Utilization and Timing of Signal Change Interval

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The problem of timing the signal change interval has received increased attention in recent years. Much of this attention is focused on two issues: whether a constant yellow interval should be used and whether the timing equations suggested by the Institute of Transportation Engineers (ITE) can realistically reflect driver needs for the change interval. These two issues are examined on the basis of observed driver behavior. The 95th percentile yellow interval requirements are found to vary from 3 to 5 sec. Such requirements do not have a positive linear correlation with the approach speed. The 85th and 95th total change interval requirements have strong linear correlations with vehicle clearance time. The ITE's timing equations should be replaced by simpler ones that can better explain driver behavior.

A signal change interval is a short time period in a traffic signal cycle between conflicting green intervals. A yellow signal indication is displayed in this interval, which is often followed by an all-red signal indication. There are two major problems in timing the signal change interval for the vehicles on an intersection approach. One is to determine the total change interval requirement, and the other is to divide this total requirement into the yellow interval and the all-red interval requirement.

In general, the total-change interval requirement refers to the length of a change interval needed for a safe transfer of the right-of-way. The yellow interval requirement represents the length of a yellow interval that is needed to allow a reasonable driver to take proper action before a red signal indication is exhibited. The all-red interval requirement is the additional time following a yellow interval that is needed to clear vehicles from the intersection before a green signal indication is displayed for the vehicles on other approaches.

Current practices in determining these various requirements associated with the signal change interval vary among traffic engineering agencies. Nevertheless, the following equation suggested by the Institute of Transportation Engineers (ITE) *(1)* has been adopted by many agencies for determining the change interval requirement:

$$T = t + V/(2a) + (W + L)/V \tag{1}$$

where

- T = change interval requirement, in sec;
- t = driver reaction time, in sec;
- V = vehicle approach speed, in ft/sec;
- a = vehicle deceleration rate, in ft/sec^2;
- W = intersection width, in ft; and
- L = vehicle length, in ft.

The sum of the first two terms on the right side of this equation has also been used by a number of agencies in determining the yellow interval requirement.

The use of Equation 1 requires the selection of representative values for the driver reaction time, vehicle deceleration rate, and vehicle length. Reported values of the mean and the 85th percentile reaction times and deceleration rates vary significantly from one intersection to another *(2)*. ITE suggests a value of 1 sec for the reaction time and 10 ft/sec^2 for the deceleration rate. The value for the vehicle length is commonly assumed to be 20 ft.

Equation 1 has been expanded in several studies *(3, 4)*. In May 1985, ITE *(5)* also extended this equation and proposed a recommended practice in timing the change interval. This recommended practice determines the yellow interval according to

$$Y = t + V/2/(a \pm 0.322G) \tag{2}$$

where t, V, and a are as defined for Equation 1, and Y = yellow interval requirement, in sec; and G = the grade of approach lane, in percent. ITE recommends that the 85th percentile approach speed always be used in Equation 2.

In addition, the ITE's proposed recommended practice allows the use of $(W + L)/V$, or P/V, or $(P + L)/V$ to determine the length of the required all-red interval. The notations used in these terms are to be interpreted as follows:

- W = width of the intersection, measured from the near-side stop line to the far-side edge of the conflicting traffic lane along the actual vehicle path, in ft;
- P = width of intersection, measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path, in ft;
- L = length of vehicle, recommended as 20 ft; and
- V = speed of the vehicle through the intersection, in ft/sec.

According to ITE, $(W + L)/V$ is to be used if there is no pedestrian traffic present; the longer of $(W + L)/V$ and P/V should be used if there is the probability of pedestrian crossings, and $(P + L)/V$ should be used if there is significant

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pedestrian traffic or the crosswalk is protected by pedestrian signals.

To determine the entire change interval, the ITE's proposed recommended practice requires that the sum of the yellow interval and the all-red interval be calculated twice—once with the 15th percentile speed and again with the 85th percentile speed. If the 15th percentile speed produces a longer interval, the all-red interval calculated at the 85th percentile speed is to be increased by the difference.

Several recent studies have raised doubt about the wisdom of using Equation 1 or its expanded forms in determining the change interval and of using Equation 2, or the sum of t and $V/(2a)$, for determining the yellow interval. For example, Chang et al. have found that the behavior of the drivers who entered the intersection after the yellow onset did not change significantly with the approach speed *(6)*. Their study showed that, over a speed range of 25 to 55 mph, 85 percent of the vehicles entering the intersection after the yellow onset took less than approximately 3.5 to 3.8 sec to reach the stop line. And, over the same speed range, 95 percent of the entering vehicles took less than about 4.2 to 4.6 sec to reach the stop line after the yellow onset. These findings prompted the three investigators to suggest that the use of a constant yellow interval of 4.5 sec may be warranted. A study by Wortman and Fox further reinforces the notion that the needs for the yellow interval is independent of the approach speed *(7)*.

Regarding the length of the change interval, a study by Lin has shown that the change interval requirement can be better estimated as a linear function of the time required for the vehicles to clear the intersection *(2)*. However, Lin's study was based on a rather limited data base. Subsequent to this study, additional data were collected in order to provide a better understanding of how the change interval should be designed.

The objective of this paper is to use the available data to discuss the utilization and timing of both the yellow interval and the change interval as a whole.

