Incorporating the Effects of Signal Transitions in the Selection of Timing Plans in Traffic Responsive Signal Systems

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16. Abstract

As part of this research project, a computer program, SIGTRAN, was developed to estimate the impacts of the transition phase when changing timing plans. The program uses the same procedures as PASSER II-90 to compute the estimated delay due to transition. Using the SIGTRAN program, a proposed algorithm for selecting new timing plans in a traffic responsive mode was developed. The algorithm compares the delay associated with retaining the old traffic signal timing to the delay associated the implementing a new timing plan plus the delay caused by transitioning to the new timing plan. If the delay for the old timing plan exceeded the delay estimates for the new timing plan plus the delay accrued during transition, then the new timing plan was adopted; otherwise, the old timing plan was retained. The new algorithm was tested using data from NASA Road 1 in Houston, Texas. It was found that the traffic responsive timing plans developed using the algorithm produced substantial delay savings in the A.M. Peak and Noon periods over operating the signals in a time-of-day mode. Less significant delay savings were generated during the A.M. Off-Peak, P.M. Off-Peak, and P.M. Peak periods. During these periods, either traffic volumes did not change significantly or the differences in the timing plans were so slight that most of the benefits of changing timing plans were offset by the increase in delays caused by transitioning to the new timing plans.

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EXECUTIVE SUMMARY

Many transportation agencies are in the process of installing closed-loop traffic signal systems. However, most are not fully utilizing the capabilities of these systems. Most closed-loop traffic signal systems are set up to operate in a time-of-day mode, where timing plans change based on the time of day and not changing traffic patterns. Traffic responsive control, however, attempts to optimize the signal settings by monitoring traffic patterns and selecting timing plans that more closely match the measured traffic demands. There is a disadvantage, however, to changing timing plans. Traffic signals must pass through a transition phase every time a timing plan is changed. Limited research has shown that the transition phase can cause significant increases in delay, but these delays are not considered in the process for deciding when to change timing plans in a traffic responsive system.

As part of this research project, a computer program, SIGTRAN, was developed to estimate the impacts of the transition phase when changing timing plans. The program uses the same procedures as PASSER II-90 to compute the estimated delay due to transition. Like PASSER II-90, delay estimates are only valid when the volume-to-capacity ratio is 1.0 or less.

Using the SIGTRAN program, a proposed algorithm for selecting new timing plans in a traffic responsive mode was developed. The algorithm compares the delay associated with retaining the old traffic signal timing to the delay associated the implementing a new timing plan plus the delay caused by transitioning to the new timing plan. If the delay for the old timing plan exceeded the delay estimates for the new timing plan plus the delay accrued during transition, then the new timing plan was adopted; otherwise, the old timing plan was retained.

The new algorithm was tested using data from NASA Road 1 in Houston, Texas. It was found that the traffic responsive timing plans developed using the algorithm produced substantial delay savings in the A.M. Peak and Noon periods over operating the signals in a time-of-day mode. Less significant delay savings were generated during the A.M. Off-Peak, P.M. Off-Peak, and P.M. Peak periods. During these periods, either traffic volumes did not change significantly or the differences in the timing plans were so slight that most of the benefits of changing timing plans were offset by the increase in delays caused by transitioning to the new timing plans.

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CHAPTER I.
INTRODUCTION

A closed-loop signal system is an integrated coordinated traffic signal system which has the capability of monitoring traffic characteristics of the roadways where it is installed. The system can monitor occupancy, vehicular speed, and vehicles entering or leaving the system. A closed-loop system also has the capability of changing signal timing plans throughout the day based on changing demand. There are three modes of operation used in closed-loop signal systems: time-of-day, traffic responsive, and manual.

In a time-of-day operating mode, traffic signal timing plans are automatically selected from a library of timing plans on a time-of-day and day-of-week basis, regardless of current traffic conditions. In effect, a traffic engineer attempts to implement the appropriate timing plan based on historical data. With traffic responsive control, a master controller monitors changing traffic patterns throughout the day and implements new timing plans as traffic conditions warrant. During a typical day many fluctuations in traffic demand occur, causing the system to change to a new timing plan. The time used to adjust to the new timing plan is defined as the transition phase. The transition phase begins when the first intersection starts adjusting timing plans and ends when the last intersection completes adjusting timing plans (*1*).

PROBLEM STATEMENT

Operators face the problem of deciding which of the three operating modes to use in their coordinated signal systems. Manual operating modes are impractical because of the large cost associated with manually changing a coordinated signal system in day-to-day operation. The operator needs to determine which of the remaining two operating modes is most beneficial. This is not to say that one operating mode is more beneficial in all cases. Different corridors may require different operating modes based on their needs and relative traffic patterns. An objective of this research is to document a method an engineer could use to decide which operating mode to implement. The benefits and drawbacks of each operating mode must be considered to help determine which is a better operating mode in a given situation.

With a traffic responsive mode, changes in timing plans can occur frequently throughout the day; however, there is little information quantifying the effects of transition. When evaluating a traffic responsive system, the effects of transitioning from an old timing plan to a new timing plan should be considered. Transitioning between timing plans causes a disruption when phases are adjusted to achieve the correct timings and offsets of the new timing plan. The transition effect can be neglected in a time-of-day mode because few transitions occur in a typical day.

The optimum setting of the new timing plan is more beneficial to the user, because they will experience less delay. Despite the obvious benefits, the transition to the new timing plan may cause disruptions that may make it more beneficial to keep the old timing plan. The system cannot constantly switch from one timing plan to a new timing plan because transition periods will overly disrupt traffic, and benefits of a new timing plan will never be realized. Two questions arise:

- When should a transition to a new timing plan occur?
- How much disruption occurs during the transition phase?

RESEARCH OBJECTIVES

The purpose of this research is to examine these two questions in greater detail. Specifically, the objectives of this research study were as follows:

- Develop a method for estimating the impacts of transitioning between two traffic signal timing plans.
- Illustrate how the procedure can be used in deciding when to change timing plans in response to changing traffic conditions.

SCOPE

This simulation study focused on the issues associated with measuring and estimating the impacts of transitioning from one traffic signal timing plan to the next. This project was limited in its scope to a simulation/laboratory-type evaluation of the issues associated with transitioning. As such, all of the results of this study are based on analytical and simulation evaluations. Field studies need to be performed to verify the results of these simulation studies.

STRUCTURE OF REPORT

The results of the research study are contained in five chapters. Chapter II of this reports provides background information on current methods used in both research and practice to transition from one timing plan to the next. Chapter III presents a model developed as part of this research to estimate the impacts associated with transitioning between timing plans. Chapter IV illustrates how estimates of the delays that are incurred during the transition period can be used to assess whether traffic conditions have changed enough to warrant implementing a new timing plan. Finally, Chapter V provides a summary of the findings associated with this research.

CHAPTER II.
BACKGROUND

Three different modes are generally used to operate traffic signal systems: manual, time-of-day, and traffic responsive.(*1*) In the manual mode, a traffic engineer manually changes the signal timing plans to adopt to the traffic demands. A manual change in the timing plan might be used in the case of a special event (for example, a major sporting event). The traffic engineer would manually change the timing plan to suit the traffic demands for the special event.

Most closed-loop systems presently operate in a time-of-day mode. In this operating mode, timing plans are changed based on a time-of-day schedule. The plans are derived to suit the average traffic flows that are expected to occur during a particular time period. Since average traffic flows vary with time, it is common practice to derive a number of plans for different times of the day. For example, a typical time-of-day timing plan is shown in Figure II-1. Because timing plans do not change in response to changing traffic conditions, time-of-day mode performs best when traffic patterns are relatively stable and predictable throughout the day.

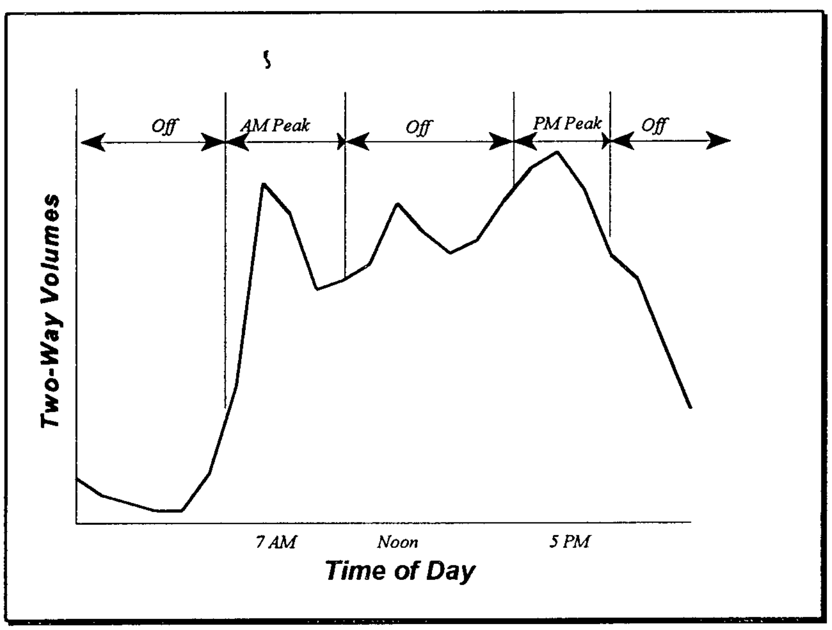

Figure II-1. Typical Time-Of-Day Timing Plan

Since only a few timing plans are implemented in a typical day, the effects of transitioning from an old timing plan to a new timing plan are generally neglected with time-of-day mode. One disadvantage to this mode of operation is that traffic demands can vary greatly from day-to-day. A time-of-day operating mode may operate in an inappropriate timing plan if a non-typical fluctuation in traffic occurs (See Figure II-2). The figure shows a situation when traffic conditions are nontypical (represented by the dotted line in figure) and how the signal system operating in a time-of-day mode would be operating in the inappropriate timing plan.

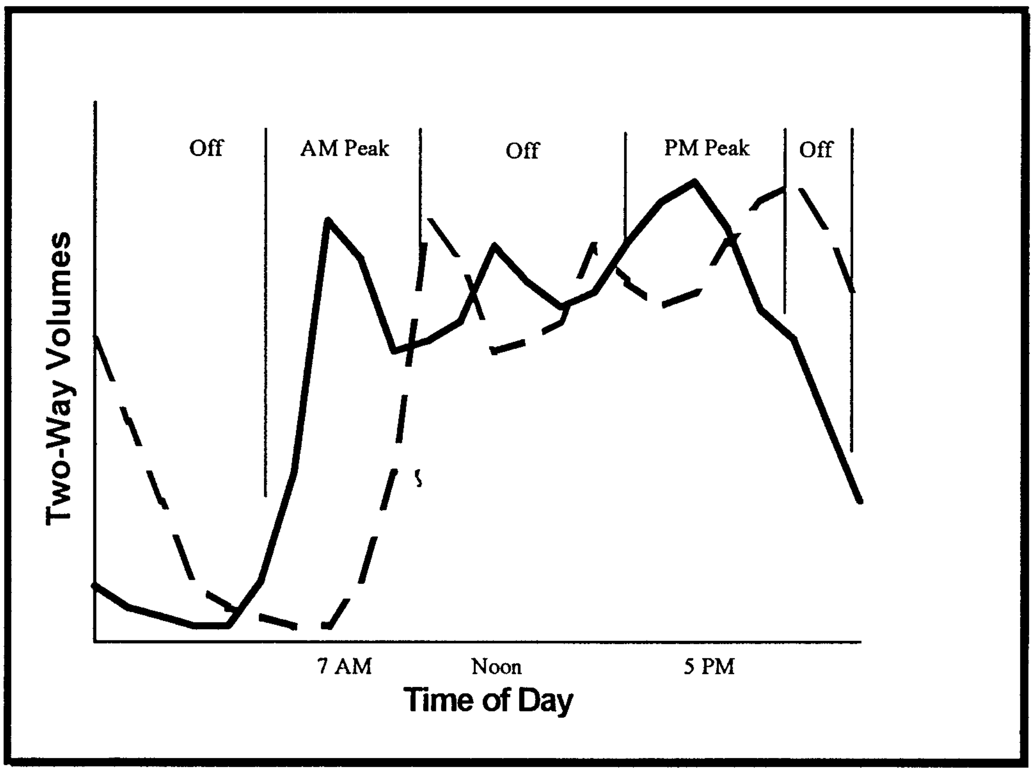

Figure II-2. Shift of Time-Of-Day Traffic Pattern

Traffic responsive systems use algorithms that examine traffic patterns throughout the day and implement new timing plans as conditions` warrant. Using real-time information, this mode of operation for a signal system is sensitive to actual traffic demands on the corridor. With many fluctuations in traffic during a typical day, many different timing plans could be implemented.

