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16. Abstract This research report summarizes findings from a research project and is organized into four chapters. Chapter 1 identifies operational problems associated with congestion and provides a summary of existing technology. Chapter 2 summarizes the results of field studies that measure headways during oversaturated traffic conditions. These field studies were conducted in Houston and Austin, Texas. The purpose of these studies was to investigate whether headways increased, decreased, or remained constant on approaches with long queue lengths and long green times. These studies show that driver expectancy plays an important role in determining headways. These studies also show that average headway is independent of a vehicle's position in queue; however, variations in headways increase for vehicles further back in the queue. Chapter 3 summarizes findings from simulation studies conducted to compare five coordination strategies for a five-intersection arterial with one oversaturated direction. The results of this study show that coordination of actuated signals for progressing traffic flow in the congested direction produces lower delays, fewer stops, and shorter queues. Chapter 4 presents the architecture of a real-time traffic control system for coordinated-actuated control, and discusses various issues related to demand estimation. This chapter also describes how these ideas are being implemented in Richardson, Texas, and concludes with the description of a prototype real-time graphical interface for use by operators.					
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**REAL-TIME COORDINATED-ACTUATED  
TRAFFIC CONTROL DURING  
CONGESTED CONDITIONS**

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# CHAPTER I INTRODUCTION

## BACKGROUND

In recent years, urban traffic demand in Texas has grown at an alarming rate. Most signalized arterials in all medium to large Texas cities currently face, oftentimes severe, traffic congestion. This trend is expected to continue through the turn of the century. Due to financial and right-of-way constraints it is increasingly difficult to add traffic lanes to relieve this situation. In many cases, making full use of the existing roadway capacity remains the only feasible option. This objective can be achieved by implementing real-time traffic management and control strategies that provide efficient, responsive, and/or adaptive control of arterial signal systems. To this end, the statewide traffic control system development is rapidly moving toward real-time traffic control to respond to the dynamic nature of traffic demand. These systems can take advantage of the existing infrastructure of loop detectors.

This report summarizes the findings of project 0-1288 that deals with real-time coordinated-actuated control during oversaturated traffic conditions on signalized arterials. This chapter provides background material and discusses problems associated with traffic congestion. The following chapters provide a summary of research conducted under various sub-tasks.

## URBAN TRAFFIC SIGNALS

The main reason for installing a traffic signal on an urban roadway intersection is to provide safe right-of-way and equitable service to a number of competing traffic movements. Once a traffic signal is installed, it must be operated to move traffic efficiently across it. The need for coordinating adjacent traffic signals on an arterial arises from the dependencies in the flow of traffic from one signal to the next. Signal parameters needed for coordinating adjacent signals depend on whether the signal controllers are pretimed or actuated. Table 1 provides a list of parameters for the two types of controllers.

**Table 1. Types of Signal Control Parameters**

<b>Pretimed Signals</b>	<b>Actuated Signals</b>
Green splits Yellow and all-red clearance times Sequence of signal phases Signal cycle length Offsets	Minimum and maximum green times Green-time extension Yellow and all-red clearance Sequence of phases Yield points and Force-off points

Coordination of adjacent traffic signals on an arterial brings stability to the system and improves overall traffic flow. However, it requires that all signals operate at a common cycle length and the timings at various signals on the arterial system are tied through the specifications of certain timing parameters at each signal. Thus, coordination imposes constraints on individual signals that may compromise the overall efficiency of some signals in order to improve the total efficiency of the coordinated signal system.

In a pretimed controller, the cycle length and green splits (phase times) are fixed. Hence, it is simple to synchronize pretimed signals by specifying offsets between adjacent signals. However, in an actuated controller, the cycle length normally varies from cycle to cycle depending on vehicle demand. Thus, a fixed background cycle length is artificially imposed to achieve coordinated operation of two or more controllers. To accomplish this feature, actuated controllers provide functions such as, permissive periods, hold, yield points, and force-off points. The use of these functions forces the controller to operate within the constraints of a background cycle while still allowing the controller to operate in an actuated mode. In addition, coordination requirements are achieved by defining a background cycle, yield points for coordinated phases, and force-off points for actuated phases. The next section describes the basic operation of an actuated controller.

### Actuated Controllers

Figure 1 uses the example of three-phase control to illustrate the operation of an actuated controller. In this example, we assume phase 1 is coordinated and phases 2 and 3 are actuated. Key definitions are provided below.

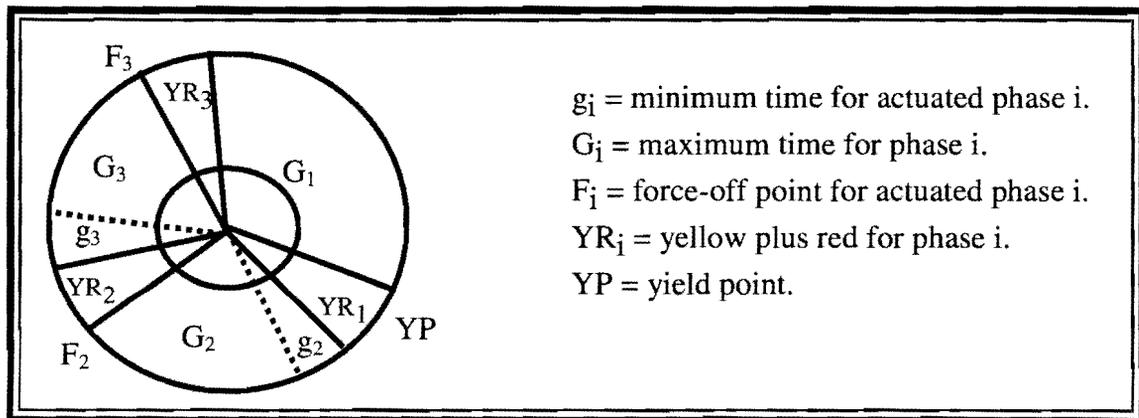


Figure 1. Actuated Controller

#### *Yield Point*

It is the time at which the green time of coordinated phase terminates. This point serves as the reference point for coordination. The yield point is a fixed point in the background cycle, and is equivalent to the offset for the coordination of pretimed signals.

### *Minimum and Maximum Greens*

The synchronization or coordinated phase is not actuated and usually has a guaranteed minimum green. However, other phases are actuated and their green times vary during each signal cycle. The green time of an actuated phase varies between specified minimum (min) and maximum (max) green times. The extension of the green time beyond the minimum green depends on the arrival rate of vehicles and the vehicle extension interval.

### *Permissive Periods*

A permissive period is a duration in which a secondary (actuated) phase is allowed to register a request for service from the coordinated phase(s). A permissive period is characterized by a start and end of the period by the release of hold on the coordinated phase(s), while omits are released on specific secondary phase(s). Which phases are allowed service depends on the time left in the cycle. For example, in a four-phase controller with phase two (2) as coordinated phase, the first permissive might allow service to phases one (1), three (3), and four (4). The second permissive might allow service to phases three (3) and four (4) - but not one (1). The third and last permissive would allow service to phase four (4) only.

### *Hold*

Under coordinated operation, controller internally applies hold to the phases designed as the coordinated phases when no permissive periods are active. As each permissive period begins, hold becomes inactive and when the permissive period ends, hold becomes active once more, unless another permissive period immediately becomes active, whereby hold is again released.

### *Pedestrian Permissive*

The pedestrian (ped) permissive designates a specific time duration at the beginning of each vehicle permissive period to respond to calls for pedestrian service. Since the controller cannot force off during the walk or ped clearance of a phase, the ped permissive period restricts pedestrian service to the beginning of each vehicle permissive so that late ped calls will not inhibit a force-off from terminating the phase at the proper time.

### *Force-Off*

A force-off is used to terminate a specific actuated phase (or phases) at a specific time within the cycle. This allows subsequent phases to be serviced at the proper time in the cycle. The force-off should always terminate an actuated phase since there is always a call on at least another phase - namely the coordinated phase. Force-off occurs such that the minimum green and/or walk plus ped clearance of the phase has already been timed out.

## **Signal Control Strategies**

A number of signal control/coordination objectives may exist depending on traffic conditions, the number of signals in an arterial system, roadway geometry, and intersection geometry. These objectives include maintaining driver safety, minimizing vehicular delay, maximizing progression bandwidth, minimizing stops to vehicles, minimizing travel time, and minimizing queue lengths. Of these objectives, safety of drivers has the highest priority. Furthermore, some of these objectives support each other, while some conflict with others. Examples of the conflicting nature of some of these objectives are given below:

Example 1: In order to maximize arterial progression during uncongested conditions, traffic engineers often implement signal timings that provide more green time to arterial approaches at the expense of side-street approaches to a signal. Thus, the objective of providing equity among signalized approaches is compromised to promote arterial efficiency and driver expectancy. Another consequence of maximizing arterial progression is additional delay for vehicles at minor approaches. Depending on the side-street demand, maximizing bandwidth may also result in higher total system delay.

Example 2: Traffic engineers often use split phasing to provide safety at the expense of signal capacity.

Control objectives other than those mentioned above also exist. The selection of a set of control objectives and the optimal balance between several conflicting objectives (safety excluded) depends on the traffic situation at hand. Later, we will provide further discussion of control objectives, especially those that need to be considered for congested systems. But first, congestion and related problems will be discussed.

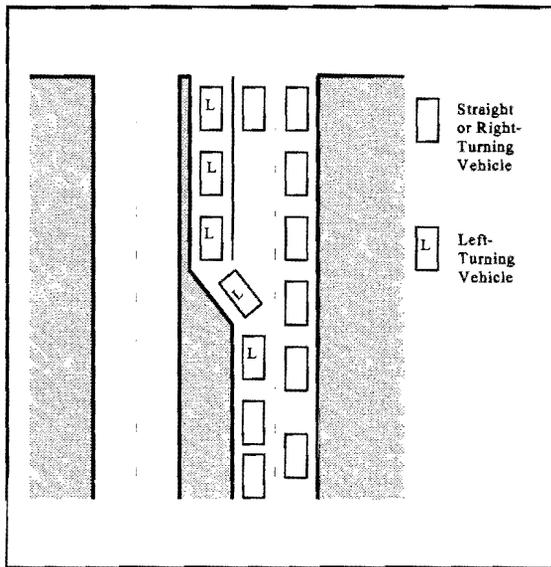
## **Congestion and Associated Operational Problems**

At higher demand levels, non-optimal signal timings can result in a reduction in capacity, which can cause congestion. Therefore, it is essential to prevent this situation by implementing optimal signal timings which take into account randomness in traffic streams. True congestion at a signal approach occurs when traffic demand equals or exceeds the available capacity of that approach. When demand approaches capacity (demand to capacity ratio exceeds 0.95), queues forming at signalized approaches to intersections fail to clear during a signal cycle. The uncontrolled growth and eventual spillback of these queues result in severe operational problems by restricting traffic flow. These problems are discussed below.

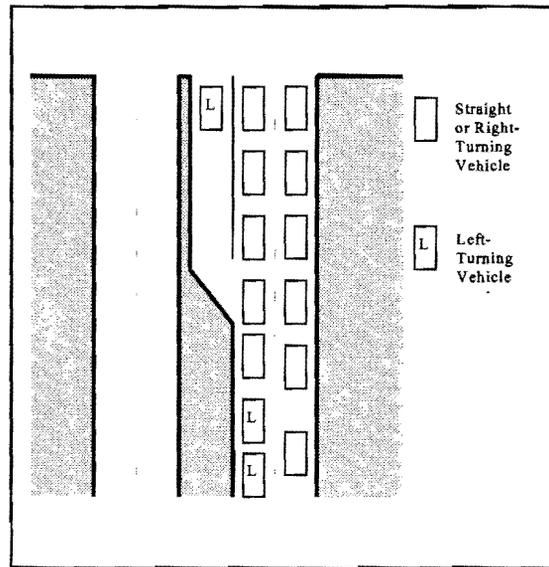
### *Starvation*

One immediate effect of a growing queue (primary congestion) at a signalized approach is a condition known as starvation. Starvation occurs when the queue from one movement prevents

demand for another movement from being serviced during the green indication for that approach. Figures 2 and 3 illustrate two cases of starvation. Case I shows queue spillback from the left-turn bay causing starvation of the through movement. In this situation, the signal indication is green for the through movement but the through vehicles in the adjacent lane cannot move. Case II shows a queue in the through lane blocking access to the left-turn bay. In this situation, the signal is green for the left-turn movement but the left-turn vehicles blocked by the queue cannot use this green time. If steps are not immediately taken to resolve these problems, queue growth will start to affect the vehicular flow at the upstream signal.