YELLOW INTERVAL REQUIREMENTS

Faced with a yellow signal indication, a driver will either decide to stop or proceed through the intersection. A yellow interval should be long enough to allow a proper choice by a driver under such a circumstance. Whether or not a yellow interval is adequate can be evaluated in terms of the percent of vehicles entering the intersection after the termination of the yellow interval *(5)*. A shorter yellow interval will likely force a larger percent of vehicles to enter the intersection on a red signal indication, or force more drivers to take potentially dangerous actions. The yellow interval requirement is defined in this study as the length of a yellow interval that will allow a specified percent of signal change intervals to be free of vehicles entering the intersection on a red signal indication.

To examine the nature of this requirement, data related to six straight-through movements and two left-turn movements at a total of five intersections were collected for analysis. Each of the subject movements represents the vehicular flows in one or more traffic lanes. All the five subject intersections were located in the state of New York. Three were on Central Avenue in Albany, one was on Almond Street in Syracuse, and the remaining one was on Market Street in Potsdam. The two left-turn movements had their own separate signal phases. As can be observed in Table 1, the clearance widths for the eight movements varied from 77 to 135 ft. Pedestrian interferences were negligible at the time of the data collection. Therefore, for each of the straight-through movements, the clearance width was measured from the stop line of the approach lane to the farthest potential conflicting point on the far side of the intersection. Similarly, the clearance widths for the two left-turn movements were measured as the length of a representative turning path from the stop line to the farthest potential conflicting point downstream.

The approach speeds given in Table 1 were based on those vehicles approaching the intersection near the end of the green interval. They were measured with stopwatches as the travel times over a distance of 100 to 150 ft. The lowest mean approach speed was 21.9 mph for Movement 8 and the highest was 32.5 mph for Movement 4. The grades for all the movements were gentle.

On average, each of the subject movements was observed for about 2-1/2 hr. The number of change intervals encountered in such an observation period ranged from 68 for Movement 8 to 255 for Movement 6. Not every one of these change intervals was utilized by vehicles either to enter or to clear the intersection after the yellow onset. For each utilized change interval, the elapsed time from the yellow onset to the moment the last entering vehicle reached the stop line was measured with a stopwatch. Such an elapsed time represents the length of the yellow interval that is needed for a change interval to be free of vehicles entering on a red signal indication. The resulting

TABLE 1 CHARACTERISTICS OF MOVEMENTS EXAMINED FOR YELLOW INTERVAL REQUIREMENTS

Movement	Movement Type	Existing Yellow sec	Clearance Width w, ft	Approach Speed, V mph			Grade %
				15th	Mean	85th	
1	Straight	3.9	77	25.6	30.0	41.9	+3.0
2	Straight	3.9	135	26.2	29.8	36.0	+1.7
3	Straight	4.0	105	22.1	27.6	38.0	+0.9
4	Straight	3.5	96	26.6	32.5	37.8	+0.7
5	Straight	3.5	92	22.6	26.6	30.9	+0.8
6	Straight	3.1	93	27.0	31.8	38.2	+0.9
7	Left	3.0	115	21.9	26.3	31.7	+0.9
8	Left	3.9	105	18.2	21.9	24.6	-0.6

measurements for each subject movement were used to construct a cumulative distribution of the yellow interval requirement. Figure 1 shows the cumulative distributions of the yellow interval requirements for the eight subject movements. Based on these distributions, a yellow interval can be chosen that will allow a reasonably high percent (e.g., 85 or 95 percent) of the signal cycles to be free of vehicles entering on a red signal. The 85th and 95th percentile yellow interval requirements for the eight subject movements are given in Table 2 along with related statistics.

The 95th percentile yellow interval requirements varied from about 3 to 5 sec, and the 85th percentile requirements were between 2.2 and 4.2 sec. These variations cannot be explained by the differences in the approach speeds of the various movements. Figure 1 shows that it can be quite erroneous to assume that the yellow interval requirements has a positive linear correlation with the approach speed.

Among the eight movements examined, Movement 4 and Movement 8 (Table 1) had the largest difference in approach speed. Thus, if the approach speed governs the yellow interval requirement in a manner as indicated by Equation 2, the cumulative distributions of the yellow interval requirements of these two movements should have exhibited the largest difference. To the contrary, Figure 1 shows that they were nearly identical. On the other hand, Movement 4 and Movement 6 had virtually the same approach speed. Yet, their yellow interval requirements displayed a large difference. Movement 4 and Movement 5 did show an increase in the yellow interval requirement as the approach speed increased. But, the yellow interval requirements of Movement 7 and Movement 8 ex-

