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Static Optimization Model for Congested Signalized Intersections

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CONTENTS

요 약

지난 수십년간 수많은 신호교차로제어모형이 제시되어 왔으나 과포화교차로를 특별히 다루는 기법은 거의 없었다. 본고에서는 과포화상태의 신호교차로제어를 위한 최적화모형이 제시된다. 지체도최소화나 연동폭최대화가 전통적인 제어목표로 사용되어 왔으나 혼잡교차로 제어에는 생산성최대화(maximum productivity)가 적절하며 본 모형에 사용되었다.

제시된 최적화모형은 Mixed Integer Linear Programming의 형태를 취한다. 본 모형은 두개의 교차로 문제에 적용되었으며, 모형화 과정에 사용된 가정들의 적절함이 민감도분석에 의해 증명되었다. 최적해의 검증을 위하여 microscopic simulation model인 TRAF-NETSIM을 사용하였다.

Ⅰ. INTRODUCTION

1. Background and Statement of Problem

Congestion on urban street networks has become a familiar occurrence in central business districts and other high-activity centers in many urban areas. Pignataro et al. (1) defined *congestion* as a condition in which all waiting vehicles cannot pass through the intersection in one signal cycle. They also defined *saturation* (or *oversaturation*) as the condition when vehicles are prevented from moving freely either because of the presence of vehicles in the intersection itself or because of queue back-ups in any of the exit links of the intersection.

Oversaturation occurs when traffic demand exceeds intersection capacity. Queues fill entire blocks and interfere with the performance of adjacent upstream intersections when this heavy demand continues for a long time period. Sometimes cross-street traffic is blocked by extended queues. Queue spillback to upstream intersections is common in heavily loaded urban networks, especially when intersections are closely spaced. In oversaturated conditions, congestion is unavoidable; thus the control policy should aim to postpone the onset and/or the severity of secondary congestion caused by intersection blockage that is not the originator of the congestion.

Several theories have been developed and applied successfully for the control of undersaturated intersections, but most of them seem ineffective or invalid when traffic volumes become excessively high. There has been relatively limited research for oversaturated conditions, and most of the research performed has been too theoretical for real situations. Moreover, none of this research has treated two-way arterial systems with multi-phase signal operation, even though there is a need for optimal control strategies in such an oversaturated arterial system. A number of practical computer programs like PASSER II (2) and TRANSYT-7F (3) can develop optimal signal control strategies for undersaturated arterials. None of these are applicable for oversaturated environments.

2. Research Objectives

The goal of this research is to develop an optimization model to provide optimal traffic signal control for oversaturated complex urban arterials. This model is designed to accommodate two-way arterials whose intersections have multi-phase operation. Generally, the control objective of traffic signal timing has been to obtain maximum bandwidth and/or minimum delay. The control objective for oversaturated environments, however, is to maximize throughput in the system during the control period, i.e., the *productivity*. When demands are extremely high, the control policy dictates that queue lengths at all internal links of the roadway system do not exceed queue storage capacity. Simultaneously, all available green times are utilized as fully as possible to obtain maximum system productivity. The maximum productivity has been used as the control objective for the control of freeway on-ramps (4, 5).

The specific objective of this research is to develop a static optimization model for traffic signal control that maximizes productivity for a system of two oversaturated intersections and perform a sensitivity analysis to investigate the performance of this model

3. Scope of Research

The traffic signal timing products developed in this research have the following scope of development :

- The optimization models for traffic signal control were designed primarily to address congested traffic environments;
- The models were developed for pretimed traffic signal control based on historic data that varies by sequential time periods;
- The stochastic nature of traffic demand within these time periods was not explicitly considered in the formulation of the models;
- The roadway system under study was a two-way arterial experiencing congestion. Exclusive left-turn phases must be provided for the left-turn movements at the arterial approaches. Cross streets can be either one-or two-way;
- Since real traffic data were not available at the time of this study, artificial traffic data representing congested conditions were generated and used to test the models; and
- TRAF-NETSIM (6), a microscopic simulation model, was used to evaluate the control strategies.

Ⅱ. REVIEW OF LITERATURE

Since Webster (7, 8) initiated studies on traffic signal timing in the late 1950s, research on traffic signal control has been performed on various problems. Most research focused on undersaturated traffic conditions. Only a few studies have addressed the area of traffic control for oversaturated environments. This chapter reviews the previous research on traffic control during congested traffic conditions. The studies can be categorized into two classes: theoretical approaches and practical guidelines.

1. Theoretical Approaches

Several researchers have attempted theoretical approaches to develop a control policy for oversaturated environments over the last thirty years. For pretimed signals, the problem of signal control during the peak hour was first considered by Gazis and Potts (9), who derived optimality conditions for an oversaturated one-way no-turn intersection. In another paper, Gazis (10) extended the control policy to two oversaturated linked intersections with one-way operation. Singh and Tamura (11) formulated a dynamic optimization problem for oversaturated networks as a linear quadratic problem based on Gazis's theories.

Michalopoulos (12, 13, 14) proposed an optimal control policy for both pretimed and real time control. His control policy was to minimize total system delay subject to queue length constraints. time control of isolated intersections. Longley and Gordon's control philosophy was based upon the fact that traffic signals cannot clear queues in conditions of primary congestion. Their signal control objective thus was to maintain the growth of the queues in a predetermined ratio in order to postpone the onset of secondary congestion.

Pignataro, et al. (1) suggested queue actuated control as a highly responsive signal control strategy. This strategy is a control policy that automatically provides an approach with a green indication when the queues on that approach become equal to, or greater than, some predetermined length. While the above studies offer substantial potential for relieving congestion, none of them has been extended to solve the two-way, two-intersection problem.

2. Practical Guidelines

Practical guidelines have been published to assist traffic engineers in understanding the cause and severity of traffic congestion and to provide control strategies associated with the congestion types. pignataro, et al. (1) presented guidelines for the treatment of traffic congestion on street networks. The guidelines provided both a tutorial and an illustrated reference on what techniques to consider and how to systematically consider them. OECD (17) provided policy-makers and traffic engineers with an up-to-date assessment of traffic congestion management. ITE (18) published the proceedings of its 1987 national conference dealing with traffic congestion.

Shibata and Yamamoto(19) suggested on-line real-time control for isolated intersections with multi-phase operation. Rathi (20) and Lieberman (21) proposed queue management contorl, a form of internal metering, designed to manage queue length to reduce the probability of spillback. They showed that backward progression was optimal or near optimal for a street with long queues and slow discharge headways.

Ⅲ. STATIC OPTIMIZATION MODEL

1. Introduction

Conventional traffic engineering models provide optimal signal timing to obtain maximum bandwidth and/or minimum delay at signalized intersections. In congested roadway system, however, these are not the desirable control objectives; instead, signal control should produce maximum system productivity.

System productivity is defined as the total number of vehicles discharged in the roadway system under consideration during the control period. In other words, as many vehicles as possible should be serviced through the specified roadway system during a given time period. Wattleworth and Berry (4) theoretically proved the equivalency of maximizing system output rate and minimizing travel time in dynamic freeway on-ramp control. This proof can be applied to the traffic signal control of surface streets.

This chapter describes the development of an optimization model for signal timing in a system of two oversaturated intersections to maximize the system productivity. The timing with respect to the system productivity based on one set of peak traffic demand, but not considering time-varying demand. The following control objectives should be accomplished so as to achieve maximum system productivity:

- Full utilization of green indication time;

– Maximization of output during the green indication time;
– Full utilization of queue storage capacity of internal links;
– Stabilization of queue lengths; and
– Prevention of queue spillback.

The optimization model uses mixed integer linear programming(MILP) to mathematically model the above requirements. MILP has been successfully used in formulating several signal timing optimization models (2, 22, 23).

2. Formulation

The formulation of the static model is initially described for a "unit problem" consisting of a single arterial and its two intersections, shown in Figure 1. One–way cross streets were used for simplicity. The model for the unit problem can be modified and extended for actual roadway systems having more than two intersections and/or two–way cross streets without any difficulty. Here, approaches 1, 2, 3, and 4 are regarded as external approaches; movements 6, 7, 16, and 37 as internal movements.

