ASSESSING DETERIORATION OF PRETIMED, ACTUATED-COORDINATED, AND SCOOT CONTROL REGIMES IN SIMULATION ENVIRONMENT

by

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I have read the thesis of Aleksandar Stevanovic in its final form and have found that (1) its format, citations, and bibliographic style are consistent and acceptable; (2) its illustrative materials including figures, tables, and charts are in place; and (3) the final manuscript is satisfactory to the supervisory committee and is ready for submission to The Graduate School.

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ABSTRACT

Regular updating of traffic signal timing plans is very important for successful traffic control. However, many jurisdictions fail to update signal timings because it is labor intensive and costly. Without updating, signal timings become obsolete as traffic volumes change. Adaptive traffic control is sometimes perceived as a way to avoid retiming traffic signals. There are no findings that support this belief.

This research investigates the deterioration of pretimed, actuated-coordinated, and SCOOT traffic control regimes through the use of microsimulation. Deterioration of actuated-coordinated and SCOOT adaptive controls have not been investigated before. Previous attempts to investigate the deterioration of pretimed control did not use microsimulation. Two major objectives of the study were to develop a methodology to assess the deterioration of the traffic control regimes through microsimulation and to evaluate the deterioration of the traffic control regimes with respect to modeled changes in traffic demand and distribution.

The experimental nine-node grid network is used as a test bed to model deterministic and stochastic traffic demand and distribution changes in link flows. Traffic signal plans developed for the base traffic conditions serve as the non-optimized plans for all other conditions. Optimized plans are developed for each scenario of changed traffic flows using macroscopic optimization tools.

The results show that all traffic control regimes deteriorate. The results for pretimed and actuated controls show that there is a benefit of up to 3% for up to 5%

of uniform growth in traffic demand for networks with unchanged traffic distributions. When stochastic variations of traffic demand and distribution are introduced, the benefits rise to an average of 35% for pretimed control and 27% for actuated control. Assessment of SCOOT ageing shows that SCOOT performance highly fluctuates with changes in traffic flows. The SCOOT control performs worse than optimized pretimed control for most of the scenarios. Had the SCOOT control been replaced by optimized pretimed plans, benefits of 11 to 16% could be achieved. The roots of SCOOT ageing have been found in its inability to accurately model traffic at the intersection approaches for changed traffic flows.

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ACRONYMS

ATCS	Adaptive Traffic Control System
CF	Changed Flows
CFP	Cyclic Flow Profile
CL	Cycle Length
CORSIM	CORridor SIMulation
DDE	Dynamic Data Exchange
DLL	Dynamic Link Library
ELAG	End LAG
FHWA	Federal HighWay Administration
GA	Genetic Algorithms
HCM	Highway Capacity Manual
HCS	Highway Capacity Software
ITE	Institute of Transportation Engineers
JNYT	JourNeY Time
LOS	Level Of Service
MOE	Measures Of Effectiveness
MC3	Managing Congestion, Communications, and Control
NEMA	National Electrical Manufacturer Association
NO	NonOptimized
NPNT	New Plan New Traffic

OP	OPtimized
OPAC	Optimized Policies for Adaptive Control
OPNT	Old Plan New Traffic
PASSER	Progression Analysis and Signal System Evaluation Routine
PI	Performance Index
QCMQ	Clear time for Maximum Queue
PRIMAVERA	PRIority MAnagement for Vehicle efficiency, Environment Road safety on Arterials
RHODES	Real-Time Hierarchical Optimized Distributed and Effective System
SCATS	Sydney Coordinated Adaptive Traffic System
SCJ	Signal Control Junction
SCOOT	Split Cycle Offset Optimization Technique
SIDRA	Signalized Intersection Design and Research Aid
SLAG	Start LAG
SOAP	Signal Operations Analysis Package
SSD	Sum of Squared Differences
STOC	SaTuration Occupancy
TCP/IP	Transmission Control Protocol / Internet Protocol
TM	Turning Movement
TOD	Time Of Day
TRANSYT	TRAffic Network StudY Tool
TRL	Traffic Research Laboratory

UDOT	Utah Department Of Transportation
UTDF	Universal Traffic Data Format
UTL	Utah Traffic Lab
UTCS	Urban Traffic Control System
VAP	Vehicle Actuated Programming
V/C	Volume/Capacity
VISSIM	Verkehr In Stadten SIMulation (Traffic in Towns Simulation)

CHAPTER 1

INTRODUCTION

This chapter addresses the essential background of traffic control systems and the importance of their maintenance. Signal timing parameters are discussed as one of the key factors for updating traffic control systems. Then, after a basic literature review of the most important studies that have been done on the topic, the research problem is defined. The research goal and objectives are stated in the next part of the chapter. The final part of the chapter provides an overview to guide readers through the remainder of the dissertation.

1.1 Traffic Congestion and Traffic Control

The growth in urban traffic congestion has been recognized as a serious problem in all large metropolitan areas in the country, with significant effects on the economy, travel behavior and land use, as well as a cause of discomfort for millions of motorists.

Solutions to the congestion problems are neither simple nor unique. The traditional approach of simply adding more capacity is often not possible or desirable. Therefore, improvements are often sought that increase the efficiency of the existing systems. One of the most important tools to alleviate urban congestion is traffic control. Maintenance of traffic control systems is as important as their optimizing methods and initial design.

The first traffic signals were installed in the beginning of the 20th century. Their main objective was to prevent accidents by alternately assigning the right of way. Not much attention was given to other objectives that are relevant today, such as minimizing traffic delay and fuel consumption. Over time, however, as traffic volumes have increased, the objective has broadened to include maximizing the capacity of the roadway system and improving traffic flow.

The Federal Highway Administration (FHWA) (1995) has reported that there are more than 300,000 traffic signals in North America. The same study estimated that two-thirds of all miles driven each year occur on roadways controlled by traffic signals. Despite their important role in traffic management, after traffic signals are installed, the timing settings are often not given enough attention. Further, more than half of the signals in North America are in need of repair (or replacement) of the traffic signal hardware, or in need of upgrade of the timing plans and traffic control software (FHWA 1995).

Making improvements to traffic signals can be one of the most cost-effective tools to increase mobility on arterials. A few simple, low-cost adjustments to a traffic signal system can often significantly improve traffic flow. In many cases, traffic signal equipment can be updated. This allows for greater flexibility of timing plans, including coordination with other nearby signals for progression. In some cases, existing equipment may be adequate; however, due to changing traffic patterns, timing plan improvements may be needed to accommodate current traffic flows more efficiently.

Much of the delay experienced by motorists during the day occurs as they wait for the light to turn green at signalized intersections. Delays can be reduced, however, by optimizing signal timings. Although more than half of the signalized intersections in the United States would benefit from equipment upgrades, nearly 1.5 % of the intersections would benefit from signal timing adjustments alone - without any hardware changes (FHWA 1995).

Highway agencies are supposed to regularly monitor traffic at intersections and then update their traffic control strategies, including signal timing plans, to ensure that a signal system is working properly so that traffic is flowing efficiently and safely. Unfortunately, this is often not the case (FHWA 2002). Many jurisdictions fail to update timing control strategies because it is labor intensive and costly. As a result, the original traffic signal plan for an intersection often continues operating long after changing traffic volumes have made it outdated.

1.2 Signal Timing Parameters

Federal Highway Administration defines fundamental signal timing variables as follows (FHWA 1996):

- Cycle Length - the time required to complete one sequence of signal intervals (phases).
- Phase – the portion of signal cycle allocated to any single combination of one or more traffic movements simultaneously receiving right-of-way during one or more intervals.

- Interval – a discrete portion of the signal cycle during which the signal indications (pedestrian or vehicle) remain unchanged.
- Split – the percentage of a cycle length allocated to each of the various phases in a signal cycle.
- Offset – the time difference between the start of the green indication at one intersection as related to the system time reference point.

According to the same handbook (FHWA 1996) cycle length, split and offset are the three fundamental signal timing variables (also called signal timing parameters). As a group they are often referred to as ‘signal timings’. The following sentences further clarify these concepts.

Traffic control and coordination is achieved by applying proper phases, splits, cycle times, and offsets. The offset is the difference in time between when one approach (e.g., northbound) on one signal gives green and the time when an adjacent signal (e.g., also northbound) gives green. The offset is often related to the time it takes a vehicle to travel between the signals. Coordination is what results from properly tuned offset values along a corridor.

The cycle time is time from the beginning of green, through amber and red to the beginning of green on a single signal phase. It is generally more straightforward to maintain coordination when cycle lengths are equal between adjacent signals. The split is the proportion of time allotted to each approach (i.e., 20% for the North and South legs and 80% for the East and West legs).

The three basic types of traffic controls are pretimed control, actuated control, and adaptive control. These three types represent different approaches to the problem of keeping traffic control up-to-date with changes in traffic conditions. More about each type will be presented in later chapters. However, all of these traffic control types have one thing in common: each requires an input to the controller consisting of initial control settings for the traffic signal timings. Range and depth to which the traffic control settings (a.k.a. signal timings) are specified in initial timing design differ significantly among various traffic control types. Hence, the pretimed traffic control requires that all basic parameters (cycle length, phase splits, intergreen intervals, and offset) are predetermined. Once these settings are entered, they remain constant until the next retiming project, which could be years away. The other types of control (actuated and adaptive) are responsive traffic controls. They respond to various changes in traffic conditions. While actuated traffic control responds mostly to changes in traffic distribution (by allocating appropriate amounts of green time to each traffic movement), adaptive control responds both to changes in traffic demand and traffic distribution. These control types usually require only broader limits within which these parameters may vary based on the online variations of traffic conditions. The initial design for actuated traffic control requires defining parameters, such as cycle length, offset, minimum green times, maximum green times, passage times, minimum gaps, etc. An Adaptive Traffic Control System (ATCS) requires either an extensive library of signal timing parameters or a range within which the system itself calculates the parameters, depending on the adaptive method used.

It is well known among signal timing practitioners that pretimed traffic control needs regular updates of the signal timing parameters to cope with changed traffic conditions (Tarnoff and Ordonez 2004, Sunkari 2004). The other two types of traffic control are known to be effective in coping with changes in traffic demand and/or traffic distribution. However, the extent and consistency of their effectiveness has not yet been thoroughly investigated.

Actuated traffic control is generally perceived to need regular updates for some of the signal timing parameters (cycle length and offsets). However, little is known about the extensiveness of deterioration of this control type when compared with pretimed traffic control. On the other hand, most traffic signal practitioners believe that ATCS do not require any updates. Unlike pretimed and actuated control systems, which usually change the signal timing plans based on Time-Of-Day (TOD) traffic patterns prepared off-line, ATCS account for changes in traffic demand (at an intersection approach). For this reason it is generally believed (and often advertised by ATCS vendors) that the ATCS do not deteriorate over time (TRL 2003, TYCO 2003). In other words, it is believed that once the signal timings are initially entered and calibrated, the systems take care of their adjustment regardless of the changes in traffic. The literature does not provide enough evidence to support or reject these beliefs. There is no study or article describing the behavior of ATCS when traffic conditions are changed over the long term. Therefore, either the ageing of ATCS has not been investigated enough, or the results of such investigations have not been published.

1.3 Problem Definition

In the last 20 years, investigation of the benefits and disbenefits of the deterioration of traffic control systems has not drawn a lot of attention from traffic researchers. The first significant attempts to overcome the obsolescence of pretimed signals were made in the 1970s. The pretimed TOD traffic signals' inability to respond to traffic fluctuations led to the development of several ATCS (Hunt et al. 1981, Lowrie 1982, Gartner 1983) in the late 1970s and the early 1980s. The benefits of ATCS over pretimed signal systems were perceived and quantified right after the first ATCS were implemented. These benefits were well documented in several studies (Robertson and Hunt 1982, Luk et al. 1982, 1983, Robertson 1987). However, most of these benefits are associated with diurnal or weekly changes of traffic demand and distribution. They are not associated with any long-term changes in traffic conditions.

The benefits of updating the pretimed control systems (or disbenefits of not updating, which is an equivalent measure of deterioration) were not investigated until 1986. The key research was conducted by Bell (1985), comprehensively quantifying the disbenefits of not updating the aged pretimed traffic signal systems. The results showed that the disbenefits for grid networks were around 3.8% per year, with up to four years elapsed time between updates. Much later, two additional studies estimated the disbenefits of traffic signal ageing. The FHWA Primer (FHWA 1995) reports that improvement of coordinated traffic signal timing plans reduces travel times by 12% on average. A study from the Institute of Transportation Engineers (ITE) (Sunkari 2004) illustrates that user costs increase substantially if timing plans are not updated at least every three years. These two studies, however, do not provide details for the assumptions and methodology used to estimate the costs of not updating timing plans. So, the Bell and Bretherton study remains the major research. It is frequently cited in reference to the obsoleteness of pretimed signal systems (Luyanda et al. 2003, Fehon 2004). More on the shortcomings of previous research on the deterioration of pretimed control is given in the Chapter 2 of the dissertation.