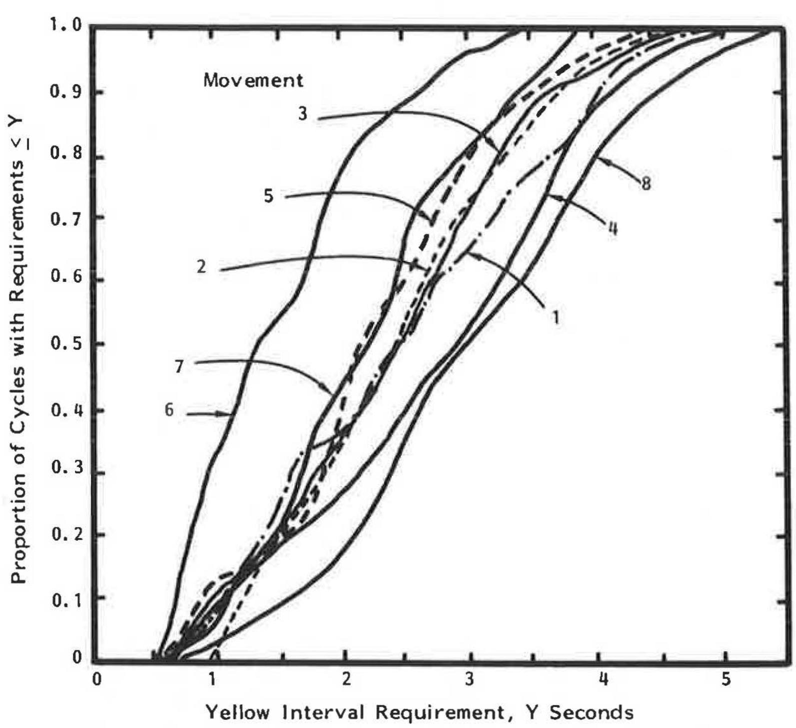

FIGURE 1 Cumulative frequency distributions of yellow interval requirements.

TABLE 2 YELLOW INTERVAL REQUIREMENTS

Movement	Requirement in Seconds 85th	Requirement in Seconds 95th	Mean Approach Speed mph	Change Interval Number Utilized	Change Interval Percent Utilized
1	3.9	4.2	30.0	55	49
2	3.5	4.0	29.8	55	74
3	3.4	4.2	27.6	60	70
4	3.9	4.5	32.5	101	76
5	3.2	3.9	26.6	55	59
6	2.2	3.0	31.8	74	29
7	3.2	3.7	26.3	57	46
8	4.2	5.0	21.9	54	79

hibited a relationship contradictory to that implied in Equation 2.

Thus, a question can be raised as to what really accounted for the variations in the cumulative frequency distributions shown in Figure 1. With each additional hour of field observations made by the authors, it became increasingly clear that the supply of vehicles that were in a position to enter the intersection within 5 sec after the yellow onset was a major source of such variations. At one extreme, Movement 8 (left turns from Wolf Road onto Central Avenue in Albany) had frequent carryovers of long queues from one cycle to the next because of the inability of a rather short green interval to discharge all of the queueing vehicles in a cycle. As a result, the vehicles would often continue entering the intersection long after the yellow interval (3.9 sec) expired.

Similarly, Movement 4 (straight-through flows on Almond Street at Harrison Street in Syracuse) provided a high level of vehicle supply at the time of the data collection. This movement occupied three straight-through lanes and another lane shared by straight-through and right-turn vehicles. During the evening peak hours in which most of the observations were made, a large number of vehicles were frequently within short travel times from the stop line at the yellow onset, and long queues often began to develop immediately after the change interval expired. Consequently, the yellow interval requirement of this movement differed very little from that of Movement 8.

At the other extreme, Movement 6 (a straight-through flow on Market Street at Sandstone Road in Potsdam) had a low flow rate of about 300 vph and was regulated by a traffic-actuated signal. The level of vehicle supply at the yellow onset was low because the vehicles were usually more than 4 sec away from the intersection when the green interval expired. Over a 4-hr observation period, the longest recorded yellow interval requirement for a cycle was 3.4 sec, and the 95th percentile yellow interval requirement was only 3 sec. Movement 7 (left turns from Central Avenue onto Everlet Avenue in Albany) had a similar characteristic. The yellow interval for this movement often began when there were no vehicles within a travel time of less than 4 sec from the stop line. This phenomenon was created by the signal controls for Movement 7 and for the movements at the upstream intersection. The resulting 95th percentile yellow interval requirement of 3.7 sec was significantly lower than that of Movements 4 and 8.

Although it is evident that the level of vehicle supply at the yellow onset is a governing factor of the yellow interval requirement, there are currently no quantitative methods for defining such a causal relationship. The percent of change intervals utilized by vehicles to enter the intersection may be a potential measure of the level of vehicle supply. Figure 2 shows that such a measure has an apparent correlation with both the 95th and 85th percentile yellow interval requirements of the eight movements.

Let F be the proportion of change intervals utilized by vehicles to enter the intersection after the yellow onset. Then, the 95th percentile yellow interval requirements of the eight movements given in Table 2 can be related to F according to

$$Y = 2.36 + 2.83F \qquad (3)$$

where Y represents the specified percentile yellow interval requirement. This equation has an R^2 value of 0.73 and a standard error of estimate of 0.33 sec. The corresponding equation for the 85th percentile yellow interval requirement is

