Timing Design of Signal Change Intervals

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ABSTRACT

Existing practices in the timing design of the signal change interval vary considerably. Attempts to provide a logical basis for the timing design are often focused on the understanding of how individual drivers react when they are confronted by a change interval. Driver behavior can be affected by numerous factors and is difficult to measure. In contrast, direct measurement of drivers' aggregated needs for the change interval can be performed easily. It can also provide information useful to the development of timing methods and to the assessment of existing change intervals. For these reasons, field observations of drivers' aggregated needs for the change interval were made at 10 intersections. The resulting data are used to evaluate various timing methods. The observed needs for the change interval indicate that timing models based on the specification of constant reaction time and deceleration rate may not adequately explain the aggregated needs of drivers.

The signal change interval is an indispensable element of signal control at an intersection. This interval usually begins with the display of a yellow signal indication, which may be followed by the display of a red signal indication in every direction. The signal change interval is short, but it has two major implications. One concerns traffic safety and the other is related to law enforcement. Rear-end collisions and right-angle collisions during signal changes may take place because of improper driver behavior compounded by poor signal timing practices. On the other hand, indiscreetly implemented combinations of the yellow interval and the all-red interval may result in driver disrespect for the red indication and foster habitual violations of signal control. Therefore, it is important that the change interval be properly determined for every signalized intersection.

Unfortunately, the proper determination of the signal change interval is a difficult task. For one thing, there is a lack of consensus as to what criteria should be used for such a task. The meaning of the change interval, particularly that of the yellow indication, is often confusing, even to traffic engineers (1). These problems are further aggravated by the highly variable nature of driver behavior. As a result, existing practices in the timing design of the change interval vary among traffic agencies.

Researchers have attempted to provide a logical basis for timing the change interval. Most studies in this connection are focused on measuring the behavior of individual drivers who are confronted by the signal change interval. Drivers' reaction time, deceleration rate, and decision-making process in response to the change interval are the primary subjects of these studies.

Interestingly enough, although the central issue is how long the change interval should be, direct measurement of drivers' needs for the change interval has rarely been made. As a diversion from the emphasis on the behavior of individual drivers, this study examines drivers' aggregated needs for the change interval at 10 intersections in the state of New York. On the basis of the observed needs, several methods for timing the change interval are compared.

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LITERATURE REVIEW

The timing design of the signal change interval involves the determination of the total length of the change interval and its division into a yellow interval and an all-red interval. Ideally, the timing design should conform to the legal interpretation of the change interval. Under a restrictive rule, vehicles must have cleared the intersection by the time the yellow interval expires. In contrast, a permissive rule allows vehicles to enter the intersection during the yellow interval and to clear the intersection after the red interval begins. The current Uniform Vehicle Code (2) adopts the permissive rule. This rule has a greater need for the all-red interval in comparison with the other rule. A survey by Benioff and Rorabaugh (3), however, revealed that the procedure used for timing the change interval was statistically independent of the state law regarding the meaning of the yellow indication.

So far, there has been no consensus as to the method for timing the change interval. The U.S. Manual on Uniform Traffic Control Devices (4) suggests the use of 3- to 6-sec yellow intervals. It also states that a short all-red interval may be used to permit the intersection to clear before cross traffic is released. No specific methods are recommended in this manual for determining the change interval. Similarly, New York State's Manual of Uniform Traffic Control Devices (5) recommends the use of 2- to 5-sec yellow intervals. If a yellow interval of more than 5 sec is needed, this manual suggests that a 3-sec yellow interval be used in conjunction with an all-red interval of up to 5 sec.

To provide a systematic method of determining the change interval, Gazis et al. (6) developed the following equation based on theoretical considerations:

$$T = t + (V/2a) + [(W + L)/V] \qquad (1)$$

where

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

The sum of the first two terms of this equation represents the time required for a driver to come to a stop after the yellow interval begins. The last term of the equation is the time required for a vehicle to cross the intersection. The Transportation and Traffic Engineering Handbook (7) suggests that a reaction time of 1 sec, a deceleration rate of 10 ft/sec², and a representative vehicle length of 20 ft be used in Equation 1.

Equation 1 has been modified in various ways. Williams (8), for example, added another element to account for the time required for a vehicle in the cross traffic to reach the conflict point. The resulting equation is

$$T = t + (v/2a_1) + [(W + L)/v] - [k + (2D/a_2)^{1/2}] \quad (2)$$

where

- t = reaction time (1.1 sec),
- v = 85th-percentile approach speed,
- a_1 = deceleration accepted 85 percent of the time (6.5 ft/sec²),
- L = length of vehicle (17 ft for cars),
- k = reaction time of cross traffic (0.4 sec),
- D = distance between vehicles and cross traffic, and
- a_2 = maximum acceleration of cross traffic (16 ft/sec²).

Williams indicated that the time deduction in Equation 2 for cross-vehicle acceleration needs to be applied with caution. He suggested that no deduction be made if cross vehicles may move into the intersection before the green indication is displayed.