**Figure 2. Starvation, Case I**



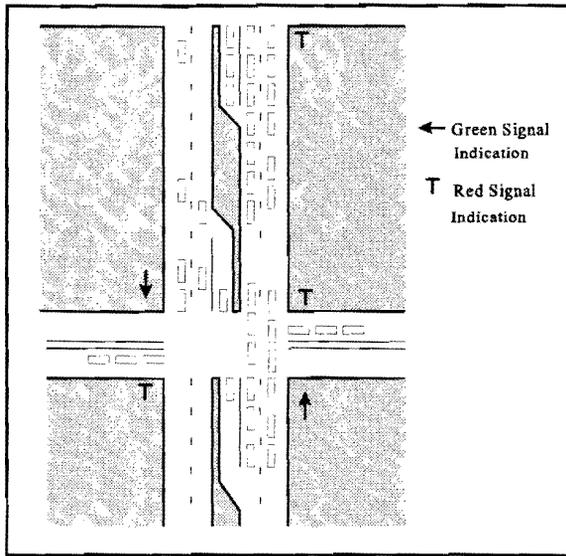
**Figure 3. Starvation, Case II**

### *Upstream Blockage and Queue Spillback*

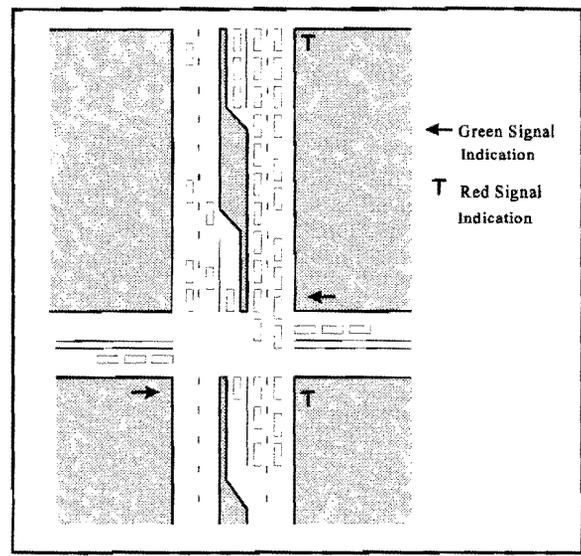
If the queue at a link is allowed to grow, it will eventually start to interfere with the traffic flow at the upstream signal by creating partial or full blockage. Partial blockage (Figure 4) occurs when the back of a standing queue reaches close (i.e., within 70 meters) to the upstream signal. This situation results in a significant reduction of capacity of the traffic movement joining the back of the queue. Full blockage (Figure 5) occurs when the back of the queue extends to or spills back into the upstream signal. Full blockage can affect all movements at the upstream signal by reducing their capacities to zero and causing a gridlock situation.

### *Control Objectives during Congested Conditions*

When the above problems arise, traditional control objectives (i.e., delay minimization and bandwidth maximization) become secondary due to their ineffectiveness or inapplicability. Since the formation of queues is inevitable during congested conditions, queue management becomes the top priority objective.



**Figure 4. Partial Upstream Blockage**



**Figure 5. Full Upstream Blockage**

Queue management includes several sub-objectives. These objective include: minimizing queue growth rate, controlling maximum queue length, eliminating or minimizing queue spillback and starvation, and maximizing throughput of vehicles in the system.

## **CURRENT TECHNOLOGY AND ITS LIMITATIONS**

Since the advent of traffic signals, a large number of researchers have investigated various issues related to the safety and operational efficiency of isolated as well as coordinated traffic signals. However, the current technology is limited in its application to congested arterial.

### **Signal Timing Optimization and Simulation Software**

A host of computer programs exist for assisting the traffic engineers in optimizing the timings of coordinated signals which include TRANSYT 7F (1), PASSER II (2), PASSER II (3), and PASSER IV (4). However, these optimization tools can only be used for signal systems facing undersaturated traffic conditions. This limitation is due to their inability to explicitly model queues in time as well as space. Another limitation of these programs is their inability to explicitly deal with actuated controllers. Guidelines are available to transform timings produced by these programs for use in actuated controllers (5).

CORSIM (6), a widely accepted microscopic-stochastic simulation model, is the only tool that applies to all traffic conditions. CORSIM can only evaluate specified scenarios and it is extremely cumbersome to use as a tool for generating signal control strategies. In addition, it can only emulate basic actuated control and does not have the ability to simulate many coordinated-actuated control

features available in modern actuated controllers. Nevertheless, CORSIM is a useful tool in assessing various strategies.

### **Guidelines for Operating Traffic Signals**

Literature is full of articles and reports that present guidelines for the safe and efficient operation of pretimed traffic signals in undersaturated conditions (7, 8, 9, 10, 11, 12, 13, 14). These include guidelines for selecting left-turn treatments and determining saturation flow rates. Lin (15) used computer simulation to develop a knowledge base to assist in the choice between permissive versus protected/permissive phasing for actuated signal control. The factors include left-turn volume, opposing volume, the number of opposing lanes, the length of left-turn bay, and cross-street volume. A major limitation of the existing methodologies is their inability to explicitly deal with traffic queues and the true demand under congested conditions. Researchers' interest in developing tools and guidelines for dealing with traffic congestion is not new: however, few practical guidelines currently exist for operating coordinated-actuated signals during congested conditions. A brief summary of literature and current limitations of technology is provided below.

Michalopoulos (16) presented a numerical model for analyzing congested arterials. This model is based on the concept of discretizing the problem in time and space. In other words, the roadway segment is divided in small sections which are analyzed in several discrete time slices; however, the model has not been validated through field testing. Kim and Messer (17) proposed a dynamic optimization model for maximizing system productivity and minimizing delay at conventional and three-level interchanges facing oversaturated conditions. The model divides the control period into a number of fixed time-slices and keeps track of queue carryover from one time slice to the next. But, because of the close distance between the signals of an interchange, discretization (division into short consecutive sections) of link distance was not taken into consideration. A recent project conducted the most thorough research dealing with oversaturated traffic signals in urban networks. The project report (18) describes the problems associated with controlling congested approaches, provides a list of all possible objectives, and discusses their conflicting nature. The report also presents guidelines and a mathematical formulation for optimally timing a signal system to achieve the internal metering of demand. Although this research is significant, it is short of what is needed for implementation in the field. The main limitation of this research is it being applicable to two-phase pretimed signals only. In addition, the guidelines are embedded deeply in mathematical equations, and it is difficult for an engineer to utilize them.

### **Real-Time Control**

Following the introduction of computer-based signal control systems in the 1960's, numerous experiments were conducted to develop more advanced control strategies. One of the most comprehensive studies was carried out by the U.S. Federal Highway Administration (FHWA) in Washington, D.C. and was labeled the UTCS (Urban Traffic Control System) experiment. The UTCS project was directed toward the development and testing of various network control concepts and strategies. This research and testing of control strategies was divided into three generations (19):

- First Generation Control (1-GC) - This mode of control uses a pre-stored library of signal timing plans which are calculated off-line, based on historical traffic data. The system provides options for selecting a timing plan on a time-of-day basis, by the operator, or by automatically matching a timing plan from an existing library that best suits to recently measured traffic conditions (volumes and occupancies). The recommended frequency of updates in a traffic-responsive mode is 15 minutes. TxDOT's FACTS (later renamed as ATMS) system (20) falls into this category;
- Second Generation Control (2-GC) - This is an on-line strategy that computes and implements signal timing plans based on surveillance data and predicted values. The optimization process can be repeated at five-minute intervals, however, plans cannot be implemented more often than every 10 minutes. 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; and
- Third Generation Control (3-GC) - This strategy was designed to implement and evaluate a fully responsive, on-line traffic control system. Similar to 2-GC, 3-GC computes control plans to optimize a network-wide objective using as input predicted traffic conditions. Differences from 2-GC were that the times between updates were reduced (3 to 5 minutes), and it allowed the cycle length to vary from one signal to another and from one control period to the next.

The advanced real-time systems were unsuccessful due to the lack of infrastructure, limitations of computer hardware and signal controllers, and limitations of communications technology. Due to the recent advances in technology, especially computer hardware, interest in real-time traffic control systems has resurfaced, and many U.S. agencies have taken steps to develop real-time traffic control systems as described below.

Under its RT-TRACS (Real-Time Traffic Adaptive Control Systems) initiative, FHWA has recently funded several research projects to develop real-time traffic control strategies. A number of strategies have been proposed by researchers and are currently being evaluated/tested in a laboratory environment using simulation. These tests use video technology for vehicle detection. FHWA plans to field test the most promising of these strategies in the near future; however, the full implementation of such strategies will take some time. This is due to the fact that the infrastructure needed (i.e., advanced surveillance capabilities) for implementing these strategies is not yet in place.

In recent years, TxDOT has funded several projects to bring the current technology and state-of-practice one step closer to meeting the needs for real-time traffic control. Since the video detection technology has not yet reached a mature state, the emphasis of these projects is to make full use of the extensive infrastructure of loop detectors in Texas. The second objective of these projects was to employ off-the-shelf signal control hardware, personal computers, and operating systems. One of these projects enhanced PASSER IV and developed guidelines for its use in a 1.5 generation

system (21). A 1.5 generation system is similar to a traffic responsive system (1-GC), but performs on-line signal timing optimization to match with the detected traffic conditions instead of selecting signal timings from a library of pre-determined signal timings. This research project overlaps the above project and expands the results of the previous project. One additional objective of this project is to propose a real-time control system that provides for the full utilization of a vast number of actuated controllers already operating in the field.

## **ORGANIZATION OF THIS REPORT**

The remainder of this report consists of three chapters as follows:

- Chapter 2 provides the results of field studies conducted to investigate the characteristics of vehicle discharge headways in the presence of long queues and long green splits. These field studies only consider situations when no spillback or blocking effect are present;
- Chapter 3 presents a summary of simulation studies conducted to compare five progression strategies during congested traffic conditions. These simulation studies use synthetic demand data to study congested traffic conditions. This allowed us to conduct a comparison of several progression strategies without having to divert our attention to the issues of demand estimation during congested conditions; and
- Chapter 4 discusses the architecture of a proposed real-time system. The bulk of this chapter discusses real-time data needs and issues related to accurate estimation of traffic demand during congested conditions.



## **CHAPTER II**

### **HEADWAYS IN OVERSATURATED CONDITIONS**

#### **BACKGROUND**

Many drivers believe that congestion can be alleviated by increasing the cycle length to provide more green time to congested approaches. Traffic engineers/analysts, on the other hand, believe that long green times on an approach tend to lower the capacity of an intersection because the headways between vehicles increase. We performed field studies to measure headways at congested intersections to ascertain whether headways increased, decreased, or remained constant on approaches with long queue lengths and long green times. This chapter presents a summary of results from these field studies.

#### **STUDY SITES**

Initially, we selected four study sites in Texas. Three of the sites were located in Houston and one was in Austin. We used the following criteria to select these sites:

- Presence of congestion and long queues during at least one peak period, and
- Existing signal timings with long phase lengths or an ability to implement long phase lengths on at least one of the study approaches.

One of the Houston sites was dropped due to the following two reasons:

1. During the data-collection effort for one approach (westbound) with heavy traffic, we observed Harris County Sheriff deputies halting westbound traffic so that vehicles could turn into and out of a major shopping center located on the northeast corner of the intersection. Although the driveway was located approximately 150 meters away from the intersection, the actions of the officers disrupted the normal headway patterns on this approach.
2. On the northbound approach, queues formed by heavy through demand often prevented left-turning vehicles from accessing the turn lanes. As the queue progressed forward during the green phase, left-turning vehicles exiting the through lane created artificially high headways.

The researchers successfully collected data on the following three sites:

1. State Highway (SH) 6 at Clay Road on the northwest side of Houston,
2. FM 1960 at Stubner Airline/Veterans Memorial located on the north side of Houston, and
3. Ben White Blvd. at I-35 West Frontage Road, located in Austin.

## DATA COLLECTION

Data were collected to study the relationship between headways and a vehicle's position in queue. Headways were measured as the time difference between the rear axle of the first vehicle to the rear axle of the following vehicle. A computer program recorded the time when the rear axle of a vehicle crossed the stop line. Different keys were used to record the times for automobiles and trucks (defined as a vehicle having three or more axles). The program computes headways by calculating the time difference between the current vehicle and the previous vehicle in the same lane.

Data were collected during peak traffic periods (either the A.M. or P.M. peak) when significant queues were present and long cycle lengths were used. Data were collected as long as significant queuing was present or a maximum of 30 cycles. While data were collected for all vehicles on all approaches, only those cycles where a truck was not present were used in the final analysis.

## DATA ANALYSIS

Table 2 summarizes where and when the headway data were collected in the field studies at each site. Because the headway data were collected at different locations, during different study periods, and in different lanes, the first analysis step was to determine if a significant difference existed in the headway data because of these factors. If these factors were judged not to impact the headway data significantly, then all of the data could be combined into one database; however, if a factor was determined to impact the headway data significantly, then the data would have to be analyzed based on that factor.