At the intersection of a two–way arterial and a one–way cross street, there are usually theree non–conflicting phases, as shown in Figure 2 (24). Phase A is dedicated to through and right–turning traffic on the arterial. Phase B provides exclusive right–of–way to all cross street movements. Phase C is necessary to clear traffic between intersections, particularly the outbound left–turn movement. Figure 2 illustrates a leading left turn sequence at both intersections. The following notations were used in the static model formulation:

C = system cycle length, sec,
G_i = effective green time for movement i, sec,
S_i = saturation flow for movement i, veh/sec,
V_i = average arrival rate for movement i. veh/sec,
N_i = number of vehicles moving during green time for movement i, veh/cycle, which is minimum $\{S_iG_i, V_iC\}$,
Z_i = 0 for oversaturation, that is, $N_i=S_iG$, 1 for undersaturation, that is, $N_i=V_iC$,
$\underline{M}$ = very large positive value, called *Big–M*
$\underline{l}$ = lost time for individual phase, sec,
Q_i = queue storage for internal movement i, veh,
P_{ij} = proportion of turning movement shown in Figure 3,
T_i = queue growing speed at external movement i, veh/sec.

1) Objective Fuction

Congestion on the internal links often adversely affects the performance of upstream intersections. In a system of short internal links, the queues generated at congested internal approaches sometimes extend into and consequently block upstream intersections. Under this condition, the vehicle discharge rate at the upstream intersection (or input rate to the system)becomes less than ideal. By eliminating this congestion, the upstream signal is able to serve more vehicles. If all external approaches are full of vehicles waiting for service and the internal links are not congested, input and output of the roadway system would be balanced. Under such a situation, engineers can increase system output (or system productivity) by increasing system input. The concept is similar to that of freeway on-ramp control (4).

The static model provides several important traffic control features. The objective function of the static model is to maximize the input flow to the roadway system. Operational constraints for preventing queue backup are also incorporated into the MILP formulation. If an external approach is fully saturated, the number of vehicles discharged during one cycle is proportional to the phase duration associated with the external approach. The traffic signal cycle at each intersection consists of two external phases (A and B) and one internal phase (C), shown in Figure 2. Increasing the durations of phases A and B can increase the number of vehicles entering the roadway system. Consequently, maximum productivity can be obtained by maximizing phase durations for the external approaches and minimizing phase durations for the internal approaches, subject to the constraints noted below. The objective function of the static model is:

$$\text{Maximize} \quad P = G_1+G_2+G_3+G_4-G_6-G_7 \quad (1)$$

2) Constraints

System productivity can be increased by increasing the phase durations for external movements; however, there are upper limits on the external phase durations. The upper limits can be formulated using proper constraints. The phase duration can be interpreted as effective green time in the following discussions. The constraint sets satisfying the five objectives listed in the introduction of this section are described in the following section.

Set 1. In a coordinated signal system, the sum of phase durations at each of the individual intersections shown in Figure 1 must be equal to the system cycle length.

$$G_1+G_2+G_6+3l = C \quad (2a)$$

$$G_3+G_4+G_7+3l = C \quad (2b)$$

Set 2. For internal links, the input to the system must be less than or equal to the output in order to obtain the stability of queue lengths over many cycles(Objective 4). Thus, for the four internal links,

$$P_{17}N_1+P_{27}N_2 \leq \alpha S_7 G_7 \quad (3a)$$

$$P_{13}N_1+P_{23}N_2 \leq \alpha S_{37}(G_3+G_7) \quad (3b)$$

$$P_{36}N_3+P_{46}N_4 \leq \alpha S_6 G_6 \quad (3c)$$

$$P_{31}N_3+P_{41}N_4 \leq \alpha S_{16}(G_1+G_6) \quad (3d)$$

where α is an adjustment factor for green split, usually not greater than 1.

According to the constraint set above, oversaturation would never occur at the internal approaches. Due to the stochastic nature of vehicle arrival and discharge headways, however, the deterministic balance of the input and output might not be valid, resulting in unexpected oversaturation on the internal approaches.

It is desirable to provide additional green indication times for the internal phases to reduce the possibility of unexpected oversaturation. The smaller the adjustment factor α, the larger the internal phase duration. A value of one was used for the factor α as default value in the following discussion, unless otherwise noted.

The left-hand side of the Constraint Set 2 represents the number of vehicles entering the internal links during one cycle. The right-hand side represents the capacity of individual movements on the internal links. With this constraint set, demand on the internal approaches never exceeds capacity. This constraint set determines the necessary proportions of all phase durations, ensuring stable queue lengths on the internal links over the cycle. Theoretically, no vehicles stay at internal links for more that two cycles. As small phase durations as possible should be provided for the internal phases through the objective function. The small phase durations force platoons to compress when they discharge at the internal phases. That is, vehicles are discharged at compressed headway, fully utilizing phase durations (Objective 1) and maximizing output during the phase duration (Objective 2).

Set 3. The maximum number of vehicles stored on internal link i must be less than queue storage, Q_i.

$$P_{17}N_1 + P_{27}N_2 \leq \beta Q_7 \tag{4a}$$

$$P_{13}N_1 + P_{23}N_2 \leq \beta Q_{37} \tag{4b}$$

$$P_{36}N_3 + P_{46}N_4 \leq \beta Q_6 \tag{4c}$$

$$P_{31}N_3 + P_{41}N_4 \leq \beta Q_{16} \tag{4d}$$

where β is an adjustment factor for queue storage, usually not greater than 1.

The left-hand side of the Constraint Set 3 is identical to that of Constraint Set 2, which is the number of vehicles entering internal links during one cycle. The maximum queue lengths might be affected by the quality of traffic progression between intersections. The queue lengths expressed in these constraints are formulated in a conservative manner. Assuming that every vehicle entering the internal link per cycle stops, the maximum queue length is identical to the number of vehicles entering the internal link.

It should be noted that Constraint Set 3 determines an optimal system cycle length. Constraint Set 2 plays a role in stabilizing queue lengths over time by adjusting green split; yet, it cannot control actual queue lengths. These queue lengths can be controlled by adjusting cycle lengths. A large cycle length gives a more effective green indication time to the intersection than does a small cycle length; however, the former increases the possibility of queue spillback into the upstream intersection. The relationship between system cycle length and system productivity has the form of a concave function, which will be demonstrated in the next chapter. This constraint set provides the optimal cycle length, which fully utilizes queue storage (Objective 3) and prevents queue spillback (Objective 5).

The queue storage capacity of internal link i, Q_i, is the maximum queue length that traffic engineers want to maintain over the cycle. This storage capacity is calculated as follows:

$$Q_i = \frac{\text{(link length, feet)}}{\text{(average vehicle storage length, feet)}} \times \text{(number of lanes)}$$

According to the above constraints, queued vehicles never spillback to the upstream intersection. Yet, due to the stochastic nature of vehicle arrivals and lane utilization, actual queue lengths fluctuate around the average value, which might cause queue spillback. In determining the queue storage, Q_i, a storage buffer should be provided to absorb such natural fluctuations; the adjustment factor β is used for this purpose. The smaller the adjustment factor β, the smaller the optimal cycle length. A value of one was used in the following discussion for convenience, unless otherwise noted.

Set 4. The number of vehicles entering during green indication time for movement "i" is expressed as

$$N_i = \text{Minimum}\ \{S_iG_i,\ V_iC\} \qquad (5)$$

Mathematical expressions for the number of the vehicles discharged (N_i) during the green time (G_i) depend on whether the corresponding approach is oversaturated or not. If the approach is oversaturated, then N_i is equal to the product of saturation flow (S_i) and phase duration (G_i). That is, the productivity during the phase is proportional to the phase duration; thus, increasing the external phase duration as much as possible for a given cycle length results in maximum productivity. If the approach is undersaturated, then N_i becomes the product of demand (vehicles per second) and cycle length (seconds). The productivity does not depend on the phase duration, but on the approach demand and the cycle length. It should be noted that whether an approach is oversaturated or undersaturated cannot be predetermined. The reason is that the oversaturation of the approach depends on how much green time is assigned to the approach. The static model automatically determines the state of saturation during the optimization procedure.