Because of the many changes to different timing plans, the effect of transition when changing timing plans must be considered. The timing plans are adjusted by adding or subtracting time during certain intervals to reach the new setting. An increase in delay is associated with the transition phase because disruption to traffic is caused when phases are lengthened or shortened. Also, the offsets for through progression need to be corrected to reestablish good traffic progression.

Generally, the philosophy of traffic signal control is to match the traffic signal timing plans with the existing traffic conditions that exist on the arterial; however, every time a new timing plan is enacted in a system, there is a period of time with the traffic signal is operating with signal settings that are less optimal. Frequently, these signal settings can extend over multiple cycles until the new timing plan can be implemented. This can frequently increase the amount of delay experienced on the arterial. Because of the disruptive nature of changing from one timing plan to another, some traffic engineers feel that changing timing plans at the wrong time (say the peak period) could be more harmful to traffic operations that having the wrong timing in operation.(*2*)

TRANSITION

The fundamental problem with changing timing plans is how to get from "Plan A" to "Plan B" without allowing the following:(*2*)

- Red displays that are so short that pedestrians become stranded in front of moving traffic.
- Green displays that are so short that drivers get confused and have rear-end crashes as one stops but the other does not.
- Excessive queues that build on intersection approaches because of extended red intervals.
- Some approaches become "starved" for vehicles due too long red displays upstream of the signal.

To prevent these problems, traffic signal controller manufacturers have incorporated strategies that allow traffic signals to *transition* from one timing plan to the next. A *transition* is the process of changing the phasing, timing, and offsets in a coordinated signal system from one timing plan to the next. The *transition period* is the time required for the transition from one timing plan to another to occur.

METHODS OF TRANSITION

Different controller manufacturers use different methods to transition between timing plans. The following describes the methods that are available to transition between timing plans for two venders of traffic signal controllers commonly used by the Texas Department of Transportation (TxDOT).

Eagle

The Eagle controller has four methods of effecting an offset change (*3*):

- Infinite Dwell
- Shortway,
- Shortway Add Only, and
- Dwell with Interrupt.

With the *Infinite Dwell* transition method, the controller dwells in the coordinated phase until it receives a proper synchronization pulse from the master controller. To use this method, the master controller must contain an offset interrupter. The offset interrupter is a device that imposes a number of shifting interrupter pulses onto the interconnect line containing the real synchronization pulse. The interrupter keeps the local controller from receiving the proper synchronization pulse until the desired offset is achieved. Once the proper offset is achieved, a synchronization pulse is received by the local controllers and the rest of the phasing is allowed to occur.

As the name implies, the *Shortway* transition method establishes a new offset by the shortest way possible. With this transition method, time is added or subtracted to different phases until the new offset is achieved. By using this method, the time required to transition to a new offset plan is never more than 50 percent of the cycle length.

With the *Shortway* transition method used in the Eagle controller, the transition can occur over multiple cycles. The decision whether to add or subtract time during transition depends upon the total amount of transition time (i.e., the time difference between the existing and the proposed offset). If this time difference is less than 50 percent of the cycle length, then time is added until the proposed offset is reached. If the difference between the existing and the proposed offset is more than 50 percent of the cycle length, then time is subtracted. When time is being added, it is added only to the coordinated phase(s). When time is being subtracted, an equal portion of the total transition time is subtracted from all phases, subject to the availability of time (i.e., the phase is not currently running at a minimum time). The maximum amount of time that can be added or subtracted during each cycle is 18.75 percent of the cycle length. For example, for a 100-second cycle, the maximum amount of time that can be added or subtract during a single cycle while the system is in transition is 18.75 seconds. In those cases where the new offset cannot be reached withing five cycles by subtracting time from the phases, the offset changes is affected by adding time.

A variation on the *Shortway* transition method is the *Shortway Add Only* method. With this method, the transition between two offsets is accomplished by dwelling in the green portion of the coordinated phase. The maximum time the controller can dwell in the coordinated phase using this transition method is 18.75 percent of the cycle length. After dwelling the prescribed amount of time, the controller releases and begins timing the other phases in the plan. If the new offset is not reached during the first dwell time, the process is repeated until the desired offset is reached.

A final transition option available with an Eagle controller is the *Dwell with Interrupt* (or *Maximum Dwell*) method. This method of transitioning between offsets is similar to the *Shortway Add Only* method in that the controller is forced to dwell in the coordinated phase; however, the maximum amount of time that the controller can dwell in this phase is set by the user (instead of being defined as 18.75 percent of the cycle length). The user is allowed to set the dwell time to range between 1 second and 999 seconds. (A user-entered value greater than the cycle length will cause the controller to transition in a similar fashion as the *Infinite Dwell* method.) After the controller has dwelled in the coordinated phase for the allotted time, it services the remainder of the phases in the cycle. This process is repeated until the desired offset is reached.

Econolite

With an Econolite controller, the user has three options for effecting an offset change: *(4)*

- Smooth,
- Add Only, and
- Dwell

With the *Smooth* transition method, an offset is changed by moving the current offset toward the desired offset in the shortest time possible. This change is made by either adding a maximum of 20 percent or subtracting a maximum of 17 percent of the cycle length to the coordinated phase. After each transition, the controller computes the difference between the current offset and the desired offset. If the desired offset is greater than the current offset by more than 50 percent of the cycle, the controller will add time to the coordinated phase. If the desired offset is less than the current offset by more than 50 percent of the cycle or if the desired offset is greater than the current offset by more than 50 percent, the controller will subtract time to the coordinated phase. If the controller determines that subtracting time from the coordinated phase results in a cycle length that is below the minimum cycle length, the controller will force the offset change to occur by adding time.

With the *Add Only* transition method, changes in offsets are affected by only adding time to the coordinated phase, regardless of the magnitude of the offset change. Under this option, time is added to the coordinated phase every cycle until the desired offset is reached. The maximum amount of time that can be added during each cycle is equal to 20 percent of the cycle length.

In addition to *Smooth* and *Add Only* options for transitioning, the user can also effect an offset change by using a *Dwell* transition method. With the *Dwell* method, the controller holds the coordinated phase at the beginning of the green portion for a time interval specified by the user. This dwell time can be entered by the user as either a percentage of the cycle length (0-99 percent) or in seconds (0-255 seconds). After the dwell interval expires, the controller releases the coordinated phase and normal timing resumes. The dwell interval is repeated once each cycle until the desired offset is achieved.

EVALUATION OF TRANSITION STRATEGIES

Very little research existing on finding a "best" or optimum way of transitioning from one plan to another. In the early 1980s, Basu conducted a study of the factors influencing the number of signal timing plans required for UTCS 1st Generation traffic control systems.*(1)* In this study, Basu used the NETSIM and TRANSYT models to examine the effects on network traffic of timing plan changes and transitions under different conditions of changing demand. In the study, a shortway transition algorithm was applied — one in which the transition at any signal is accomplished by expanding or contracting the cycle length to achieve the offset change in the least possible time. In this case, the minimum cycle length allowed was equal to the sum of the phase minimums (minimum green plus clear interval times), while the maximum allowed was the smaller of 255 or twice the new cycle length. All transitions computed for the control strategies were completed in two cycles —one expanded or contracted cycle to achieve the desired offset change, and the second cycle to implement the new timing plan. The results of this study showed the following:

- The effects of transitions between timing plans are significantly more deleterious than may have been recognized until now.
- For increasing demand conditions, the detrimental effects of transition increase in magnitude for higher transition frequencies and increasing rates of demand change.
- Thirty-minute periods seem too short for the benefits of new timing plans to offset the effects of transition.
- The percent increase in total network delay caused by transition increases with increases in network saturation, and the degree of this increase in delay increases significantly as the instantaneous rate of change of network saturation goes up.

- For increasing demand conditions, the total transition time for the entire section varies very little with changing demand or with different rates of demand changes.

In another study, Ross examined six theoretical methods of transitioning between timing plans.(5) Using the frontage road system of the Central Expressway in Dallas as a test network and the NETSIM traffic simulation model, the impacts of the different transition algorithms on average speed and stops per vehicle were examined under three different volume conditions: volumes increasing 10 percent every 5 minutes, volumes constant at their base value, and volumes decreasing every 5 minutes. This study found that the best method of transitioning between timing plans was to either extend the main-street green, or gradually adjust the offset over multiple cycle by adding or reducing the green of each phase. [Note: this approach is similar to the Shortway transition method used by many controller manufacturers.] The gradual transition produced the highest average speeds and the lowest average number of stops whenever traffic volumes were changing and was good (second or third best) when traffic volumes were constant. The extended main-street green transition method was superior when traffic volumes were constant and good when volumes were changing. Because of these findings, Ross concluded that there was no difference between these two algorithms; however, the way the study was conducted might have influenced this recommendation. As mentioned above, traffic volumes were gradually increased by 10 percent every 5 minutes. At the end of each 5-minutes period, a new timing plan was generated based on the traffic demands measured by the simulation model. Because the demand changes were relatively small, the changes in the signal settings (and in particular the relative changes in the signal offsets) were likely small as well, resulting in only minor adjustments to the offset. When only small changes in offset are needed, one would expect the extend method and gradual method of transitioning to produce similar impacts on traffic operations. If on the other hand, large changes in offset levels were required, the authors might have found that there were significant differences in the way the two transition strategies impacted traffic operation. This study showed, however, that the choice of a transition algorithm is extremely important, especially in computer-controlled traffic signal systems that are designed to the responsive to changing traffic conditions. A poorly selected transition algorithm could degrade the performance of the system by as much a 20 percent, completely negating the expected benefits of real-time, responsive control.

Bretherton used a similar approach to examine the impacts of five different strategies for transitioning between timing plans.(*6*) The transition strategies examined in this study are similar in concept to those used by Ross. Fifty-four timing plan changes were simulated for each transition method. Each method was tested at 90 percent, 100 percent, and 105 percent of the flow normally experienced when changing from an off-peak to a P.M. peak plan in Glasgow, England. For each flow level, the timing plans were changed at three different points in the cycle: 1 second, 31 seconds, and 71 seconds. A total of nine simulation runs, each lasting for 3½ hours, was performed for every transition strategy. Every 10 minutes, the total number of vehicle-kilometers traveled and the total number of vehicle hours spent in the network was recorded. The additional delay caused by a timing plan change was assessed by comparing the totals for the 10 minutes immediately following a plan change, with the totals for the remaining 20 minutes between plan changes.

For each transition method, the total travel time (veh hr/hr) in the 10-minute periods following a timing plan change were plotted against the demand (veh km/hr) in the corresponding periods. Analysis of covariance procedures (used to account for the dependence of travel time on demand) was used to compare regression lines for each of the transition strategies to the "no change" regression line. This comparison found that there was a significant difference in the travel time between the plan change regression lines and the "no change" regression lines. This difference was significant at 1 percent. The study also found that there was not significant difference between the Modified Abrupt and the Minimax transition strategies. These two strategies, however, showed a significant improvement (at a 5 percent level of confidence) over the other three transition strategies. As shown in Table II-1, the Modified Abrupt and the Minimax strategies also had approximately the same impact on the average amount of delay experienced by vehicles in the system.