**Table 2. Summary of Data Collection Site Characteristics**

Study Location	Study Period	Study Lane
Ben White at I-35 West Frontage Road	A. M. and P. M. Peaks	Inside Only
FM 1960 at Stubner Airline/Veteran's Memorial	P. M. Peak Only	Inside and Middle
SH 6 at Clay Road	P. M. Peak Only	Inside and Middle

We used the Statistical Analysis System (SAS) computer software for statistical analysis. Analysis of variance (ANOVA) procedures were employed to test whether any of the above factors (i.e., study location, study period, or lane) significantly impacted the headway data at a 95 percent confidence level. Table 3 summarizes the results of the ANOVA procedures. This table shows that the headway data were significantly different as a result of a vehicle's position in a queue and by the study location. Both the period when the headway data were collected and the lane in which vehicles were traveling were determined to not impact the headway data significantly. The results of the analysis

implied that the headway data, as a function of a vehicle's position in queue, would have to be analyzed on a site-by-site basis.

**Table 3. Summary of Results from ANOVA Investigation of Data Collection Factors**

Model Variable	F Value	Prob > F	Significant Factor ?
Position in Queue	12.42	0.0004	Yes
Location	7.55	0.0005	Yes
Period	0.68	0.4113	No
Lane	1.38	0.241	No

Although the ANOVA results showed that the headway data differed significantly as a function of a vehicle's position in the queue, they do not show whether the headways were increasing or decreasing. A regression analysis was used to examine the relationship between headways and positions of vehicles in the queue. The general form of the linear model used in the regression analysis is as follows:

$$\text{Headway} = B_0 + B_1 * \text{Position in Queue},$$

where:

$B_0$  = Intercept of the regression line, and

$B_1$  = Slope of the regression line.

Because the ANOVA determined that headways differed by study location, separate regression analyses were performed on the headway data collected at each location. The model significance was tested at a 95 percent confidence level.

## RESULTS

Table 4 summarizes the results of the regression analysis. This table shows that the regression model used to represent the relationship between headways and position in queue was significant at all three sites. Table 5 summarizes the parameter estimates of the regression equation and their significance level. Figures 6, 7, and 8 show the headway data as a function of a vehicle's position in queue and the resulting regression equation.

**Table 4. Summary of Results of Regression Analysis**

Location	Source	DF	Sum of Squares	Mean Square	F Value	Prob > F	Correlation Coefficient (R <sup>2</sup> )
Ben White	Model	1	17.39	17.39	37.107	0.0001	0.0351
	Error	1019	477.51	0.46860			
	Total	1020	494.90				
FM 1960	Model	1	26.03	26.03	23.72	0.0001	0.0206
	Error	1126	1235.85	1.10			
	Total	1127	1261.88				
SH 6	Model	1	12.52	12.52	9.88	0.0017	0.0083
	Error	1176	1491.16	1.27			
	Total	1177	1503.69				

**Table 5. Parameter Estimates of Regression Equations**

	Variable	DF	Parameter Estimate	Standard Error	T for H0: Parameter=0	Prob >  T
Ben White	Intercept	1	1.991810	0.0506	39.364	0.0001
	Position in Queue	1	-0.0163161	0.0027	-6.092	0.0001
FM 1960	Intercept	1	1.695296	0.0693	24.472	0.0001
	Position in Queue	1	0.015369	0.0031	4.870	0.0001
SH 6	Intercept	1	1.724308	0.0728	23.687	0.0001
	Position in Queue	1	0.009791	0.0031	3.143	0.0017

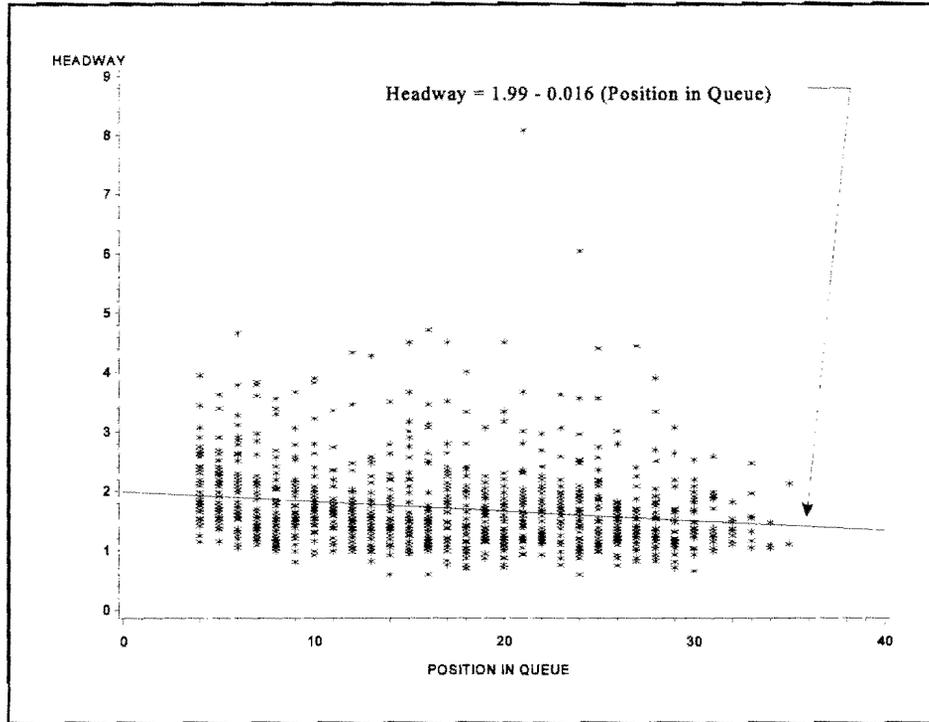


Figure 6. Headway Versus Position in Queue for Ben White Blvd.

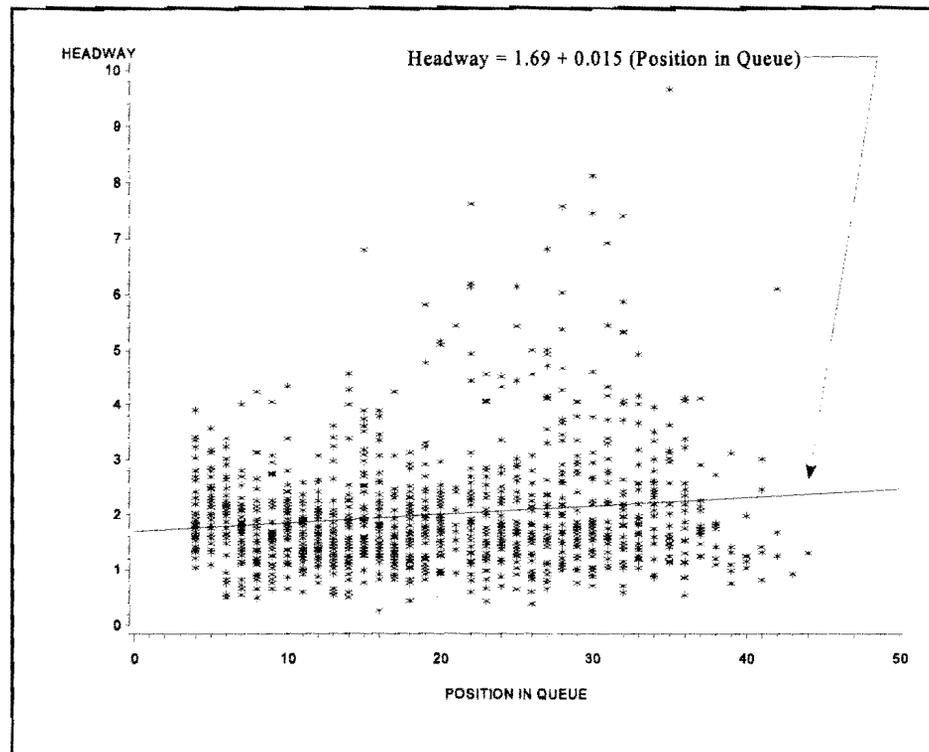
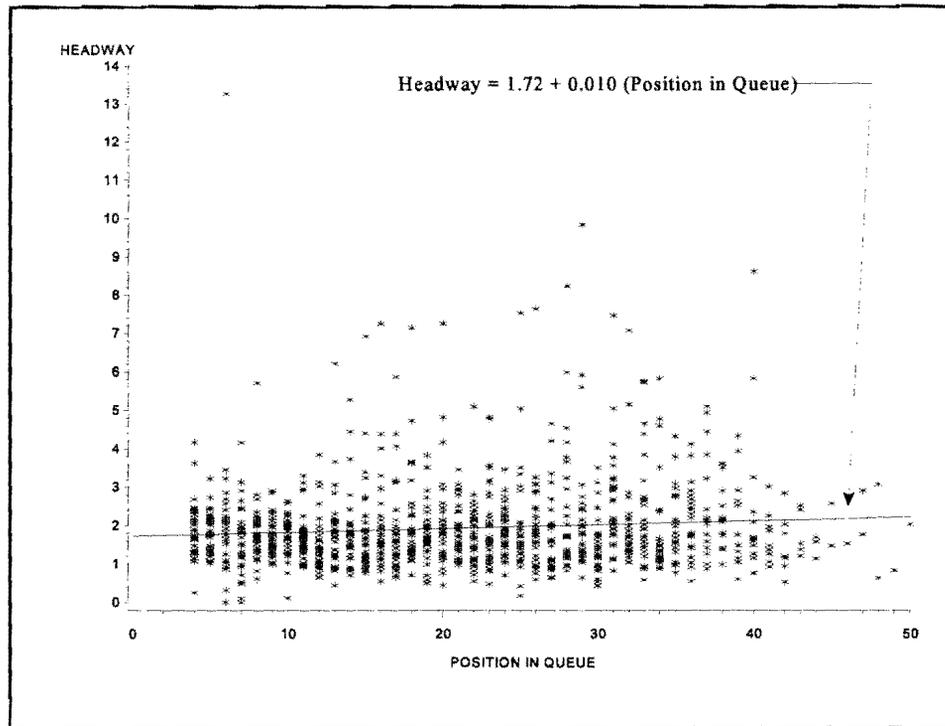


Figure 7. Headway Versus Position in Queue for FM 1960



**Figure 8. Headway Versus Position in Queue for SH 6**

### Discussion of Results

Figure 6 and Table 5 show that at the Ben White Blvd. at I-35 West Frontage Road site, headways are actually decreasing the further back in a queue a vehicle is located. In other words, the further back a vehicle is located in the queue, the closer in time vehicles travel to other vehicles. This relationship is opposite to those observed at the other two sites: FM 1960 at Stubner Airline/Veteran's Memorial and SH 6 at Clay Road. At both of these sites, headways increased as a vehicle's position in the queue increased. One possible explanation for the difference in the relationships is that at the Ben White intersection, drivers are accustomed to having a long cycle length and push to keep headways close together. At the other two sites, the cycle length was increased specifically for this study, and drivers may not have been accustomed to seeing longer cycle lengths and drove more tentatively through the intersection.

Although the statistical tests indicated a relationship between headway and position in the queue, it was a weak one. To measure the degree of this relationship, statisticians use the correlation coefficient ( $R^2$ ). The correlation coefficient represents that portion of the total variability of the headway values that can be accounted for by a vehicle position in the queue.

A correlation coefficient close to 1.0 implies that 100% of the variation in headways is caused by a vehicle's position in the queue. Table 4 shows that the correlation coefficient approaches zero.

This implies that hardly any of the variation in the headway data can be attributed to a vehicle's position in queue. A review of Figures 6 through 8 shows that headways vary widely from cycle to cycle for all queue positions, and a vehicle's position in the queue may not be a good indicator of its headway.

## **CONCLUSIONS**

Because different phenomenon were observed at different sites, the results of this study were inconclusive. Based on the data obtained, the researchers were unable to ascertain if headways actually increased, decreased, or remained constant the further back a vehicle was located in the queue. Most of the variation that results in headways at an intersection is caused by random factors which are not measured in this study.

The results showed that if there is a change in headways as a result of a vehicle's position in the queue, it is slight. For all practical purposes, it can be assumed that the average time headway between vehicles in the queue remains constant for all positions in the queue. One interesting observation from the study is that the variations in headways tended to increase further back in the queue. This implies that it is more difficult to get the same number of vehicles through an intersection on a consistent basis as the queue size grows.



## **CHAPTER III**

### **STRATEGIES FOR MAINTAINING PROGRESSION IN OVERSATURATED CONDITIONS**

#### **BACKGROUND**

Maintaining progression during oversaturated conditions is difficult. Queues that form as a result of insufficient capacity can inhibit the movement of platoons through a coordinated signal system, thereby reducing the overall effectiveness of the signal system. This chapter summarizes the findings from a simulation study which evaluates four potential strategies for maintaining some level of progression on an oversaturated arterial.

#### **COORDINATION STRATEGIES**

The objective of this study was to identify and test different coordination strategies for maintaining some level of progression on an arterial during oversaturated conditions. The goal was to identify strategies that traffic engineers could develop using the existing PASSER II traffic signal optimization software, which is a bandwidth optimization program [2].

The following five coordination strategies were identified for setting the signal timings for oversaturated arterials:

- Provide standard two-way progression with equal bandwidths;
- Provide maximum bandwidth for traffic moving in the congested direction;
- Provide maximum bandwidth for traffic traveling in the uncongested direction;
- Provide two-way progression up to and away from the congested intersection; and
- Provide continuous progression in the uncongested direction, but break the progression band at the congested intersection in the congested direction.

