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Optimal Timing Settings and Detector Lengths of Presence Mode Full-Actuated Control

FENG-BOR LIN

ABSTRACT

The operation of presence mode full-actuated signal control at individual intersections is governed primarily by the choice of detector length and the timing settings of vehicle interval and maximum green. The relationships between these control variables and the control efficiency vary with the flow pattern at an intersection. Based on the results of computer simulations, the optimal combinations of detector length, vehicle interval, and maximum green are identified for a wide range of flow conditions. The analyses performed in this study concern only intersections where vehicle approach speeds are less than 35 mph.

Full-actuated signals based on long loop presence detectors are being widely used for the regulation of traffic flows at individual intersections. This presence mode control, which is also referred to as loop-occupancy control, can rely on a variety of timing settings and detectors. Nevertheless, the typical operation of this mode of control is governed by three basic control variables: vehicle interval, maximum green, and detector length. Vehicle interval determines the longest duration in which detectors can be left unoccupied without prompting the termination of a green duration. Maximum green limits the maximum green duration allowable to a signal phase after a vehicle actuates a detector of a competing phase.

Some researchers have attempted to quantify the performance of the presence mode control under certain operating conditions, but so far the findings are inconclusive regarding the optimal use of presence mode control. for example, Cribbins and Meyer (1) examined the effects of detector length on the control efficiency under real-life conditions. They concluded that the longer the length of the presence detector on the major approach to an intersection, the longer the delay. The conditions under which various detector lengths were examined, however, are unknown.

To provide further insights into the effects of detector length, Tarnoff and Parsonson (2) used the NETSIM simulation model (3) to compare detector lengths of 30 to 90 ft. They found that the efficiency of the presence mode control increased as the detector length was shortened. But they also cautioned that the simulation results did not properly account for the possibility that a signal phase could be prematurely terminated because of the variations in queue discharge headways. Tarnoff and Parsonson's caution is not unwarranted. A recent study by Lin and Percy (4) has indicated that the risk of the premature phase termination is not negligible and can significantly affect the operating characteristics of the presence mode control.

Generally, current understanding of the performance characteristics of the presence mode control is piecemeal and mostly intuitive in nature. As a result, it is not clear how this mode of control can be used to achieve the highest possible control efficiency. To bridge this gap in the state of the art of signal control, this study was conducted to determine the relationships between the optimal use of the presence mode control and the flow patterns at individual intersections where vehicle approach speeds are less than 35 mph.

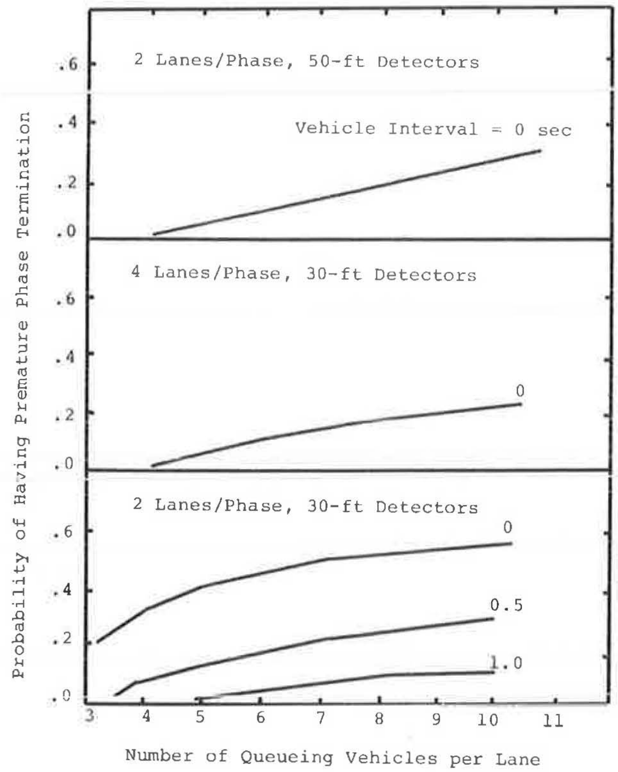

FIGURE 1 Probabilities that queuing vehicles face prematurely terminated green phases.

METHOD OF PERFORMANCE ANALYSES

It is generally impractical to conduct field studies to determine how the performance of the presence mode control would change under different operating conditions. A practical alternative is to use computer simulation for the performance analysis of such a signal control. But past efforts in developing simulation models largely overlooked the importance of a reasonably accurate representation of the interactions between queuing vehicles and presence detectors. Consequently, existing models may not be suitable for use as a tool to identify the optimal use of the presence mode control.

The NETSIM model (3), for example, assumes that every vehicle in a queue can extend a green duration until the queue dissipates completely. Field data collected in this study (5), however, indicate that this is not always the case. In fact, the probability of premature phase termination caused by the failure of queuing vehicles to extend green durations can be rather high, even when 50-ft detectors are used. Figure 1 presents a few examples of this phenomenon. The existence of prematurely terminated green durations can be expected to result in poor operation of the presence mode control. Ignoring it would certainly lead to underestimates of vehicle delays.

To avoid introducing systematic biases into the analysis of presence mode signal operations, a simulation model referred to as the RAPID model (5) was used in this study. This model is a microscopic simulation model capable of duplicating the dynamic and probabilistic interactions between queuing vehicles and presence detectors. This capability is indispensable because the performance of the presence mode control is dictated by such interactions. Furthermore, the model does not contain any assumption that would misrepresent the actual operation of the presence mode control.