$$Y = 1.81 + 2.70F \qquad (4)$$

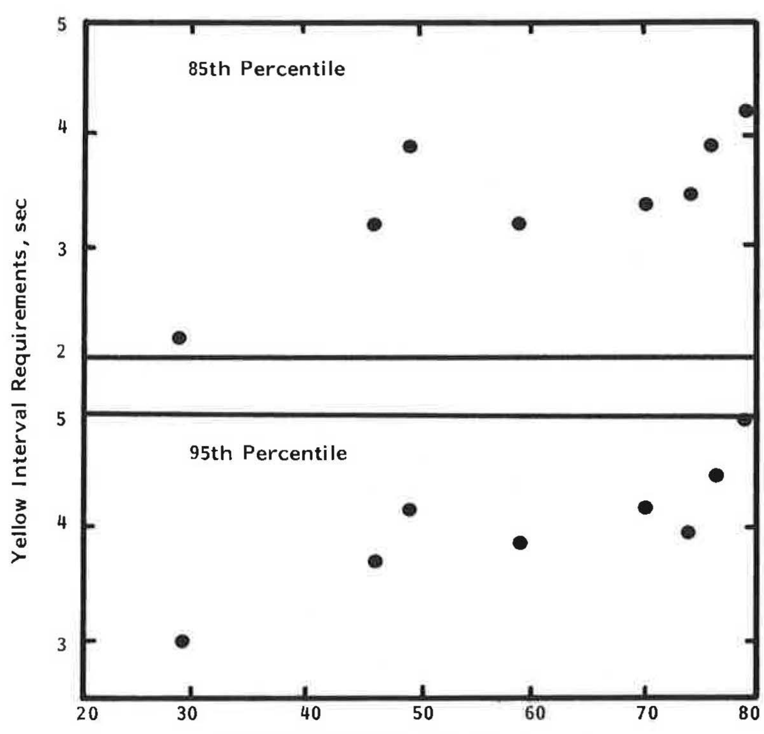

FIGURE 2 Variation of yellow interval requirement with the rate of change interval utilization.

The R^2 value of this equation is 0.60 and the standard error of estimate is 0.42 sec.

CHANGE INTERVAL REQUIREMENTS

The signal change interval may comprise only a yellow interval or include a yellow interval and an all-red interval. The current Uniform Vehicle Code *(8)* allows vehicles to enter the intersection during the yellow interval and to clear the intersection after the red interval begins. This "permissive rule" has a greater need for the all-red interval in comparison with the "restrictive rule," which requires vehicles to clear the intersection by the end of the yellow interval.

Regardless of which rule drivers should follow, the change interval requirement can be defined as the length of a change interval that is needed to allow all vehicles to clear the intersection in a specified percent (e.g., 85 percent) of signal cycles. In order to analyze the nature of this requirement, data related to the interactions between the change intervals and the vehicles of 22 movements were collected. These 22 movements were associated with 15 intersections in 5 urban areas in the state of New York. Five of the intersections were located in Syracuse, four in Albany, one in Rochester, three in Potsdam, and two in Canton. Four of the 22 movements were the same as Movement 1 through Movement 4 described previously in Table 1.

As can be observed in Table 3, all but one of the movements had clearance widths between 74 and 135 ft. The grades of the approach lanes were within ±4 percent. The mean approach speeds varied from 21.9 to 43.9 mph, and the mean turning speeds of the left-turn movements were about 20 mph. At the time of the data collection, none of the 22 movements had vehicles blocking the intersections because of congestion downstream of the stop lines. The "permissive rule" was in place at all the intersections for the use of the change interval. Pedestrian interferences with the vehicular movements at all the intersections were negligible at the time of the data collection. Therefore, the clearance widths were also measured according to the definitions described previously.

The traffic conditions in an approach lane of a signalized intersection may vary substantially within a signal cycle because of the formation and dissipation of queues. Consequently, the speeds of those vehicles that may interact with the change interval may be significantly affected by such changing conditions. For this reason, the determination of the vehicle speeds took into consideration only those vehicles approaching or crossing the intersection near the end of the green interval or immediately after the yellow onset. Stopwatches were used to measure the travel times of such vehicles over a distance of 100

TABLE 3 CHANGE INTERVAL REQUIREMENTS

Movement	Clearance Width ft	Grade %	Approach Speed, mph			C.I. Requirement, sec		% of C.I. Utilized
			15th	Mean	85th	85th	95th	
1	89	-1.0	25.7	28.9	32.3	6.3	7.0	51
2	89	+4.0	23.8	27.2	35.2	5.6	6.4	60
3	117	-0.5	27.5	31.1	35.8	5.0	6.0	60
4	74	-0.9	24.3	27.9	32.0	4.8	5.5	36
5	107	-0.3	28.4	32.2	36.5	5.3	6.0	15
6	106	-3.9	27.6	30.1	31.7	5.5	6.0	34
7	90	+0.7	38.6	43.9	49.2	5.5	5.8	47
8	96	+0.2	28.1	33.0	37.6	5.0	5.5	61
9	195	+1.0	24.2	30.6	35.8	7.4	8.3	23
10	74	+0.4	27.3	30.8	35.1	5.1	5.7	55
11	130	+0.9	27.7	32.5	38.1	5.8	6.1	40
12	77	+3.0	25.6	30.0	41.9	5.6	6.3	46
13	135	+1.7	26.2	29.8	36.0	6.8	8.3	79
14	76	-0.1	23.5	28.2	35.6	5.7	6.6	60
15	105	+0.9	22.1	27.6	38.0	7.2	8.1	67
16	110	-3.5	20.8	24.5	28.9	6.8	8.0	44
17	96	+0.7	26.6	32.5	37.8	5.4	6.0	66
18	130	+0.6	25.9	30.5	35.7	6.2	7.1	39
19	105	+0.8	17.9 (14.9)	21.9 (18.0)	25.2 (22.7)	8.8	9.3	79
20	115	+0.9	22.6 (17.4)	26.3 (20.0)	31.1 (24.4)	7.4	8.0	26
21	96	-0.5	24.3 (17.9)	28.3 (20.2)	33.6 (23.2)	7.4	7.8	44
22	101	+0.8	22.5 (16.2)	26.8 (18.8)	30.8 (21.1)	8.4	9.4	69

Note: Values in parentheses are turning speeds to 150 ft. For the straight-through movements, the approach speeds determined in this manner were equivalent to the speeds at which the vehicles cleared the intersection. For the left-turn movements, the approach speeds were not a good approximation of the clearance speeds. Therefore, both approach speeds and clearance speeds were determined for the left-turn movements.