#### **SIMULATION STUDY**

A simulation study was performed to compare the performance of a five-intersection arterial under control of each of the candidate signal control strategies, including the standard two-way traffic signal-timing strategy. The following sections describe the measures of effectiveness obtained from the simulation studies to compare various strategies.

## Measures of Effectiveness

### *Primary*

The primary measures of effectiveness (MOEs) used in the evaluation were as follows:

- Total system delay and average system delay. The unit for the total system delay was vehicle-hours, and the unit for the average system delay was minutes/vehicle;
- Throughput in both directions on the main street. Throughput is the number of vehicles per time unit discharged from any movement. The throughput values used in this research were obtained by expressing the total traffic volume discharged in 1120 seconds of congested simulation in units of vehicle per minutes;
- The average queue length at each intersection. The queues were expressed in units of vehicles; and
- Phase failures at each intersection. A phase failure is the number of phases that the queue failed to clear each intersection at the end of the green phase. The phase failures do not have any units and were expressed as numbers.

Researchers measured total system delay and average system delay at the end of two simulation periods. During the second simulation period, no new traffic was allowed to enter the system. This technique allowed all the queues that had formed during the simulation to clear the system and be accounted for in the measures of effectiveness. This technique resulted in a more accurate measurement of system delay (both total and average). Researchers collected all the other MOEs at the end of the first simulation period.

### *Exploratory*

Since total system delay represents the amount of delay experienced by all drivers in the system, individual drivers have no perception of total system delay. Instead, drivers tend to be more concerned with the number of stops they have to make once they enter the system. It may be more frustrating to the driver to experience stops at several intersections than several stops at a single intersection.

Along with the primary MOEs, two exploratory MOEs were also investigated for this research. These were the total number of stops a randomly selected car made at the intersections on the arterial and the individual number of stops at each intersection. These unconventional MOEs were chosen to explore whether they might be valuable for use in future studies. They take into consideration what an individual driver experiences while traveling on an arterial.

Visual inspection of the animation on the computer monitor was used to measure how many stops drivers had to make at each intersection once they entered the arterial. The vehicular motion of one particular car was observed for the selected demand-to-capacity (D/C) ratios of 0.86, 1.00, and 1.21 for each of the five strategies and for the first five replication runs for each of the strategies. This resulted in a total of 75 cars being tracked on the arterial.

### Test Network

Researchers developed a hypothetical arterial street of five intersections for this research. Since only through traffic was used in the simulations, no left-turning lanes were provided at the intersections. The traffic signals at all the intersections were operated in a fully actuated, but coordinated mode. By design, the third intersection in the peak flow direction was the critical intersection. Figure 9 shows the spacing of the intersections in the arterial system.

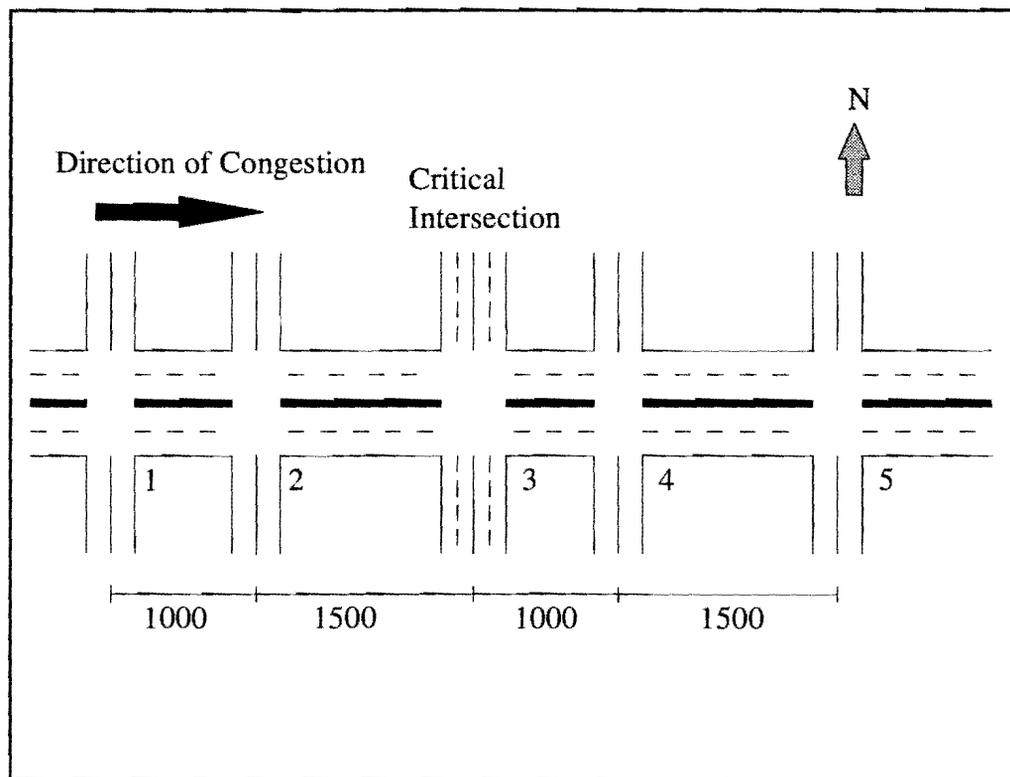


Figure 9. Hypothetical Network Used to Evaluate Coordination Strategies

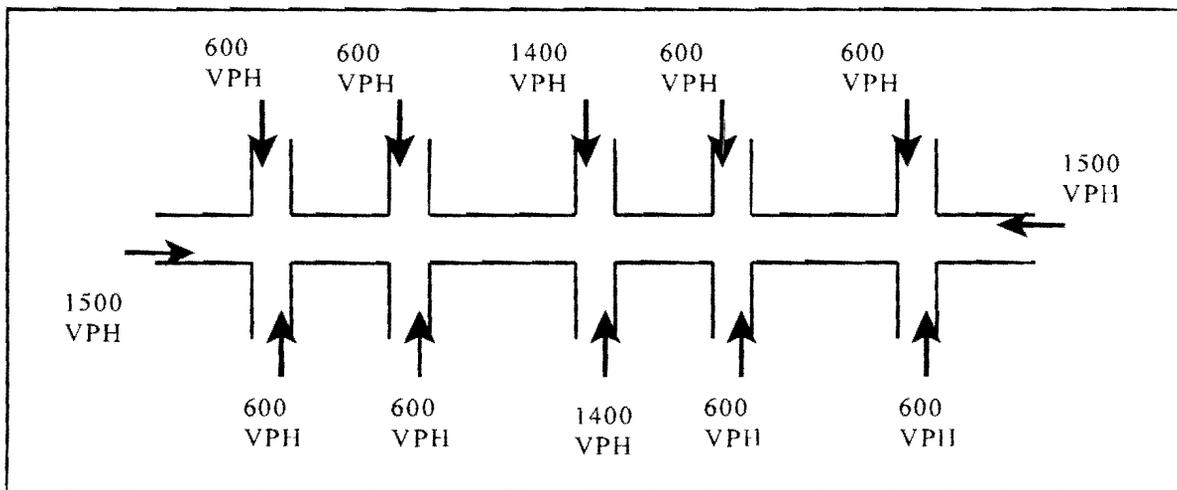
### Traffic Volumes

Before traffic volumes simulating oversaturated traffic could be developed, it was first necessary to assume a cycle length. It was also important not to select a cycle length that is too long because intersections operating with a long cycle length can block the adjacent upstream intersection,

especially when the distance between the intersections is small. For the purposes of this study, a 140-second cycle length was selected for the following two reasons:

- Many agencies in Texas commonly use 140 seconds as the maximum cycle length for oversaturated intersections; and
- A 140-second cycle length was close to the system minimum delay cycle computed by PASSER II.

Once the cycle length was established, the next step was to assume a suitable travel speed for the arterial. An arterial speed of 38 mph was arbitrarily selected. The cycle length and the assumed speed were then used to estimate the volume required to load the arterial to the desired demand-to-capacity (D/C) ratios. To simulate the conditions of near saturation, the arterial was loaded with traffic so that all the links had an approximate D/C ratio of about 0.86. This was done by using the PASSER II software package and by employing a trial and error procedure. Figure 10 gives the volumes required for achieving demand-to-capacity ratios of 0.86.



**Figure 10. Cross-Street and Main-Street Volumes Required to Achieve D/C Ratio of 0.86**

The next stage was to increase the through movement in the eastbound direction to simulate the building of congestion in one direction. Rising congestion was simulated by increasing the D/C ratio at the critical intersection (third intersection) from near saturation (D/C of 0.86) to oversaturation (D/C of 1.21). For simplicity, all the traffic movements in the arterial street were through movements. Researchers entered the traffic volumes into PASSER II to compute the splits at the critical intersection. Table 6 shows the volume levels, the resulting D/C ratios, and signal timing splits used in the simulation.

**Table 6. Volume Conditions, D/C Ratios, and Phase Splits Simulated During Study**

Eastbound Through Volume (Vph)	Computed D/C Ratio for Critical Link Using Rounded Webster's Splits	Signal Splits at Critical Intersection	
		Main Street	Cross-Street
1500	0.86	72.0	68.0
1600	0.89	74.0	66.0
1800	0.95	78.0	62.0
2000	1.00	82.0	58.0
2200	1.06	85.0	55.0
2300	1.09	86.0	54.0
2500	1.14	89.0	51.0
2700	1.21	91.0	49.0

These traffic volumes were coded in both PASSER II and TRAF-NETSIM software packages for the different strategies. Offsets obtained from the PASSER II runs were consequently coded into TRAF-NETSIM for simulation.

### Data Collection

TRAF-NETSIM simulations for a total of 1,820 seconds (30.33 minutes) were performed for each of the five strategies. The total simulation time was divided into two periods. The first period was 1,120 seconds (18.67 minutes). During this period, the network was fully loaded with the design demand. The second period was 700 seconds (11.67 minutes), during which no demand was allowed to enter the system. The reason for using the second time period was to allow vehicles queued up at the end of the first simulation period to clear the arterial.

Only passenger vehicles were used for the purpose of simulation. Each strategy was evaluated under the eight different volume conditions. Each simulation run was then replicated nine times for a total of 10 runs for each volume condition and strategy. This resulted in a total of 400 simulation runs.

To collect exploratory MOE data for the congested directions, the following procedures were used:

- A car entering the system at a specified time was observed as it traveled in the congested direction on the arterial. The observation times were chosen in a way to include the entire range of the heavily loaded simulation period. Table 7 shows the time intervals at which the vehicles were selected for tracking across the network for D/C ratios of 0.86, 1.00, and 1.21, respectively. The same entry time was used to track vehicles for each strategy. Researchers

discovered that by using the identical entry time, the performance of the same vehicles could be monitored under each strategy; and

- The cars entering the arterial at each of these times were tracked, and the number of stops the car made at each of the intersections was recorded.

**Table 7. Time at Which Vehicles Were Selected for Observation**

Vehicle Number	Select Entry Time for Vehicle Observation		
	D/C = 0.86	D/C = 1.00	D/C = 1.21
1	150 sec	30 sec	170 sec
2	210 sec	264 sec	290 sec
3	526 sec	508 sec	459 sec
4	790 sec	723 sec	663 sec
5	999 sec	811 sec	898 sec

## STUDY RESULTS

### Total System Delay and Average System Delay

Total system delay and average system delay are two primary measures of effectiveness used in this study to gauge the performance of the different coordination strategies. Total system delay represented the cumulative delay experienced by all vehicles in the system. Average system delay represented the amount of delay experienced by a typical vehicle under the different coordination strategies. This was computed by dividing the total system delay by the total number of vehicles in the simulation. Total system delay versus D/C ratio plots were obtained to compare the five strategies. From these plots, we observed that the rate of increase of delay for D/C ratios between 0.86 to 1.00 is gradual and then increases rapidly after a D/C ratio of 1.06. The following observations were made from these plots.

- For D/C ratio range 0.86–1.00, there appeared to be no difference in the total system delay or average system delay produced by each strategy;
- For D/C ratio range 0.86–1.00, delay remains relatively constant for all the strategies including the standard two-way coordination strategy;
- For D/C ratio range 1.06–1.21, delay increases rapidly for all the strategies; and
- Differences among strategies become more prominent in the D/C ratio range 1.06–1.21.

We also plotted the delay difference curves for the total system delay and the average system delay. The delay difference values of the strategies were obtained by subtracting the delays resulting from operating the signal system under one of the different strategies from the delay occurring when the standard two-way coordination strategy was used to operate the traffic signals. The delay difference values were then plotted as a function of the D/C ratios. From these plots, the following observations were made:

- All of the strategies provide slightly lower delays than the standard, two-way progression scheme produced by PASSER II;
- The delay difference curves appear to show that breaking the progression at the critical intersection produced the lowest delay for the D/C ratio range 0.86–1.00;
- One-way progression in the congested direction produced the lowest delay for D/C ratios greater than 1.00; and
- Breaking the progression in both directions at the critical intersection appeared to produce the highest delay for D/C ratios greater than 1.00.