There is no investigation on deterioration of performance due to change in traffic conditions for the other two major types of traffic controls (actuated and adaptive). There are two major reasons for the lack of research in deterioration of adaptive control. First, the ATCS are expensive and, therefore, not easily available to traffic researchers. As such, the systems are usually installed and used by government agencies for traffic control purposes only. Second, the ATCS developers and vendors which are able to investigate their systems might be hesitant to publish findings revealing potential shortcomings of the systems. However, even more surprising is that there is no published research quantifying the deterioration of actuated traffic control – the form of control that is currently used at most intersections in the USA.

The purpose of this study is to quantify the level of deterioration for actuated and adaptive controls and to give an update on the deterioration of pretimed control. The first contribution of this dissertation to the body of knowledge comes from the fact that no previous research has assessed the deterioration of actuated and adaptive traffic controls. The second contribution of this study is the development of the methodology to quantify this deterioration for adaptive traffic control. More on this subject is provided in the Chapter 3 of the dissertation.

Pretimed and actuated traffic controls are standard controls that can be found at most traffic signals throughout the world. Unlike pretimed and actuated controls, deploying adaptive traffic control means purchasing a license and installing technology, which enables use of real-time detector data to adjust/optimize traffic flows in the network. Adaptive installations often require even more than that. They are customized and tailored to satisfy specific requirements considering the idiosyncrasies of specific road and traffic conditions. More information about various adaptive controls is given in the Chapter 2 of the dissertation.

SCOOT (Split Cycle Offset Optimization Technique) is selected to be a representative of adaptive control for this study. There are two reasons for selecting SCOOT. First, SCOOT is one of the most well-known and widely used adaptive techniques in the world. Second, the first academic license to use SCOOT was presented to the University of Utah in Salt Lake City, Utah. By conducting this research at the Utah Traffic Lab (UTL), the author has the unique opportunity of using the actual SCOOT control interfaced to the VISSIM traffic simulation software (Hansen and Martin 1998, Feng and Martin 2002).

The research questions that motivated this study relate unknown impact of changes in traffic flows on the degradation of the performance of various traffic control regimes. The first question for each control type is whether the control regime deteriorates with changes in traffic flows. The second question is whether the deterioration of each control type is smaller or larger than the deterioration of other control types.

1.4 Research Goal and Objectives

The goal of the study is to investigate the deterioration of various traffic control regimes. Three traffic control regimes are selected representing pretimed traffic control, actuated traffic control, and adaptive traffic control. Their performances are evaluated on the network of urban arterials for various changes in traffic demand and distribution. The major objectives of the study are:

- Develop methodology to assess deterioration of the traffic control regimes through microsimulation
- Evaluate deterioration of the traffic control regimes with respect to modeled changes in traffic demand and distribution

The foundation of this research lies in three major hypotheses. The hypotheses are based on the case that signal timing parameters are not updated regularly. The hypotheses are:

1. $H_{0(1)}$ - Pretimed traffic control plans do not deteriorate with changes in traffic demand and distribution
2. $H_{0(2)}$ - Actuated traffic control plans do not deteriorate with changes in traffic demand and distribution
3. $H_{0(3)}$ – SCOOT adaptive traffic control does not deteriorate with changes in traffic demand and distribution

1.5 Dissertation Organization

This dissertation is divided into six chapters. Chapter 1 – Introduction introduces the reader to the essential background of traffic control systems and the importance of their maintenance. Signal timing parameters are discussed as one of the key factors for updating traffic control systems. In the later part of the chapter the research problem was stated and the goal and the objectives of the research were stated.

Chapter 2 – Literature Review provides a comprehensive overview of the related research. The literature review concentrates on two topics. The first topic deals with various types of traffic control used in this study. A short history of traffic control systems, types of traffic control, and, their most essential features are presented. Special emphasis is given to the SCOOT system since it is the most complex and least known control type used in this study.

Chapter 3 – Research Methodology describes the approach to conducting the research. The first part of the chapter introduces the concept of ageing of traffic control regimes. A method is proposed to measure the extent of the signal timing plans' ageing. The next part of the chapter presents the network and the set of assumptions used to design simulation experiments. Selection of appropriate tools for assessing the ageing of signal timing plans follows. This section represents a study within the main study. It has its own literature review, methodology, results, and discussion. The next part of the chapter deals with the main modeling process. It describes modeling experiments for the three types of traffic control and validation of the SCOOT model. The last part of the chapter summarizes the research methodology and presents its findings.

Chapter 4 – Results provides the findings of the experiments. The results are presented in the form of graphs and tables with short discussion of their meanings. The chapter is divided into four sections. The first part of the chapter presents a general finding about the methodology used in this study. The second section deals with the reliability of the ageing measure used in Bell's work. The third part of the chapter presents the results for ageing of pretimed and actuated-coordinated traffic control regimes for deterministic and stochastic changes in traffic flows. The final part of the chapter shows the results of assessing the ageing of SCOOT adaptive control.

Chapter 5 – Discussion analyzes the results presented in the previous chapter. First, the general methodology used to assess the ageing of traffic control is discussed. Next, the ageing measure used by Bell is discussed, along with the reason for its inability to provide a reliable measure of aged traffic flows. The third part of this chapter discusses the results of assessed ageing of pretimed and actuated traffic control regimes. Specific reasons for the particular results are provided. The last part of the chapter discusses SCOOT performance and the ageing of the SCOOT control.

Chapter 6 – Conclusions provides conclusions of the research and directions for future research.

CHAPTER 2

LITERATURE REVIEW

This chapter presents the findings of the literature review. A comprehensive literature search is done and findings are grouped into three subchapters. The first section reviews the different types of traffic control used in this study. It presents a short history of traffic control systems, types of traffic control and their most essential features. Special emphasis is given to the SCOOT system since it is the most complex and least known control type used in this study. The second part of the chapter provides a review of a few studies that have investigated the deterioration of traffic control systems. The final part of the chapter summarizes the literature review.

2.1 Traffic Control Systems

Over the past few decades, traffic signal control systems have evolved along with technological advancements in electronic, communication, control, and computer fields. Computers have allowed the development of off-line traffic signal optimization. One type of optimization philosophy, represented by Traffic Network Study Tool 7F (TRANSYT 7F) (Robertson 1969) and SYNCHRO (Husch and Albeck 2003 (I)), models vehicle arrivals at each approach to an intersection and attempts to minimize delays, stops, and queue lengths. A second type of philosophy uses bandwidth maximization to determine the number of vehicles that can progress along a series of signals based on a time-space relationship. Some specific applications of this philosophy are MAXBAND (Little et al. 1981), PASSER-II (Chang 1988) and REALBAND (Dell'Olmo and Mirchandani 1995). These optimization techniques are used with historical traffic flows to produce a fixed-time plan. These plans are typically implemented during times of the day when traffic flows are distinct, such as during the morning and evening commute hours. Because these plans are based on historical flows, they require periodic updating. It has been shown that these plans degrade (deteriorate) by 3-5% per year (Bell and Bretherton 1986). Often, municipalities will update these plans infrequently because of cost constraints, resulting in an inefficient signal-timing plan.

The FHWA sponsored a research program called the UTCS (Urban Traffic Control System) in the 1970s as a way to improve performance of traffic signal systems. The research developed and tested three strategies (or generations) of adaptive traffic signal control (MacGowan and Fullerton 1979):

Generation 1 (1GC) uses prestored signal timing plans that are calculated off-line and are based on historical data. These either go into effect at a specific time of day or by an operator selecting the best-suited plan from an existing library of plans for the various traffic conditions on that network.

Generation 1.5 (1.5GC) is the same as 1GC, except new timing plans are generated automatically when traffic conditions warrant them.

Generation 2 (2GC) calculates and implements timing plans based on surveillance data. This is repeated at 5-minute intervals. To avoid too much change, the system is not allowed to implement a change at two 5-minute intervals in a row, or to have varying cycle lengths on signals within a group.

Generation 3 (3GC) is similar to 2GC, except it is allowed to change cycle lengths at 3-5-minute intervals and the cycle length is allowed to vary between signals as well as during the same control period.

Field tests performed on these control strategies yielded surprising results. Generation 1 performed the best. Within 1GC, plans selected by an operator functioned better than the time-of-day plans. Generation 2 had a mix of success and failure. The average benefit, however, was inferior to 1GC. Generation 3 was unsuccessful altogether and degraded conditions in almost every case.

Some hypotheses for why the failures occurred are:

- inaccuracies in flow measurement caused 2GC and 3GC to not be able to respond quickly enough
- inadequate transition logic was used between plans, causing traffic to be caught in the middle of progression (transients), and
- the benefit of coordination outweighed the benefit of local signal optimization.

Modern traffic control systems can be divided into three major categories: pretimed, actuated, and adaptive. Pretimed traffic control and actuated traffic control are frequently referred to as fixed timing controls.

2.1.1 Pretimed Traffic Control Systems

Pretimed controllers represent traffic control in its most basic form. They operate on a predetermined and regularly repeated sequence of signal indications. The pretimed traffic control systems use controllers that require constant cycle time length, split length and split sequence. Signal timing plans are developed off-line and optimized according to historical traffic flow data. Traffic control systems commonly have a series of predetermined plans to accommodate variations in traffic volume during the day, such as peak and off-peak traffic conditions.

Figure 2.1 shows two coordinated intersections with pretimed control and four phases for traffic movements. Coincidently, these two intersections have the same splits. In reality, the splits do not have to be equal but if intersections are coordinated they will always have the same cycle length. Offset between intersections defines beginning of the coordinated phase at the slave intersection (Phase 4 in Figure 2.1) in order for traffic to get progression between these two intersections. Progression refers to the nonstop movement of vehicles along a signalized street system.

Pretimed controllers are best suited for intersections where traffic volumes are predictable, stable, and fairly constant. Once the timing programs are set, they are updated to adjust to changes in traffic flow. The frequency of the updating largely depends on the variation of traffic flow and available resources. Generally, pretimed controllers are cheaper to purchase, install, and maintain than traffic-actuated controllers. Their repetitive nature facilitates coordination with adjacent signals, and they are useful where progression is needed. Properly timed signal systems facilitate progression.

Many algorithms for developing pretimed signal plans were developed. Software packages were developed to design isolated-intersection signal times, including Highway Capacity Software (HCS), Signal Operations Analysis Package

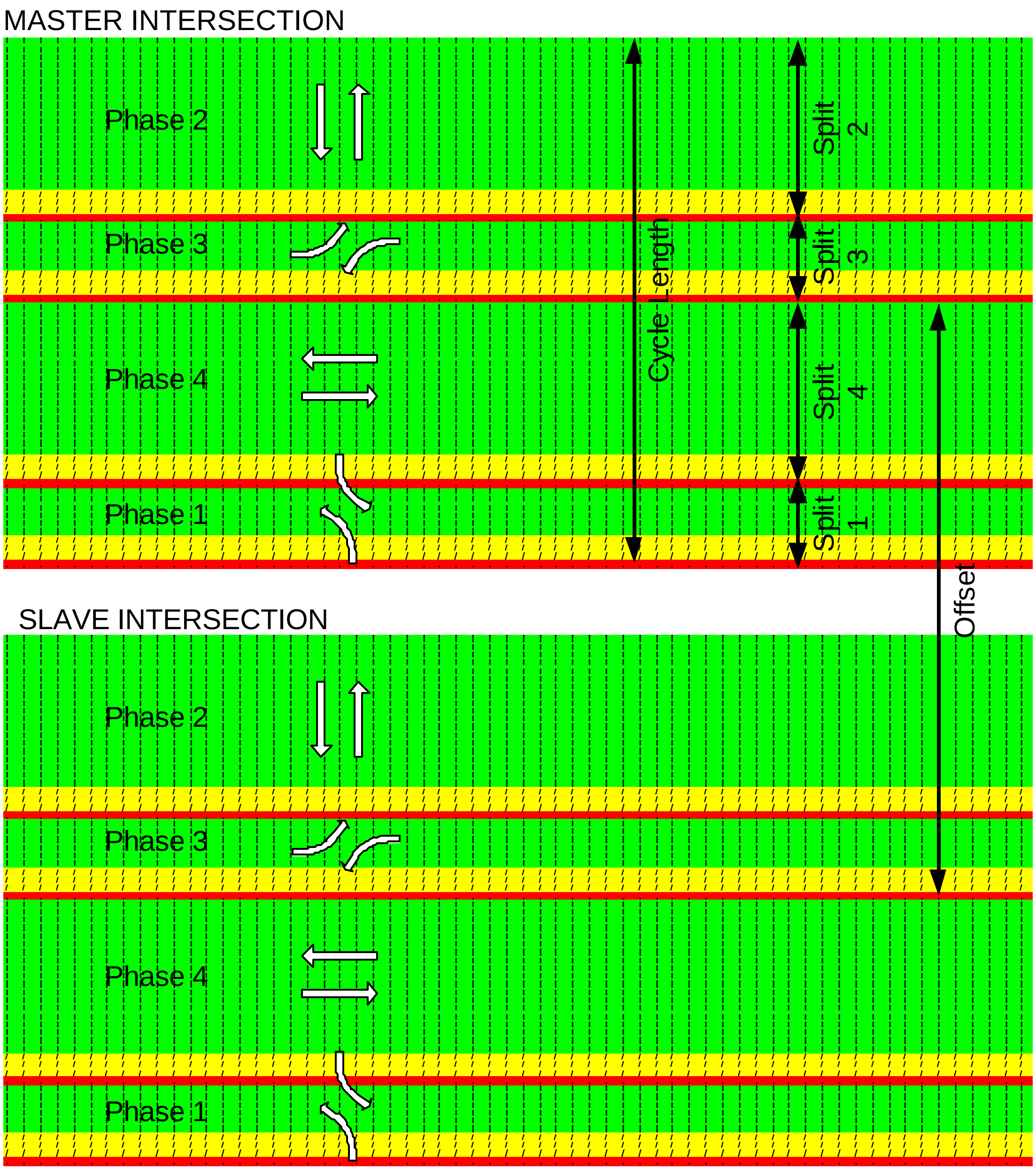


Figure 2.1 – Pretimed Signal Timing Plans for Two Adjacent Intersections

(SOAP) (Courage et al. 1979), SIGNAL94 (Strong Concepts 1995), and Signalized Intersection Design and Research Aid (SIDRA) (Akcelik and Besley 1998). Some packages, such as HCS, Progression Analysis and Signal System Evaluation Routine (PASSER III), MAXBAND, TRANSYT-7F, and SYNCHRO, are for arterial signal timing design. TRANSYT-7F, SYNCHRO, and PASSER IV can also design signal-timing plans on networks. A report (Sabra et al. 2000) compared several factors of these signal-timing packages, including applications, animation, measures of effectiveness, data input requirements, operating system, and minimum hardware requirements.