The deceleration rate in Equation 1 can be affected by the grade of an approach lane. On the basis of theoretical considerations, Parsonson and Santiago (9,pp.67-71), adjusted the deceleration rate according to a ± 0.322G, where G is the grade in percent. With this modification, Equation 1 can be transformed into

$$T = t + [V/2\ (a \pm 0.322G)] + [(W + L)/V)] \quad (3)$$

The effect of grade on the deceleration rate was recently examined by Chang et al. (10) on the basis of field observations made at 13 sites. They found that for grades ranging from +1 to -6.5 percent, the effect of grade can be accounted for by using a = 10.5 ± 0.075G. On the basis of this relationship, Equation 1 can be rewritten as

$$T = t + [V/2\ (10.5 \pm 0.075G)] + [(W + L)/V] \quad (4)$$

It can be seen from this equation that for grades between +1 and -6.5 percent, the effect of grade on the calculated change interval is generally negligibly small. The inclusion of grade as a variable complicates the equation and does not appear to be necessary for such a range of grades.

The recommended practice, Determining Vehicle Change Intervals, as proposed by the Institute of Transportation Engineers (ITE) (11) relies on the sum of t and $V/[(a \pm 0.322G)/2]$ as shown in Equation 3 to determine the yellow interval. The recommended value of t is 1 sec and that of a is 10 ft/sec². If an all-red interval is to be provided, ITE's proposed practice allows the use of (W + L)/V, or P/V, or (P + L)/V to determine the length of the required interval. W, P, and L are defined 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;
- P = width of the intersection, measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path; and
- L = length of vehicle, recommended as 20 ft.

According to ITE's proposed practice, (W + L)/V is to be used when there is no pedestrian traffic; the longer of either (W + L)/V or P/V is to be used when there is the probability of pedestrian crossings; and (P + L)/V is to be used when there is significant pedestrian traffic or the crosswalk is protected by pedestrian signals. Furthermore, the entire change interval should be calculated twice, once with the 15th-percentile speed and again with the 85th-percentile speed. In the cases where the 15th-percentile speed produces a longer interval, the all-red interval calculated at the 85th-percentile speed is increased by the difference.

Equation 1 has been employed by many agencies. Of the agencies responding to a survey by May (12), 22 percent indicated that they use this equation. A later survey of 232 traffic agencies by Benioff and Rorabaugh (3) found that most of the agencies use Equation 1.

The all-red duration is generally considered to be a function of intersection width and approach speed. For example, the last term of Equation 1 has been used for determining the all-red duration (13). The methods for timing the yellow interval can be classified into two general approaches. One is to employ a uniform yellow interval at all or a selected group of intersections. Such a yellow interval is often used in conjunction with an all-red interval. The survey conducted by Benioff and Rorabaugh (3) revealed that about one-quarter of the agencies responding to the survey had at one time utilized a uniform yellow interval throughout their jurisdiction or in selected portions of it. Two of the subject agencies making extensive use of the uniform yellow interval indicated that they had observed no change in accidents. Uniform yellow intervals were also reported to have been used by 47 percent of the agencies responding to May's survey (12).

Several researchers have also recommended the use of a uniform yellow interval. Olson and Rothery (14), for example, stated that a uniform yellow interval of 5.5 sec would allow all or nearly all motorists to clear the intersections that they investigated. Williams (8) also suggested the use of a uniform yellow interval. This interval is then subtracted from the change interval calculated from Equation 2 to determine the required all-red interval.

Chang et al. (10) examined drivers' decisions to cross the intersection after the yellow interval begins. They found that 95 percent of the vehicles going through the intersection took less than 4.5 sec to reach the stop line. They also found that this time element was relatively stable for vehicle approach speeds ranging from 25 to 55 mph. On the basis of these observations, it was contended that the use of a uniform yellow interval of 4.5 sec may be warranted.

Another approach for timing the yellow interval considers the yellow interval as a function of approach speed and other variables. Many agencies have used the sum of t and V/2a to determine the yellow interval (12). The yellow interval determined in this manner decreases as the approach speed decreases. According to the observations made by Chang et al. (10), however, the yellow interval should not be shortened at lower approach speeds.

The use of Equation 1 and other similar equations has also generated controversies. The problem is in the selection of a constant reaction time and a constant deceleration rate for such equations. Driver

TABLE 1 Reported Reaction Times and Deceleration Rates

Description	Value	Source
Deceleration (ft/sec^2)		
Avg maximum	9.7	Williams *(8)*
Mean	7.0-13.9	Wortman and Matthias *(15)*
85th percentile	11.5-18.2	Wortman and Matthias *(15)*
Mean	8.3-13.2	Wortman and Witkowski *(16)*
85th percentile	10.8-17.7	Wortman and Witkowski *(16)*
Mean	7.8-13.4	Chang et al. *(10)*
Reaction time (sec)		
Mean	1.14	Gazis et al. *(6)*
Mean	1.4	Jenkins *(17)*
Mean	1.09-1.55	Wortman and Matthias *(15)*
85th percentile	1.5-2.1	Wortman and Matthias *(15)*
Mean	1.1-1.4	Wortman and Witkowski *(16)*
85th percentile	1.4-2.0	Wortman and Witkowski *(16)*
Mean	0.7-1.5	Chang et al. *(10)*
85th percentile	1.0-2.2	Chang et al. *(10)*

Note: Range of value reflects variations among different sites.

behavior varies with individuals, traffic flow conditions, geometric design, and many other factors. Consequently, it is doubtful that a single reaction time and a single deceleration rate will allow the determination of a desirable change interval under all circumstances. Table 1, which shows the highly variable nature of the reaction time and the deceleration rate, underlines this problem. Further studies on the reaction time and the deceleration rate will not make the timing design of the change interval easier.