To facilitate this study, the RAPID model was calibrated with field data collected at two intersections (5). These data concern the interactions between queuing vehicles and detectors 30 to 120 ft long. Based on this calibrated model, the optimal combinations of detector length, vehicle interval, and maximum green were determined for a variety of flow patterns. Vehicle delays were used as the measure of performance in search of such optimal combinations.

The average vehicle delay associated with a presence mode operation is a random variable. Its true value can only be estimated. Therefore, the optimal control referred to herein for a given flow pattern is in fact an approximate solution. The procedure used to search for such an optimal control was simple but tedious. Detector lengths of 30, 50, 65, 80, and 120 ft are evaluated separately first. For each detector length, the best combinations of vehicle interval and maximum green were identified for a number of flow patterns. The average delay produced by each combination of vehicle interval and maximum green was estimated on the basis of the outputs of at least four simulation runs. For a flow pattern with heavy lane flows, the average delay could vary substantially from one simulation run to another. In such a case additional simulation runs were performed to obtain a better estimate of the average delay.

The flow patterns examined in this study represent a number of combinations of flow rate per lane, distribution of traffic volume among lanes, and temporal variations in flow rate. The vehicles associated with these flow patterns included straight-through and right-turn movements with a negligible number of trucks and buses. These two directional movements represent two extreme flow conditions and were analyzed separately. Both two- and four-phase operations of the presence mode control were analyzed for each flow pattern. The rest-in-red feature was assumed to be in effect. Also, each signal phase contained up to four lanes. The ratio of the critical lane flow in one phase to that in another phase was varied from 1 to 2. The flow rate in a lane ranged from 50 to 100 percent of the critical lane flow of the same phase.

Each combination of flow pattern, detector length, vehicle interval, and maximum green was analyzed on the basis of a 1-hr operation of the signal control. In such an hourly operation, the flow rate in each lane was allowed to vary from time to time at 5-min intervals. A factor, referred to herein as the peaking factor (PF), was used to represent the degree of such temporal variations in the flow rate. This factor is defined as

$$PF = \text{Hourly volume}/(4 \times \text{peak 15-min volume}) \qquad (1)$$

A peaking factor of 1.0 indicates a uniform flow rate. A lower peaking factor implies that there is a higher concentration of traffic in a short period of time. The signal operation for each flow pattern was analyzed, respectively, at peaking factors of 1.0, 0.85, and 0.7. At any flow rate, the arrivals of the vehicles were assumed to be random.

PERFORMANCE CHARACTERISTICS

The choice of detector length, vehicle interval, and maximum green can affect the risk of the premature phase termination. It can also affect the speed profile of a vehicle before and after the actuation of a detector and the degree of easiness or difficulty for vehicles not in a queue to extend a green phase. The resulting relationships between the control efficiency and the control variables are complex.

Among the three control variables, maximum green plays a relatively simple and easily identifiable role in shaping the operation of the presence mode control. The maximum green chosen for a specific signal operation is dormant until the arriving vehicles are able to extend a green phase continuously. The potential impact of this signal control variable is shown in Figure 2.

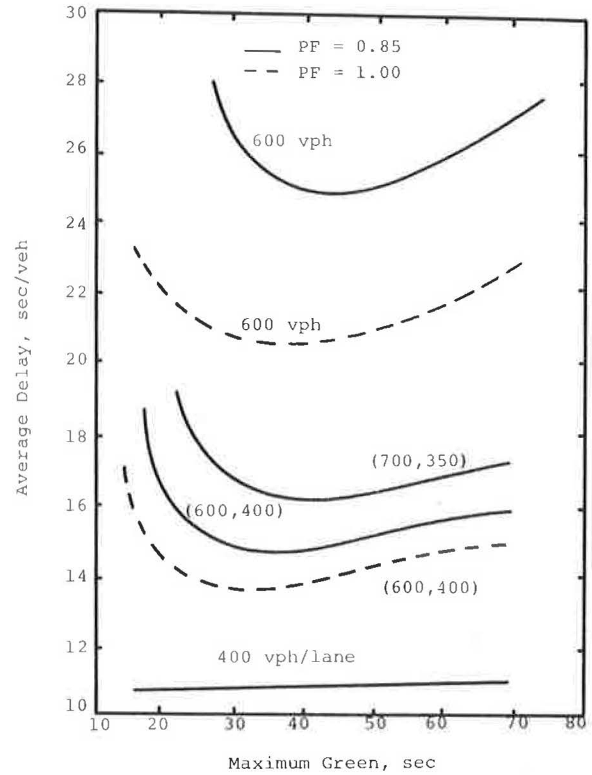

FIGURE 2 Average delay as a function of maximum green and lane flow for two-phase operations.

One feature revealed in this figure is that the delays are insensitive to maximum green when the flows are relatively low [e.g., 400 vehicles per hour (vph) per lane]. For a flow pattern with heavier flows, short maximum greens become undesirable. In such a case the average delay begins to increase rapidly when the maximum greens fall below a certain level. Long maximum greens, however, may not have a significant adverse impact on the average delay.

Figure 2 also shows that optimal maximum green can vary with the peaking factor. The general trend as revealed by the simulation data is that the optimal maximum green decreases when the peaking factor increases. This is not an unexpected result. A flow pattern with a strong peaking characteristic (i.e., small peaking factor) implies a high concentration of traffic volume in a short period of time. For a given hourly flow rate, the smaller the peaking factor, the heavier the traffic becomes in such a period and the longer the maximum green should be.

