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By

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ABSTRACT

ANALYSIS OF ADAPTIVE SIGNAL CONTROL STRATEGIES IN URBAN CORRIDORS

By

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The objective of this study was to determine the merits of alternative signal control strategies and to quantify the magnitude of the incremental benefits that might be achieved through changing the control of traffic signals in urban corridors from fixed-time or actuated control to more advanced computer-based adaptive control systems. Two adaptive signal control strategies, with two different techniques for traffic profile predictions, were examined in the study. The analysis was conducted using simulation modeling of a hypothetical corridor with traffic profiles representing moderate and high peak-hour conditions. A real-time adaptive control algorithm was incorporated with the CORSIM simulation model. The algorithm provided real-time adaptive control for the traffic signals along the corridor based on detector data exchanged with the simulation model.

The effectiveness of different signal control strategies was examined using the same platform. The output of the simulation model showed that adaptive control strategies resulted in a reduction in average intersection delay and corridor travel time over optimized fixed-time signals. The reduction was higher during the non-peak periods

and decreased as traffic demand reached its peak with average saving in total travel time ranging from 1.89% to 5.90%. When compared with the coordinated actuated signals, neither of the two adaptive control strategies showed a significant difference in corridor travel time or intersection delay parameters. The saving in corridor travel time under adaptive signal control system is more sensitive to the PHF of the minor-street traffic than any other traffic parameter. There was no significant difference in the performance of the two adaptive control systems that used different traffic-profile prediction techniques.

A field study to examine the effectiveness of deploying the SCATS adaptive control system in the Orchard Lake Road corridor showed that the reduction in corridor travel time achieved under SCATS control was within the benefit limits predicted by the simulation model. The results also showed that, similar to what the simulation model output demonstrated, coordinated actuated signals might have achieved a similar reduction in travel time.

This research has contributed to the understanding of the potential benefits of adaptive signal control strategies in urban corridors. It also identifies several areas where there is a need for further research that can, ultimately, lead to a set of rules and guidelines for choosing among different signal control strategies within an ITS context.

This work is dedicated to my parents, my wife, and my two children

Who all dream to witness my accomplishment.

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Chapter 1

INTRODUCTION

1.1 Description of the Problem

In 1991 the Federal Highway Administration (FHWA) issued a solicitation for the development and evaluation of a Real-Time TRaffic Adaptive Control System (RT-TRACS) suitable for use in an Intelligent Transportation Systems (ITS) environment. The FHWA ITS program consisted of research and operational tests designed to combat traffic congestion. The thrust of the program was to develop and implement the technology necessary to mitigate the effects of congestion by providing more efficient transportation facilities. Some of the elements included in ITS are transportation management, in-vehicle route guidance systems, integration of multi-modal transportation, integration of surface streets and freeway networks, and incident management. An adaptive traffic control system, together with a surveillance system that provides real time data, are fundamental to integrate different ITS elements. This adaptive control needs to include forecasting capabilities such that proactive, not reactive, traffic control is provided.

With the anticipated rapid development of ITS, considerable potential exists to change the control of many of the nation's traffic signals, primarily controlled at present in fixed-time or actuated modes, to more efficient on-line advanced adaptive control (Davies, 1991). Thus, there is a need to determine the merits of alternative signal control strategies and to precisely quantify the magnitude of the incremental benefits that might be achieved through changing to adaptive control strategies. While the potential benefits of these adaptive control strategies are thought to be significant, few research studies have compared the effect of adaptive control against alternative control strategies. The objective of this research is to examine the benefits of deploying an adaptive signal control system on an arterial street in an urban traffic network.

1.2 Evolution of adaptive signal control strategies

After the introduction of computer-based traffic signal control systems in the 1960s, numerous experiments were conducted to develop more responsive control strategies. One of the most comprehensive studies was the Urban Traffic Control System (UTCS) experiment in the 1970s by the FHWA. The UTCS project was directed toward developing and testing a variety of advanced network control concepts and strategies and lasted for almost a decade, (Henry, Ferlis, and Kay, 1976). Research and testing in the UTCS project were divided into three generations. Table 1.1 presents the key features for the three generations of control.

Table 1.1 Key features for the UTCS three generations of control strategies

Feature	First Generation	Second Generation	Third Generation
Optimization	Off-line	On-line	On-line
Frequency of update	15 minutes	5 minutes	3-6 minutes
No. of timing patterns	up to 40	Unlimited	Unlimited
Traffic predictions	No	Yes	Yes
Critical Intersection Control (CIC)	Adjust Split	Adjust splits and offsets	Adjust Split, offset, and cycle
Hierarchies of control	Pattern Selection	Computation	Computation
Fixed cycle length	Yes	Within group of intersections	No fixed cycle length

First-Generation Control (I-GC) uses pre-stored signal timing plans that are calculated off-line and based on historical traffic data. The plan controlling the traffic system can be selected on the basis of time of day, by direct operator selection, or by matching from the existing library a plan best suited to traffic conditions (volumes and occupancies). This is called the traffic responsive (TRSP) mode of plan selection. Plans in 1-GC can be determined by any off-line signal optimization method. The operator determines the mode of plan selection with a frequency of update in the traffic-responsive mode of 15 minutes. The I-GC also included logic to enable a smooth transition between different signal timing plans and a critical intersection control (CIC) feature, that enables vehicle-actuated adjustment of green splits at selected signals.

The Second-Generation Control (2-GC) is an on-line strategy that computes in real-time and implements signal-timing plans based on surveillance data and predicted values. The optimization process can be repeated at 5-minute intervals; however, to avoid transition disturbances, new timing plans cannot be implemented more often than once every 10 minutes. The 2-GC software contains an optimization algorithm, a traffic prediction model, sub-network configuration models, critical intersection control, and a transition model to minimize transition time between two plans.

Finally, the Third-Generation Control (3-GC) strategy was designed to implement and evaluate a fully responsive, on-line traffic control system. Similar to 2-GC, it computed control plans to minimize a network-wide objective using predicted traffic conditions for input. The differences with 3-GC were that the time period for revising timing plans was shortened to 3 to 5 minutes and the cycle length was allowed to vary among the signals as well as the same signal during the control period. The switching points within each control interval were also determined by the on-line optimization procedure. It is noteworthy that 3-GC is similar in concept to 2-GC. Because of the inherent inaccuracies in the measurement-prediction cycle, neither the 2-GC nor the 3-GC strategies could respond adequately to rapid changes in traffic flows. Table 1.2 presents the characteristics of different generations of signal control strategies.

Table 1.2 Characteristics of the UTCS control strategies

	Generation of Control			
Benefits	Fixed-Time	First	Second	Third
+ Signal coordination				
+ Change timing easily				
+ Monitor performance and equipment status				
+ Data collection for future policies				
+ Responsive to major variation				
+Responsive to all variation				
+ Detector needs	None	Low	Moderate	High

[Source: The Urban Traffic Control System in Washington, D.C., U.S. Department of Transportation, Federal Highway Administration, 1974]

A significant advance toward more effective on-line adaptive control was achieved during the 1980s with the introduction of Split, Cycle and Offset Optimization Technique (SCOOT) in the United Kingdom and Sydney Coordinated Adaptive Traffic System (SCATS) in Australia. SCATS is an example of first generation UTCS control strategy (Lowrie, 1982). It updates signal-timing parameters based on changes on traffic demand from a library of pre-defined plans. SCOOT, however, uses an on-line optimization technique and may be considered a 2-GC strategy (Bretherton, 1990).

1.3 Current practices of signal control strategies

There are several levels of sophistication for signal control systems. These range from fixed time plans selected by time-of-day through dynamic selection of fixed time plans to full dynamic selection of cycle length, phase splits, and offsets. The various signal control systems for urban arterial street networks currently in operation fall into three broad categories; fixed-time control with plans selected by time-of-day, semi-actuated or full-actuated control, and on-line dynamic adaptive control.

Fixed-time control is by far the simplest type of control, as it involves the pre-calculation of signal timing plans for the entire control period. Typically, fixed-time plans are optimized off-line based on average historical arrival rates, and then implemented in the signal controller. They, therefore, have no ability to react to changes in traffic patterns. However, as traffic volumes, except at capacity, are seldom constant for an entire control period, this type of control is optimal for traffic conditions that, while average, may still only occur for brief periods during each control period, if at all. At all other times, the fixed-time signal plan may not be optimal. This limitation can be overcome by implementing time-of-day control where the timing plan is changed automatically at certain times of the day based on expected temporal traffic peaking trends. This control strategy, other than during transition periods, maintains the simplicity of fixed-time control.

The main drawback to this type of control is its inability to react to unexpected deviations from historical trends, such as diversions arising from incidents, or simple day-to-day random variations of the timing and the severity of the peaks. In addition, even for predicted traffic conditions, there are only a finite number of time-of-day plans that can be handled by current controllers. Therefore, during periods of build up or decay of the peak, the selected time-of-day plan may still not be optimal. These two limitations spurred the development of adaptive control algorithms. These algorithms, at least in theory, should provide optimal solutions for all conditions.

Actuated signals allocate the green time for each phase on a cycle-by-cycle basis. The basic characteristic of actuated control is that the cycle length and phase splits, and even phase sequence, may vary from cycle to cycle in response to detector actuation. Detectors, placed upstream from the intersection, provide the actuated controller with information concerning current demand. Signal actuation, accordingly, permits phases to start early or to end late to meet the variation in demand. This flexibility should make it possible to improve the performance of the control strategy.

In urban corridors, where intersections are relatively closely spaced, signals, operating in fixed-time or actuated modes, can be coordinated to provide progression for vehicles traveling through the corridor. In this case, all signals along the corridor should operate in a common cycle-length to maintain offsets and progression patterns.

While coordinating actuated signals would deprive the controller of most of its ability to vary cycle length to meet demand, they can improve the signal operation by reallocating green time among approaches to meet the cycle-by-cycle variation in demand.

It should be noted that none of the above control strategies has the ability to dynamically change the phase structure (number, type, and order of phases). Currently, this decision must still be optimized off-line in most controllers, and is then held fixed for a specified duration. In actuated signals, however, some phases might be skipped if there is no demand to activate the actuated controller. Current practice is to use non-actuated signal control plans if the arrival pattern is predictable, such as when networks are heavily congested and intersections are closely spaced. Actuated control is used instead when arrival patterns are less predictable, such as during light traffic conditions. Current practice also recommends that signals spaced half a mile or less should be coordinated (Skabardonis, 1997)

Several traffic-responsive signal control strategies were developed for intersections furnished with traffic detectors. The British SCOOT system is simply an on-line version of the TRAfic Network StuY Tool (TRANSYT). The control strategy is to minimize a function defined by vehicle delay and stops at all intersection in the network in all ranges of demand. Signal splits change incrementally based on current demand obtained from detectors every four seconds. Offsets and cycle length are adjusted every few minutes (Robenson, 1991). SCATS of Australia has different control strategies for various demand levels. Signal plans are selected from embedded plans developed off-line, while the cycle length is calculated every cycle. The system also makes use of some local controls to accommodate local traffic conditions at each intersection (Rathi, 1992).

In the 1990s, several adaptive systems were developed and implemented in the United States. An example of these systems is the Optimized Policies for Adaptive Control (OPAC), developed by Gartner (1983 and 1991). OPAC uses a mathematical optimization of multistage decision process to dynamically optimize the signal setting. The latest version of OPAC (version 3.0) incorporated major enhancements that are important for efficient signal operation. The enhancements included phase skipping, optimization of all phases, platoon identification, and modeling algorithm that provides a coordination mechanism for coordinated signals. OPAC was implemented in New Jersey in 1996 (Andrews, et. al., 1997).

1.4 Statement of the problem

The benefits of computer-based adaptive signal control systems are not yet fully understood. The deployment of several adaptive signal control strategies have led to a mixed perception of their benefits. Field evaluations of the deployed systems reported different, sometimes contradictory results. Thus, there is a need for research that quantify the incremental benefits achieved from changing the control of an existing signal system to adaptive control. There is also a need to examine the different factors that might impact the benefits of these systems. The research conducted in this study is original because it tests the benefits of deploying adaptive signal control strategies in an urban corridor. The performance of alternative signal control strategies was compared under the same environment using simulation modeling.

1.5 Objectives and scope of the research

The research addresses the potential benefits of adaptive signal control strategies for urban corridors. The benefits of adaptive signal control strategies were tested against fixed-time and actuated signals under different demand levels. The scope of the research includes:

1. identification of measures of effectiveness (MOEs),
2. development of adaptive control strategies and tests of their effectiveness,
3. determination of the limits of effectiveness of these control strategies with respect to demand and peak-hour factors,
4. identify which characteristics of the adaptive control logic contributed to the changes in traffic parameters, and
5. evaluate the effectiveness of different prediction techniques used in the adaptive control strategies.

1.6 Research approach

This research was based on traffic simulation. Simulation models permit the comparison of different signal control strategies under the same road network and traffic conditions. CORSIM (CORridor SIMulation), a microscopic interval-based simulation model, was selected for this study. It has run-time extension capability that allows external programs to interact with CORSIM and exchange data with it while it is executing. This feature allows the user to replicate adaptive signal control logic within the model. CORSIM version 1.04, which was used in this study, includes many advanced features such as run-time extension capability, traffic surveillance system, and new enhanced actuated control logic. CORSIM can provide data on the MOEs suitable for the analysis (Federal Highway Administration, 1998).

The research was based on a hypothetical corridor with demand characteristics representative of traffic conditions on the Orchard Lake corridor, an urban corridor in Oakland County, Michigan. The effectiveness of adaptive control strategies was compared with two conventional corridor signal control strategies namely: coordinated fixed-time and coordinated actuated signals. Two adaptive control strategies were developed and examined in the study. Both adaptive systems attempt to optimize the signal setting by equalizing the degree of saturation in competing approaches. The difference between the two lies in the way traffic profiles are predicted. In the first adaptive logic, traffic predictions are made based on detectors placed downstream from the intersection. Predictions for the traffic profile in any given cycle were based on the volumes of previous cycles. For the second adaptive control strategy, predictions were made based on detectors placed upstream from the intersection by using a "scan ahead" technique that predicts the arrival of vehicles to the intersection based on estimated time of travel of the platoon from the detector location to the intersection.

The study is organized into five parts. In the first part, several MOEs were used to compare the effectiveness of the four signal control strategies under two different demand levels representing moderate and high peak-hour conditions, respectively. Having examined the potential benefits of the adaptive control logic, the second part of the study was a signal timing analysis to examine which characteristics of the adaptive control logic contributed to the changes in traffic parameters. The third part of the study was a sensitivity analysis to examine the benefits of adaptive control strategies under different demand levels and peak-hour factors. The fourth part of the study aimed to compare the effectiveness of the two prediction techniques used in the adaptive control strategies. Finally, a field study aimed to validate the output of the simulation model was performed. A comparison of the cost and benefits of different signal control strategies was also illustrated in the field study.

1.7 Structure of the dissertation

The dissertation is organized into seven chapters. A brief review of the existing literature (Chapter 2) follows the introduction. The literature review aims at familiarizing the reader with current research efforts in the area of signal control strategies. Chapter 3 contains a detailed description of the simulation model development. It also includes details of different signal control strategies examined in the study.

Chapter 4 contains a detailed description of the design of the study, including a discussion of different MOEs used in the analysis, processing the simulation model output, corridor geometry, and traffic demand levels. The output of the simulation analysis is presented in chapter 5. Chapter 6 includes the details of the field study aimed to validate the output of the simulation model. Finally chapter 7 summarizes the conclusions and suggestions for future research topics.

Chapter 2

LITERATURE REVIEW

A literature search was performed to accomplish four primary goals. First, the current limits of knowledge in the field of traffic engineering relative to signal control strategies were explored. Second, an understanding of the research gaps in this area, including the absence of studies performed specifically on the comparative benefits gained from adaptive signal control were identified. The literature review also established a base of knowledge from which to launch the proposed study and demonstrated certain techniques that have been successful in past research. Finally, the review of past published literature gave insights into the way in which current theories, technology, and implementation of adaptive traffic signal systems have developed over the years.

2.1 Optimization of traffic signal control systems

The first generation of signal control strategies was based on off-line calculations for a fixed-time signal control system. Webster and Cobbe (1958) suggest that for an isolated signal, a signal split strategy should equalize the degree of saturation (DS) of all critical approaches to approximately yield the minimum overall intersection delay. This technique became general practice for most fixed-time control.

For arterial and network considerations, where throughput is more important than overall delay, Little (1966) introduced an off-line mathematical technique to maximize the through bandwidth. The principle was to maximize the number of vehicles able to successfully encounter green signals when traveling along a street. Over the years, several variations of this approach were developed. NCHRP Report 73 (1969) evaluated several of these offset strategies. Off-line control techniques investigated in this report were Yardeni's time-space design, Little's maximal bandwidth, and delay/difference-of-offset. Three responsive control strategies, namely basic queue control, cycle and offset selection, and mixed cycle mode, were also evaluated in this report. The results indicate that the cycle and offset selection method and delay/difference-of-offsets techniques rank the best for off-peak periods, whereas the mixed cycle mode and basic queue control were the best for peak periods.

Perhaps the most comprehensive and most widely applied control strategy for fixed-time control setting on an arterial is based on a computer optimization method. The TRANSYT method of delay minimization is a popular method to determine signal timings (Woods, 1993). TRANSYT (Robertson, 1968) is an off-line program utilizing a modified Webster's method to calculate green splits, and a hill-climbing optimization technique to determine the offset and cycle length which minimizes a performance index. The logic for the offset calculation is similar to the delay/difference-of-offset method evaluated in NCHRP Report 73. Similar programs to TRANSYT are SIGOP (Signal Optimization program) and PASSER II (Progression Analysis and Signal System Evaluation Routine), which have different optimization procedures. The control strategies obtained from off-line calculations are effective for average traffic conditions, but are not responsive to changing traffic patterns. When a traffic pattern changes, the solutions from these programs are no longer optimal.

Several traffic-responsive signal control strategies were developed for an individual intersection furnished with traffic detectors. Gazis and Potts (1964) developed a technique for time-dependent signal setting under varying demand. They used queue length as an input to minimize total aggregate intersection delay. The technique is also called "bang-bang" because the green time is set at a predetermined maximum value for the queued approach, and at a minimum value in other directions. When a queue in the first approach is cleared, the setting is reversed. This signal setting does not, in general, minimize the period during which one approach is congested. D'Ans and Gazis (1976) furthered this control method by means of linear programming. Church and Revelle (1978) formulated similar control strategies with consideration of maximum waiting time and queue length. When the maximum queue length was used as a control objective, they found that the solution tended to balance the queue lengths on the most saturated approaches of each signal phase, and the signal frequently switched between phases. Michalopoulos and Stephanopolous (1977) reported that the queue constraint was effective when the demand increases to the limiting value and that the optimal control strategy at saturation is simply the balance of input-output to maintain constant queue length.

The results of testing four different intersection control strategies, namely basic queue control, queue-length/arrival rate control, modified space-presence control, and delay-equalization control are presented in the NCHRP Report 32 (1967). The results showed that the modified space-presence control strategy yielded the lowest delay under low to medium intersection demand (up to 2000 vehicles per hour for 4-lane, 4-leg junction). When the demand was greater than 2000 vehicles per hour, the basic queue control strategy was better than the others.

Many control strategies have been developed for oversaturated traffic conditions at an isolated intersection. Gordon (1969) suggested that the control objective should be to maintain a constant ratio among the respective storage spaces. Longley (1968) attempted to balance queue lengths on all approaches. NCHRP Report 194 (1976) showed that, although the Longley control logic yielded lower delay than the off-line calculation, the queue-actuated control resulted in lower delay when the degree of saturation was above 0.5. The report stated that the objective of signal control should be to avoid spillback and to provide equitable service. The report also gave some tactical control strategies to ease queue blockage at an intersection.

Dell'Olmo and Mirchandani (1995), used a simulation study to examine an approach of real-time coordination of traffic flow in an urban network. The approach considers available real time data for computing signal timing. It first identifies platoons and predicts their movement in the network (i.e., their arrival times at intersection, their sizes and their speeds) by fusing and filtering the traffic data obtained in the last few minutes. A traffic model is used to propagate the predicted platoons through the network for a given time horizon. The study showed that this real-time coordination examined performed as well as or better than off-line coordination methods such as PASSER II, or TRANSYT. They also concluded that this approach is suitable for light-to-moderate traffic conditions, but not oversaturated conditions.

2.2 Comparative analysis of traffic control systems

A step toward today's advanced traffic management systems was made in 1967 with the development of a computer program and methodology to gather and process data from loop detectors placed near intersections. The Urban Traffic Control System (UTCS) in the early 1970's resulted in the development of three generations of traffic control software (Henry, et al 1976). The first generation uses a set of predefined timing plans, which were developed off-line. These plans may be called into operation, manually, by time of day, or by a traffic responsive mechanism. Special features include dynamic control of critical intersections (CIC). The Second generation uses real time software that computes timing plans based on surveillance data and predicted changes. The plans are, however, constrained to fixed cycle lengths, but may include critical intersection control. The third generation is a real-time software system that computes timing plans based on surveillance data and predicted changes (approximately every three minutes). The third generation, however, controls intersections independently with a fully adaptive offset and cycle length determination.

Although the three generations of control reflected increasing level of comprehensiveness and complexity, they were not simple evolution of the same basic system. That is, each generation represented a unique development in control philosophy. For this reason, the second or third generations may not be "better' in any or all applications than the first generation, (Henry, et al 1976 and Kay, Allen, and Brugerman 1975).

The UTCS, which was installed in 114 intersections in Washington, D.C, operated standard traffic control signals directly from a central computer. Loop detectors were used as the basic source of traffic flow data. Data from the loop detectors is transmitted to the computer site using leased telephone lines. The computer translates the detector signals into traffic flow measures which are used to select the most appropriate signal timing plan in the case of first generation software, or to serve as the basis of computing the optimal signal setting for the second and third generations. A two-phase research program to evaluate the UTCS control strategies was conducted at these intersections. The first phase of the research consisted of an evaluation of four alternatives of first generation control strategies. Phase II of the research evaluated the second and the third generation control strategies as well as the most effective first generation control strategy. Results of the study are presented in Tables 2.1 and 2.2 (Henry, et al 1975 and 1976).

The first generation was found to be operationally effective and the least expensive to apply. Second generation control systems also proved to be effective especially in arterial streets. The third generation however, did not prove effective. Bell (1990) listed three reasons that the UTCS demand responsive systems failed when they were introduced in the late sixties. The first was the difficulty of predicting short term changes in traffic conditions (based on a time scale 5 to 10 minutes). The second was the disruption to traffic caused when changing from one signal plan to the next. The third was the limitation of the technologies available.

Much research has been performed to try to quantify the expected benefits of adaptive control systems. Several studies on isolated intersections claim to show substantial travel time benefits. Elahi and Goul (1991) stated that the application of expert systems to traffic control can provide significant improvements over

Table 2.1 Comparison of travel time by route under different control strategies

Comparison	Route NO.	Percent Difference A. M	Noon	P.M
Traffic Responsive vs. D.C. fixed-time Plans	11	-8.2*	-2.1	15.3*
	13	-1.3	-7.4	13.1*
	22	13.3*	33.8*	10.2*
	24	5.1*	6.7	6.2
	30	5.9	22.4*	9.4
	40	4.1	17.4*	-0.7
Traffic Responsive vs. Time-of-day plan	11	-0.2	-1.0	-1.2
	13	3.3	1.9	4.9
	22	4.7	10.6 *	2.2
	24	4.6	-4.5	2.1*
	30	-9.6	9.7*	1.1
	40	-10.2	-7.7*	4.7
Traffic Responsive vs. Critical Intersection Control (CIC)	11	4.2	0.3	4.2
	13	7.1*	4.1	-1.1
	22	-4.1	1.4	-3.5
	24	-9.7	4.3	-29.7
	30	-11.2*	7.1*	-6.7
	40	-0.5	10.5*	-4.4

* Statistically significant at the 95% level.

Positive values indicate lower travel time with the traffic responsive alternative relative to the other alternatives.

Tables 2.2 Comparison of total travel time under alternative control strategies with first generation control

	Percent Difference			
time period	Second Generation Control 2-GC	Second Generation Control with Critical Intersection Control	Third Generation Control 3-GC	Washington, D.C. fixed-time signal plans
Morning peak period	-2.0	+1.0	+6.4	-5.5
Non-peak period	-13.5*	-8.4	-6.8	-16.6*
Noon peak period	-10.0*	-3.3	-9.0*	-11.1*
Afternoon peak period	-10.6*	-11.6*	-13.5*	-19.8*
Total	-9.9*	-6.3*	-5.6	-14.0*

*** Denotes the difference is statistically significant at the 95% level.**
A negative sign indicates a "better" performance relative to the base case (fewer vehicle-minutes of travel). A positive sign indicates degradation in performance

conventional control systems. Pierce and Webb (1990) reported a reduction in delay of between 3-24% as a result of using MOVA, a self-optimizing adaptive control system designed to reduce delay and stops at isolated intersections, to control 20 isolated traffic signals in England. Barcelo et al (1991) stated that a demand-responsive traffic control system would improve the network performance between 10-25%.

Stewart and Van Aerde (1997) presented the findings from a systematic simulation study that attempts to show what types of adaptive control strategies have the greatest potential of producing incremental travel time benefits for different types of traffic conditions. The study compared two types of fixed-time control with two types of adaptive control under different traffic volumes and peak hour factors. The analysis was performed using the INTEGRATION simulation model. The results showed that, at low traffic volumes and at a low level of peaking there is little difference in the performance of any control strategy. In contrast, the study showed that that when traffic is highly peaked, well-optimized time-of-day plans produce better results than fixed-time plans and adaptive control strategies. The study showed that adaptive control strategies yield lower travel times than all other types of control when traffic demand levels evolve slowly.

2.3 Current practice of adaptive control systems

Several types of advanced control systems are currently in operation, serving in experimental traffic control management roles. Two such systems, SCATS and SCOOT, have developed into the most prevalent real time adaptive traffic signal control systems. The SCOOT system was developed in Great Britain in the mid-1970's. The main idea of the SCOOT system was to take an "off-line" model, like the fixed time signal optimization program TRANSYT, and have it operate "on-line. Specifically, the system uses traffic data measurements collected from the existing stream and makes short and long term decisions regarding the traffic signal settings.

Many studies have been completed which claim to have evaluated adaptive traffic signal systems. Among the earliest reported benefits, a 1966 project in Wichita Falls, Texas, reported a 16% reduction in stops, a 31% reduction in vehicle delay, a 8.5% reduction in accidents, and an increase in speeds of over 50%. This analysis compared the computerized system to the single-dial system it replaced. The Fuel Efficient Traffic Signal Management (FETSIM) and Automated Traffic Surveillance and Control (ATSAC) programs in California (1976) showed benefit/cost ratios of 58:1 and 9.8:1 respectively. ATSAC, which includes computerized signal control, reported a 13% reduction in travel time, a 35% reduction in vehicle stops, a 14% increase in average speed, a 20% decrease in intersection delay, a 12.5% decrease in fuel consumption, a 10% decrease in HC, and a 10% decrease in CO levels.

Several of the more recent studies have involved the two most widely used real-time traffic responsive signal systems, SCATS and SCOOT. The creators of SCATS, the Australian Road Research Board (ARRB) and the Road and Traffic Authority of New South Wales carried out many studies to compare the system against various less sophisticated forms of signal control. One study measured the performance of SCATS against the control characteristics afforded by systems with isolated fixed time signal phasing and TRANSYT optimized fixed time control with and without local vehicle actuation.

The ARRB study conducted their comparison using the floating car travel time estimation technique to record the "journey" or travel time on each link, the number of stops in each link, the stopped time in each link, and the amount of fuel used in each trip. The recorded stopped times were later found to be unreliable, so they could not be used in the analysis. The study was able to compare the different signal systems in terms of travel time, number of stops, and a derived "Performance Index." The Performance Index was a weighted-measure of travel time incorporating the number of stops during the trip. The study found that on one arterial highway, SCATS resulted in a 23% reduction in travel time and a 46% reduction in stops over isolated fixed time signals. In the central business district (CBD) study area, the travel time was not effected and the reduction in stops was 8%.

When compared to Linked Vehicle Actuated (LVA) control, SCATS showed some benefits and some degradation in the recorded performance measures on the arterial and in the CBD areas. The comparison of SCATS and TRANSYT optimized fixed times concluded that SCATS can improve travel time and number of stops from 3% to 18%. The actual improvement depends upon the type of road system and traffic patterns for the area under study (Well 1987).

ARRB also conducted a comparative study of SCATS versus SCOOT. It detailed the similarities and differences in the data requirements, hardware, and operation of the two systems. However, a direct field comparison of the operational differences between SCATS and SCOOT was not possible. Direct comparisons using simulated or actual data are very difficult, due to the different locations where traffic flow data is gathered. In SCATS, traffic information is collected at the approach stop lines. The required traffic flow information for SCOOT is collected upstream of the stop lines. The two systems also differ in their operating requirements for computer processing. The paper stated that SCATS is better in some applications because it has the capacity to estimate congestion better than SCOOT. By contrast, SCOOT can be more effective in certain heavy flow situations because it incorporates an automatic double-cycling mechanism, which SCATS did not have.

Kelman (1991) presented a performance report on the Metropolitan Toronta SCOOT system. The study showed that time-of-day plans, in certain circumstances, were better than adaptive strategies. This is further supported by work done by Gartner et al (1994, 1995). They indicated that, while the largest possible benefits, (+10% improvement) could be achieved using the most traffic responsive type of control, the highest dis-benefits, (-10% deterioration) were also associated with the most traffic responsive controls. Finally, it was suggested by Ort (1995), and Richeson and Underwood (1996), that the majority of ITS benefits are related to increased safety, and that these benefits were not directly attributable to the implementation of adaptive versus fixed-time control strategies.

Abdel-Rahim and Taylor (1998) reported the results of a study conducted to determine the change in travel time following the implementation of the SCATS in Oakland County, Michigan. A before/after comparison was used to examine the change in travel time on a specific corridor. The results showed that the corridor travel-time improved for both directions for both the peak and the non-peak periods. The reduction in corridor travel time ranged from 6.6% to 31.8%, with savings in travel time being higher during non-peak periods. Before/after intersection delay studies showed that the approach delay for the main street traffic decreased at the intersections as a result of SCATS implementation. SCATS extended the green time for the through traffic, reducing the average degree of saturation from 1.02 to 0.87

during peak periods and from 0.73 to 0.56 during non-peak periods. SCATS reduced the green time for other approaches increasing the degree of saturation on the minor approach from 0.52 to 0.63 during peak periods and from 0.22 to 0.31 during non-peak periods. A before/after offset study showed that the through bandwidth increased during all time periods for both directions, mainly as a result of extending the green time for the main street traffic.

Andrews et al (1997) evaluated the New Jersey Optimized Policies for Adaptive Control (OPAC) system. The evaluation compared the performance of OPAC against a well-designed time-of-day fixed-time signal. The evaluation was performed under various traffic demand conditions and included both isolated intersections and arterial sections. The analysis indicated a significant improvement with OPAC control. OPAC performed its best during oversaturated conditions. It reduced the travel time and number of stops by about 26 percent and 55 percent, respectively, for the entire arterial section. OPAC also improved traffic performance during changing demand conditions. It significantly improved the performance of an isolated intersection during unsaturated conditions. OPAC reduced stopped delay on the major-street approach by 40 percent without affecting the minor street performance.

2.4 Literature Review Conclusions

Many research studies have been performed to quantify the benefits of adaptive signal control systems. Several studies on currently deployed adaptive control systems claim to show significant travel time and intersection delay benefits. However, the nature of the benchmark against which these improvements were measured is not always clearly identified in these studies. For example, it is not known whether these studies used as their benchmarks optimized time-of-the- day plans or simply used the plans that were already in place. Therefore, it is possible that any improvements gained through the implementation of adaptive signal control might also have been achieved by simply updating the existing time-of-day fixed-time plans to more optimized plans.

While simulation is an effective tool to compare the effectiveness of different signal control strategies under the same environment, the literature search found few research studies using simulation to compare alternative control strategies. This may be due to the fact that none of the existing simulation models can fully replicate the adaptive control logic.

The literature review helped identify the research gaps in assessing the benefits of adaptive signal control strategies. First, none of the past studies compared different adaptive control strategies for urban corridors against optimized coordinated fixed time or actuated signals. Second, none of past studies used simulation models that can fully replicate adaptive control logic. Third, none of the past studies examined different traffic prediction techniques employed in the adaptive control logic, Finally, none of these studies examined the sensitivity of the adaptive control benefits to changes in demand levels and peak hour factors.

The review of published literature has demonstrated the interest in and importance of comparative studies of the various forms of traffic signal control. As a result, studies that can assist current and future users of real-time adaptive signal control systems to determine their expected benefits are extremely valuable. The research conducted in this study is original in covering some of the areas that have not yet been addressed. The research used the new version of CORSIM, which has the ability to model adaptive signal control logic, to examine the benefits of adaptive control logic against conventional coordinated fixed-time and full-actuated signals in an urban corridor. The sensitivity of the benefits to changes in demand level and peak-hour factor was also examined. The effectiveness of different prediction techniques used for adaptive control logic was also examined.

Chapter 3

DEVELOPMENT OF THE SIMULATION MODEL

3.1 Simulation Modeling

Simulation can be used with some degree of confidence to examine the microscopic as well as macroscopic aspects of travel in traffic networks. A number of widely disseminated traffic simulation packages are available to support the analysis, design, and evaluation of a traffic control system's operation. Simulation models incorporate analytical models, such as traffic flow models, car-following models, shock-wave analysis and queuing theory, into a framework for simulating complex components or systems of interactive components (Waugh, Clark, and Kanan, 1994).

Traffic simulation models can be classified based on the level of simulation detail to three categories: microscopic, macroscopic, or mesoscopic. Microscopic simulation models describe the detailed, time-varying trajectories of individual vehicles in the traffic stream. With this level of details in the simulation, the microscopic models can be used to study the effects of detailed control strategies on different traffic parameters (Kosonen, 1990). Examples of the microscopic simulation models are NETSIM for surface streets and FRESIM for freeways from the integrated traffic family of simulation models (TRAF). The two models were recently integrated into one microscopic model for both the freeway and surface street traffic (CORSIM).

Macroscopic models represent the traffic stream in some aggregate form (e.g., employing a fluid flow analogy or a statistical representation). Less detailed strategies, involving changes in circulation patterns, for example, may be studied with macroscopic models. These models may also be used to gauge the impacts of very detailed strategies outside the boundaries of the area in which these strategies are implemented (Rakha, 1989 and Rathi, 1990). Examples of macroscopic simulation models are NETFLO 1, NETFLO 2 for surface streets and FREFLO for freeways from the TRAF family of simulation models. The three models were also integrated into one macroscopic model for both freeway and surface streets (CORFLO).

Another example of a macroscopic traffic simulation model is TRANSYT. It is among the most realistic of those available in the family of computerized macroscopic traffic models. The traffic model utilizes analytical flow-density-speed relationships and platoon dispersion algorithm to simulate the normal dispersion of platoons as they travel throughout the traffic network, (Wallace et. al., 1998).

Recently, several "mesoscopic" models have been developed. These models track the movements of individual vehicles, as microscopic models do, but model their movements using macroscopic flow equations. An example of mesoscopic simulation models is the INTEGRATION model, which was conceived during mid 1980s as an integrated simulation and traffic assignment tool (Stewart and Van Aerde, 1998).

The selection of the level of simulation detail and the model that should be applied is based on the physical environment to be studied, the level of detailed analysis desired, the availability of data, and the objectives of the study (Waugh, Clark, and Kanaan, 1994). As the scope of this study involved studying microscopic signal control strategies in an urban traffic network, a higher level of analysis is required, and a decision was made to consider a microscopic or mesoscopic simulation model for the analysis.