The major task of the data collection was to use stopwatches to measure, on a cycle-to-cycle basis, the elapsed time from the yellow onset to the moment the last entering vehicle cleared the intersection. Only those vehicles entering the intersection after the yellow onset were included in the data collection. This task was performed for an average of about 2 hr for each subject movement. The resulting data were used to construct a cumulative frequency distribution of the change interval requirement for each subject movement. The 85th and 95th percentile change interval requirements determined from such distributions are summarized in Table 3 along with other relevant statistics.

The data given in Table 3 can be used to examine alternative models for estimating the change interval requirement. One such model suggested by Lin *(2)* can be written as

$$T = A + B(W + L)/V \tag{5}$$

where

T = specified percentile requirement of the change interval, in sec;
A, B = coefficients to be calibrated;
W = clearance width, in ft;
L = representative vehicle length, 20 ft; and
V = mean clearance speed, in ft/sec.

This model implicitly assumes that the relationship between the change interval requirement and the average clearance time $(W + L)/V$ is linear. Figure 3, which is based on the 95th percentile change interval requirements given in Table 3, confirms the existence of a strong linear relationship between T and $(W + L)/V$.

A least-square regression based on the 85th percentile change interval requirement given in Table 3 results in the following equation:

$$T = 2.84 + 1.09(W + L)/V \tag{6}$$

This equation has an R^2 value of 0.75 and a standard error of estimate of 0.58 sec. When the 95th percentile requirements are used for the regression, the resulting equation is

$$T = 3.33 + 1.17(W + L)/V \tag{7}$$

The R^2 value of this equation is 0.74 and the standard error of estimate is 0.64 sec.

No attempt was made to analyze the confidence intervals of these regression equations and the variances of the regression coefficients. Such an analysis requires the assumption that the 85th and 95th percentile change interval requirements are distributed normally. The existing data do not support such an assumption.

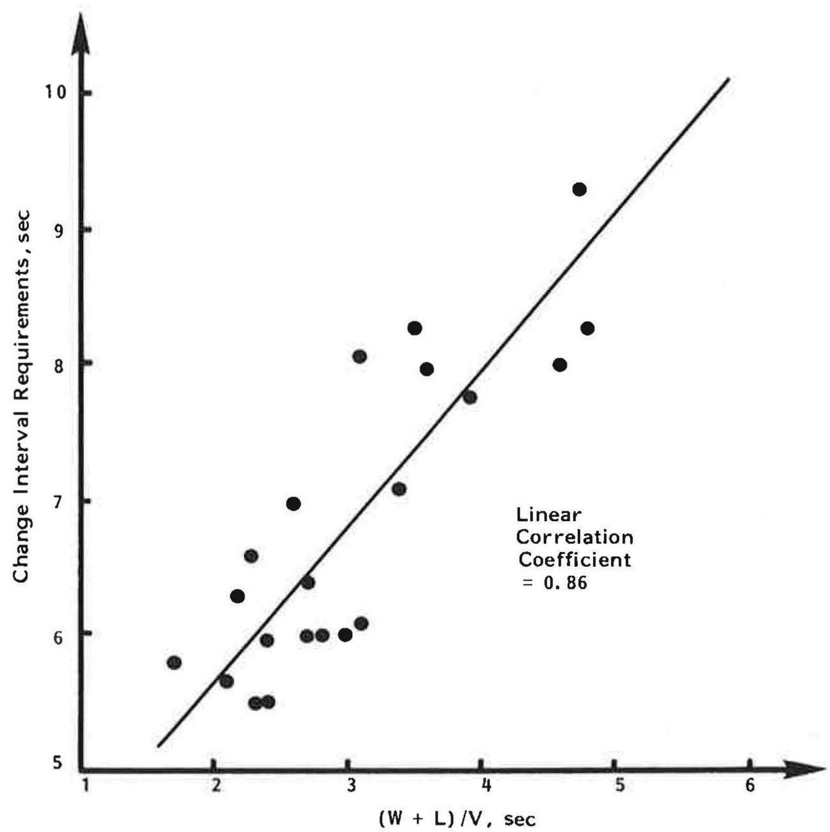

FIGURE 3 Variation of the 95th percentile change interval requirement with clearance time.

For the subject movements, the last term of Equation 7 is virtually the same as the clearance time determined from the 15th percentile clearance speed. Similarly, the last term of Equation 6 has values that are on the average only 0.2 sec shorter than the clearance times determined from the 15th percentile clearance speeds of the various movements. Therefore, if the last terms of these two equations are used to determine the all-red interval requirement, they would satisfy the clearance needs of about 85 percent of the entering vehicles.