The effects of detector length and vehicle interval on control efficiency are much more difficult to generalize. Nevertheless, the operation of the presence mode control is governed primarily by the sum of the dwell time of a vehicle in a detection area and the vehicle interval provided. This sum can be referred to as the effective vehicle interval faced by a vehicle. The dwell time is a function of detector length and can vary from one vehicle to another. Consequently, the effective vehicle intervals faced by the arriving vehicles also vary.

If such effective vehicle intervals are short, both queuing vehicles and vehicles not in a queue may have great difficulties extending a green phase. On the other hand, long effective vehicle intervals may allow vehicles separated by long headways to extend a green phase. In either case, control efficiency can be expected to be poor.

After a green phase begins, the queue in a lane will grow and decay at the same time. Eventually such a queue will dissipate. For the vehicles in a queue, the premature termination of a green phase should be prevented. Otherwise the queue length may grow from one cycle to another, thus inducing excessive delays. Under highly variable flow conditions, the premature phase termination can be effectively prevented if long detectors (e.g., 80 ft) are used. Once the risk of the premature phase termination is negligibly small because of the use of long detectors (e.g., 80 ft or longer) or because of the presence of light traffic flows, then longer vehicle intervals may lead to increased delays. These characteristics of the presence mode control are clearly revealed in Figure 3.

After a queue dissipates from a detection area, it may be desirable to allow some vehicles that are following behind to extend the green phase. The vehicles not in the queue generally have longer headways than the queuing vehicles in the same lane. Furthermore, they are faced with effective vehicle intervals that are shorter than those encountered by the queuing vehicles. Consequently, such vehicles can be expected to have substantial difficulties in extending a green phase. An approximate analysis given in the following paragraphs underscores this phenomenon.

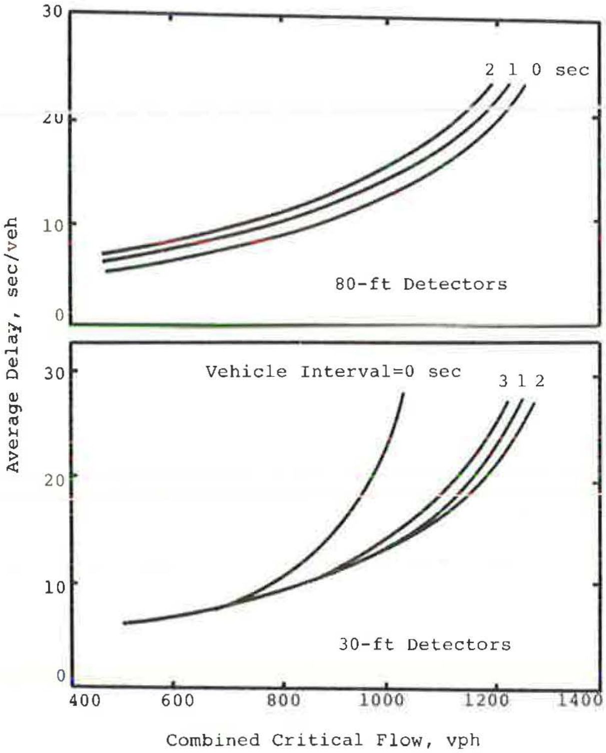

FIGURE 3 Variations in delays with combined critical flow and vehicle interval (two phases, four lanes per phase, PF = 0.85).

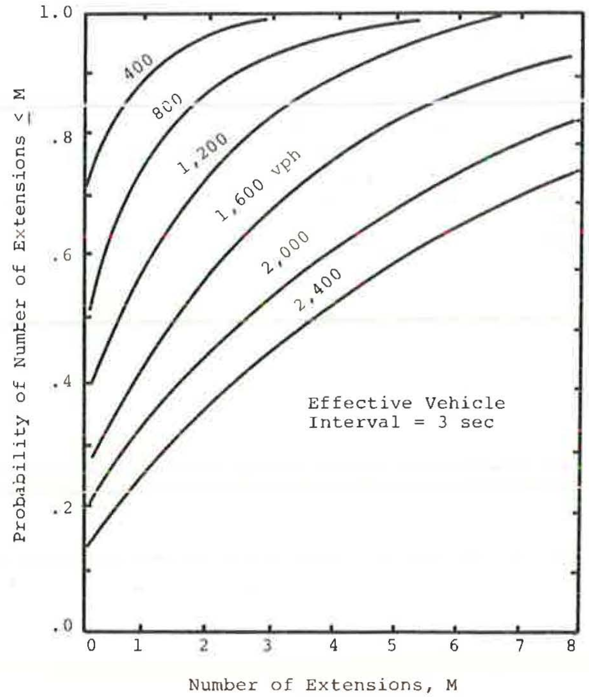

FIGURE 4 Probabilities that M or fewer vehicles not in a queue will be able to extend a green phase.

Consider a green phase that is associated with more than one lane and assume that vehicles that cannot join a queue arrive randomly at the upstream end of the detector in a lane. With these random arrivals, it can be shown (6) that the arrival headways of the combined flow can be represented by the following probability density function:

$$F(h \geq t) = \exp\left[-(\lambda_1 + \lambda_2 + \ldots + \lambda_i \ldots)t\right] = e^{-\lambda t} \quad (2)$$

where

$F(h \geq t)$ = probability that a headway h is greater than or equal to t,
λ_i = flow rate in lane i, and
λ = combined flow rate.