3.1.1 Selection of the Simulation Model Suitable for ITS applications

While traffic simulation covers a wide spectrum of traffic theory and engineering topics, the current interest in ITS technologies raises significant challenges for traffic simulation models. Since ITS technologies encompass both local and area-wide traffic control strategies, the scope of the traffic simulation model must incorporate microscopic signal control, a vehicle rerouting mechanism, traffic surveillance, and a dynamic optimization algorithm that is capable of optimizing traffic signal control on an on-line basis, (Mahmassani, 1992).

A growing variety of simulation models are being developed for ITS applications. Three simulation models, that have the potential to simulate adaptive signal control strategies, were examined in the early stages of this study. The three models are the INTEGRATION simulation model, the Traffic and Highway Objects for Research, Analysis and Understanding (THOREAU) simulation model, and

CORSIM form the FHWA's TRAF family of simulation models.

The INTEGRATION simulation model, developed in the University of Queens-Canada, has the ability to model a responsive signal control system through an automatic signal re-timing algorithm included in the model. When this automatic signal re-timing is utilized, only the offsets and the lost time specified in the original signal-timing plan are kept constant. The other timing plan parameters (cycle length and phase splits) are optimized each time interval based on the volume/saturation flow ratios in the competing approaches, (Stewart and Van Aerde, 1998). While the model provides a framework for the simulation of ITS implementation alternatives, it does not allow the users to incorporate their own signal optimization algorithm or adaptive control logic within the model, which limits the number of control strategies that can be tested using this model. The model also does not have the capabilities to test dynamic offset plan alternatives.

THOREAU is an object-oriented, trip specific, microscopic and macroscopic traffic simulation model designed for comparative ITS analysis. The model adopts object-oriented, discrete event simulation for vehicle movements along arterial streets, through intersections and on the freeways. The model incorporates adaptive signal control strategies similar to the UTCS strategies, which, allows an analyst to study different adaptive signal control strategies, (Hsin and Wang, 1992). A major disadvantage of the model is that it requires a dynamic input-output matrix to generate vehicle trips, something that is not generally available. Similar to INTEGRATION, the model does not allow users to use their own control logic, or to modify the control strategies included in the model.

The current version of CORSIM has the capability of modeling the operation of real-time traffic adaptive control signals. The model allows the transfer of detector surveillance data to an external real-time signal control algorithm, which updates the signal control parameters on an on-line basis. The user defines the frequency and type of information to be sent between the model and the external real-time control algorithm. Based on the review of the available simulation models, a decision was made to use CORSIM as the simulation tool for this study. The latest version of CORSIM (1.05 Alpha/TSIS v4.5), issued in March 1998, was used in this study.

3.1.2 Description of CORSIM (Structure, Principles, and Features)

The first version of NETSIM was developed for the Federal Highway Administration (FHWA) nearly two decades ago. The program, formerly called UTCS-1, was later integrated into the TRAF simulation system. Over the years, NETSIM and FRESIM have gained acceptance in the traffic engineering community as presenting a reasonable representation and MOEs of traffic movement along surface street and freeway systems, though not in a combined network. In 1997, CORSIM was introduced as the product of combining FRESIM and NETSIM into a singularly defined and integrated traffic network, (FHWA, 1997).

The CORSIM model is based on the application of interval-based, discrete event simulation to describe the dynamics of traffic operations. The vehicles are represented individually and their operational performance is determined uniquely every second. Furthermore, each vehicle is identified by category, type, and driver-behavioral characteristic. There are 16 different vehicle types with various operating and performance characteristics. Drivers are classified into three different driver behavioral categories (aggressive, normal, and passive). The driver-vehicle characteristics are assigned stochastically, as are other performance and behavioral attributes including turning movements, free-flow speed, queue discharge headway, and acceptable gaps. Consequently, each vehicle's behavior may be simulated in a manner reflecting real world processes. In general, most operational conditions experienced in an urban street network environment are realistically described (Rathi and Santiago, 1990).

Each time a vehicle is moved, its position (both lateral and longitudinal) on the network, its relationship to other vehicles, and its kinematics properties (speed, acceleration and status) are recalculated and updated. Vehicles are moved according to car-following logic while responding to traffic control devices, pedestrian activity, transit operations, performance of neighboring vehicles, and other conditions that influence driver behavior. Actuated control and interaction between passenger cars and buses are explicitly modeled.

CORSIM has the capability to simulate the effects of all types of intersection control. Signal controllers can be fixed-time, multi-dial, semi-actuated, or full-actuated signals. Signals can also be set to operate as real-time adaptive signals using the optional run-time extension capabilities.

The simulation procedure in CORSIM consists of a warm-up (initialization) period and an actual simulation period during which statistical data are accumulated. During the warm-up time, traffic generators feed vehicles into the empty simulated network until equilibrium conditions are reached, that is, the rate at which vehicles enter and discharge from the network is equal. Having reached equilibrium the model starts to accumulate statistics on different traffic parameters in the network.

3.1.3 Modeling an External Controller within CORSIM

CORSIM's functionality has been extended through the development of the new Traffic Software Integrated System (TSIS). TSIS is a Microsoft Windows (95/NT)-based environment for CORSIM. It provides an intuitive user-interface to CORSIM (as well as other FHWA models in the future). TSIS has also provided a fundamentally new approach to using the model to research ITS technologies by allowing other application programs to interact with CORSIM while the simulation is executing, and further, these other programs can have an effect on internal CORSIM operations. This type of application is known as a Run Time Extension (RTE).

RTE can be used to test adaptive signal control algorithms for effectiveness, before they are implemented in the field. With the rapid evolution of technology, these types of experiments are necessary, not only to assess the benefits of advanced ITS applications, but also to verify their operational capability before field deployment. This new capability is a critically important step in extending the usefulness and effectiveness of simulation-based research and analysis.

To understand how real-time extension programs function within CORSIM, the structure and the component of TSIS/CORSIM should be understood. The TSIS is composed of several major components including the "shell" application TSIS.EXE and several Windows Dynamic Link Libraries (DLL). The four major components of the TSIS/CORSIM are:

1. TSIS.EXE, the executable MS-Windows program, the new host for operating CORSIM.

2. CORSIM.DLL, which is built from the configuration-controlled FORTRAN source code.

3. TSISINTF.DLL, the TSIS component that contains the shared memory and the communications interface, all of which are utilized by CORSIM and CORSIM RTE.

4. Optional RTE in DLL format, can be designed and implemented to interact with CORSIM at run-time

Figure 3.1 illustrates the overall functional diagram of TSIS and its components (FHWA, 1997)

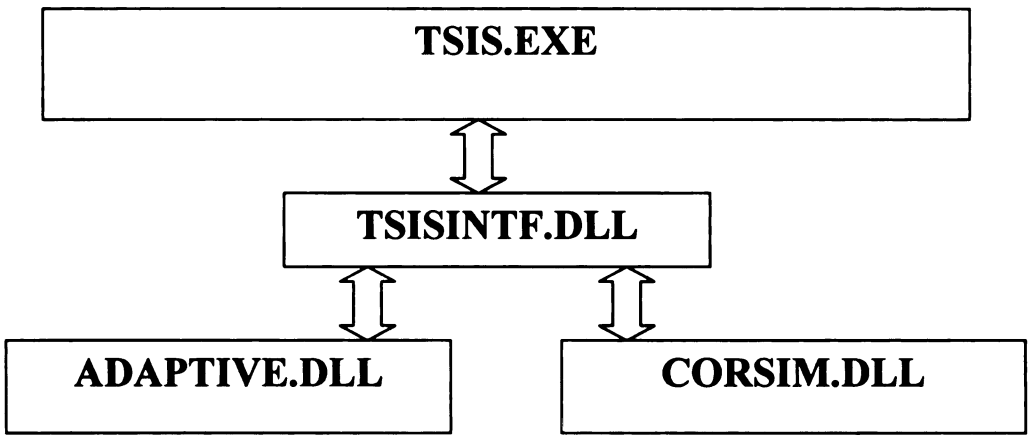

Figure 3.1 TSIS-CORSIM RTE interface structure

3.2 Signal Control strategies

The study was formulated to test the effectiveness of adaptive signal control strategies against two conventional, widely applied corridor signal control strategies namely: coordinated fixed-time and actuated signals. Two adaptive control systems that equalize the degree of saturation on critical approaches were examined. The difference between the two adaptive strategies lies in the way traffic flow predictions were made. In the first adaptive control strategy, traffic predictions were made based on detectors placed downstream from the intersection, whereas, in the second adaptive control strategy the predictions were based on detectors placed upstream of the intersection.

3.2.1 Coordinated Fixed-Time Signals

3.2.1.1 Definition

In a coordinated fixed time signal system, the cycle length is constant for all signals during any given control period. Cycle length, green split, and offsets are optimized off-line based on historical traffic profile data. The values of all signal timing parameters are kept constant during any given control period, but the signals may have different settings based on time-of-day. In this study, the signal settings were kept constant during the one-hour analysis period.

3.2.1.2 Optimizing Fixed Time Signal Plans

The design of corridor traffic signal timings has significant impact on traffic flow parameters. As the major objective of this study is to compare the effectiveness of different control strategies, it was important to develop optimal settings for the fixed-time signal control under different traffic conditions. TRANSYT and PASSER are the most widely used tools for optimizing signal-timing plans for corridors and traffic networks.

There are five essential elements that constitute pretimed signal timing in a coordinated signal system: cycle length, number of phases, phase sequence, phase lengths, and offsets. TRANSYT explicitly optimizes phase lengths and offsets for a given cycle length. The program is applied at the network and/or corridor level wherein a consistent set of traffic conditions is known and the system hardware can be integrated and coordinated with respect to a fixed cycle length and coordinated offsets.

Optimization is performed in TRANSYT through the simulation of vehicle responses to various signal settings. The traffic simulation model in TRANSYT is among the most realistic of those available in the family of computerized macroscopic traffic models. The traffic model utilizes a platoon dispersion algorithm that simulates the normal dispersion of platoons as they travel downstream. To determine the best cycle length, an evaluation of a user-specified range of cycle lengths may be made. TRANSYT does not select the phase sequence--these are required inputs. To examine alternative phase sequences, multiple computer runs are required. When optimizing, TRANSYT minimizes (or maximizes, depending on the selection) an objective function called the Performance Index (PI) which is either a linear combination of delay and stops, fuel consumption and excessive maximum back of queue; or excess operating cost. This PI is often referred to as "performance optimization", and is to be minimized, (Wallace, 1998 and Hadi, 1992)

Another performance index alternative is the forward progression opportunities "PROS". It can be used either alone or in combination with the delay and stops (or fuel consumption) disutility function. This PI is maximized to increase the quality of perceived progression. The components of the PI are:

1) Standard TRANSYT disutility index (DI), the "standard" delay and stops is a linear combination of these measures:

   DI=[delay on a link * link-delay weighting factor] + [stop penalty * stop * link-specific stops weighting factors]

2) Excess operating cost,
3) Progression Opportunities (PROS), and
4) Optimization objective function which may be defined by the user in a number of ways:

$$DI = \begin{vmatrix} \text{Standard delay and stops} \\ \text{Excess fuel consumption} \\ \text{Excess operation cost} \end{vmatrix} + [\text{double count progressive links}] + [\text{queue penalty}]$$

PASSER is another tool for corridor signal timing optimization. The model consists of three optimization software programs that optimize traffic timings on single roadways or entire networks of roadways. PASSER-II can be effectively used for arterial progression optimization (bandwidth maximization), existing system evaluation (simulation), and intersection capacity evaluation. PASSER helps traffic engineers and planners develop timing strategies that optimize the flow of traffic on a single road or through the entire network by maximizing the progression band. Cycle and offsets optimization in PASSER is based on bandwidth efficiency optimization. By determining the maximum band in both directions of a roadway, the program can calculate timings that allows the greatest number of cars pass through the system without stopping, (Abu-Lebdeh and Benekohal, 1997).

A key factor in the comparative study conducted in this research is to compare the adaptive signal control strategies against well-optimized fixed-time and actuated signals. This will ensure that any benefits reported in the study would be solely the result of the adaptive control logic and could not be achieved through any other fixed-time or actuated signal settings. For this reason, different settings for the fixed-time signal control were examined to determine the optimal signal parameters. The Highway Capacity Manual (HCM) defines arterial/corridor performance in terms of the average speed of through vehicles, (Anderson, 1996). An optimal setting, as defined in this study, is the setting that would achieve the highest average speed/lowest average corridor travel time among all alternatives.

TRANSYT-7F and PASSER II-90 were used to obtain alternative "optimal" signal settings (offset, cycle length and green split). The CORSIM simulation model was used to compare the performance of these alternative settings. Average corridor speed and total travel time were used as measures of effectiveness to determine the optimal signal setting among these alternatives. The results showed that offsets and green split, obtained from PASSER II-90, provided the highest average speed and lowest total travel time.

3.2.2 Coordinated Actuated Signal Systems

3.2.2.1 Actuated control features and operation

Actuated signals allocate the green time for each phase on a cycle-by-cycle basis in response to detector actuation. Detectors, placed upstream to the intersection, provide the actuated controller with information concerning current demand. Signal actuation, accordingly, permits phases to start early or to end late to meet the variation in demand. This flexibility should make it possible to improve the performance of the control strategy.

Each actuated phase has the following features that must be set on the controller:

Minimum Green Time: Each phase has a minimum green time. Older controllers divided this time into two portions: an initial green interval and a unit extension. The initial green interval was intended to provide sufficient time for all vehicles potentially stored between the detector and the stop line to enter the intersection, the unit extension allowed another vehicle to travel from the detector to the stop line.

For all actuated phases, the minimum green time must be established. This is generally equal to an initial interval that allows all vehicles potentially stored between the detector and the stop line to enter the intersection. A start-up time of 4 seconds is incorporated in addition to 2 seconds for each vehicle (the latter based on a saturation flow rate of 1800 vph).

Passage Time Interval: The passage time interval allows a vehicle to travel from the detector to the stop line, and is analogous to the "unit extension" of older controllers. The passage time setting, however, also defines the maximum gap between vehicles arriving at the detector to retain a given green phase.

Maximum Green Time: Each phase has a maximum green time. If demand is sufficient to retain a given green phase to this limit, the green will arbitrarily terminate. Maximum green times are generally set by working out an optimal cycle length and phase splits as if the controller were pretimed.

Figure 3.2 illustrates the operation of an actuated phase based on these three settings. When a green indication is initiated, it will be retained for at least the specified minimum green period. Additional detector actuation may occur during this minimum green period. If they occur, and there is at least one passage time left before the minimum green terminates, no green time is added. If a vehicle arrives during the minimum green and there is less than one passage time left before its termination, an amount of green time equal to the passage time is added. The controller now enters the extension portion of the phase. If a subsequent actuation occurs within one passage time interval, another passage time interval is added to the green. The green is terminated by one of two mechanisms:

a) a passage time elapses without an additional actuation, or

b) the maximum green time is reached and there is a call on another phase.

Coordinated actuated signal systems operate on a common background cycle length. Synchronization is provided through the yield point, which is a fixed point in the cycle length, normally during or at the end of the non-actuated phase.

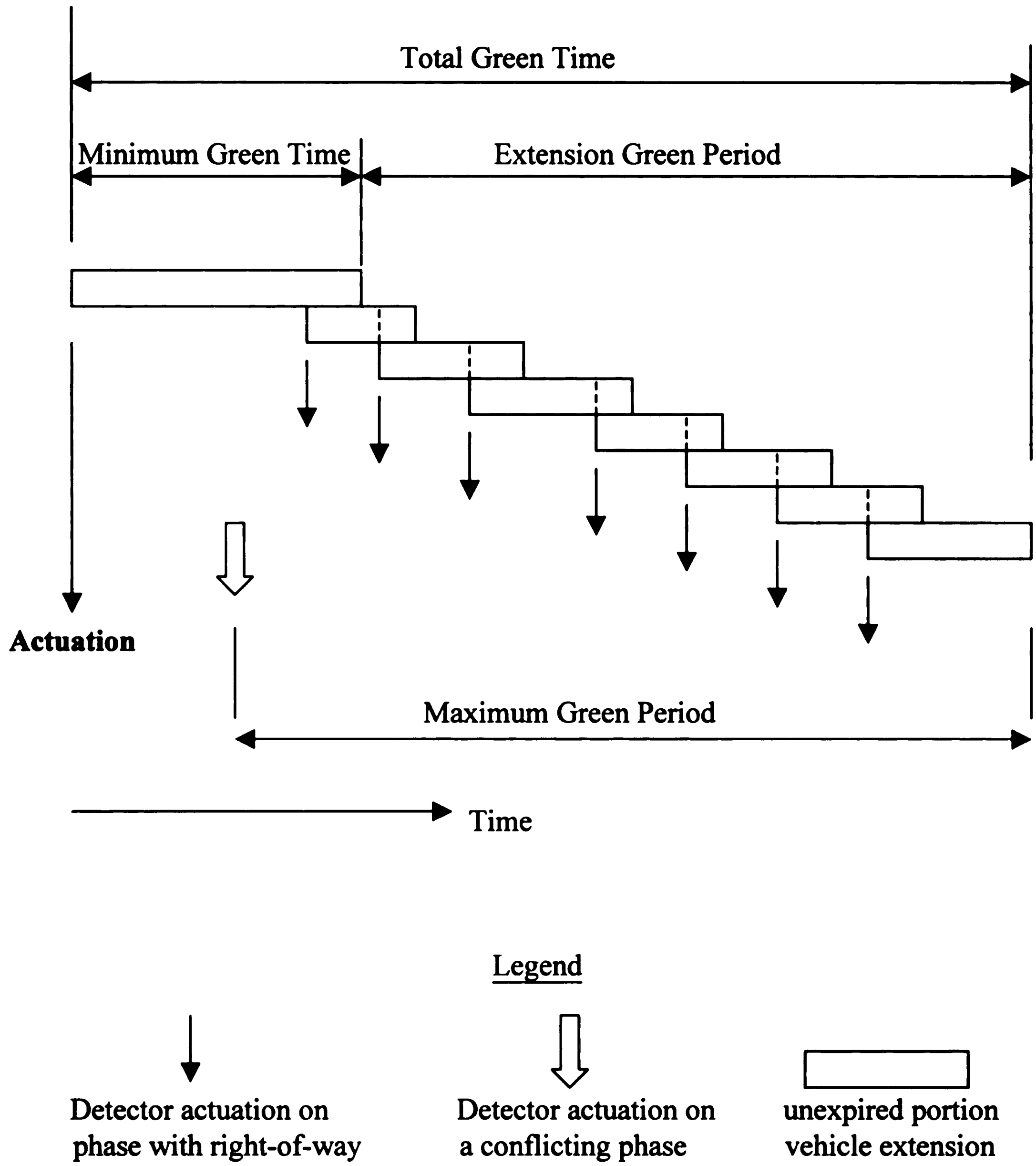

Figure 3.2 Operation of an actuated phase

The non-actuated phase has some minimum green time, plus all slack time not used by actuated phases. The green times on all actuated phases vary on each cycle between a minimum green and a maximum green, depending on the arrival rate of vehicles and the value of the extension intervals. Fixed force-off points in the background of the cycle are used to terminate the duration of the actuated phases. If an actuated phase terminates early, then any green time not used by the actuated phase is transferred to the non-actuated phase. For this study, the green time for the major corridor was set as the non-actuated phase. Figure 3.3 illustrates the coordinated actuated signal controller.

3.2.2.2 Optimization of Actuated Signal Control Settings

Optimization of coordinated actuated signals include two main objectives:

1) optimal design of cycle length and offsets

2) optimal design of detection scheme (detector length and location) and gap-time settings.

TRANSYT-7F has the capability of optimizing actuated signal time settings. However, the program does the optimization through calculating the average values of green time and cycle length. For the purpose of this study, the optimal cycle length and offsets obtained form the fixed time signal settings were used as the background cycle length and offsets for the actuated signals.

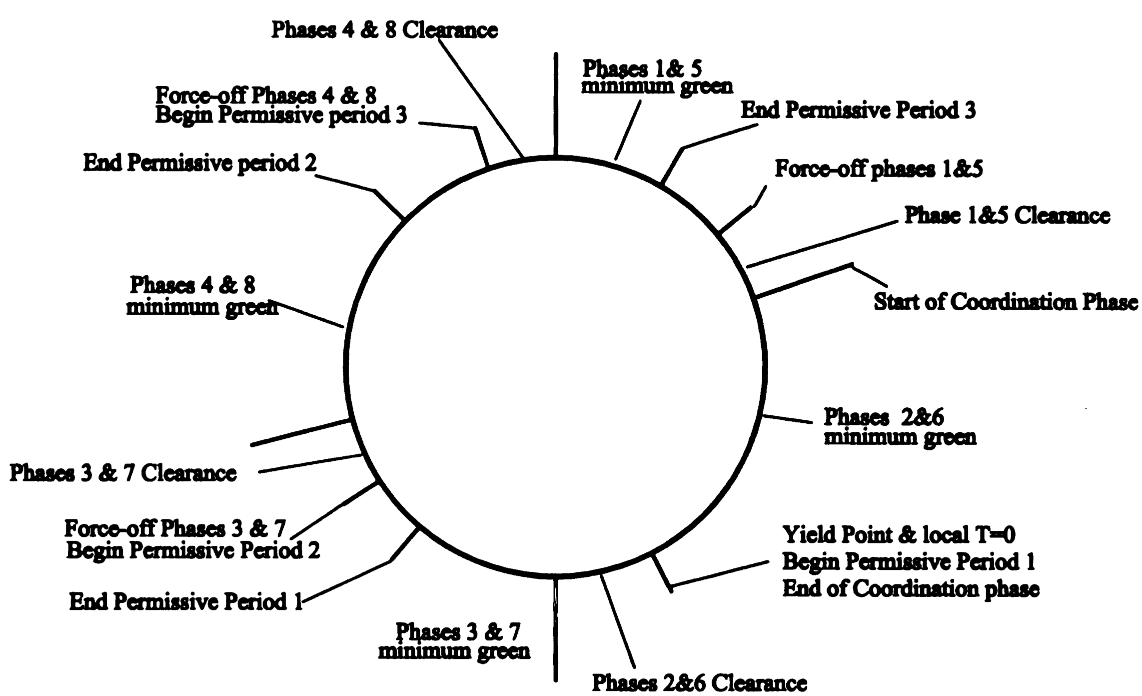

Figure 3.3 Typical timing dial diagram for coordinated actuated signals in CORSIM

A considerable number of studies have examined the optimal detector length and location for fully actuated signal systems. Lin (1992), recommended that a 20-foot detector located approximately 120-200 feet upstream from the intersection would be optimal for most urban area actuated controllers. Many studies (Lin, 1992), and (Rathi 1994) recommended a gap-out of 3 to 5 seconds to provide an optimal control for urban area traffic. For this study, traffic simulation was used to determine the optimal detector location and gap-out among alternative settings. Detectors placed 180 feet upstream to the intersection with a gap-out time equal to 3 seconds provided the optimal setting.

3.2.3 Real-time adaptive signal control

Figure 3.4 shows the components of real-time adaptive control. The control consists of a detection method, traffic prediction model, and signal optimization model.

3.2.3.1 Prediction models for adaptive Signal Control

The effectiveness of adaptive systems is dependent on the accuracy of the real-time traffic predictions. Traffic predictions are made for some time in the future. During a computational process for optimization of a specific cycle, only the predictions from the previous cycle/cycles data are available. Plans optimized using data collected during the cycle (N) would be implemented in the cycle (N+2) (Figure 3.5).

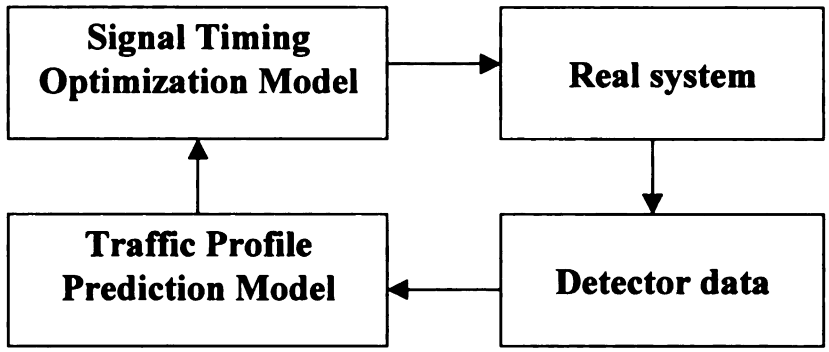

Figure 3.4 Network flow model for real-time adaptive control

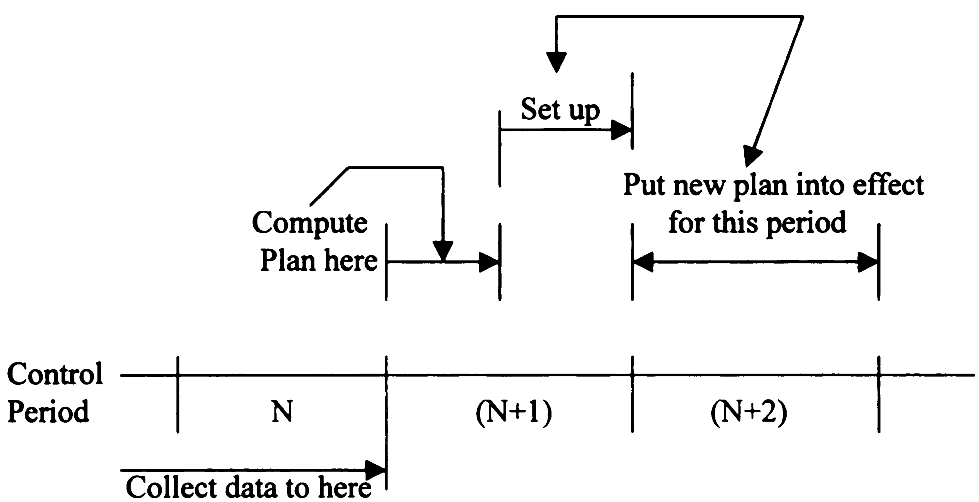

Figure 3.5 Prediction sequence for adaptive control strategies

Prediction of the traffic profile can be made in two ways, using detectors placed upstream or downstream from the intersection. Detectors placed downstream from the intersection predict a traffic profile for a specific cycle based on the volume(s) of previous cycles. In essence, they predict changes in traffic demand over time rather than cycle-by-cycle variation in demand. Detectors placed upstream to the intersection, however, use "scan-ahead" and predict the arrival of vehicles during a specific cycle based on estimated time of arrival of the platoons to junctions further downstream.

For this study, the effectiveness of both prediction techniques was examined. In the first adaptive control strategy, predictions of the traffic profile at the intersection was obtained from detectors placed downstream from the stop-bar at the intersection. The expected volume for the new cycle (n+1), was estimated, similar to SCATS adaptive control, based on volumes departed during cycles (n, n-1, n-2) as follows:

$$V_{n+1} = 0.5\, V_n + 0.3\, V_{n-1} + 0.2\, V_{n-2}$$

In the second adaptive control strategy, predictions were made based on detectors placed upstream from the intersection. Detector locations are presented in Figure 3.6. The use of upstream detector to predict the arrival of vehicle platoons at the intersection is demonstrated in Figure 3.7. The optimization sequence of the adaptive control strategies is presented in Figure 3.8.

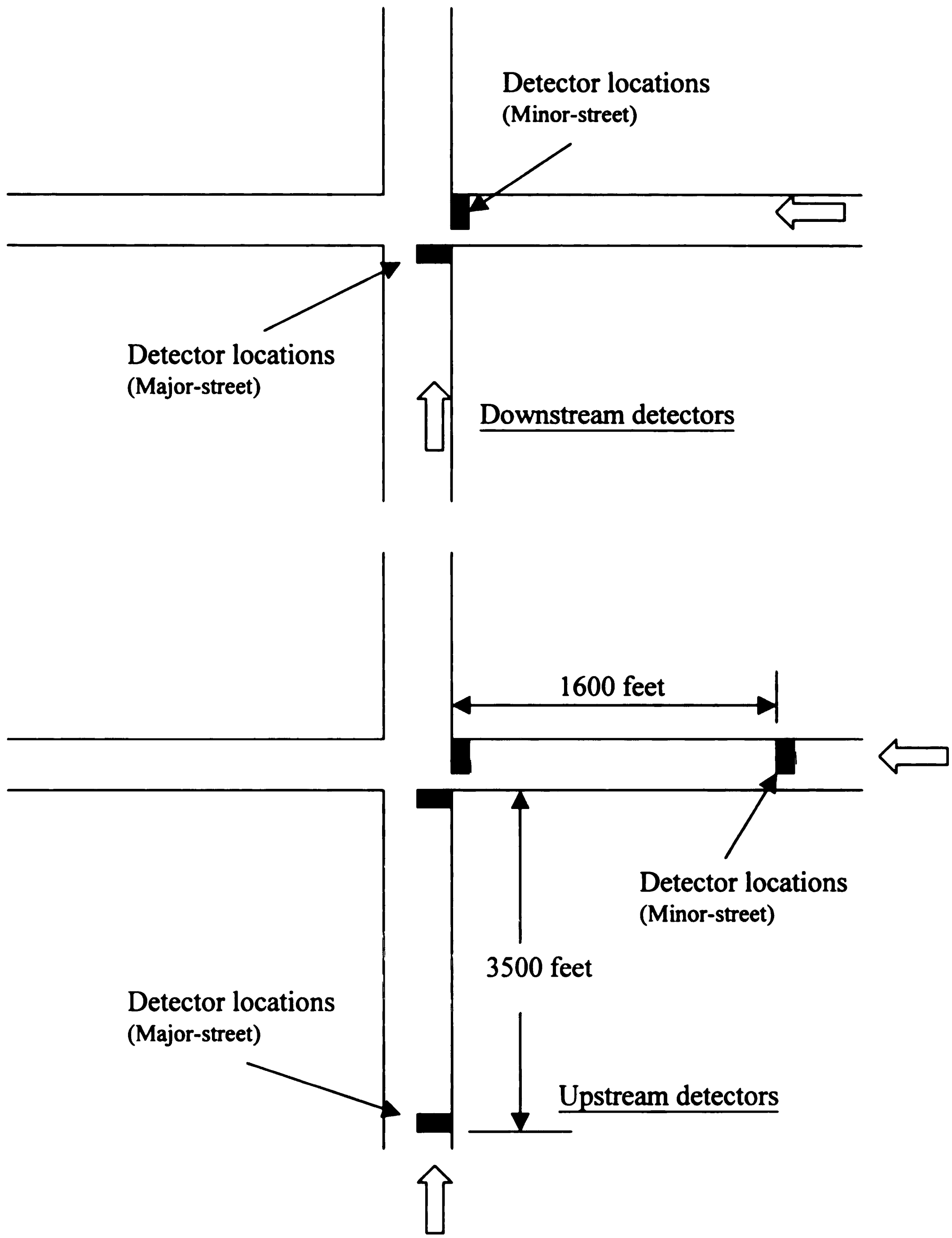

Figure 3.6 Detector locations for the two adaptive control strategies

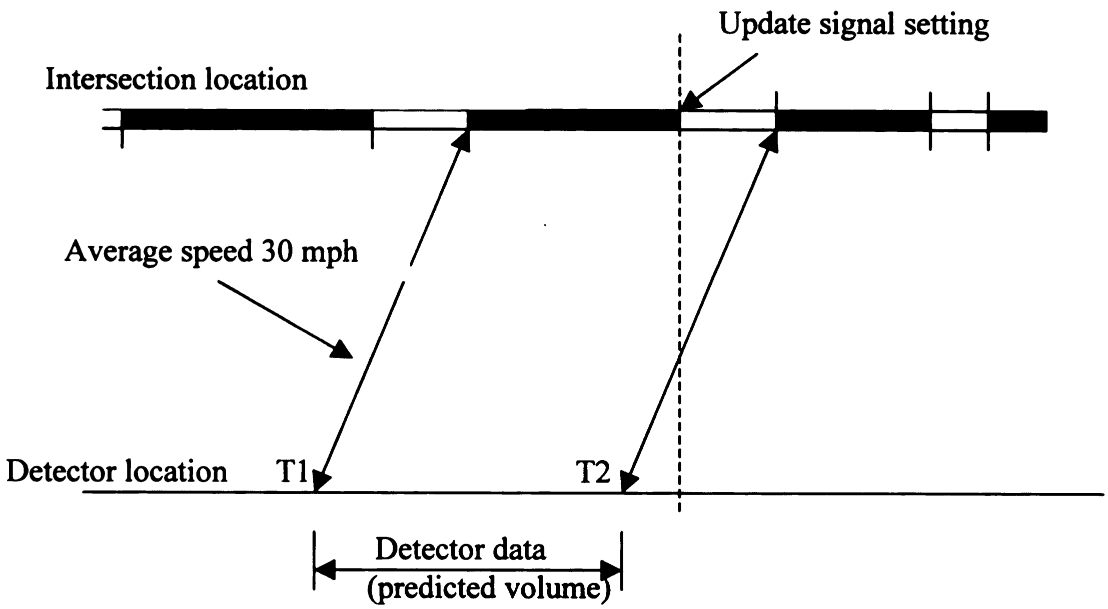

Figure 3.7 Use upstream detectors to predict vehicle arrivals

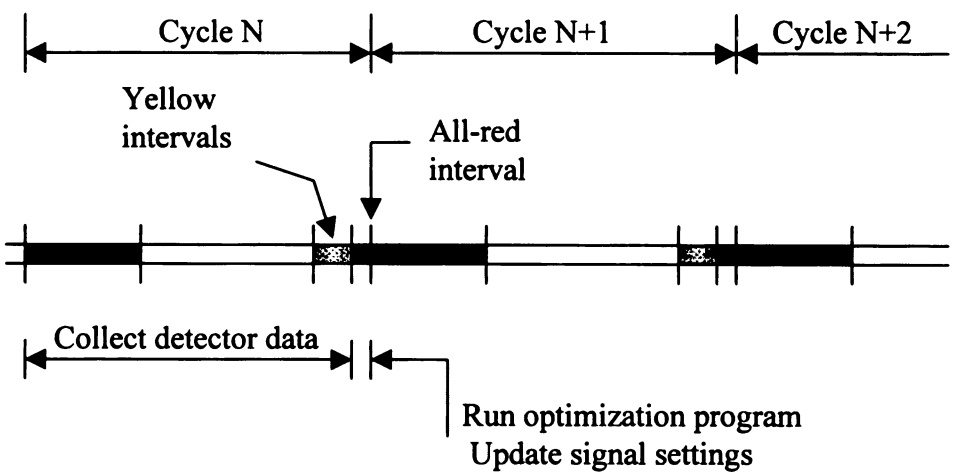

Figure 3.8 Optimization sequence for adaptive control strategies

3.2.3.2 Optimization models for Adaptive Signal control

Once predictions were made for the expected number of vehicles in a subsequent cycle, these data were used to calculate the optimal cycle length and green split. Cycle length was updated every five minutes. The cycle length was increased/decreased to obtain an optimal degree of saturation of not higher than 0.9 or lower than 0.8 on the corridor link that had the highest expected traffic volume. The green splits, however, were optimized on a cycle-by-cycle basis for each intersection. Optimal green split was obtained by maintaining an optimal degree of saturation of 0.92 for the minor-street traffic and assigning the rest of the green time for the major-street traffic. This method would maximize the throughput of the corridor and maintain adequate delay levels for the minor-street traffic. Offsets were chosen based on the value of the cycle length from a library of pre-optimized offset/cycle length values. This library was developed from TRANSYT-7F optimization of the corridor under different demand levels.

Before using run-time extensions for testing adaptive signal control algorithms, the input file must be edited to allow non-source nodes of the network to be placed under the control of a signal control algorithm separate from and external to CORSIM. The signal state for these nodes is controlled by the external control algorithm. The run-time extension program was written in FORTRAN 90 and was compiled using DIGITAL Visual Fortran V5.0 to obtain the DLL file used by CORSIM. Details of the optimization program are provided in Appendixes

CHAPTER 4

DESIGN OF THE STUDY

There are a number of unanswered issues that should be addressed before large-scale deployment of adaptive signal control technologies. The magnitude and consistency of the benefits of these control strategies under various traffic conditions should be fully evaluated. While field demonstrations are the only way to test new strategies in real-world conditions, traffic simulation, as an alternative, provides the mechanism for testing theories, modeling concepts, and control strategies. Simulation models are particularly valuable in identifying key operational and performance issues for different control strategies under a range of scenarios. Simulation studies have many advantages over field demonstrations; they are less costly, the results are obtained quickly, many variables can be held constant, and the data generated through simulation include several MOEs that may not be easily obtained through field studies.