Table II-1. The Delay Caused by Five Transition Methods.

Transition Method	Average additional delay per vehicle in seconds	
	Off peak to PM peak	PM peak to off peak
Basic Abrupt	36	45
Modified Abrupt	14	25
Maximum Green	36	43
Slope	48	60
Minimax	15	20

CHAPTER III.
ESTIMATING THE IMPACT OF SIGNAL TRANSITION

As shown in the previous chapter, the transition from one timing plan to the next can cause severe disruption to the normal timing of a traffic signal. Sometimes this disruption can last for multiple cycles. Because timing plans should only be changed if an improvement to traffic operations can be achieved, a method for estimating the impact of changing timing plans on traffic operation is needed. As part of this study, a simple procedure for estimating the delay associated with timing plan transition was developed. The procedure is based on the PASSER II-90 delay estimation procedures. This chapter summarizes the development effort and illustrates the required input and output of the program.

DELAY ESTIMATION IN PASSER II-90

As shown in Figure III-1, PASSER II-90 uses platoon length at the upstream intersection to estimate platoon length at the downstream intersection.*(7,8)* The length of the platoon at the upstream intersection i, LP_i, is given by the following equation:

$$LP_i = g_o + PVG(g-g_o)^2/g \qquad \text{(III-1)}$$

where,

LP_i	=	Length of platoon at upstream intersection, i (sec),
g_o	=	Time required for queued vehicles to clear the intersection at i (sec),
PVG	=	Percent of vehicles arriving on green at i, and
g	=	effective green time for the progressed movement at i.

The percent vehicles arriving during green (PVG) above is computed as follows:

$$PVG = PTT_j * GO_j / LP_j + (1\text{-}PTT_j)\, RO_j / (C\text{-}LP_j) \qquad \text{(III-2)}$$

where,

PTT_j	=	percent of total through traffic at j arriving from i,
GO_j	=	green overlap for the platoon traffic from i at j as shown in figure 1 (sec),
RO_j	=	green overlap for the nonplatoon traffic component from i at j (sec), and
LP_j	=	platoon length at the downstream intersection j (sec).

Using these relationships, the platoon length at the downstream intersection j, LP_j, can be estimated using the following equation.

$$LP_j = LP_i * PD_{ij} + 0.8*(0.9 + 0.056t_{ij}) \qquad \text{(III-3)}$$

where,

LP_j	=	Length of platoon at downstream intersection, j (sec),
LP_i	=	Length of platoon at upstream intersection, i (sec),
PD_{ij}	=	Platoon dispersion factor (*8*),
	=	$1.0 + (0.026 - 0.0014*NP)*t_{ij}$
t_{ij}	=	travel time between i and j in seconds, and
NP	=	number of vehicles in platoon at i.

Three arrival rates during the red period at the downstream intersection are defined: a flow rate for the early traffic arrivals, which are part of the main street platoon traffic; a flow rate for late arrivals, also part of the main street platoon traffic; and a flow rate for the nonprogressed traffic during red. Assuming a constant flow rate in the platoon length LP_i., the flow rates q_{re} and q_{rl} (in Figure III-1) are equal. The nonplatoon flow will be late whenever the platoon flow is early or straddles the red. Similarly, nonplatoon traffic will be early whenever the platoon traffic is late.

Using the assumptions about the platoon and nonplatoon flow rates, and the length of the platoon, as described above, PASSER II estimates the uniform delay (UD) through stepwise demand integration. The uniform delay equation consists of three parts. The first part (UD1) represents platoon traffic delay in red, the second part (UD2) represents the nonplatoon traffic delay in red, and the third part (UD3) represents the delay during queue dissipation.

The first part of the equation is shown below:

$$UD1 = [q_{rp} r^2 / (2qC)] * FEAL \tag{III-4}$$

where,

q_{rp}	=	platoon flow rate in red, (veh/sec),
	=	$PTT_j (1 - GO_j / LP_j) * q*C/r$,
q	=	average flow rate (veh/sec),
r	=	effective red for the progressed movement at j (sec),
C	=	cycle length (sec),
FEAL	=	factor for early and/or late arrivals as given by $[(r_e - r_l)/r] + [2 * r_l / (r_l + r_e)]$, and all other variables are as defined earlier (see figure 1 for r_e and r_l).

The second part of the uniform delay equation is shown in the equation below:

$$UD2 = [q_{rnp} r^2 / (2qC)] * FEAL \tag{III-5}$$

where,

q_{rnp}	=	nonplatoon flow rate in red, (veh/sec),
	=	$(1-PTT_j)* [1 - RO_j/(C- LP_j)] * q*C/r$, and

All other variables are as defined above, except FEAL. FEAL will be different because the values of r_e and r_l (see Figure III-1) are different for the nonplatoon traffic flow. The adjustment factor FEAL is derived based on the platoon arrival patterns. It not only differs from platoon to nonplatoon traffic, but also from pattern to pattern. For further details please refer to the discussion by Malakapalli (*9*).

The third part of the uniform delay equation represents the delay for queue dissipation after the start of green. It is given as:

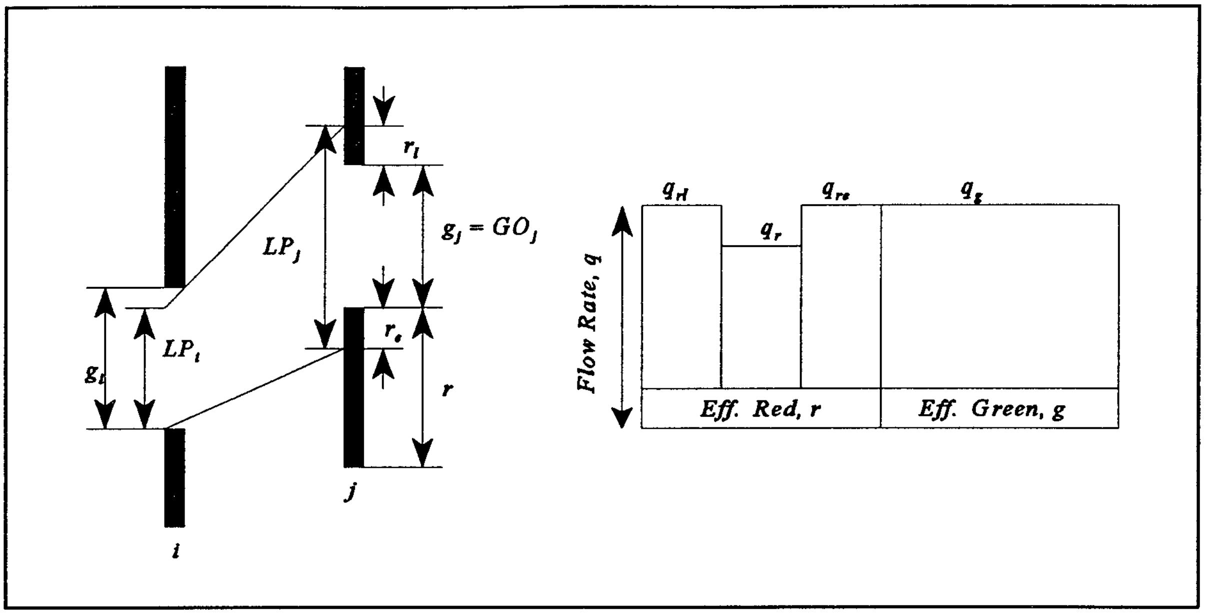

Figure III-1. Flow Rate Definition in PASSER II-90 Delay Estimation Model.

$$UD3 = q_r * r^2 / (2qC) * [1/(s - q_g)] \qquad \text{(III-6)}$$

where,

q_r	=	combined mean flow rate during red (veh/sec),
q_g	=	combined, mean flow rate during green (veh/sec), and
s	=	saturation flow rate (veh/sec).

Combining the three parts, UD1, UD2, and UD3 the total uniform delay is obtained. The random delay is computed using the HCM second-term delay for incremental random and overflow effects. (*10*)

DEVELOPMENT OF TRANSITION DELAY ESTIMATION METHOD

Representation of Transition

In a fixed-time signal system, the sequence of fixed durations of red and green repeat over time during regular, non-transition periods. If the system is coordinated, then the time relationship between consecutive signals in a system also remains constant. Therefore, the platoon arrival patterns can be assumed to be similar cycle-after-cycle. During transition, however, neither the duration of red and green nor the time relationships between consecutive signals are constant. The duration of red and/or green vary from cycle to cycle until the new offsets, splits and sequence are attained.

Because of the variation in timing at individual signals, as well as variation in the time relationship between different signals (i.e., variation in offset), the platoon arrival patterns change from cycle to cycle during transition. In order to estimate delay, therefore, the sequence of red, and green periods for each movement is constructed from the beginning of transition until the new timing plan is attained. Figure III-2 depicts the sequence at four consecutive signals in a coordinated system, for one of the progressed movements. Similar sequences are constructed for nonprogressed movements. The time relationship between consecutive signals, however, is not important for nonprogressed movements.

Transition Duration

In Figure III-2, R_1 and G_1 represent the red and green durations in the old plan and R_2 and G_2 represent the splits for the new plan. All durations in between represent transition cycles. It should be noted from Figure III-2 that the sequence of red and green (R_2 and G_2) is continued even after the new timing is attained at intersections 2, 3 and 4. Although the new timing is attained, the platoons arriving at the intersection during these periods are generated during an upstream transition cycle. Hence, the platoon arrivals during these cycles are not regular patterns. In the proposed methodology, all cycles that are either actually transition cycles, or those that receive platoons affected by transition at an upstream signal are considered as transition cycles. For nonprogressed movements, since the arrival is assumed to be random, the actual transition cycles are the only transition cycles. The sum of all transition cycles equals the transition duration. It should be noted that different movements experience different durations of transition.