Equation 5 itself cannot be solved directly using linear programming. This equation must be transformed into the following equivalent linear form :

$N_i \leq S_iG_i$

$N_i \leq V_iC$

$S_iG_i - N_i \leq \underline{M}Z_i$

$V_iC - N_i \leq \underline{M}(1 - Z_i)$

where Z_i is an integer variable having binary values. For $Z_i = 0$ (oversaturated condition), N_i is equal to S_iG_i; otherwise, N_i becomes V_iC. Unfortunately, the static model now becomes a complex Mixed Integer Linear Programming (MILP) problem because of the integer variable, Z_i, in the above formulation.

Set 5. Maximum cycle length constraint.

$$C \leq C_{max} \qquad (6)$$

For long internal links, the static model produces long cycle lengths as the optimum solution. The cycle lengths should be constrained by a practical upper limit considering queue storage requirements.

Set 6. Minimum green constraints.

$$G_i \leq G_{i\ min} \qquad (7)$$

where $G_{i\ min}$ is a minimum green indication time for phase i. The minimum green time can be determined from pedestrian crossing requirements or driver expectancy considerations.

3. Green Split for External Phases

One special feature of the static model is that phase durations for external movements are adjustable by adding optional constraints. The sum of phase durations of the external phases can be expressed as follows:

$$G_1+G_2+G_3+G_4+4\underline{l}=2C-(G_6+G_7+2\underline{l}) \quad (8)$$

According to Equation 8, a two-intersection system can be analyzed as an isolated intersection with four-phase operation. as shown in Figure 4. Cycle length (C) and internal phases (G_6 and G_7) are calculated automatically in the MILP formulation after weighting factors are applied to the external phases (G_1, G_2, G_3, and G_4). The generalized form of the optional constraint set of weighting factors is expressed as follows:

$$\frac{G_1}{W_1}=\frac{G_2}{W_2}=\frac{G_3}{W_3}=\frac{G_4}{W_4} \quad (9)$$

where W_i is the selected weighting factor of approach i.

The weighting factor, W_i, should be selected with care, based on geometric and traffic conditions. The larger the W_i factor for approach i, the bigger is G_i for the approach. Examples are presented in the following sections.

1) Green Split Based on Demand

A conventional method of calculating green splits is to allocate green times according to flow ratios. That is,

$$\frac{G_1}{Y_1}=\frac{G_2}{Y_2}=\frac{G_3}{Y_3}=\frac{G_4}{Y_4} \quad (10)$$

where Y_i is the V_i/S_i external movement i.

This scheme is desirable when the arterial system is not saturated and/or when the queue lengths for external movements are not critical. The scheme also gives the least overall delay to external approaches (7).

2) Green Split Based on External Queue Lengths

When an arterial system is oversaturated and the queue-storage capacities for external approaches are insufficient, engineers may want to control external queue lengths so that they do not hurt the performance of the total system. Under this condition. the green split based on demand only is not appropriate for queue management: Longley's queue control policy (15) appears more desirable. When the intersections are oversaturated, the queue lengths continue to grow as long as demand volumes exceed intersection capacity. The control strategy should aim to postpone queue spillback to adjacent intersections as long as possible and hence reduce its severity.

The queue-growing speed per cycle (T_i) for external approach i is defined as the amount of demand exceeding capacity per cycle, which is expressed as follows:

$$T_i = V_iC - G_iS_i \quad (11)$$

The green split should be adjusted so that the four competing queues simultaneously fill up the queue storage of their associated links. This green split can prevent a queue on the shortest link from reaching its maximum earlier than the others; The fill-up ratio for external link i is expressed as T_i/L_i Thus, the optional constraint for the green split based on the queue lengths is expressed as follows:

$$\frac{T_1}{L_1}=\frac{T_2}{L_2}=\frac{T_3}{L_3}=\frac{T_4}{L_4} \quad (11)$$

where L_i is the queue storage capacity of external approach i. This constraint ensures that the queues for all approaches simultaneously reach their allowable maximums as long as demands are constant.

This scheme is desirable for oversaturated systems with limited queue–storage capacity.

4. Offset

The static model produces an optimal cycle length and green splits, but not offsets. An important characteristic of the static model is that its optimal solution for maximum productivity is not sensitive to offset between intersections. To fully utilize the capacity during an internal phase, the optimal solution produced by this model is designed so that the vehicles entering from external link are forced to stop at the internal links. They are then released at saturation headways during the next cycle, which is accomplished by assigning the minimum green time required for undersaturation to internal clearance phases. In the static model, the adjustment of the internal offset may reduce average delay at the internal links, but not significantly increase the system productivity. The appropriateness of this assumption is demonstrated in the next chapter.

Ⅳ. SENSITIVITY ANALYSIS

1. Introduction

An efficient method of evaluating mathematical models is to examine the sensitivity of the model predictions to small changes in major variables. Elements of traffic signal control are cycle length, green split, offset, and phase sequence. In developing the static model described in the previous chapter, theree hypotheses about the traffic signal control elements were involved as follows:

- There exists an optimum cycle length which maximizes system productivity. System productivity would be lost due to lost time for cycle lengths less than the optimum, and due to queue spillback into upstream intersections for cycle lengths larger than the optimum. Refer to Figure 5 (a).
- There exists an optimum green split which maximizes system productivity. System productivity would be lost due to increasing queue lengths on internal links for Phase–C duration (left–turn phase) less than the optimum, and due to unused green time for a duration greater than the optimum. Refer to Figure 5(b).
- Offset is not very sensitive to system productivity at optimum cycle length and green split.

The purpose of this sensitivity analysis is to demonstrate the appropriateness of the above hypotheses. This research investigated the relationships between the major signal timing elements and system productivity through sensitivity analysis. It also studied the effect of the timing elements on system delay.

2. Study Approach

1) Experimental Design

The test procedure followed four straightforward steps:

Step 1. Experimental plan,

Step 2. Optimization using the static model, Step 3. Simulation using TRAF-NETSIM, and

Step 4. Analysis of results.

The input data for the base case were generated artificially, The input data represented oversaturated traffic conditions for an arterial with two signalized intersections. To investigate the effects of the input data on the optimal solution, the base case was modified as follows:

- Lengths of internal links (200, 300, and 500 feet),
- Cycle lengths (5-second increments from 40 to 110 seconds)
- Turning percentages (three cases of origin-destination patterns), and
- Offsets (5-second increments from 0 up to the cycle length).

The static model was used to obtain the optimal signal timings for the above cases. Green splits were calculated based on approach demands in all the cases. These signal timing plans were then simulated by TRAF-NETSIM. Signal timings deviating from the optimal timings were also simulated, and their performances were compared to those of the optimal timing. Signal timing optimality will be demonstrated by comparing the performance between the optimal timing and the other timings deviating from the optimal timing.

TRAF-NETSIM was used to evaluate the signal timing plans. Due to its inherent variability, a simulation trial for each signal timing plan was replicated four times, using different random number seeds. A 15-minute simulation time was used for every simulation trial.

2) Description of Base Case

As illustrated in Figure 6, the roadway for the base case is an arterial with two lanes in each direction. Two intersections are spaced 300 feet apart. Left-turn traffic on the arterial has a left-turn bay with an exclusive phase. Cross streets are one-way facilities with two moving lanes. Traffic volumes and turning percentages are shown in Figure 7. Assuming a vehicle discharge headway of 2 seconds per vehicle, the intersections are oversaturated having the traffic volumes and patterns depicted in Figure 7. The zero offset was used for the base case signal timing.