4.1 Study Plan

For this study, simulation provided an opportunity to compare different signal control strategies for an arterial corridor using the same network configuration and traffic pattern. The study is designed to test different signal control strategies under a controlled environment using the CORSIM traffic simulation model. The control strategies considered in the study are:

1) coordinated fixed-time signals,

2) coordinated actuated signals,

3) adaptive signal control that equalizes the degree of saturation on critical approaches with traffic flow prediction based on detectors placed downstream from the intersection (adaptive_1); and

4) adaptive signal control that equalizes the degree of saturation on critical approaches with traffic flow predictions based on detectors placed upstream of the intersection (adaptive_2).

The study was organized in five parts, each designed to achieve one of the objectives of the research. The first part was a comparative analysis of the effectiveness of the four signal control strategies under two different demand levels: moderate peak-hour demand and high peak-hour demand. The second part was a signal timing analysis designed to test the changes in traffic parameters reported in the comparative analysis. The third part of the study was a sensitivity analysis to examine the benefits of the adaptive control systems over fixed time signals under four different demand levels and peak-hour factors for both major-street and minor-street traffic. Changes in the adaptive control system benefits resulting from adding two left turn phases to the signal operation were also examined in this section. The fourth part of the study was a comparison of the accuracy of the prediction techniques used in the adaptive control systems in this study. The final part was a field study to validate the output of the simulation model. A comparison of the cost and the benefits of different signal control strategies is also presented in the final part of the study. The study plan is shown in figure 4.1

4.2 Hypothetical Corridor Network

There is a general resistance among transport modelers to use hypothetical networks for simulation work because of the potential for introducing unrealistic features (Van Vuren and Leonard, 1994). Nonetheless, a theoretical network is commonly used when the main objective is to test a serious of scenarios that have not been applied in an existing network. Using a hypothetical network allows for the generalization of the output of the study beyond any site-specific configuration of an existing network.

A postulated surface street corridor was constructed for this study. The corridor has a total length of 3.6 mile and includes five intersections. The geometric configuration of the corridor is presented in Figure 4.2. The link-node diagram of the tested corridor is also presented on Figure 4.3

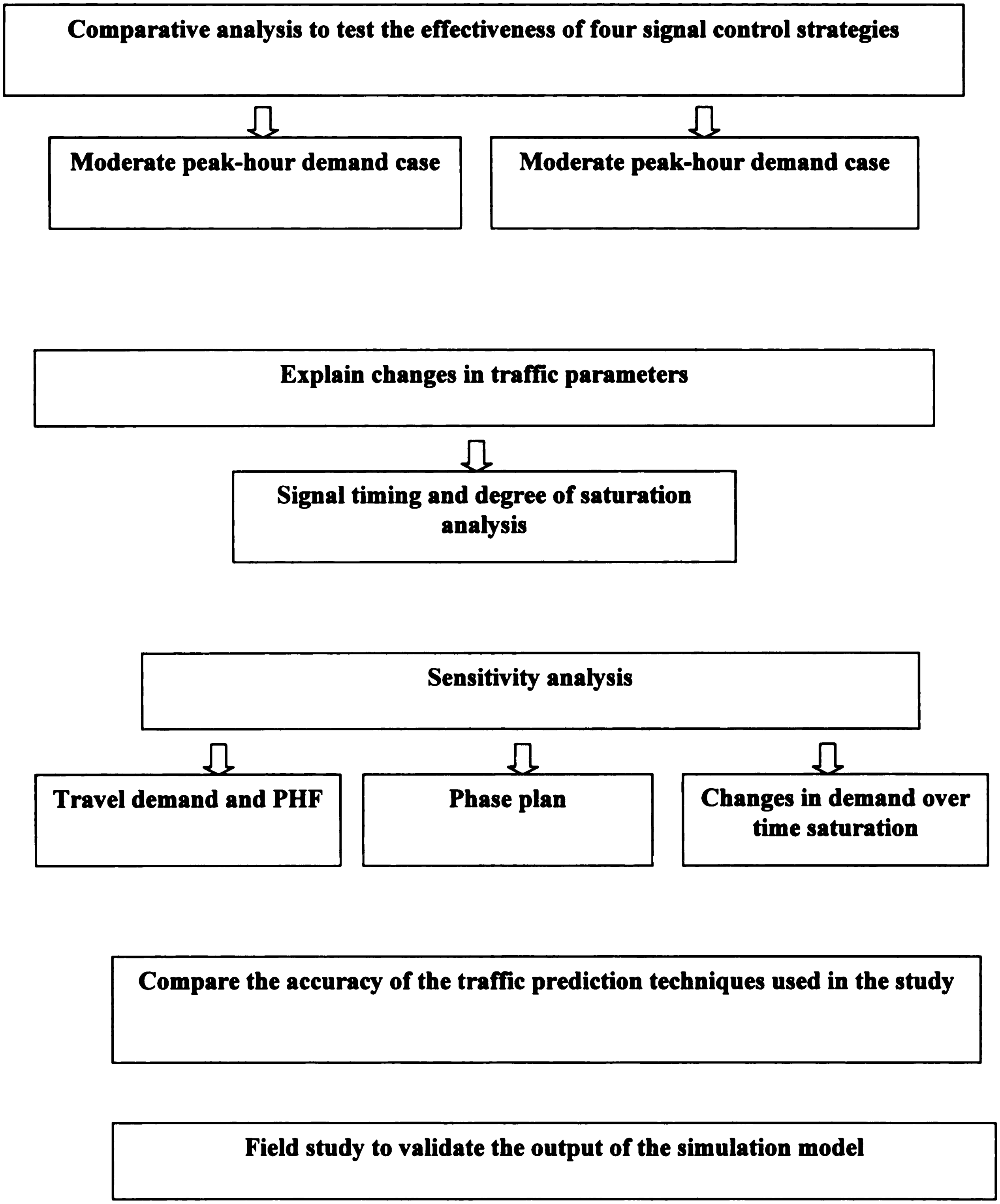

Figure 4.1 Study Plan

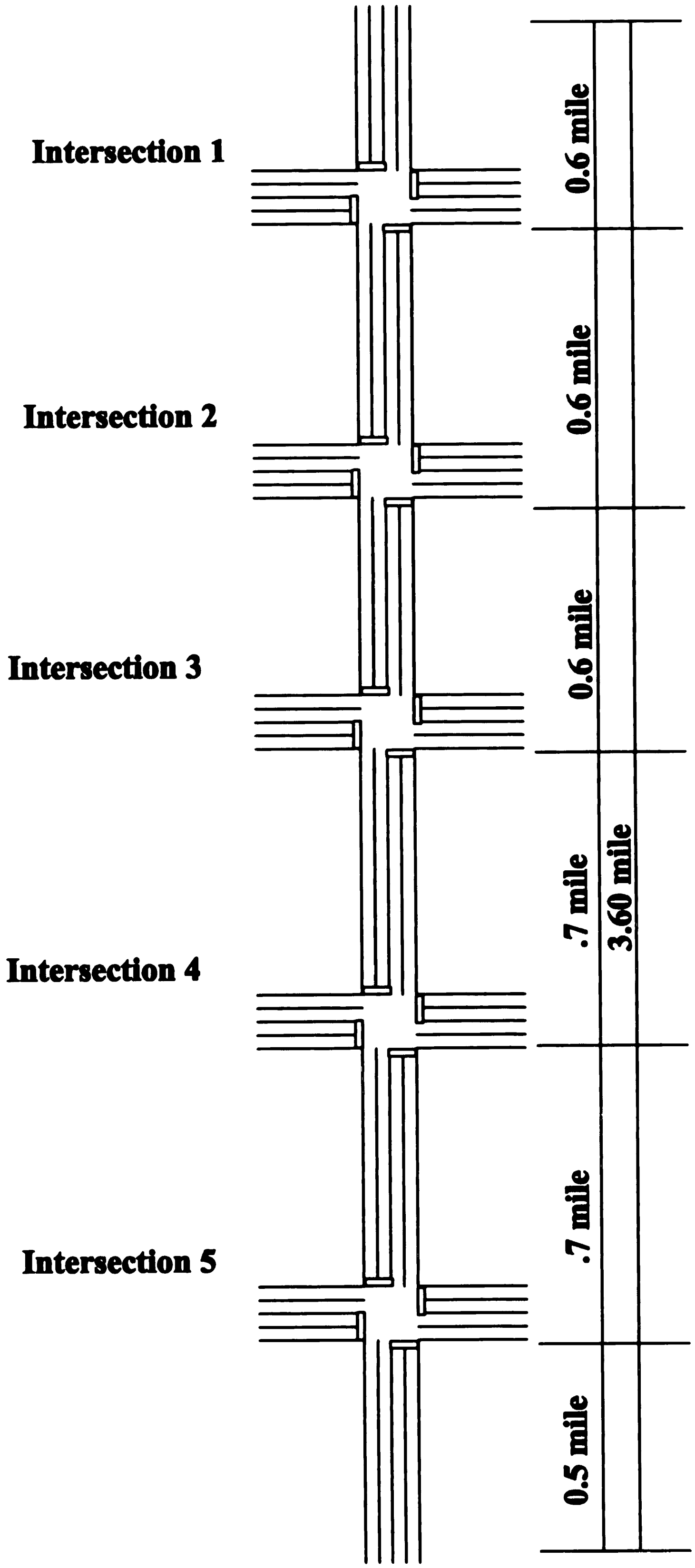

Figure 4.2 Geometric Configuration of the Corridor

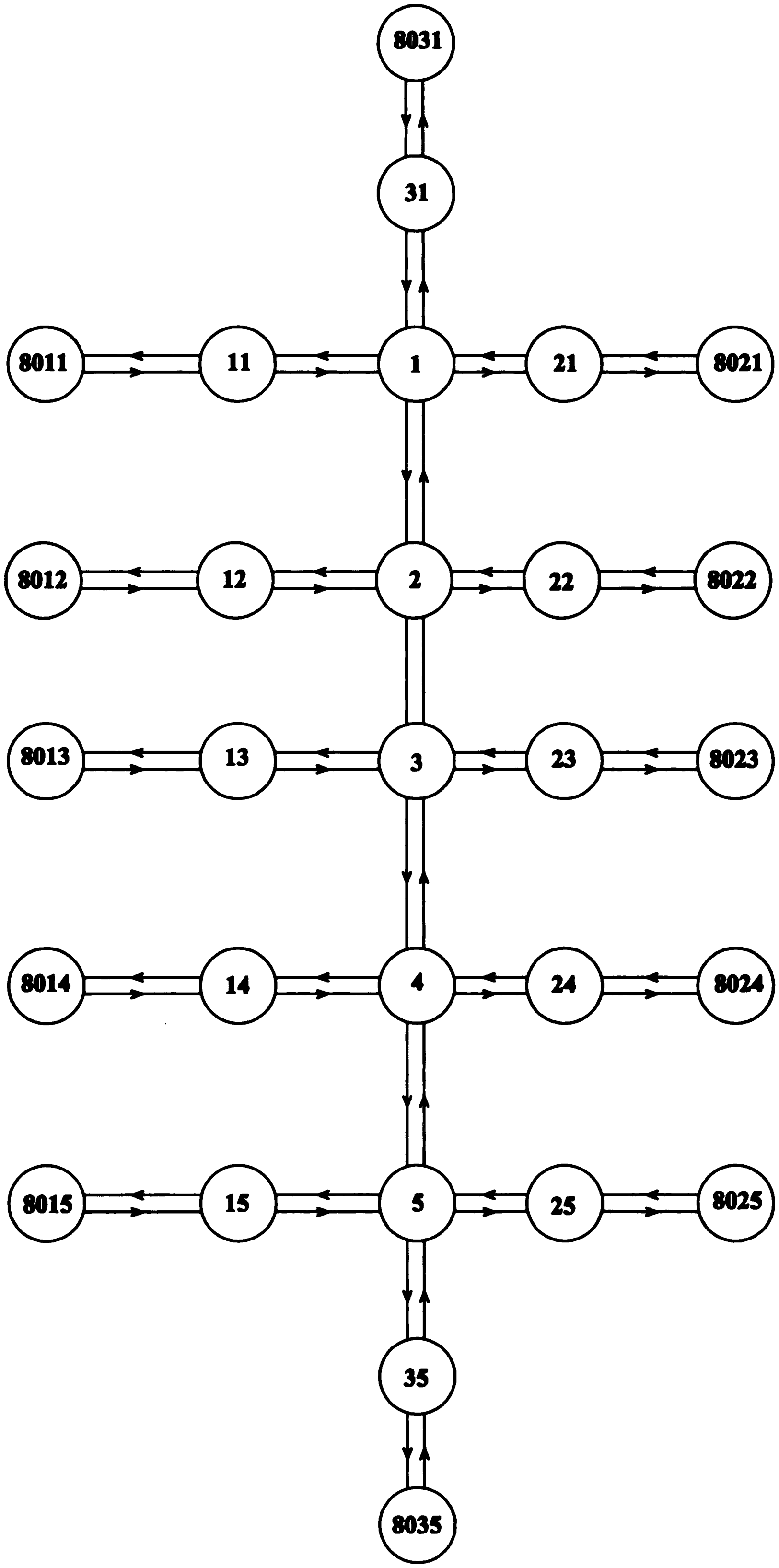

Figure 4.2 Link Node Diagram for the Corridor

4.3 Traffic Demand Levels

Demand levels were selected to represent different peak-period traffic conditions in urban areas. Two demand levels with different peaking factors, representing moderate and high peak-periods were selected for both the major corridor and minor streets. The traffic volumes and peak-hour factors were based on an analysis of the volumes on Orchard Lake Road, an urban corridor in Oakland County, Michigan. The distribution of the traffic over an hour period for different demand levels is presented in Figure 4.4. As in most corridors in urban areas, during any specific time period, the demand varies between the two directions of travel along the corridor (peak and non-peak directions). The demand for the non-peak direction of travel was set at 75% of the peak-direction demand for both major and minor streets.

4.4 Measures of Effectiveness

The improvement brought about by changes in traffic control can be assessed using different measures. To compare the effectiveness of alternative control strategies, several measures of effectiveness that are appropriate for characterizing traffic flow conditions were studied. The MOEs selected for this study are:

1-Average corridor travel time

2-Total corridor travel time

3-Average stopped delay per vehicle per approach

4-Average stopped delay per vehicle for the intersections.

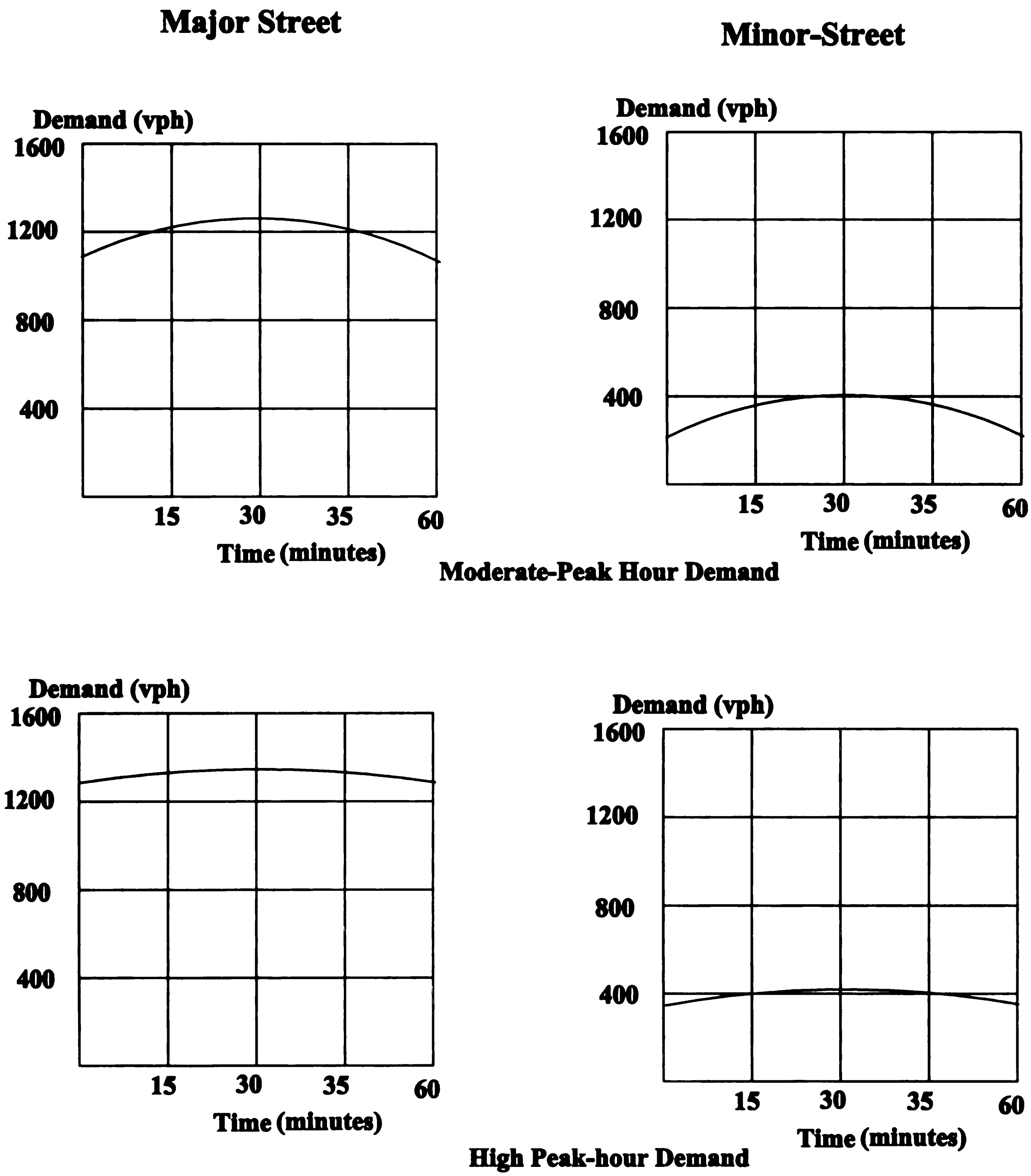

Figure 4.4 Deamnd Level For Major Corridor and Minor Streets

4.4.1 Average corridor travel time

The efficiency of any traffic signal control system can be measured in terms of the difference in total time taken by a vehicle to complete a designated trip from a specific origin to a specific destination. Travel time studies are commonly used to measure arterial levels of service. The travel time characteristics within a corridor are very useful as an indicator of overall system performance. Travel time is an indication of delay on a broader scale than intersection delay, which only measures the delay at a single intersection or specific approach. Average link travel time, as reported by CORSIM, is the average travel time of vehicles that have completed their trip across the link.

4.4.2 Total corridor travel time

Total travel time is the sum of the travel time for all individual vehicles completing their trips. This measure considers the number of vehicles, distance of travel, speed and delay associated with travel. Total travel-time may be the most appropriate indicator for a system operator, who seeks to minimize the overall system-wide travel time.

4.4.3 Average delay per approach

Delay reflects the traffic operations as perceived by system users. The research study assessed the difference in average approach and stopped delay. The term approach and stopped delay, as used in this study, are defined as the following:

Average Approach Delay: is the length of time that vehicles approaching the intersection from a particular direction are delayed due to a red signal and/or stopped queue which prohibits their free flow travel through the intersection. Free flow travel time represents the time the vehicle takes to travel through the intersection approach section using the average speed of vehicles on the approach. Average approach delay is the sum of the approach delay for all vehicles divided by the total number of vehicles.

Average Stopped Delay: is the amount of time that a vehicle spends in a queue plus the time it takes to travel from its position in the queue to the approach stop line. This is the measure used in the Highway Capacity Manual to assign a Level of Service (LOS) rating to signal controlled intersections. Average stopped delay is the sum of stopped delay divided by the total number of vehicles.

4.4.4 Average intersection delay

The weighted-average of the intersection delay for all approaches is a good indicator of how well the control strategy served the overall intersection rather than for a single approach.

4.5 Analyzing CORSIM Output/ simulation iteration

CORSIM uses the ratio of sample means of observations to calculate the average per vehicle parameters (average travel time per vehicle, average number of stops per vehicle, average speed, and average delay time per vehicle). Each of these MOEs is the ratio of the means of two random variables X and Y. For example, the average delay per vehicle is obtained by dividing the total delay time accumulated on each link by the total number of vehicles discharged from the link. To make valid statistical statements about these parameters, one must apply statistical techniques to the model output that are based on ratio estimates. The most common methods to analyze the output of the stochastic simulation models are the batch mean and the replication methods.

The batch mean method is performed by running the simulation model for one long run, then dividing it into smaller time intervals (batches). For each batch, statistics are collected and variability among batches is used to build a confidence interval on the simulation output parameters. The advantage of the batch means method is that it requires only one initialization period, which reduces the simulation execution time significantly. However the length of the intervals should be set long enough to reduce the correlation among batches.

The replication method is performed by running the simulation for a number of independent runs, each run has its unique initialization time (warm-up period) until the system reaches an equilibrium condition. After the warm-up period, statistics on system performance are collected. Each run should have a unique random number seed to generate random variation among the runs.

Gafarian and Halati (1986), developed a statistically valid method for using the value of the ratio X/Y as a point estimate for the ratio x/y. The method provides a measure of the accuracy of the estimates in term of confidence intervals. The development of the confidence interval for the estimate x/y is based on the observations $\{(X_i/Y_i), i=1, 2, \ldots ,n)\}$ of all independent runs of the simulation model. The validity of this method depends on how well the assumptions made in the derivation are met by the CORSIM model. These assumptions are: system in steady state, independent runs, and normality of X and Y.

Steady state in the CORSIM is achieved by the warm-up procedure. To obtain independent runs, each run is started with a different seed number. The only assumption that is only approximately met is that of normality, and it has been shown that the method is not sensitive to this requirement, (Gartner and Hou 1994)

According to the Gafarian and Halati study, (Gafarian1986), when the number of simulation runs exceed 40, then the coverage probability for the 95% confidence intervals of the estimator will approach 0.95. In a comparative study for evaluating alternative control strategies, Gartner and Hou (Gartner, 1992) stated that a sample size of four simulation runs would provide 0.80 coverage probability of the 95% confidence intervals. In this research, a pilot study was used to determine the optimal number of iterations to be applied throughout the course of the simulation model and to study the variability associated with the randomness of the traffic.

4.6 Number of simulation iterations

The CORSIM model requires a random seed number for the random number generators that govern the flow of vehicles into the network. CORSIM generates different traffic patterns of traffic flow and responses to traffic choices. The user defines two random number seeds. The first one is used for generating vehicles for traffic streams. The second one is used for all time dependent stochastic processes. By keeping all other modeling elements constant and varying the seed number for traffic flow the variance of performance measures due to variation in traffic patterns can be illustrated.

The preliminary pilot study consisted of 40 identical CORSIM input data sets in terms of geometry, traffic control, traffic volumes, and turning percentages.

However, different random number seeds were used in each run. The simulation runs produced 40 simulation outputs. Average corridor travel time for the 40 runs is presented in Figure 4.5. The output shows the variability of the travel time values due to the randomness of the traffic patterns. The graph indicates that the randomness of the traffic pattern accounts for variations of up to ±1.8% of the mean for corridor travel time (Figure 4.5).

The data from the 40 simulation runs were further analyzed to determine the number of iterations required for each simulation run. Based on the cumulative averages from iteration 1 to 40, the average values were plotted against the number of iterations included, Figure 4.6. The graph show that the cumulative average improves significantly after 4 to 5 iterations. Accordingly, it was decided to conduct at least five iterations with different seed numbers for each case.

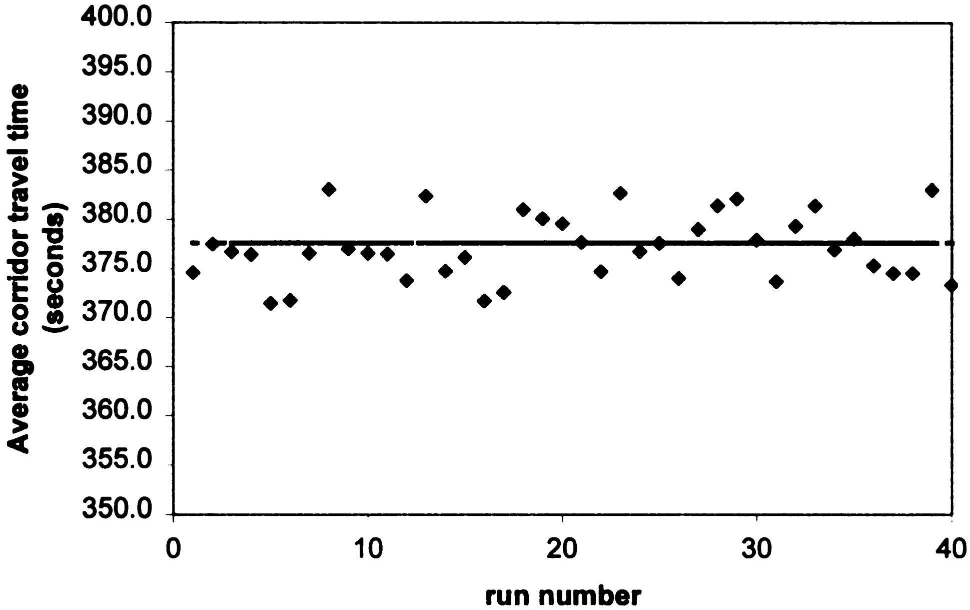

Figure 4.5 Average Corridor Travel time for 40 simulation iterations

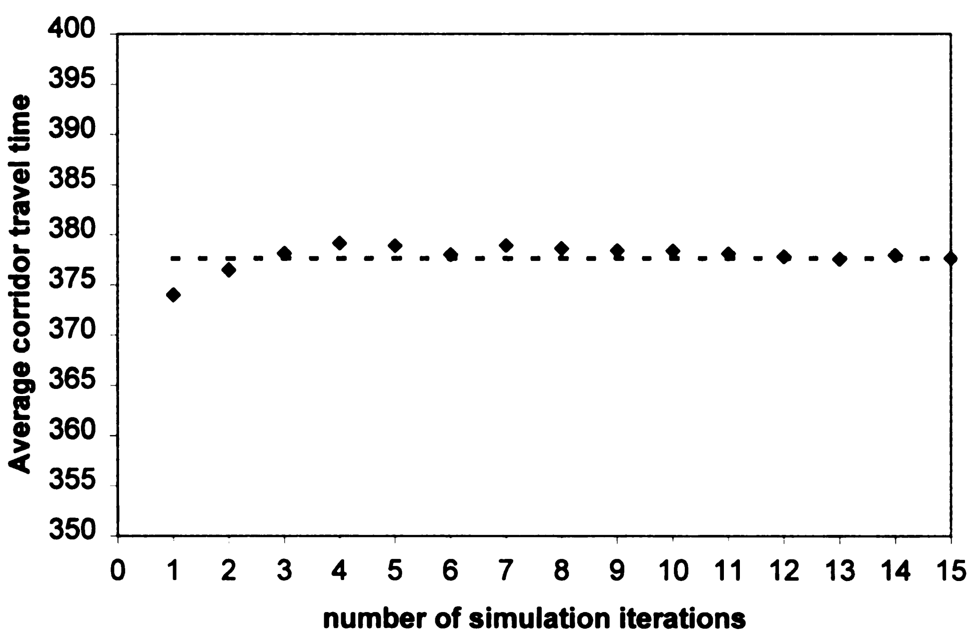

Figure 4.6 Number of simulation iterations

Chapter 5

ANALYSIS AND RESULTS

The output of the simulation study is analyzed and presented in this chapter. The chapter is divided into four main sections. The first section includes the results of a comparison of the effectiveness of different control strategies under different demand levels. A common set of traffic parameters, generated from the simulation runs, was used to measure and compare the effectiveness of these control strategies. A comparison of the signal timing characteristics under different control strategies is presented in the second section. The signal timing analysis was conducted to examine how the dynamic green time allocation in both actuated and adaptive signal control systems contributed to the changes in the traffic parameters.

The benefits of the adaptive control systems over fixed time signals were examined under four different demand levels and peak-hour factors for both major-street and minor-street traffic. The results of a sensitivity analysis are presented in the third section of this chapter. Changes in the adaptive control system benefits resulting from adding two left-turn phases to the signal operation was also examined. Finally, a comparison of the accuracy of the two prediction techniques used in the adaptive control systems in this study is presented in the fourth section of this chapter.

5.1 Comparative analysis of signal control strategies

The CORSIM model was used to simulate traffic conditions under four different control strategies; fixed-time signals, actuated control signals, and two different adaptive control systems. The effectiveness of these control strategies was examined for under two different demand levels, representing moderate and high peak-hour cases in an urban corridor. Traffic parameters, generated from the simulation runs, were used as measures of effectiveness in the comparative analysis. The parameters included were; average corridor travel time, total corridor travel time, and average delay per approach and for the intersection.

5.1.1 Corridor travel time

The efficiency of any traffic signal control system can be measured in terms of the total time taken for all vehicles to complete their trip throughout the corridor. The travel time characteristics within a traffic network are very useful as an indicator of the overall system performance. Travel time studies are commonly used to measure corridor levels of service and to compare the effectiveness of alternative signal control strategies.

Figure 5.1 shows the average corridor travel time under four different control strategies for the moderate peak-hour demand case. The results are aggregated over 5-minutes intervals. The results are also presented in Table 5.1. All three control strategies showed an improvement in average corridor travel time over fixed-time signal control. The average corridor travel time was 399.02 seconds/vehicle under fixed time control. This average decreased to 385.92 seconds/vehicle, 382.58 seconds/vehicle, and 383.57 seconds/vehicle under actuated, adaptive_1 and adaptive_2 controls, respectively.

The overall savings in corridor travel time were 3.28% under actuated-signal control, 4.12% under adaptive_1 control, and 3.87% under adaptive_2 control. For the three control strategies, the savings in travel time decreased as the traffic demand approached its 15-minute peak period, then increased again as traffic demand gradually decreased. The savings in travel time was as high as 8.45% in periods of low demand and as low as 0.0% during the peak 15-minute period, Table 5.2. The average savings in travel time during the first 15 minutes of the peak-period were 6.05%, 7.13%, and 6.77% for the three control strategies, respectively. These values were 0.59%, 0.96%, and 0.73%, respectively, during the peak 15-minute period in the peak-hour.

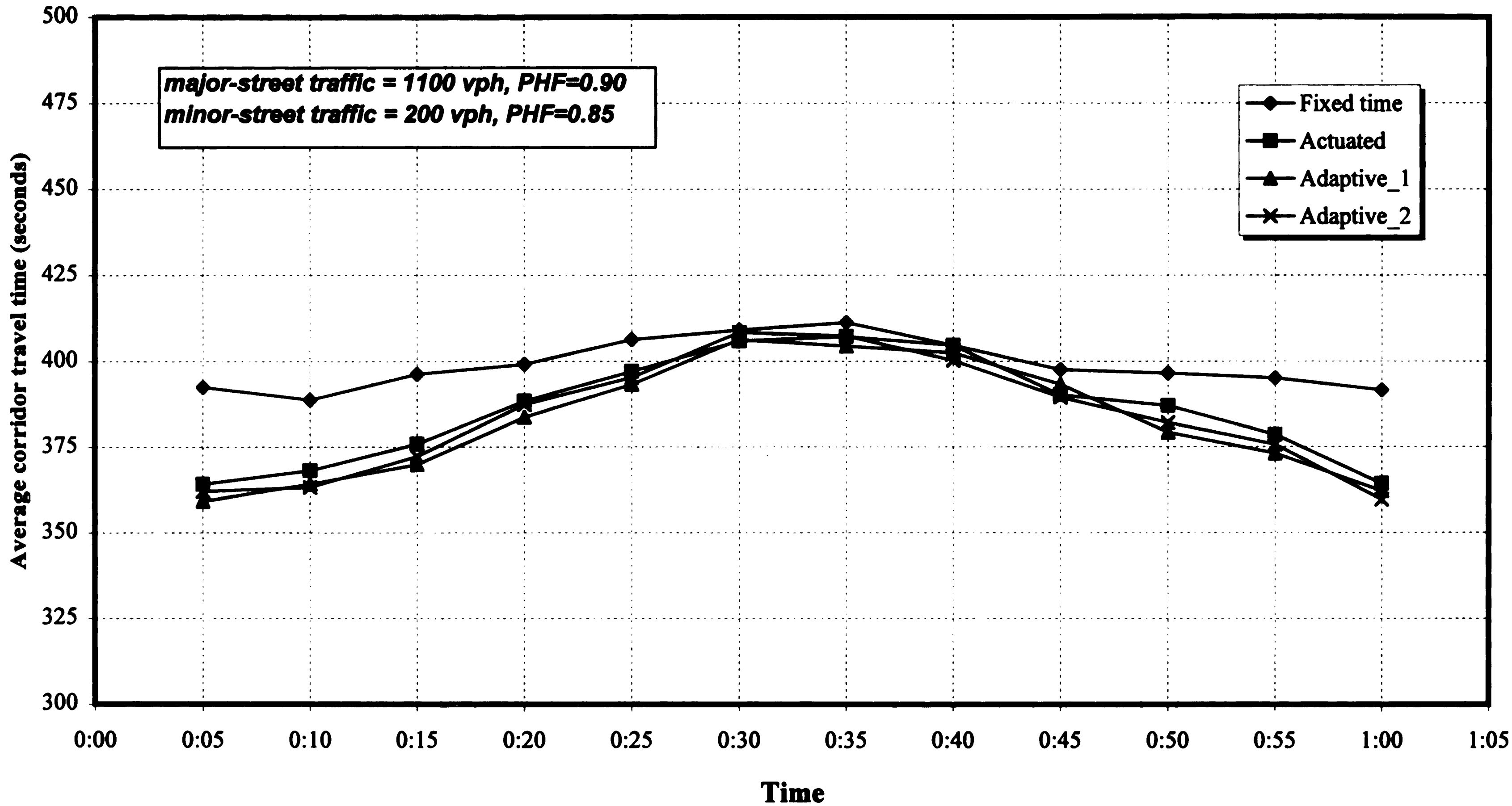

Figure 5.1 Average corridor travel time under different signal control strategies (moderate rush-hour case)

Table 5.1 Average corridor travel time* under different control strategies (moderate peak-hour demand)

Time	Fixed-Time		Actuated		Adaptive_1		Adaptive_2	
	Average	S.D.	Average	S.D.	Average	S.D.	Average	S.D.
0:05	392	3.29	364	3.11	359	3.13	362	2.60
0:10	389	3.37	368	3.62	364	3.43	363	3.12
0:15	396	2.88	376	3.59	370	3.25	372	2.63
0:20	399	2.37	388	2.91	384	2.81	387	2.70
0:25	406	3.54	397	3.85	393	3.56	395	3.40
0:30	409	3.74	406	3.43	406	3.12	408	3.51
0:35	411	3.63	407	3.05	404	2.97	407	3.09
0:40	405	3.41	405	3.24	402	2.62	400	3.40
0:45	398	2.95	390	3.65	393	3.52	389	3.83
0:50	397	3.12	387	3.62	379	3.14	382	3.30
0:55	395	4.06	379	2.57	373	3.00	376	3.52
1:00	392	3.05	364	3.84	362	2.99	360	2.83

* Average corridor travel time in seconds/vehicle

Table 5.2 Savings* in corridor travel time under different control strategies (moderate peak-hour demand)

Time	Actuated		Adaptive_1		Adaptive_2	
	Savings	Percent	Savings	Percent	Savings	Percent
0:05	28.15**	7.17	33.15**	8.45	30.26**	7.71
0:10	20.61**	5.30	24.50**	6.30	25.48**	6.55
0:15	20.40**	5.15	26.41**	6.66	24.07**	6.07
0:20	10.77**	2.70	15.46**	3.87	11.79**	2.95
0:25	9.24**	2.27	13.00**	3.20	10.99**	2.71
0:30	3.20**	0.78	2.78	0.68	0.69	0.17
0:35	4.11**	1.00	6.90**	1.68	4.00**	0.97
0:40	0.00	0.00	2.20	0.54	4.33**	1.07
0:45	7.39**	1.86	4.26**	1.07	8.09**	2.04
0:50	9.54**	2.41	17.31**	4.37	14.43**	3.64
0:55	16.55**	4.19	21.99**	5.57	19.42**	4.92
1:00	27.33**	6.98	29.36**	7.50	31.90**	8.15
Average	13.11**	3.28	16.44**	4.12	15.45**	3.87

*** savings over fixed-time signal control (seconds)**

**** the difference between the two means is statistically significant at $\alpha = 0.05$**

Figure 5.2 and Tables 5.3 and 5.4 present the average corridor travel time under four different control strategies for the high peak-hour demand case. The overall savings in corridor travel time were 1.11% under actuated-signal control, 1.23% under adaptive_1 control, and 1.43% under adaptive_2 control. Similar to the moderate peak-hour demand, the savings in travel time were higher during the low demand period in the first and last 15-minute periods, and decreased as traffic reached its peak 15-minute period. The average savings in travel time during the first non-peak periods were 1.11%, 1.63%, and 2.2% for the three control strategies, respectively. These values were 0.59%, 0.37%, and 0.64%, respectively, during the peak 15-minute period. The differences in average corridor travel time among the three control strategies were not significant during either the moderate or high demand cases.