Based on these observations, one-way progression in the congested direction appears to be superior over a range of D/C ratios greater than 1.00. One explanation for the superior performance of this strategy at D/C ratios greater than 1.06 is that it prevents queue from spilling back from the critical intersection into the upstream intersections.

Breaking the progression at the critical intersection resulted in lower delays for D/C ratios less than 1.00. The reason could be that the critical intersection was isolated, and the controller was able to choose a cycle length and allocate the green splits based on demand.

Two statistical tests were conducted on average system delay values obtained for the simulation: Analysis of Variance (ANOVA) and Duncan's Multiple Range Test. The tests were conducted using the Statistical Analysis System (SAS) software package. ANOVA was used to determine if the average system delays produced by any of the strategies were statistically different from each other. Researchers used the Duncan's Multiple Range Test to determine which of the strategies were statistically different from the rest. All tests were conducted to a 95 percent confidence level.

#### *Analysis of Variance (ANOVA)*

The ANOVA was used to determine if the average system delays produced by the treatments were statistically the same. The null hypothesis for this test was as follows:

$H_0$ : The average system delay produced by each strategy was the same.

The alternative hypothesis for this test was as follows:

$H_a$ : The average system delay produced by at least one of the strategies was statistically different from the rest.

Additional statistical tests would be required to determine which strategy produced different average system delay.

The entire data set was divided into two sets for conducting the ANOVA test because the plot of average system delay showed appreciable amount of interaction. The first set comprised of the D/C ratios from 0.86 to 1.00 and the second class comprised of D/C ratios from 1.06 to 1.21. These two data sets were analyzed by making separate SAS runs. Tables 8 and 9 give the ANOVA table generated by the SAS software package.

**Table 8. ANOVA Table for D/C Ratio Range 0.86–1.00**

Source	Degrees of Freedom	Sum of Squares	Mean Square	Observed F Value	Tabulated F Value	Pr>F
D/C	3	0.4275	0.1425	608.83	2.72	0.0001
Strategy	4	1.3621	0.3405	1454.84	2.43	0.0001
Interaction	12	0.0347	0.0029	12.36	1.81	0.0001

**Table 9. ANOVA Table for D/C Ratio Range 1.06–1.21**

Source	Degrees of Freedom	Sum of Squares	Mean Square	Observed F Value	Tabulated F Value	Pr>F
D/C	3	95.1225	31.7075	5103.87	2.72	0.0001
Strategy	4	6.3597	1.5899	255.93	2.43	0.0001
Interaction	12	2.5683	0.2140	34.45	1.81	0.0001

Tables 8 and 9 indicate there was interaction between D/C ratio and strategies. However, because the average system delays are responding similarly for each strategy (implying an orderly interaction), the effects of the interaction were ignored. The tables show that the effects of both D/C ratio and strategy are statistically significant.

*Duncan's Multiple Range Test*

A Duncan's Multiple Range Test was used to determine which of the individual strategies produced statistically different average system delays. The Duncan Multiple Range Test permits the pairwise comparison of all strategies.

Like the ANOVA test, the data set was again divided into two sets. Two separate SAS analyses were performed. Tables 10 and 11 tabulate the results. In these tables, means with the same letter are not statistically different.

**Table 10. Duncan's Grouping for D/C Ratio Range 0.86–1.00**

Strategies	Sample Size	Mean (Min/Veh)	Duncan Groupings
Two-way	40	1.3510	A
Continuous (uncongested)/ Broken (congested)	40	1.2403	B
One-way, congested	40	1.2178	C
One-way, uncongested	40	1.1585	D
Broken Progression	40	1.1068	E

**Table 11. Duncan's Grouping for D/C Ratio Range 1.06–1.21**

Strategies	Sample Size	Mean (Min/Veh)	Duncan Groupings
Broken Progression	40	2.3343	A
Two-way	40	2.2638	B
Continuous (uncongested)/ Broken (congested)	40	2.2378	B
One-way, uncongested	40	2.0495	C
One-way, congested	40	1.8428	D

From Table 10, the following observations can be made about the performance of the different coordination strategies when traffic conditions were near saturation ( $D/C < 1.0$ ):

- All strategies produced delays that were significantly different from one another; and
- Breaking the progression at the critical intersection produced the least average delay.

From Table 11, the following observations can be made about the performance of the different coordination strategies when traffic conditions were oversaturated ( $D/C > 1.0$ ):

- Breaking the progression in both directions at the critical intersection produced the highest average system delay of all the strategies in oversaturated conditions;
- Providing continuous progression in the uncongested direction while breaking the progression band at the critical intersection for traffic traveling in the congested direction resulted in the same level of average system delay statistically as the standard two-way coordination strategy. Both treatments produced statistically lower system delays than breaking the progression band in both directions at the critical intersection;
- Providing one-way progression in the uncongested direction resulted in statistically lower average system delays than any of the previously mentioned coordination strategies, but not as low as providing one-way progression in the congested direction; and
- Providing one-way progression in the congested direction resulted in the lowest average system delay of any of the coordination strategies in oversaturated conditions (i.e., when the  $D/C$  ratio was greater than 1.0).

### **Throughput**

Total throughput for the main street (total for both directions), throughput in the congested direction, and throughput in the uncongested direction were all plotted against  $D/C$  ratios for the different strategies. Plots of total throughput, throughput in the congested direction, and throughput in the uncongested direction provided the following information:

- For  $D/C$  ratios less than 1.00, throughput in both the congested and uncongested directions and total throughput appear to be the same for all strategies; and
- For  $D/C$  ratios greater than 1.06, one-way progression in the congested direction seemed to produce greater throughput than the other strategies.

### **Average Queue Lengths**

The following observations were made concerning queue lengths in this study:

- The standard, two-way coordination strategy produced the longest queues on the cross-street approaches for almost all volume conditions tested;

- The one-way progression in the congested direction strategy produced the shortest queues in the critical link of the critical intersection (Link 2==>3);
- For D/C ratios greater than 1.06, providing one-way progression in the congested direction achieved dramatic reduction in queue length over the other strategies in the congested approach at the critical intersection; however, the queue lengths on the other links produced by this strategy were comparable to the other strategies;
- For all conditions, providing one-way progression in the congested direction produced the smallest queues on all approaches in that direction, especially at higher volumes;
- The one-way progression in the uncongested direction strategy produced the smallest queues in the uncongested direction for all the volume conditions. This is because this strategy ensures the maximum possible progression band in the uncongested direction;
- The queue lengths at the entry links of the non-critical intersections remained constant, irrespective of the strategies used. This was expected as the strategies had no influence over incoming vehicles at the external links. The queue lengths at these links remained fairly constant up to a D/C ratio of 1.09 and rapidly increased beyond a D/C ratio of 1.09;
- Queues at the entry link to the first intersection in the eastbound direction remained relatively stable until a D/C of 1.14 and rapidly increased at a D/C ratio of 1.21;
- The one-way progression in the congested direction strategy produced the longest queues compared to the other strategies at entry link to the first intersection in the eastbound direction for the D/C ratio of 1.21. Unlike the other strategies, this strategy assured that almost every vehicle able to pass the first intersection will clear the network without stopping because of the large progression band provided in the congested direction. Thus, this strategy kept the interior of the arterial clear of vehicles traveling in the congested direction, while causing queue to form outside the network;
- For a D/C ratio of 1.21, the cross streets had the smallest queues under the one-way progression in the uncongested direction strategy; and
- In reviewing the cross-street queue lengths for a D/C ratio of 1.14, breaking the progression at the critical intersection produced the highest queues. Breaking the progression band at the critical intersection caused queues to spillback from the critical intersection into the second intersection. The abnormally high queue lengths (almost twice that of the other strategies) at the cross streets of the second intersection were primarily due to the spillback as well.

## **Phase Failures**

The average phase failure values were also collected on each link. Using the results of the simulation, the following observations were made concerning phase failures:

- For D/C ratios less than 1.06, phase failures occurred only at the critical intersection for all the strategies. This is to be expected because the critical intersection represents the bottleneck for the entire corridor;
- For D/C ratios less than 1.06, phase failures mainly occurred on the cross-street approaches of the critical intersection;
- For D/C ratios equal to and below 1.14, phase failures never occurred in the main-street approaches of the non-critical intersections;
- Phase failures never occurred in the uncongested direction for any of the volume conditions. This was probably because traffic volume in the uncongested direction was fixed for that direction;
- At a D/C ratio of 1.21, all the strategies produced similar percentages of phase failures at all the intersections. The phase failures were almost evenly distributed among all the intersections at this volume level;
- Breaking progression at the critical intersection prevented phase failures from occurring at all the intersections below a D/C ratio of 1.00; and
- The one-way progression in the congested direction strategy produced the least number of phase failures at the congested approach of the critical intersection in the congested direction. It also produced fewer phase failures compared to the other strategies for the fourth and fifth intersection cross-street approaches.

## **Experimental MOE**

As discussed above, two experimental MOEs were also used to assess the performance of the different progression strategies:

- The number of vehicles required to stop during a single trip on an arterial; and
- The number of stops drivers made at each intersection in the system.

These measures were collected by monitoring the same five vehicles under each strategy. Data were collected at three D/C ratios. Tables were constructed to summarize the number of vehicles that were able to pass through the system without making any stops, one stop, two stops, etc. at each of

the three D/C ratios. Researchers observed that providing one-way progression in the congested direction allowed the greatest number of drivers traveling in that direction to pass through the arterial without making any or one stop. The tables show that one-way progression in the congested direction resulted in the fewest number of multiple stops as compared to the other strategies. Again, this implies that this strategy provided better opportunities for vehicles to progress through the system without stopping.

Providing one-way progression in the uncongested duration produced the greatest number of stops in the congested direction for all the strategies. The probable cause for this observation is that this strategy was designed to maximize progression in the uncongested direction. The net effect was that the cars in the congested directions had to stop more frequently.

## **RECOMMENDATIONS**

Based on the results of this simulation study, researchers recommend that agencies set up their signal systems to provide one-way progression in the congested direction for arterial approaching or experiencing oversaturation. The use of this strategy is recommended for the following reasons:

- When traffic conditions are approaching capacity (e.g., 0.86 to 1.0), setting up the traffic signals to provide one-way progression in the congested direction results in approximately the same amount of system delay and average system delay as the standard PASSER II signal timings; and
- When traffic demands exceed capacity, (e.g., 1.0 to 1.21) setting up the signal system to provide one-way progressive flow in the congested direction, can produce lower system delays, fewer stops, and shorter queues than the standard PASSER II signal timings.



## **CHAPTER IV**

### **DATA NEEDS AND ARCHITECTURE FOR REAL-TIME CONTROL**

#### **INTRODUCTION**

Most operational systems-level approaches make signal control decisions at a central location and pass these decisions to local controllers. These systems operate, in a time-of-day basis or traffic-responsive mode (1-GC), by implementing signal timings developed off-line using historic demand data. Systems-level approaches perform well during medium to high traffic demand periods and when traffic patterns can be reasonably established using historic data. Signal control at the local (or isolated) level is traditionally accomplished through the use of (semi or fully) actuated controllers. Actuated traffic signals perform very well under low traffic conditions, however, as traffic demand increases, their performance degrades, and in many cases, it is not as effective as coordinated pretimed control. This is due to the fact that isolated-actuated control only takes into consideration local conditions and ignores arrival times of platoons from upstream signals.

Effective coordination of actuated signals can be achieved by explicitly taking into account dependencies among adjacent traffic signals. We believe this control philosophy has the potential to provide better performance than the pretimed coordination strategies because temporal variations in demand occur even under heavy traffic conditions, and actuated controllers are better equipped to handle these variations.

#### **PROPOSED REAL-TIME SYSTEM**

Although original implementations of real-time systems in this country were not fully successful, the traffic engineering community still believes that real-time systems can be a cost-effective means of relieving traffic congestion by providing efficient flow of traffic. The key strengths of a real-time system are its ability to automatically:

- Collect, aggregate, and analyze traffic data to assess traffic conditions;
- Make appropriate signal control decisions; and
- Ability to adapt to changes.

In order to function properly, a real-time system must have the following additional features:

- Ensure stable traffic flow,
- Able to handle the full range of traffic conditions,
- Be responsive and adaptive to changing traffic patterns, and
- Be able to calibrate itself.

## **System Architecture**

Here we propose the architecture of a real-time system which combines centralized- and traffic-actuated control. We also discuss various components needed to implement such a system. The proposed system can maintain stability by making certain control decisions at the systems level. It can also provide responsiveness to random fluctuations in traffic patterns by providing constrained flexibility to actuated controllers. This approach essentially divides the real-time decision-making process into two major levels; strategic and tactical.

### *Strategic Level*

At the strategic or central level, the system makes traffic control decisions for the entire system of signals. This analysis uses real-time as well as historic traffic data. Here, we consider real-time data as the data collected by the system immediately prior to the data analysis process. The strategic-level data analysis includes the following steps:

- demand estimation,
- demand projection,
- signal timing optimization, and
- signal timing transition cost analysis.