Equations 6 and 7 are capable of explaining about 74 percent of the variations in the 85th and the 95th percentile change interval requirements. The unaccounted-for variations can be attributed to differences in vehicle speeds, vehicle lengths, vehicle supply patterns at the yellow onset, and so forth. Because the yellow interval requirement is linearly correlated to some extent with the proportion F of change intervals utilized, Equation 5 can be improved by adding onto it another term as follows:

$$T = A + CF + B(W + L)/V \tag{8}$$

The resulting regression equation based on the 95th percentile requirements is

$$T = 2.24 + 2.15F + 1.18(W + L)/V \tag{9}$$

with an R^2 value of 0.84 and a standard error of estimate of 0.52 sec. The corresponding equation for the 85th percentile requirements is

$$T = 2.0 + 1.7F + 1.10(W + L)/V \tag{10}$$

This equation has an R^2 value of 0.82 and a standard error of estimate of 0.51 sec.

Although Equations 9 and 10 are more powerful than Equations 6 and 7 in explaining the length requirements of the change interval, the improvements do not appear to be large enough to warrant the use of either Equation 9 or Equation 10. In fact, the inclusion of F in these equations would make the equations difficult and expensive to use because data for F would have to be collected at each intersection. For the same reason, Equations 3 and 4 presented earlier would have little use for timing applications.

An alternative to Equations 5 and 8 for estimating the change interval requirements is ITE's proposed recommended practice. For movements that have little pedestrian interference, this practice implies a model form of

$$T = t + V/2/(a \pm 0.322G) + (W + L)/V \tag{11}$$

As described previously, the use of Equation 11 requires two calculations: once with the 15th percentile speed and once with the 85th percentile speed. However, ITE is vague about how such calculations are to be performed for protected left turns. The last two terms in Equation 11 are a function of vehicle speed. If both terms are determined from the same percentile approach speed for left-turn movements, it can be shown that the resulting T values and the 95th percentile change interval requirements given in Table 3 have a linear correlation coefficient of 0.57. If the approach speed is used for the second term on the right side of Equation 11 and the turning speed is used for the last term, the resulting linear correlation coefficient would become 0.79 (Figure 4). This level of correlation with the observed 95th percentile requirements is respectable, but it is still below that which can be achieved by a much simpler model such as Equation 7 (Figure 3). Therefore, there is no reason to adopt ITE's proposed recommended practice unless it has a superior theoretical basis for explaining driver behavior.

The soundness of the theoretical basis for Equation 11 can be addressed by rewriting this equation as

$$Z = T - (W + L)/V = t + V/2/(a \pm 0.322G) \tag{12}$$

The term Z in this equation represents the yellow interval requirement. If Equation 11 is a valid representation of the behavior of drivers in their use of the change interval, the values of Z determined as $T - (W + L)/V$ from field observations should be strongly and positively correlated with the approach speed.

Figure 5 shows that such a linear correlation does not exist between Z and the approach speed as far as the 22 subject movements are concerned. In this figure, the values of Z are determined as $T - (W + L)/V$ from Table 3 based on the mean clearance speeds and the 95th percentile change interval requirements. A least-square regression of these Z values on the mean approach speeds (in mph) results in the following equation:

$$Z = 5.32 - 0.049V \tag{13}$$

The R^2 value of this equation is 0.10 and the standard error of estimate is 0.62 sec.

As can be observed from Figure 5, there is only one Z value for mean approach speeds exceeding 34 mph. This Z value is weighted more heavily in Equation 13 than the other values. If this value is deleted, the resulting regression equation becomes

$$Z = 7.85 - 0.14V \tag{14}$$

for mean approach speeds ranging from 22 to 33 mph. Equation 14 has an R^2 value of 0.34 and a standard error of estimate of 0.54 sec.

The regression coefficients of both Equation 13 and Equation 14 are quite different from the values recommended by the ITE. According to ITE, the constant terms in Equations 12 and 13 should have been 1.0 sec and, for the subject movements, the coefficient of V should have been in the range of 0.044 to 0.057. Therefore, even if Z is really a linear function of the approach speed, the sum of t and $V/2/(a \pm 0.322G)$ is a poor representation of driver behavior. The negative signs of the coefficients of V in Equations 13 and 14 further indicate that an increase in the approach speed tends to cause a reduction instead of an increase in the yellow interval requirement. It is uncertain, however, whether this negative correlation between Z and V really exists because the R^2 values of Equations 13 and 14 are rather small and data are lacking for mean approach speeds exceeding 32 mph and for speeds below 26 mph. Overall, it is evident that the causal relationship between Z and V is very weak and; thus, it is meaningless to treat the yellow interval requirement as a function of the approach speed.