So far, the research on the signal change interval has largely ignored drivers' aggregated needs for the change interval. Such needs relate to the time required for the intersection to clear after the yellow onset. They can be measured with relative ease for testing existing timing design methods and for developing new methods. The findings of a preliminary study along these lines are presented.

DATA COLLECTION

The data were collected at five intersections in Syracuse, three intersections in Potsdam, and two intersections in Canton, all in New York State. Syracuse has a population of about 200,000. Potsdam and Canton are small urban areas with populations of approximately 10,000 and 6,000, respectively.

Only straight-through vehicles were considered for the data collection. Two lanes in one of the intersections in Canton and one lane in each of the other intersections were used.

The following types of data were the focus of the data collection:

1. Intersection width and grade,
2. Flow rate and vehicle approach speed,
3. Timing settings of the yellow interval and the all-red interval, and
4. Signal change interval requirements measured as the elapsed time from the onset of the yellow interval to the moment when the last crossing vehicle clears the intersection.

The data collection covered a total of 1,202 signal change intervals during daytime hours and on dry pavement. At each of the study sites, two observers were used for simultaneous measurement of flow-related characteristics. Stopwatches were the primary tool for the data collection. Trucks accounted for only 1.5 to 5 percent of the observed vehicles at each of the study sites. They did not have appreciable effects on the information derived from the data.

The data concerning intersection width, grade, timing of the signal change interval, flow rate, and speed are summarized in Table 2. In Table 3 the characteristics are presented of the observed signal change interval requirements in comparison with the change intervals provided. Figure 1 shows the cumulative frequency distributions of the change interval requirements at five of the study sites. The distributions at the other sites are all confined by the distributions at Sites 1 and 5 as shown in the figure.

OBSERVED CHANGE INTERVAL REQUIREMENTS

Of the 1,202 signal change intervals observed, in 709 at least one vehicle crossed the intersection after the yellow onset. In these utilized change intervals, it took an average of 2.8 to 4.1 sec after the yellow onset to clear the intersection (Table 3). The 85th- and the 95th-percentile change interval requirements were considerably longer than the average requirements. The maximum observed requirements ranged from 5.8 to 8.9 sec.

In terms of the need to clear the intersection before the cross traffic is released, the actual change intervals provided at the study sites were inadequate. The percentage of the used change intervals that had insufficient lengths varied from 9.3 to 67.5; the overall average was 28 percent. To satisfy the 95th-percentile requirements for the change interval, the change intervals provided at the study sites would have to be raised by an average of 1.2 sec.

TABLE 2 Geometric Design, Signal Control, and Flow Characteristics at Study Sites

Site	Intersection Width (ft)	Grade (%)	Signal (sec) Yellow	Signal (sec) All Red	Flow Rate (vph)	Speed (mph) Mean	Speed (mph) 15th Percentile	Speed (mph) 85th Percentile	Speed (mph) SD
1	89	-1.0	3.4	1.1	350	28.9	25.7	32.3	2.9
2	89	+4.0	3.4	1.1	394	27.2	23.8	35.2	3.4
3	117	-0.5	3.1	1.9	228	31.1	27.5	35.8	3.2
4	74	-0.9	3.1	1.6	600	27.9	24.3	32.0	3.7
5	107	-0.3	3.2	1.8	170	32.2	28.4	36.5	3.5
6	106	-3.9	3.1	0.8	644	30.1	27.6	31.7	2.8
7	77	+3.3	3.6	0.0	420	20.4	17.2	23.4	2.9
8	90	+0.7	4.0	1.0	170	43.9	38.6	49.2	5.9
9	96	+0.2	3.1	1.7	270	33.0	28.1	37.6	4.8
10	195	+1.0	3.0	2.9	320	30.6	24.2	35.8	5.1
11	74	+0.4	3.1	1.8	390	30.8	27.3	35.1	3.8

Note: SD = standard deviation.

TABLE 3 Observed Signal Change Interval Requirements

Site	Sample Size	Percent of Sample Used	Percent of Sample Failing to Clear Intersection	Change Interval Needed (sec) Mean	85th Percentile	95th Percentile	Maximum	SD	Existing Interval (sec)
1	144	57.6	43.1	4.3	6.1	6.7	8.0	1.6	4.5
2	74	64.9	39.8	4.1	5.5	6.2	6.9	1.3	4.5
3	150	90.7	9.3	3.3	4.6	5.4	6.5	1.1	5.0
4	150	54.0	11.2	3.2	4.3	5.3	6.7	1.4	4.7
5	150	29.3	9.2	2.8	4.7	5.4	6.7	1.5	5.0
6	150	43.3	57.6	4.0	5.4	5.8	6.9	1.3	3.9
7	48	83.3	67.5	4.1	5.3	5.8	6.5	1.3	3.6
8	89	60.7	18.5	3.2	5.4	5.8	6.3	1.7	5.0
9	57	70.2	20.0	3.9	4.9	5.4	5.8	1.1	4.8
10	130	60.8	17.5	3.6	6.0	7.4	8.9	2.2	5.9
11	60	65.0	23.1	4.0	5.0	5.6	6.0	1.5	4.9

Note: Sample size = number of change intervals observed. SD = standard deviation.