The decision-making process performs the above procedures at regular intervals and decides whether it is worthwhile to change the existing signal timing plan. If so, it downloads new signal timing parameters to each controller. Depending on the size of the signal system, several minutes may be required to perform all of the analysis steps given above. In addition, depending on the differences in the existing and new timing plan, there will be a period of unstable flow (the transition time). Any strategic level change must allow sufficient time for stable flow before implementing a new plan. Thus, we propose the time between successive signal timing updates at strategic level be no less than 15 minutes. This ensures stability of traffic flow as well as stability of data. Signal control decisions made at this level set constraints on the operation of each signal controller's actuated mode of operation.

### *Tactical Level*

In the proposed system, actuated controllers make tactical level decisions to account for temporal variability in actual demand. These decisions, however, are constrained to stay within the boundaries specified by the strategic decisions. The depth of analysis performed at this level depends on the capabilities/intelligence built into the controllers. For instance, a smart controller with adequate detection hardware could provide the following functions:

- better demand estimation,
- capacity and performance analysis, and
- detection of incident and blockage.

Depending on the capabilities of the controllers used, these functions could be used to provide cycle-by-cycle signal control decisions at the local level. For instance, quick response time required to address an incident or a blocking/spillback (primary congestion) situation requires continual monitoring of traffic flow and immediate changes in signal timings. This level of control can only be achieved by building smarts into the controllers. Furthermore, a smart controller could also be programmed to temporarily drop out of coordination (disregard strategic constraints) to resolve such situations because the objective of coordinating traffic signals to provide smooth flow becomes secondary to the objective of utilizing full signal capacity.

### *Variations in Levels of Control*

The optimum balance between the decisions made at each level and the interaction or dependencies between them is a function of traffic conditions, geometric characteristics of the signalized network, and the capabilities of the hardware. The proposed architecture provides for a wide range of traffic control between, and inclusive of, the following two extreme boundaries:

1. All decisions made at the central location. This extreme is similar to traffic-responsive control, but with an additional capability to calculate signal timings using real-time data.
2. All decisions made at the local level. This extreme provides for the operation of signal controllers in a fully actuated mode using signal parameter settings stored in the controller's memory.

Although advanced (i.e., 2070) controllers are currently under development and planned for near-term installation in some cities, their cost will prohibit mass scale installation for a few more years. Thus, the research in this project focused on existing controller technology with minor enhancements. In the next section, we discuss real-time data needs and how these needs could be met by the existing hardware technology.

### *Manual Control*

In addition to the automated data exchange and control functions, the system must also provide an ability that allows an operator to monitor the system and to implement manual control. An integral part of this functionality is an operator interface. We will discuss such an interface in a later section.

## **REAL-TIME DATA NEEDS**

The determination of optimal traffic control depends heavily on the ability to assess traffic conditions. No matter how sophisticated the analysis capabilities a real-time system has, effective traffic control cannot be achieved unless it can obtain accurate data from the field. Furthermore, the system should take into account the effects of traffic conditions on signal control decisions and vice versa. Thus, we set the following two fundamental requirements for the proposed real-time system:

1. Ability to accurately assess traffic demand and
2. Ability to assess traffic performance measures.

Video-based surveillance systems currently under development have the potential to provide a wide range of information. But at this time, these systems have not reached a mature state. Even when such time arrives, cost and other factors will prohibit their wide use. Therefore, in this project, we focused on the use of loop detectors and signal controllers for the collection of traffic data. In the following sections, we discuss the types of data that can be obtained from the field and the usefulness of these data in the real-time system. In addition, we discuss how each type of data can be used to satisfy the above requirements.

### Data from Loop Detectors

Loop detectors can provide two types of data; vehicle counts and loop occupancy. Vehicle count is the number of vehicles that pass over a loop during a specified period of time. Loop occupancy, measured continuously over a specified time, is the percent of that time the loop was occupied. These two variables can be used to estimate traffic demand.

Figure 11 shows how volume and occupancy increase with the increase in traffic demand on an uninterrupted traffic lane (i.e., a freeway lane). From this figure, we can observe the following changes in volume and occupancy with the increase in demand:

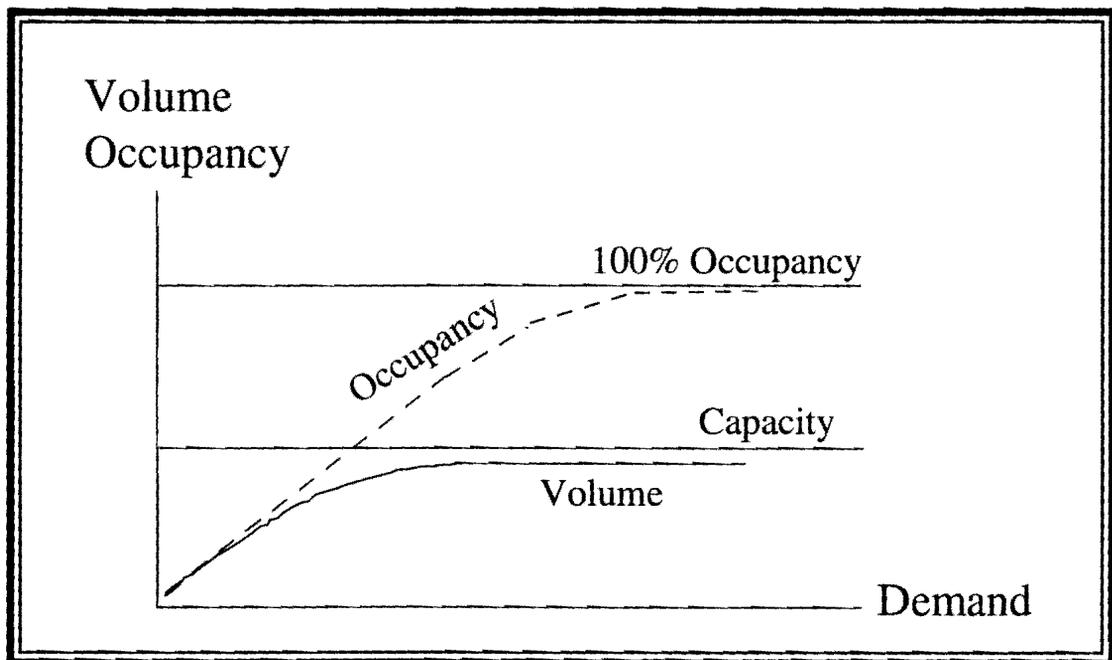


Figure 11. Volume/Occupancy Versus Demand Curve for an Uninterrupted Lane

1. Occupancy increases and tapers off at its maximum value of 100 percent; and
2. Initially, service volume is equal to demand and increases linearly. As demand approaches operational capacity of the lane, volume starts to taper off and then becomes constant.

Thus, service volume is equal to demand as long as demand is below capacity. When demand nears or exceeds capacity, occupancy can be used to estimate demand as follows:

$$Demand = Volume + (K * Occupancy)$$

In the above equation, K is a constant multiplication factor. In addition, this equation applies when occupancy increases beyond a certain threshold value. Earlier studies show that an occupancy value over 25 percent is a reliable indicator of the onset of congestion. One situation not illustrated in the figure arises when occupancy is high and volume drops significantly below the operational capacity. For instance, this situation can occur when a downstream incident affects the flow of traffic. Thus, occupancy and volumes from loop detectors can also be used to identify other operational problems.

The demand to volume/occupancy relationships discussed above become more complicated for detectors in signalized lanes. The first complication is caused by the fact that the operational capacity of an approach lane is dependent on the effective green time, which is significantly lower than the saturation flow rate (maximum capacity) of the lane. Thus, green-time information from the signal may be additionally needed to determine demand. The second complication is caused by the location of the detector. On a signal approach, loop detectors can be placed at a number of locations to provide information about traffic conditions. Figure 12 illustrates two typical placements of a loop detector on a single-lane approach to a traffic signal. On the northbound approach, the detector is located at the stop-bar. We refer to this as a stop-bar detector. On the southbound approach, the detector is placed a certain distance upstream of the stop-bar. We refer to this as an approach or system detector. In the following sub-sections, we describe the usefulness of data from these types of detectors and what improvements can be made.

### *Stop-Bar Detectors*

By definition, vehicle count from a stop-bar detector is the number of vehicles in the approach lane serviced (serviced volume) by the signal during a specified time (i.e., in one cycle). During undersaturated traffic conditions all of the vehicles queued during the red period and those arriving during the successive green period receive service during that green period. Thus, vehicle count from a stop-bar detector is an estimate of traffic demand. In addition, during these conditions, higher occupancy from a stop-bar detector indicates higher demand but does not provide any additional information than that provided by traffic counts. During saturated, or oversaturated conditions, the green period is not long enough to service all (queued or arriving) vehicles, and thus, the vehicle count (serviced volume) from a stop-bar detector underestimates demand. Thus, additional data is needed to estimate demand during saturated or oversaturated traffic conditions.

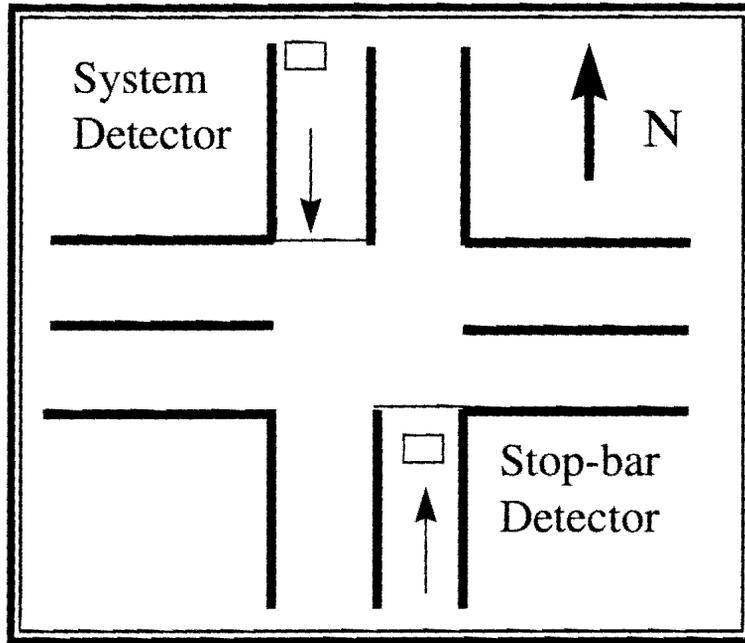


Figure 12. Example of Typical Detector Locations

For a stop-bar detector, a continuous measure of occupancy, with a properly validated occupancy threshold value, can be used to detect the onset of congestion. However, as in the case of uninterrupted facilities, this measure cannot be used to accurately estimate demand. Figure 13 illustrates a volume/occupancy versus demand curve for a stop-bar detector. As shown in this figure, continuous presence of a queue, no matter how small it is, will result in a steep rise in the occupancy value. Furthermore, there is no way to tell exactly how many cars are in the queue.

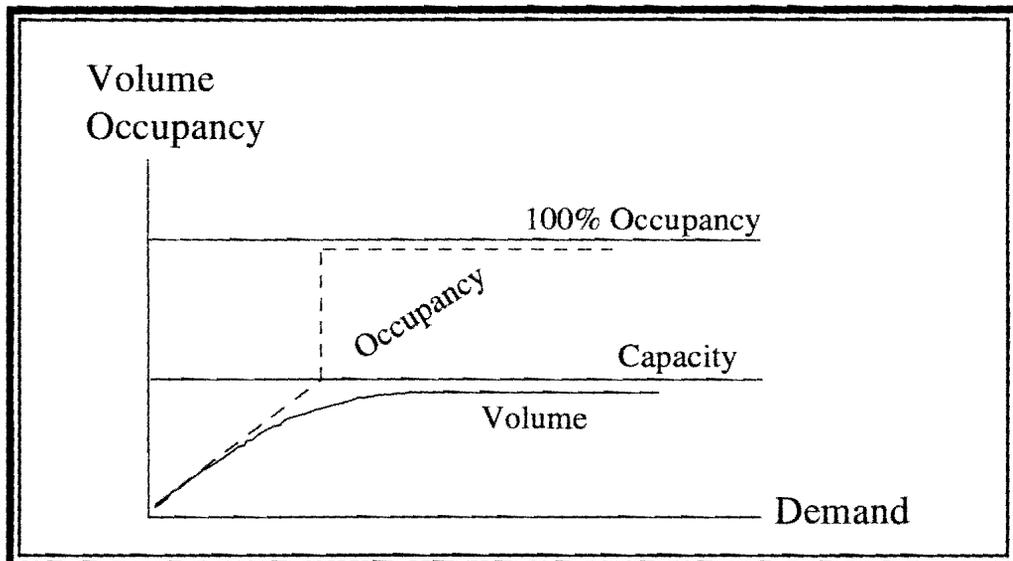


Figure 13. Volume/Occupancy Versus Demand Curve for a Signalized Approach Lane with a Stop-Bar Detector