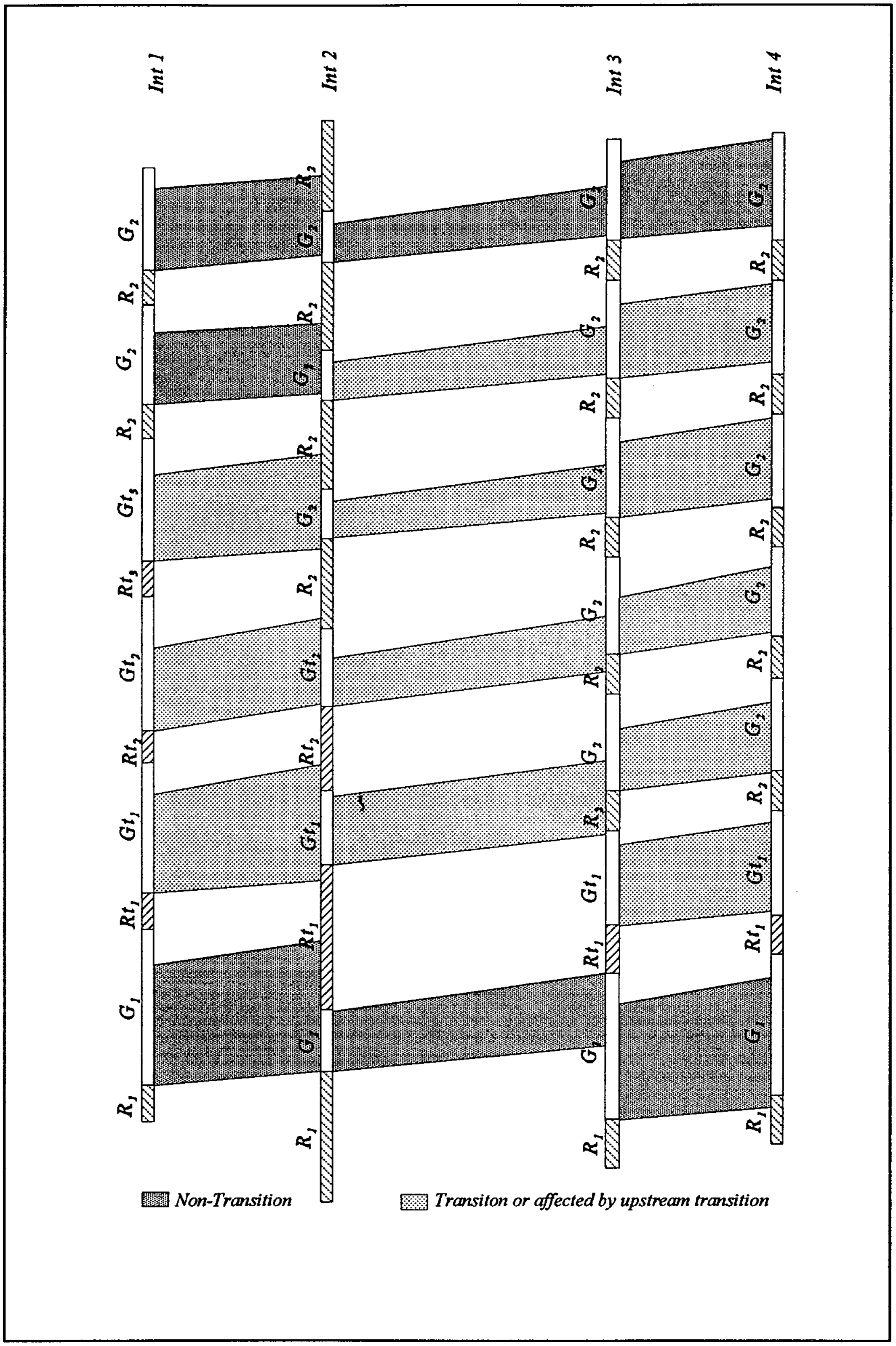

Figure III-2. Signal Timing and Platoon Arrival Patterns During Transition in a Coordinated System.

Delay Estimation for Progressed Movements

The delay during transition for the progressed movements is estimated using the PASSER II methodology described above. Based on the arrival patterns during red, different flow rates for platoon and nonplatoon flows are used to compute the total queue, delay during red, and delay during queue dissipation. The only distinction between this methodology and the PASSER II implementation is that the early-and-late factors are not used here. Since the time of platoon arrival is taken into consideration for each transition cycle to compute delay, the resulting estimate will be the same. Figure III-3 depicts how the delay is computed. The shaded area is the delay during the cycle. It should be noted that since the platoon arrival times and patterns during successive transition cycles vary, the delay also varies. The delay estimated using this procedure was found to be comparable to the delay estimated using PASSER II methodology for nontransition cycles.

Delay Estimation for Nonprogressed Movements

As mentioned earlier, the sequence of red and green periods is also constructed for the nonprogressed movements, as in Figure III-2. A constant arrival rate during red and green periods is assumed. The delay estimation is based on the HCM delay equation.

IMPLEMENTATION

The proposed methodology for estimating delay during transition is closely related to the PASSER II-90 methodology described above. A FORTRAN program was developed to implement the methodology. The program, called SIGTRAN, reads the travel times, traffic volumes, saturation flow rates, and the old and new signal timing plans as input. One method of transition, popularly called shortway transition by transportation professionals, was implemented to test the delay estimation methodology. It is assumed that all signals in the system use the same method of transition.

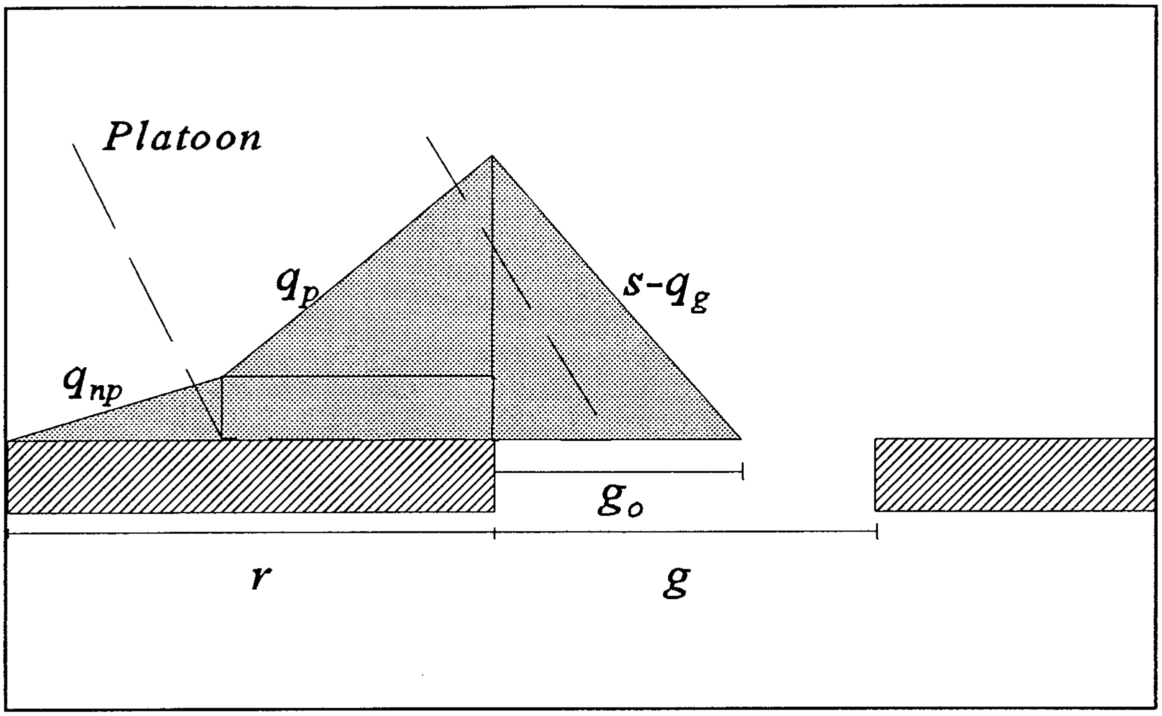

Figure III-3 Delay Estimation for Progressed Movements.

Input File

To execute the program, data is entered in an ASCII file in a DOS text editor. The data needed by the algorithm to estimate the amount of delay that occurs during transition includes the following:

- The number of intersections in the coordinated signal system.
- The travel time between each intersection, in both directions.
- The equivalent hourly flow rate of each movement at each intersection in the system.
- The saturation flow rate for each movement at each intersection in the system.
- The timing plan (offset, phasing pattern, and phase durations) of both the existing signal plan and the proposed new timing plan.
- The phases that can be used by the algorithm to transition to the new timing plan.

The structure of the input data file is shown in Figure III-4. Appendix A provides a sample input file.

Output

In terms of output, the program provides estimates of the following for each phase at each intersection:

- The total delay (in vehicle seconds) experienced during transition.
- The duration that each phase is in transition.
- The average stop delay experienced by individual vehicles on each phase during transition (in seconds/vehicle).

Figure III-5 shows an example of the output for a system of eight intersections. The sample assumes the first intersection in the system begins the transition sequence. Similar output is provided assuming each of the other intersections is the first in the system to begin transition. Therefore, the engineer can examine how the total system delay varies depending upon which intersection controls the transition sequence.

```
1. Number of intersections  (I5)
2. For each intersection
   travel time in direction A, travel time in direction B (f7.2,1x,f7.2)
   (travel time from upstream intersection. Blank or zero for intersection 1
    direction A, and intersection 'nsig' direction B)
3. For each intersection
   volume rate for each nema phase (vph) (8(f7.2,1x))
   saturation flow rate for each nema phase (vph) (8(f7.2,1x))
4. Signal timing plan 1
   For each intersection
    offset (f7.2)
    sequence (6(i2,1x))
    split (sec) (6(f7.2,1x))
5  Signal timing plan 2
   For each intersection
    offset (f7.2)
    sequence (6(f7.2,1x))
    Correction Phase (6(f7.2,1x))
    ( enter 1 if it is a correction phase, else blank or 0.
      Only one correction phase at this time!!)
    split (sec) (6f7.2,1x)
```

Figure III-4. Structure of Input File for SIGTRAN Program.

Verification

The estimation of delay is similar to the PASSER II methodology. The delay estimates from the routine for a nontransition cycle compared well with PASSER II estimates. Further validation of the estimates using real traffic controllers is underway.

Anchor = 1								
Phase (Nema)	5	6	1	2	3	4	7	8
Tot Del (sec)	0.00	99.48	68.86	0.00	4.41	0.00	0.00	90.44
Duration (sec)	0.00	100.00	100.00	0.00	100.00	0.00	0.00	100.00
Delay(sec/veh)	0.00	2.85	41.31	0.00	39.70	0.00	0.00	42.84
Tot Del (sec)	306.16	5559.66	287.05	1653.37	450.43	179.03	99.63	885.56
Duration (sec)	109.00	507.10	107.10	205.10	126.10	76.10	76.10	126.10
Delay(sec/veh)	46.81	29.81	44.67	27.27	54.49	23.27	23.57	53.56
Tot Del (sec)	0.00	57.79	92.88	1721.69	0.00	4.28	0.00	422.64
Duration (sec)	0.00	548.80	347.50	448.80	0.00	338.80	0.00	338.80
Delay(sec/veh)	0.00	0.28	48.11	8.97	0.00	45.43	0.00	46.78
Tot Del (sec)	0.00	1010.95	35.95	52.11	0.00	87.63	0.00	0.00
Duration (sec)	0.00	545.40	228.75	445.40	0.00	234.40	234.40	0.00
Delay(sec/veh)	0.00	5.89	47.14	0.29	0.00	48.07	0.00	0.00
Tot Del (sec)	383.77	888.44	199.81	1610.22	312.12	3.15	50.84	191.08
Duration (sec)	235.80	435.80	228.75	435.80	233.80	234.80	234.80	233.80
Delay(sec/veh)	50.51	7.84	49.13	12.89	52.24	48.25	48.72	49.04
Tot Del (sec)	186.46	341.17	0.00	311.18	0.00	126.99	107.13	0.00
Duration (sec)	108.00	394.50	0.00	394.50	0.00	93.50	93.50	0.00
Delay(sec/veh)	45.70	3.06	0.00	3.53	0.00	38.20	39.66	0.00
Tot Del (sec)	167.05	1413.98	385.36	1972.87	495.79	420.66	138.63	550.61
Duration (sec)	347.50	435.90	335.90	536.90	338.90	334.90	334.90	338.90
Delay(sec/veh)	48.07	14.04	49.17	14.76	45.40	47.10	46.57	45.70
Tot Del (sec)	0.00	0.00	0.00	661.76	406.36	0.00	0.00	214.27
Duration (sec)	0.00	0.00	344.50	341.00	340.00	0.00	0.00	340.00
Delay(sec/veh)	0.00	0.00	0.00	8.96	44.82	0.00	0.00	43.63

Figure III-5. Sample Output of SIGTRAN Program.

Constraints and Limitations

The program should be further enhanced to overcome some of the limitations and to realize the full benefits from the methodology. A primary advantage of the methodology presented here is its ability to estimate transition delay for any method of transition quickly and easily. To realize its full potential, more transition methods should be built into the program. Incorporating different transition methods will allow comparative evaluation of different methods of transition for any user-specified pair of timing plans and traffic conditions. Also, individual signal controllers may be allowed to transition using different methods if that is found to be more efficient.