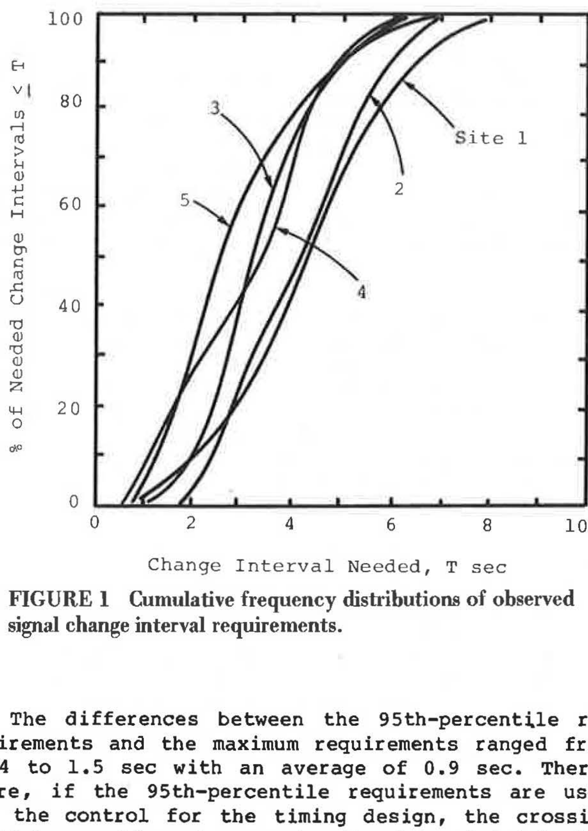

FIGURE 1 Cumulative frequency distributions of observed signal change interval requirements.

The differences between the 95th-percentile requirements and the maximum requirements ranged from 0.4 to 1.5 sec with an average of 0.9 sec. Therefore, if the 95th-percentile requirements are used as the control for the timing design, the crossing vehicles would need a margin of safety of 0.4 to 1.5 sec to avoid a collision. This margin of safety can usually be provided by two sources. One is the starting delay of the cross traffic and the time required for accelerating to the conflict point. On the basis of the characteristics of 3,527 vehicles, Chang et al. (10) found that 95 percent of the vehicles took more than 0.8 sec to start. Adding the acceleration time to this starting delay would result in a margin of safety of more than 1 sec.

Another source of the needed margin of safety is the low probability that a crossing vehicle with the maximum change interval requirement will encounter a vehicle that jumps the light in the cross traffic. No jumping of the light was observed at the study sites in the course of the data collection. Therefore, it appears logical to use the 95th-percentile change interval requirements as the design control. Longer change interval requirements should be considered for design if needed margins of safety cannot be adequately provided by the two sources just described.

COMPARISONS OF SIGNAL TIMING METHODS

Equations 1, 2, and 3 and the ITE proposed recommended practice are tested on the basis of the observed change interval requirements. The first step of this test involves the use of these equations to calculate the required change intervals for the study sites. On the basis of the cumulative frequency distributions of the observed change interval requirements, the percentage of these requirements satisfied by each of the calculated change intervals is then determined.

For this test, the observed 85th-percentile speeds are used for V in both Equations 1 and 3 and the distance between a crossing vehicle and the cross traffic at the conflict point is assumed to be 10 ft for Equation 2 (i.e., D = 10 ft). It should be noted that the change interval determined from Equation 1 or 3 on the basis of 85th-percentile speed can be either longer or shorter than the interval determined on the basis of average speed. Generally, however, the change interval calculated from these equations is not very sensitive to the approach speed. Pedestrians did not affect the measured change interval requirements at the study sites. Therefore, $(W + L)/V$ is used for determining the all-red interval in testing the ITE proposed practice.

Table 4 shows the results of the test. Several observations can be made from this table. First, none of the change intervals calculated from any of the equations can satisfy the 95th-percentile change interval requirements at all the study sites. This reflects the difficulty in using constant reaction time and deceleration rate to consistently produce good timing designs. Second, with the exception of Sites 2 and 6, the change intervals calculated respectively from Equations 1 and 3 for each of the

TABLE 4 Comparison of Timing Methods

Site	Equation 1 T (sec)	Equation 1 RS (%)	Equation 2 T (sec)	Equation 2 RS (%)	Equation 3 T (sec)	Equation 3 RS (%)	ITE T (sec)	ITE RS (%)
1	5.7	79	5.5	78	5.7	79	5.8	81
2	5.7	89	5.6	87	5.4	83	5.7	89
3	6.2	99	6.2	99	6.3	99	6.4	99
4	5.4	96	5.2	94	5.4	96	5.5	97
5	6.0	98	6.0	98	6.1	98	6.2	98
6	6.0	97	5.8	95	6.4	97	6.4	97
7	5.5	90	5.0	72	5.4	87	5.9	95
8	6.1	98	6.6	100	6.0	98	6.1	98
9	5.9	97	5.9	97	5.8	97	5.9	97
10	7.7	96	7.7	96	7.6	96	8.2	100
11	5.4	82	5.3	80	5.4	82	5.4	82