5.1.2 Total travel time

Total travel time is the sum of the travel time of all individual vehicles completing their trips throughout the corridor. This measure considers the number of motorists, distance of travel, speed, and delay associated with the trips. Figures 5.3 and 5.4 show the total travel time under different control strategies for the moderate and high demand cases, respectively. Similar to the observations made from the average travel time graphs, all three control strategies showed a decrease in total travel time over fixed-time control. The decrease, however, was not significant in the high peak-hour demand case.

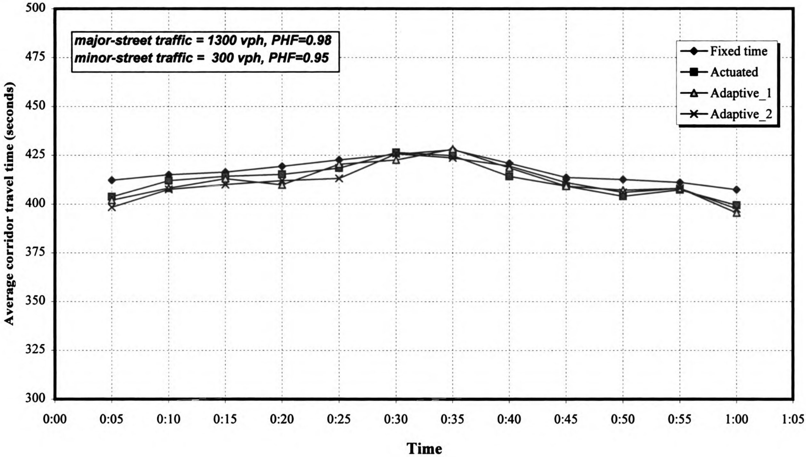

Figure 5.2 Average corridor travel time under different signal control strategies (high rush-hour)

Table 5.3 Average corridor travel time* under different control strategies (high peak-hour demand)

Time	Fixed-Time		Actuated		Adaptive_1		Adaptive_2	
	Average	S.D.	Average	S.D.	Average	S.D.	Average	S.D.
0:05	412	2.53	404	2.38	402	2.39	398	1.96
0:10	415	2.59	412	2.80	408	2.63	408	2.38
0:15	416	2.19	414	2.77	413	2.49	410	1.98
0:20	419	1.77	415	2.21	410	2.13	412	2.05
0:25	423	2.72	418	2.99	420	2.74	413	2.61
0:30	426	2.89	426	2.64	423	2.38	426	2.70
0:35	428	2.80	425	2.33	428	2.27	424	2.36
0:40	421	2.62	414	2.48	419	1.97	420	2.61
0:45	414	2.25	409	2.82	409	2.72	411	2.96
0:50	413	2.39	404	2.79	407	2.40	406	2.53
0:55	411	3.16	407	1.94	408	2.29	408	2.71
1:00	407	2.33	399	2.97	396	2.28	398	2.15

* Average corridor travel time in seconds/vehicle

Table 5.4 Savings* in corridor travel time under different control strategies (high peak-hour demand)

Time	Actuated		Adaptive_1		Adaptive_2	
	Savings	Percent	Savings	Percent	Savings	Percent
0:05	8.45**	2.05	10.21**	2.48	13.80**	3.35
0:10	3.00**	0.72	6.77**	1.63	7.44**	1.79
0:15	2.16	0.52	3.34	0.80	6.34**	1.52
0:20	4.10	0.98	9.47**	2.26	7.32**	1.74
0:25	4.24	1.00	2.20	0.52	9.47**	2.24
0:30	-0.91	-0.21	2.89	0.68	-0.23	-0.05
0:35	2.99	0.70	-0.33	-0.08	4.16	0.97
0:40	6.71**	1.59	2.29	0.54	1.39	0.33
0:45	4.43	1.07	4.43**	1.07	2.60	0.63
0:50	8.60**	2.09	5.41**	1.31	6.58**	1.59
0:55	3.74**	0.91	3.07**	0.75	3.07**	0.75
1:00	7.98**	1.96	11.85**	2.91	9.85**	2.42
Average	4.63**	1.11	5.13**	1.23	5.98**	1.43

***Savings over fixed-time signal control (seconds)**

**** The difference between the two means is statistically significant at $\alpha = 0.05$**

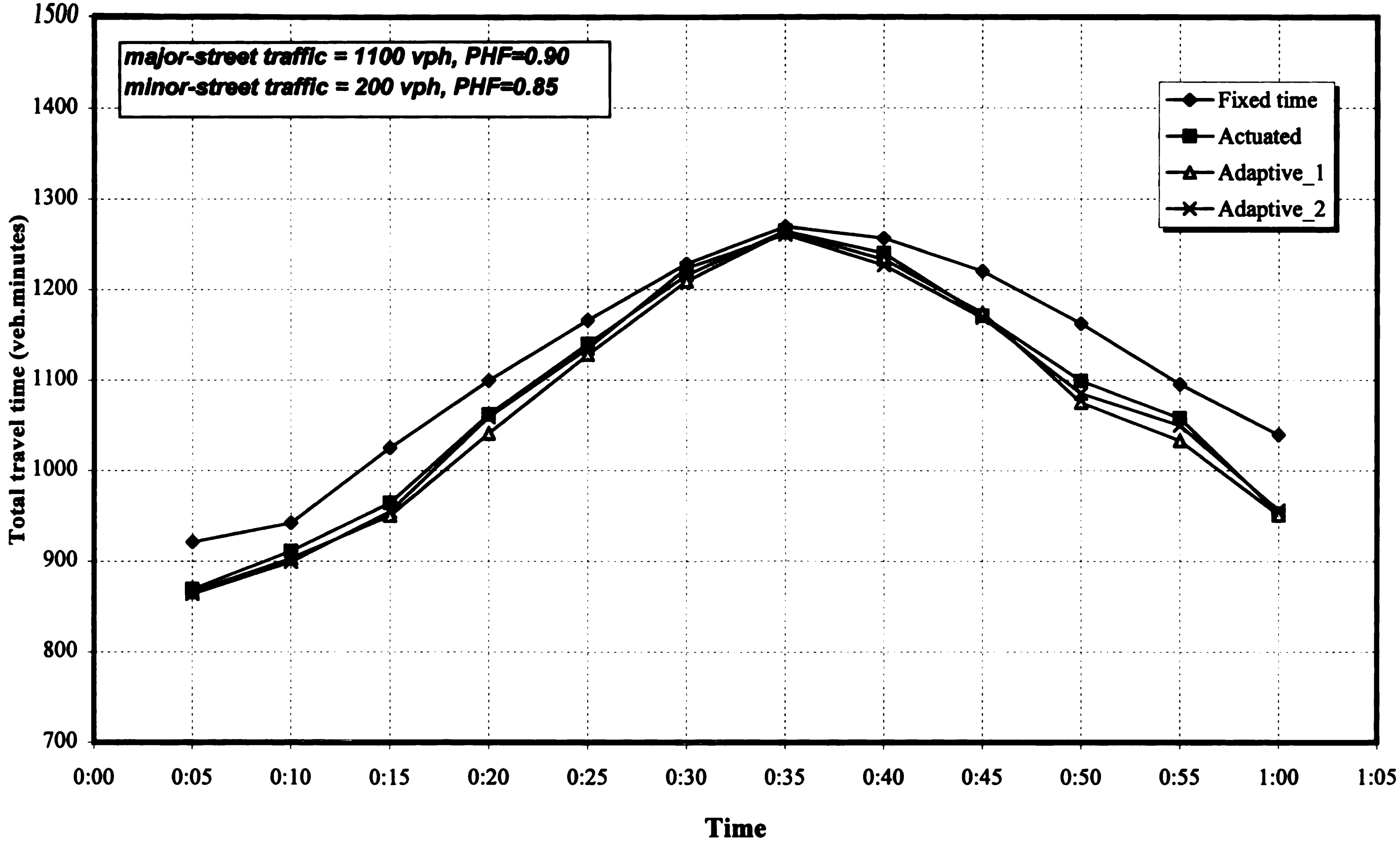

Figure 5.3 Total travel time under different signal control strategies
(moderate rush-hour case)

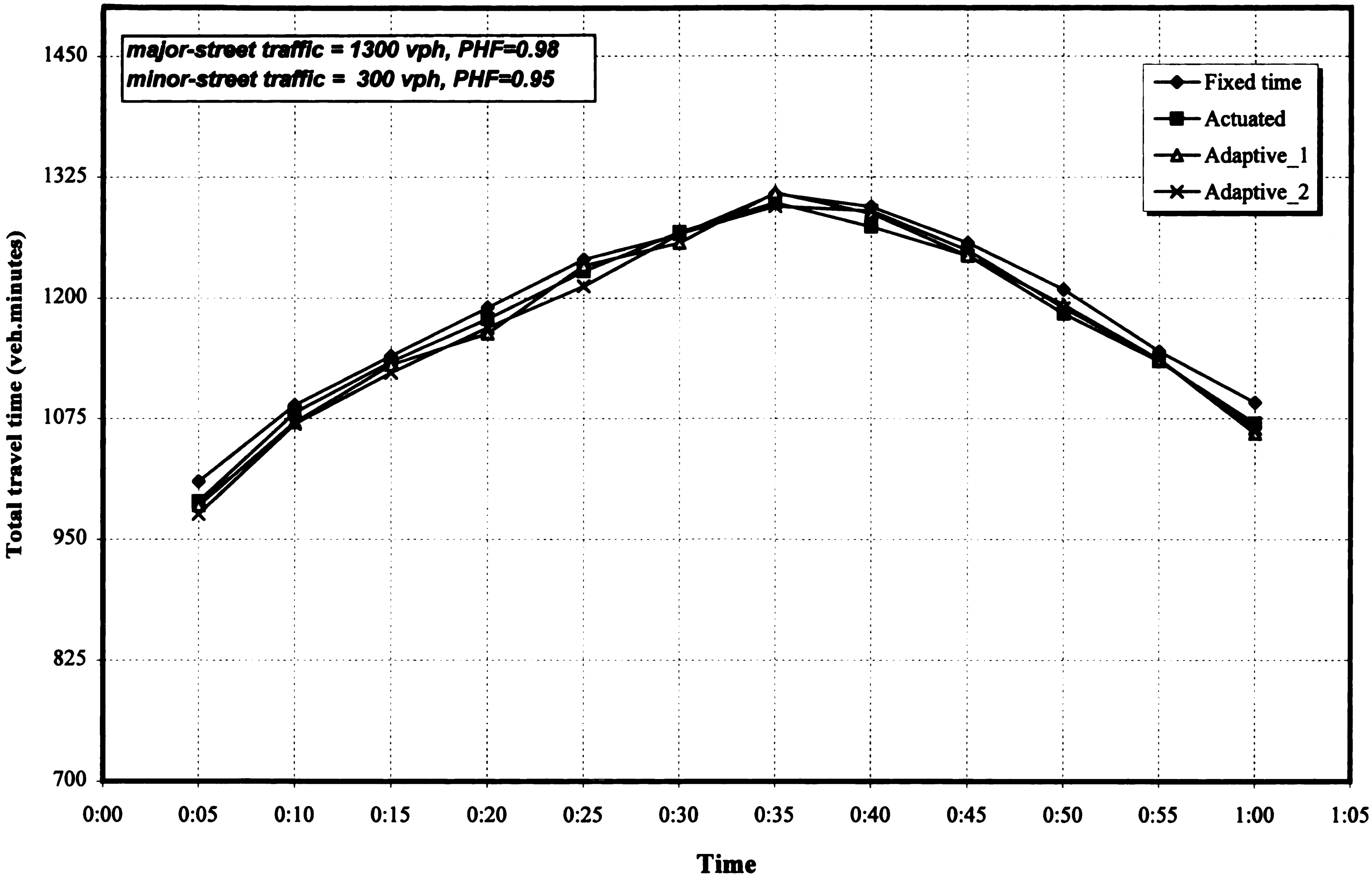

Figure 5.4 Total travel time under different signal control strategies (high rush-hour)

Tables 5.5 and 5.6 show the total travel time under different control strategies for the moderate and high demand cases, respectively. The tables list the average total travel time for the full one-hour period and for the peak and non-peak 15-minute periods. For the moderate peak-hour demand case, the overall decrease in total travel time was 3.57% under actuated control, 4.46% under adaptive_1 control, and 4.05% under adaptive_2 control. These values were 0.91%, 1.30%, and 1.17%, respectively during the peak 15-minute period. For the high peak-hour demand case, the decrease in total travel time averaged between 0.38% to 2.18% under the three control strategies with no significant difference between peak and non-peak 15-minute periods. Again, the difference in total travel time among the three control strategies was not significant for either the high or the moderate demand cases.

5.1.3 Intersection Delay

Corridor travel time is a measure of the overall system performance for the main traffic along the corridor. It does not, however, address the impacts of the control strategies on the minor street traffic at the intersections along the corridor. An intersection delay analysis was necessary to examine the changes in approach delay under different control strategies. The term approach delay, as used in this study, is defined as the length of the time that vehicles approaching the intersection from a particular direction are delayed due to a red signal and/or stopped queue which prohibits their free flow travel through the intersection. Intersection delay parameters

Table 5.5 Total corridor travel time (vehicle-minutes) under different control strategies (moderate peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	1118.80	1078.90	39.91	3.57	1068.89	49.92	4.46	1073.54	45.27	4.05
15 minutes (peak)	1251.50	1240.12	11.38	0.91	1235.22	16.28	1.30	1236.91	14.59	1.17
15 minute (non-peak)**	962.68	914.67	48.02	4.99	906.33	56.35	5.85	905.85	56.84	5.90

*Savings in total corridor travel time over fixed-time signal control
** the 15-minute period with the least amount of traffic during the one-hour period

Table 5.6 Total corridor travel time (vehicle-minutes) under different control strategies (high peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	1186.51	1173.5	13.01	1.10	1172.38	14.12	1.19	1170.03	16.47	1.39
15 minutes (peak)	1289.04	1280.02	9.02	0.70	1284.17	4.87	0.38	1283.6	5.44	0.42
15 minute (non-peak)**	1079.67	1068.17	11.50	1.06	1062.35	17.31	1.60	1056.1	23.57	2.18

*Savings in total corridor travel time over fixed-time signal control
** the 15-minute period with the least amount of traffic during the one-hour period

for an intermediate intersection along the corridor (intersection 3) are presented in this section. The average delay per approach output are the delay for the northbound traffic (peak direction for the major-street traffic) and for the westbound traffic (peak direction for the minor-street traffic). Average intersection delay, however, is the weighted-average of the delay in all four approaches.

5.1.3.1 Average approach delay for major-street traffic

Figures 5.5 and 5.6 show the average approach delay for the major-street traffic under different control strategies for both the moderate and high demand cases, respectively. Similar to average and total corridor travel time, average approach delay for the major-street traffic improved under the three control strategies. The reduction in the major-street approach delay explains the reduction reported in both average and total corridor travel time. Again, the reduction in intersection delay is higher in the moderate peak-hour demand case than in the high peak-hour demand case.

Tables 5.7 and 5.8 present the average approach delay data for major-street traffic under different control strategies. The tables present the average delay over the full one-hour analysis period, the peak 15-minute period, and the non-peak 15-minute period. In the moderate peak hour demand case, the approach delay for major-street traffic decreased by 12.78% under actuated signal control, 16.17% under adaptive_1 control and 15.16% under adaptive_2 control. These values were 0.34%, 1.77%, and

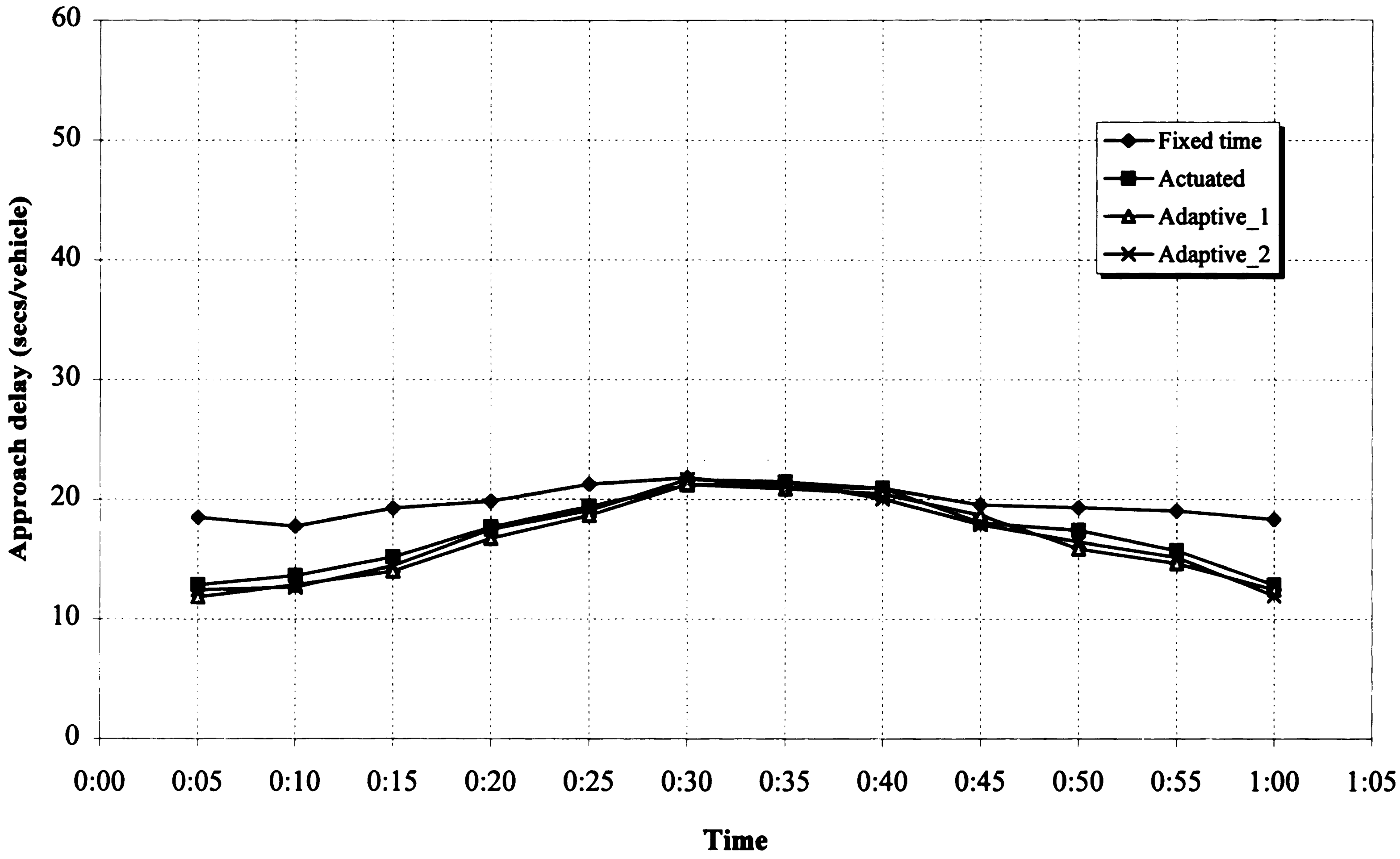

Figure 5.5 Average approach delay for major-street traffic under different control strategies(moderate peak-hour demand case)

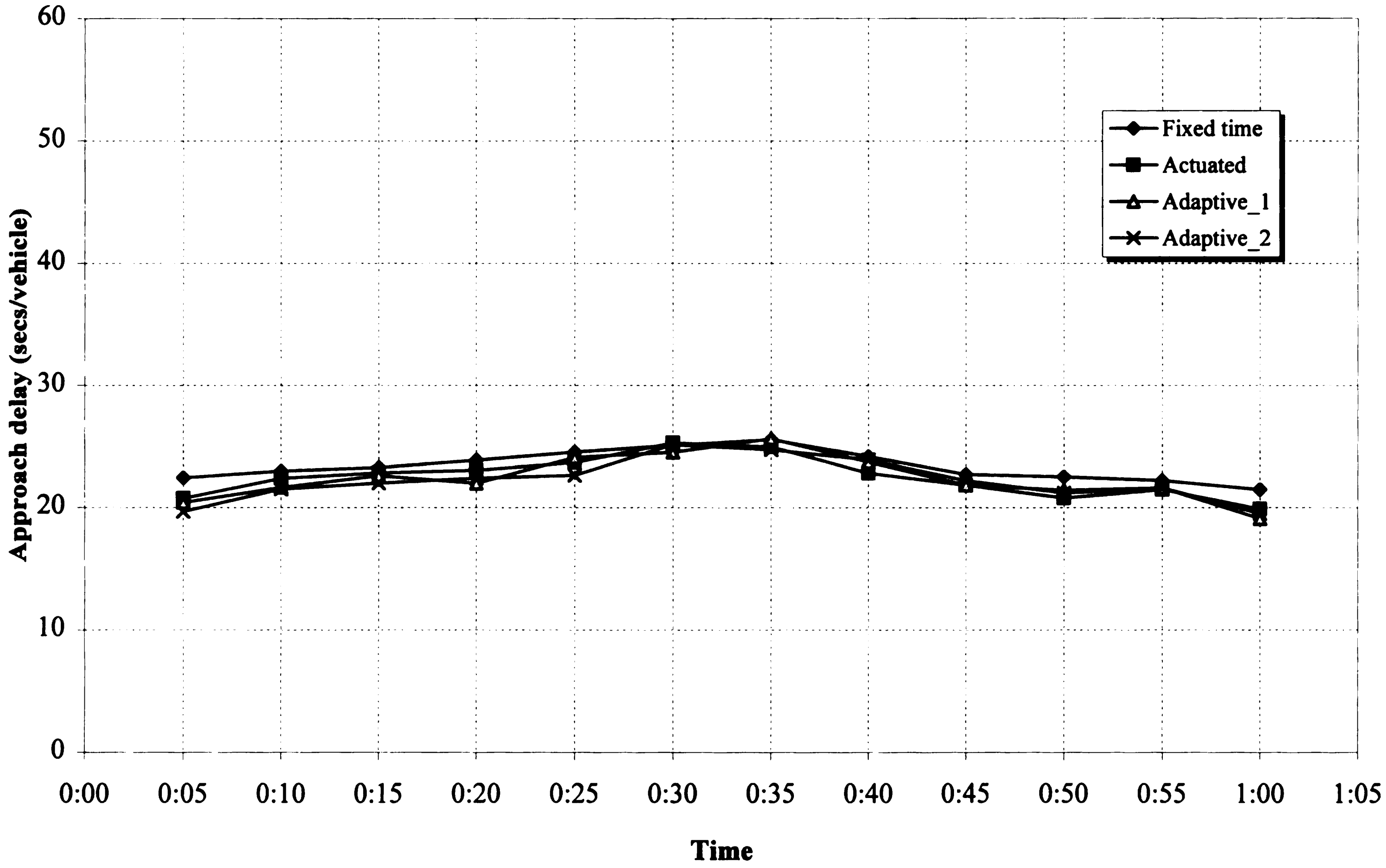

Figure 5.6 Average approach delay for major-street traffic under different control strategies (high peak-hour demand case)

Table 5.7 Average approach delay for major-street traffic under different control strategies (moderate peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	19.70	17.18	2.52	12.78	16.51	3.18	16.17	16.7	2.99	15.16
15 minutes (peak)	21.23	21.16	0.07	0.34	20.86	0.38	1.77	21.05	0.19	0.88
15 minute (non-peak)**	18.49	13.88	4.61	24.93	12.88	5.60	30.30	13.17	5.32	28.77

*Savings in approach delay over fixed-time signal control (secs / vehicle)
** the 15-minute period with the least amount of traffic during the one-hour period

Table 5.8 Average approach delay for major-street traffic under different control strategies (high peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	23.40	22.48	0.93	3.95	22.38	1.03	4.39	22.21	1.20	5.11
15 minutes (peak)	24.95	24.36	0.59	2.35	24.62	0.32	1.30	24.59	0.36	1.42
15 minute (non-peak)**	22.90	21.99	0.91	3.96	21.54	1.35	5.92	21.06	1.84	8.03

*Savings in approach delay over fixed-time signal control (secs / vehicle)
** the 15-minute period with the least amount of traffic during the one-hour period

0.88%, respectively during the peak 15-minute period. During the non-peak 15 minutes period, the reduction in intersection delay was as high as 30.30%.

In the high peak-hour demand case, the average reduction in intersection delay was 3.93% under actuated signal control, 4.39% under adaptive_1 control and 5.11% under adaptive_2 control. The average increase during the non-peak 15-minute period was as high as 8.03%, and during the peak 15-minute period was as low as 1.30%. The difference in approach delay for major-street traffic among the three control strategies was not significant.

5.1.3.2 Average approach delay for minor-street traffic

Figures 5.7 and 5.8 show the average approach delay for minor-street traffic under different control strategies for the moderate and the high demand cases, respectively. The results are also summarized in Tables 5.9 and 5.10. Average approach delay for the minor-street traffic increased under the three control strategies. The average increase in delay was 6.3% under actuated signal control, 7.15% under adaptive_1 control, and 7.79% under adaptive_2 control in the moderate demand case. These averages were 2.06%, 2.95%, and 4.22%, respectively, in the high demand case.

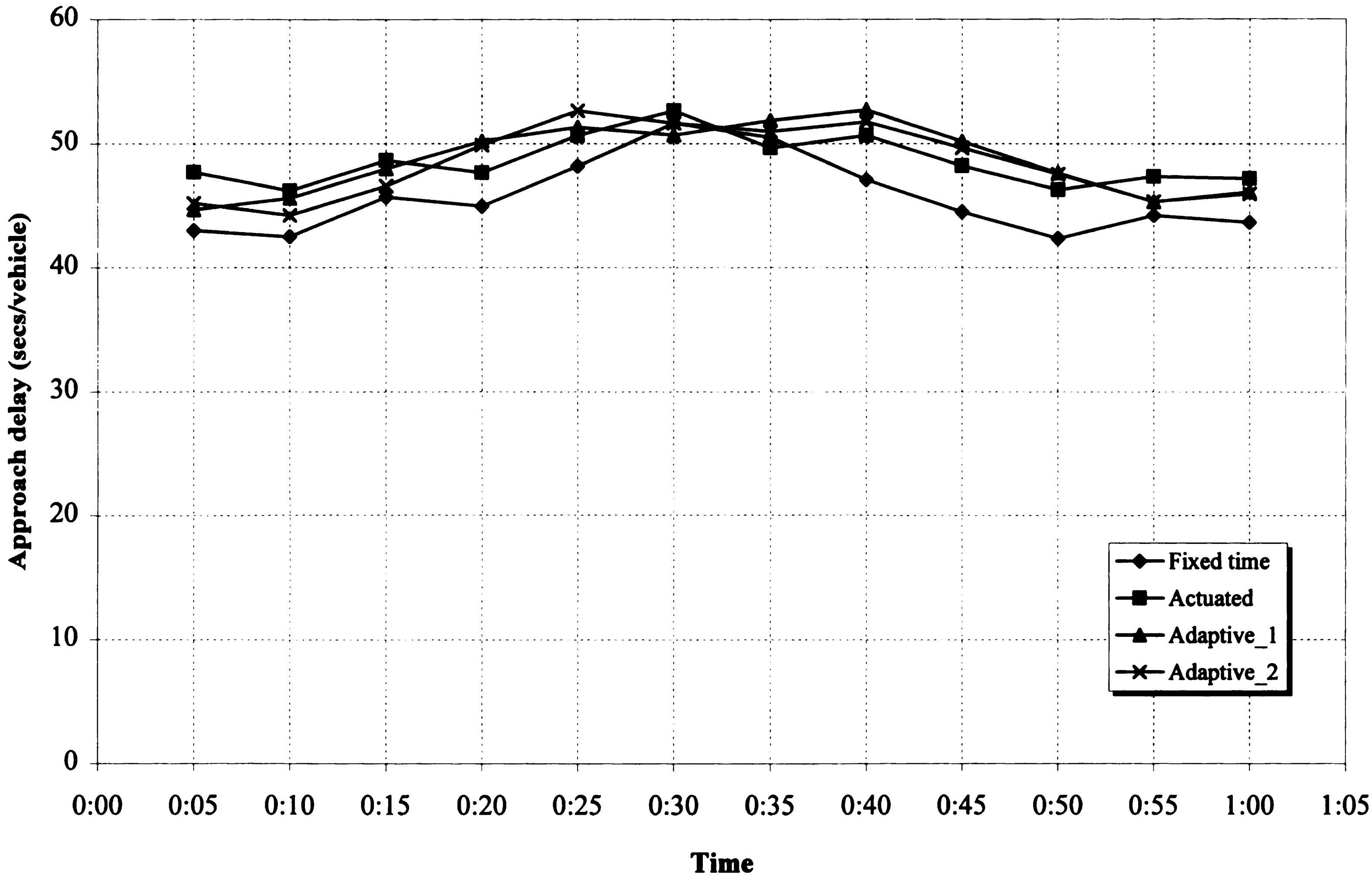

Figure 5.7 Average approach delay for minor-street traffic under different control strategies(moderate peak-hour demand case)

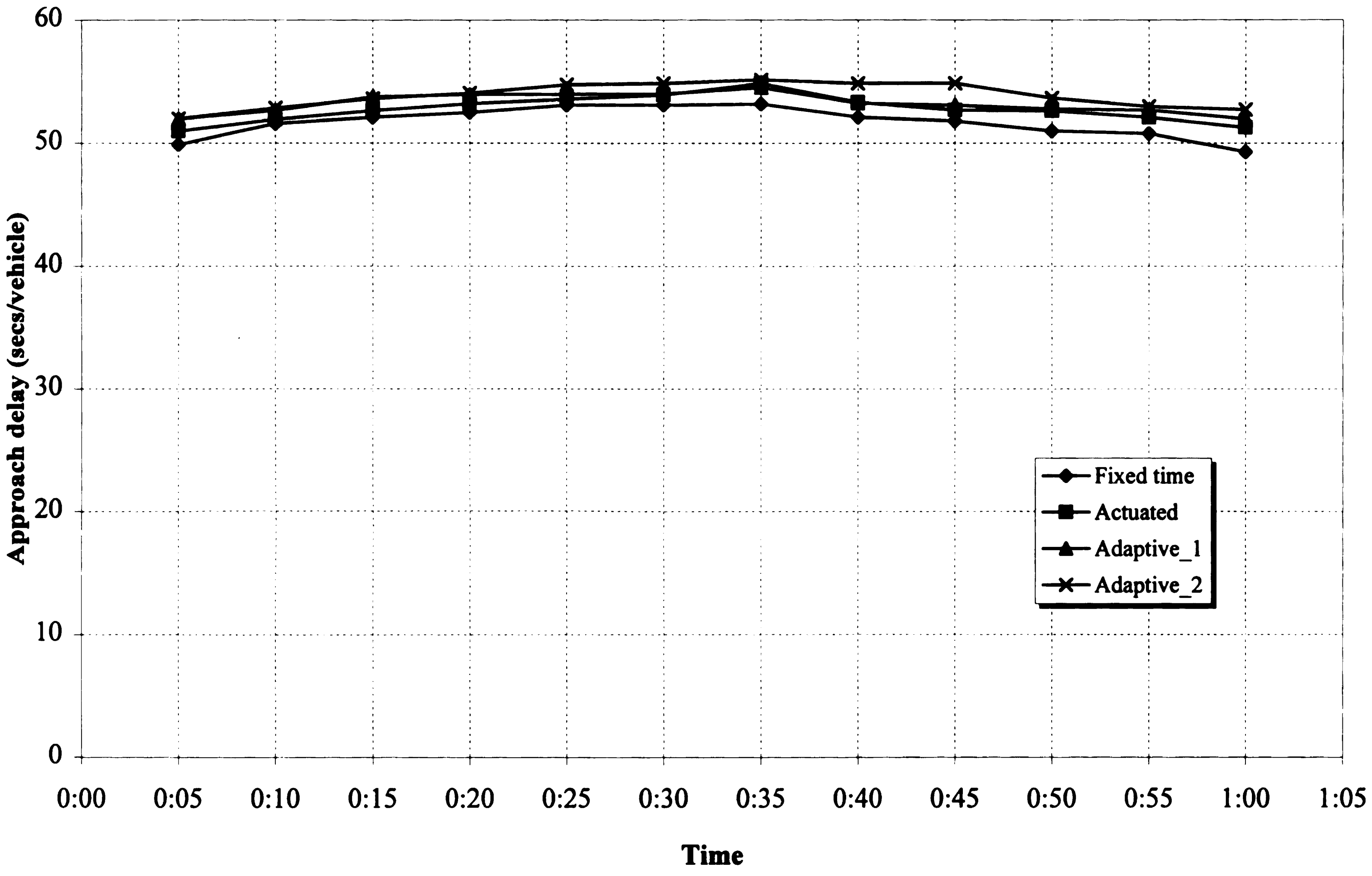

Figure 5.8 Average approach delay for minor-street traffic under different control strategies (high peak-hour demand case)

Table 5.9 Average approach delay for minor-street traffic under different control strategies (moderate peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	45.69	48.57	-2.88	-6.30	48.96	-3.27	-7.15	49.34	-3.64	-7.97
15 minutes (peak)	49.75	50.99	-1.24	-2.49	48.77	-2.02	-4.06	49.47	-1.72	-3.46
15 minute (non-peak)**	43.73	47.51	-3.78	-8.65	45.14	-3.41	-7.80	44.51	-3.78	-8.64

*Savings in approach delay over fixed-time signal control (secs / vehicle)
** the 15-minute period with the least amount of traffic during the one-hour period

Table 5.10 Average approach delay for minor-street traffic under different control strategies (high peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	51.70	52.77	-1.07	-2.06	53.23	-1.53	-2.95	53.89	-2.18	-4.22
15 minutes (peak)	52.81	54.05	-1.24	-2.35	53.94	-1.13	-2.14	54.99	-2.18	-4.12
15 minute (non-peak)**	51.20	51.86	-0.67	-1.30	52.82	-1.62	-3.16	52.84	-1.65	-3.22

*Savings in approach delay over fixed-time signal control (secs / vehicle)
** the 15-minute period with the least amount of traffic during the one-hour period

As both adaptive control strategies used a shorter average cycle length (100 seconds) during the non-peak periods, the increase in minor street delay was lower than that under actuated control, which was forced to use a cycle length of 120 seconds throughout the analysis period. The three control strategies, however, achieved the same reduction in major street approach delay.