As an approximation, let the effective vehicle interval faced by each of such vehicles be the same. Denote this effective vehicle interval as U. Then, for a vehicle in the combined flow to extend a green phase, its headway should not exceed U. The probability that a vehicle will be able to extend the green becomes $1 - e^{-\lambda U}$. The corresponding probability Y that exactly M vehicles will be able to extend the green in succession is

$$Y = (1 - e^{-\lambda U})^M e^{-\lambda U} \quad (3)$$

Based on this equation, the probability that M or fewer vehicles will be able to extend a green phase can be estimated for various combinations of λ and U. Figure 4 shows that, with an effective vehicle interval of 3 sec, the median number of vehicles that can extend a green phase in succession is only about 2.5 when the combined flow is 2,000 vph. If the effective vehicle interval is increased to 4 sec, it can be shown that the corresponding median value is still only five vehicles. This is an average of 1.25 vehicles per lane if the combined flow is distributed among four lanes. Therefore, unless the effective vehicle intervals faced by the arriving vehicles are longer than 4 sec and the combined flow is extremely heavy, the presence mode control will rarely allow a vehicle not in a queue to extend a green phase.

Allowing vehicles not in a queue to extend a green phase is desirable only when the combined critical flow of a traffic pattern is heavy. With a combined critical flow exceeding 1,200 vph, for example, 1-sec vehicle intervals tend to produce more efficient operations than 0-sec vehicle intervals when 65-ft detectors are used. No field observations have been made on vehicle movements over 65-ft detectors. Nevertheless, the probability of the premature phase termination associated with the use of such detectors can be expected to be negligibly small. This implies that the 1-sec vehicle intervals needed to minimize delays are primarily to allow some vehicles not in a queue to extend a green phase.

OPTIMAL UTILIZATION

Optimal Maximum Green

Maximum green is usually set between 30 and 60 sec (7). Current practices in selecting the maximum green appear to be arbitrary. For the purpose of preventing a green phase from becoming unreasonably long to waiting drivers, the maximum green may be set in accordance with a tolerable waiting time. How long a waiting time is tolerable is, of course, subject to intuitive judgment.

To maintain high control efficiency under varying flow conditions, it has also been suggested (7) that the maximum green be selected to correspond to the desired cycle length and split at an intersection. Following this suggestion, the optimal pretimed cycle length and green durations for a flow pattern being considered can be determined first. The computed green durations are then multiplied by a factor ranging between 1.25 and 1.50 to obtain the maximum greens (7). This approach is logical, but the basis for choosing a value between 1.25 and 1.50 as the multiplication factor is not clear.

The simulation results obtained in this study indicate that the optimal maximum green of a phase can be related to the corresponding optimal pretimed green and the peaking factor. Figure 5 shows such relationships. The optimal pretimed green, denoted

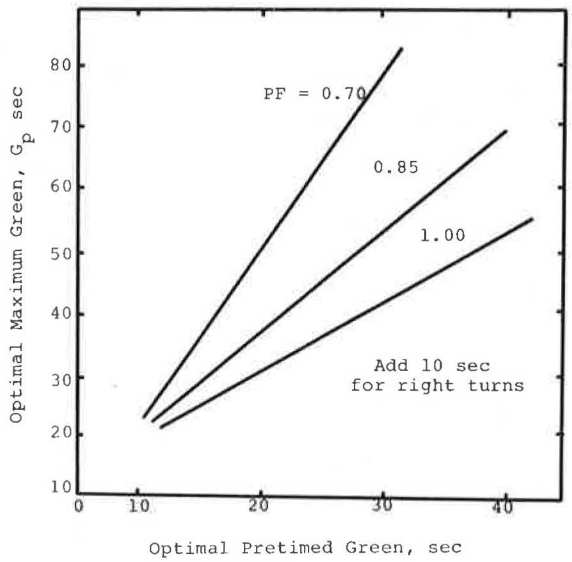

FIGURE 5 Recommended maximum greens.

as G_p in the figure, is determined from the following optimal pretimed cycle length (8):

$$C_o = (1.5\ L + 5)\bigg/\left(1 - \sum_{j=1}^{N} Z_i\right) \quad (4)$$

where

C_o = optimal pretimed cycle length (sec);
L = loss time per cycle, taken as 5 sec per phase;
Z_i = ratio of critical lane volume of phase i to saturation flow of 1,800 vph; and
N = number of signal phases.

Given C_o, the available green time is allocated to each phase in proportion to the critical lane volume; that is,

$$G_p = \left(C_o - \sum_{j=1}^{N} Y_j\right)\bigg/\left(Q_{ci}/\sum_{j=1}^{N} Q_{cj}\right) \quad (5)$$

where

G_p = pretimed green duration of phase i,
Y_j = clearance interval of phase j, and
Q_{ci}, Q_{cj} = critical lane volumes of phase i and phase j, respectively.

A few observations can be made from Figure 5. First, with a peaking factor of 1.0, the optimal maximum greens are about 10 sec longer than the corresponding optimal pretimed greens. Second, when the peaking factor decreases to 0.85, the optimal maximum greens are about 80 percent longer than the pretimed greens. Finally, the optimal maximum greens for a peaking factor of 0.7 are about 2.5 times the optimal pretimed greens. The optimal maximum greens for flow patterns with only right-turn flows are longer than those for straight-through flows. The difference is about 10 sec.