2.1.2 Actuated Traffic Control System

A simple traffic-actuated signal installation consists of four basic components: detectors, the controller unit, signal heads (the traffic lights), and communications. Traffic-actuated control systems differ from pretimed control systems. Their signal indications are not of fixed duration, but rather change in response to variations in the traffic volumes. Traffic-actuated controllers are typically used where traffic volumes fluctuate irregularly or where it is necessary to minimize interruptions to traffic flow on the street carrying the greater volume of traffic.

There are two major types of actuated signal control systems: semi-actuated and fully-actuated. Semi-actuated systems give green time to minor streets only when a vehicle is detected. These systems are most appropriate for locations with a low volume of minor street traffic. Fully-actuated control systems detect vehicles for all approaches and serve phases as demand on all approaches. Both semi-actuated and fully-actuated signal control systems can be uncoordinated or coordinated. Some systems can select the best signal-timing plan from a library according to recently measured traffic conditions (called Traffic Responsive Pattern Selection).

This research investigates only coordinated actuated systems. Uncoordinated actuated control is good only for individual intersections where coordination of green times at adjacent intersections is not important for progression of vehicles. A grid network of relatively closely spaced intersections is used for the experiments conducted in this research. Coordinated actuated traffic control is the only method that makes sense in the study network, due to the geometric conditions of the network. Uncoordinated actuated control would not be able to yield performance comparable to any type of coordinated traffic control.

Actuated signal timing plans are also predesigned by off-line signal timing design packages, including SOAP, SIGNAL94, TRANSYT-7F, SIDRA, and SYNCHRO. The cycle time lengths and offsets remain constant. However, actuated signal control systems can adjust the lengths of phases between minimum and maximum thresholds in response to vehicle actuations, or they can skip phases. The systems do not predict traffic flow.

Figure 2.2 shows a timing plan for a single intersection with coordinated actuated control. Unlike pretimed control, this timing plan has eight phases. Four of the eight phases (left turn phases) can be skipped if there is no demand for them. The length of a cycle can be extended only if the previous cycle was shortened due to lack of demand at certain phases. Phase splits change in order to respond to variable demands of turning movements. The coordinated phase (Phase 2 in Figure 2.2)

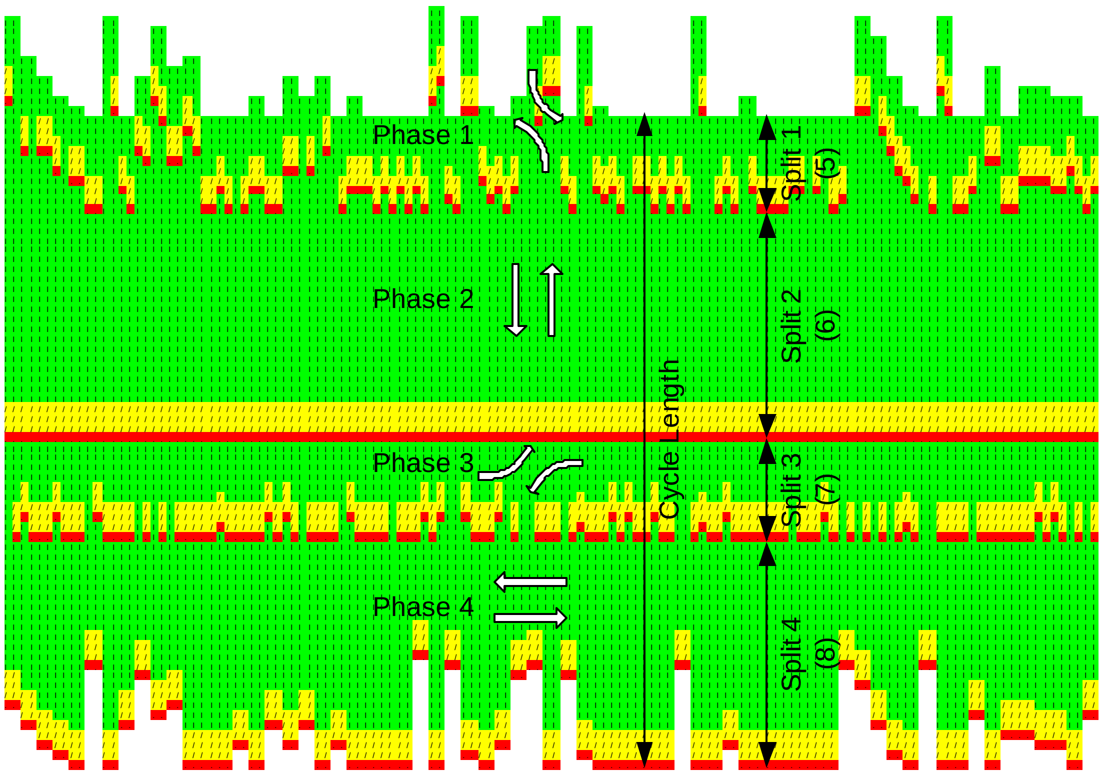


Figure 2.2 – Actuated Signal Timing Plan for an Intersection

always ends at the same time as the defined cycle length. This is necessary to provide progression at the adjacent intersections.

2.1.3 Adaptive Traffic Control Systems

ATCSs optimize signal timings on-line. They adjust traffic signal timings in response to variations in traffic flow. Systems use upstream and/or stop line detectors to measure real-time traffic flow information, usually consisting of volume and occupancy. They use this information to predict traffic flow conditions, such as vehicle arrivals, queues and turning percentages. Systems optimize traffic signals to reduce delay, stops, travel time, and emissions based on measured and predicted information. Figure 2.3 shows how basic signal timing parameters in SCOOT change in time according to changes in traffic flows. Unlike pretimed and actuated-coordinated control, all three parameters change, allowing the system to choose parameters that yield the smallest congestion and delays for all users in the region. The beginning of Phase 4 in Figure 2.3 is kept constant only for the purpose of presenting the signal timings. The actual starting time of Phase 4 varies too.

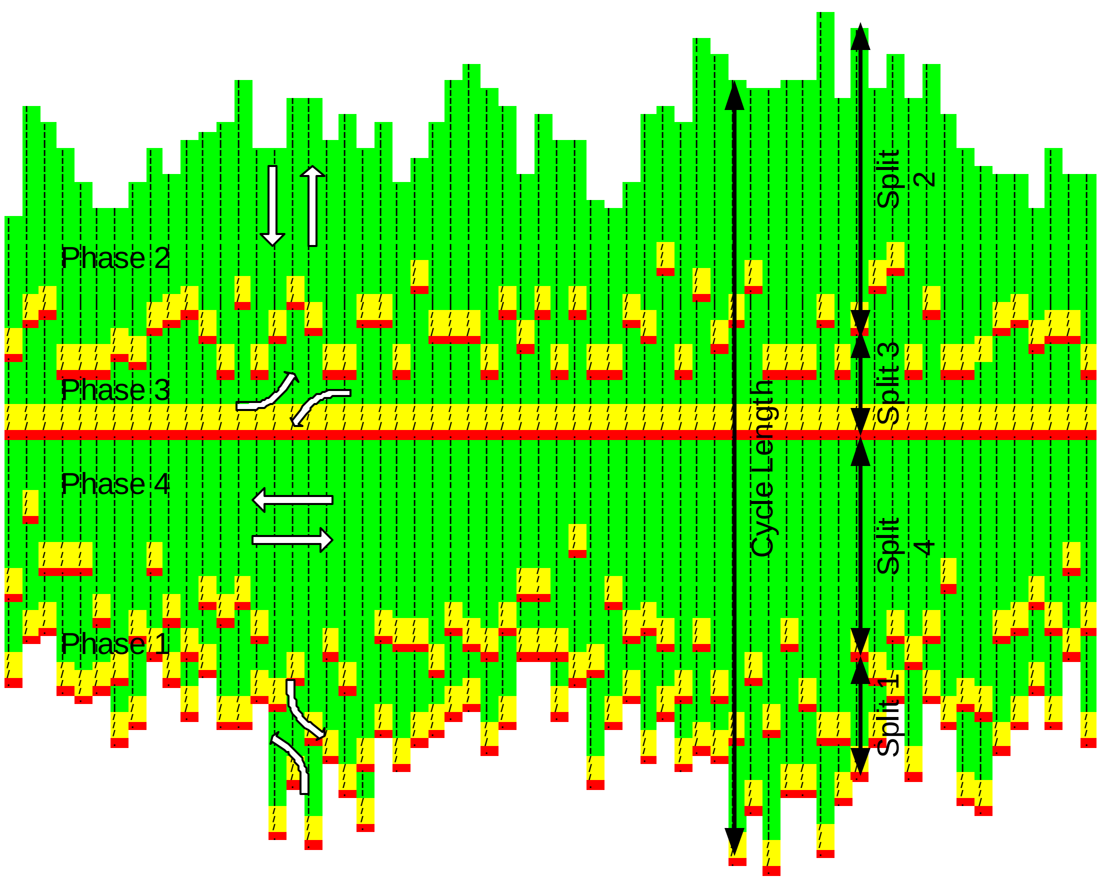


Figure 2.3 – SCOOT Adaptive Timing Parameters for an Intersection

The most widely deployed adaptive signal control systems in North America are SCATS (developed in Australia) (Lowrie 1982), SCOOT (developed in the U.K.) (Hunt et al. 1981), RHODES (Head et al. 1992) and OPAC (Gartner 1983) (developed in the U.S.).

Other adaptive signal control systems include LA-ATCS (developed in the U.S.) (Hu 2000), UTOPIA (developed in Italy) (Mauro and Di Taranto 1990), PRODYN and CRONOS (developed in France) (Khoudour et al. 1991, and Boillot et al. 1992). All of these adaptive signal control algorithms can be classified as either 2GC or 3GC. Essential operations and philosophies of the major systems deployed in the U.S. are described below. The SCOOT system represents adaptive traffic control in this research. For this reason, SCOOT is described in a separate section to provide more details about its philosophy and operations.

2.1.3.1 SCATS

SCATS stands for Sydney Coordinated Adaptive Traffic System. SCATS is a hierarchical adaptive signal control system installed in over 50 locations worldwide. SCATS was originally developed for the New South Wales Roads and Traffic Authority for application in Sydney and other Australian cities. In this system, signal timings are adjusted in response to traffic data measured by stop line detectors. SCATS minimizes travel time, stops (for light traffic), and delays (for heavy traffic). Cycle length optimization aims to keep the degree of saturation below the target and split optimization aims to minimize delay at intersections. Offset plans are selected from a predetermined library based on traffic flow on the links. The system does not determine signal timings in advance but it reacts to changes in traffic flows that occurred in the previous cycle. The system can be loosely described as a feedback control system (Lowrie 1982).

SCATS has a hierarchical control architecture consisting of two levels, strategic and tactical (Lowrie 1992). At the strategic level, a ‘subsystem’ or a network of up to 10 intersections is controlled by a regional computer to coordinate signal timings. These subsystems can link together to form a larger ‘system’ operating on a common cycle time. At the tactical level, optimization occurs at the intersection level within the constraints imposed by the regional computer’s strategic control. Tactical control allows early termination of green phases when the demand is less than average and for phases to be omitted entirely when there is no demand. All the extra green time is added to the main phase or can be used by subsequent phases.