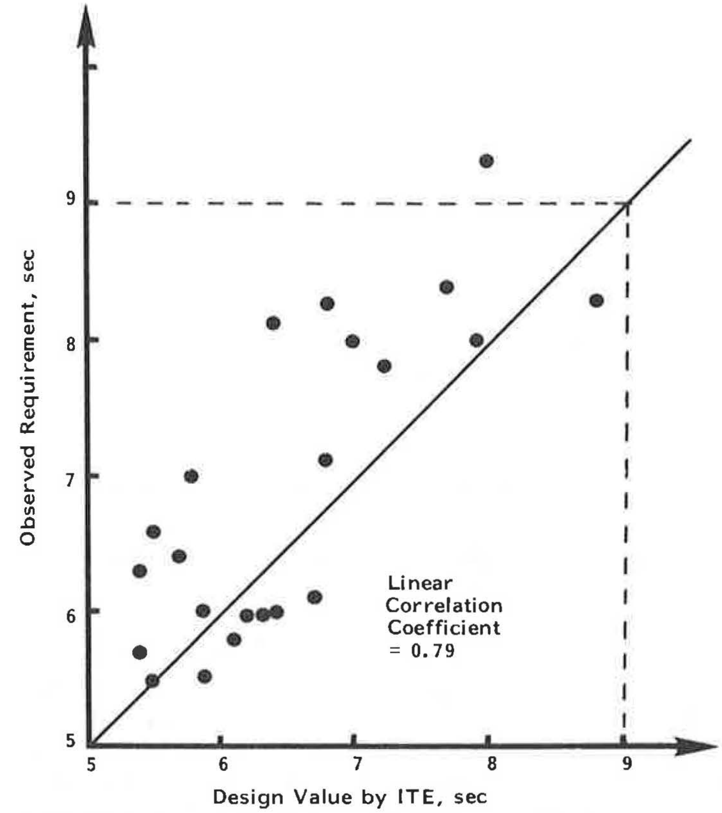

FIGURE 4 Correlation between the 95th percentile change interval requirements and values determined from the ITE's proposed recommended practice.

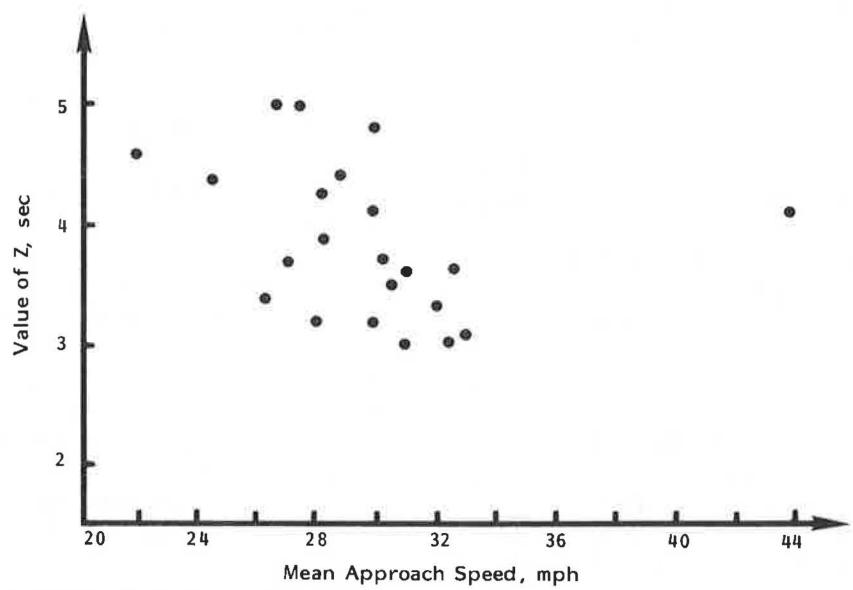

FIGURE 5 Variation of Z with mean approach speed.

TIMING DESIGN APPLICATIONS

The cumulative frequency distributions of the yellow interval requirement shown in Figure 1 are bounded by the distributions of Movements 6 and 8, which had vehicle supply patterns of opposite extremes at the yellow onset. The corresponding 95th percentile yellow interval requirements are between 3 and 5 sec and the 85th percentile requirements are between 2.2 and 4.2 sec. The Z values shown in Figure 5, which approximate the 95th percentile yellow interval requirements of 22 movements, also lie between 3 and 5 sec. The same Z values plotted in Figure 6 against the percentage of change intervals utilized further show that the Z values remain above 3 sec even when the rate of change interval utilization drops to as low as 13 percent. Therefore, a reasonable range of the design values for

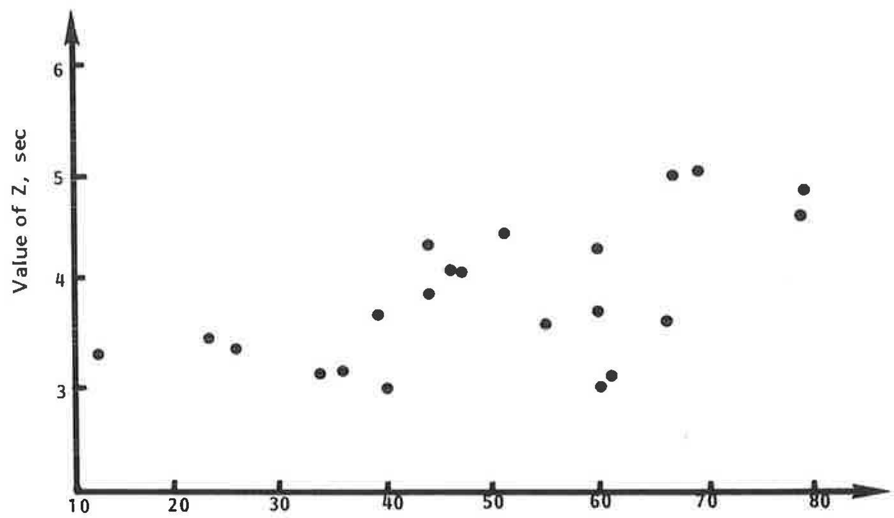

FIGURE 6 Variation of Z with the rate of change interval utilization.

the yellow interval is 3 to 5 sec. Yellow intervals shorter than 3 sec are not recommended because they may cause some drivers to apply excessively high decelerations in order to avoid entering the intersection on red.