Only one phase can be designated as a correction phase in the current version of the program. In several systems, however, correction during transition is applied to more than one phase. The program should be enhanced to permit more than one correction phase. This does not require any modifications to the delay estimation methodology.

The delay estimation methodology assumes undersaturated conditions. One of the serious detrimental effects of transition is that oversaturation might occur for some phases because of a long red interval (due to the extension of a conflicting phase green interval) followed by a regular green interval, or a reduction in the green interval for the phase. When such a condition arises, the program issues a warning, but forces saturated conditions (i.e, volume/capacity <1.0) for computing delay, resulting in an underestimation. The methodology should be enhanced to compute delay for oversaturated conditions.

CHAPTER IV.
CONSIDERATION OF TRANSITIONS IN SELECTING TIMING PLANS

Delay represents an indirect cost to motorists in terms of lost time and a direct cost in terms of fuel consumption during idle operation. (*10*) In a coordinated signal system, it is desirable to optimize the signals to the appropriate offsets and intervals for good traffic progression. Good traffic progression lessens delay whereas frequent stops by the driver at sequential intersections disrupt travel patterns and make the driver more irritable and frustrated.

In the control of a traffic signal network, the decision of implementing one signal timing plan in place of another is based on an assumption that if traffic conditions change sufficiently, a timing plan more suited to the new conditions will provide significant benefits to the traffic in the network in the form of reduced delays, stops, and increased speeds. However, there is added delay when transitioning to the new timing plan. The benefits derived from implementing a new timing plan need to outweigh the cost of transitioning from the old plan to the new plan.

This principle can be used to determine when timing plans in a traffic responsive system need to be changed. If the delay of the new timing plan plus the delay of the transition period to achieve the new timing plan is less than the delay of implementing the old timing plan, the new timing plan should be implemented. If the effects of transitioning outweigh the benefits of changing to the new timing plan, the old timing plan should remain as the timing plan for that period. An equation can be written to help determine when to change timing plans.

$$Delay_{New} + Delay_{Transition} < Delay_{Old} \qquad \text{(IV-1)}$$

This chapter describes how the above equation was used to select a traffic responsive timing plan that incorporates the effect of the transition phase in the selection process. Using the concept described above, an algorithm was developed for determining when to change timing plans. Traffic data and signal timing plans from Nasa Road 1 in Houston, Texas were used to test the algorithm. Delays with the new and old timing plans for each 15-minute period were estimated using PASSER II-90. SIGTRAN (described in the previous chapter) was used to estimate the impacts of transitioning from an old timing plan to a new timing plan. If the delay from the new timing plan plus the delay associated with the transition phase did not exceed the delay with keeping the timing plan from the previous 15-minute period, then the new timing plan was implemented. If the sum of the delay from the new timing plan and the transition phase exceeded the delay of the timing plan from the previous 15-minute period, then new timing plan was not implemented. Cumulative PASSER II-90 delays were used to compare the performance of the traffic responsive plan to the current time-of-day plan used on NASA Road 1.

METHODOLOGY

The methodology used to determine the traffic responsive timing plan is shown in Figure IV-1. This is a summary of the decision process used for each time period analyzed during the chosen study period. The timing plan designed would mimic the timing plans implemented if a traffic responsive system were in use on the system. If the delay of the new optimum timing plan plus the delay incurred to reach the new timing plan was less than the delay of the previous timing plan, the new timing plan was implemented. If this was not the case, the old timing plan was kept for the present traffic conditions. In either case the kept timing plan was now the old timing plan for the next interval evaluation. This process was repeated for each 15-minute increment of data.

Data

NASA Road 1, southeast of the City of Houston, served as a test bed for this study. It is an arterial connecting Interstate 45 to the Johnson Space Center. It consists of eight signalized intersections from the western limit of Kings Row to the eastern limit of Hospital. The Texas Department of Transportation (TxDOT) collected vehicle turning movement counts at each intersection in 15-minute increments from essentially 7:00 a.m. to 6:00 p.m (with some minor gaps). Geometries and travel speeds were collected by the Texas Transportation Institute (TTI) during site visits. A schematic of the study area is shown in Figure IV-2.

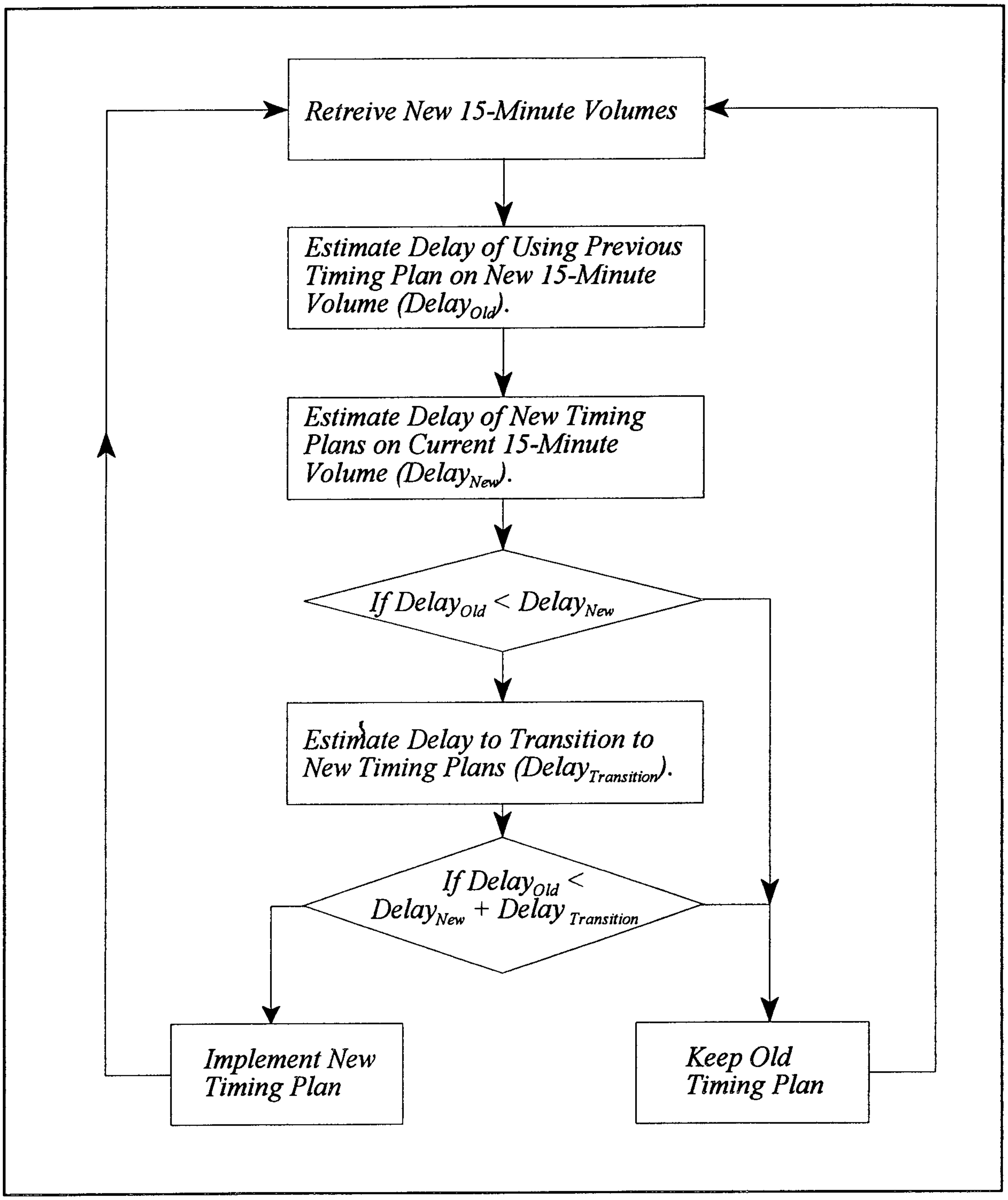

Figure IV-1. Flowchart of Timing Plan Selection Algorithm.

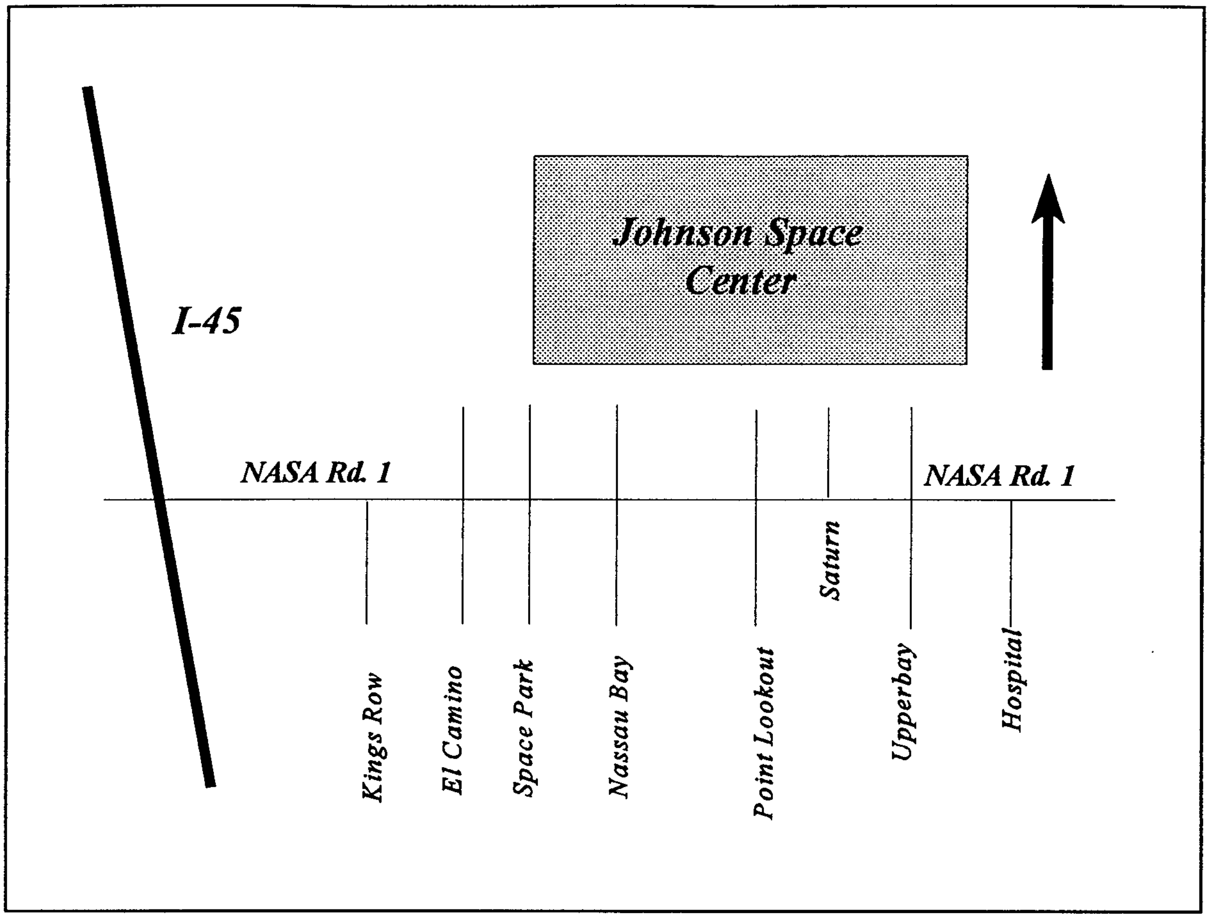

Figure IV-2. Schematic of Nasa Road 1 Study Site.

Traffic signal timing plans for each intersection were also provided by TxDOT. Currently, the traffic signals operate in a time-of-day mode. Timing plans based on 90-, 100-, 110-, 120-, and 140-second cycle lengths have been developed by TxDOT. An outbound, bi-directional, and an inbound offset plan have been developed for each cycle length. These timing plans are contained in Appendix B.