Field tests conducted in the U.S. showed that SCATS was efficient in improving network performance. In Troy, Michigan the SCATS system reduced travel time by 20% in the AM peak period and by 32% in the nonpeak period. The system also reduced delays by 20% (Abraham 2000). In Oakland County, Michigan, a 3.1-mile corridor was controlled by SCATS. The test results indicated that travel time was decreased by 8.6% in the AM peak, by 7% in the PM peak, and by 6.6%-31.8% in the nonpeak periods (Abdel-Rahim et al. 1998).

2.1.3.2 RHODES

RHODES (Real-Time Hierarchical Optimized Distributed and Effective System) prototype was developed at the University of Arizona. As a hierarchical system, RHODES has three levels, including dynamic network loading, network flow control, and intersection control. RHODES proactively responds to stochastic variations in traffic flow by explicitly predicting individual vehicle arrivals, platoon arrivals, and traffic flow rates for the three levels. The optimization objectives can minimize average vehicle delay, average queues, and number of stops (Head et al. 1992).

At the intersection control level, RHODES uses a dynamic programming-based algorithm to make decisions on signal stages based on predicted vehicle arrivals, coordination constraints, flow predictions, and operational constraints. At the network flow control level, a model called REALBAND optimizes the coordination of observed and predicted platoons in the sub-network. At the dynamic network loading level, the slowly-varying characteristics of traffic flow are controlled. The five core-logics in the RHODES prototype are intersection optimization logic, link flow prediction logic, network flow optimization logic, platoon flow prediction logic, and parameter and state estimation logic (Mirchandani and Head 2001).

RHODES has also been linked to the CORSIM traffic simulation software. A simulation experiment was taken on an arterial with nine intersections in Atlanta, Georgia. Compared to semi-actuated timing plans generated by PASSER and TRANSYT-7F, RHODES reduced average vehicle delays by 50% (for low traffic) and 30% (for high traffic) (Mirchandani and Head 2001).

RHODES has been implemented in several locations, including Tempe and Tucson (Arizona), Santa Clara County (Sunnyvale, California), Seattle (Washington) and Oakville (Canada). Field test results from Seattle, WA showed that RHODES performed generally better or the same as optimized fixed-time plans in terms of arterial travel times (ITT Industries 2002).

2.1.3.3 OPAC

Parsons Brinkerhoff Farradyne Inc., and the University of Massachusetts at Lowell jointly developed OPAC (Optimized Policies for Adaptive Control Strategy) (Gartner 1983). For optimization, the network is divided into subnetworks, which are considered independently. OPAC transitions between two models: one for congested networks and the other for uncongested networks.

In the uncongested model, the signal timings are determined one of two ways: Fixed-time plans are obtained off-line, or a “virtual cycle” is calculated dynamically. The level of local to network control can be configured by the user. The local signal timings are based both on detected data (15 seconds) and predicted data (60 seconds). These are implemented for a time-step (roll period) of 2-5 seconds. In the congested model, OPAC considers the saturation flows and maximizes the number of vehicles that can pass through an intersection. It also considers the critical links as those that are on the verge of spillback. Except for the computation of cycle length, OPAC is not controlled by a central computer. Hence, it can run autonomously if communications to the central server fail.

In the late 1990s and early 2000, OPAC was deployed on Route 18 in New Jersey, Reston Parkway, VA, and Vancouver, WA. Some of the tests done in the simulation environment showed the benefits of deploying OPAC over existing pretimed plans (Stallard and Owen 1998). Unpublished literature (Ghaman Undated) reported that results from the field tests on the Reston Parkway in VA showed that OPAC could improve traffic performance by 5 to 6%. Currently, OPAC is not fully operational anywhere (Fehon 2005).

2.1.4 SCOOT

SCOOT (Split Cycle Offset Optimization Technique) is a centralized traffic adaptive signal control system. SCOOT was developed in the UK in the early 1980s by the Transport Research Lab (Hunt et al. 1981). SCOOT continuously measures traffic volumes on all approaches to intersections in the network and changes the signal timings to minimize a Performance Index (PI). This PI is a composite measure of delay, queue length and stops in the network. These changes in signal timings are small enough to avoid major disruptions in traffic flow, but frequent enough to allow quick response to changing traffic conditions.

2.1.4.1 Working Principles

The principal publication on SCOOT is the original report presented by the Transport Research Laboratory (Hunt et al. 1981). It describes the details of the SCOOT philosophy. The system has been modified and enhanced several times since then (Bretherton and Bowen 1990, Bretherton 1996, Bretherton and Bowen 1998). The most recently developed version is called SCOOT MC3 (which stands for Managing Congestion, Communications and Control). The traffic lab at the University of Utah currently uses an older version, 4.2 (versions 4.5 and MC3 were developed after version 4.2).

SCOOT's most fundamental goal is to minimize a measure of effectiveness called the PI (Performance Index) (Hunt 1981). The PI is a composite measure of the average queue lengths and the number of stops at all approaches. To this end, real-time traffic flow data must be obtained from detectors in the street. These data are fed into a model that develops a Cyclic Flow Profile (CFP). It then projects platoon movement and dispersion downstream. Figure 2.4 shows this prediction process along a 300-meter road segment. In this way, SCOOT's traffic model estimates the accumulation of queues at each approach of an intersection. The model then estimates the queue discharge when the signal allows traffic to proceed.

Several parameters are needed to allow SCOOT to model these activities with sufficient accuracy to create an optimum signal timing plan. These model parameters

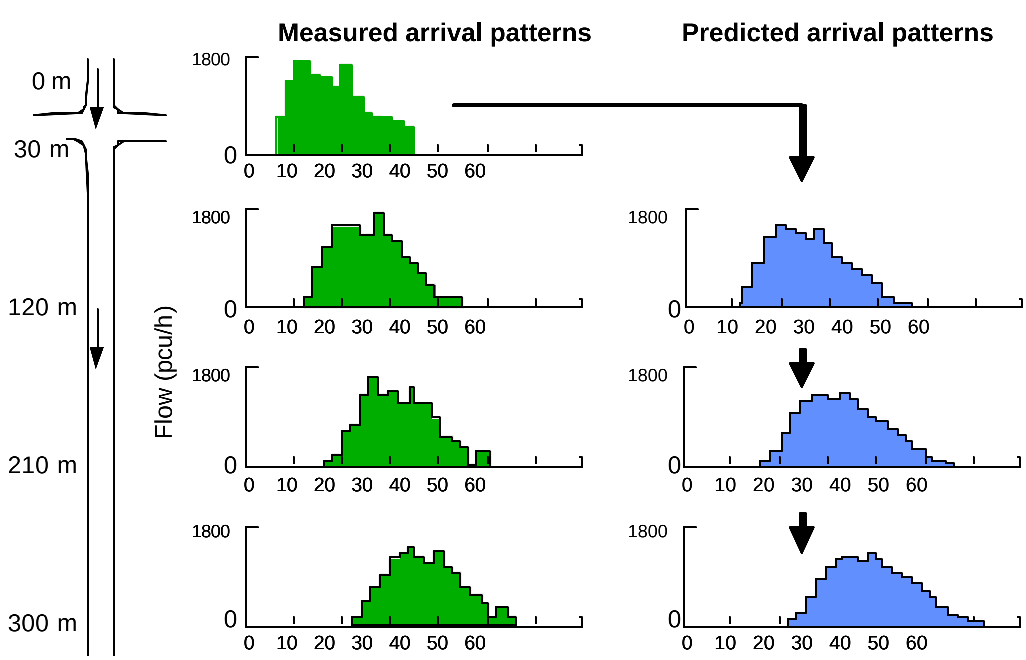


Figure 2.4 – SCOOT Predictions of Downstream Flow

are start-up delay (SLAG), journey time (the average travel time from the detector to the stop bar) (JNYT), saturation occupancy (a measure of the flow capacity through the intersection) (STOC), and Maximum Queue Clear Time (the time it takes for a link to discharge a queue reaching from the stop line to the upstream detector) (QCMQ).

All of this information is used to develop optimum coordination plans, much in the same way as the TRANSYT-7F software. SCOOT is often referred to as an "on-line TRANSYT" because of this (Martin and Hockaday 1995). The signal timings that are sent to the system reflect movement toward an optimum timing plan by increasing or decreasing the cycle time, offsets, and cycles. When implementing these timings, it is unwise to change the cycle lengths and offsets dramatically. Doing so can have adverse affects on the behavior of drivers because the changes are unexpected. The system therefore changes the offsets and cycle lengths by 4, 8, or 16 seconds. SCOOT allows faster change rates at higher cycle times.

SCOOT has three optimizers - one for cycle time, one for offsets between intersections, and one for splits at each intersection. The split optimizer adjusts the green split between the various legs of an intersection. A few seconds before the end of each phase change, the optimizer is run to see whether early termination or an extension of the phase would improve traffic. The optimizer makes two classes of changes: temporary changes that are likely due to random variation in traffic flow, and long term changes that are smaller and accumulate over time to accommodate true traffic flow variations. The splits are also optimized to work within the offsets determined by the offset optimizer.

The offset optimizer recalculates the offsets for each intersection at a predetermined point in the cycle. The process involves a calculation to see if the offset can be adjusted to improve progression between it and all adjacent intersections. The accumulation of all of these changes occurs over time to accommodate longer-term changes. The offsets are optimized to function within the cycle time determined by the cycle time optimizer.

The cycle time optimizer recalculates the optimum cycle time according to a critical node in the region. A region is a set of adjacent signals that operate at the same cycle time to allow for maximum progression. A critical node is selected in each region. This is generally the node that requires the maximum cycle time.

Once SCOOT is installed on a network, each link must be validated. The validation process entails going out onto the street and making sure that SCOOT is modeling the queues as closely as possible to the way they actually occur on the street. Doing this ensures a proper signal timing plan and the best results. In the SCOOT-simulation environment, validation requires many observations of simulated traffic and relevant SCOOT outputs. SCOOT parameters are adjusted in an iterative process so that SCOOT queues are as close as possible to queues from simulated traffic.

SCOOT has several other features that give the system more flexibility. Similar to fixed-time plans, which require adjustments to get it tuned on the street, SCOOT needs fine-tuning during the initial setup.

2.1.4.2 Evaluation Studies

Several agencies have compared SCOOT's performance to previous signal control strategies. The benefits realized from SCOOT depend on the prior control strategy and how well it was optimized.

The most recently published evaluation was performed by Companhia de Engenharia de Trafego, the traffic engineering company responsible for managing traffic in Sao Paulo, Brazil (Mazzamatti et al. 1998). The evaluation done in Nijmegen, Netherlands compared SCOOT to fixed-time plans (Taale et al. 1998). Before and after studies in Toronto, Canada also compared SCOOT's performance to coordinated fixed-time plans (Quan et al. 1993). Benefits in Beijing, China were higher than the most of the others as SCOOT was compared to uncoordinated fixed-time control (Peck et al. 1990). SCOOT benefits were higher when compared to isolated vehicle actuation than coordinated fixed-time plans in Worcester, UK (Hereford Department of Transport et al. 1986). Evaluations in London (Chandler and Cook 1985), Southampton (Powell 1985) and Coventry in the UK also showed significant benefits from SCOOT in the early 1980s. It should be noted, however, that most of these results were not reported to be significant at a 95% confidence level. Evaluations were also done in Santiago, Chile but were not expressed as percent benefit and hence were not comparable to the other results.

Comparison with fixed-time control showed that SCOOT reduced delay and travel time, thereby improving traffic network performance. Early evaluations in the UK showed that SCOOT typically reduced delay by up to 33% and travel time by up to 8%. The literature also indicated that validating SCOOT during installation is extremely important. A nonvalidated SCOOT system in Nijmegen worsened system performance, but when properly validated, it improved delay by 25% and travel time by 11%.

However, some exceptions to SCOOT performance exist. A report (Jayakrishnan et al. 2001) evaluated the SCOOT system in Anaheim, California using “non-idealized” detectors which were placed 250 feet ahead of the intersection approach stop line and only partially met the requirements of SCOOT. The field test results showed that in some cases SCOOT performed worse than the baseline system and in other cases SCOOT performed better. The variation range was within 10%. Overall, the test did not show significant SCOOT benefits. The author indicated that SCOOT was not beneficial due to the nonidealized detectors. Another reason may be that minimal time was spent fine-tuning SCOOT’s parameters.

In addition to the field tests, SCOOT has been evaluated many times in the simulation environment. Some researchers emulated SCOOT performance through a continual time-sliced application of TRANSYT, an offline optimization program on which the initial SCOOT program was based (Rakha and Van Aerde 1995, Liu and Cheu 2004). These attempts, however, provide questionable findings because SCOOT not only applies TRANSYT optimizations in real time, but also decides when to apply these optimizations to minimize possible transients.