The variations in the change interval requirements cannot be accounted for by the differences in the approach speeds. Relating such variations to another traffic or signal variable is likely to make the resulting timing method difficult to apply. Therefore, it is preferred that simple guidelines be established in the future for the choice of the yellow interval.

Meanwhile, the yellow interval may be determined according to

$$Y = 4.0 + C_1 \tag{15}$$

in order to satisfy the 95th percentile requirements. In this equation, C_1 is a correction factor with a value between −1.0 and +1.0 sec.

A value between +0.5 and +1.0 sec may be chosen for C_1 if one of the following vehicle supply patterns exists: (a) frequent carryovers of long queues from one cycle to the next; (b) a rapid build-up of long queues immediately after the change interval expires; and (c) the percent of change intervals utilized exceeding 70 percent. On the other hand, a reasonable choice of C_1 would be between −1.0 and −0.5 sec if (a) the movements of concern have low flow rates and are regulated by traffic-actuated controls; or (b) the vehicle supply to the intersection at the yellow onset is frequently cut off due to cyclic flow patterns created by signal coordination; or (c) the rate of change interval utilization is less than 30 percent. For movements with vehicle supply patterns in between the two extremes, C_1 may be set to 0 sec.

To reflect the actual requirements of individual movements, the lengths of the change interval determined from either Equation 6 or Equation 7 may be adjusted upward or downward. For the 95th percentile requirements, the adjustment may take the form of

$$T = 3.33 + 1.17\,(W + L)/V + C_2 \tag{16}$$

where C_2 is a correction factor.

For the 22 subject movements given in Table 3, the values of C_2 range from about −1.0 to +1.1 sec. These values correspond to the deviations of the measured 95th percentile requirements from the regression line shown in Figure 3. Again, a reasonable choice of C_2 is between +0.5 to +1.0 sec for movements with high levels of vehicle supply at the yellow onset (e.g., Movements 1, 7, 13, 14, 15, 16, 19, and 22). In contrast, C_2 would most likely assume a value between −1.0 and −0.5 sec for movements with very low levels of vehicle supply (e.g., Movements 3, 5, 8, 9, 11, and 20).

For timing applications, C_1 and C_2 can be considered to be the same. Therefore, given a yellow interval and a change interval as determined from Equations 15 and 16, the all-red interval R can be calculated as

$$\begin{aligned} R &= 3.33 + 1.17\,(W + L)/V + C_2 - 4 - C_1 \\ &= 1.17\,(W + L)/V - 0.67 \end{aligned} \tag{17}$$

It should be noted that the all-red interval requirements determined from Equation 17 do not take into consideration a safety margin that may be provided by the cross traffic. This safety margin is the amount of time required for the first vehicle in the cross traffic to reach the conflicting point after the change interval expires. If queueing vehicles are present on the cross street after the change interval expires, the safety margin would equal the time needed for the driver in the first queueing vehicle to accelerate the vehicle to the conflicting point. If no queueing vehicles are present, it will also take the first vehicle arriving on the cross street some time to reach the conflicting point.

If this vehicle approaches the intersection at a speed V_o and has an intended deceleration b, then the minimum safety margin provided by this vehicle can be approximated by $V_o/(2b)$. The deceleration rate b can be as high as 18 ft/sec² *(6)*. Using this rate and measuring V_o in ft/sec gives a safety margin of $V_o/36$ sec. This safety margin can be determined by choosing an appropriate value (e.g., the 15th percentile approach speed) for V_o. The safety margin allowed for the timing design could be taken as the lesser of $V_o/36$ and the minimum (or near minimum) time for the first queueing vehicle in the cross traffic to reach the conflicting point.

CONCLUSIONS

The yellow interval requirement correlates poorly with the approach speed. This requirement appears to be governed by the vehicle supply pattern at the yellow onset. When frequent carryovers of long queues from one cycle to the next exist, the 95th percentile yellow interval requirement can reach 5 sec. This requirement can be reduced to about 3 sec if vehicular movements with low flow rates and under the control of traffic-actuated signals are involved.

A constant yellow interval of 4.5 sec would be able to accommodate the 90th to the 100th percentile requirements at nearly all the intersections. However, because the 95th percentile yellow interval requirement can vary from 3 to 5 sec, the use of a constant yellow interval may not appeal to some traffic engineering agencies. On the other hand, introducing additional variables into a timing procedure in order to account for such a variation would certainly make the resulting procedure impractical. Therefore, it is recommended that simple guidelines be established to assist in the choice of the yellow interval. Such guidelines may evolve on the basis of Equation 15.

At intersections where grades are within ±4 percent, the change interval requirements are strongly and linearly correlated with the vehicle clearance time. Therefore, simple regression equations such as Equations 6 and 7 can adequately serve as a basis for timing design. The ITE's proposed recommended practice lacks a sound theoretical basis and is unnecessarily tedious to apply.

Equation 17 provides a convenient and logical tool for determining the all-red interval requirements. The coefficients of this and other regression equations presented previously can be modified if additional data become available. The existing data base can be enhanced in several respects. Of particular interest are vehicular movements with mean approach speeds exceeding 32 mph, intersections with clearance widths of more than 130 ft, and intersections where grades are steep.

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