The consequences of using maximum greens that deviate from the values given in Figure 5 are shown in Figure 6. Each curve of this figure represents the average delays for a given flow pattern when maximum green is varied. It is obvious from this figure that the use of maximum greens 10 sec shorter than the values given in Figure 5 should be avoided. On the other hand, maximum greens 20 sec longer than such values may increase average delays only slightly.

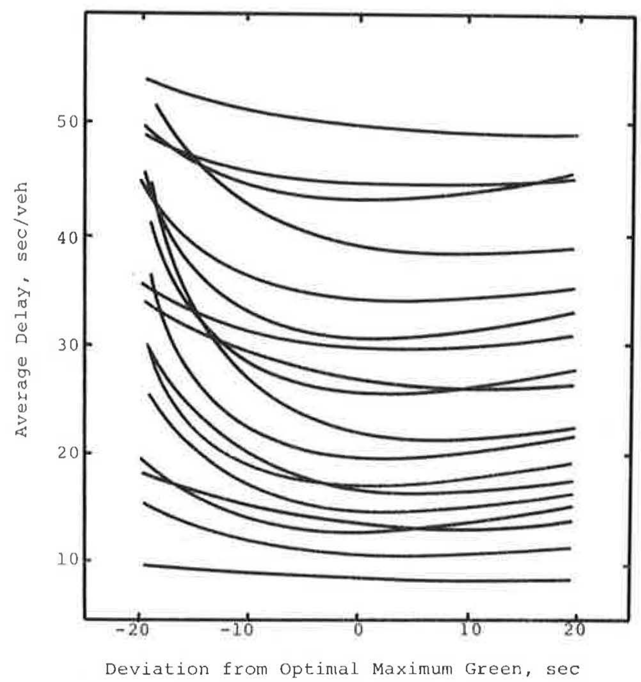

FIGURE 6 Variations in average delays as a function of maximum green.

Because the traffic volume at an intersection varies from time to time, the optimal maximum green of a signal phase should be determined on the basis of the peak-hour flow pattern when only one maximum green per phase is allowed. If two settings of maximum green are allowed, one setting should be based on the peak-hour flow patterns and the other based on a pattern with moderate flow rates. The maximum green should be limited by drivers' tolerance to waiting.

Optimal Vehicle Interval

For detectors at least 80 ft in length, 0-sec vehicle intervals can be expected to produce the most efficient signal operations. When shorter detectors are used, the optimal vehicle intervals depend primarily on the combined critical flow of the traffic pattern. A heavier combined critical flow generally requires a longer vehicle interval in order to minimize delays.

When 1-sec vehicle intervals are chosen over 2-sec vehicle intervals for 30-ft detectors, Figure 7 shows that the average delays may be reduced by up to 2 sec per vehicle for straight-through flows and up to 4 sec per vehicle for right-turn flows if the combined critical flow is less than 900 vph. Under heavier flow conditions, 2-sec vehicle intervals become much more desirable than 1-sec vehicle intervals. For straight-through flows with a combined critical flow of more than 1,000 vph, there is an increasing need to use 3-sec vehicle intervals. For right-turn flows it becomes advantageous to use 3-sec vehicle intervals only when the combined critical flow approaches 1,400 vph. The use of vehicle intervals longer than 3 sec, however, would reduce control efficiency.

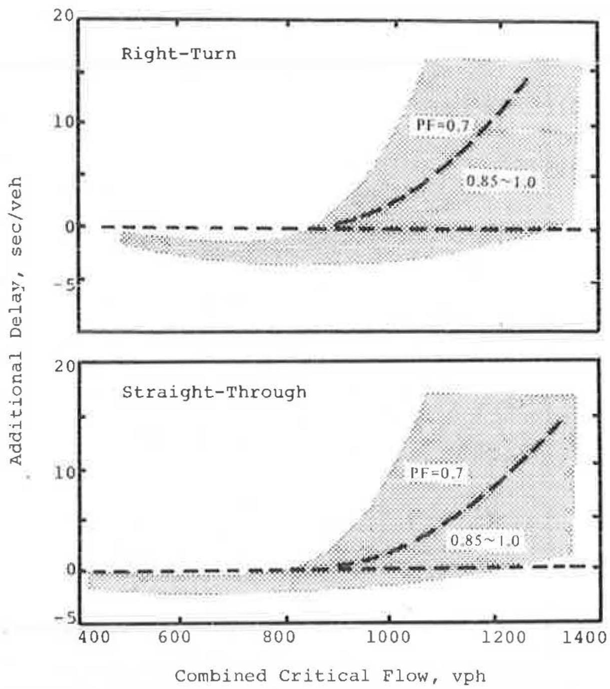

FIGURE 7 Additional delays caused by the choice of 1-sec vehicle intervals over 2-sec vehicle intervals (30-ft detectors).

For 50-ft detectors serving straight-through flows, 1-sec vehicle intervals are always better than 2-sec vehicle intervals when the combined critical flow is less than 1,000 vph. Above this flow level the relative efficiencies of 1- and 2-sec vehicle intervals depend on the specific flow pattern at an intersection. Generally, 2-sec vehicle intervals can become slightly better when the peaking factor approaches 0.7, whereas 1-sec vehicle intervals are preferred when the peaking factor is greater than 0.85. The differences in the resulting delays, however, are less than 1.5 sec per vehicle and thus can be ignored. When the combined critical flow is less than 800 vph, 0-sec vehicle intervals are preferred to 1-sec vehicle intervals. Under heavier flow conditions, it becomes important to use 1-sec vehicle intervals.