3) Measures of Effectiveness

TRAF-NETSIM provides various measures of effectiveness(MOE's) for traffic operations produced by given signal control strategies. In this research the objective of traffic signal control was to maximize system productivity during congested periods. The total number of *vehicles discharged* during the simulation period appeared to be the most appropriate of all MOE's because the number of vehicles discharged indicated the system productivity for a given signal timing. The number of vehicles discharged is seldom used in a normal traffic study while *average delay* is widelay used in traffic engineering.

The average delay was also investigated to test the performance of signal timing. TRAF-NETSIM gives four MOE's in seconds per vehicle: total time, delay time, queue time and stop time. The *queue time* is comparable to the average queue delay of other deterministic models; so the average delay used in this dissertation is the queue time in TRAF-NETSIM output.

3. Results

1) Cycle Length

According to pignataro et al (1), one of the most prevalent and erroneous beliefs in the traffic engineering community is that the capacity of an intersection increases substantially as the cycle length increases. His concern was that cycle lengths should be determined from lengths of feeding links in order to avoid excessively long queues. Intersection capacity, the sum of critical lane volumes (ΣV), is a function of cycle length. The formula that expresses this relationship is:

$$\Sigma V = \frac{3600}{h}\left(1 - \frac{\Sigma l}{C}\right) \tag{13}$$

where C is the cycle length in seconds, Σl is the total lost time in seconds, and h is the saturation headway in seconds per vehicle. For h = 2.0 sec, as C approaches infinity, ΣV converges to 1,800 vph per lane. When traffic demand is near or over this value, increasing the cycle length has little effect on an oversaturation problem. Unnecessarily long cycle lengths tend to create excessive queue lengths, which often cause serious operational problems at upstream intersections as a result of queue spillback.

A plot of cycle length versus vehicles discharged over a range of link lengths is presented in Figure 8. The data points in this figure were obtained from the TRAF-NETSIM simulation. The ideal case was simulated using TRAF-NETSIM to show ideal system productivity. The ideal case provides an infinite length for the internal links on an arterial. On an arterial with limited intersection spacing, the vehicles discharging at the upstream intersection are often impeded by vehicles stalled in the receiving link. In the ideal case, vehicles can be discharged freely at the stop line without being impeded by the stalled vehicles. As shown in Figure 8, vehicle discharge increases continuously as cycle length increases. Long cycle lengths reduce loss of system productivity due to lost time. As the cycle length is longer, the curve for the ideal case becomes flatter.

Arterials having three different link lengths were also simulated to show actual system productivity (Figure 8). A plot for a link length of 300 feet shows a typical concave curve. Vehicle discharge increases continuously up to an optimal cycle length (around 75 sec) and decreases beyond this point. This trend was an expected result, as depicted in Figure 5(a). System productivity is lost due to lost time for cycle lengths less than the optimum and due to queue spillback to the upstream intersection for cycle lengths larger than the optimum. A plot for a 200-foot link shows a shape similar to the 300-foot link. Maximum productivity occurs at a 50-second cycle. For the 500-foot link case, vehicle discharge increases continuously to a 75-second cycle and then flattens for cycle lengths between 75 and 100 seconds. The curve slightly decreases beyond the 100-second cycle.

Figure 9 shows the relationship between

cycle length and total arterial delay produced from the TRAF−NETSIM simulation. The curves in this figure have the form of convex functions. The minimum delay cycle lengths are 55 seconds for the 200−foot link and 90 seconds for the 300−foot link, respectively. For the 500−foot link, the curve is relatively flat for cycle lengths between 90 and 100 seconds. Delay increases beyond the 100−second cycle. Figures 8 and 9 demonstrate that optimal cycle length is related to link length. The optimum cycle length increases as link lengths become longer.

The static model, as a deterministic model, optimizes signal timing based on average queue lengths over many cycles. In the real situation, queue lengths fluctuate around the average values due to their stochastic nature. Even at the optimal cycle length, there is a chance of queue spillback. The side effect of queue spillback in potentially serious. In selecting a system cycle length, it is safer to choose the cycle length slightly less than the optimal in order to reduce the queue−spillback probability. The static model produced optimal cycle lengths of 54, 76, and 105 seconds for links lengths of 200, 300 and 500 feet, respectively, when average vehicle storage lengths were assumed to be 25 feet.

2) Green Split

This research studied the effect of green split on system productivity to demonstrate the appropriateness of the hypothesis with respect to green split in developing the static model (Hypothesis 2). Whether the optimal green split produced by the static model was really optimal was also tested through TRAF−NETSIM simulation. Three cases of turning percentages were prepared, as shown in Figure 10. Case 1 has turning percentages identical to the base case. In Case 2, traffic entering from the cross streets onto the arterial in double that of Case 1. In Case 3, the right intersection has turning percentages identical to Case 1 and the left intersection is identical to Case 2. Cases 1 and 2 have a symmetric traffic pattern, while Case 3 has an a symmetric pattern.

Figures 11, 12, and 13 show the effect of green split on system productivity and total arterial delay. In Figure 11, the green time for internal clearance phase (phase C) was increased by 2 seconds starting from 5 seconds, and green times for external phases (phases A and B) were simultaneously reduced by 1 second, while keeping the cycle length constant. The green time did not include intersection clearance time. A curve showing the relationship between internal phase duration and system productivity became a concave function. The vehicle discharge increases to the internal phase duration of 9 seconds, then decreases continuously for durations above 9 seconds. This result agrees with the hypothesis made in the static model for green splits in Figure 5 (b). System productivity was lost due to increasing queue length on the internal link for the internal phase durations less than the optimum and unused green time for the internal phase durations greater than the optimum. A curve for total arterial delay is noted to be convex, as expected.

The relationship between the internal phase duration, vehicle discharge, and total arterial delay for Cases 2 and 3 (Figures 12 and 13) show a pattern similar to Case 1. The vehicle discharge functions are concave while the total arterial delay functions are convex. The three cases, however, give different optimal internal phase durations. This result means turning percentage is a major factor in determining green split. When the signal timings for the three cases were developed using the static model, it produced optimal internal phase durations of 9, 11, and 10 seconds for Cases 1, 2, and 3, respectively. These optimal durations are slightly larger that those predicted by the sensitivity analysis using the TRAF−NETSIM simulation.

3) Offset

According to current practice in signal timing design, the best offsets are selected based on maximum bandwidth (PASSER II (2), MAXBAND (22)) or minimum delay and stops (TRANSYT−7F (3)). While a number of studies have been conducted to determine the effects of offset on arterial progression, average delay, and stops, none has considered system productivity. The objective of this section is to examine the effect of offset on system productivity and average delay.

Given the roadway and traffic characteristics of the base case (Figure 7), optimal cycle lengths and green splits were generated by the static model. It should be noted that the offset reference points are the starting point of phase A of both intersections, as demonstrated in Figure 14. Two cycle lengths were simulated using TRAF−NETSIM. One was the optimal cycle length (75 seconds), and the other was a cycle length below optimal (60 seconds). Green splits were fixed for the two cycle lengths.

Figure 15 shows the effect of offset on the average delay for the internal links. For both cycle lengths, minimum internal delay is observed at the offset value of half the cycle lengths (alternate offset), and maximum delay is observed around the zero offset (simultaneous offset). The percent differences between minimum and maximum delay are approximately 100 percent for the 60−second cycle and 75 percent for the 75−second cycle. From these results,, it is concluded that average delay on the internal links is sensitive to offset.

Figure 16 shows the effect of offset on total vehicle discharge for the arterial system. The curves in these figures have a different trend to that of offset versus internal delay (Figure 15). First of all, the curves are almost flat for the entire range of offset. Maximum vehicle discharge was obtained near simultaneous offset (zero offset) for both cycle lengths. Percent differences between the minimum and maximum are only 1 percent for the 60−second cycle and 4 percent for the 75−second cycle. These differences appear relatively trivial compared to the internal delay. From this result, it is concluded that system productivity is not very sensitive to offset. Figure 17 shows the effect of offset on total interchange delay at 60−and 75−second cycle lengths. The curve of total delay has a trend similar to that of system vehicle discharge. Total delay also is not as sensitive as internal delay over the range of the entire cycle.