Estimating Delay for Old Timing Plan

The delay was estimated for the signal timings from the "old" timing plan by running the previous timing plan for the present period's traffic conditions. The previous timing plan's cycle length, phase sequencing, splits, and offsets along with the current 15-minute turning movement traffic volumes were entered into PASSER II. PASSER II is a computer program developed at

TTI to assist engineers in optimization of progression along an arterial street. Using the data that was collected for the arterial, PASSER II can determine phase splits and sequencing for each intersection using procedures similar to those in the 1985 Highway Capacity Manual. (*10*) It can also be used to develop estimates of delay for a given set of signal timing parameters (i.e., cycle length, phasing plan, phase durations, and offsets). The output of PASSER II includes the estimate of total vehicular delay. PASSER II estimates vehicular delay in vehicle-hour/hour for the entire system.

Using the simulation mode of PASSER II, estimates of the total vehicular delay in veh-hour/hour for the entire system were developed for the "old" timing plan. These estimate in delay represents Delay $_{Old}$ in Equation IV-1.

Estimating Delay for New Timing Plans

PASSER II was also used to estimate the delay incurred with implementing a new timing plan on the current 15-minute volume. Not every timing plan was considered to be feasible for implementation as a new timing plan. Most traffic responsive systems permit only a one level change from the current cycle length and offset level. Depending upon the cycle length and the offset level of the old timing plan, thé number of timing plans considered as potential new timing plans at each volume level were restricted. For example, if the cycle length from the previous 15-minute period was 120 seconds, only those timing plans one cycle length level up (i.e., 140-seconds) and one cycle length level down (110-seconds) were considered as feasible under the traffic responsive plan. Likewise, if the current offset plan was designed for outbound traffic, only those offset plans one level removed were considered as viable options (i.e., outbound to bi-directional, bi-directional to either outbound or inbound, and inbound to bi-directional). Table IV-1 shows the timing plan numbers that are considered as viable alternatives for each timing plan in the traffic responsive mode. The specific cycle length, phasing, and offset for each intersection under each plan is provided in Appendix B.

Table IV-1. Viable Timing Plans for Implementation in Traffic Responsive Mode.

Current Timing Plan		Permitted Timing Plan Changes			
Cycle Length	Offset Level				
140	Inbound (Plan 20)	140 Bi-directional (Plan 14)	120 Inbound (Plan 19)	-	-
	Bi-directional (Plan 14)	120 Bi-directional (Plan 12)	140 Inbound (Plan 20)	140 Outbound (Plan 8)	-
	Outbound (Plan 8)	140 Bi-directional (Plan 14)	120 Outbound (Plan 19)	-	-
120	Inbound (Plan 19)	120 Bi-directional (Plan 12)	140 Inbound (Plan 20)	110 Inbound (Plan 18)	-
	Bi-directional (Plan 12)	140 Bi-directional (Plan 14)	120 Bi-directional (Plan 12)	120 Inbound (Plan 19)	120 Outbound (Plan 7)
	Outbound (Plan 7)	120 Bi-directional (Plan 12)	140 Outbound (Plan 8)	110 Outbound (Plan 6)	-
110	Inbound (Plan 18)	110 Bi-directional (Plan 11)	120 Inbound (Plan 19)	100 Inbound (Plan 17)	-
	Bi-directional (Plan 11)	120 Bi-directional (Plan 12)	100 Bi-directional (Plan 10)	110 Inbound (Plan 17)	110 Outbound (Plan 6)
	Outbound (Plan 6)	110 Bi-directional (Plan 11)	120 Outbound (Plan 7)	100 Outbound (Plan 5)	-
100	Inbound (Plan 17)	100 Bi-directional (Plan 10)	110 Inbound (Plan 18)	100 Inbound (Plan 6)	-
	Bi-directional (Plan 10)	110 Bi-directional (Plan 11)	90 Bi-directional (Plan 9)	100 Inbound (Plan 16)	100 Outbound (Plan 5)
	Outbound (Plan 5)	100 Bi-directional (Plan 10)	110 Outbound (Plan 6)	90 Outbound (Plan 4)	-
90	Inbound (Plan 16)	90 Bi-directional (Plan 9)	100 Inbound (Plan 17)	-	-
	Bi-directional (Plan 9)	100 Bi-directional (Plan 10)	90 Inbound (Plan 16)	90 Outbound (Plan 4)	-
	Outbound (Plan 4)	90 Bi-directional (Plan 9)	100 Outbound (Plan 5)	-	-

Estimating Delay Due To Transition

The delays associated with transitioning from the previous timing plan to the present optimum timing plan were also computed. SIGTRAN is a computer program developed at TTI that automates the calculations of delay during the transition phase. As stated before, the short-way transition algorithm was used in this research. With this transition method, no more than 18.75 percent of the cycle length was expanded or contracted from the dual mainline green at each intersection to achieve the new coordinated offset. Time was added or subtracted from the dual mainline green depending on which scenario would reach the new offset in the least amount of time.

The call to transition could come at any intersection along the arterial. For this reason, the transition was estimated assuming that each intersection was the first to begin the transition phase. SIGTRAN outputs estimated delay in vehicle-hour/hour. The highest amount of delay was considered to represent the worst case scenario. This estimate in delay represents $Delay_{Transition}$ in Equation IV-1.

Development of Traffic Responsive Timing Plan

Using the above mentioned delay estimates, a traffic responsive timing plan was developed for the A.M. Peak (7:00 to 9:00 A.M.), A.M. Off-Peak (9:30 to 11:30 A.M.), Noon Peak (12:00 to 1:30 P.M.), P.M. Off-Peak (14:00 to 15:30 P.M.), and P.M. Peak (16:15 to 18:00 P.M.). Tables IV-2 through IV-6 show the delays associated with each of the timing plans evaluated during these periods. Using Equation IV-1, the delay associated with keeping the timing plan from the previous 15-minute period was compared to the delay from all the viable timing plan options plus any transition delay that may result from implementing the new plan. The timing plan that resulted in the lowest delay (including the transition delay) was selected to be implemented in a traffic responsive mode. The bold entry in the tables show which timing plans were selected for each 15-minute volume.

Table IV-2. Determination of Traffic Responsive Plan for the A.M. Peak Period.

Time-of-Day	Existing Timing Plan: Timing Plan #	Existing Timing Plan: Total System Delay (veh-hr/hr)	Potential Timing Plans: Timing Plan #	Potential Timing Plans: Total System Delay (veh-hr/hr)	Potential Timing Plans: Transition Delay (veh-hr/hr)	Executed Timing Plan
7:15	-	-	**14**	**341.3**	-	**14**
			8	544.1		
			2	410.9		
7:30	**14**	**908.4**	8	1588.9	–[1]	**14**
			19	2033.2	–[1]	
			20	1053.0	–[1]	
7:45	**14**	**2059.5**	12	2926.5	–[1]	14
			20	2690.3	–[1]	
			8	3423.6	–[1]	
8:00	14	1182.5	8	1653.4	–[1]	20
			12	1361.2	–[1]	
			20	**1083.5**	**18.0**	
8:15	20	1449.1	12	1552.5	–[1]	**14**
			14	**1151.2**	**16.6**	
			8	1954.1	–[1]	
8:30	**14**	**306.6**	12	417.7	–[1]	**14**
			8	479.9	–[1]	
			20	408.5	–[1]	
8:45	**14**	**150.0**	12	167.6	–[1]	**14**
			20	163.7	–[1]	
			8	217.9	–[1]	
9:00	**14**	**124.3**	12	118.7	6.9	**14**
			20	121.6	8.5	
			8	161.9	–[1]	

–[1] Not computed because delay of potential timing plan exceeds delay caused by existing timing plan.

Table IV-3. Determination of Traffic Responsive Plan for the A.M. Off-Peak Period.

Time-of-Day	Existing Timing Plan		Potential Timing Plans			Executed Timing Plan
	Timing Plan #	Total System Delay (veh-hr/hr)	Timing Plan #	Total System Delay (veh-hr/hr)	Transition Delay (veh-hr/hr)	
9:45	-	-	**18** 17 19	**106.6** 135.8 250.0	– – –	**18**
10:00	**18**	**90.6**	17 19 10	90.1 96.9 88.7	 –[1] 6.9	**18**
10:15	**18**	**97.9**	10 11 17	94.7 90.7 95.5	7.5 7.5 6.8	**18**
10:30	**18**	**98.4**	10 11 17	95.4 100.3 97.6	7.4 –[1] 7.0	**18**
10:45	**18**	**112.4**	11 17 19	126.4 118.9 170.5	–[1] –[1] –[1]	18
11:00	18	122.9	17 19 **10**	109.5 129.3 **102.3**	7.7 –[1] 9.0	**10**
11:15	**10**	**143.1**	9 11 17	167.6 154.2 146.1	–[1] –[1] –[1]	**10**
11:30	**10**	**220.8**	9 11 17	303.1 250.6 283.7	–[1] –[1] –[1]	**10**

–[1] Not computed because delay of potential timing plan exceeds delay caused by existing timing plan.

Table IV-4. Determination of Traffic Responsive Plan for the Noon Period.

Time-of-Day	Existing Timing Plan: Timing Plan #	Existing Timing Plan: Total System Delay (veh-hr/hr)	Potential Timing Plans: Timing Plan #	Potential Timing Plans: Total System Delay (veh-hr/hr)	Potential Timing Plans: Transition Delay (veh-hr/hr)	Executed Timing Plan
12:15	-	-	10	291.5	–	**14**
			11	291.9	–	
			12	305.8	–	
			14	**274.9**		
12:30	**14**	**350.0**	9	614.4	–[1]	**14**
			10	374.1	–[1]	
			11	339.8	14.9	
			17	358.3	16.8	
12:45	14	675.5	10	619.0	–[1]	**18**
			11	526.3	18.5	
			12	460.6	13.3	
			18	**442.6**	**12.5**	
13:00	**18**	**238.7**	11	249.2	–[1]	**18**
			17	247.7	–[1]	
			19	304.5	–[1]	
13:15	**18**	**285.9**	11	382.3	–[1]	**18**
			17	334.7	–[1]	
			19	626.7	–[1]	
13:30	**18**	**166.8**	11	202.6	–[1]	**18**
			17	184.2	–[1]	
			19	304.4	–[1]	

–[1] Not computed because delay of potential timing plan exceeds delay caused by existing timing plan.

Table IV-5. Determination of Traffic Responsive Plan for the P.M. Off-Peak Period.

Time-of-Day	Existing Timing Plan		Potential Timing Plans			Executed Timing Plan
	Timing Plan #	Total System Delay (veh-hr/hr)	Timing Plan #	Total System Delay (veh-hr/hr)	Transition Delay (veh-hr/hr)	
14:15	-	-	11 **12** 14 19	217.3 **214.6** 257.3 312.0	– – –	**12**
14:30	**12**	**146.8**	10 11 14 19	141.0 146.8 155.6 162.7	8.5 –[1] –[1] –[1]	**12**
14:45	**12**	**144.8**	10 11 14 19	142.5 144.8 159.8 238.5	9.3 –[1] –[1] –[1]	**12**
15:00	12	117.8	**10** 11 14 17	**107.2** 113.8 134.2 110.2	**7.0** 6.9 –[1] 10.0	**10**
15:15	**10**	**140.3**	9 11 17	303.6 145.0 138.3	–[1] –[1] 7.9	**10**
15:30	**10**	**117.8**	9 11 17	125.3 129.8 146.6	–[1] –[1] –[1]	**10**

–[1] Not computed because delay of potential timing plan exceeds delay caused by existing timing plan.