Note: RS = requirements satisfied.

study sites are within 0.1 sec of each other. If Equation 3 is replaced by Equation 4, these small differences in the calculated change intervals would become even smaller. Third, eight of the change intervals calculated from Equation 2 are in close agreement with those determined from Equation 1. For Site 7, which has a rather low 85th-percentile speed of 23.4 mph, Equation 2 produces an appreciably shorter change interval. In contrast, Site 8 has a relatively high 85th-percentile speed of 49.2 mph and, as a result, Equation 2 produces a substantially longer change interval. And, finally, the ITE proposed recommended practice shows a slight improvement over Equations 1, 2, and 3 in terms of the level of the change interval requirements satisfied by the calculated values of the change interval. This is because the ITE approach requires the use of the larger value of the change intervals calculated respectively on the basis of the 85th-percentile speed and the 15th-percentile speed. According to this recommended practice, the change intervals needed at the majority of the study sites would be governed by the 15th-percentile speed.

Overall, the ITE proposed practice and the use of any of the three equations can satisfy about the same level of the change interval requirements, and the level of satisfaction appears to be adequate for most of the study sites. Of course, the level of satisfaction can be easily manipulated by altering the parameter values used in calculating the change interval.

For the study sites, Equation 1 appears to be the most appealing because of its relative simplicity. This equation, however, has weaknesses. The derivation of Equation 1 is based on an implicit assumption that drivers behave according to a well-defined rule, which results in perceived change interval requirements that can be represented by the following general model:

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

The current practices consider A as the reaction time and B as equivalent to 1/(2a). Thus, the sum of A and BV represents the distance required for a vehicle to come to a stop in front of the stop line after the yellow onset. In reality, there are numerous combinations of A and BV that can result in the same stopping distance. Furthermore, A is not likely to be affected by the reaction time alone and B can be a function of the deceleration rate and many other variables. Therefore, it is perhaps more sensible to determine A and B based on aggregated change interval requirements observed at a large number of intersections.

Consider T as the 95th-percentile change interval requirements at the study sites and V as the average approach speed. A regression analysis based on Equation 5 results in the following equation:

$$T = 1.71 + 0.032\,V + [(W + L)/V] \tag{6}$$

where V is measured in feet per second, W and L are in feet, and T is in seconds.

This regression equation has an r^2-value of 0.19, and its standard error for estimating the observed 95th-percentile change interval requirements is 0.56 sec. Although the standard error is small, the low r^2-value suggests that the relationship as represented by Equation 5 is not a strong one.

Equation 5 may be modified into a more general form as follows:

$$T = A + BV + C[(W + L)/V] \tag{7}$$

The regression equation for this model on the basis of the same data is

$$T = 3.38 + 0.017V + 0.63\,[(W + L)/V] \tag{8}$$

Equation 8 has an r^2-value of 0.50 and a standard error of the estimate of 0.51 sec. Therefore, Equation 8 is a marked improvement over Equation 6. Still, it can be seen that the second term of Equation 8 will generally contribute very little to the calculated change interval. This suggests that the first two terms of Equation 7 may be combined into a single constant term. The resulting model becomes

$$T = A + B\,[(W + L)/V] \tag{9}$$

The regression equation for this model based on the field observations is

$$T = 4.36 + 0.56\,[(W + L)/V] \tag{10}$$

This equation can explain 47 percent (r^2 = 0.47) of the 95th-percentile change interval requirements at the study sites. The standard error of the estimate is 0.50 sec. Therefore, this equation is as good as the more complicated Equation 8.

One interesting feature of the model as represented by Equation 9 is that it conforms to timing designs that use a constant yellow interval. The constant term A in this equation may represent the yellow interval and the second term of the equation is the required all-red interval. The value of A in Equation 10 is very close to the constant yellow of 4.5 sec suggested by Chang et al. (10). This may be a mere coincidence because the data base used for the regression analysis is rather limited. To apply such an equation as Equation 10 for timing design, the calculated value of T should be increased slightly (e.g., by one standard error of the estimate) to account for the deviation of the observed requirements from the regression values.

CONCLUSIONS

The signal change intervals employed at the study sites were inadequate for clearing the intersection. To satisfy the 95th-percentile change interval requirements, the change intervals at the study sites have to be raised by an average of 1.2 sec.

Regression analyses of the field data indicate that the first two terms of Equation 1 can be combined into a single constant term to form a model in the form of T = A + B[(W + L)/V]. This model is much more logical than a generalized form of Equation 1 (i.e., Equation 5) in explaining the observed change interval requirements, and it also implies that the use of a constant yellow interval at a variety of intersections may be a rational approach. These findings, however, are strictly applicable only to the study sites. Further studies based on a larger data base are needed to draw definitive conclusions.

The timing design of the change interval can be based on the consideration of such driver behavior as reaction time and deceleration. But drivers' needs for the change interval can be affected by numerous factors. Therefore, the use of aggregated change interval requirements for timing purposes represents a simpler and at least equally logical alternative.

Future studies based on aggregated change interval requirements can rely on a straightforward regression analysis to develop a model for signal timing. Such studies should include intersections covering a wide range of geometric design and traffic flow conditions. The impact of steep grades, heavy trucks, and wide intersections on the signal change interval requirements is of particular concern.