To serve right-turn flows, there is no advantage of using vehicle intervals longer than 1 sec for 50-ft detectors. For such directional flows, 0-sec vehicle intervals produce more efficient control than 1-sec vehicle intervals when the combined critical flow is less than 900 vph. Once the combined critical flow exceeds 900 vph, there is an increasing need to use 1-sec vehicle intervals.

The optimal vehicle intervals for 65-ft detectors are in the range of 0 to 1 sec. The use of longer vehicle intervals can be expected to induce additional delays. For 65-ft detectors used to serve straight-through flows, 0-sec vehicle intervals are preferred to 1-sec vehicle intervals when the combined critical flow is less than 1,000 vph. Under heavier flow conditions, 1-sec vehicle intervals should be used. For right-turn flows, the use of 0-sec vehicle intervals is generally desirable when the combined critical flow is less than 1,100 vph. Above this level of combined critical flow there is an increasing need to use 1-sec vehicle intervals.

Based on these findings, a set of vehicle intervals is determined and recommended for timing design applications. These recommended vehicle intervals are shown in Figure 8. The shaded areas in this figure represent various ranges of vehicle intervals in which the control efficiency is not likely to vary significantly. Nevertheless, it is desirable to use the upper bounds of such ranges to choose a vehicle interval when the peaking factor of a flow pattern approaches 0.7. The lower bounds are to be used when the peaking factor is between 0.85 and 1.0. As in the case of selecting a maximum green, the timing design can be based on the peak-hour flow pattern expected at an intersection.

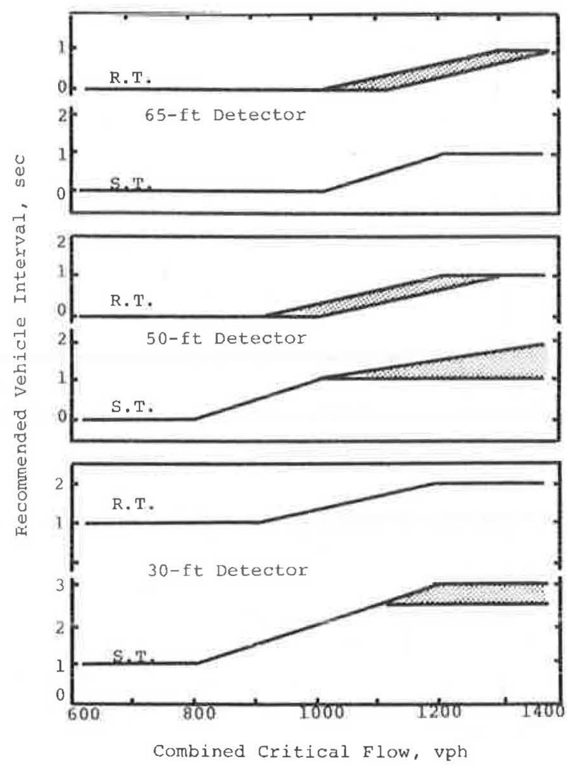

FIGURE 8 Recommended vehicle intervals.

For detector lengths not shown in Figure 8, their optimal vehicle intervals can be estimated through interpolations.

Optimal Detector Length

It has been suggested (9) in the past that required detector lengths be determined from the following equation:

$$D = 1.47\, V(U - E) - L \quad (6)$$

where

D = detector length (ft),
V = design approach speed (mph),
U = desired effective vehicle interval (sec),
E = vehicle interval (sec), and
L = design vehicle length (ft).

The primary concern of this equation is to give vehicles not in a queue a reasonable chance to extend a green phase. To serve this purpose, Equation 6 in fact equates the sum of the dwell time (D + L)/(1.47V) of such vehicles and the vehicle interval E to a desired effective vehicle interval U. An effective vehicle interval of 3 sec is usually considered to be adequate.

Equation 6 is convenient to use, but the detector lengths determined from it are unlikely to be the most desirable. A major reason for this is that the effective vehicle interval U that should be used in the equation has not been clearly specified. For a detector of 30 to 65 ft, it has been shown (Figure 8) that the vehicle interval needed to minimize delays increases with the combined critical flow of a flow pattern. This implies that the effective vehicle interval U should be related to traffic volume. Furthermore, the operation of the presence mode control is governed primarily by the interactions between queuing vehicles and detectors. Therefore, the use of Equation 6 may lead to good choices of detector length for some flow patterns and poor choices for others.

When compared with the other detector lengths examined in this study, 80-ft detectors with 0-sec vehicle intervals were found to be able to produce either better or at least equally efficient signal operations. Figure 9 provides an insight into the relative efficiencies of the various detector lengths. Each delay curve shown in the figure is associated with a flow pattern under either a two- or four-phase signal control. It can be seen from the figure that the control efficiencies produced by 65- and 80-ft detectors are comparable over a wide range of flow conditions.

Detectors shorter than 80 ft are frequently used because of budget constraints. Before such detectors are used, their potential impact on control efficiency should be evaluated. Figure 10 shows an example of the additional delays that may result from the use of detectors shorter than 80 ft. The information contained in a figure such as this can be used to assist in the choice of detector lengths. For example, the data in Figure 10 indicate that, when the combined critical flow is less than 1,000 vph, the average delays caused by the use of 65-ft detectors are less than 1.5 sec per vehicle longer than those produced by the use of 80-ft detectors. Therefore, if this magnitude of the added delays is deemed to be insignificant, then 65-ft detectors may be employed.