Real SCOOT-simulation modeling started in 1998 when the first SCOOT-CORSIM interface was developed by Hansen and Martin at the University of Utah Traffic Lab (UTL) (Hansen and Martin 1999). The authors compared SCOOT and

TRANSYT-7F performance on a small, 6-intersection network in an urban area. The results showed that SCOOT can reduce delay and stops by up to 30%.

Research in the UTL continued with evaluations of SCOOT control in various traffic conditions. One of the studies (Jhaveri et al. 2001) compared SCOOT performance with fixed-time plans for a range of congestion intensities on a four-intersection corridor. The results showed that most of SCOOT's benefits (8 -13%) come for volume/capacity (v/c) ratios between 0.7 and 0.9. The research showed that SCOOT operates much like a fixed-time control once traffic becomes saturated.

Another study (Chilukuri et al. 2004) investigated the benefits of SCOOT control during incidents. SCOOT performance was again compared to fixed-time control. Incidents with durations of 15, 30, and 45 minutes were modeled in one experimental and two real-world networks. The results showed that SCOOT outperformed fixed-time control with benefits of up to 40% in total network delay, travel time, and intersection delay.

In 2002 another interface between SCOOT and a micro simulation model was developed. Feng and Martin (2002) developed a SCOOT-VISSIM interface on the same basis as the previous SCOOT-CORSIM interface. The SCOOT-VISSIM interface was used to evaluate SCOOT performance and the impacts of SCOOT bus priority on buses and private vehicles under varying traffic conditions. These conditions included three corridor congestion scenarios, three bus frequency scenarios and two bus location scenarios. The study compared SCOOT control with bus priority, SCOOT control without bus priority, and optimized pretimed signal timings. A real-world corridor with nine intersections was also modeled to validate the findings. The research found that SCOOT controls performed better than pretimed control under noncongested conditions. All of the SCOOT controls reduced nonbus traffic delay by 1.5 – 3 seconds (7% - 13%) per person per intersection and bus-person delay by 1 – 9 seconds (3% - 33%). SCOOT control with bus priority was slightly better then SCOOT control without bus priority.

Although most of the SCOOT-simulation research was done in the UTL, there were a few outside studies that investigated SCOOT performance with other simulation tools. Two of those studies were done as part of the PRIMAVERA project (PRIority MAnagement for Vehicle efficiency, Environment & Road safety on Arterials). In the first study (Fox et al. 1994), SCOOT was connected to AIMSUN2 (a traffic microsimulation program) and CLAIRE (an expert system for congestion management). The idea was to pass detector data from AIMSUN2 to SCOOT, which would return signal timings to the simulated traffic network. At the same time, link congestion information from SCOOT was sent to CLAIRE. The CLAIRE program was then used to recommend interventions via link control actions if necessary. The interventions had to be translated into appropriate SCOOT parameter changes. There was no publication available on the field tests of this method.

The second study (Fox et al. 1995) developed an interface program to link the Italian simulation model NEMIS to the UK SCOOT system, allowing them to interact. This interaction was part of the wider project's goal in which co-operative strategies that combine queue management, public transport priority and traffic calming methods on urban arterial roads were developed. Traffic flows on the arterial and surrounding network were collected automatically and passed to computers, which rapidly calculated signal plans to minimize the overall travel time. Public transport vehicle location systems were used to send data to these computers to let them adjust the signal plans to ensure that public transport vehicles got priority when they passed through intersections. The major benefits due to adoption of the PRIMAVERA strategies in Leeds were a 10% reduction in travel time for buses and improvements in safety due to ‘removal’ of the speeding vehicles on entry points to the network.

Another SCOOT-simulation interface was developed in 1998 by Kosonen and Bargiela (1999). This interface connected SCOOT control with HUTSIM microscopic simulation model. The authors described a framework for a connection between SCOOT and HUTSIM, but they did not conduct any comparison between SCOOT and other traffic control systems. They reported that simulation somewhat matched observed vehicle counts, but they did not perform any validity tests.

2.2 Review of the Studies on Ageing of Traffic Signal Timing Plans

Investigation of the deterioration of traffic control systems has drawn little attention from traffic researchers over the last 20 years. Although embedded into few research projects on traffic signal optimization (Park et al. 2000, Sunkari 2004, Swayampakala and Graham 2005), deterioration of signal timings has not been explicitly investigated.

2.2.1 Bell’s Research on Ageing of Fixed-Time Traffic Signal Plans

The key research conducted by Bell and Bretherton (Bell 1985, Bell and Bretherton 1986) comprehensively quantified the disbenefits of aged pretimed traffic signal systems. The authors used TRANSYT 8 (UK version) to simulate the performance index (a composite measure representing the costs of traffic performance) for deterministic traffic flows. The disbenefits of not updating pretimed signal plans were calculated for various traffic networks. Traffic variability was modeled with the Monte Carlo method through a series of random and uniform changes to link flows.

The ageing disbenefits were estimated for both experimental and real networks. The experimental results showed that the disbenefits of not updating grid networks were one tenth of the average difference between base and aged link flows over the entire network. The results from the real world showed that the disbenefits for grid networks were around 3.8% per year, with up to four years between updates.

The study broke new ground, but was constrained by the computational constraints of the mid-1980s. There are four major limitations of the study:

- Small traffic flow variations
- Unbalanced traffic flows between intersections
- No testing through microsimulation
- Unreliable ageing measure

Each of these limitations is commented on separately in the following paragraphs.

Small traffic flow variations – Bell used coefficients from -10% to 10% to represent uniform decreases/increases in background traffic demand for the whole network. Similarly, coefficients from 5% to 25% were used as random fluctuations of the turning movement proportions for each link. When experimental estimations were compared to real traffic data, the author concluded that the range of the variations in simulated flows was smaller than the range from real traffic. To avoid this mistake, this study encompasses a wider range of the similarly constructed coefficients for traffic demand and distribution.

Unbalanced traffic flows between intersections – The version of TRANSYT used in the mid-1980s in the UK did not provide a tool for balancing traffic flows in the network. For this reason, the conservation rule of traffic flows (analogue to the first Kirchhoff's law - conservation of electric charge) was not satisfied. This means that traffic flow on each link was separately changed (increased or decreased), regardless of the changes in traffic flows on the upstream links. This approach could potentially impact the final results of the disbenefits of aged signal timings. All traffic flows in the network used in this study were balanced automatically after any change in the flows.

No testing through microsimulation - The third limitation of Bell's study is that the results were evaluated with the model used to predict the differences. Ideally, if the deterioration is evaluated properly, the differences between optimized and non-optimized performances should be validated with separate simulations or an analytical tool. In this research, microsimulation is selected to check the validity of the differences obtained from macroscopic optimization models.

Unreliable ageing measure – The previous three limitations were either obvious from Bell’s research scope (TRANSYT use only) or recognized by the author (unbalanced flows and too small traffic variations applied). The fourth limitation is implied in Bell’s research paper (1985), but is not recognized. However, the author of this dissertation believes that this ageing measure is the reason for the inconsistency between the results from the experimental study and the real-world data from Bell's study.

Bell developed two ageing measures (A and B) to express changes in traffic flows on the network over the years. These measures are very similar. Measure A considered total flows on the links between any two nodes in the network. Measure B dealt with three link flows (left, through, and right) from the same link used for A (one for each turning movement at the intersection). Consequently, the number of links on the network is three times greater for Measure B than for Measure A.

The concept behind Measure B is denoted CF (Changed Flows) and explained here. This measure was supposed to be a universal measure that shows how much traffic in the network has changed since the last optimization of traffic signals. Bell (1985) calculated Measure B (CF) as an average absolute difference between traffic flows for aged traffic conditions and base traffic conditions, whereby:

$$CF = \frac{\sum_{i=1}^{N} \left| F_b - F_a \right|}{N} \qquad [2.1]$$

Where:

CF = Ageing measure of changed link flows

F_b = Base link flow (for all turning movement links) [veh/hour]

F_a = Aged link flow (for all turning movement links) [veh/hour]

N = Total number of links in the network (all turning movement links)

However, this measure by definition seems to be an unreliable indicator of link flow changes. The measure might work for one type of change in link flows (increase or decrease). However, when changes in link flows are triggered by simultaneous changes in traffic demand and turning movement proportions, Measure B would not be a reliable indication of the aged traffic conditions. One of the secondary objectives of this research is to investigate the reliability of this measure.

2.2.2 Other Studies

The FHWA Primer (FHWA 1995) reported that improvement of coordinated traffic signal timing plans reduces travel times by an average of 12%. This study also gave an estimation of the benefits for several improvements in traffic signal operations. The benefits range from 12% to 25%, depending on the traffic signal operation conditions before and after improvement. However, in general this report is more of a qualitative assessment than quantitative evaluation of the achieved benefits.

In 2000, Park et al. (2000) provided a more comprehensive assessment of the direct Genetic Algorithm (GA) optimization within the CORSIM micro simulation program. Two types of traffic demand changes were investigated. First, the mean rates of demand for the network entry points were changed randomly to ±15% from the base demand to account for changes in mean demand rates since CORSIM always produces demand distribution around predetermined mean rates of demand. Second, a 10% increase over the entire network was modeled to account for systematic changes in travel demand. This approach emulated the ageing of traffic signals. No explicit modeling beyond CORSIM's own stochastic variability was made to simulate changes in turning movement proportions at the intersections. The study found that optimizing timing plans for a 10% higher traffic demand on the network yields better performance than optimizing for the current demand, if demand is expected to grow. In other words, if demand is expected to grow, signal timings should be optimized for higher demand and the benefits will increase over the long term. The benefits of optimizing the timings for 10% higher demand surpass the cost of not optimizing the timings for higher demand while the demand is still low.

An ITE study (Sunkari 2004) illustrated that user costs increase substantially if timing plans are not updated at least every three years. This study also represents qualitative assessment of benefits of retiming traffic signals without details for the assumptions and methodology used to estimate these benefits.

Swayampakala and Graham (2005) investigated the optimal time between traffic signal retiming. They analyzed more than 6,400 Synchro files representing existing and future conditions at 13 signalized intersections. They concluded that signals should be retimed every 18 to 30 months depending on the variability of traffic volumes at the intersections. Microscopic simulation was not used in this study.

2.3 Summary of Literature Review

The first section of the literature review introduces basic concepts of the three most common types of traffic control. Pretimed, actuated, and adaptive controls are described, with the most emphasis given to adaptive control. Four major types of adaptive traffic controls deployed in the U.S. are described. Special emphasis among adaptive controls is given to the SCOOT – the adaptive control used in this research.

SCOOT's major working principles are described, along with a review of both field and simulation tests of SCOOT installations.

The second part of the literature review presents summaries from a few studies on the ageing of traffic signal systems. Special attention is given to Bell's study (Bell 1985, Bell and Bretherton 1986) because this study has been the only study that has explicitly investigated the ageing of traffic signals for the last 20 years. The major limitations of Bell's study have been listed along with proposed approaches to avoid the same limitations in this research. Finally, a few other studies that have touched on the same topic have been reviewed.

Summarizing the literature review on ageing of traffic signals, there is no comprehensive study that investigates the ageing of signal timing plans using macrosimulation and microsimulation analysis tools. Further, the last attempts to investigate the reliability of macro optimization plans did not include variations in turning movements. The methodology used 20 years ago to assess the ageing of pretimed traffic signal plans was limited by the processing power available at the time.

CHAPTER 3

RESEARCH METHODOLOGY

The first part of this chapter describes the concept of ageing (deterioration) of signal timing plans. A method is proposed to measure the extent of the signal timing plans' ageing. The next part of the chapter presents the network and the set of assumptions used to design simulation experiments. Selection of appropriate tools for assessing the ageing of signal timing plans follows. This section represents a study within the main research. It has its own literature review, methodology, results, and discussion. The next part of the chapter deals with the main modeling process. It describes modeling experiments for the three types of traffic control and validation of the SCOOT model. The last part of the chapter summarizes the research methodology and presents its findings.

3.1 The Concept of Ageing of Traffic Control Systems

The deterioration of traffic signal systems is a complex process. The two major factors that influence the traffic signal systems' deterioration over time are:

1. A traffic control regime does not have information about the current traffic in the network that it is supposed to control
2. The traffic control system is unable to find the optimal control for traffic in the network.

The first reason for poor traffic control is associated with the traffic control system's inability to gather updated traffic flow information from the road network. Situations that keep the traffic control system from having sufficient information about traffic conditions are: no detectors, too few detectors, faulty detectors, detector data that is insufficient for good control, poor detector locations, etc. The second reason is associated with the traffic control system's inability to find the optimal control based on the gathered information.

This research deals with both factors that influence the quality of a traffic control regime. In the case of pretimed and actuated control, deterioration of control regimes is mostly associated with the limited amount of traffic data available to these systems. In the case of SCOOT, adaptive control deterioration might be associated with both a lack of information from the detectors and the inability of the system's algorithm to cope with changed traffic flows. However, timeliness and accuracy of traffic information fed into the system is crucial for all traffic control regimes. Information on traffic flows and how closely they compare with estimated traffic flows has a huge impact on the performance of traffic flows. In other words, the less traffic flows deviate from the expected flows, the better the traffic control.