V. CONCLUSION

In this paper, static optimization model was presented for traffic signal control of oversaturated signalized intersections. Sensitivity analysis was also performed to demonstrate the appropriateness of the hypotheses used for developing the optimization model. Conclusions drawn from this study are described as follows:

- The control objective of signal timing for oversaturated intersections should be maximum productivity instead of conventional objective of minimum delay and/or maximum bandwidth.
- In order to achieve maximum productivity, the following objectives should be accomplished: full utilization of green indication time, maximization of output during the green indication time, full utilization of queue storage capacity of internal links stabilization of queue lengths, and prevention of queue spillback.
- From the sensitivity analysis, it was found that there exist optimum cycle length and green split to maximize system productivity.
- TRAF−NETSIM was used as a tool for the sensitivity analysis. The capability of simulating queue spillback and intersection blockage was critical in evaluating control strategies for oversaturated conditions. It was found that TRAF−NETSIM was able to simulate these phenomena.

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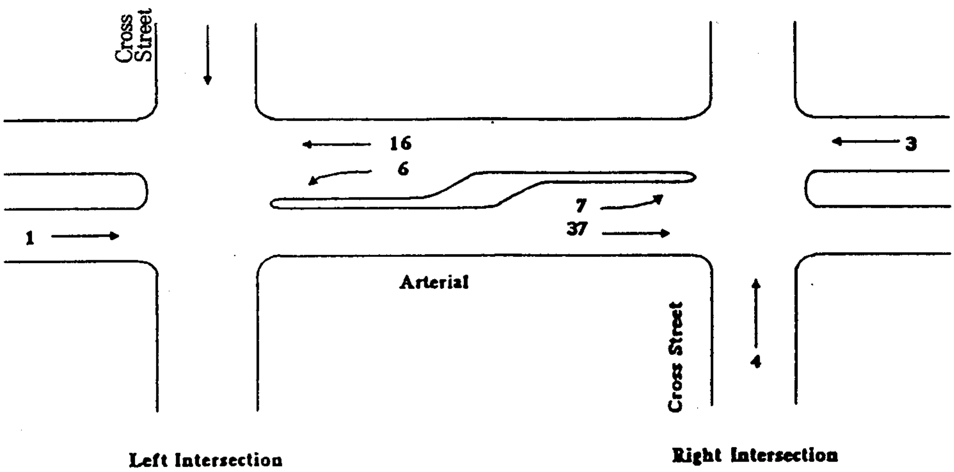
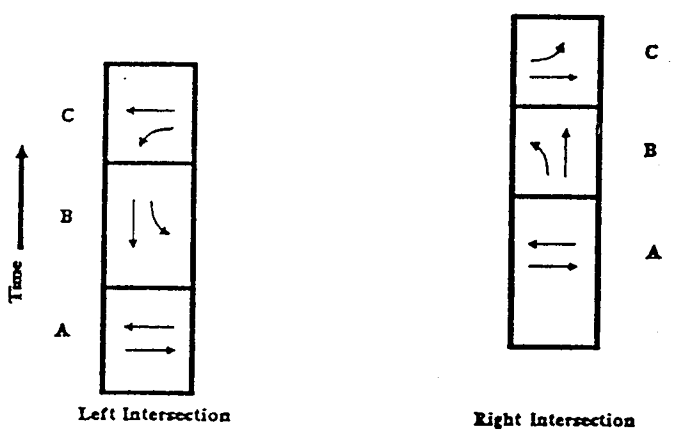
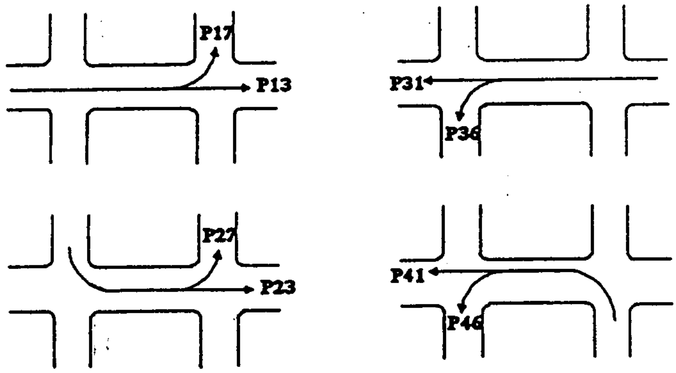
FIGURES



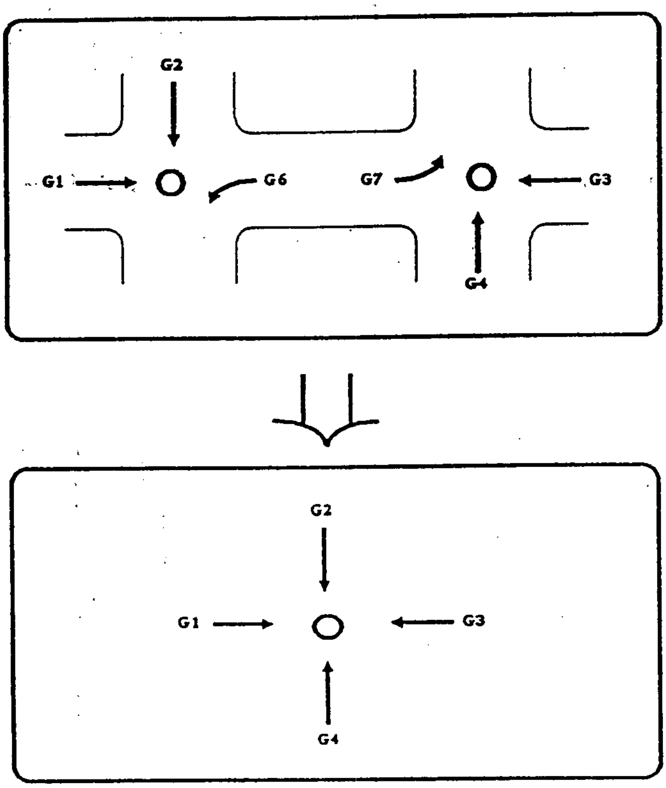
〈FIGURE 1〉 Arterial with Two Intersections.



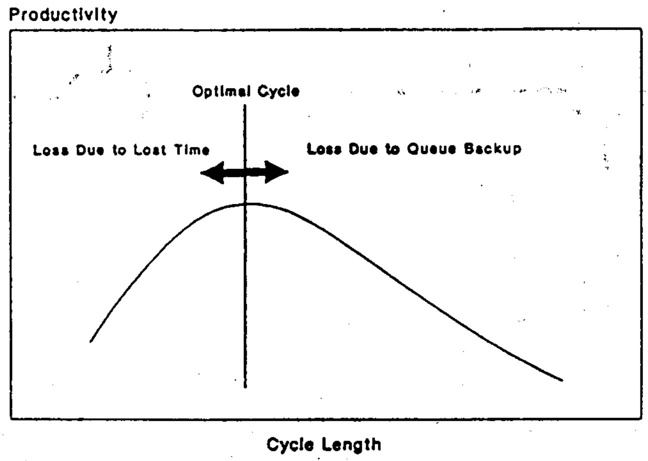
〈FIGURE 2〉 Three Basic Phases and Phase Sequences.



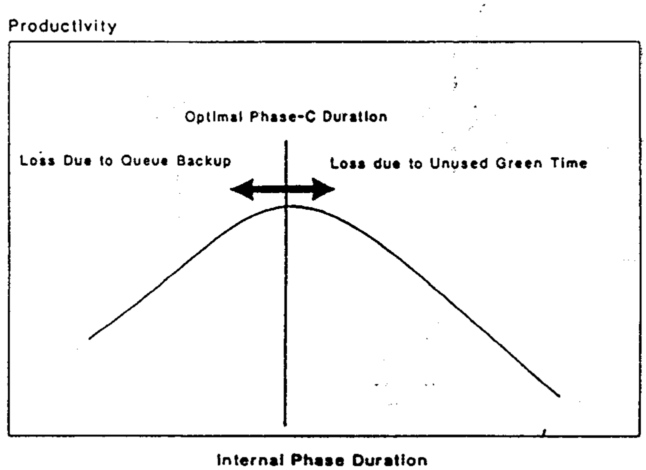
⟨FIGURE 3⟩ Notation for Turning Percentages.