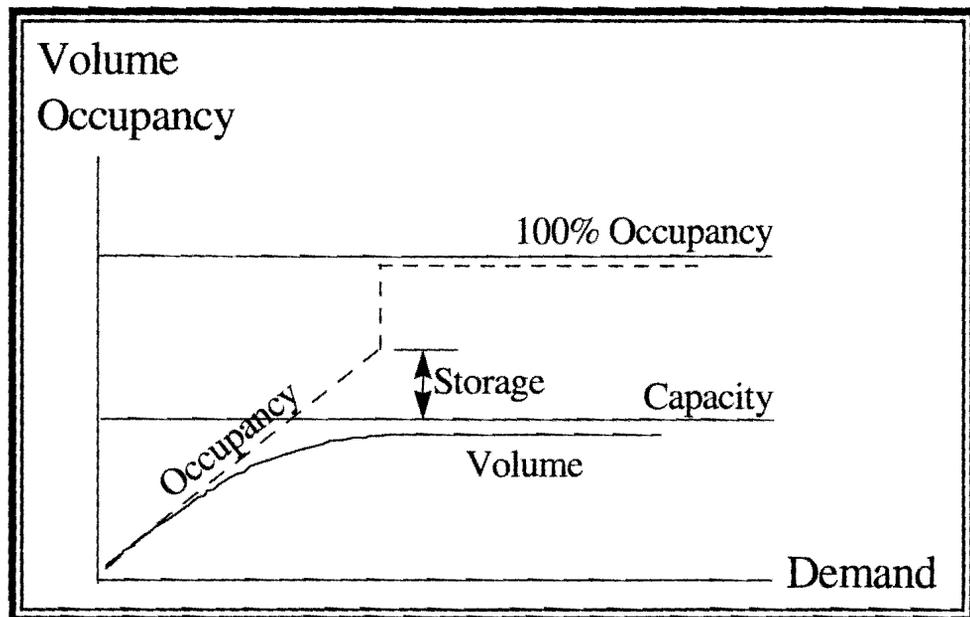
Thus, in order to be a useful tool in demand estimation during congested traffic conditions, occupancy for a stop-bar detector should be measured during those parts of a signal cycle during which vehicles are moving and queue size is the smallest. We recommend investigation of three types of occupancy measures for this purpose. These are:

1. Occupancy during green signal indication,
2. Occupancy during green plus clearance, and
3. Occupancy during clearance.

Commonly used controllers in Texas currently do not provide these measures. Thus, there is a need to enhance signal controller software to provide these measures of occupancy. Also, further research is needed to determine proper values of the occupancy threshold and the multiplication factor (K) for use in estimating demand using each of these occupancy measures. We believe that these values will be different for different detector locations.

#### *System Detectors*

System detectors are placed a certain distance upstream of the stop-bar and entrances to turn bays. As shown in Figure 14, these detectors provide a better estimate of demand because their placement accounts for vehicles serviced by the signal, plus those vehicles queued between the detector and the stop-bar. The maximum upper limit to which demand can be accurately measured depends on the signal capacity and vehicle storage capacity between the stop-bar and the downstream edge of the detector. However, these detectors cannot accurately estimate service volume.



**Figure 14. Volume/Occupancy Versus Demand Curve for a Signalized Approach Lane with a System Detector**

## **Data from Controllers**

In the proposed system, the strategic-level decision-making process uses average traffic conditions to determine green splits for each signal in the system. Then it downloads these splits to each controller. The controllers are permitted to make constrained variations in the splits to account for variability in actual demand during the control period. But, the strategic level does not know the actual splits implemented by the controllers. Without this information, the strategic level cannot assess accurate demand or system performance. As a result, it cannot respond to problems created by spillback and blocking. Inexpensive modifications can allow the controllers to provide this information to the control center and for use by each controller to make more intelligent decisions. In a later section, we will describe how this additional information can be used to provide improved real-time control.

## **Summary of Data Needs**

As discussed above, stop-bar and system (approach) detectors have different capabilities to provide data. We strongly recommend the use of both types of detectors in each continuous lane and stop-bar detectors in turn bays. In addition, we recommend system detectors at key mid-block locations. The following information can be obtained from these detectors and signal controllers:

- Vehicle counts from stop-bar and system detectors,
- Occupancies from stop-bar and system detectors, and
- Green splits and other signal timing information from the controllers.

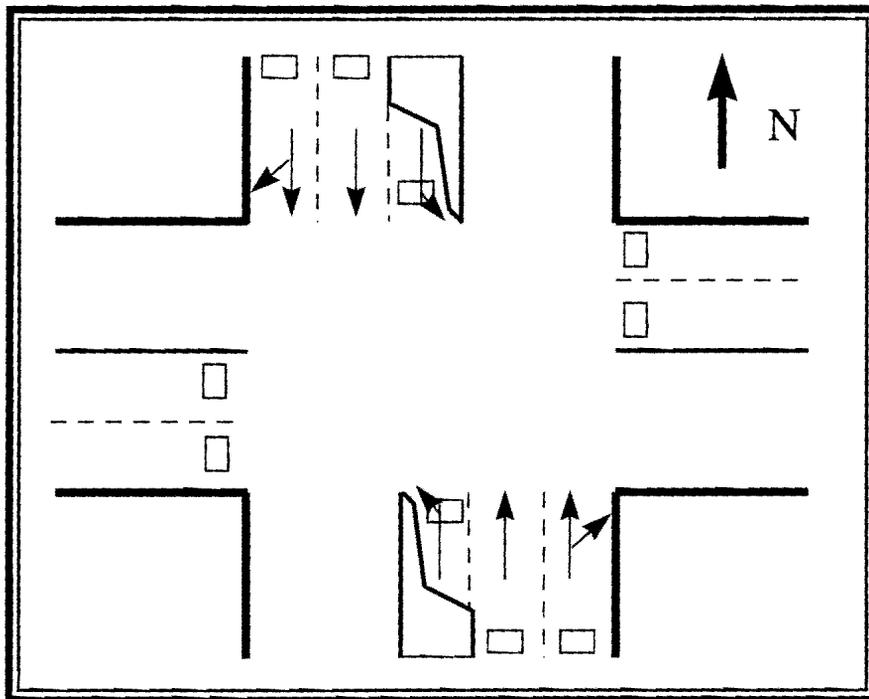
This information can be used to estimate demand and to assess system performance. In many situations, it will not be feasible to install detectors at all of the recommended locations. In such cases, a subset of detectors could be used. In the next section, we illustrate this using our test bed in Richardson, Texas.

## **RICHARDSON TEST BED**

In the previous sections, we provided the overall architecture of a real-time traffic control system and discussed the data needed to implement such a system. In the early stages of this research project, TTI, the city of Richardson, and Naztec, Inc. had joined in a cooperative partnership to develop, test, and validate various components of the proposed system. Based on the recommendations of the research team, Naztec and Richardson have already completed software enhancements to provide real-time detector and signal timing data. In addition, the enhanced controllers have also been installed in the field. These data are also available to TTI over the Internet. Furthermore, the researchers have developed a prototype operator interface to retrieve, analyze, and display the real-time data. In the following subsections, we describe different components of a system.

## Description of Detectors

Figure 15 shows typical detector layout and lane assignments in Richardson. As illustrated in the figure, only a subset of recommended detectors are available. This subset is described below:



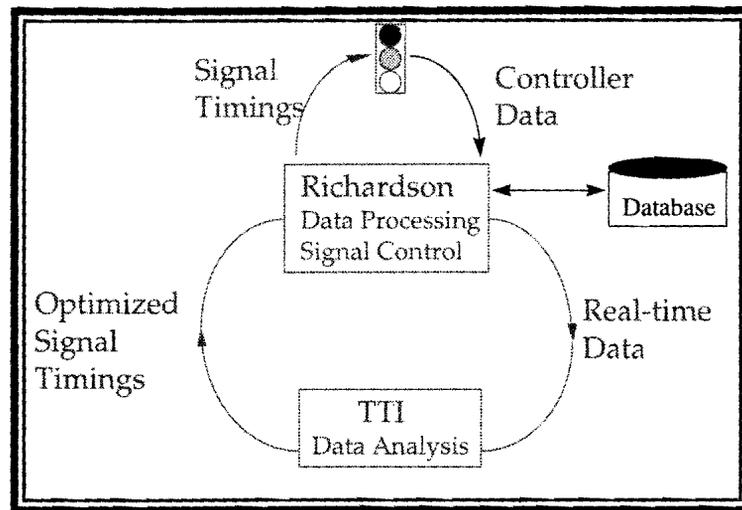
**Figure 15. Location of Detectors in Richardson**

- All lanes on minor approaches and all left-turn bays have stop-bar detectors.
- All through lanes at major approaches have approach detectors. These detectors are located approximately 100 meters upstream of the stop-bar. This placement of the approach detectors upstream of the entrance to the turn pockets provides for their use as system detectors.

With a few exceptions, all right-turn traffic shares the lane with through traffic. Therefore, right-turn volume and demand estimation must rely on predetermined time-of-day turning percentage estimates.

## Data Flow and Data Descriptions

Figure 16 illustrates the data flow process between the field hardware, Richardson control center, and TTI. Each controller gathers and uploads to the central computer, the following data in 15 minute blocks of time:



**Figure 16. Data Flow between Richardson and TTI**

- Vehicle counts for each lane,
- Occupancies for detectors in each lane,
- Status data for each detector,
- Average green splits for all phases,
- Average cycle length, and
- Phasing sequences.

Depending on user selection, either one of the following four occupancy measures can be obtained from one controller:

1. Average occupancy during the entire 15 minute period;
2. Occupancy during green, averaged over 15 minutes;
3. Occupancy during green plus clearance (yellow and red), averaged over 15 minutes; or
4. Occupancy during the last two seconds of green plus clearance, averaged over 15 minutes.

The only restriction is that one selection applies to all detectors at an intersection. Thus, the selection must work for both stop-bar and approach detectors. This rules out the possibility of using the first option, as discussed earlier. The communication between the central computer and the field hardware takes place over the cable television network. Then the central computer conducts the following operations:

- Checks data for errors,
- Aggregates data and calculates turning movement counts,
- Stores the counts into a database, and
- Copies raw data (counts, occupancies, and signal timings) to a file for use by TTI.

A real-time process running at TTI uploads the data file over the Internet. On the average, the central computer takes about five minutes to obtain and process the data. Although the processing time is fairly constant, data communication delays cause a significant (two to three minute) variation in time of file availability. These unpredictable delays make it impossible to predict the exact time the next data file will be ready for download by TTI. We resolved this problem by putting a time stamp in the file name. For instance, a file named *RT1015.TXT* contains data from 10:00 A.M. to 10:15 A.M.

At TTI facilities in College Station, Texas, a computer routine constantly checks for the availability of the file with a specific time stamp. When the file becomes available, a real-time process uploads it using the Internet file transfer protocol (FTP). In the next section, we describe the real-time analysis process running at TTI.

#### DATA ANALYSIS PROCESS AT TTI

At TTI, a real-time process performs further analysis of the data as soon as the next 15 minute data file becomes available. Figure 17 provides a flow diagram of the process. This process can reside at the central location in Richardson. In the following sub-sections, we discuss various components in the analysis process addressed in this research project.

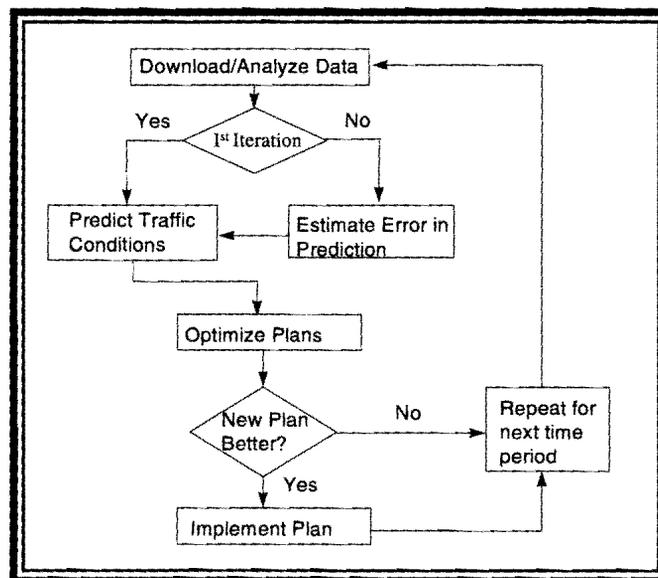


Figure 17. Flow Chart of Data Analysis

## **Turning Movement Count and Demand Estimation**

The first step is the assessment of turning movement volumes from the raw data from stop-bar and approach detectors. As described earlier, Richardson only has a subset of recommended detectors. In addition, most right-turn lanes are shared. This creates the need to use some historic data to estimate the right-turn volumes counts. This count estimation process for each lane is given below.

### *Minor Approaches*

Recall that all minor approaches have stop-bar detectors. Turning volumes for these approaches are obtained as follows:

- For exclusive lanes, volume is equal to detector count.
- For shared lanes, a turn-percentage value is applied to determine the left- and/or right-turn volume. We use time-of-day (morning, off-peak, and evening peak) percentage values obtained by Richardson staff through field studies.