Table IV-6. Determination of Traffic Responsive Plan for the P.M. Peak Period.

Time-of-Day	Existing Timing Plan		Potential Timing Plans			Executed Timing Plan
	Timing Plan #	Total System Delay (veh-hr/hr)	Timing Plan #	Total System Delay (veh-hr/hr)	Transition Delay (veh-hr/hr)	
16:30	-	-	6 7 **8** 14 19	601.3 668.8 **438.0** 770.5 1347.3	– – – – –	**8**
16:45	8	854.8	**7** 14 20	**805.4** 1897.0 4798.2	**15.6** –[1] –[1]	**7**
17:00	7	531.8	**8** 14 20	**440.9** 929.5 5685.1	**24.0** –[1] –[1]	**8**
17:15	**8**	**1245.4**	7 14 20	3676.8 8056.0 20755.9	–[1] –[1] –[1]	**8**
17:30	**8**	**1115.6**	7 14 20	1235.2 1451.9 5664.1	–[1] –[1] –[1]	**8**
17:45	8	135.0	7 **14** 20	993.5 **905.7** 5418.8	17.1 **18.3** –[1]	**14**
18:00	14	207.8	**7** 8 12	**171.6** 195.2 467.7	**10.8** 12.2 –[1]	**7**

–[1] Not computed because delay of potential timing plan exceeds delay caused by existing timing plan.

FINDINGS

Table IV-7 compare the total system delay (including any transition delay) of the signal system operating in a traffic responsive mode to the total system delay (also including any transition delay) of the signal system operating in a time-of-day mode. The total system delay plus the transition delays were computed for each 15-minute period with the derived traffic responsive plan and the existing time-of-day plan used on Nasa Road 1. Table IV-8 shows the total system delays of the signal system operating in a traffic responsive and time-of-day mode without the effects of the transition delay considered. These system delay and transition delays are contained in Appendix C.

Table IV-7. Comparison of Total System Delay for Traffic Responsive and Time-of-Day Modes for Nasa Road 1.

Period	Total System Delay (veh-hrs)		Delay Savings (veh-hrs)	Percent Reduction
	Time-of-Day	Traffic Responsive		
A.M. Peak	1881.3	1562.3	316.1	16.8%
A.M. Off-Peak	274.6	252.0	22.6	8.2%
Noon	549.1	456.0	93.1	16.9%
P.M. Off-Peak	230.1	224.8	5.3	2.3%
P.M. Peak	1362.8	1352.3	10.5	0.8%

Table IV-8. Comparison of Total System Delay for Traffic Responsive and Time-of-Day Modes without Considering the Effects of Transition.

Period	Total System Delay (veh-hrs)		Delay Savings (veh-hrs)	Percent Reduction
	Time-of-Day	Traffic Responsive		
A.M. Peak	1874.7	1530.6	343.8	18.3%
A.M. Off-Peak	254.6	243.0	11.6	4.5%
Noon	536.2	439.6	96.6	18.0%
P.M. Off-Peak	223.5	217.8	5.7	2.6%
P.M. Peak	1362.8	1283.6	79.2	5.8%

A couple of interesting findings can be generated from these tables. First, in every period, the traffic responsive timing plans produced a delay savings over the time-of-day mode, even when the impacts of transition were incorporated. This was expected since the idea behind using a traffic responsive mode of operation is to match as closely as possible the traffic signal timings with the measured traffic volumes and patterns.

Another interesting finding is that the greatest amount of delay savings occurred during the A.M. Peak and Noon periods. Both Off-Peak periods showed only relatively small reductions in delay while no delay savings were generated in operating the signals in a responsive mode during the P.M. Peak period. A review of the traffic volume and signal timing data show that the greatest changes in the traffic conditions occurred during the A.M. Peak and the Noon periods, and that the largest differences existed between the offset and the phase duration used by the timing plans to accommodate the volume. During the Off-Peak periods, however, only minor changes (i.e., the addition or subtraction of one or two seconds from various phases) exist between timing plans.

One problem encountered during this study was that NASA Road. 1 is oversaturated, with volume to capacity (v/c) ratios greater than 1.0 for much of the analyzed time period. This over saturation leads to overflows in the corridor. Delays become increasingly more difficult to estimate with large v/c ratios as the delay curve increases nearly asymptotically when v/c's are greater than 1.0. For this reason, the v/c ratio was reverted back to the saturation level of v/c equal to 1.0 when estimating the transition delay. This causes a lower estimation of transition delay than actually exists on the system. As a result, a new timing plan was selected more frequently than common practice would dictate. For example, from site observation it is known that arterial is near breakdown during the hours of 4:00 p.m to 5:45 p.m. At this point it would be inappropriate to transition to a new timing plan. However, because of the underestimation in the cost of transitioning at oversaturated conditions, the results indicate changes in timing plan would be appropriate (see Table IV-6).

It is also noted that great care should be taken when managing data during oversaturated conditions. For instance, during the P.M. Peak, vehicle turning movement counts decreased because of the oversaturated condition. Because of this, only a minimal number of vehicles were able to be processed by the signal. The data did not reflect the traffic demand that existed, but instead indicated the number of vehicles the signal was able to process. Impractically small cycle lengths were selected by PASSER II during these time periods because of the low traffic volumes indicated. Common practice says to extend the cycle length to an appropriate length so that more vehicles can be processed by the signal.

Finally, the importance of including the effects of the transition phase in estimating the benefits of changing timing plans can be seen by comparing Tables IV-7 and IV-8. In all but the Off-Peak periods, the delay savings generated by operating the signal system in a traffic responsive mode were substantially higher when the effects of transitioning were not included. In the A.M. Peak and Noon periods, the transition delay reduced the delay savings by about 2 percent. In the A.M. Off-Peak, the transition delay was responsible for approximately half of the total system delay. The transition delay basically offset any benefits of using a traffic responsive mode in the P.M. Peak. These findings illustrate the importance of incorporating the impacts of transition in the timing plan selection process.

CHAPTER V. SUMMARY

Many transportation agencies are in the process of installing closed-loop traffic signal systems. However, most are not fully utilizing the capabilities of these systems. Most closed-loop traffic signal systems are set up to operate in a time-of-day mode, where timing plans change based on the time of day and not changing traffic patterns. Traffic responsive control, however, attempts to optimize the signal settings by monitoring traffic patterns and selecting timing plans that more closely match the measured traffic demands. There is a disadvantage, however, to changing timing plans. Traffic signals must pass through a transition phase every time a timing plan is changed. Limited research has shown that the transition phase can cause significant increases in delay, but these delays are not considered in the process for deciding when to change timing plans in a traffic responsive system.

As part of this research project, a computer program, SIGTRAN, was developed to estimate the impacts of the transition phase when changing timing plans. The program uses the same procedures as PASSER II-90 to compute the estimated delay due to transition. Like PASSER II-90, delay estimates are only valid when the volume-to-capacity ratio is 1.0 or less.

Using the SIGTRAN program, a proposed algorithm for selecting new timing plans in a traffic responsive mode was developed. The algorithm compares the delay associated with retaining the old traffic signal timing to the delay associated the implementing a new timing plan plus the delay caused by transitioning to the new timing plan. If the delay for the old timing plan exceeded the delay estimates for the new timing plan plus the delay accrued during transition, then the new timing plan was adopted; otherwise, the old timing plan was retained.

The new algorithm was tested using data from NASA Road 1 in Houston, Texas. It was found that the traffic responsive timing plans developed using the algorithm produced substantial delay savings in the A.M. Peak and Noon periods over operating the signals in a time-of-day mode. Less significant delay savings were generated during the A.M. Off-Peak, P.M. Off-Peak, and P.M. Peak periods. During these periods, either traffic volumes did not change significantly or the differences in the timing plans were so slight that most of the benefits of changing timing plans were offset by the increase in delays caused by transitioning to the new timing plans.

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APPENDIX A: Example of Input File for SIGTRAN

```
8
  00.00   13.24
  13.24   20.77
  20.77   12.78
  12.78   18.51
  18.51   19.48
  19.48   14.22
  14.22   13.64
  13.64
  64.00 2068.00    4.00    0.00    0.00 1300.00    0.00  124.00
1710.00 3423.00 1710.00    0.00    0.00 3429.00    0.00 1453.00
 152.00 1616.00  232.00  328.00   84.00  904.00  500.00 1212.00
3155.00 4864.00 1710.00 3309.00 3155.00 4753.00 1710.00 3224.00
  44.00 2752.00    0.00   12.00    0.00 1116.00    0.00   64.00
1710.00 3394.00    0.00 1710.00    0.00 3429.00    0.00 3372.00
  32.00 2340.00    0.00   36.00    0.00 1092.00    0.00  180.00
1710.00 3366.00    0.00 1710.00    0.00 3427.00    0.00 3409.00
 132.00 2728.00  120.00    4.00   40.00 1408.00    1.00  104.00
1710.00 3366.00 1710.00 3172.00 3155.00 3413.00 1710.00 2915.00
   0.00 1060.00    0.00   88.00  196.00  984.00  124.00    0.00
   0.00 3429.00    0.00 2915.00 3155.00 4841.00 1710.00    0.00
  76.00 1296.00  140.00   60.00  372.00  920.00   16.00  188.00
1710.00 3315.00 1710.00 3187.00 3155.00 3403.00 1710.00 1764.00
   0.00 1272.00   84.00    0.00    0.00 1708.00    0.00   64.00
1710.00 3252.00 1710.00    0.00    0.00 3429.00    0.00 1530.00
   0.00
26 16 38
 105.00   17.00   18.00
  12.60
25 26 16 38 47
  27.00   25.00   22.00   35.00   31.00
 119.00
16 26 48
  15.00   94.00   31.00
 131.60
16 26 47
  15.00  101.00   24.00
  63.00
16 26 25 47 38
  24.00   66.00   15.00   15.00   20.00
 116.20
25 26 47
  22.00   94.00   24.00
 138.60
25 26 16 47 38
  18.00   59.00   20.00   15.00   28.00
 137.20
16 26 38
  15.00   95.00   30.00
   0.00
26 16 38
 1  0  0
 110.00   15.00   15.00
  11.20
25 26 16 47 38
 0  1  0  0  0
  15.00   44.00   18.00   31.00   32.00
 109.20
16 26 48
 0  1  0
  15.00  110.00   15.00
 126.00
16 26 47
 0  1  0
  15.00  108.00   17.00
  49.00
16 26 25 47 38
 0  1  0  0  0
```

```
  15.00   76.00   16.00   15.00   18.00
 106.40
25 26 47
 0  1  0
  23.00  102.00   15.00
 124.60
25 26 16 47 38
 0  1  0  0  0
  27.00   45.00   28.00   15.00   25.00
 107.80
16 26 38
 0  1  0
  29.00   90.00   21.00
```