⟨FIGURE 4⟩ Conversion of a Two–Intersection System into a Single Intersection.

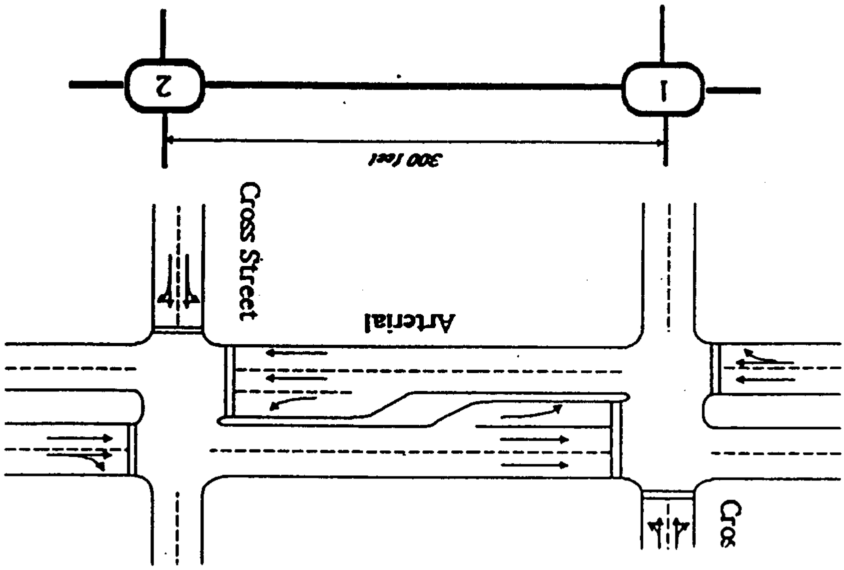


(a) Cycle Length versus Productivity

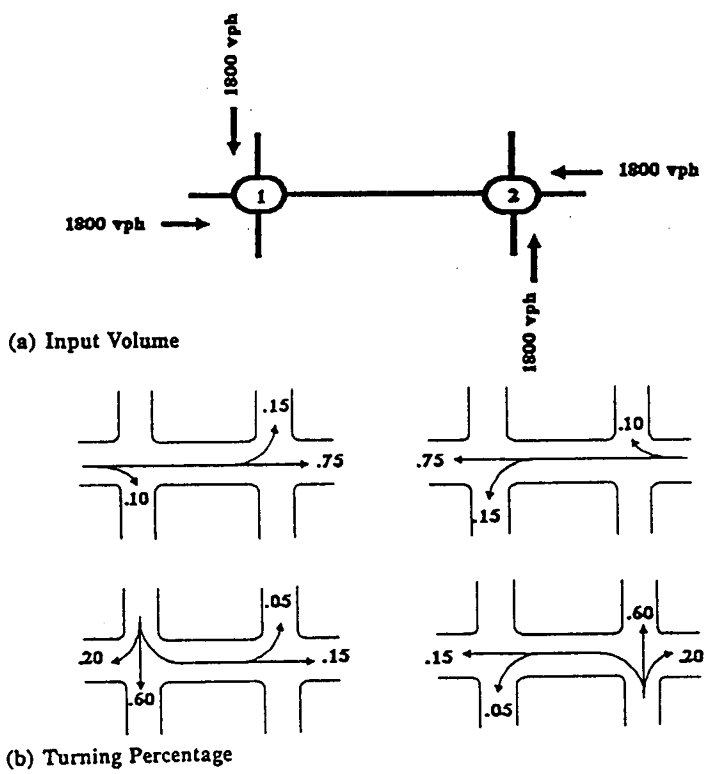


(b) Internal Phase Duration versus Productivity

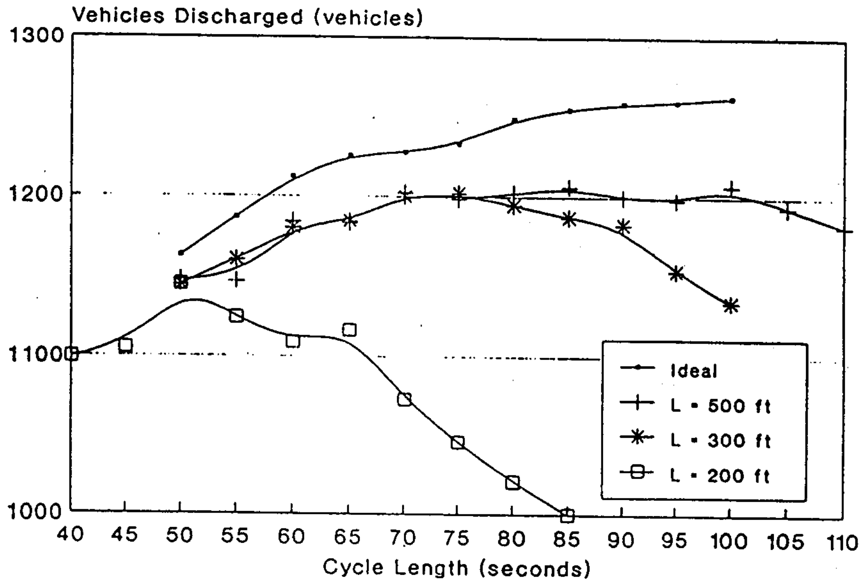
⟨FIGURE 5⟩ Relationships between Signal Timing Variables and Productivity.



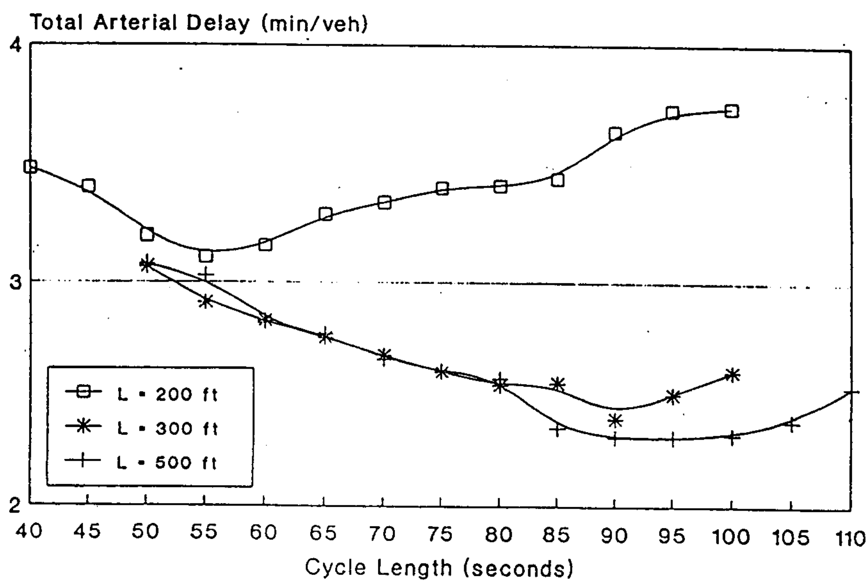
〈FIGURE 6〉 Roadway System for Base Case.