3.1.1 Variability of Traffic Flows

In general, a traffic control regime can perform suboptimally due to three basic types of changes in traffic flows:

1. *Short term changes* result from sudden fluctuations in travel demand and distribution lasting from several minutes to an hour (for example, a 15-minute

event-driven increase in traffic). They are stochastic and not recursive with any recognizable pattern.

2. *Medium term changes* are mostly associated with diurnal or seasonal changes in traffic flows resulting from various trip purposes, such as increase in traffic demand during a peak period. They are a combination of stochastic and deterministic factors in traffic demand.

3. *Long term changes* depict permanent changes in traffic demand, and distribution and can be modeled as deterministic growth factors applied to base traffic volumes.

These changes, along with their temporal natures, modeling approaches and coping mechanisms, are summarized in Table 3.1.

We can distinguish between the performance losses in traffic control due to traffic fluctuations, diurnal/seasonal traffic variations, and traffic growth based on the classification of traffic flow changes. For each type of traffic change, there is a term representing the ability of a traffic control regime to cope with the changes. Flexibility, responsiveness, and adaptability refer to the system's ability to cope with traffic fluctuations, diurnal/seasonal traffic variations, and growth in traffic,

Table 3.1 – Changes in Traffic Flows and Associated Terms

Term	Traffic Demand	Coping Mechanism	Temporal Nature	Modeling Approach
Short	Fluctuation	Flexibility	Temporary	Stochastic
Medium	Diurnal/Seasonal Change	Responsiveness	Cyclical	Stochastic & Deterministic
Long	Growth	Adaptability	Permanent	Deterministic

respectively. Although these three words can sometimes be used synonymously, there is a bit of logic in associating each term with each type of traffic change.

1. Flexibility represents the ability of the system to cope with short term changes. Although often used synonymously with adaptability, this term assumes a short time in responding to changes and the ability to quickly recover to the previous state.
2. Responsiveness, in this case, represents the ability of the system to cope with diurnal and seasonal traffic variations. When variation occurs, the system is supposed to adjust itself to the new conditions. This adjustment does not have to be as fast as the one associated with the traffic fluctuations. A responsive system should be able to recover to the previous conditions with the same slower speed. The term "responsiveness" is used for medium term changes because of the traffic-responsive signal systems that choose plans based on pattern matching.
3. Adaptability represents the system's ability to adjust to long term traffic changes. This term is often used as a synonym for flexibility but it does not involve any underlying meaning of the quickness of this process.

Ageing in this study is defined as the inability of a traffic control regime to adapt to long term changes. Similarly, a traffic control system is inflexible if it does not adjust to traffic fluctuations. Finally, a traffic control system is irresponsive if it does not respond to medium term traffic variations. It seems that flexibility and responsiveness can accommodate for some portion of ageing, but not all of it.

The term “ageing” is synonymous with deterioration or degradation over time. However, ageing does not assume any reversibility. For example, a traffic signal system can fail to perform optimally the day after it is installed because of high traffic fluctuations. However, this does not mean the system has aged. If the system were again able to perform optimally, then the earlier performance loss would be attributed to the system’s unresponsiveness to varying traffic. However, if volumes were to increase so that, regardless of traffic fluctuations and variations, two years later the system never again reached optimal performance, then the system would have aged. The ageing concept presented here considers only long term changes in traffic flows.

3.1.2 Measuring the Ageing of Traffic Signal Timing Plans

There are various ways to measure the success of traffic control. The major role of traffic control, other than providing safe movement for conflicting traffic streams, is to make the movements as efficient as possible. The number of stops, travel time, delay time, pollutant emissions, and fuel consumption are all measures of the success of the traffic control regime. In general, the goal is to keep all these measures as low as possible. An increase in any of these factors represents a cost to society.

When traffic conditions change, signal timing plans (once optimized to yield to the smallest impedance on traffic) need retiming or updating. Retiming of signal timing plans represents a process in which signal timing parameters are chosen to minimize costs for new traffic conditions. When this process is performed, signal timing plans again become efficient and they improve the performance of the traffic control regime. By measuring the differences in performance of a traffic control regime before and after retiming, one can find the extent of deterioration of the traffic control regime, or, more accurately, the deterioration of the signal timing plan. When this process is performed in the field, it requires measuring or expert judgment of vehicle delays, stops and queues before and after retiming takes place. In this research, performance measures of control regimes are taken from simulation outputs. The following subchapters describe how we deduce deterioration for pretimed, actuated and adaptive traffic regimes.

3.1.2.1 Ageing of Pretimed and Actuated Traffic Control Regimes

Figure 3.1 shows a theoretical representation of the hypothetical relationship between traffic control performance and change in traffic flows. Its purpose is to represent the relationship between the set of factors that influence deterioration of traffic signal regimes. It does not show actual results. This relationship can be applied to single intersections and urban arterials, or networks. The X and Y axes represent changes in traffic flows and a traffic control Performance Index (PI), respectively. Point A in the figure represents the PI associated with an optimal timing plan during a peak traffic period for base traffic conditions (Base Flows). If the traffic flows increase or decrease, the PI, which is used as an objective function in the optimization process, is supposed to increase or remain unchanged (if there is more than one optimal PI).

If the traffic flows increase from the base traffic flows to the traffic flows shown as ?8, the PI changes for two reasons. First, the existing timing plan becomes obsolete and the PI increases by amount D2. Second, the ability of the road network

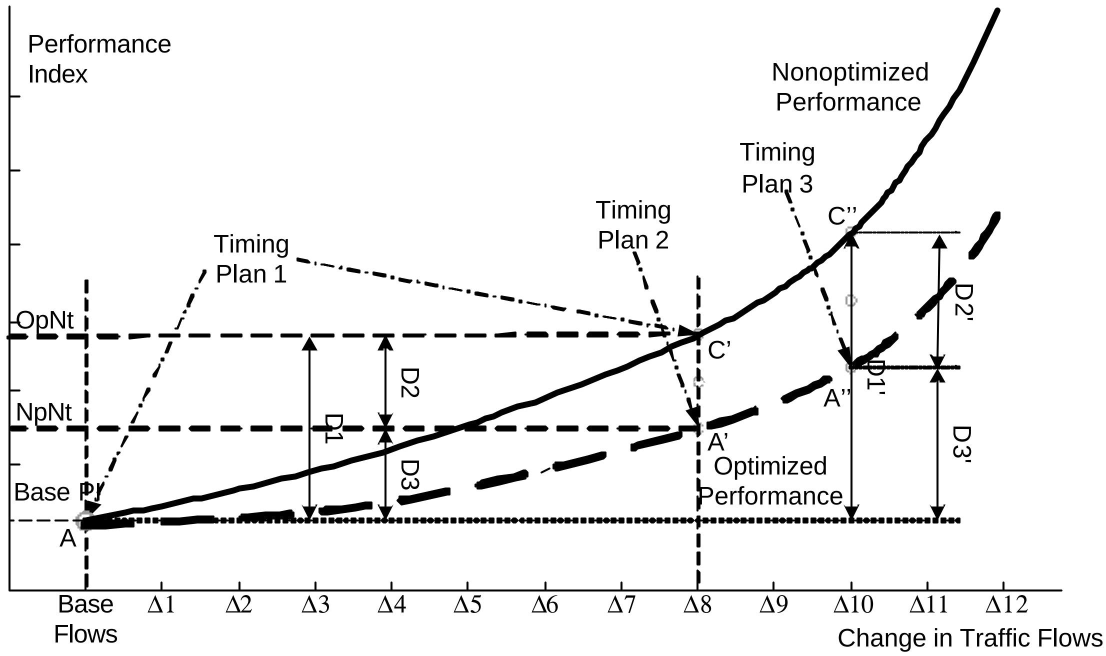


Figure 3.1 - Impact of the Traffic Changes on the Nonadaptive Traffic Controls

and traffic control system to cope with traffic demands decreases with the increase in traffic flows by amount D3. These two processes work in the same direction when traffic flows increase. However, if the traffic flows decrease, then the increase in PI due to a suboptimal timing plan could be countered by the positive effects of the decrease in traffic demand. Point C' represents the PI with existing timing plans and increased traffic flow. The PI produced for these conditions is labeled OpNt – Old Plan New Traffic. Had the timing plan been optimized for new traffic, its PI would be called NpNt – New Plan New Traffic. These conditions are represented by point A'.

The difference between OpNt and NpNt is labeled D2 and represents an increase in PI, after a certain change in traffic flow, due to "suboptimal" signal performance. Similarly, the difference between NpNt and Base PI (D3) represents an increase in PI due to limitations in intersection (or network) capacity. This portion of

PI increase will exist, with an increase in traffic flows, regardless of whether the timing plan is optimal. Finally, the total increase in PI (or total system degradation after the change in traffic flow ?8) is represented as D1, which is the difference between OpNt and Base PI.

The difference D2 can be interpreted as a benefit of updating the old timing plan after a certain change in traffic conditions. Therefore, the percentage benefit of updating a timing plan (ß) can be shown by:

$$b = \frac{D2}{OpNt} \times 100\ \% \qquad [3.1]$$

Where:

D2 = Measure of the benefits (reduction in PI) obtained by updating the timing plan

OpNt = The PI of the old timing plan after a certain change in traffic flow (?8)

Similarly, the percentage disbenefit of not updating a timing plan (d) can be calculated as:

$$d = \frac{D2}{NpNt} \times 100\ \% \qquad [3.2]$$

where NpNt represents the PI of the new timing plan after a certain change in traffic flow. These disbenefits, or opportunity costs, represent the loss associated with not updating signal timing plans.

3.1.2.2 Ageing of Adaptive Traffic Control Systems

Figure 3.2 shows a hypothetical prediction of the ageing process for both pretimed and adaptive traffic controls. Pretimed traffic and actuated control timing plans are optimized in such a way that the optimal PI is achieved for the base traffic state (Base Flow). If the traffic flow increases or decreases, the PI may increase. Point B in Figure 3.2 represents the PI for the same traffic characteristics when an adaptive control regime controls the traffic. The notation "Base ATCS PI" represents ATCS performance (PI) for base traffic flow. Point B can be either below or above point A.

The previous section shows how to assess the measure of the ageing of pretimed and actuated traffic control regimes (D2). The system performance is measured both before and after updating the signal timing plan to estimate the decrease in system performance due to the obsolescence of the timing plan.

A similar procedure cannot be used to assess the ageing of an ATCS when the ageing results from any type of traffic flow changes. This is because the ATCS gradually changes signal timing plans by responding to changes in traffic flow. In other words, there is only a single curve for ATCS performance in Figure 3.2. For this reason, it is not possible, as with pretimed and actuated controls, to measure D2 after introducing a new timing plan. Instead, one can measure only the total deterioration of the traffic control regime expressed as D1a.

D1a (total increase in PI for ATCS) includes both the potential deterioration of the ATCS control regime and the road network's inability to cope with changes in