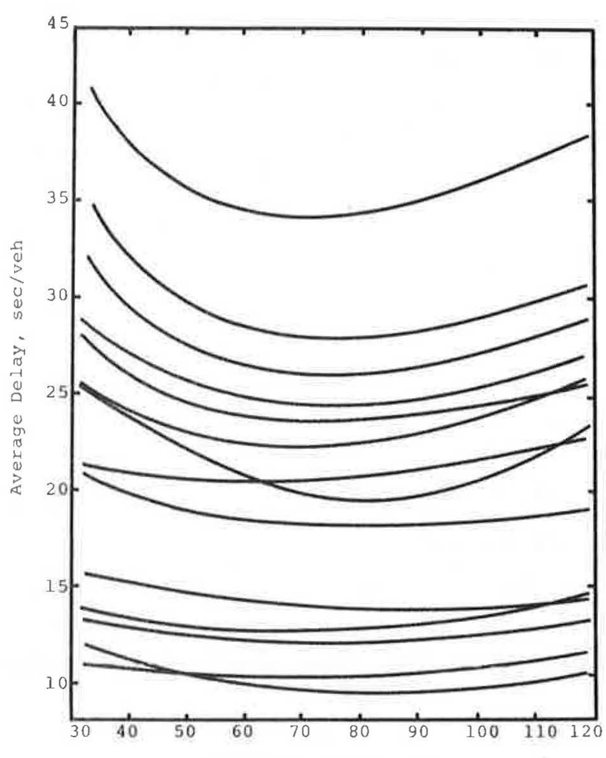

FIGURE 9 Variations in average delays with detector length.

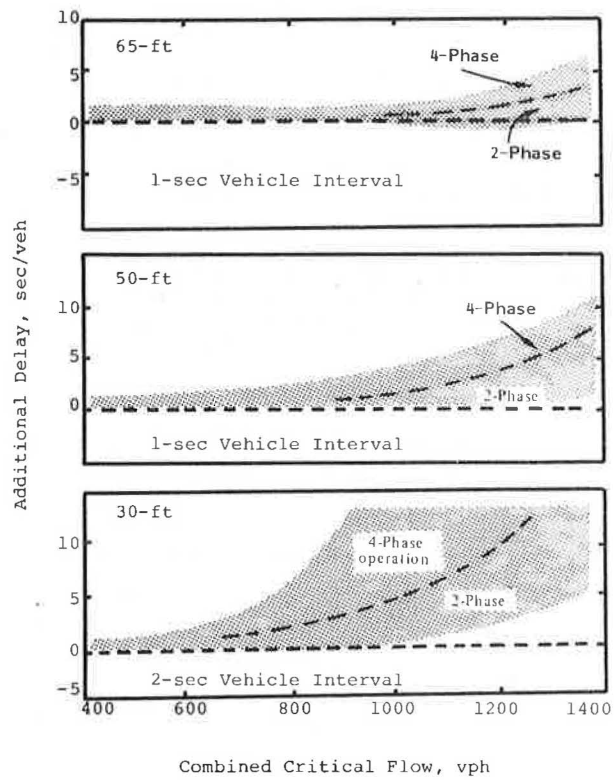

FIGURE 10 Additional delays caused by the use of detectors shorter than 80 ft (straight-through flows).

Short detectors may be used in place of 80-ft detectors as long as their use is not likely to incur undue additional delays. Figures 11 and 12 show such acceptable detector lengths at two levels of allowable added delays for straight-through and right-turn flows, respectively. These figures indicate that, to maintain a specified level of control efficiency, detector length should increase with combined critical flow. They also reveal that, if an added delay of 5 sec per vehicle is acceptable, 30-ft detectors can be used for flow patterns with combined critical flows of up to about 800 vph. Four-phase operations require longer detector lengths than two-phase operations because they incur longer delays, and such delays are more sensitive to the choice of detector length.

The detector lengths determined from Figures 11 and 12 should be used in conjunction with the

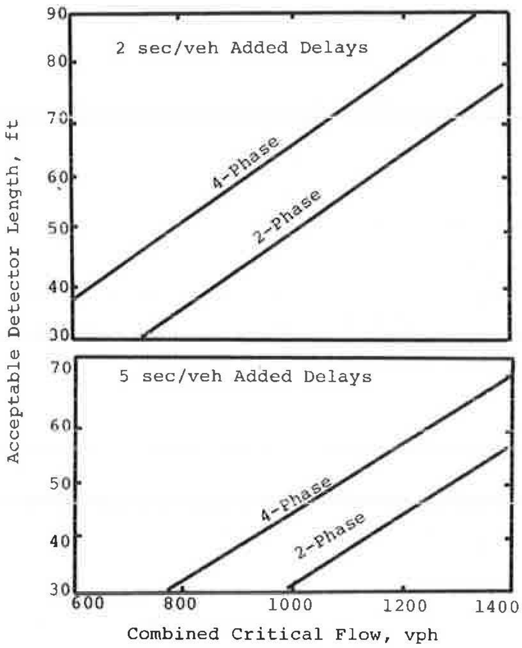

FIGURE 11 Acceptable detector lengths and allowable added delays (straight-through flows).

optimal vehicle intervals for respective levels of combined critical flow. Otherwise, the resulting added delays may be significantly longer than the acceptable values indicated in these figures.