5.1.3.3 Average intersection approach delay

The average approach delay for the intersection is the weighted average of the delay on all approaches. It takes into consideration the average delay per approach as well as traffic volumes for each approach. Thus, it is an indictor of the overall benefits of alternative signal control strategies at an intersection. Figures 5.9 and 5.10 present the average intersection approach delay under different control strategies. Tables 5.11 and 5.12 present the average over the full one-hour analysis period, the peak 15-minute period, and the non-peak 15-minute period. Overall, the intersection delay decreased by 6.4% under actuated signal control, by 8.36% under adaptive_1 control, and by 7.42% under adaptive_2 control in the moderate demand case. The delay decreased by 2.11%, 2.14% and 2.25% for the three control strategies, respectively, in the high demand case. Similar to all other parameters, the reduction in delay is higher during the non-peak 15-minute period, and decreases as traffic volumes reach their 15-minute peak period.

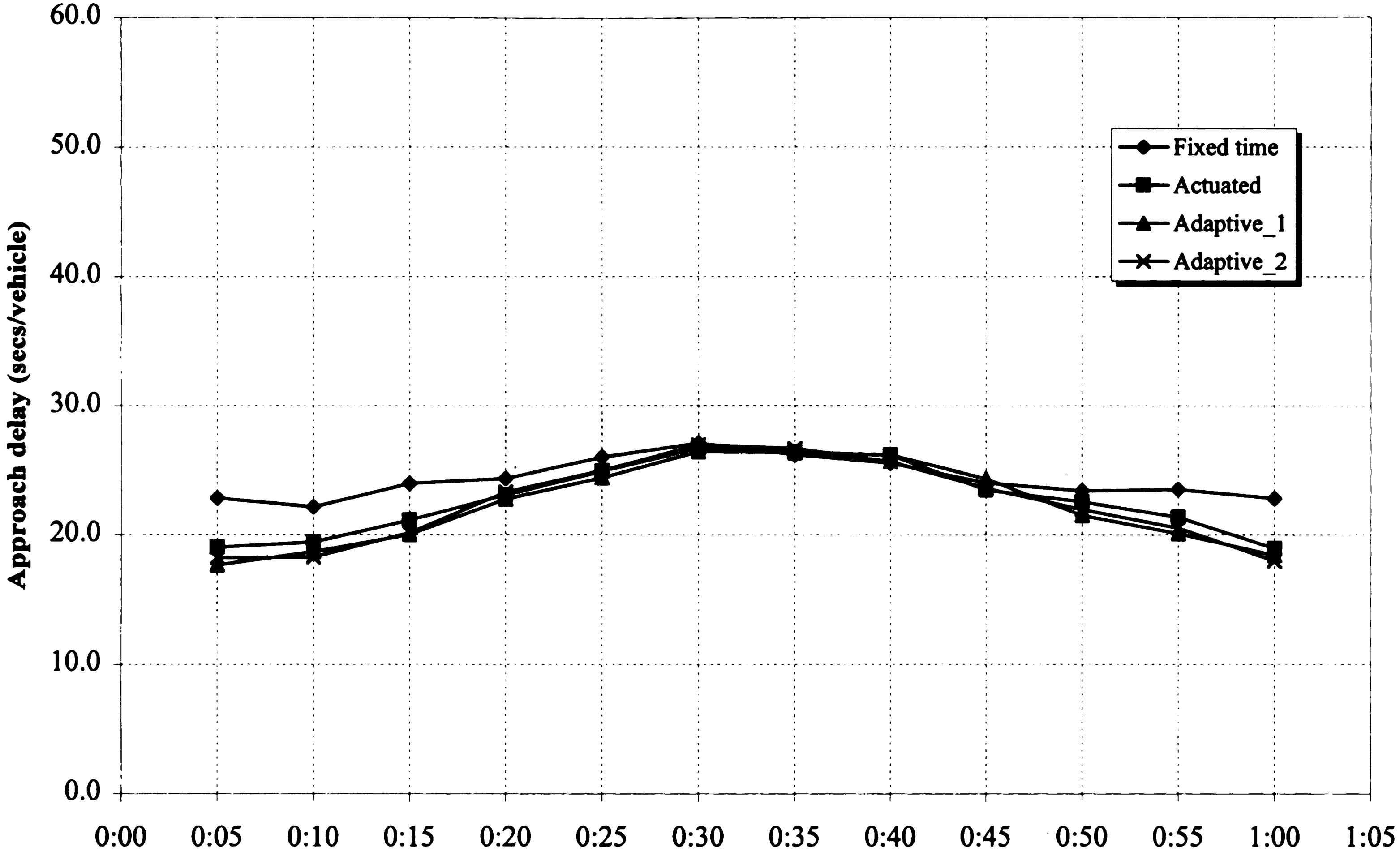

Figure 5.9 Average intersection approach delay under different control strategies(moderate peak-hour demand case)

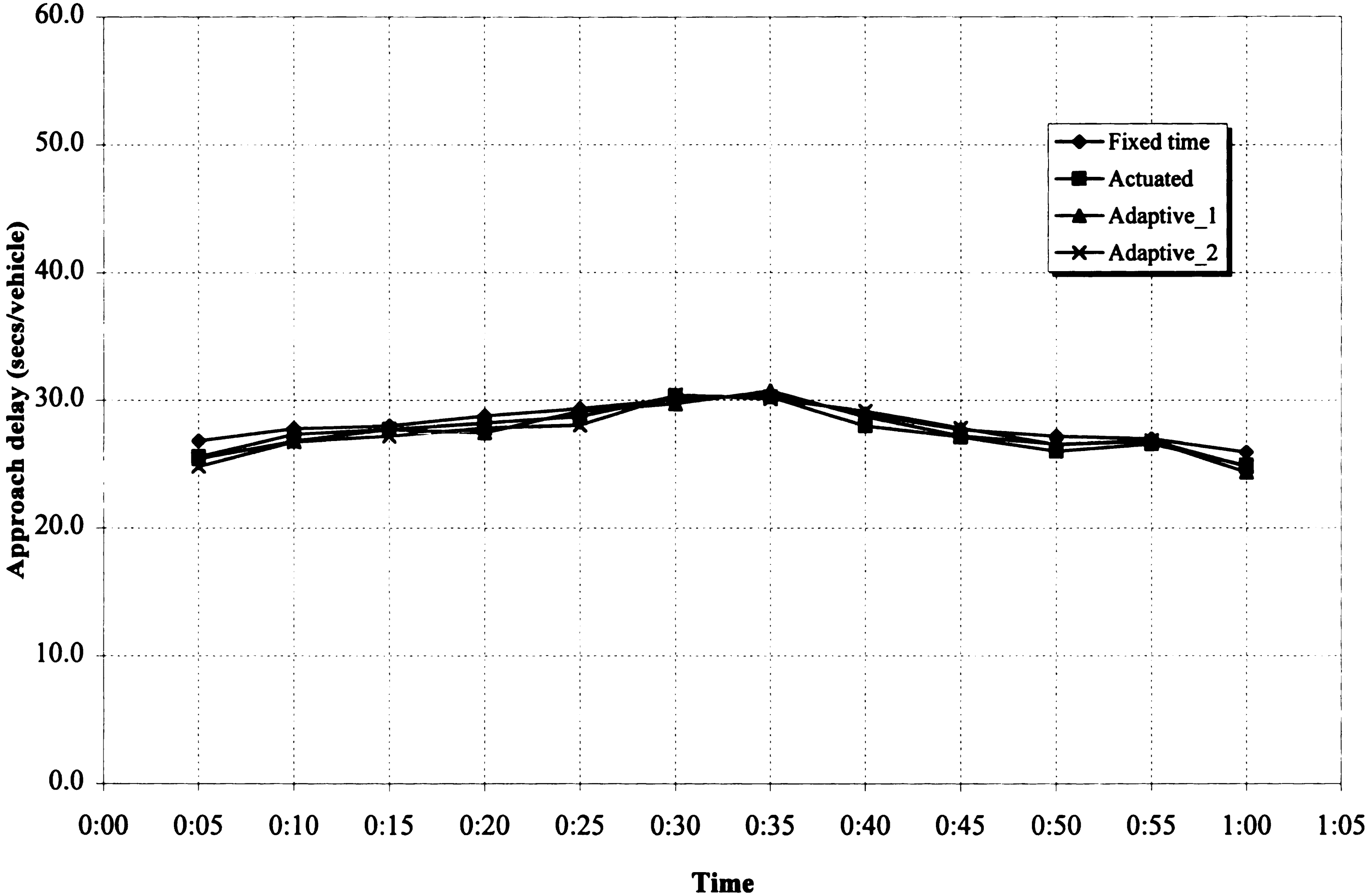

Figure 5.10 Average intersection approach delay under different control strategies (high peak-hour demand case)

Table 5.11 Average intersection approach delay under different control strategies (moderate peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	24.32	22.77	1.56	6.40	22.29	2.03	8.36	22.52	1.80	7.42
15 minutes (peak)	26.27	26.43	-0.16	-0.61	26.32	-0.05	-0.18	26.42	-0.15	-0.58
15 minute (non-peak)**	22.98	19.87	3.12	13.56	18.98	4.00	17.40	19.28	3.70	16.10

*Savings in approach delay over fixed-time signal control (secs / vehicle)
** the 15-minute period with the least amount of traffic during the one-hour period

Table 5.12 Average intersection approach delay under different control strategies (high peak-hour demand)

Average	Fixed Time	Actuated			Adaptive_1			Adaptive_2		
		Average	Savings*	Percent	Average	Savings*	Percent	Average	Savings*	Percent
(one hour)	28.16	27.56	0.59	2.11	27.55	0.60	2.14	27.52	0.63	2.25
15 minutes (peak)	29.79	29.52	0.27	0.91	29.72	0.07	0.24	29.88	-0.08	-0.28
15 minute (non-peak)**	27.53	26.87	0.65	2.37	26.66	0.87	3.16	26.25	1.27	4.62

*Savings in approach delay over fixed-time signal control (secs / vehicle)
** the 15-minute period with the least amount of traffic during the one-hour period

5.2 Signal timing analysis

There are three possible ways an adaptive signal can contribute to a reduction in delay for the major street traffic. The first is to allocate more green time to this approach; the second is to utilize the existing green time more efficiently, (i.e. with less lost time); and the third is to improve the corridor progression, and thus reduce the stopped time component of delay, Figure 5.11. The total effective green time allocated to each approach, aggregated over 5-minute periods, under different control strategies was compared. The effective green time for each phase represents the combined green and amber periods minus the lost time at the beginning and end of that phase. The lost time was assumed to be three and half seconds per cycle.

The results of the green time comparison for both major-street and minor-street traffic are presented in Figures 5.12 and 5. 13 for the moderate demand case, and Figures 5.14 and 5.15 for the high demand case. The results showed that all three signal controls, actuated signals, adaptive_1 control, and adaptive_2 control, allocated additional green time to the major-street traffic while reducing the green time for the minor-street traffic. The green time added to the major-street traffic was higher during the 15-minute non-peak periods and decreased as traffic volumes approached their 15-minute peak. During the 15-minute non-peak period in the moderate demand case, the green time allocated for the major street traffic increased from 10.88 minutes under fixed time control to an average of 11.80 minutes under actuated control, 12.16

minutes under adaptive_1 control, and 11.64 minutes under adaptive_2 control. During the peak 15-minute period, all three signal control strategies assigned approximately the same green time allocated to the approach under fixed time signals. In the high demand case, the green time allocated for major street traffic, except during the first and last 10 where it was slightly higher, was equal to that assigned to the approach under fixed-time signal control. The analysis showed no significant differences in the green time allocated for each approach among the three control strategies, which explains the similarity in travel time and intersection delay reduction achieved by the three control strategies.

To measure how effectively the green time allocation was utilized the degree of saturation, aggregated over 5-minute periods was compared under different control strategies. The ideal saturation flow was assumed to be 1800 vehicles per hour. The results showed that, in general, when the three control strategies extended the green time for the major-street traffic, they reduced the average degree of saturation, reducing approach delay and hence corridor travel time. Consequently, when they reduced the green time for the minor approach, the degree of saturation increased, increasing the delay for this approach. The reduction/increase in intersection delay for each approach was a function of the amount of increase/decrease in the green time allocated for the approach. The results are presented in Figures 5.16, 5.17, 5.18, and 5.19.

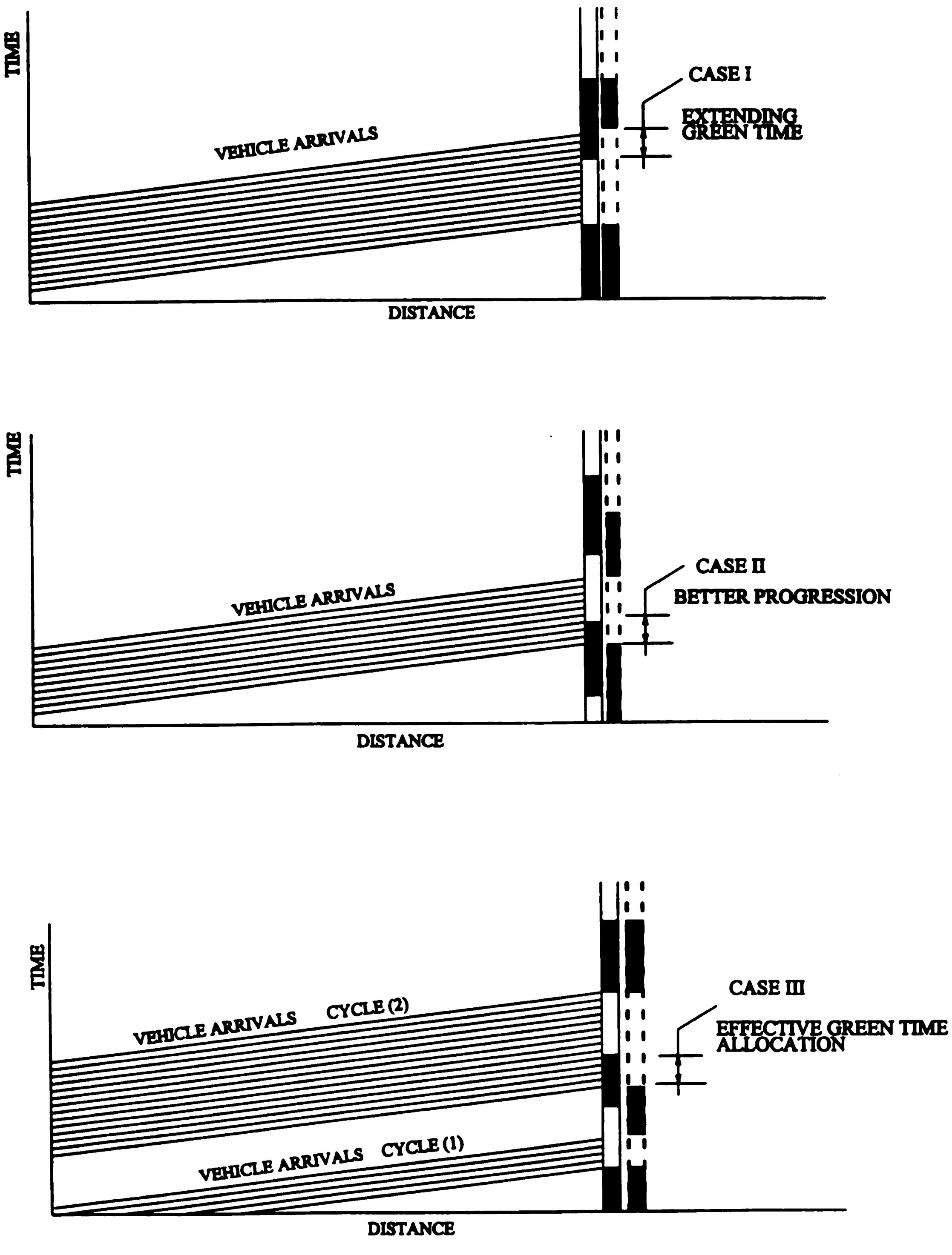

Figure 5.11 Possible delay reduction by an adaptive control system

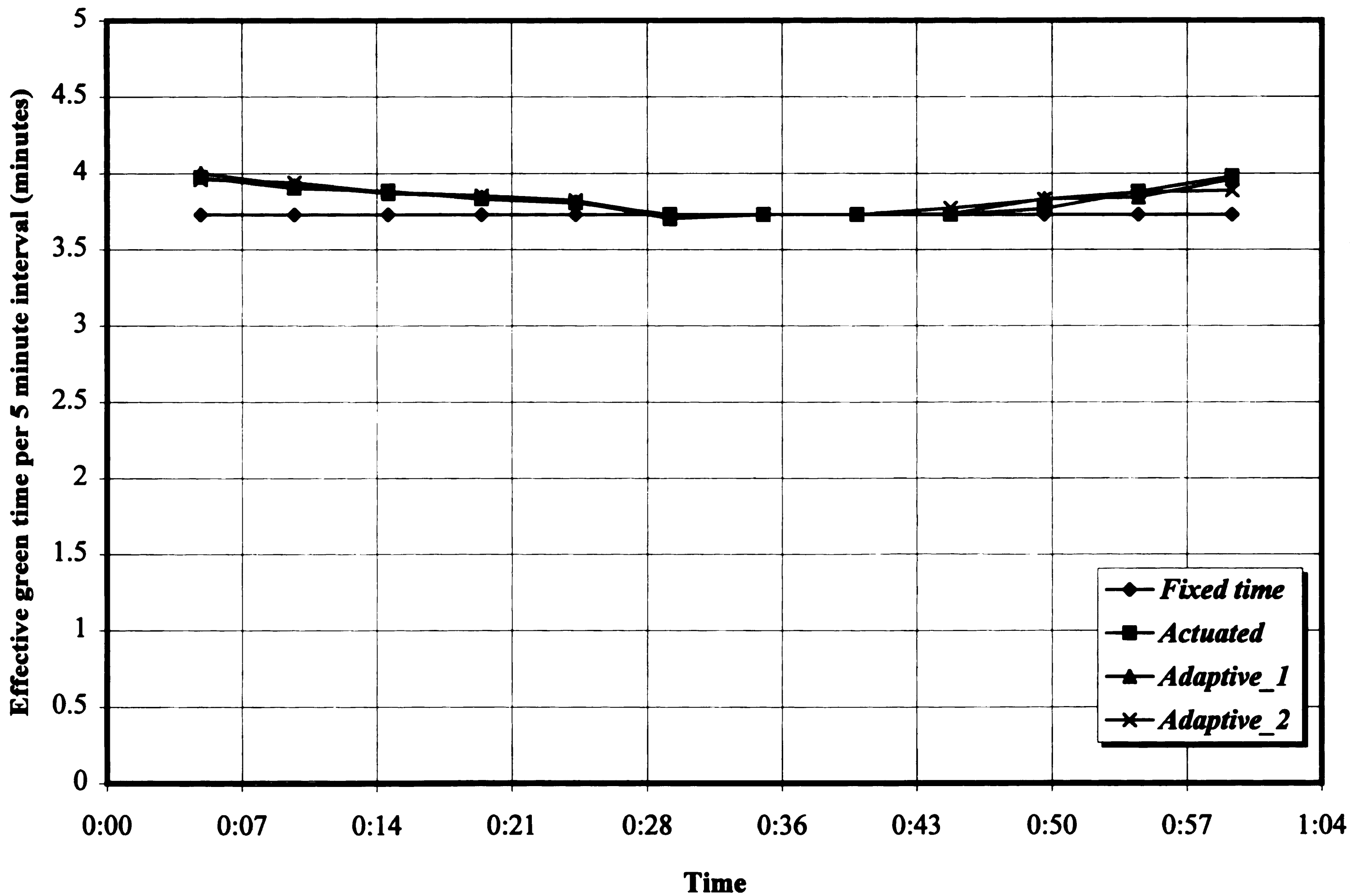

Figure 5.12 Effective green time for major-street traffic
(moderate peak-hour demand case)

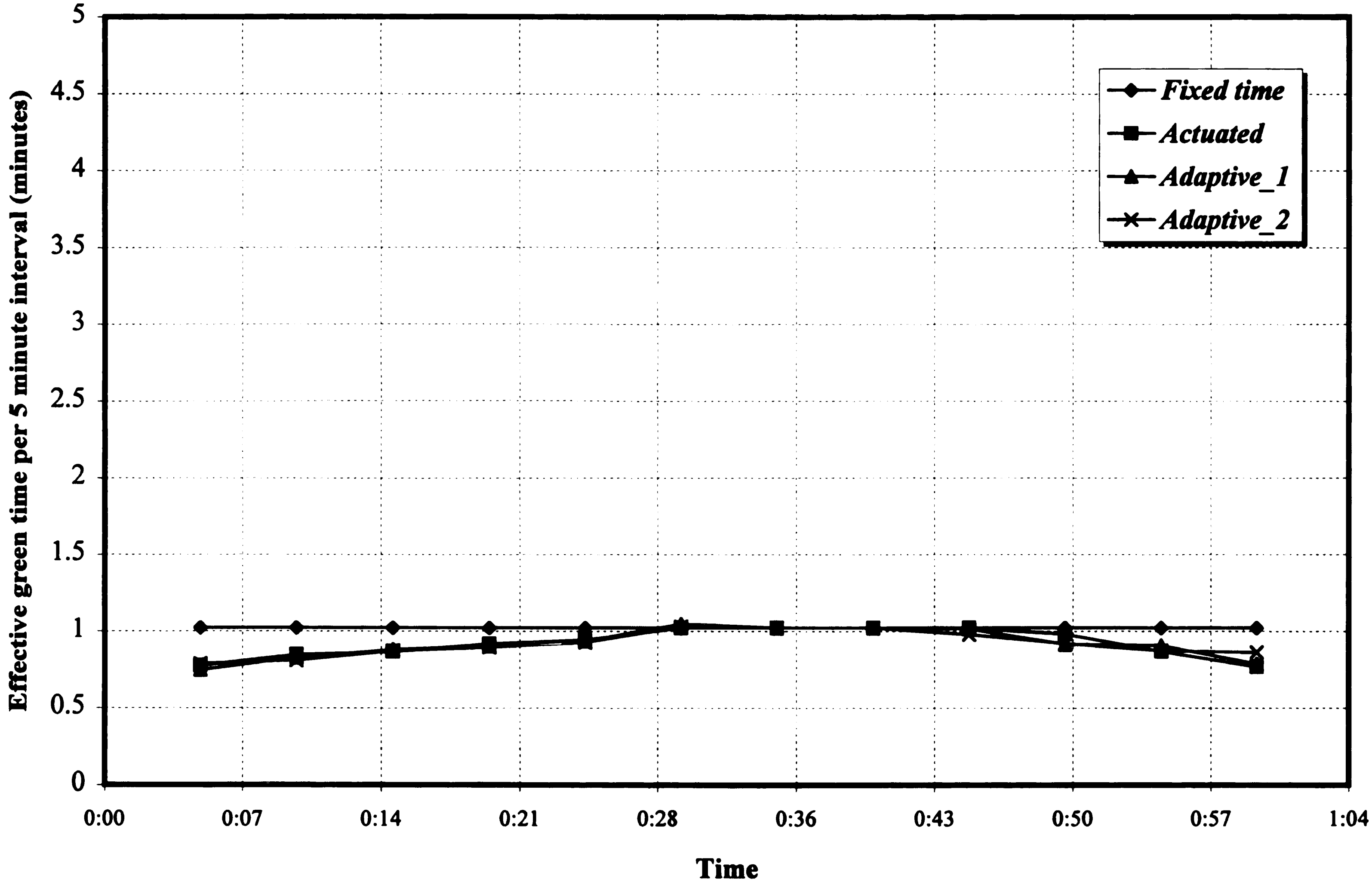

Figure 5.13 Effective green time for minor-street traffic (moderate peak-hour demand case)

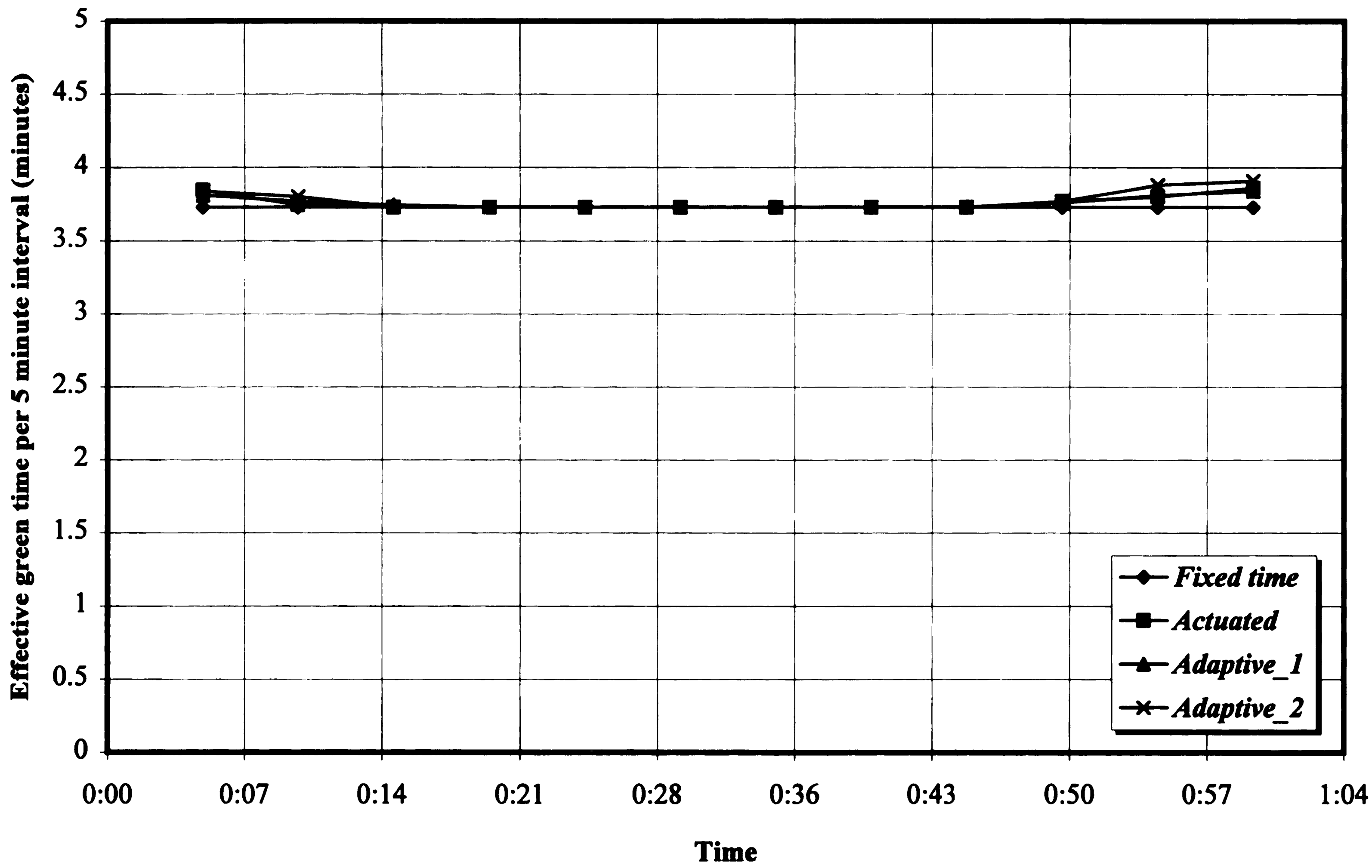

Figure 5.14 Effective green time for major-street traffic (high peak-hour demand case)

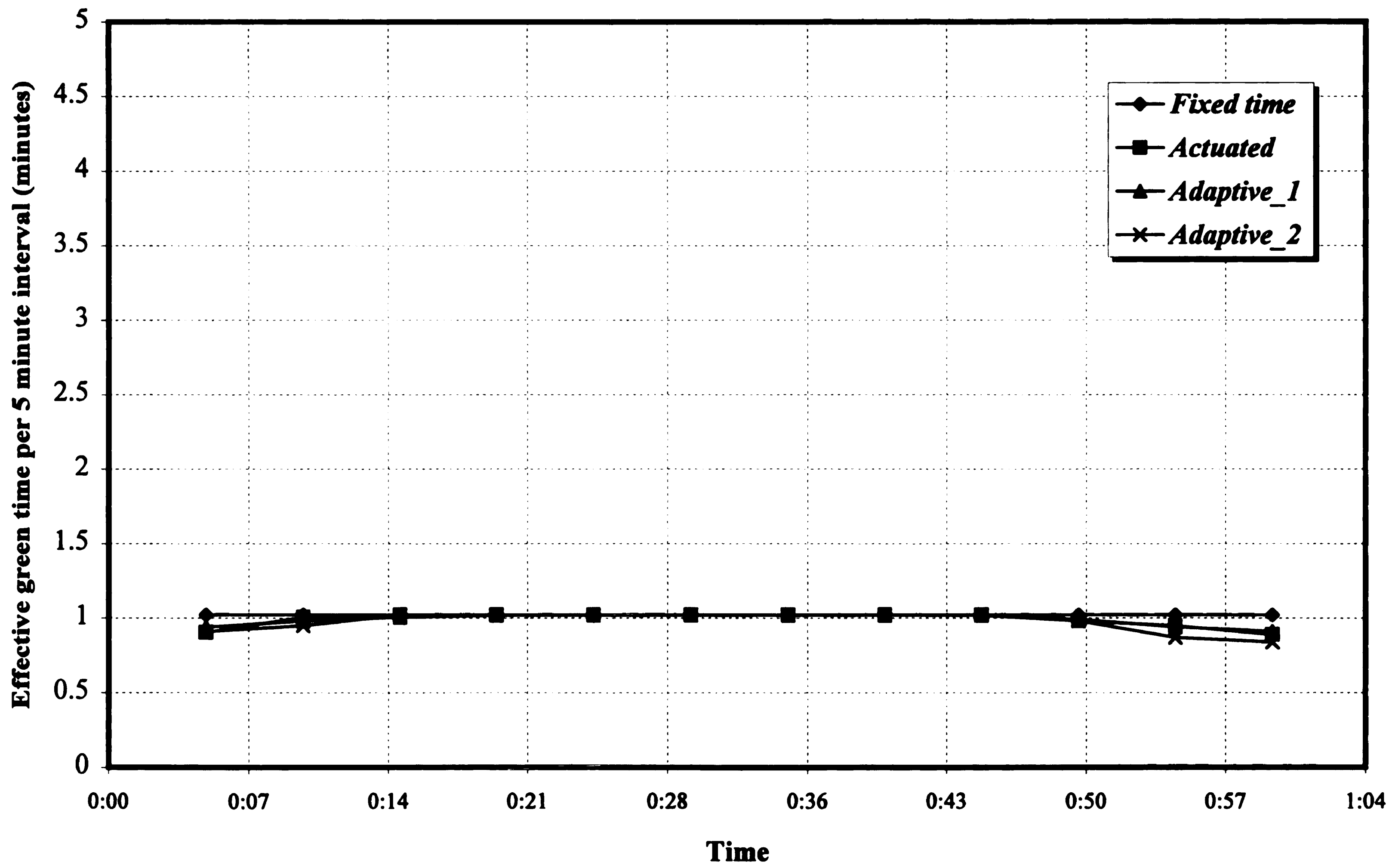

Figure 5.15 Effective green time for minor-street traffic (high peak-hour demand case)

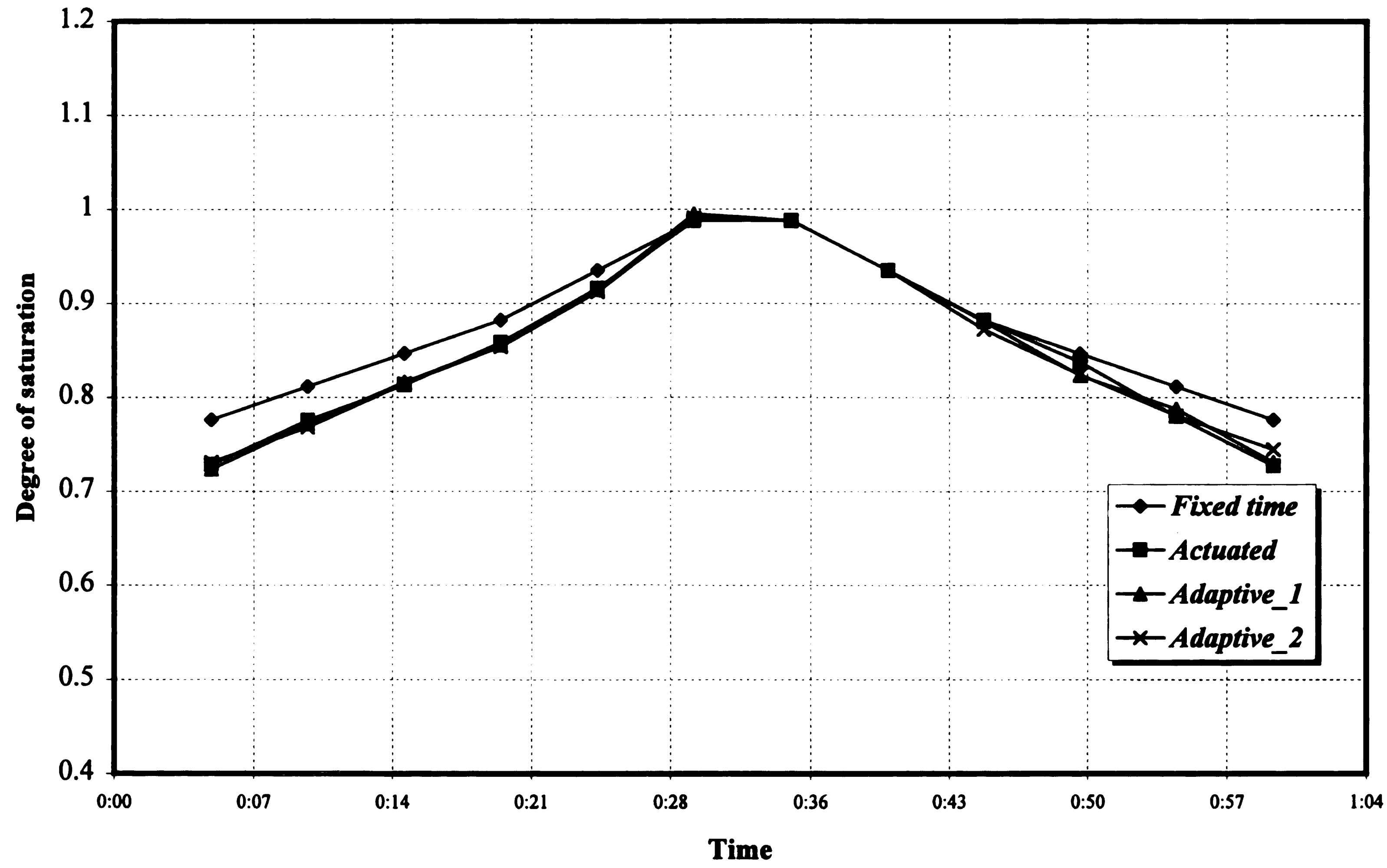

Figure 5.16 Degree of saturation for major-street traffic (moderate peak-hour demand case)