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Discussion

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The author has presented data on the clearance-time requirements of a limited number of vehicles at 11 sites, which were compared with the clearance-time requirements calculated by the formula recommended by ITE and with the actual signal change intervals. Alternatives to the ITE timing formula were derived from analyses of traffic flow and intersection characteristics. From these analyses, the author claimed that the first two terms of the ITE formula ($t + V/2a$) can be combined into a constant term that is a much more "logical form" than the ITE formula and that will result in more accurate estimates of timing requirements. Closer examination of this study reveals that, in fact, the clearance-time data presented by the author are not inconsistent with the ITE formula and that the ITE formula and procedures are adequate for most intersections.

In arguing for alternative equations, the author has disregarded the statistical variation associated with their estimated parameters. Detailed comparison of the ITE formula and Equations 7 and 9 from Lin's analysis shows that they are not statistically different (Table 5). If the ITE parameters are compared with Equation 7, the estimates for the coefficients for A, B, and C are all within the 95 percent confidence interval of the comparable parameters in the ITE equation. If the average of the 85th-percentile vehicle speeds at all the sites (51.4 ft/sec) is

TABLE 5 Comparison of Timing Equations

Parameter	Estimate	Standard Error
ITE formula: $T = t + (V/2a) + C[(W + L)/V]$		
t	1.0	–
1/2a	0.05	–
C	1.0	–
Equation 7: $T = A + BV + C[(W + L)/V]$		
A	3.38	1.36
B	0.017	0.022
C	0.63	0.22
Equation 9: $T = A + B[(W + L)/V]$		
A	4.36	0.57
B	0.56	0.20

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substituted as a constant for V in the first term of the ITE formula (t + V/2a), it becomes

$$T = 3.57 + [(W + L)/V]$$

The first term (3.57) of this equation is within the statistical variability of Equation 9, but the coefficient for the second term is not (1.0 compared with 0.95). Overall, these alternative equations are not meaningfully different from the ITE formula.

Lin's data generally confirm the validity of the ITE formula and procedures. Although a timing equation with a constant term, such as that provided by Lin, may be simple, the ITE formula is preferable because it provides adequate timing that is based on specific combinations of traffic flow and intersection characteristics. In general, the alternative formulas derived by Lin will result in longer change intervals except for approach streets with slow traffic that must cross a wide cross street. Two recent studies have found that this scenario tends to have less-than-adequate change intervals and higher rear-end and right-angle crash rates (1,2).

Finally, the author has misinterpreted the first two terms of the ITE equation (t + V/2a) as representing "the time required for a driver to come to a stop after the yellow interval begins." On the contrary, this expression represents the time it takes vehicles to travel the distance within which most drivers would reject stopping. The actual time it takes to stop a vehicle is t + V/a, which is longer than the previous expression. Time is not a constraining factor for stopping but distance is.

The data provided by Lin in Table 4 show that the ITE formula typically provides sufficient signal change interval time for 95 percent of vehicles at most sites. The author has failed to demonstrate that either of the proposed alternative formulas is superior or significantly different from the ITE formula in terms of underlying theory or performance. There were, however, some sites for which the ITE formula, as well as Lin's equations, would have accommodated less than 90 percent of the clearing vehicles. It would be of interest to examine these intersections for specific characteristics that may contribute to this failure.

ACKNOWLEDGMENT

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REFERENCES

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Author's Closure

The evaluation and choice of regression models should take into account a number of factors. Among such factors are causal relationship between dependent and independent variables, ability of a model to explain the variation in the values of the dependent variable, standard error of the estimate, and so on. In this connection, the confidence interval as discussed by Stein is not a sufficient criterion. There are ample reasons why the ITE's formula is a poor choice.

Stein is correct in pointing out that t + V/(2a) does not mean what is indicated in my paper. A brief discussion of t + V/(2a) would shed some light on a troublesome aspect of the ITE formula. Let x be the distance between a vehicle and the stop line at the yellow onset, V, the vehicle approach speed at a location defined by x, t, the driver reaction time, and a, the deceleration rate to be applied to the vehicle. Then the following condition should be satisfied for the vehicle to come to a stop before the stop line:

$$x/V \geq t + V/(2a)$$

In deriving the ITE formula, x/V is set equal to t + V/(2a). Thus, t + V/(2a) represents the minimum travel time x/V to the stop line needed for a vehicle to come to a stop after the yellow onset. If the travel time x/V is less than t + V/(2a), that vehicle will go through the intersection. The study by Chang et al. (1) has revealed that, regardless of the approach speed, 95 percent of vehicles going through the intersection after the yellow onset require less than 4.5 sec to reach the stop line. This observation is further substantiated by a study carried out by Wortman and Fox and described in another paper in this Record. Additional field observations of straight-through movements at three sites by myself and a graduate student also reveal that 95 percent of through vehicles have travel times x/V less than 4.1 to 4.4 sec. These findings by various researchers suggest that the use of t + V/(2a) in timing the change interval contradicts the actual driver behavior. The field data indicate that t + V/(2a) can be replaced by a constant to better serve the timing purpose.