CONCLUSIONS

Generally, optimal maximum greens for the presence mode control are longer than the green durations required to achieve optimal pretimed control. Flow patterns with higher degrees of concentration of traffic in short periods of time need longer optimal maximum greens. The optimal maximum greens for hourly flow patterns with a peaking factor of 1.0 are about 10 sec longer than the corresponding optimal pretimed greens. With a peaking factor of 0.85, the optimal maximum greens are approximately 80 percent longer than the corresponding optimal pretimed greens.

Optimal vehicle intervals are a function of detector length and flow rate. For detectors 30 ft long, the use of 2-sec vehicle intervals can lead to the best signal performance over a wide range of operating conditions. For 50-ft detectors, 1-sec vehicle intervals are desirable under a variety of flow conditions. When detectors 80 ft or longer are used, 0-sec vehicle intervals can minimize delays. The use of vehicle intervals longer than 0 sec for such detector lengths is not desirable unless the combined critical flow at an intersection exceeds 1,400 vph.

Detectors 80 ft long can consistently produce the best signal performance. For a combined critical flow of less than 1,100 vph at an intersection, however, 65-ft detectors can produce comparable performance. For a combined critical flow of less than 900 vph, the use of 50-ft detectors in place of 80-ft detectors would only incur an added delay of up to 2 sec per vehicle. And, for a combined critical flow of less than 600 vph, 30-ft detectors may also be used to replace 80-ft detectors without incurring undue excess delays. For a specific signal control problem that involves the use of presence detectors, it is recommended that Figures 5, 8, 11, and 12 be consulted to determine an efficient combination of detector length, vehicle interval, and maximum green. For a signal phase with various directional flows, the critical lane flow of a representative peak-hour flow pattern may be used as a basis for determining such a combination.

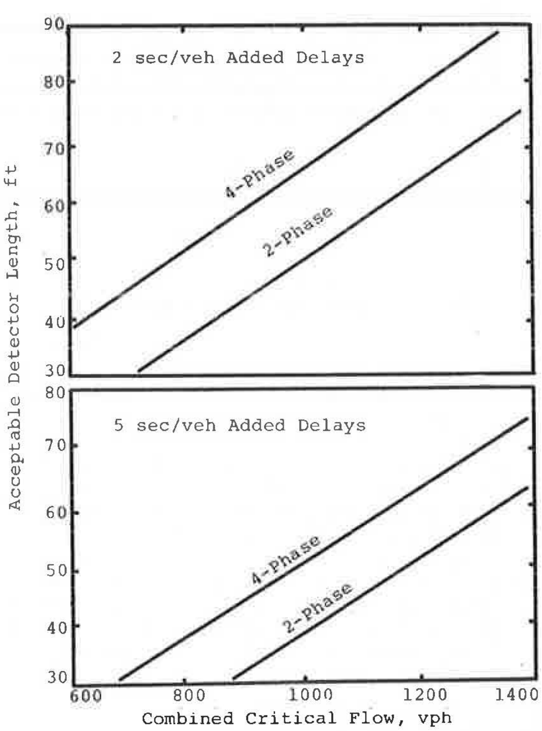

FIGURE 12 Acceptable detector lengths and allowable added delays (right-turn flows).

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Multiway Stop Sign Removal Procedures

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ABSTRACT

In recent years local jurisdictions have successfully converted unwarranted multiway stop-controlled intersections to less restrictive forms of control. However, there is wide variation in the approaches used and factors considered in the conversion decision. Therefore, FHWA initiated a national study of the processes, with two primary objectives: (a) to develop and test procedures to convert multiway stop-sign-controlled intersections to two-way stop-sign-controlled intersections, and (b) to document the safety effects of converting multiway stop controls to two-way controls. In this paper the study is summarized and the results are presented in the form of recommended conversion procedures. Thirty separate geographically distributed jurisdictions were visited and information and data regarding the various conversion experiences were collected. Data from more than 170 separate intersections were studied by the research team in arriving at the conclusions and recommended procedures in this paper. Laboratory driver preference studies were conducted to determine the most suitable warning and information signs. In addition to local government officials, several consultants as well as professionals in quasi-public agencies were interviewed and their experiences and knowledge of the conversion process were incorporated, where appropriate. The emphasis of the study has been on the safety aspects of the conversion process.

Within the past few decades there has been an increase in the use of multiway stop signs as the traffic control scheme at many intersections. Many elected officials believe that multiway stop signs are a panacea for intersection safety problems because they promote speed control, accident reduction, and pedestrian safety. Even though the Manual on Uniform Traffic Control Devices (MUTCD) (1) has warrants for the application of multiway stop control, in some cases the "political" warrant is the only one that is met. Multiway stop signs should ordinarily be used only where the intersecting road volumes are approximately equal. The MUTCD states that a stop sign should not be used for speed control.

Research has indicated that stop signs installed to control speed do not result in speed reduction (2-5). Also, studies have indicated that stop signs do not always result in increased safety (6).

Unwarranted stop signs increase stops, cause delays, and increase fuel consumption and pollutants. Further, installation of unwarranted traffic control devices breeds disrespect for such devices and can result in potentially dangerous behavior. For these reasons, it is desirable to remove unwarranted and unneeded stop signs that hinder traffic flow rather than aid it. Concern for the environment and for fuel conservation has led to a different attitude toward traffic control.

For several decades traffic engineering changes have, almost without exception, involved installing more positive or rigid control; for example, going from no control to two-way stop control or two-way to four-way stop control. Traffic engineers as well as the general public are conditioned to increasing degrees of control. Local jurisdictions are beginning to realize the mistakes of the past and understand that there are air pollution, delay, and