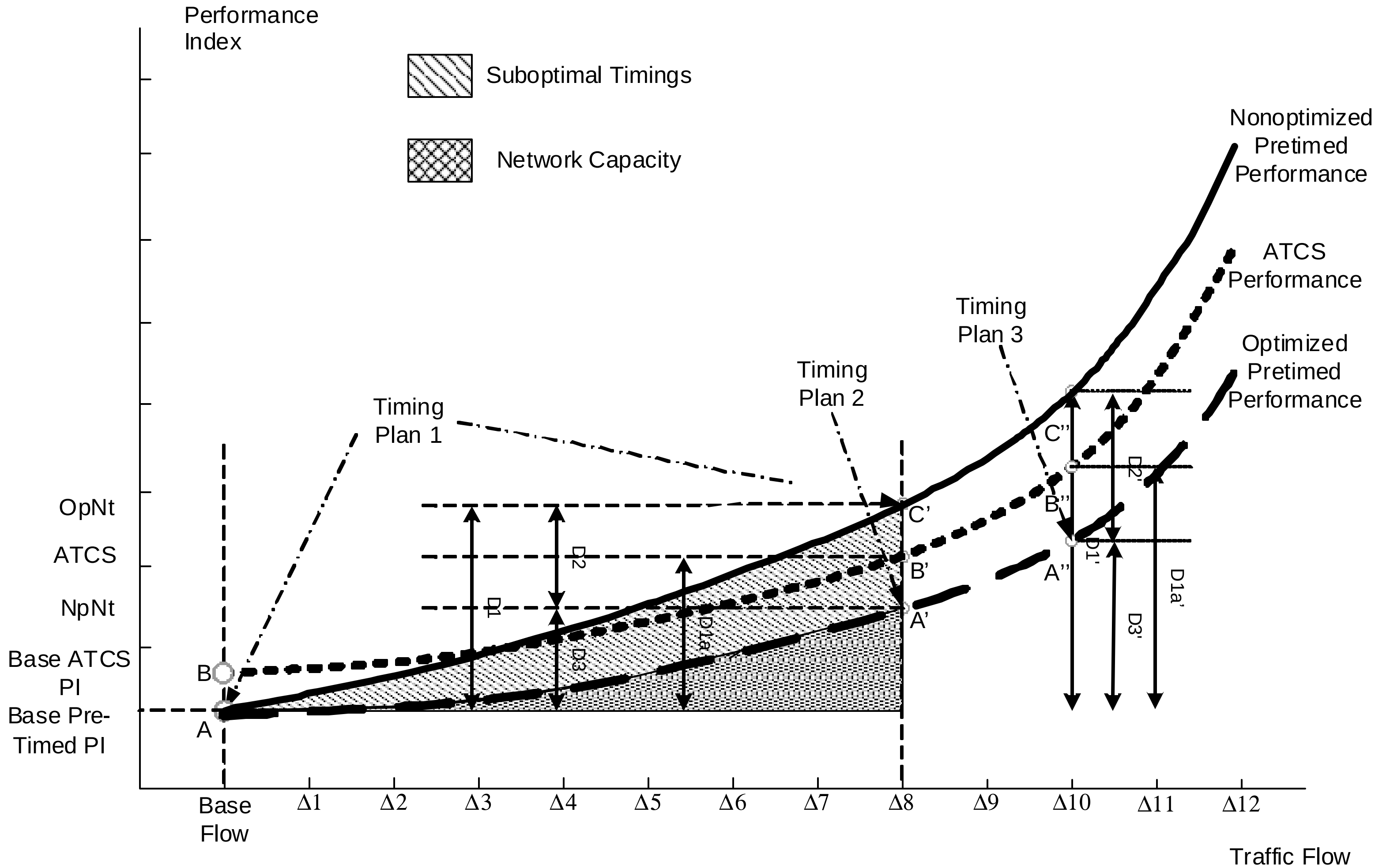


Figure 3.2 – Ageing of the Traffic Control Regimes with Change in Traffic Flow

traffic. For the sake of presenting the concept, the ATCS performance curve is placed between optimized and nonoptimized pretimed performance curves (points B, B', and B''). To assess the ageing of an ATCS, one would have to analyze the difference between D1a and D3.

Ideally, the ATCS performance curve should overlie the optimized pretimed performance curve and the difference between D1a and D3 should be zero. However, this is not likely to be the case for actual ATCS implementations. Optimized pretimed (or actuated) timing plans are likely to perform better than any ATCS for projected constant traffic demand and distribution. In other words, if traffic demand and distribution do not vary from the values used to optimize signal timing plans, optimized pretimed or actuated timing plans will likely perform better than any adaptive traffic control. Therefore, ATCS performance is likely to be represented by a curve (in Figure 3.2), which is above the hypothetical optimized performance curve for pretimed signals. An assumption used here is that traffic flows remain constant throughout the period of analysis.

The major challenge is to recognize a trend in ATCS performance which would imply an influence of changes in traffic flow on ATCS performance. The general idea is to look for differences between D1a and D3 to detect any deterioration trend in ATCS performance.

The difference between ATCS PI and PI of pretimed control is equal to B-A for base traffic flows. If ATCS does not deteriorate at all, this difference should remain constant or decrease for any further change in traffic flow. One should not forget that performance of any of the traffic control regimes will worsen with an increase in traffic flow. However, even if the ATCS is worse than pretimed performance for all ranges in traffic flows, it should keep pace with optimized pretimed signal timing plans.

Therefore, if the difference between ATCS performance and pretimed performance does not increase with changes in traffic flows, one can say that the ATCS does not deteriorate. Otherwise, it can be said that ATCS ages with changes in traffic flows. Mathematically, if we assume that D1a is greater than D3, the following inequality shows a condition for the existence of the ATCS ageing:

$$(D_{1a} - D_3) \quad < \quad (D_{1a}' - D_3') \qquad [3.3]$$

Or

$$\frac{(D_{1a} - D_3)}{(D_{1a}' - D_3')} \quad < \quad 1 \qquad [3.4]$$

In other words, if the difference between these two measures increases over time, we can say that an ATCS deteriorates or ages. If the ratio between (D1a-D3) and (D1a'-D3') is one, then the ATCS does not age. However, this ratio should never be greater than one because that would mean that ATCS improve performance with increase in traffic flow. In Figure 3.2, $D_{1a}' - D_3' = B'' - A''$ represents the difference for changed traffic flows for Δ10, while $D_{1a} - D_3 = B' - A'$ is the difference for Δ8. Since the former term is greater than the latter, we could say, if the results were obtained from the simulation, that the ATCS ages. If there is ageing of ATCS, the percentage of ATCS ageing (?) can be expressed as:

$$g = \left(1 - \frac{(D_{1a} - D_3)}{(D_{1a}' - D_3')}\right) \times 100\,\% \qquad [3.5]$$

In general, the entire process of determining ? requires many traffic simulations with controlled traffic flow inputs. This enables estimation of the aforementioned measures D1, D2, D3, and D1a. The analysis of the measures requires statistical testing for the difference between the measures. Once these measures are estimated, it is possible to assess the potential deterioration of ATCS. The situation presented here is the simplest case. If the differences between ATCS and pretimed PIs vary, additional tools can be necessary to characterize the ageing process. The methodology for estimating the ageing of ATCS presented in this section is original and represents one of the contributions of the dissertation.

3.1.3 Selecting a Performance Measure

There are various ways to measure the performance of traffic control regimes. The number of stops, travel time, delay time, pollutant emissions, and fuel consumption are all traffic control regime measures. In general, the goal is to minimize these measures. An increase in any of these measures represents a cost to society. The PI from Synchro optimization software serves as the performance measure. The PI is a composite measure of delay time and stops, which are two of the most important MOEs for undersaturated traffic conditions (Husch and Albeck 2003 (I)). It represents the objective function in Synchro optimization and takes the form:

$$PI = D + 10 \cdot \frac{S}{3600} \qquad [3.6]$$

Where:

D = Total delay (hours) per hour

S = Total number of stops per hour

10 = Weighting factor for stops

In order to make sure that the pretimed curves in Figures 3.1 and 3.2 represents the best timing plans possible, the performance measure used as an objective during the optimization of the timing plans must also be used to assess deterioration of those plans. Any other measure would introduce uncertainty about the quality of the optimized timing plans.

However, the PI used in SYNCHRO (a similar PI is used in TRANSYT-7F) has some weaknesses. One of the major weaknesses results from the inability of some simulation programs to account for delay and stops of those vehicles that did not leave the network by the end of a simulation run. Another similar problem comes from the fact that most types of simulation software do not account for the delays experienced by vehicles that are not able to enter the network (due to congested traffic conditions at the entry point). For this purpose, two additional measures of effectiveness (MOEs) are gathered for each simulation run. The first measure is network throughput. This measure represents a ratio between the number of exited vehicles and the number of entered vehicles. The second measure is input ratio. This measure is a ratio between the actual number of entering vehicles and the number of vehicles intended to enter. These two ratios are usually around 99-100% percent. However, when traffic conditions are saturated, more vehicles are locked in the network by the end of the simulation and these ratios drop by a few percentages. In such situations, these ratios are reported and the reliability of the PI is questioned.

3.2 Design of the Simulation Experiments

3.2.1 Experimental Grid Network

The design of the experiments was motivated by a need to set up a highly experimental approach. There were several requirements that had to be satisfied for the experimental network:

- The network was supposed to be like a real-world network with a complex geometrical layout. A single arterial with 4-5 intersections would not challenge optimization from any optimization software. Deterioration of one (critical) signal at such a network would possibly cause more harm to the performance of the whole system than if the network were simple.
- Traffic flows were supposed to be high enough that the benefits of optimization would be gained either for an increase or a decrease in traffic demand. The same applies for turning movement proportions.
- Traffic flows and turning movement proportions were supposed to be equal (for most of the links) and symmetrical. As such, they represent controllable variables that enable the development of a cause-effect relationship in the postexperimental analysis.
- When selecting a test network for assessing the ageing of signal timing plans, findings are needed that are not associated with particular road and traffic conditions. Case studies can be difficult to generalize.

A grid urban network (the kind found in most urban districts) represents a experimental grid network. Figure 3.3 shows a nine-node experimental grid network with base traffic volumes. All links in the network are between 800 and 1000 feet long, representing intersection spacing common for some urban districts (e.g., Salt Lake City, UT). Two central arterials have three lanes in each direction, while four peripheral arterials have two lanes in each direction. All intersection approaches have 250-foot left turn pockets and 150-foot right turn pockets.

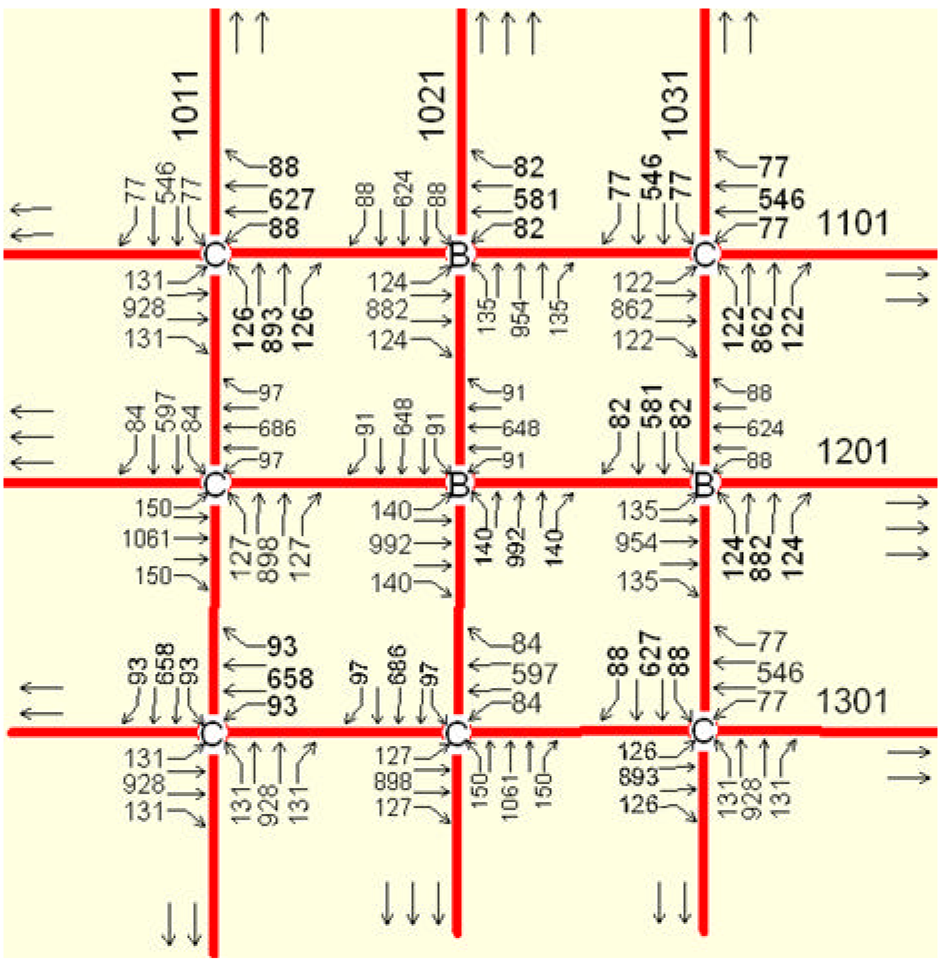


Figure 3.3 - Test Bed Network with Base Traffic Flows (Synchro 6)

Pretimed and actuated coordinated controls are modeled separately. There was no need to model actuated uncoordinated control considering the short spacing between intersections and high coordinatability factors (Husch and Albeck 2003 (I)). For pretimed and SCOOT controls, all intersections have signals with four major phases: two for leading left turn movements and two for major through movements. Actuated-coordinated control deploys eight phases instead of four. Operations of actuated control with only four phases would limit performance of this control type. The phases allow no permitted left turns. Right turns on red are available for all right turn movements.

Base flows at the network entry points are 700 veh/hour for 2-lane arterials and 800 veh/hour for 3-lane arterials. These flows are multiplied by 1.7 for all northbound and eastbound approaches to model directional peak traffic demand. Free flow speeds are 35 mph for 2-lane arterials and 45 mph for 3-lane arterials. Heavy vehicles contribute 2% of the total traffic demand. There are no peaks in demand flows and peak demand is constant during the peak hour. All turning movement proportions in the network are equal for the base traffic conditions. All approaches have 78% through traffic and 11% left and right traffic. This enables the modeling to capture the relationship between changes in turning proportions and ageing of timing plans.

All intersections are assumed to be fully coordinated within the network. The intersection of two 3-lane arterials has a zero offset. However, the base traffic conditions governed one of the peripheral intersections (intersection of 1011 and

1301 arterials in Figure 3.3) to become the critical intersection. There is no pedestrian or transit traffic.