Stein mentioned that the ITE formula is preferable because it provides adequate timing that is based on specific combinations of traffic flow and intersection characteristics. In fact, there is nothing the ITE formula can do that cannot be done by a much simpler formula such as Equation 9. Both formulas contain the same variables (t and a are constants in the ITE formula).

It should also be noted that the analysis of the confidence interval of a regression equation as suggested by Stein is possible only if it is assumed that the dependent variable (T in this case) has a normal distribution. Users of packaged computer programs may not be aware of this constraint because such programs are mainly concerned with producing statistics rather than providing the users with the theoretical background of a particular statistical analysis technique. At the present time there is no evidence to support the assumption that the 95th-percentile change interval requirements have a normal distribution. Therefore, the analysis of the confidence interval of Equations 7 and 9 is really an academic exercise.

If the normality assumption is accepted, it also becomes possible to determine whether the resulting regression coefficients are significantly different from zero in a statistical sense. In such a case, Stein should have checked the significance of the resulting coefficients in Equations 7 and 9 before analyzing the related confidence intervals. Furthermore, on the basis of the normality assumption and the data given in Table 5, it can be shown that coefficient B of Equation 7 is not significantly different from zero at a 5 percent level of significance. In contrast, coefficient C in Equation 7 and coefficient B in Equation 9 are significantly different from zero at the same level of significance. In other words, if the normality assumption is correct, then Equation 7 is invalid because it should not include the second term BV. Without the normality assumption, BV can also be dropped from Equation 7 by noting that BV is small in comparison with the sum of the other two terms in the same equation.

The ITE formula is also inferior to Equation 9 in terms of its ability to explain the variation in the observed change interval requirements. This feature has already been discussed in my paper. Figures 2 through 5 further demonstrate this weakness of the ITE formula. These figures are based on the data reported in my paper and additional data collected at eight sites in Albany, New York.

The readers of this paper should be advised not to use Equation 10 for direct applications. This equation serves the purpose of illustrating a need to reexamine the philosophy in timing the change interval. A larger data base is needed to provide a more reliable estimation of the related regression coefficients. Moreover, the resulting equation should

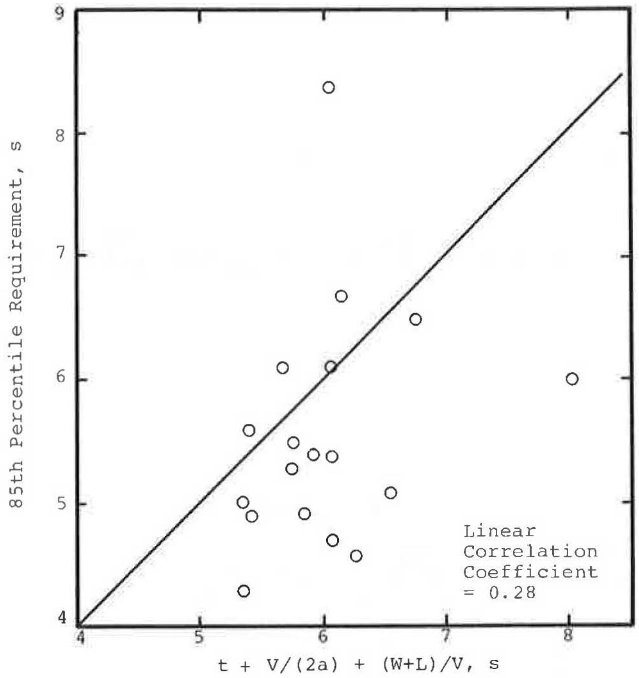

FIGURE 2 Correlation between the 85th-percentile change interval requirements and values determined from ITE formula ($t = 1$ sec, $a = 10$ ft/sec^2, V = mean speed).

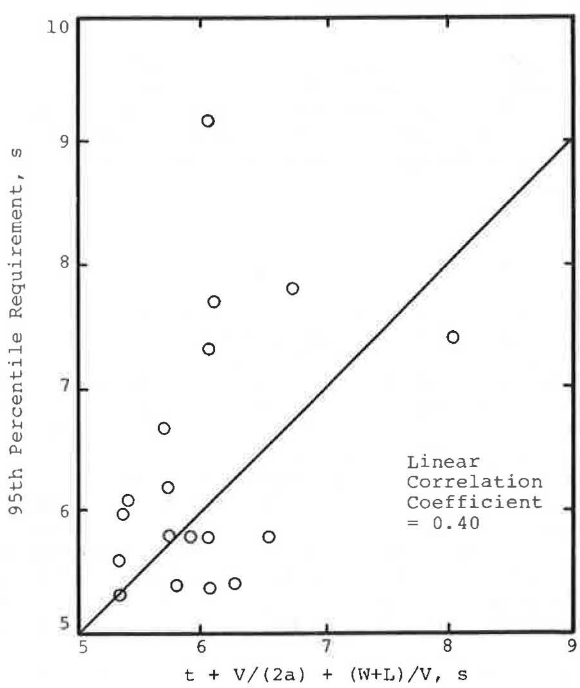

FIGURE 3 Correlation between 95th-percentile change interval requirements and values determined from ITE formula.