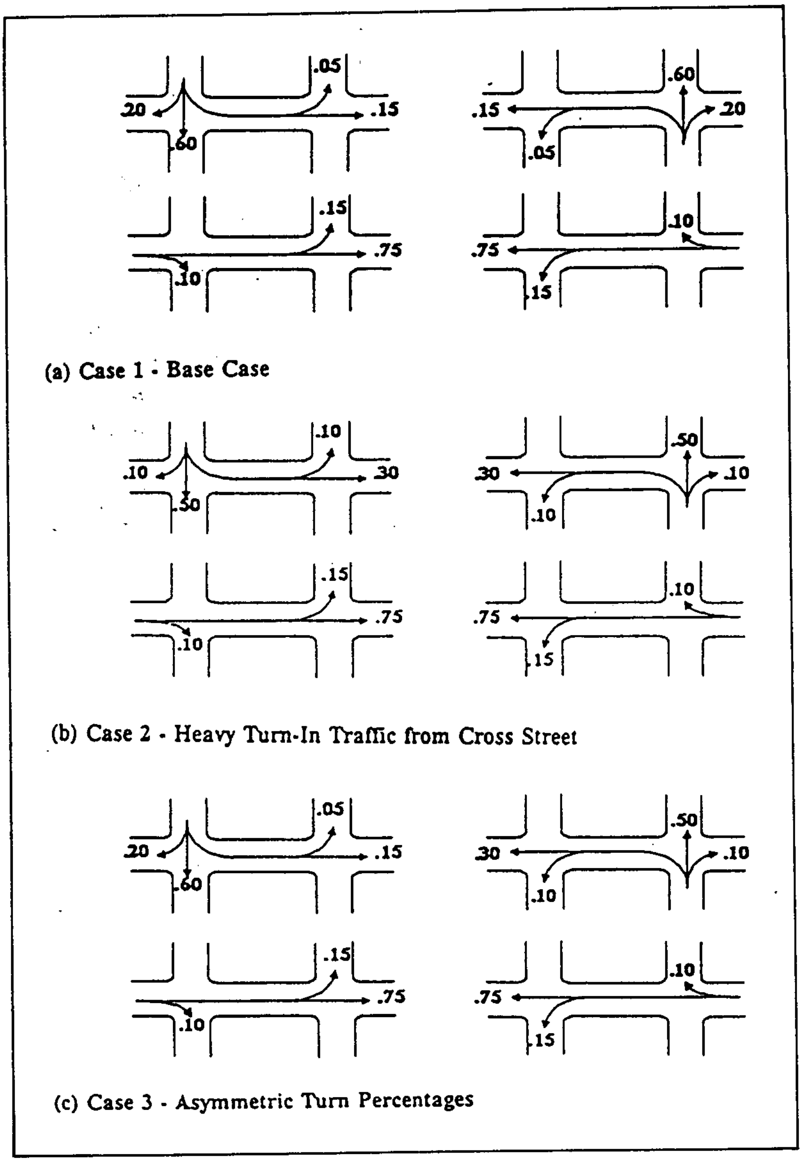
〈FIGURE 7〉 Traffic Characteristics for Base Case.



〈FIGURE 8〉 Plot of Cycle Length versus Vehicles Discharged.



〈FIGURE 9〉 Plot of Cycle Length versus Total Arterial Delay.

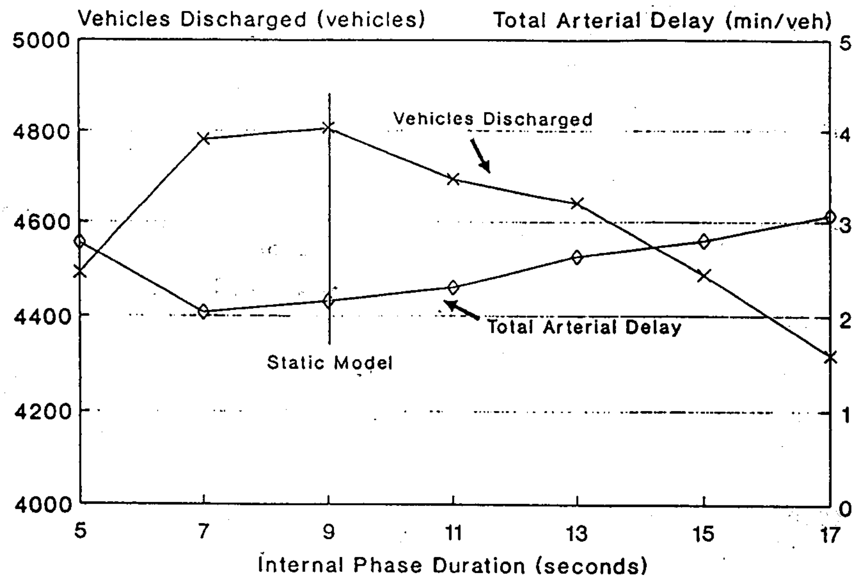


(a) Case 1 - Base Case

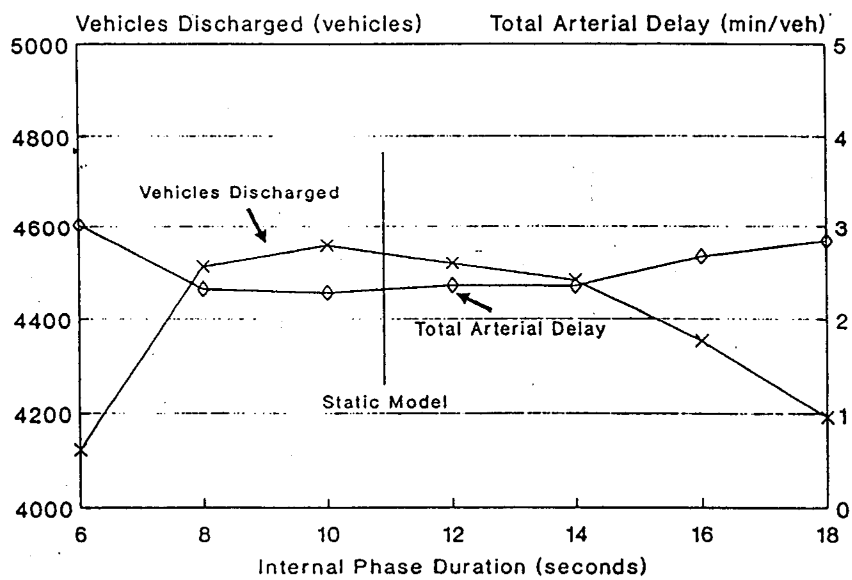
(b) Case 2 - Heavy Turn-In Traffic from Cross Street

(c) Case 3 - Asymmetric Turn Percentages

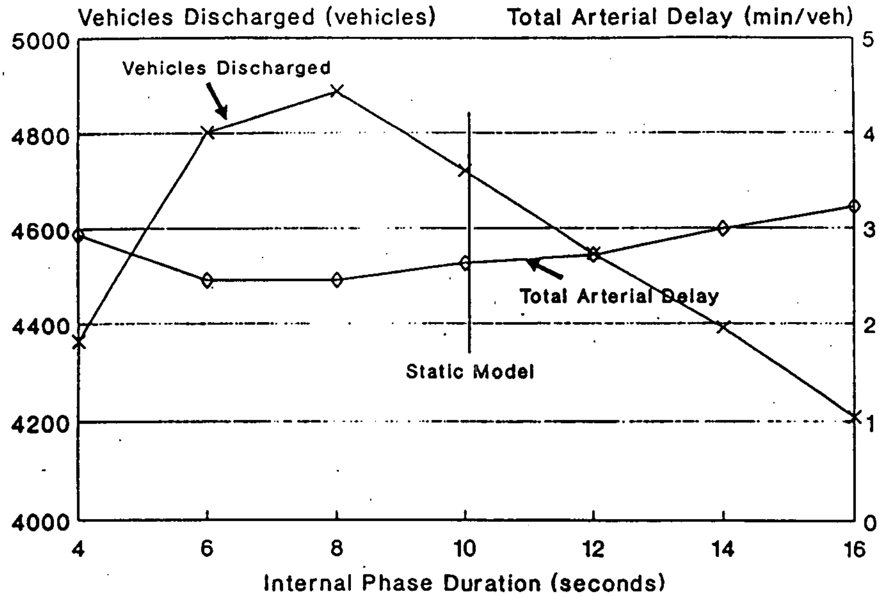
⟨FIGURE 10⟩ Three Cases of Turning Percentages.