### *Major Approaches*

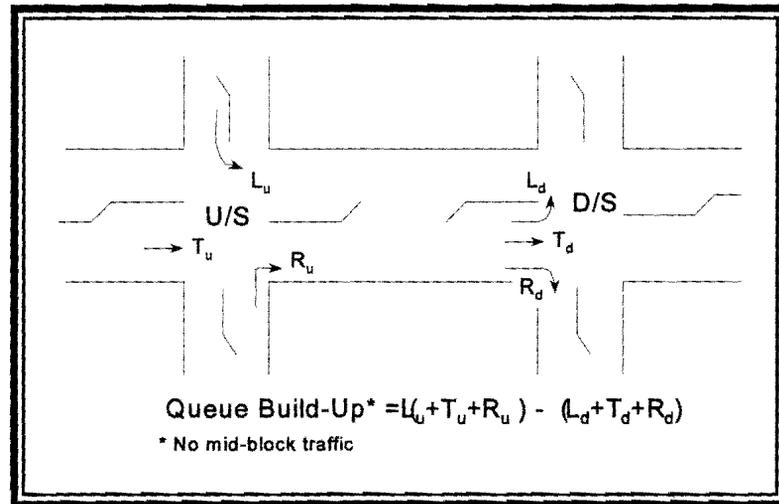
As described earlier, all major approaches in Richardson have left-turn bays. A majority of these have single lanes and some have two lanes. In most cases, right-turn traffic shares a lane with through traffic. Thus, we calculate turning volumes as follows:

- Left-turn volume is equal to the count from the stop-bar detector;
- Through volume for an exclusive lane adjacent to the turn bay is equal to the approach detector count, minus the stop-bar detector count for the turn bay; and
- For a shared through plus right lane, apply the right-turn percentage for that time of day to the approach detector count to obtain right-turn volumes. In case the shared lane is adjacent to a left-turn bay, we used the count as adjusted in the above step. Then, we subtract the right-turn volume from the total count for that lane to obtain the through volume.

### *Demand Estimation during Congested Conditions*

The count from the approach detectors is a fairly good estimate of demand as long as the back of the queue stays on the downstream side of the detector. This is true during a major part of the day. During congested conditions, occupancies will be needed for accurate demand estimation. However, there is a need to perform field studies to determine occupancy threshold and multiplication factor values for the new occupancy measures described earlier. These field studies will allow the selection of the best occupancy measure from the available set.

Another way of estimating traffic demand is by link input-output analysis. This concept, illustrated in Figure 18, makes use of stop-bar detectors at upstream signals as system detectors. The additional information from these detectors can be extremely useful, especially for those links that have insignificant or no mid-block source or sink.



**Figure 18. Input-Output Analysis on a Link**

The following is a list of potential uses of counts from upstream detectors:

1. Can be used in lieu of mid-block demand detectors,
2. Can be used as backup when downstream detectors malfunction, and
3. Can be used to assess when queue is growing or dissipating.

Using data from these detectors, turning movement demand at the downstream end of a link can be estimated as follows:

1. Obtain the sum of counts from lanes feeding left-turn, through, and right-turn traffic to the subject link. This provides the total traffic entering the link.
2. Determine the turning percentages using counts obtained from detectors located at the downstream end of the link.
3. Multiply the total entering traffic from step 1 with each turning percentage to obtain demand for each movement.

Depending on the travel time on the link, these demand estimates will also provide some projection into the future.

## Prediction

The above analysis is performed using data from the immediate past. In order to provide adaptive capabilities, the estimated demand needs to be projected to estimate traffic conditions during the next control period. Thus, the next step in the process is predicting traffic conditions. One method of projecting or predicting demand is described below.

Let,

$t$  be the present time period,

$t+1$  be the next time period,

$V(t)$  be the demand just estimated,

$V'(t+1)$  be the historic demand for the next control period,

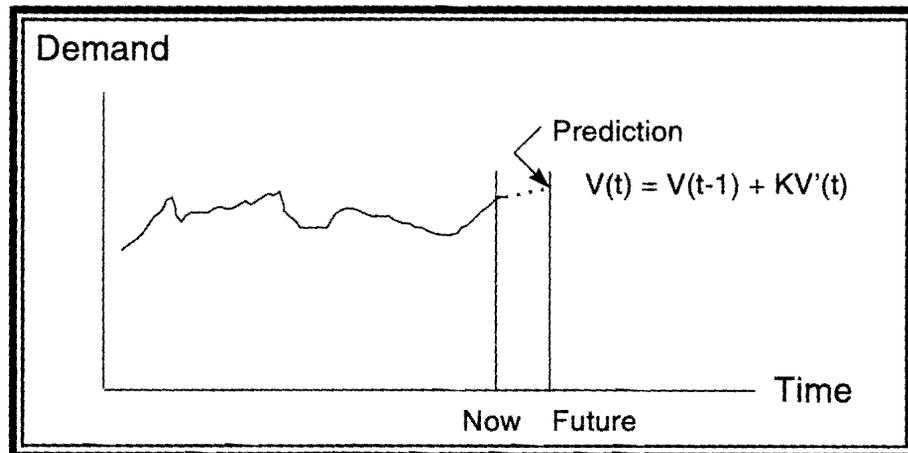
$V(t+1)$  be the projected demand for the next control period, and

$K$  be a number between 0 and 1, inclusive.

Then:

$$V(t+1) = (1-K)V(t) + K*V'(t+1)$$

In the above equation, the value of  $K$  determines how much weight is to be given to historic data. Figure 19 illustrates the demand prediction process.



**Figure 19. Concept of Demand Prediction**

The above equation can also be expanded to include real-time demand from a specified number of previous time slices. The demand prediction also could be enhanced to provide a real-time calibration capability by using the differences or squared-differences in actual and predicted demand.

## Assessment of System Performance

In this section, we describe how turning volumes and signal timing data from the controllers can be used to assess the performance of the system.

### *Capacity Analysis*

Use the demand and signal timing information to determine demand-to-capacity (D/C) ratio for each movement at a signal and the overall D/C ratio for the signal. This information can be used as follows to improve signal operation:

1. If the overall D/C ratio for the signal is less than one and that for one or more movements is equal to or greater than one, then splits can be adjusted to bring the D/C ratio of all movements below one; and
2. If the overall D/C ratio for the signal is more than one, adjustments to green splits will be needed at upstream signals as well. This will be done in two steps:
  - A: Adjust splits locally to ensure equal saturation for all movements; and
  - B: Adjust splits at the upstream signals. Depending on the severity of congestion at a signal, this may require minor changes in the system cycle length or major changes such as metering upstream signal approaches feeding traffic to the subject intersection.

### *Detecting Spillback and/or Blocking*

Simple logic could be used to detect spillback/starvation due to interference in the vicinity of entrance to turn bays, or queue spillback and/or blocking at a downstream link. The accuracy of these detections depends on the amount of information. We explain these concepts using the following example:

Example: A signal approach with two through lanes and one left-turn bay.  
The left-turn bay has a stop-bar detector.  
The through lanes have approach detectors as in Richardson.  
Volumes, green split information, and occupancies are available.

Case 1: Spillback from the left-turn bay could be detected by the following conditions:

1. V/C ratio for the through lane adjacent to the left-turn bay is low, and the occupancy points to congestion in this lane; and
2. V/C ratio for the other through lane is higher and the occupancy is low.

These conditions could also be caused by an incident in the through lane.

- Case 2: Spillback/blocking at the downstream link could be detected by the following conditions:
1. V/C ratio for the through lane adjacent to the left-turn bay is low, and the occupancy points to congestion in this lane; and
  2. V/C ratio for the other through lane is low and the occupancy points to congestion in this lane as well.

The detection of conditions described in the above example indicates reduced capacity in certain lanes. These conditions do not provide information about the exact cause of the problem. In order to detect the exact cause, more information is needed. For instance, if the system has the capability to promptly report these conditions, the operator could use video cameras to determine the exact cause and take measures to resolve the situation. In Richardson, this is possible because the city has several cameras located at strategic locations. The only component needed is an operator interface that provides alarms when such situations are detected. In the next section, we describe a prototype operator interface that was developed in this research project and could be used to provide audio and visual signals to assist the operators.

## **GRAPHICAL TOOL FOR DATA ANALYSIS AND VALIDATION**

In the previous sections, we describe the various components of real-time data analysis. The main focus of these discussions is on the assessment of demand and traffic conditions. There is a need to verify and validate these procedures using real-time data. In order to efficiently accomplish these tasks, we felt a need for a capability to graphically display real-time data. A prototype graphical user interface to real-time data was developed and installed in Richardson. In this section, we provide a brief description of this software. The graphical interface integrates with other components described earlier. Figure 20 illustrates a screen from this interface.

This graphical interface uses a database describing the signal system and provides the following functions:

- Allows the user to select a signal from the signal list (top-left corner in Figure 20) and one approach (i.e., northbound) for the intersection. At any time the user can switch between various approaches or signals.
- Provides real-time color-coded plots of traffic volumes for user selected movements (middle-top part of the screen in Figure 20). The user can also choose to display either critical or total volumes.
- In the lower-middle part of the screen, it displays lane assignments for the selected approach. It also identifies the types of detectors and detector data for each lane.
- In the right-bottom part of the screen, it provides a bar chart showing link analysis. In this analysis, the selected approach identifies the downstream end of the link and the upstream

end of the link uses data from the stop-bar detectors at the upstream signal that feeds traffic to the selected approach.

As mentioned earlier, this prototype software has been installed in Richardson and is currently being used for data verification and validation purposes.

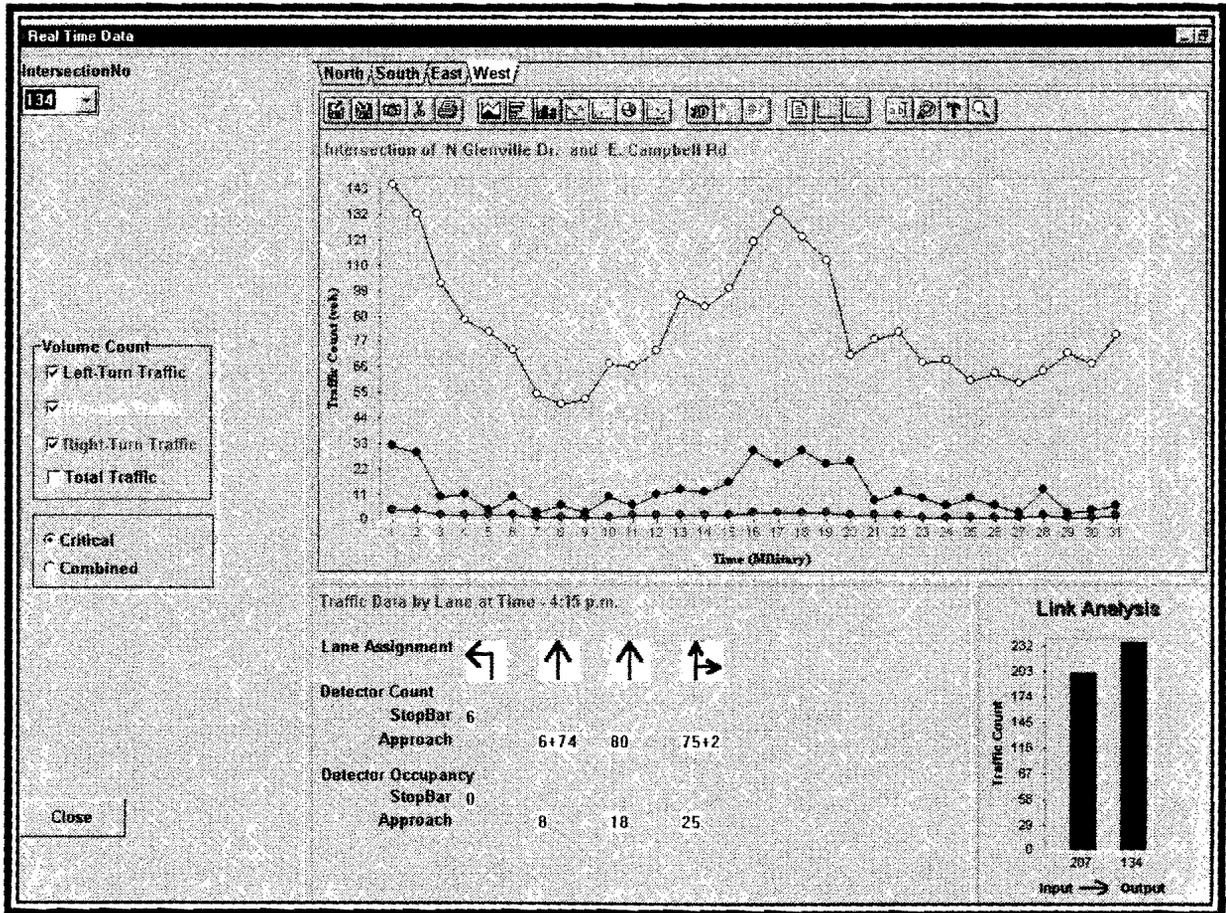


Figure 20. Real-Time Operator Interface



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