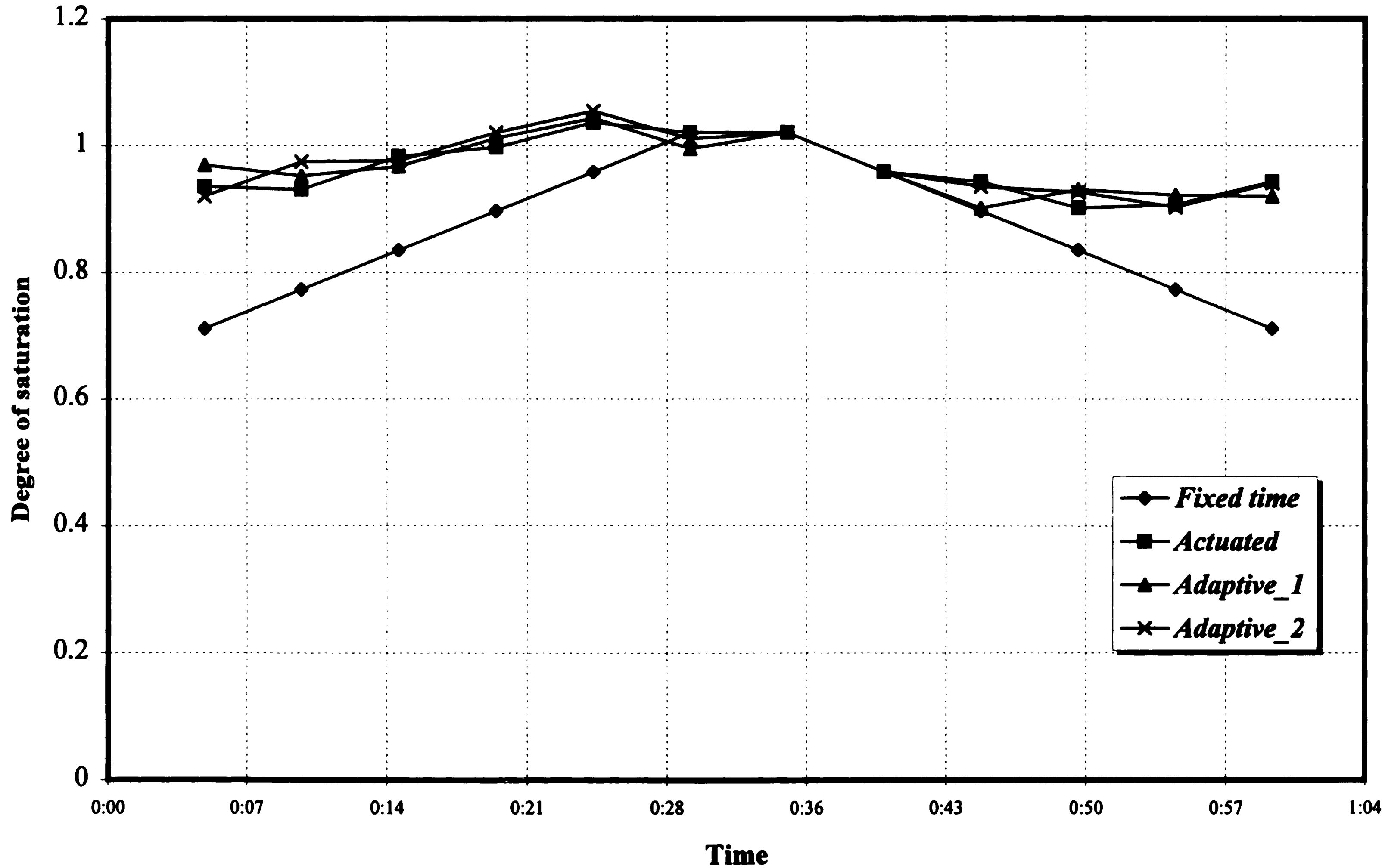

Figure 5.17 Degree of saturation for minor-street traffic (moderate peak-hour demand case)

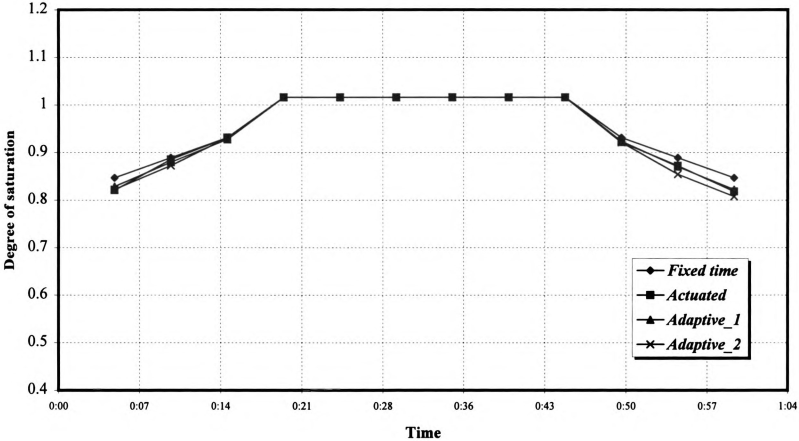

Figure 5.18 Degree of saturation for major-street traffic (high peak-hour demand case)

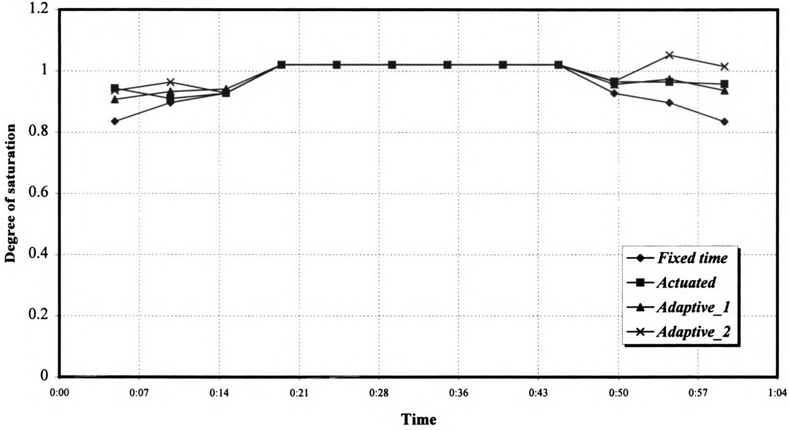

Figure 5.19 Degree of saturation for minor-street traffic (high peak-hour demand case)

During the 15-minute non-peak period in the moderate demand case, the degree of saturation for the major-street traffic decreased from an average of 0.81 under fixed time control to an average of 0.77 under the three other control strategies. This reduction was a result of increasing the green time for the major-street traffic. Simultaneously, the degree of saturation for the minor-street traffic increased from an average of 0.77 to an average 0.95 as a result of reducing the green time for this approach. During the peak 15-minute period the three control strategies as well as the fixed time control maintained the degree of saturation for the two approaches at 0.99%.

5.3 Sensitivity analysis

This part of the analysis explores the sensitivity of the benefits of the adaptive control strategies to changes in demand levels and peak hour factors for minor-street and major-street traffic. The benefit, in this part of the study, is defined as the percent reduction in average corridor travel time achieved by the adaptive control strategy. Due to the similarity in the performance of both adaptive control strategies examined in this study, the sensitivity analysis was carried out only for the adaptive_1 control strategy.

5.3.1 Sensitivity of adaptive control benefits to minor-street demand level

The sensitivity of adaptive control benefits was first examined under different peak-hour factors (PHF) for the minor-street traffic. The traffic volume and the PHF for the major-street traffic were kept constant. The traffic volume for the minor-street traffic was also kept constant with four different values of PHF: 0.85, 0.90, 95, and 0.99. Five simulation iterations were performed for each PHF. The results of the analysis are presented in Figure 5.20. As can be seen, the percent reduction in average corridor travel time is sensitive to the PHF for the minor street traffic. That is, as the PHF decreases the benefit increases. The percent reduction in travel time ranged from 13% to 7.8% when the PHF was 0.85, and from -1.2% to 1.44% when the PHF was 0.99.

Lower values of PHF mean that the ratio of the volume during the non-peak periods to the volume during the 15-minute peak period is lower. Because the green time allocated to this approach, under fixed-time control is calculated based on the peak 15-minute traffic, the approach is served with an excess amount of green time during periods other than the peak 15-minute. The adaptive control system utilized this extra-green time by reallocating it to the major-street traffic. As the PHF decreases, the excess green time allocated for the minor-street traffic increases, providing the adaptive control logic with more potential benefits. Similarly, during the peak 15-minute period, there is little or no excess green time to be reallocated, eliminating the potential benefits of the adaptive control system.

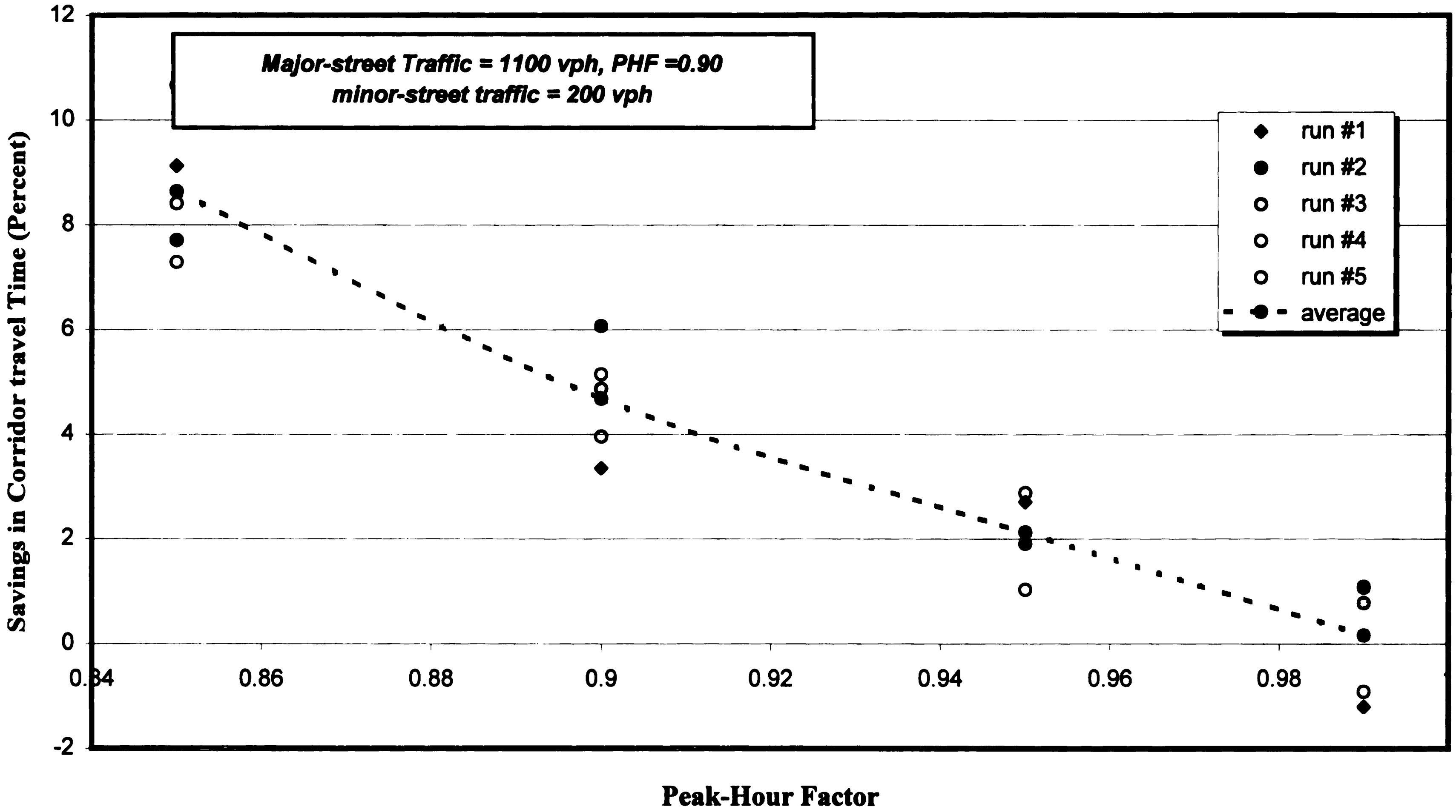

Figure 5.20 Benefit Sensitivity to minor-street Peak Hour Factor (PHF)

To examine the sensitivity of the benefits to minor-street traffic demand, the PHF was kept constant for four different demand levels: 150, 200, 250, and 300 vehicles per hour. Figure 5.21 shows the savings in corridor travel time under these four demand levels using four different PHFs. For the same PHF, the benefit slightly decreases as the minor-street traffic demand increases. This indicates that the percent reduction in travel time is more sensitive to changes in PHF than to changes in traffic demand, at least at these demand levels. Figure 5.22 shows the potential reduction in corridor travel time under different demand levels and peak hour factors for minor street traffic.

5.3.2 Sensitivity of adaptive control benefits to major-street demand level

To examine the sensitivity of the benefit of adaptive control systems to changes in major-street traffic volumes, four different demand levels were examined. The minor-street demand level and PHF were kept constant. The PHF for the major-street traffic was also kept constant. Five simulation iterations were performed for each demand level, the results are presented in figure 5.23. No significant changes were found among the benefits obtained under different demand levels. This indicates that changes in major-street traffic demand, within the values examined in this study, had no significant effect on the percent reduction in corridor travel time.

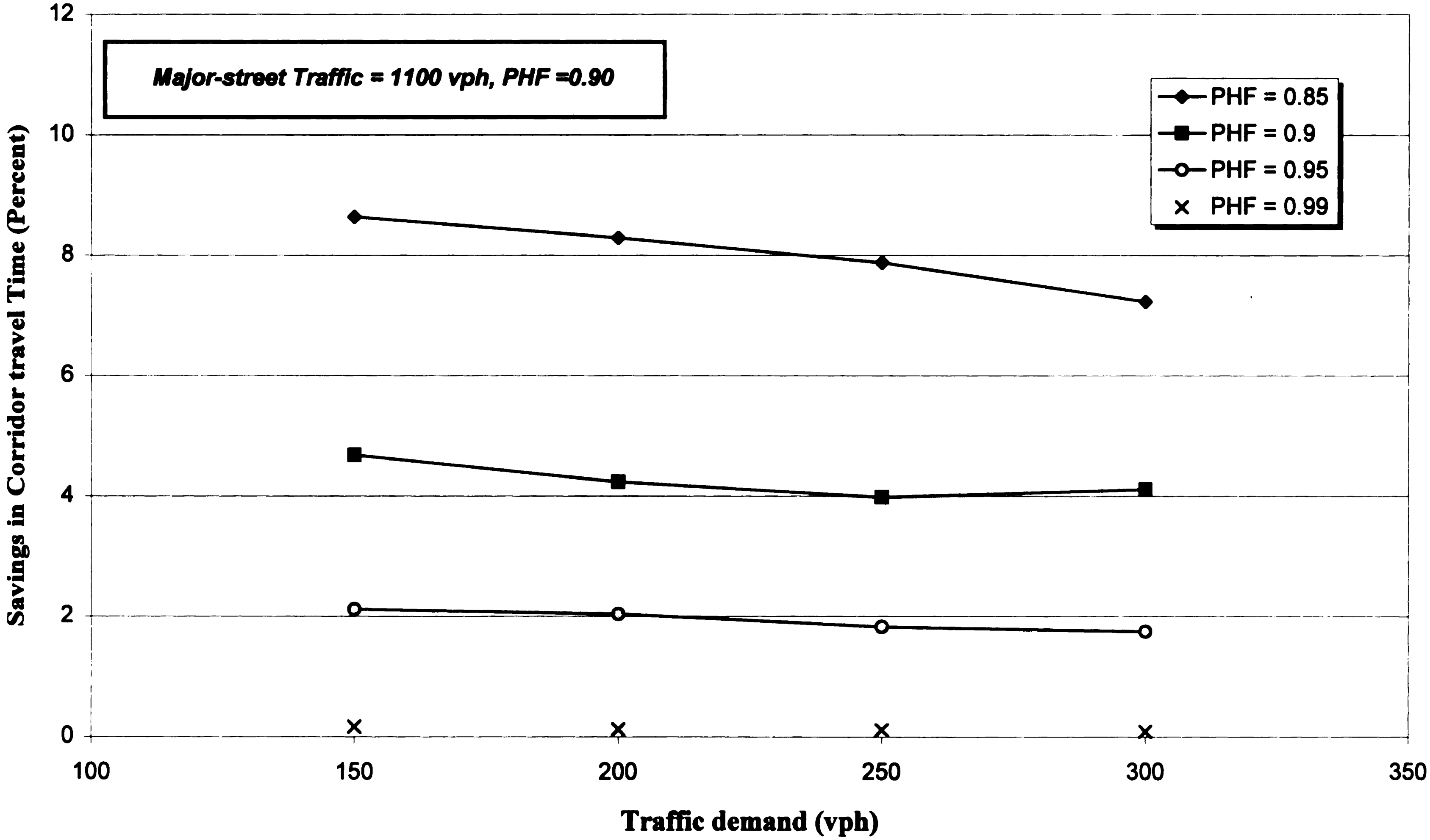

Figure 5.21 Benefit Sensitivity to minor-street traffic demand for different PHFs

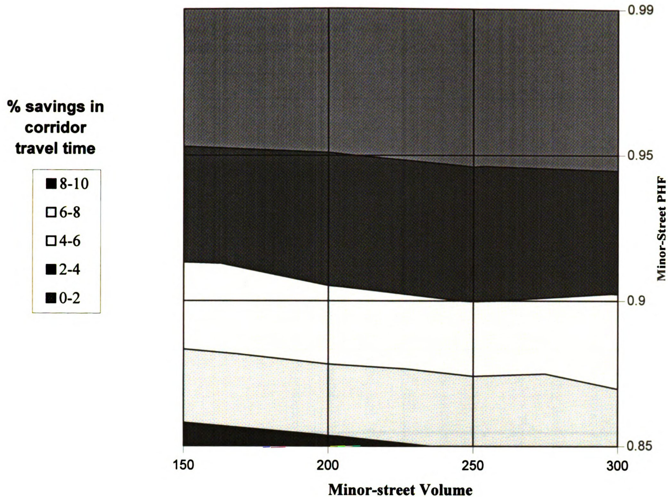

Figure 5.22 Potential Corridor travel time savings of adaptive control systems under different demand level and PHF for minor-street traffic

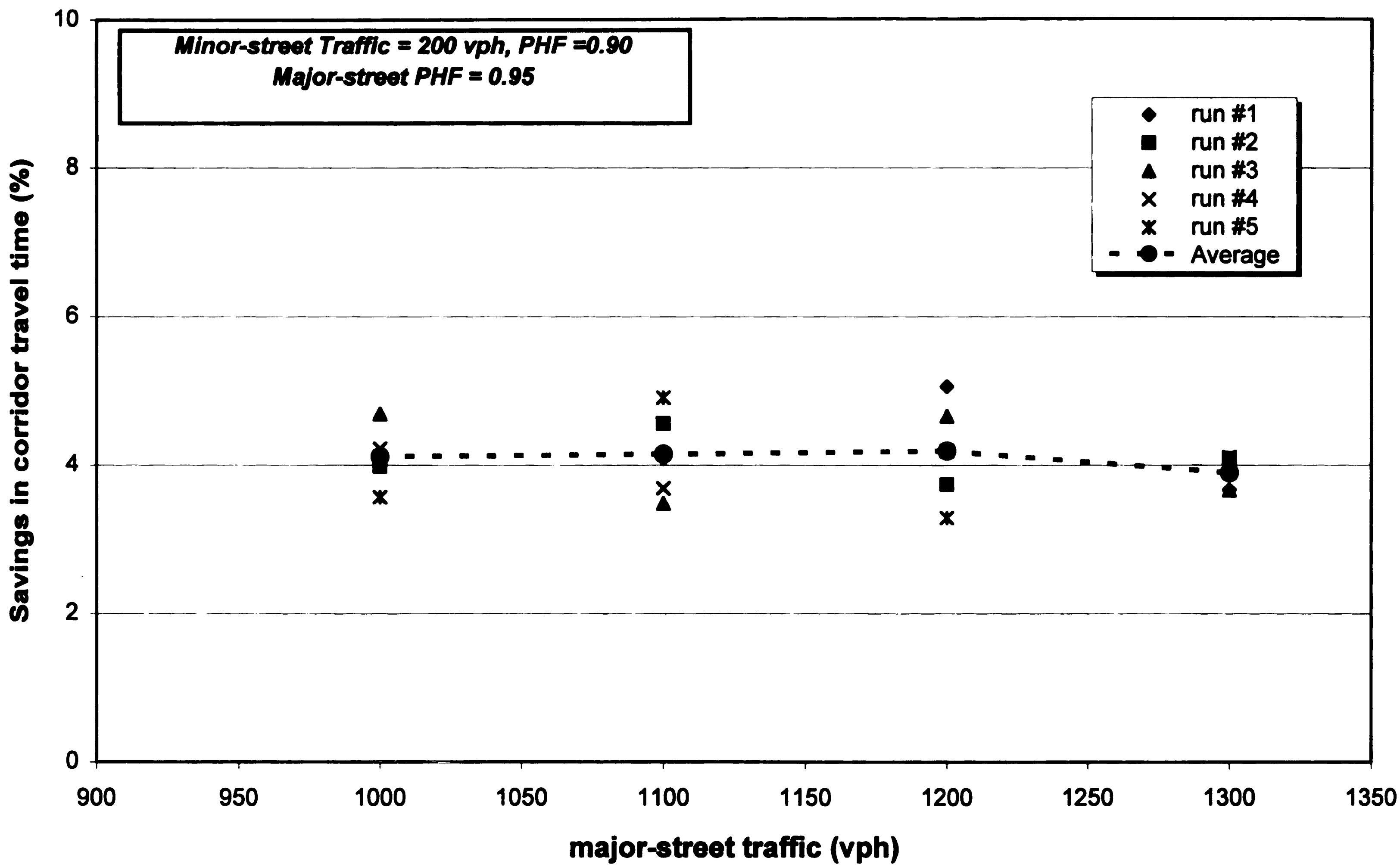

Figure 5.23 Sensitivity of savings in corridor travel time to major-street traffic demand

5.3.3 Sensitivity of adaptive control benefits to signal phase plan

The objective of this part of the study is to examine whether the reduction in corridor travel time reported in the earlier parts of the study is limited to the two-phase signal operation examined in the study or can be generalized for other signal plans. A four-phase signal plan was considered in this part of the study by adding a protected left-turn phase for major-street and minor-street traffic. Corridor travel time was examined under two different signal control strategies: fixed-time and adaptive_1 control. Five simulation iterations were performed for each case, the average corridor travel time under the two control strategies are shown in Figure 5.24.

The results showed a similar pattern to that obtained under two-phase signal operation. Similar to the results obtained earlier, the reduction in travel time was higher during the low demand period in the first and last 15-minute periods, and decreased as traffic reached its 15-minute peak period. The average reduction in travel time during the non-peak periods was 4.14%. The reduction was only 0.36% during the peak-15 minute period.

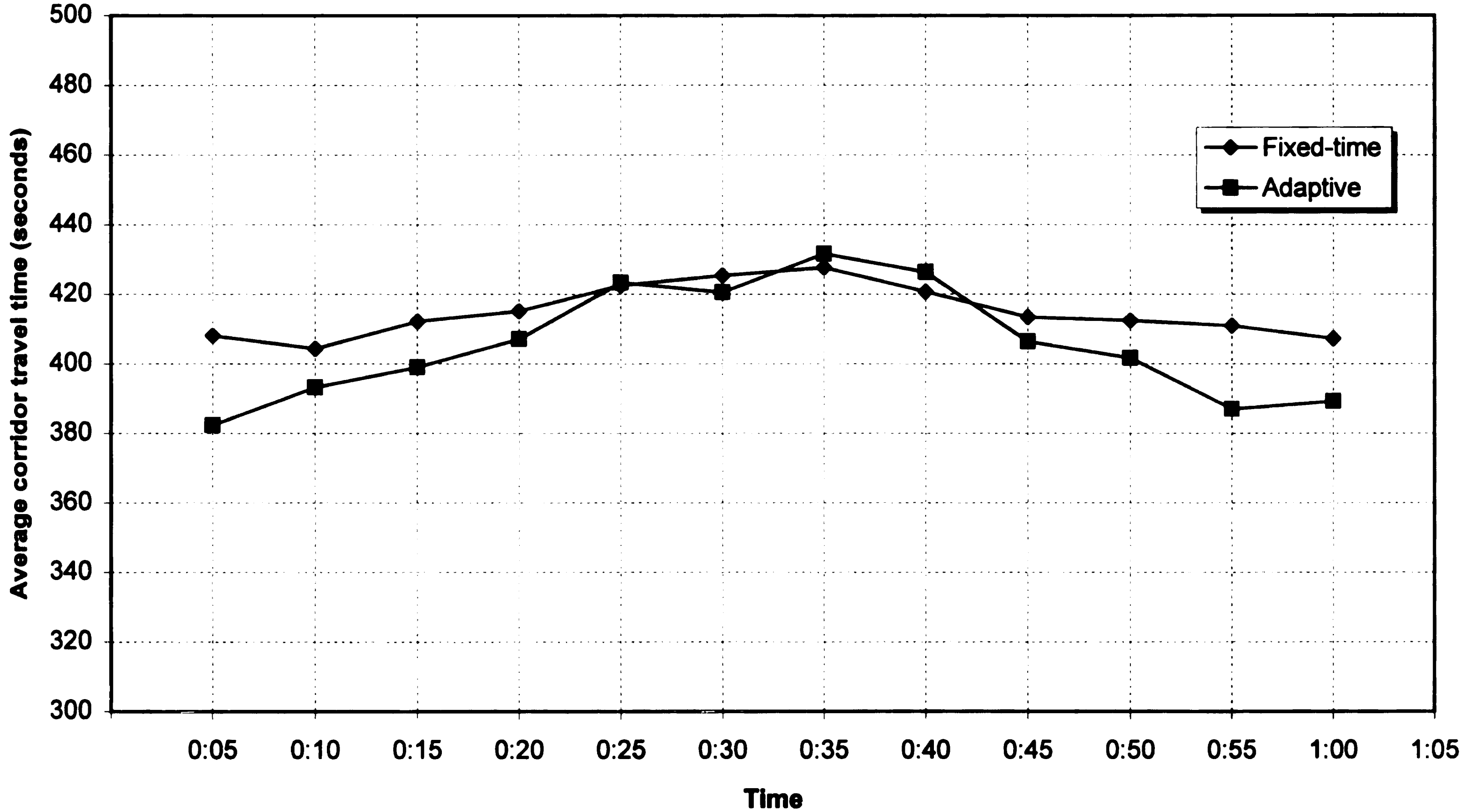

Figure 5.24 Average corridor travel time under fixed time and adaptive signal (4- pahse signal operation)

5.3.4 Sensitivity of adaptive control benefits to changes in traffic demand over time

The objective of this part is to examine the long-term benefits of adaptive control strategies. For this purpose, it was assumed that the major-street traffic would grow at an average of 6% annually. The growth rate for the minor-street traffic was set at 2% annually. The benefits of adaptive signal control were examined after one and two years under two assumptions. The first one assumed that the signal plan will remain with the same setting optimized using the base year traffic demands. The second case assumed that the signal settings would re-optimized using the new traffic demands. The results are shown in Figures 5.25 and 5.26 and summarized in Table 5.12.

Table 5.13 Percent reduction of corridor travel time under adaptive control system (two-year operation)

	Base-year	First-year	Second year
Adaptive vs. non-optimized fixed-time	4.16	9.06	26.33
Adaptive vs. optimized fixed time	4.16	3.04	1.57

When compared with the fixed-time settings, an optimized setting based on the base year traffic, the benefit of the adaptive control system increased significantly with time. The savings in travel time increased from an average of 4.16 % in the base year to an average of 26.33% after two years of operations. This represents a significant

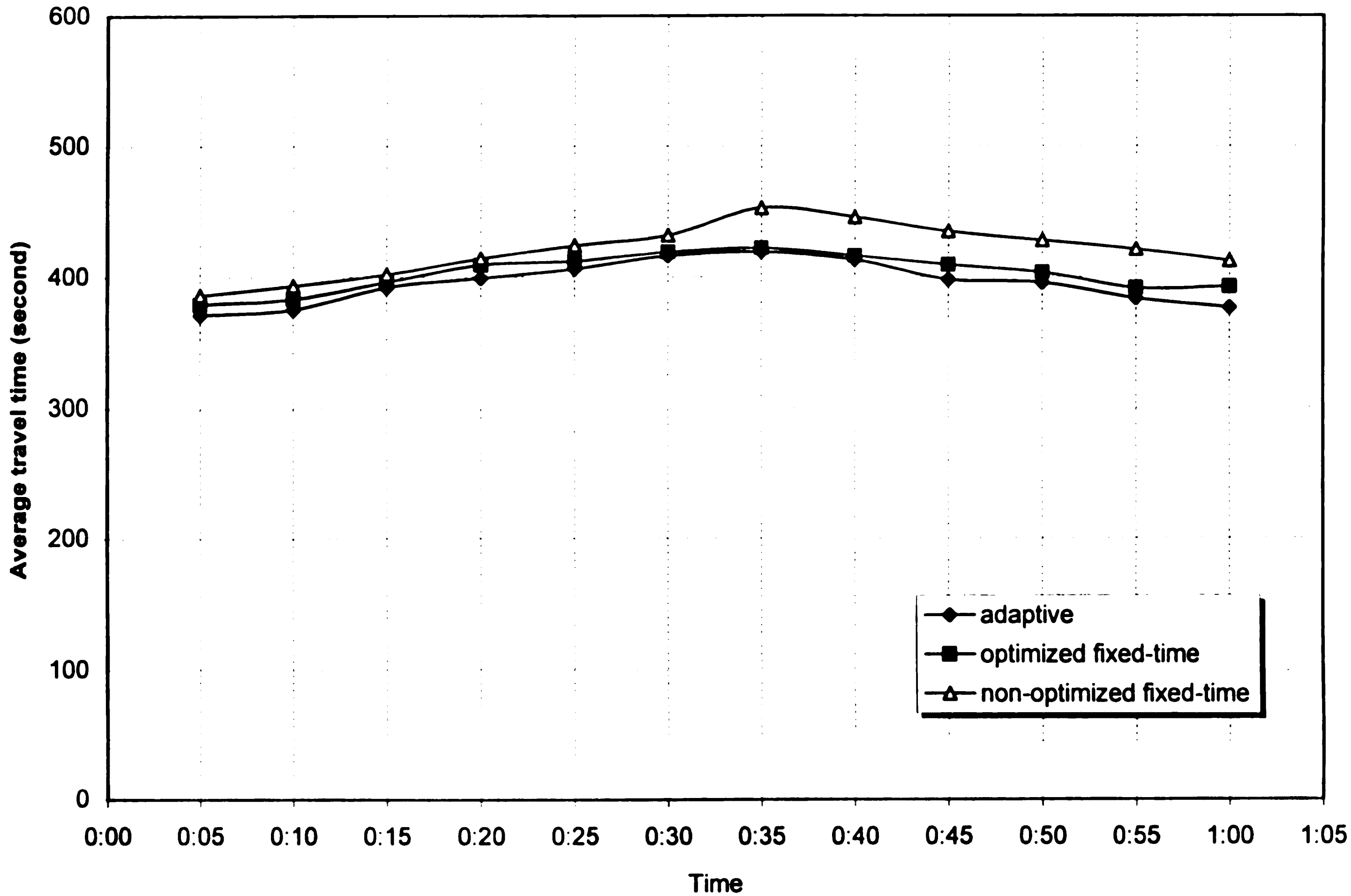

Figure 5.25 Benefit of adaptive signal control after one-year

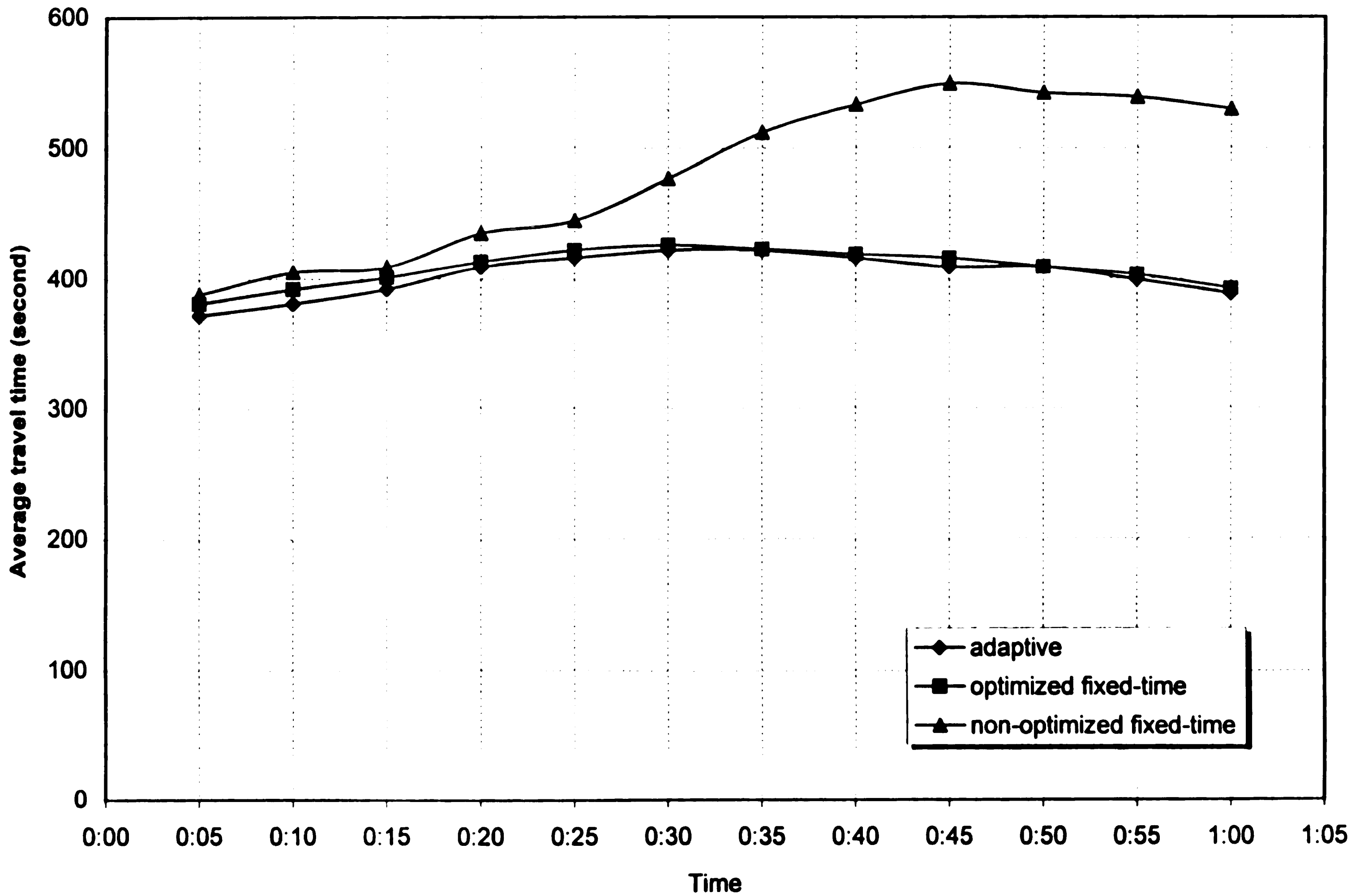

Figure 5.26 Benefit of adaptive signal control after two year

benefit of the adaptive control system as it is optimized on a day-to-day basis. When compared with a fixed-time signal that is optimized based on the new traffic demands, the benefits of the adaptive control decreased from an average of 4.16% in the base year to an average of 1.57 % after two years of operation. This is a result of the increase in the degree of saturation resulting from the growth in traffic.

5.4 Comparative analysis of adaptive control prediction logic

In the first adaptive control strategies examined in this study (adaptive_1), traffic predictions were made based on cycle-by-cycle traffic data collected downstream from the intersection. That is, the prediction of the number of vehicles expected for any specific cycle was calculated based on traffic volumes in the previous three cycles as explained in section 3.2.3.2 of this dissertation. Figure 5.27 shows the predicted and actual number of vehicles for both major-street and minor-street traffic for one-hour period. The comparison showed that the error in the predicted number of vehicles per cycle ranged from 0 to 3 vehicles for minor street traffic (average number of arrivals per cycle = 10 vehicles) and from 0 to 5 vehicles for major-street traffic (average number of arrivals =40 vehicles). When the prediction horizon was increased to 5-minute intervals, the accuracy of the predictions increased (figure 5.28).

In the second prediction technique, upstream detectors were used to predict the number of vehicles expected based on the estimated travel time between the detector location and the intersection. A comparison of the predicted and actual values is shown in Figure 5.29. The error in the predicted values ranged from 0 to 2 vehicles for both major-street and minor-street traffic. The error is basically due to the difference between the stochastic value of the vehicle speeds and the average value used in the prediction model. It should be noted that the scan-ahead prediction model introduced in this study represents an ideal case. No mid-block traffic entered or exited in the approach between the detector location and the intersection. It should be noted that while the first prediction technique was not as effective in predicting the cycle-by cycle variation in demand as the second technique, there was no significant difference in the performance of the two adaptive control systems that used these two prediction techniques.