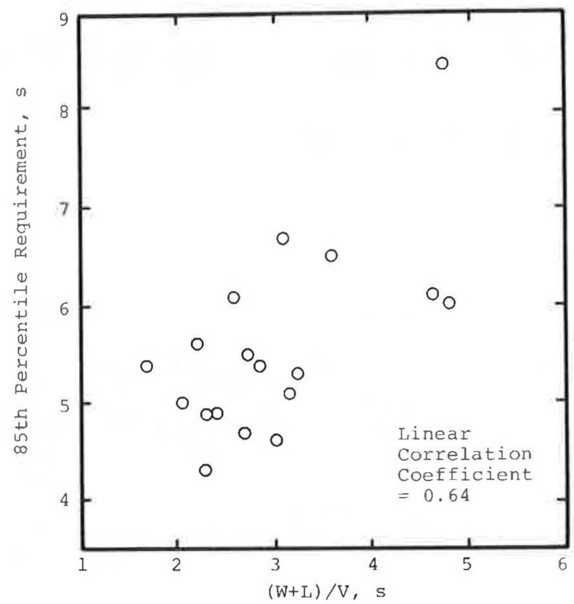

FIGURE 4 Correlation between 85th-percentile change interval requirements and (W + L)/V.

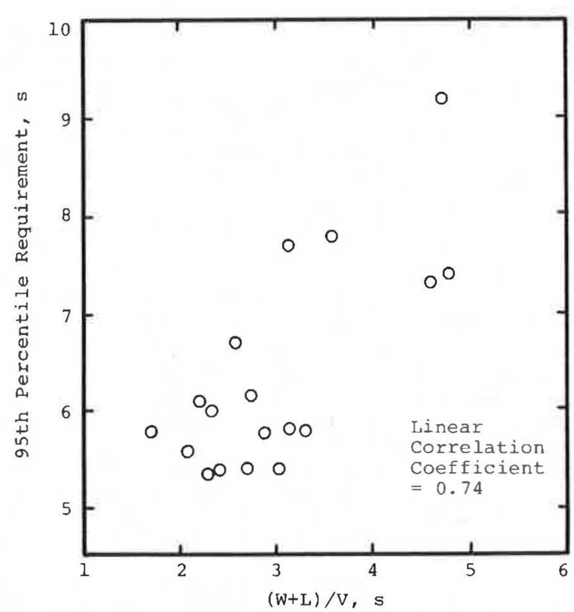

FIGURE 5 Correlation between 95th-percentile change interval requirements and (W + L)/V.

be modified into a timing tool by considering the variation from the regression line.

In summary, the ITE formula is based on a very shaky theoretical ground and it has a poor ability to explain the variation in the driver needs for the change interval. I do not recommend continued use of either T + V/(2a) to determine the yellow interval or the ITE formula to determine the entire change interval.

REFERENCE

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Guidelines for Protected/Permissive Left-Turn Signal Phasing

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ABSTRACT

Guidelines for the use of protected/permissive left-turn signal phasing were developed by collecting and analyzing data on traffic and roadway conditions for protected-only, protected/permissive, and permissive left-turn phasings to identify relationships among these conditions and signal phasings. The following left-turn signal aspects are addressed: (a) traffic volume based on the peak-hour minimum left-turn volume and the product of the peak-hour left-turn and opposing volumes with lower and upper limits, (b) annual left-turn accident experience based on the critical number and rate, (c) left-turn traffic conflict experience based on the critical number and rate, (d) left-turn delay, (e) site condition, (f) user cost savings for protected/permissive versus protected-only left-turn phasing, and (g) traffic engineering judgment.

Protected/permissive (P/P) left-turn signal phasing is a combination of a protected phase, in which a green arrow indicates a protected turn, and a permissive phase, in which the left-turning vehicles must yield to the opposing traffic during the green indication. The primary intent is to increase the efficiency of traffic flow by permitting left-turning movements through gaps in the opposing traffic at intersections where traffic volumes warrant a separate left-turn phasing. P/P phasing also reduces delay and energy consumption.

However, in two research efforts it was found that accidents involving left-turning vehicles increased after the installation of P/P signals (1,2). The number of accidents appeared to decrease as drivers became familiar with the signals, and driver understanding of the P/P phasing was identified as an important factor. However, because at some intersections operational and accident problems have not decreased over time, it appears that factors other than unfamiliarity cause problems.

Because the guidelines for a separate left-turn signal found in the literature vary considerably, no clear, consistent set of guidelines could be derived from a synthesis. Moreover, the quantitative guidelines are only for a separate left-turn phase and do not specify the selection of P/P versus protected-only (PO) phasing. The P/P guidelines lack quantitative measures that would eliminate much of the judgment and potential for error involved in selecting a P/P phasing.

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OBJECTIVE AND SCOPE

In light of the foregoing, research was undertaken to develop guidelines for the use of P/P left-turn phasing. This was achieved by collecting data on traffic and roadway conditions for the three left-turn phasings and then analyzing the data to identify relationships between the left-turn phasing and traffic and roadway conditions.

Because the majority (about 95 percent) of the P/P left-turn signals designed by the Virginia Department of Highways and Transportation contain a leading green arrow, only leading-arrow phases were considered. Study sites were limited to signalized intersections along arterial routes in suburban areas because these are of primary interest to the Department.

In establishing the guidelines for use of P/P left-turn phasing, guidelines for the use of permissive (P) and PO left-turn phasings were indirectly addressed.