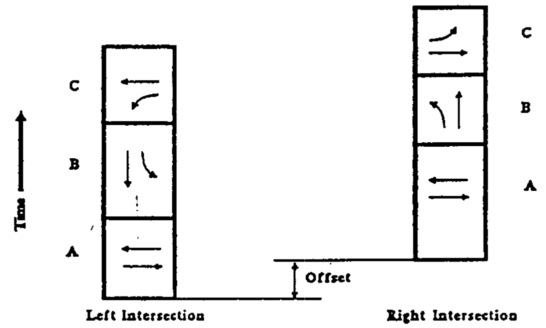
〈FIGURE 11〉 Effects of Green Split on Vehicles Discharged and Total Arterial Delay (Case 1).



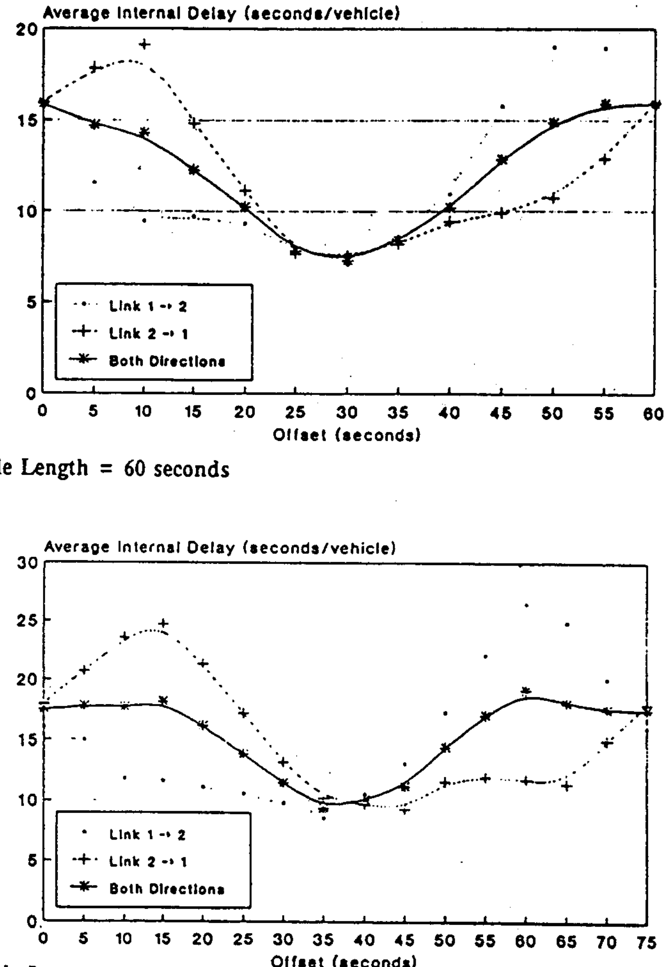
〈FIGURE 12〉 Effects of Green Split on Vehicles Discharged and Total Arterial Delay (Case 2).



⟨FIGURE 13⟩ Effects of Green Split on Vchicles Discharged and Total Arterial Delay (Case 3).



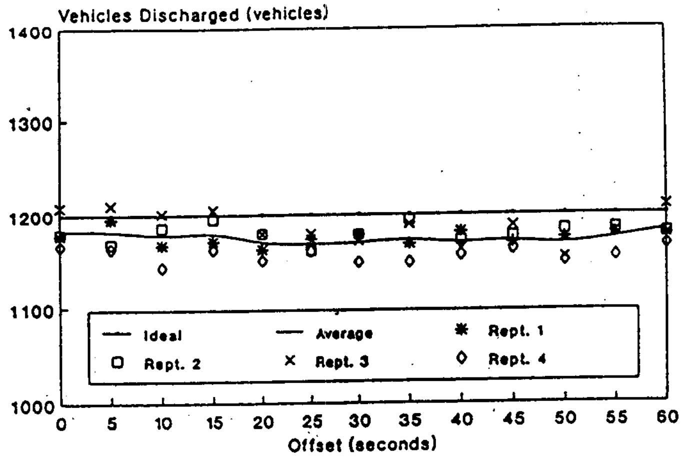
⟨FIGURE 14⟩ Definition of Offset in Static Model.



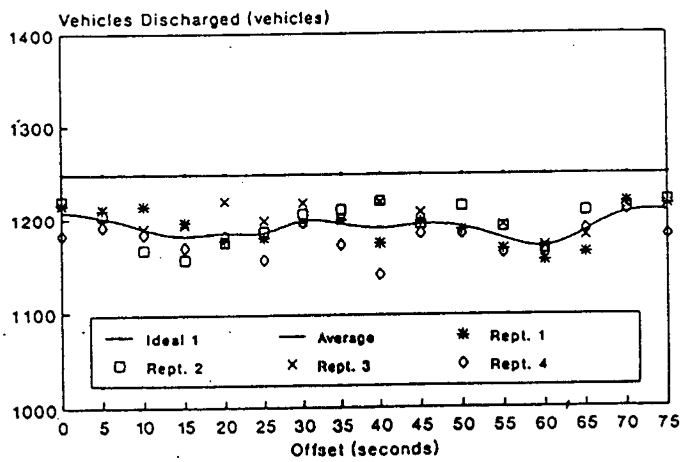
(a) Cycle Length = 60 seconds

(b) Cycle Length = 75 seconds

⟨FIGURE 15⟩ Plot of Offset versus Average Internal Delay.

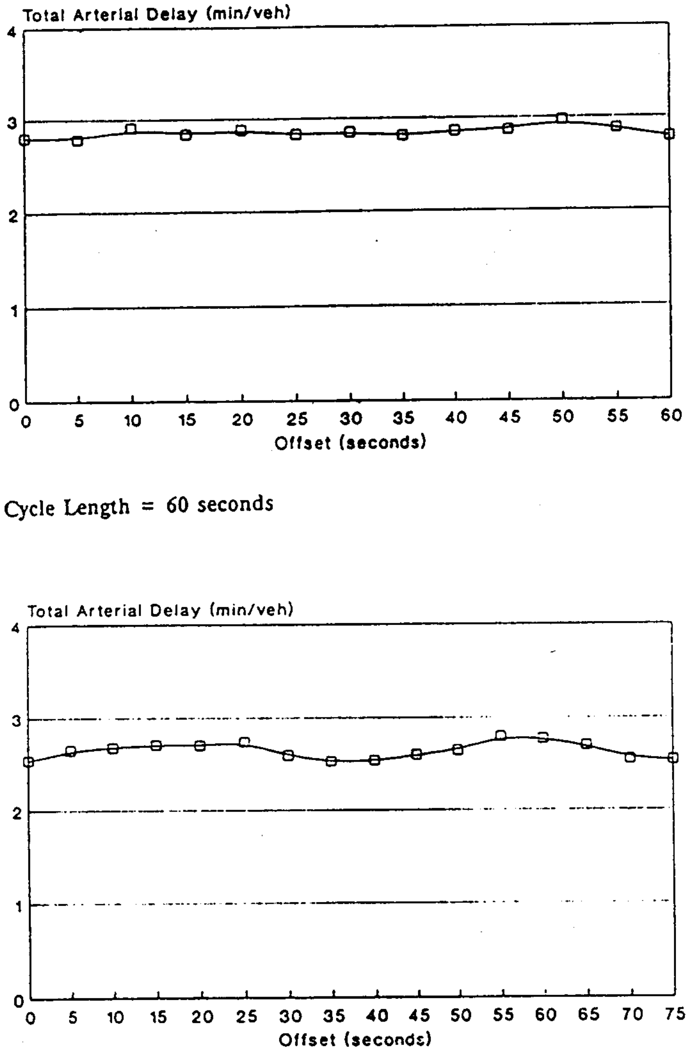


(a) Cycle Length = 60 seconds



(b) Cycle Length = 75 seconds

⟨FIGURE 16⟩ Plot of Offset versus Vehicles Discharged.



(a) Cycle Length = 60 seconds

(b) Cycle Length = 75 seconds

⟨FIGURE 17⟩ Plot of Offset versus Total Arterial Delay.