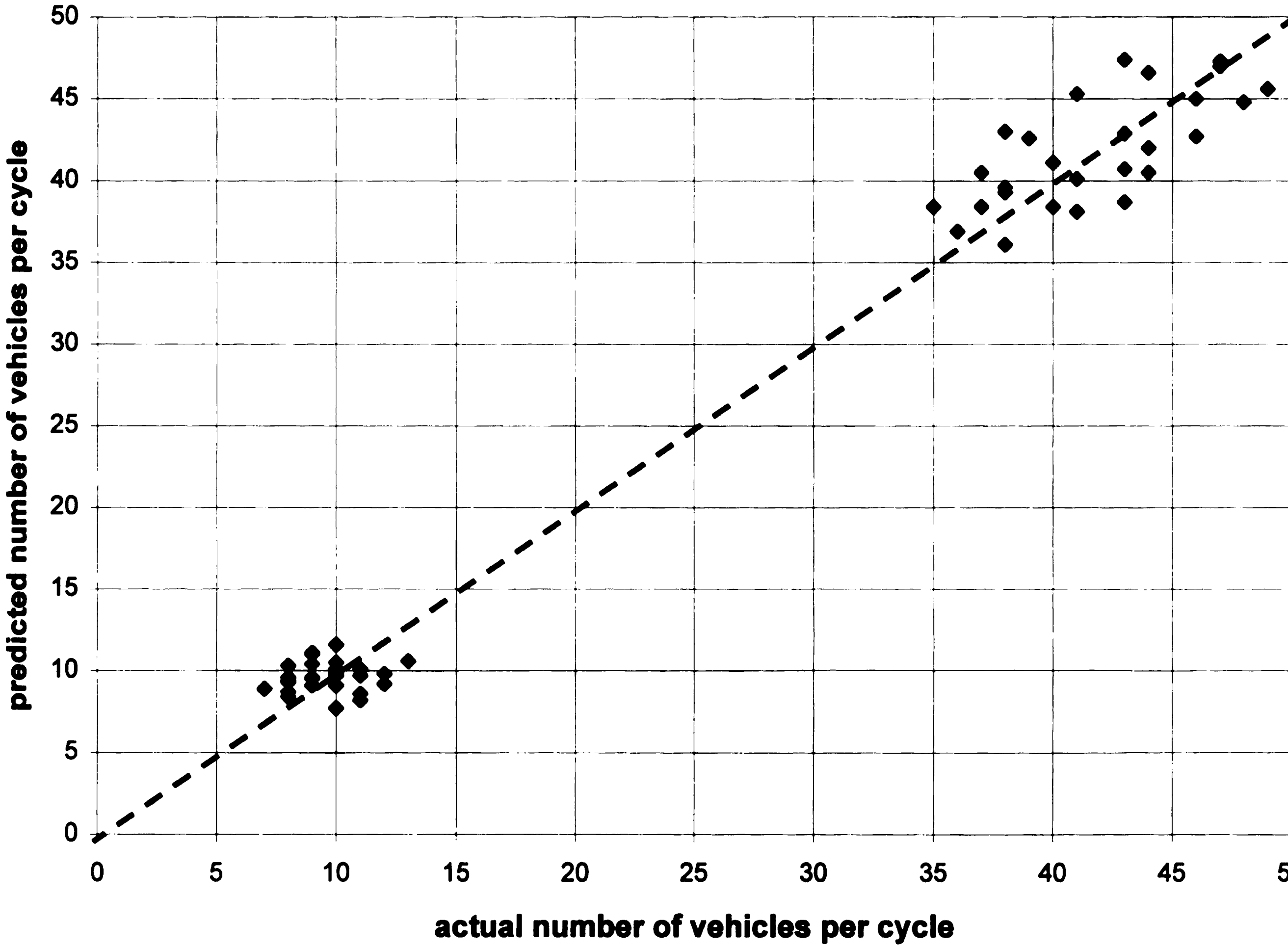

Figure 5.27 Actual vs. predicted number of vehicles for each cycle
(downstream detector method)

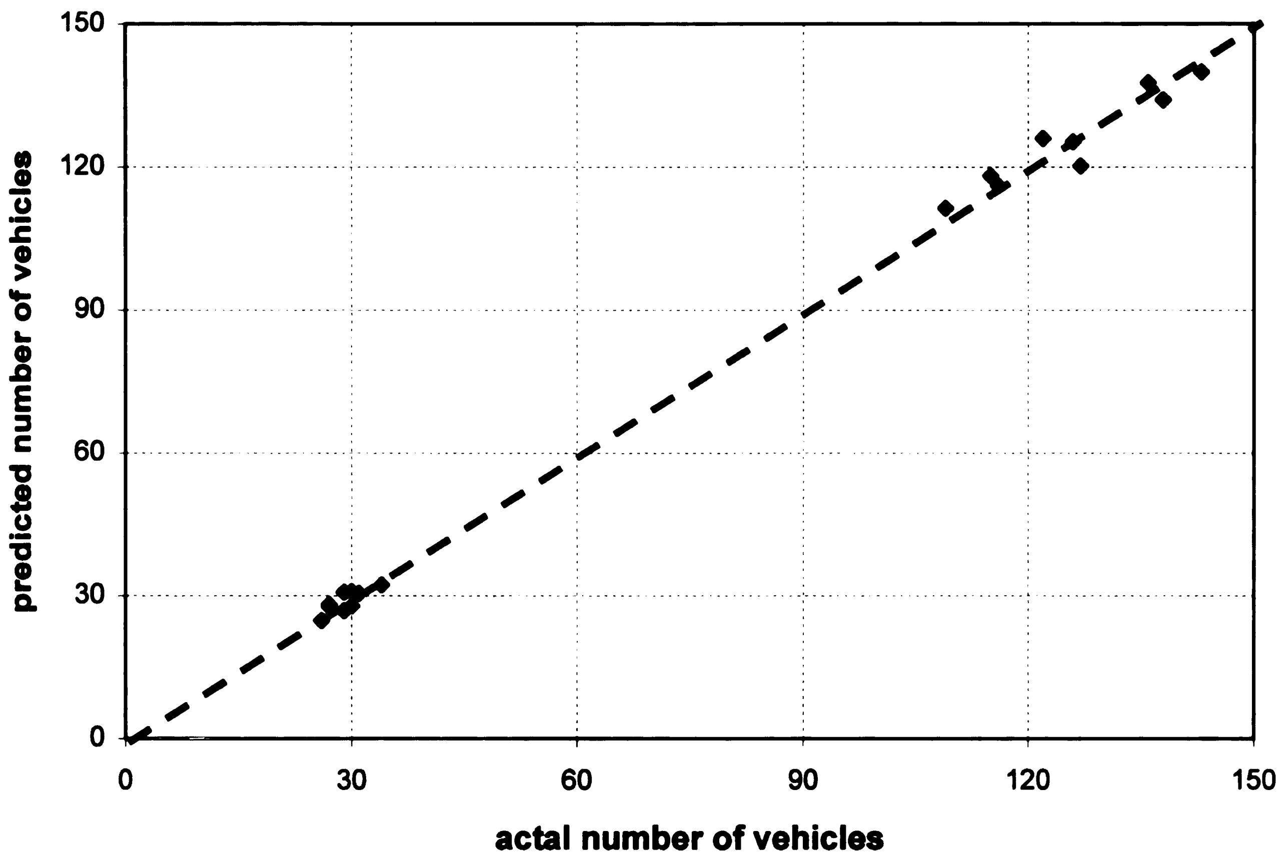

Figure 5.28 Actual vs. predicted number of vehicles for 5-minute time interval (downstream detector method)

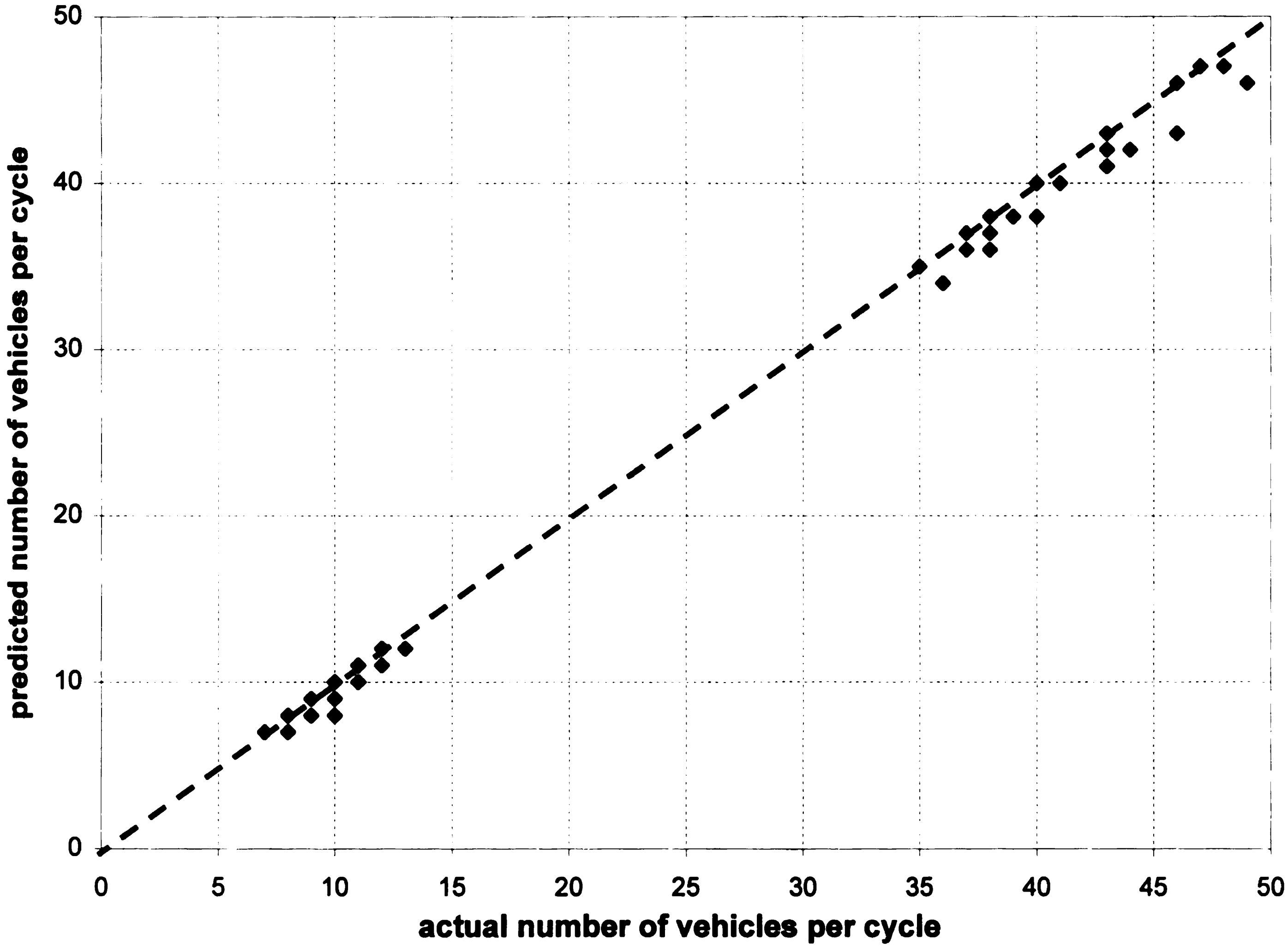

Figure 5.29 Actual vs. predicted number of vehicles for each cycle upstream detector (scan-ahead) method

Chapter 6

FIELD STUDY

In 1991, the Road Commission for Oakland County (RCOC), Michigan, embarked on a major demonstration project involving the implementation of Intelligent Transportation Systems (ITS) technology. This technology included an adaptive traffic control system, SCATS, implemented on the road network within Oakland County, Michigan. The signal improvements are part of the FAST-TRAC (Faster and Safer Travel-Traffic Routing and Advanced Controls) system, a national demonstration project for Advanced Traffic Management System (ATMS) and Advanced Traveler Information System (ATIS), (RCOC, 1996).

A before/after study, (Taylor et. al, 1997 and 1998), to examine the impacts of SCATS implementation on Orchard Lake Road traffic revealed that the corridor travel time improved for both directions of travel for both the peak and the non-peak periods. The reduction in corridor travel time ranged from 6.56% to 31.80% with savings in travel time being higher during the non-peak periods. Similar results were obtained from studies carried out by the Australian Road Research Board (ARRP) and the Department of Main Roads in New South Wales to evaluate the performance of SCATS. Their comparison of SCATS and TRANSYT optimized fixed time signals showed that SCATS can improve the travel time and number of stops from 3% to 18%, depending on the network and traffic characteristics, (ARRP 1982, and 1988).

6.1 Field study Objectives and Approach

The first part of this study was designed to determine which characteristics of SCATS adaptive control caused the savings in travel time on Orchard Lake Road. Before/after green time and offset analyses were carried out to examine how the dynamic green time allocation (cycle length and phase plans) and the dynamic progression (offset plans) contributed to the changes in intersection delay and corridor travel time.

The second part of the study was a simulation analysis to examine whether similar savings in Orchard Lake Road corridor travel time could have been achieved through conventional control strategies namely; coordinated actuated and optimized fixed-time signals. A comparison of the cost and benefits of different control systems are presented at the end of this field study.

6.2 Study Location

The selected study location is a corridor (Orchard Lake Rd) in an urban district in Oakland County, Michigan. The geometric configuration of the corridor is presented in Figure 6.1.

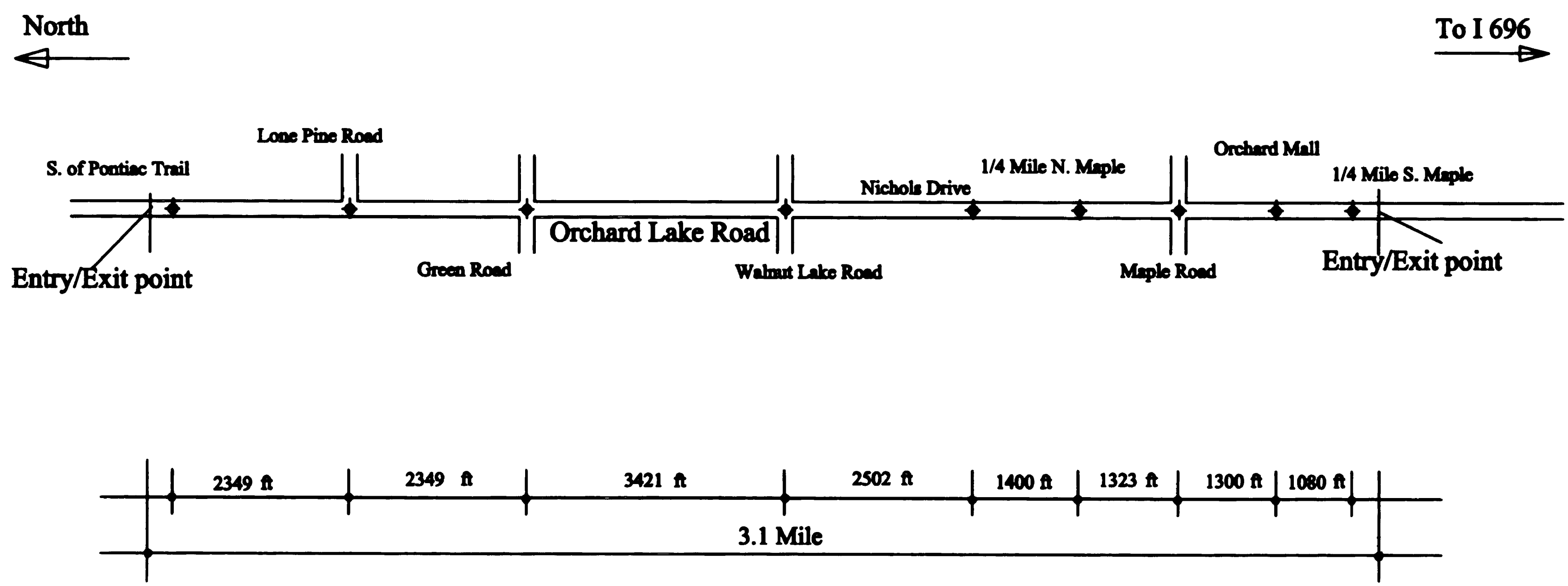

Figure 6.1 Orchard Lake Road corridor (corridor travel time study section)

6.3 Characteristics of SCATS adaptive control

Input data for the SCATS system installed in Oakland County are collected via a system of video image devices mounted overhead on the signal strain poles or attached to mast arms. The traffic information collected in the field includes the discharge characteristics (i.e. flow and occupancy during the green phase) on each intersection approach. This data is transmitted to a regional control center where the SCATS control program attempts to maintain the highest degree of saturation at the intersection. This is done by choosing optimal signal timing parameters from a library of pre-defined signal timing and offset plans that were optimized off-line under different traffic conditions (Lowrie, 1982 and 1990).

For dynamic signal coordination, SCATS chooses from four different pre-defined offset plans. Two different offset values "PP1" and "PP2" are defined for each plan. PP1 is to be used if the cycle length is equal to or less than a pre-defined value "C1" and PP2 is to be used if the cycle length is higher than or equal to a pre-defined value "C2". If the cycle length is between "C1" and "C2", the system chooses the offset linearly between PP1 and PP2. Unlike the fixed time mode where all the signals are coordinated to one intersection, SCATS coordinates the leading phase in one intersection with a designated phase in another intersection, typically the leading phase of the preceding intersection.

SCATS uses detectors placed downstream from the intersection to collect traffic data of vehicles departing from the intersection during the green interval for each approach. The system predicts the traffic profile for the next cycle based on the volumes of previous cycles. This method allows SCATS to predict changes in traffic demand over time rather than predict cycle-by-cycle variation in demand. This method is similar to the prediction technique used in the second adaptive control strategy examined in the first part of this research.

6.4 Before/after green time and offset analysis

The green time allocation and offset data for the after period were obtained from SCATS system monitoring files. The file reports the cycle-by-cycle green time allocated for each approach, the actual number of vehicles that departed during the green phase per lane per approach, and the link and offset plans employed during this cycle.

Before SCATS was employed, the signals were operating as fixed time signals with the plan pre-determined according to the time of the day and the day of the week. For the weekdays, three different plans were used, morning rush (6:00 - 9:00 AM), evening rush (3:00 - 7:00 PM) and non-rush periods (9:00 AM - 3:00 PM) and (7:00 PM -6:00 AM). The cycle length, green split and offset remained constant during each of these plans. As in any fixed-time signal system, the signal plans were optimized off-line based on average historical data. The plans were optimized using the PASSER II-90 program to maximize the through bandwidth for Orchard Lake traffic.

Because the cycle length was different for the before and after periods, and was variable in the after period, a comparison of the total effective green time per hour was used in the analysis. The total effective green time per hour allocated to each movement during the before and after periods were compared for the intersections along the corridor, the results are presented in Table 6.1. The before/after green time study revealed that SCATS, in general, extended the effective green time for the major road traffic. This increase in green time led to a corresponding reduction in delay and stops. Extending the green time increases the percentage of non-stopped vehicles and reduces the stopping time for stopped vehicles. SCATS also used longer cycle lengths, with less lost time per hour, thus utilizing the green time more efficiently.

A cycle-by-cycle degree of saturation comparison was used to test how effectively the green phase was used for each approach during a cycle. When the total entering traffic volume approaches the intersection capacity, a degree of saturation equal to or near an optimal value of 0.90, would mean that the system effectively assigned the right amount of green for that approach. Figure 6.2 presents the before/after cycle-by-cycle degree of saturation for northbound and westbound traffic, respectively (Orchard Lake Road and Walnut Lake Road intersection).

Table 6.1.a Green Time and Volumes (Walnut Lk Road Intersection)

Time	Orchard Lk. Rd.				Walnut Lk. Rd.			
	Through		Volume		Through		Volume	
	Fixed Time	SCATS	SB	NB	Fixed Time	SCATS	EB	WB
6:00 AM	26	33	757	295	18	13	351	65
7:00 AM	26	30	1678	690	18	16	620	160
8:00 AM	26	30	1703	700	18	17	589	215
9:00 AM	26	35	1204	620	16	14	353	195
10:00 AM	26	34	986	739	16	14	298	184
11:00 AM	26	34	969	948	16	14	318	198
12:00 PM	26	34	985	905	16	14	311	241
1:00 PM	26	33	931	885	16	14	271	228
2:00 PM	26	34	1076	1057	16	14	324	228
3:00 PM	24	28	946	1164	19	16	375	398
4:00 PM	24	27	890	1397	19	16	378	474
5:00 PM	24	24	918	1626	19	18	384	613
6:00 PM	24	27	968	1476	19	17	348	534
7:00 PM	26	33	885	1134	16	15	303	240
8:00 PM	26	34	880	1021	16	13	278	201
9:00 PM	26	32	887	884	16	14	273	196
10:00 PM	26	32	527	551	16	15	247	183
11:00 PM	26	30	299	388	16	14	150	131

Table 6.1.b Average Degree of Saturation (Walnut Lk Road Intersection)

Time	Orchard Lk. Rd.				Walnut Lk. Rd.			
	SB		NB		EB		WB	
	Fixed Time	SCATS	Fixed Time	SCATS	Fixed Time	SCATS	Fixed Time	SCATS
6:00 AM	0.51	0.40	0.20	0.16	0.69	0.96	0.13	0.18
7:00 AM	1.14	0.97	0.47	0.40	1.09	1.20	0.31	0.35
8:00 AM	1.16	1.00	0.48	0.41	1.15	1.22	0.42	0.44
9:00 AM	0.81	0.61	0.42	0.32	0.79	0.90	0.44	0.50
10:00 AM	0.66	0.51	0.50	0.38	0.67	0.78	0.41	0.48
11:00 AM	0.65	0.51	0.64	0.50	0.71	0.82	0.44	0.51
12:00 PM	0.66	0.52	0.61	0.48	0.70	0.78	0.54	0.60
1:00 PM	0.63	0.49	0.59	0.47	0.61	0.68	0.51	0.57
2:00 PM	0.72	0.56	0.71	0.55	0.73	0.84	0.51	0.59
3:00 PM	0.70	0.59	0.86	0.72	0.72	0.85	0.76	0.90
4:00 PM	0.65	0.59	1.03	0.93	0.72	0.83	0.90	1.04
5:00 PM	0.68	0.67	1.13	1.06	0.73	0.75	1.05	1.07
6:00 PM	0.71	0.64	1.09	0.98	0.66	0.74	1.02	1.13
7:00 PM	0.59	0.48	0.76	0.61	0.68	0.72	0.54	0.57
8:00 PM	0.59	0.46	0.69	0.54	0.62	0.74	0.45	0.53
9:00 PM	0.60	0.48	0.59	0.48	0.61	0.69	0.44	0.50
10:00 PM	0.35	0.29	0.37	0.31	0.55	0.58	0.41	0.43
11:00 PM	0.20	0.18	0.26	0.23	0.34	0.37	0.29	0.32

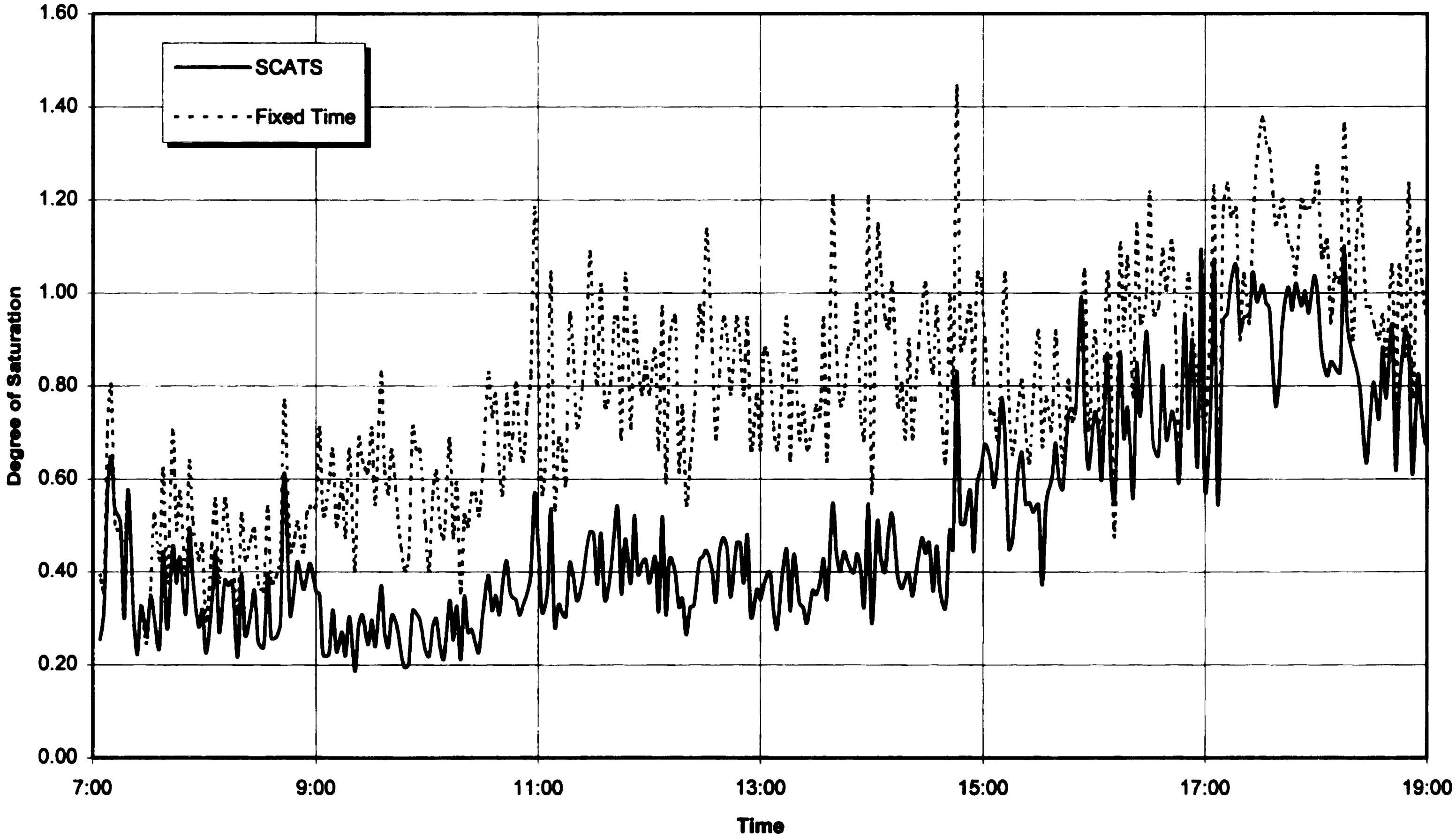

Figure 6.2 SCATS/Fixed Time Cycle-By-Cycle Degree of Saturation for Northbound Traffic

The result shows, in general that SCATS provided a consistent degree of saturation. However, the average difference in the degree of saturation between any two consecutive cycles under SCATS control was 0.14 for northbound traffic and 0.19 for westbound traffic). This is larger than might be expected from an adaptive/responsive system. This might be a result of the SCATS logic, which only allows the phase plan or cycle length to change after receiving two votes in any three consecutive cycles. This limits the ability of the system to respond to the cycle-by-cycle changes in demand. A second factor is that each intersection is forced, under the marriage mode, to have a common cycle length even if it is longer than the intersection optimal cycle length. A third factor is the prediction techniques used in SCATS which predicts the short-term variation in demand rather than cycle-to cycle variation.

Improving progression for the traffic along the corridor is another potential method of achieving a reduction in delay. Improving the progression would minimize the number of vehicles arriving during the red phase, thus reducing the delay and stops. A before/after progression analysis was conducted to investigate how SCATS dynamic progression and offset plan contributed to the savings in delay and travel time. Progression diagrams were used to compare the effectiveness of the before and after progression.

The total width of the through bandwidth per hour was chosen to be the measure of effectiveness (MOE) in this analysis. This measure takes into consideration the difference in cycle length between the before and after periods as well as the variation in cycle length during the after period. The results showed that SCATS, in general, provided a wider through bandwidth for the major street traffic during all time periods. The results also showed that both fixed time and SCATS controls achieved an optimal through bandwidth for the peak direction traffic during both the morning and the afternoon peak periods. The through bandwidth under each control had an optimal value equal to the total green time per hour for the intersection that had the lowest green time for the northbound/ southbound traffic. This increase in the through bandwidth under SCATS control is attributable mainly to the increase in the green time allocated for the northbound/southbound traffic under SCATS control.

6.5 The simulation study

The objectives of this simulation analysis were to determine if conventional signal control strategies can provide savings in travel time for Orchard Lake Road traffic similar to those achieved by SCATS. The simulation analysis was carried out for three different signal control strategies:

1) the base case, in which signal timings were set to represent the signal timing conditions during the before period. The signals were operating under fixed time

signals with a cycle length of 120 seconds during the peak-periods. The results from these simulation runs were compared against those collected in the field during the before period to calibrate the simulation model.

2) re-optimized fixed-time signals, in which the cycle length was increased to 140 seconds during the peak-periods. This cycle length value represents the average cycle length employed by SCATS during the peak period. The green split for different approaches was set to the average values employed by SCATS. The offset plans were obtained through the PASSER II-90 optimization of the network.

3) coordinated actuated signals, in which the common background cycle length and the offsets were set similar to the second control strategy. The major corridor traffic was set as the non-actuated approach and the gap-out was set to 3 seconds for all other approaches.

The analysis was carried out for two time periods; the morning peak period (7:00 AM to 9:00 AM) and the afternoon peak-period (4:00 PM to 6:00 PM). The southbound traffic was the peak direction during the morning peak-period, whereas, the northbound traffic was the peak direction during the afternoon peak period. The simulation was done using the CORSIM simulation model. One of the recent improvements to CORSIM that made the simulation output more reliable, is the ability to change traffic demand and turning movements within each time period. Traffic volumes were obtained from the SCATS monitoring file, which reports traffic volumes for each lane on a cycle-by-cycle basis. Simulations were run five times using five different random number seeds for each case. The results from the five runs were averaged to account for the variability associated with the randomness of the traffic.

6.6 Results from the simulation study

Figures 6.3 and 6.4 present average corridor travel time under different control strategies for southbound traffic (morning peak-period), and northbound traffic (afternoon-peak period), respectively. The corridor travel time obtained from simulation with the signal settings similar to that during the before SCATS period showed a similar pattern to the travel time data collected in the field. The error between the simulated and actual travel time values ranged from 1.01% to 3.07% for southbound traffic and 0.46% to 3.86% for northbound traffic.

The difference between the actual and simulated travel time is attributed to the stochastic nature of the traffic. While the simulated data is the average of all vehicles for five different runs using different random numbers, the field data is the average of 112 observations in a one-day period. While mid-block volumes were included in the simulation runs, the volume and distribution of such traffic might be different than what is actually in the field. This could be another possible source of the difference.

Nevertheless, the travel time pattern from the simulation results was similar to that observed in the field, which validates the output of the simulation analysis.

Tables 6.2 and 6.3 present the average values of corridor travel time under different control strategies during the peak 30-minute period, the non-peak 30-minute period and the overall average. Tables 6.4 and 6.5 present the percent reduction in corridor travel time under SCATS control compared to actual fixed-time data (before SCATS data), optimized fixed-time (simulated data) and coordinated actuated controllers (simulated data).

When compared to the optimized fixed-time signal, SCATS reduced arterial travel time by an average of 5.61% and 3.00% during the non-peak 30-minute periods for the southbound and northbound traffic, respectively. During the peak 30-minute period, the reduction in travel time averaged 0.26% and 0.49%, respectively. These values are similar to those obtained from the simulation model in the first part of this study. The PHF for the minor- street traffic during these time periods ranged from 0.87 to 0.93, with an average volume of 243 vehicles per lane per hour. The expected benefits from an adaptive system with minor street-traffic demand similar to these values would be in the range of 3% to 6% (Figure 5.22). The values obtained through this simulation study were within this range.