APPENDIX B: Timing Plans Used on Nasa Road 1

Plan Number	Cycle Length	Intersct.	Mvmt	Dur	Mvmt	Dur	Mvmt	Dur	Mvm	Dur	Mvmt	Dur	Offset
1	80	1	1+6	14	2+6	51	3+8	15					60
		2	2+5	14	2+6	14	1+6	14	3+8	19	4+7	19	0
		3	1+6	15	2+6	51	4+8	14					21.6
		4	1+6	12	2+6	54	4+8	14					37.6
		5	1+6	14	2+6	23	2+5	14	4+7	15	3+8	14	16
		6	2+5	15	2+6	50	4+7	15					44
		7	2+5	14	2+6	23	1+6	14	4+7	15	3+8	14	54.4
		8	1+6	15	2+6	40	3+8	25					54.4
4	90	1	2+6	62	1+6	13	3+8	15					0
		2	2+5	15	2+6	25	1+6	15	3+8	20	4+7	15	12.6
		3	1+6	13	2+6	58	4+8	19					0
		4	1+6	14	2+6	58	4+8	18					11.7
		5	1+6	15	2+6	26	2+5	15	4+7	15	3+8	19	16.2
		6	2+5	15	2+6	60	4+7	15					47.7
		7	2+5	15	2+6	25	1+6	15	4+7	15	3+8	20	57.6
		8	1+6	15	2+6	55	3+8	20					52.2
5	100	1	2+6	72	1+6	13	3+8	15					0
		2	2+5	16	2+6	29	1+6	16	3+8	23	4+7	16	5
		3	1+6	13	2+6	66	4+8	21					96
		4	1+6	14	2+6	68	4+8	18					86
		5	1+6	18	2+6	37	2+5	15	4+7	15	3+8	15	86
		6	2+5	17	2+6	68	4+7	15					23
		7	2+5	15	2+6	33	1+6	18	4+7	15	3+8	19	29
		8	1+6	15	2+6	63	3+8	22					0
6	110	1	2+6	80	1+6	15	3+8	15					0
		2	2+5	19	2+6	27	1+6	17	3+8	26	4+7	21	2.2
		3	1+6	13	2+6	75	4+8	22					93.5
		4	1+6	13	2+6	75	4+8	22					92.4
		5	1+6	17	2+6	43	2+5	15	4+7	15	3+8	20	97.9
		6	2+5	18	2+6	76	4+7	16					33
		7	2+5	18	2+6	36	1+6	17	4+7	15	3+8	24	47.3
		8	1+6	15	2+6	71	3+8	24					40.7
7	120	1	2+6	90	1+6	14	3+8	16					0
		2	2+5	22	2+6	27	1+6	18	3+8	30	4+7	23	12
		3	1+6	14	2+6	77	4+8	29					112.8
		4	1+6	14	2+6	82	4+8	24					115.2
		5	1+6	19	2+6	57	2+5	14	4+7	14	3+8	16	66
		6	2+5	16	2+6	84	4+7	20					3.6
		7	2+5	16	2+6	44	1+6	14	4+7	20	3+8	26	18
		8	1+6	18	2+6	74	3+8	28					1.2
8	140	1	2+6	104	1+6	19	3+8	17					0
		2	2+5	17	2+6	32	1+6	35	3+8	31	4+7	25	11.2
		3	1+6	15	2+6	91	4+8	34					131.6
		4	1+6	15	2+6	83	4+8	42					135.8
		5	1+6	18	2+6	72	2+5	15	4+7	15	3+8	20	42
		6	2+5	15	2+6	98	4+7	27					98
		7	2+5	15	2+6	49	1+6	28	4+7	20	3+8	28	119
		8	1+6	15	2+6	95	3+8	30					114.8

Plan Number	Cycle Length	Intersct.	Mvmt	Dur	Mvmt	Dur	Mvmt	Dur	Mvm	Dur	Mvmt	Dur	Offset
9	90	1	2+6	62	1+6	13	3+8	15					0
		2	2+5	15	2+6	26	1+6	15	3+8	15	4+7	19	6.3
		3	1+6	14	2+6	61	4+8	15					0
		4	1+6	14	2+6	61	4+8	15					0
		5	1+6	15	2+6	30	2+5	15	4+7	15	3+8	15	15.3
		6	2+5	15	2+6	60	4+7	15					43.2
		7	2+5	15	2+6	24	1+6	15	4+7	15	3+8	21	54.9
		8	1+6	15	2+6	50	3+8	25					49.5
10	100	1	2+6	70	1+6	15	3+8	15					0
		2	2+5	16	2+6	24	1+6	16	3+8	22	4+7	22	10
		3	1+6	15	2+6	68	4+8	17					84
		4	1+6	15	2+6	69	4+8	16					94
		5	1+6	15	2+6	40	2+5	15	4+7	15	3+8	15	60
		6	2+5	17	2+6	68	4+7	15					96
		7	2+5	15	2+6	33	1+6	15	4+7	15	3+8	22	9
		8	1+6	15	2+6	60	3+8	25					3
11	110	1	2+6	80	1+6	15	3+8	15					0
		2	2+5	17	2+6	23	1+6	20	3+8	26	4+7	24	9.9
		3	1+6	16	2+6	76	4+8	18					88
		4	1+6	15	2+6	77	4+8	18					94.6
		5	1+6	18	2+6	48	2+5	15	4+7	14	3+8	15	31.9
		6	2+5	18	2+6	71	4+7	21					81.4
		7	2+5	15	2+6	44	1+6	16	4+7	15	3+8	20	91.3
		8	1+6	18	2+6	70	3+8	22					84.7
12	120	1	2+6	86	1+6	16	3+8	18					0
		2	2+5	14	2+6	32	1+6	21	3+8	27	4+7	26	9.6
		3	1+6	14	2+6	88	4+8	18					102
		4	1+6	14	2+6	89	4+8	17					110.4
		5	1+6	16	2+6	57	2+5	14	4+7	14	3+8	19	60
		6	2+5	18	2+6	78	4+7	24					115.2
		7	2+5	14	2+6	51	1+6	14	4+7	16	3+8	25	12
		8	1+6	19	2+6	76	3+8	25					2.4
14	140	1	2+6	105	1+6	17	3+8	18					0
		2	2+5	27	2+6	25	1+6	22	3+8	35	4+7	31	12.6
		3	1+6	15	2+6	94	4+8	31					119
		4	1+6	15	2+6	101	4+8	24					131.6
		5	1+6	24	2+6	66	2+5	15	4+7	15	3+8	20	63
		6	2+5	22	2+6	94	4+7	24					116.2
		7	2+5	18	2+6	59	1+6	20	4+7	15	3+8	28	138.6
		8	1+6	15	2+6	95	3+8	30					137.2
16	90	1	2+6	62	1+6	13	3+8	15					0
		2	2+5	15	2+6	19	1+6	15	3+8	21	4+7	20	6.3
		3	1+6	14	2+6	61	4+8	15					5.4
		4	1+6	14	2+6	61	4+8	15					7.2
		5	1+6	15	2+6	30	2+5	15	4+7	15	3+8	15	16.2
		6	2+5	17	2+6	58	4+7	15					50.4
		7	2+5	16	2+6	29	1+6	15	4+7	15	3+8	15	72
		8	1+6	15	2+6	55	3+8	20					68.4

Plan Number	Cycle Length	Intersct.	Mvmt	Dur	Mvmt	Dur	Mvmt	Dur	Mvm	Dur	Mvmt	Dur	Offset
17	100	1	2+6	70	1+6	15	3+8	15					0
		2	2+5	16	2+6	19	1+6	17	3+8	24	4+7	24	10
		3	1+6	15	2+6	69	4+8	16					40
		4	1+6	15	2+6	69	4+8	16					40
		5	1+6	15	2+6	40	2+5	15	4+7	15	3+8	15	80
		6	2+5	15	2+6	70	4+7	15					25
		7	2+5	15	2+6	35	1+6	15	4+7	15	3+8	20	40
		8	1+6	15	2+6	65	3+8	20					40
18	110	1	2+6	80	1+6	15	3+8	15					0
		2	2+5	17	2+6	23	1+6	15	3+8	29	4+7	26	9.9
		3	1+6	15	2+6	79	4+8	16					101.2
		4	1+6	15	2+6	78	4+8	17					105.6
		5	1+6	15	2+6	48	2+5	15	4+7	16	3+8	16	46.2
		6	2+5	17	2+6	77	4+7	17					96.8
		7	2+5	15	2+6	47	1+6	16	4+7	16	3+8	16	1.1
		8	1+6	18	2+6	71	3+8	21					0
19	120	1	2+6	90	1+6	14	3+8	16					0
		2	2+5	17	2+6	38	1+6	18	3+8	23	4+7	24	12
		3	1+6	16	2+6	89	4+8	15					97.2
		4	1+6	15	2+6	86	4+8	19					104.4
		5	1+6	16	2+6	59	2+5	16	4+7	14	3+8	15	46.8
		6	2+5	18	2+6	87	4+7	15					96
		7	2+5	24	2+6	44	1+6	22	4+7	14	3+8	16	108
		8	1+6	25	2+6	75	3+8	20					99.6
20	140	1	2+6	110	1+6	15	3+8	15					0
		2	2+5	15	2+6	44	1+6	18	3+8	31	4+7	32	11.2
		3	1+6	15	2+6	110	4+8	15					109.2
		4	1+6	15	2+6	108	4+8	17					126
		5	1+6	15	2+6	76	2+5	16	4+7	15	3+8	18	49
		6	2+5	23	2+6	102	4+7	15					106.4
		7	2+5	27	2+6	45	1+6	28	4+7	15	3+8	25	124.6
		8	1+6	29	2+6	90	3+8	21					107.8

APPENDIX C. Comparison of Time-of-Day and Traffic Responsive Delays

Table C-1. Comparison of Time-of-Day and Traffic Responsive Plans in the A.M. Peak.

Time-of-Day	Time-of-Day Mode			Traffic Responsive Mode		
	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)
7:15	20	102.7	0	14	85.3	0
7:30	20	263.3	0	14	227.1	0
7:45	20	672.6	0	14	514.9	0
8:00	20	270.9	0	20	270.9	18.0
8:15	20	362.3	0	14	287.2	16.6
8:30	20	102.1	0	14	76.6	0
8:45	19	62.7	6.6	14	37.5	0
9:00	19	38.1	0	14	31.1	0
Total		1874.7	6.6		1530.6	34.6

Table C-2. Comparison of Time-of-Day and Traffic Responsive Plans in the A.M. Off-Peak.

Time-of-Day	Time-of-Day Mode			Traffic Responsive Mode		
	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)
9:45	18	26.6	0	18	26.6	0
10:00	18	22.6	0	18	22.6	0
10:15	17	23.9	6.8	18	24.5	0
10:30	17	24.4	0	18	24.6	0
10:45	17	29.7	0	18	28.1	0
11:00	17	27.3	0	10	25.6	9.0
11:15	7	45.4	13.2	10	35.8	0
11:30	7	54.7	0	10	55.2	0
Total		254.6	20		243	9

Table C-3. Comparison of Time-of-Day and Traffic Responsive Plans in the Noon Peak.

Time-of-Day	Time-of-Day Mode			Traffic Responsive Mode		
	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)
12:15	14	68.7	0	14	68.7	0
12:30	14	87.5	0	14	87.5	0
12:45	14	168.9	0	18	110.6	16.4
13:00	14	64.9	0	18	59.6	0
13:15	11	95.6	12.9	18	71.5	0
13:30	11	50.6	0	18	41.7	0
Total		536.2	12.9		439.6	16.4

Table C-4. Comparison of Time-of-Day and Traffic Responsive Plans in the P.M. Off-Peak.

Time-of-Day	Time-of-Day Mode			Traffic Responsive Mode		
	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)
14:15	10	59.6	0	12	53.6	0
14:30	10	35.2	0	12	36.7	0
14:45	10	35.6	0	12	36.2	0
15:00	10	26.8	0	10	26.8	7.0
15:15	12	35.5	6.6	10	35.1	0
15:30	12	30.8	0	10	29.4	0
Total		223.5	6.6		217.8	7

Table C-5. Comparison of Time-of-Day and Traffic Responsive Plans in the P.M. Peak.

Time-of-Day	Time-of-Day Mode			Traffic Responsive Mode		
	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)	Plan #	Total System Delay (veh-hr)	Transition Delay (veh-hr)
16:30	8	109.5	0	8	109.5	0
16:45	8	211.4	0	7	201.3	15.6
17:00	8	110.2	0	8	110.29	24.0
17:15	8	311.4	0	8	311.4	0
17:30	8	278.9	0	8	278.9	0
17:45	8	292.6	0	14	229.4	18.3
18:00	8	48.8	0	7	42.9	10.8
Total		1362.8	0		1283.69	68.7