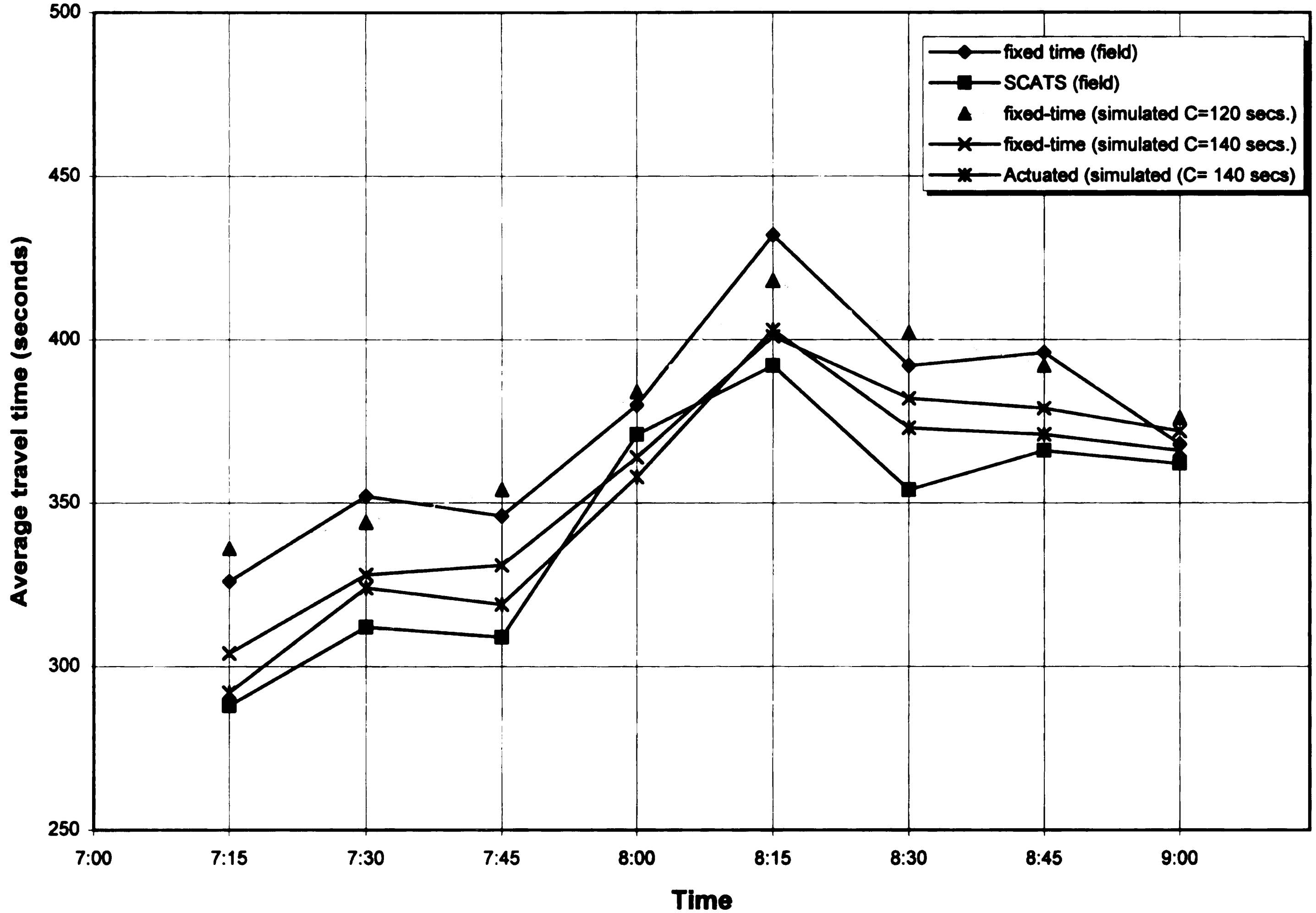

Figure 6.3 Average corridor travel time under different control strategies (southbound traffic - morning peak-period)

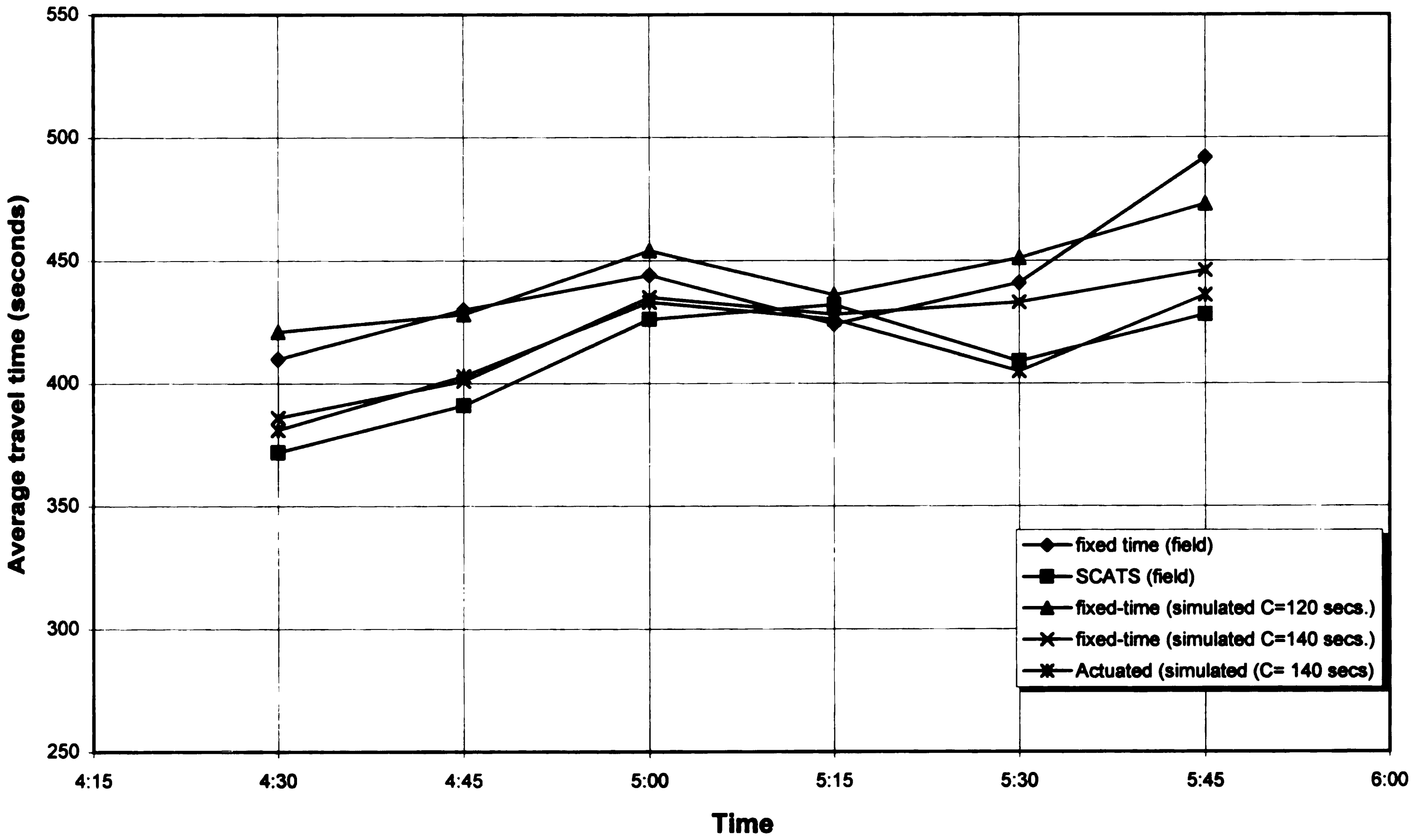

Figure 6.4 Average corridor travel time under different control strategies (northbound traffic-afternoon peak-period)

Travel time values under actuated signal control obtained through simulation were quite similar to those obtained in the field under SCATS control. The difference between the average travel time under these two types of control ranged between 0.02% to 2.76%. This validates the output of the simulation model which demonstrated that actuated signal control can achieve a reduction in travel time similar to that obtained through adaptive signal control.

Table 6.2 Average corridor travel time* (southbound traffic-morning peak-period)

	SCATS (field data)	Before SCATS (field data)	Optimized fixed-time (simulated data)	Actuated Simulated data
overall average	344.2	374.1	357.6	350.7
Average (non-peak)**	303.0	341.3	321.0	311.6
Average (peak)***	381.5	406.0	382.5	380.5

* travel time in seconds
** average values during the first 30-minute period
*** average values during the peak 30-minute period

Table 6.3 Average corridor travel time* (northbound traffic-afternoon peak-period)

	SCATS (field data)	Before SCATS (field data)	Optimized fixed-time (simulated data)	Actuated Simulated data
overall average	409.6	440.1	421.5	414.6
Average (non-peak)**	381.5	420.2	393.3	392.1
Average (peak)***	429.4	434.0	431.5	429.5

* travel time in seconds
** average values during the first 30-minute period
*** average values during the peak 30-minute period

Table 6.4 Percent reduction* in corridor travel time under SCATS control compared to different control strategies (southbound traffic-morning peak-period)

	Before SCATS Field data	Optimized fixed-time simulated data	Actuated Simulated data
overall average	7.99	3.75	1.85
Average (non-peak)	11.2	5.61	2.76
Average (peak)	6.03	0.26	-0.29

****Percent reduction = (travel time (different control strategies) - SCATS travel time) / travel time***

Table 6.5 Percent reduction in corridor travel time under SCATS control compared to different control strategies (northbound traffic-afternoon peak-period)

	Before SCATS Field data	Optimized fixed-time simulated data	Actuated Simulated data
overall average	6.93	2.82	1.21
Average (non-peak)	9.21	3.00	2.70
Average (peak)	1.06	0.49	.02

****Percent reduction = (travel time (different control strategies) - SCATS travel time) / travel time***

6.7 Comparative cost analysis of alternative signal control systems

Transportation engineers and planners use cost/benefit comparisons to decide whether the benefit of any signal control system would justify the cost of installing, operating and maintaining the system. Perhaps the most comprehensive comparison of the cost of alternative control strategies in the literature is that included in the UTCS study (Henry, et. al, 1976).

The average cost per intersection is highly dependent on the size of the network. For the first generation of control strategies, the average cost per intersection (in 1975 dollar) was $17,353 and $13,988 for networks of 100 and 500 intersections, respectively. Anderson (1984) presented the subsystem cost of an adaptive signal control system as a percent of the total cost over a ten-year period (1970-1980). The results suggest that the detector, the communication, and the central computer cost, as a percentage of the total cost, decrease over the 10-year period. He suggested that the cost of these subsystems would continue to decline over the years.

6.7.1 Cost components of alternative signal control systems

The cost of implementing a signal control system depends on many factors. The geographic location, the area type, and the size of the system are among the factors that affect the overall cost of the system. Data regarding the cost of fixed-time and actuated systems were obtained through different sources and represent average values. Data regarding the cost of SCATS adaptive system were obtained from OCRC files, and it represents the cost of installation with 1994 dollars. Table 6.7 shows the components of the cost of different systems.

Table 6. 7 Cost components of different signal control strategies

	Fixed-Time	Actuated	Adaptive
Signal Controller	Low	High	High
Detectors	None	High	High
Communication	None	None	High
Central Computers	None	None	High
System maintenance	Average	Average	High
Data maintenance	High	High	None

The average cost of replacing a fixed-time signal system with a coordinated actuated system would be in the range of $50,000 to $70,000 per intersection (Carrier and Gable, 1998). Based on OCRC cost data, the cost of implementing SCATS in the Oakland County traffic network ranged from $92,000 to $114,00 per intersection. This cost included the detection system (four Autoscope@ cameras per intersection), the local controller (one controller per intersection), and the average cost of the central computer and the communication system per intersection.

6.7.2 Benefit components of alternative signal control systems

After determining the expected cost of alternative control strategies, the next step is to assess the potential benefits of these alternatives. All relevant factors should be evaluated and quantified before any rational decision can be reached. Table 6.8 presents the components of the benefits for alternative signal control systems.

Table 6. 8 Benefit components of different signal control strategies

	Fixed-Time	Actuated	Adaptive
Improvement in traffic operation:			
-optimal control during peak-periods			
-optimal control all time periods			
-respond to incident and special event			
-Safety improvements			
-data maintenance and signal setting update			
- data for ATIS applications			

6.7.2.1 Benefit to traffic parameters (normal traffic conditions)

Campbell (1988) and Witkowski (1992) suggested that the change in the value of the estimated peak-hour person-hour of stopped delay or travel time during weekdays can be used as the measure of effectiveness when comparing different signal control strategies in urban areas. The value of the benefit can be assessed by multiplying the estimated saving in total travel time per hour by the assumed average hourly value of an individual travel time (average $4/hr in urban areas, Witkowski (1992)).

Assuming that the Orchard Lake Road corridor has a moderate peak-hour demand that lasts for two hours in the morning and two hours in the afternoon peak.

An adaptive control system would reduce the total travel time from an average of 223.76 vehicle-hours/hour to 213.78 vehicle-hours/hour, with savings 9.98 vehicle-hours/hour, (Table 5.5). Assuming an average occupancy 1.2 per vehicle, four peak-hours per day, and an average of 253 weekdays per years, the total benefits over one-year period would be:

$$\boldsymbol{9.98 \times 1.2 \times 4 \times 253 \times \$4/hr = \$48{,}488/year}$$

6.7.2.2 Benefit to traffic parameters (incident and special-event conditions)

Many studies concluded that adaptive control systems have the potential of reducing total delay resulting from incident situations. It is difficult; however, to quantify such benefit due to the randomness associated with incident occurrence, duration and severity. An average value, based of historical data of the non-recurring congestion resulting from incidents and special events can be used as guide to quantify such benefits

6.7.2.3 Changes of the adaptive control system benefit over time span

Another significant benefit of an adaptive system is the ability to optimize the system on a real-time basis, something that is very costly and labor-intensive for non-adaptive systems. As illustrated in section 5.3.4 of this study, the reduction in corridor travel time changes over time. When the fixed-time signal plans were kept unchanged, the average reduction in travel time increased from 4.16% in the base year, to 9.06% in the first year, to 26.33% in the second year. When compared with signal plans that optimized annually, the reduction in travel time decreased from 4.16% in the base year to 3.04% in the first year, then to 1.57% in the second year. The benefit of the adaptive control, thus, would be tied to the frequency of updating the signal plans in the network.

Signal plan updating requires extensive data collection and many man-hours of data management and network optimization. It would be justifiable thus, when estimating the benefit of the adaptive system over a span of time, to include the savings in data management and signal plan updating as a relevant benefit of the system. The value of this benefit would vary among agencies.

6.7.2.4 Other benefits of adaptive control systems

The benefits of adaptive signal control systems extend beyond the reduction in delay and travel time. Adaptive systems are the core of any Advanced Traveler Information Systems (ATIS) application as they provide the system with real-time traffic data required for route choice algorithms. While significant, it might be difficult to explicitly quantify its dollar value outside the ATIS benefit context. Many studies reported some safety benefits associated with adaptive signal control. The benefits ranged from marginal (Hanbali and Fornal, 1997) to significant as reported by Richeson and Underwood (1996).

Chapter 7

CONCLUSIONS

This research examined the potential benefits of adaptive signal control strategies for arterial corridors in urban networks. Two adaptive signal control strategies were examined against two conventional control methods, namely coordinated fixed-time and coordinated actuated signals. The difference between the two signal control strategies was in the way they predict traffic. In the first adaptive control strategy, a prediction of the traffic in any given cycle was made based in the traffic on the previous three cycles using traffic data collected via detectors placed downstream from the intersection. In the second adaptive control strategy, a prediction was made using detectors placed upstream to the intersection that scan the traffic expected to arrive at the intersection. The research was conducted using the CORSIM simulation model. Using the same platform, these four signal control strategies were tested. The measures of effectiveness used to identify the performance of each control strategy were average travel time, total travel time, and intersection delay parameters.

The effectiveness of the four signal control strategies was examined under two different demand levels. The demand levels were chosen to replicate moderate and high peak-hour conditions in an urban corridor. The output of the simulation study showed that both adaptive control strategies showed a reduction in corridor travel time over well-optimized fixed-time signals. The percent reduction in total travel time was higher in the moderate peak-hour demand case. Within each demand level, the savings in total travel time was higher during the non-peak periods and decreased as traffic demand reached its peaking (Table 7.1).

Table 7.1 Percent reduction in total travel time under adaptive control systems

	15-minute non-peak periods	15-minute peak period
High peak-hour demand	1.89	0.40
Moderate peak-hour demand	5.90	1.20

When compared with the coordinated actuated signals, neither of the two adaptive control strategies examined in this study showed a significant difference in corridor travel time or intersection delay parameters. This was true in both moderate and high peak-hour demand levels. Thus, it can be concluded that, within the demand levels examined in this study, a coordinated actuated signal system can provide savings in corridor travel time similar to those achieved by an adaptive control system.

The output of a signal timing analysis study showed that the reduction in corridor travel time was achieved mainly through reallocating the extra green time, provided to the minor-street traffic under fixed time control, to the major-street traffic. This happened when the minor-street traffic demand was lower than its peak design values during the non-peak periods. Similarly, during peak periods, when minor-street traffic demand reaches its peak design value, the value of the extra green time that can be reallocated decreases, decreasing the potential corridor travel time savings by adaptive signal control systems.

The sensitivity of the adaptive signal control benefit to various traffic parameters was also examined. These parameters included minor-street traffic demand and PHF, major-street traffic demand, the phase plan, and a change of demand over time. The output showed that the saving in corridor travel time under adaptive signal control system is more sensitive to the PHF of the minor-street traffic than other traffic demand parameters. As the PHF decreases, the excess green time allocated for the minor-street traffic increases, providing the adaptive control system with more potential benefits. No significant relationship was found between the percentage reduction in corridor travel time and changes in minor-street or major-street demand levels.

The corridor travel-time savings were not limited to a specific signal plan. Similar corridor travel time saving was obtained when the signals operated in two-phase and four-phase plans.

The reduction in corridor travel time changes over time. When the fixed-time signal plans were kept unchanged, the average reduction in travel time under adaptive control increased from 4.16% in the base year, to 9.06% in the first year, and to

26.33% in the second year. When compared with signal plans that optimized annually, the reduction in travel time decreased from 4.16% in the base year to 3.04% in the first year, then to 1.57% in the second year.

The accuracy of the two prediction techniques used in the adaptive control systems was examined. The output showed that detectors placed upstream to the intersection predicted the expected number of vehicles for each cycle more accurately. However, the accuracy of predictions made using detectors placed downstream from the intersection improved when the prediction horizon increased to 5-minute intervals. There was no significant difference in the performance of the two adaptive control systems that used these two prediction techniques.

A field study to examine the effectiveness of deploying SCATS adaptive control system in the Orchard Lake Road corridor showed that the reduction in corridor travel time achieved under SCATS control was within the benefit limits estimated by the simulation model. The results also showed that, similar to what the simulation model output demonstrated, coordinated actuated signals might have achieved the same reduction in travel time.

This research has contributed to the understanding of the potential benefits of adaptive signal control strategies in urban corridors. It should be noted that the output reported in this study is limited to the geometric configurations and demand levels examined in the study. The benefits of the adaptive systems might vary if the spacing between the intersections is significantly different than those examined in the study, since the signal spacing is a significant factor in determining the benefits of signal progression.

The study also identifies several areas where there is a need for further research. This research can, ultimately, lead to a set of rules and guidelines for choosing a signal control system among different alternatives within an ITS context. The followings are some proposed areas for further research in the framework of this study:

1. The study can be extended to cover additional demand levels during non-peak periods and oversaturation conditions.
2. The study can be extended to examine different geometric configurations, (intersections that are closely spaced or set further apart).
3. The same simulation model can be modified and used to examine the effectiveness of adaptive control systems for urban networks.
4. Different adaptive control logic can be tested using the same simulation model introduced in this study.
5. The same study framework can be used to test the effectiveness of adaptive control strategies in urban networks under different demand levels.

APPENDICES

Appendix A

Data structures in CORSIM

Variable name	Description	Fortran common block
IMXLNK	Maximum allowable number of links in a network	SNET
IMXNOD	Maximum allowable number of nodes in a network	SNET2
IMXDET	Maximum allowable number of detectors in a network	SNET3
YINIT	Flag that records if the simulation has reached equilibrium or not.	GLR091
DWNOD[IL]	Downstream node number of link IL, where IL is the CORSIM link ID.	SIN036
UPNOD [IL]	Upstream node number of link IL, where IL is the CORSIM link ID	SIN062
NMAP[IN]	User-specified node number that corresponds to the internally-assigned NETSIM subnetwork node, number, IN	SIN075
CLOCK	Current time since start of the simulation in seconds.	SIN104
TTLNK	Total number of links in subnetwork.	SIN116
DTLNK[DT]	Link Number surveillance detector DT is on.	SIN700
DTLEN(DT)	Length of Detector DT in tenths of a foot	SIN313
DTMOD(DT)	1-3 Detector type (0=Presence, 1=Passage) 4-10 Speed of vehicle when passing the passage detector. 11-23 Vehicle count since beginning of simulation.	SIN314
DTNLNK(DT)	Detector identification number of the next detector on the same link as detector DT.	SIN308
DTPOS(DT)	Distance between the detector's downstream edge and the downstream stop-bar, in tenths-of-a-foot.	SIN312
DTFLNK(IL)	Detector identification number of the first detector on the referenced link, IL.	SIN307

Variable name	Description	Fortran common block
DETON(DT)	Bit specific array: 1 (0,1) if detector was (on,off) for first 0.1 second 2 (0,1) if detector was (on,off) for second 0.1 second 3 (0,1) if detector was (on,off) for third 0.1 second 4 (0,1) if detector was (on,off) for forth 0.1 second 5 (0,1) if detector was (on,off) for fifth 0.1 second 6 (0,1) if detector was (on,off) for sixth 0.1 second 7 (0,1) if detector was (on,off) for seventh 0.1 second 8 (0,1) if detector was (on,off) for eighth 0.1 second 9 (0,1) if detector was (on,off) for ninth 0.1 second 10 (0,1) if detector was (on,off) for tenth 0.1 second	SIN070
TOTDET	Number of surveillance detectors specified for this subnetwork. The data for each detector is contained in arrays DTCTR1, DTCTR2 and DTCTR3.	SIN109
AMBSPC[IL]	The subscript K is the link identification number and contains the movements which are in amber. 1 (0,1) if signal for right turners (is, is not) amber 2 (0,1) if signal for through vehicles (is, is not) amber 3 (0,1) if signal for diagonal turners (is, is not) amber 4 (0,1) if signal for left turners (is, is not) amber 5 - 11 Time remaining (sec) in amber for right turners 12-18 Time remaining (sec) in amber for through vehicles 19-25 Time remaining (sec) in amber for diagonal turners 26-32 Time remaining (sec) in amber for left turners	SIN366
SDCODE[IL]	Signal codes of current, active, signal interval controlling traffic on this link, fetched from XINT1 or XINT2. 1 Signal code for right turn vehicles 2 Signal code for through vehicles 3 Signal code for diagonal (left or right) vehicles 4-5 Signal code for left turn vehicles Where Signal Code is defined as follows 0 GO, permitted and protected 1 NO GO, not permitted 2 COND, GO, permitted but unprotected (left turners	SIN050

Appendix B

Source Code for the adaptive control logic interface with CORSIM

```
C Program ADAPTIVE_1.for - PC version
C By:    Ahmed Shawky Abdel-Rahim
C Date:  4/27/98
C Last updated: 6/16/98
C *** FOR PH.D DISSERTATION *****
C Purpose: INTERFACE  on a real-time basis with CORSIM
C
C --------------   GLOSSARY OF VARIABLE NAMES   --------------------
C                  ============================
C  CLOCKR  REALTIME CLOCKTIME
C  CREAL   STRING OF FLAGS FOR SECOND-SPECIFIC DATA
C  CREALD  STRING OF FLAGS FOR DEBUG DATA
C  CRELCF  CHARACTER - PATHNAME WHERE THE REAL-TIME SOFTWARE
C          READS AND WRITES TO THE CONTROL FILES
C  CRELDF  CHARACTER - PATHNAME WHERE THE REAL-TIME SOFTWARE
C          READS AND WRITES TO THE DATA FILES (DIRECTORY WHERE
C          NETSIM WAS EXECUTED)
C  FLREAL  ARRAY OF FLAGS SPECIFYING WHICH FILES TO CREATE FOR REAL-
C          TIME TRAFFIC ADAPTIVE SIGNAL CONTROL SYSTEMS
C  ISMLEN  TIME TO COMPLETE SIMULATION TO END OF CURRENT TP, SECONDS
C  IT      INDEX OVER TIME PERIOD DURATIONS
C  IW      FLAG SET (0,1) IF CONTROL FILE (HASNT,HAS) BEEN UPDATED
C  ITOTME  TOTAL EXECUTION TIME SET FOR NETSIM SIM RUN, SECONDS
C  LENINT  LENGTH OF A TIME INTERVAL, SECONDS
C  LNTMPR  ARRAY OF TIME PERIOD DURATIONS, SECONDS
C  LU21    PERIPHERAL UNIT NUMBER 21
C  LU26    LOGICAL FILE (CONTROL FILE FOR SECOND-SPECIFIC DATA)
C  LU27    LOGICAL FILE (CONTROL FILE FOR DEBUG DATA)
C  LU33    LOGICAL FILE (FILE FOR DEFINING PATHNAME, ITOTME, LENINT,
C          AND LENPRD)
C  WRETV   FLAG SET (T,F) IF (IS,ISNT) TIME TO RETRIEVE NETSIM DATA
C
    IMPLICIT INTEGER (A-Q, S-V, X), REAL (R, Z), LOGICAL (W, Y)
C
    COMMON /GLR898/ CLOCKR
    COMMON /GLR183/ CRELCF
    COMMON /GLR184/ CRELDF
    COMMON /GLR894/ FLREAL(  5)
    COMMON /GLR060/ LENINT
    COMMON /GLR014/ LNTMPR(  19)
    COMMON /GLR888/ LU21
    COMMON /GLR893/ LU26
    COMMON /GLR895/ LU27
```

```
      COMMON /GLR185/ LU33
C
      CHARACTER CREAL*32, CREALD*11, CRELCF*35, CRELDF*35
C
      INQUIRE (FILE='RELNET.PTH', EXIST=WEX)
      IF (.NOT. WEX) THEN
        WRITE (*, 7002)
        CALL EXIT
      ELSE
C
C ----- CHECK PATHNAME FOR THE CONTROL FILES.
C
        CALL FOPEN (33)
        READ (LU33, 7003) CRELCF
        IC = ICHAR (CRELCF(1:1))
        IF (IC .LT. 65 .OR. IC .GT. 122 .OR.
     1     (IC .GT. 90 .AND. IC .LT. 97)) THEN
          WRITE (*, 7004)
          CALL EXIT
        ENDIF
        IF (CRELCF(2:2) .NE. ':') THEN
          WRITE (*, 7005)
          CALL EXIT
        ENDIF
        IC = 36
    5   CONTINUE
        IC = IC - 1
        IF (CRELCF(IC:IC) .EQ. ' ' .AND. IC .GT. 1)     GO TO 5
        IF (CRELCF(IC:IC) .EQ. '\') CRELCF(IC:IC) = ' '
C
C ----- CHECK PATHNAME FOR THE DATA FILES.
C
        READ (LU33, 7003) CRELDF
        IC = ICHAR (CRELDF(1:1))
        IF (IC .LT. 65 .OR. IC .GT. 122 .OR.
     1     (IC .GT. 90 .AND. IC .LT. 97)) THEN
          WRITE (*, 7007)
          CALL EXIT
        ENDIF
        IF (CRELDF(2:2) .NE. ':') THEN
          WRITE (*, 7008)
          CALL EXIT
        ENDIF
        IC = 36
   10   CONTINUE
        IC = IC - 1
        IF (CRELDF(IC:IC) .EQ. ' ' .AND. IC .GT. 1)     GO TO 10
        IF (CRELDF(IC:IC) .EQ. '\') CRELDF(IC:IC) = ' '
        WRITE (*, 7011) CRELCF, CRELDF
        PAUSE
        CLOSE (LU33)
      ENDIF
C
```

```
C ----- GET TIME INVARIANT DATA.
C
      CALL FOPEN (26)
      READ (LU26, 5001) ITIME, IRCODE, INCODE, CREAL
      IF (ITIME .EQ. 1 .AND. INCODE .EQ. 1) CALL RINV
      CLOSE (LU26)
C
C ----- DETERMINE TOTAL LENGTH OF CORSIM SIMULATION RUN, IN SECONDS.
C
      ITOTME = 0
        DO 20 IT = 1, 19
          ITOTME = ITOTME + LNTMPR(IT)
   20   CONTINUE
      IT = 1
      ISMLEN = LNTMPR(IT)
C
C ----- LOOP OVER SECONDS.
C
        DO 60 ICLOCK = 1, ITOTME
          CLOCKR = ICLOCK
          IW = 0
          WSEC = MOD (ICLOCK, FLREAL(1)) .EQ. 0
          WTI = FLREAL(2) .EQ. 1 .AND. MOD (ICLOCK, LENINT) .EQ. 0
          WTP = FLREAL(3) .EQ. 1 .AND. MOD (ICLOCK, ISMLEN) .EQ. 0
          WDB = FLREAL(4) .EQ. 1
          WRETV =  WSEC .OR. WTI .OR. WTP .OR. WDB
C
C ----- UPDATE THE SIMULATION RUN LENGTH IF INTO THE NEXT TIME PERIOD.
C
          IF (ICLOCK .EQ. ISMLEN + 1 .AND. ICLOCK .NE. ITOTME) THEN
            IT = IT + 1
            ISMLEN = ISMLEN + LNTMPR(IT)
          ENDIF
C
C ----- READ THE SECOND-SPECIFIC CONTROL FILE.  "WAIT" BY LOOPING
C ----- IF NECESSARY UNTIL CORSIM PROCESSES ANOTHER SECOND AND UPDATES
C ----- CONTROL FILE (INCODE = 1).
C
   25     CONTINUE
   30     CONTINUE
          CALL FOPEN (26)
          READ (LU26, 2000) ITIME, IRCODE, INCODE
          IF (INCODE .NE. 1)  THEN
            CLOSE (LU26)
                                        GO TO 30
          ENDIF
          BACKSPACE LU26
C
C ----- CORSIM HAS PROCESSED ANOTHER SECOND.  RESET FLAGS.
C
          IRCODE = 1
          INCODE = 0
C
```

```
C ----- REALTIME IS READY TO RETRIEVE CORSIM DATA (WRETV = TRUE).
C
      IF (WRETV) THEN
C
C ----- THE CORSIM SIMULATION TIME IS THE SAME AS THE REALTIME
C ----- PROCESSING TIME.
C
        IF (ITIME .EQ. ICLOCK) THEN
C
C ----- GET DESIRED NETSIM DATA.
C
          IF (WSEC) CALL RSEC (CREAL)
          IF (WTI) CALL RTI
          IF (WTP) CALL RTP
C
C ----- MAKE CHANGES TO SIGNALS AND OUTPUT IF IT IS TIME.
C
          IF (WSEC) THEN
            IDUM = 0
              DO 35 I = 1, 10000
                IDUM = IDUM + 0
  35           CONTINUE
            CALL ROUTS (CREAL)
          ENDIF
C
C ----- GET DEBUG DATA.
C
          IF (WDB) THEN
  40          CONTINUE
            CALL FOPEN (27)
            IF (ICLOCK .EQ. 1) THEN
              READ (LU27, 6000) ITIMD, IRCODD, INCODD, CREALD
            ELSE
              READ (LU27, 2000) ITIMD, IRCODD, INCODD
            ENDIF
            IF (ITIMD .EQ. ICLOCK .AND. INCODD .EQ. 1) THEN
              CALL RSECD (CREALD)
              CLOSE (LU27)
            ELSE
              CLOSE (LU27)
                                  GO TO 40
            ENDIF
          ENDIF
C
C ----- REALTIME IS READY TO RETRIEVE BUT MUST WAIT UNTIL CORSIM
C ----- CREATES THE DATA (FOR THIS SECOND). UPDATE CONTROL FILE.
C
        ELSE
          WRITE (LU26, 5000) ICLOCK, IRCODE, INCODE, CREAL
          CLOSE (LU26)
                                  GO TO 25
        ENDIF
C
```

```
C ----- REALTIME IS NOT READY TO RETRIEVE.  UPDATE CONTROL FILE AND
C ----- RETURN TO PROCESS NEXT SECOND.
C
        ELSE
          IW = 1
          WRITE (LU26, 5000) ICLOCK, IRCODE, INCODE, CREAL
          CLOSE (LU26)
        ENDIF
C
C ----- BE SURE TO UPDATE CONTROL FILE.
C
        IF (IW .EQ. 0) THEN
          WRITE (LU26, 5000) ICLOCK, IRCODE, INCODE, CREAL
          CLOSE (LU26)
        ENDIF
END
```

Appendix C

Source file for CORSIM

```
 CORSIM main source file - Prepared by Ahmed Shawky Abdel-Rahim              00
       for Ph. D. Thesis dissertation                                         00
       April, 1998                                                            00
                                                                              00
                                       9   24  98                             01
       1   0     -15     7981         300000       3 8                  7581  02
 300 300 300 300 300 300 300 300 300 300 300 300                              03
                  60                                                          04
                                                                              05
8021  21             2                     1              20  22   0          11
  21   12000         2       7T           11  31      11  20  22  30   11     11
   1  212000         2       TT         8021              20  22  30   11     11
   1  112000         2       TT         8011              20  22  30   11     11
  11   12000         2       7T           21   2      21  20  22  30   11     11
8011  11             2                     1              20  22   0          11
8022  22             2                     2              20  22   0          11
  22   22000         2       7T           12   1      12  20  22  30   11     11
   2  222000         2       TT         8022              20  22  30   11     11
   2  122000         2       TT         8012              20  22  30   11     11
  12   22000         2                    22   3      22  20  22  30   11     11
8012  12             2                     2              20  22   0          11
8023  23             2                     3              20  22   0          11
  23   32000         2       7T           13   2      13  20  22  30   11     11
   3  232000         2       TT         8023              20  22  30   11     11
   3  132000         2                  8013              20  22  30          11
  13   32000         2       7T           23   4      23  20  22  30   11     11
8013  13             2                     3              20  22   0          11
8024  24             2                     4              20  22   0          11
  24   42000         2       7T           14   3      14  20  22  30   11     11
   4  242000         2       TT         8024              20  22  30   11     11
   4  142000         2       TT         8014              20  22  30   11     11
  14   42000         2       7T           24   5      24  20  22  30   11     11
8014  14             2                     4              20  22   0          11
8025  25             2                     5              20  22   0          11
  25   52000         2       7T           15   4      15  20  22  30   11     11
   5  252000         2       TT         8025              20  22  30   11     11
   5  152000         2       TT         8015              20  22  30   11     11
  15   52000         2       7T           25  35      25  20  22  30   11     11
8015  15             2                     5              20  22   0          11
8035  35             2       TT            5              20  22   0   11     11
  35   52600         2       7T            4  25       4  20  22  45   11     11
   5  352600         2       TT         8035              20  20  45   11     11
   5   43700         2       7T            3  24       3  20  22  45   22     11
   4   53700         2       7T           35  15      35  20  20  45   11     11
   4   33700         2       7T            2  23       2  20  20  45   11     11
   3   43700         2       7T            5  14       5  20  20  45   11     11
   3   23200         2       7T            1  22       1  20  22  45   11     11
   2   33200         2       7T            4  13       4  20  20  45   11     11
   2   13200         2       7T           31  21      31  20  22  45   22     11
   1   23200         2       7T            3  12       3  20  20  45   11     11
```

1	313200			2		TT			8031				20	20	45	11		11
31	13200			2		7T			2	11		2	20	22	45	11		11
8031	31			2					1				20	22	0			11
8021	21		100															21
21	1		90	10														21
1	21		100															21
1	11		100															21
11	1		90	10														21
8011	11		100															21
8022	22		100															21
22	2		90	10														21
2	22		100															21
2	12		100															21
12	2		90	10														21
8012	12		100															21
8023	23		100															21
23	3		90	10														21
3	23		100															21
3	13		100															21
13	3		90	10														21
8013	13		100															21
8024	24		100															21
24	4		90	10														21
4	24		100															21
4	14		100															21
14	4		90	10														21
8014	14		100															21
8025	25		100															21
25	5		90	10														21
5	25		100															21
5	15		100															21
15	5		90	10														21
8015	15		100															21
8035	35		100															21
35	5		90	10														21
5	35		100															21
5	4		90	10														21
4	5		90	10														21
4	3		90	10														21
3	4		90	10														21
3	2		90	10														21
2	3		90	10														21
2	1		90	10														21
1	2		90	10														21
1	31		100															21
31	1		90	10														21
8031	31		100															21
1	90	21	11	2	31	87	5	23	5	0	0	0	0	0	0	0	0	35
2	41	22	12	3	1	87	5	23	5	0	0	0	0	0	0	0	0	35
3	112	23	13	4	2	87	5	23	5	0	0	0	0	0	0	0	0	35
4	56	24	14	5	3	87	5	23	5	0	0	0	0	0	0	0	0	35
5	0	25	15	35	4	87	5	23	5	0	0	0	0	0	0	0	0	35
35		8035	5															35
31			18031															35
21		8021	1															35
22		8022	2															35
23		8023	3															35
24		8024	4															35
25		8025	5															35
11			18011															35

```
  12        28012                                                                            35
  13        38013                                                                            35
  14        48014                                                                            35
  15        58015                                                                            35
   1 2299 2200 9922 0022 0000 0000 0000 0000 0000 0000 0000 0000                             36
   2 2299 2200 9922 0022 0000 0000 0000 0000 0000 0000 0000 0000                             36
   3 2299 2200 9922 0022 0000 0000 0000 0000 0000 0000 0000 0000                             36
   4 2299 2200 9922 0022 0000 0000 0000 0000 0000 0000 0000 0000                             36
   5 2299 2200 9922 0022 0000 0000 0000 0000 0000 0000 0000 0000                             36
  25   5   1              1   200  1                                                         42
  35   5   1    22480     1   200                                                            42
  35   5   1     2950         200                                                            42
8021  21 475                                                                                 50
8011  11 475                                                                                 50
8022  22 475                                                                                 50
8012  12 475                                                                                 50
8023  23 475                                                                                 50
8013  13 475                                                                                 50
8024  24 475                                                                                 50
8014  14 475                                                                                 50
8025  25 475                                                                                 50
8015  15 475                                                                                 50
8035  352200                                                                                 50
8031  311600                                                                                 50
2222  31    1    1    2    2    3    3    4    4    5    5   35                             90
1111  35    5    5    4    4    3    3    2    2    1    1   31                             90
   0                                                                                        170
   1     4000    18400                                                                      195
   2     4000    15200                                                                      195
   3     4000    12000                                                                      195
   4     4000     8300                                                                      195
   5     4000     4600                                                                      195
  35     4000     2000                                                                      195
  31     4000    21600                                                                      195
  21     6000    18400                                                                      195
  22     6000    15200                                                                      195
  23     6000    12000                                                                      195
  24     6000     8300                                                                      195
  25     6000     4600                                                                      195
  11     2000    18400                                                                      195
  12     2000    15200                                                                      195
  13     2000    12000                                                                      195
  14     2000     8300                                                                      195
  15     2000     4600                                                                      195
8021     7000    18400                                                                      195
8022     7000    15200                                                                      195
8023     7000    12000                                                                      195
8024     7000     8300                                                                      195
8025     7000     4600                                                                      195
8011     1000    18400                                                                      195
8012     1000    15200                                                                      195
8013     1000    12000                                                                      195
8014     1000     8300                                                                      195
8015     1000     4600                                                                      195
8035     4000      500                                                                      195
8031     4000    22600                                                                      195
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