3.2.2 Traffic Demand and Distribution Scenarios

There were two aspects of assessing the ageing of timing plans through simulation: deterministic and stochastic. The deterministic aspect is used for both selection of appropriate simulation tools and assessment of the ageing of traffic control regimes. The stochastic aspect is used only for the latter. The deterministic aspect is introduced here. The stochastic aspect is introduced later in this chapter in the section that describes the major modeling process.

Uniform deterministic traffic demand and distribution used 21 scenarios to emulate the ageing of the link traffic flows. This approach was used to constrain the number of controlled variables and provide explainable relationships between changes in link flows and performance of traffic control regimes.

Two major aspects of changes in link traffic flows are assessed: total traffic demand in the network (traffic flows at the entry points) and distribution (turning movements at the intersections). Twenty-one scenarios were developed to emulate variation in the link traffic flows. The first scenario represents the base traffic conditions. Volumes and Levels Of Service (LOS) are shown in Figure 3.3.

Scenarios 2 to 11 represent changes in traffic demand. Traffic flows at the entry points were decreased and increased by 5, 10, 15, 20, and 25% with respect to base input flows. The turning movements for these 10 scenarios remain unchanged. These scenarios emulate the increase or decrease in total network traffic growth. The same increase and decrease factors are applied to all entry flows in order to capture the influence of deterministic change in traffic demand on the ageing of signal timing plans.

Scenarios 12 to 21 represent the change in distribution of the base traffic flows. The traffic demand for these scenarios remained unchanged. These scenarios were selected to emulate long term changes in turning movement flows at the intersections. All turning movements (left and right) at each intersection were decreased and increased by 10, 20, 40, 60, and 80% with respect to base turning movements.

3.3 Selection of Proper Tools for Assessing the Ageing of Traffic Control

The concept of ageing of pretimed and actuated traffic control regimes is presented in the first part of this chapter. Figure 3.1 illustrates the concept. A crucial assumption is that optimized signal timing plans perform better than old outdated timing plans. However, this is not always the case. Optimized timing plans that are applied in the field or within simulation sometimes perform worse than non-optimized timing plans. A major reason for this contradiction is the inability of the macroscopic optimization tools to model traffic conditions properly. This inability is largely impacted by two factors. The first factor is the quality of the underlying traffic model (or, more precisely, the set of models) used by a macroscopic optimization tool to emulate real-world traffic conditions. The second factor is adjustments to this tool that have to be made to model field traffic conditions as closely as possible. Adjustments of parameters that influence performance of the traffic model are made through calibration and validation. Park et al. (2001) have shown that timings optimized by an optimization tool based on a macroscopic model (TRANSYT-7F) give inconsistent results when tested through micro simulation (CORSIM).

The following chapters introduce the definitions of different traffic analysis tools, their differences and reasons for inconsistencies. A short review of previous studies about microscopic and macroscopic simulation tools is given. Finally, the reliability of macroscopically optimized timing plans used in microscopic simulation models is assessed.

3.3.1 Traffic Analysis Models

The major difference between microscopic and macroscopic traffic models lies in the different approaches they use to model traffic behavior in the real world. The following definitions describe macroscopic, microscopic, and optimization tools.

Macroscopic simulation models are based on the deterministic relationships of flow, speed, and density of the traffic stream. Simulation in these models is done section-by-section rather than for trajectories of individual vehicles. The advantage of macroscopic tools is that they are simpler and require less time to obtain the results of the simulation.

Microscopic simulation models simulate the movement of individual vehicles based on car-following and lane-changing theories. Each vehicle, with pre-determined destination, vehicle type and driver characteristics, is tracked through the network over small time intervals (1 second or less). Simulation results are much more realistic than those from macroscopic models. However, computer time and storage requirements for these models are large, usually limiting the network size and the number of simulations that can be completed.

Traffic signal optimization tools are tools designed to develop optimal signal-phasing and timing plans for isolated intersections, arterial streets, or networks. These tools usually calculate capacity and optimize cycle lengths, splits, offsets, and phase orders for traffic signals.

Optimization processes are complex and involve several parameters. Cycle lengths, splits and offsets are all interconnected and they all have to be considered during the optimization. This increases the number of possible alternatives to test for given traffic conditions. This largely increases the time necessary to perform optimization. The time necessary to test one signal timing alternative (cycle length, splits, offsets) is a critical component of the optimization process. For this reason, optimization tools use macroscopic models to evaluate the effectiveness of various signal timing alternatives. When there is a trade-off between accuracy (microscopic models) and efficiency (macroscopic models), optimization tools generally choose the latter.

3.3.2 Review of Past Research

Investigation of the reliability of macroscopically optimized timing plans in microscopic environments has drawn little attention. Park et al. (2001) evaluated the reliability of TRANSYT-7F optimization schemes for both uncongested and congested conditions. They compared TRANSYT-7F’s MOEs to those resulting from CORSIM’s output using the same TRANSYT-7F timing plans. This is the first explicit approach of assessing macroscopically optimized timing plans within a microsimulation environment. Comparing the performance measures from TRANSYT-7F and CORSIM, they reported a high correlation of MOEs for the uncongested condition and a low correlation for the congested condition. In both cases, the performance measures for the macroscopic and microscopic models were quite different. They associated these differences to a low fidelity traffic model within TRANSYT-7F.

Rouphail et al. (2004) recognized the need for a better optimization tool to evaluate performance measures for microscopic simulations. They developed direct CORSIM optimization, a type of stochastic optimization, using the Genetic Algorithm (GA) method for optimization of performance measures. The findings showed that direct optimization consistently provided better MOEs than TRANSYT-7F strategies, even though the best TRANSYT-7F strategies were selected based on their performance in CORSIM, not in TRANSYT-7F.

Park et al. (2004) provided a more comprehensive assessment of the direct GA optimization within the CORSIM micro simulation program. This time not only TRANSYT-7F optimization schemes were compared with GA optimization, but the performance of the GA was investigated for changes in traffic demand. Two types of change were investigated. First, the mean rates of demand for the network entry points were changed randomly to ±15% from the base demand. This approach was used to account for changes in mean demand rates since CORSIM always produces demand distributions around predetermined mean rates of demand. Second, a 10% increase over the entire network was modeled to account for systematic changes in travel demand. This approach emulated the ageing of traffic signals. No explicit modeling beyond CORSIM's own stochastic variability was made to simulate changes in turning movement proportions at the intersections. This study also found that GA optimization is better than TRANSYT-7F in the CORSIM environment and performs similarly to TRANSYT-7F in the TRANSYT-7F environment. The results showed that GA plans are more reliable for the changes in mean rates of traffic demand.

Lee et al. (2004) investigated the performance of CORSIM, SimTraffic, and Synchro 4 for application to diamond interchanges for three performance measures: cycle length, average delay, and total stops. The research showed that the two microscopic models performed similarly, but were different from the macroscopic program Synchro. The study also found that SimTraffic modeled conditions more restrictively than the other two tools in some situations.

In summary, studies have compared the performance of micro- and macro-simulation tools. Discrepancies between micro- and macrosimulation tools for the same timing plans have been recognized. There follows a comprehensive evaluation of the reliability of the most popular signal optimization tools using the most popular micro simulation tools. The objective of this portion of the research is to investigate the reliability of timing plans optimized by Synchro and TRANSYT-7F in the micro-simulation environments of CORSIM, SimTraffic, and VISSIM.

3.3.3 Basic Review of the Macro- and Microsimulation Tools Used

According to Tarnoff and Ordonez (2004), TRANSYT-7F and Synchro are the most popular traffic control optimization tools among traffic signal practitioners in the U.S. For this research, the author investigated the reliability of timing plans derived by Synchro and TRANSYT-7F, which are tested by their implementation in stochastic microsimulation.

The same survey (Tarnoff and Ordonez 2004) recognized SimTraffic and CORSIM as two of the most popular micro simulation tools. Tian et al. (2002) acknowledged CORSIM, SimTraffic and VISSIM as three of the most popular types of microsimulation software in the U.S.

There have been numerous studies comparing the performance outputs from various macro and micro simulation tools (Park et al. 2001, Lee et al. 2004, Tian et al. 2002, Bloomberg and Dale 2000, Mystkowski and Khan 1999, and Trueblood 2005). Various micro- and macrosimulation tools rely on different traffic behavioral models. They also use different ways to report MOEs from their models. Consequently, they also report delays and stops in different ways. The following paragraphs provide the most essential descriptions for each traffic simulation model. The focus is on delays and stops produced by each tool as the major elements of the PI.

Synchro (Version 6) is a macroscopic, deterministic simulation and optimization model used mostly for the optimization of coordinated and uncoordinated traffic signals. Synchro computes total delay (used as part of the optimization objective) as a sum of the traditional control delay and blocking delay

(Husch and Albeck 2003 (I)). The delay is estimated based on queue polygon with multiple percentile volume scenarios. Synchro calculates the percentage of stopped vehicles based on TRANSYT-7F's relationship between vehicle delays and percent of stops (Husch and Albeck 2003 (I)). Lee et al. (2004) state that Synchro considers a vehicle stopped when its speed is less than 3 m/s (10 ft/s).

TRANSYT-7F (Version 10) is also a macroscopic, deterministic simulation and optimization model. Version 7F is an American version based on the original British software. The TRANSYT traffic model is known as one of the best macroscopic traffic models (Hale 2005). TRANSYT-7F computes total delay as a sum of total uniform and random delays, well-known concepts of delay from analytical models. Total number of stops in TRANSYT-7F is calculated as a sum of the stops for different delay types. Number of stops for each delay type is based on the delay-stop look up table.

Delay is usually defined as the entire amount of time spent not traveling at the prevailing cruise speed. Although Synchro and TRANSYT-7F can estimate these delays, they cannot explicitly trace the trajectories of individual vehicles to obtain accurate delays. Similarly, the numbers of stops they estimate are only an approximation of the "real" number of stops.

CORSIM (Version 5.0) is a stochastic microscopic simulation program for urban traffic developed by the FHWA (1999). CORSIM reports delays and stops by link. Delay time per vehicle is defined as an average delay on a link for each vehicle, calculated by taking the total delay of all vehicles that have traversed the link and dividing it by the number of vehicles. Stop percentage is defined as a ratio of the number of vehicles that have stopped at least once on a link to the total number of vehicles on the link. A vehicle is considered to be stopped when its speed is less than 3 feet/second (FHWA 1999).

SimTraffic (Version 6) is a stochastic and microscopic simulation program that is closely tied to Synchro. SimTraffic incorporates the vehicle and driver parameters developed by the FHWA for use in traffic modeling (Husch and Albeck 2003 (II)). These parameters are basically the same as CORSIM's parameters, with a few exceptions (Husch and Albeck 2003 (II)). SimTraffic calculates total delay as the sum of the ratios between actual vehicle slowdown (the difference between maximum link speed and actual speed) and maximum link speed for all time slices of 0.1 sec. Total delay also includes all time spent by denied entry vehicles while they are waiting to enter the network. SimTraffic adds a stop to the total number of stops whenever a vehicle's speed drops below 10 ft/s (3 m/s). Several studies (Lee et al. 2004, Trueblood 2005, Husch and Albeck 2003 (II)) have found that CORSIM and SimTraffic produce comparable performance outputs when calibrated properly.

VISSIM (Version 4.10) is another stochastic and microscopic simulation that has recently been widely accepted in the U.S. Delay is measured in VISSIM as the difference between free flow travel time and actual travel time on the user-defined segments. These segments can be automatically created for the intersection delay statistics or for delay on the entire network. VISSIM defines the number of stops within the queue as the total number of events when a vehicle enters the queue condition. The queue conditions are adjustable and the default value is 4.55 ft/s (1.4 m/s) (PTV AG 2005).

3.3.4 Optimization and Simulation in Macro- and Microsimulation Tools

Figure 3.4 shows a simplified flowchart of major objects and processes for the evaluation of optimization schemes in a microsimulation environment. Two different approaches are examined in this research. In the first, the reliability of timing plans from two macroscopic optimization tools is examined within three microscopic simulations. This type of optimization is termed Indirect Optimization. In the second approach, candidate plans from TRANSYT-7F's GA optimization procedure are evaluated through Direct CORSIM optimization.

First, the same 21 link flow scenarios, designed for assessing the ageing of traffic control regimes, are used to evaluate the performance of the optimized timing plans within micro and macro simulation tools. These scenarios are modeled in MS Excel and account for various traffic demands and distributions. An Excel file is designed to enable the selection of traffic demand and distribution, balancing of link flows, and preprocessing output for peak hour volume data in a "*.csv" file with the Universal Traffic Data Format (UTDF). The UTDF is a standard specification for transferring data between various software packages (Husch and Albeck 2003(I)).

In the next step, the preprocessed volume data were imported into the base Synchro file. When new volumes are imported into the old Synchro file (timings are not changed), this corresponds to a situation in which signal timings are not adjusted to the volumes: namely, the ageing of the timing plans. This type of Synchro file was saved for further analysis as a Non Optimized (NO) file. Then, the optimization process within Synchro was executed.

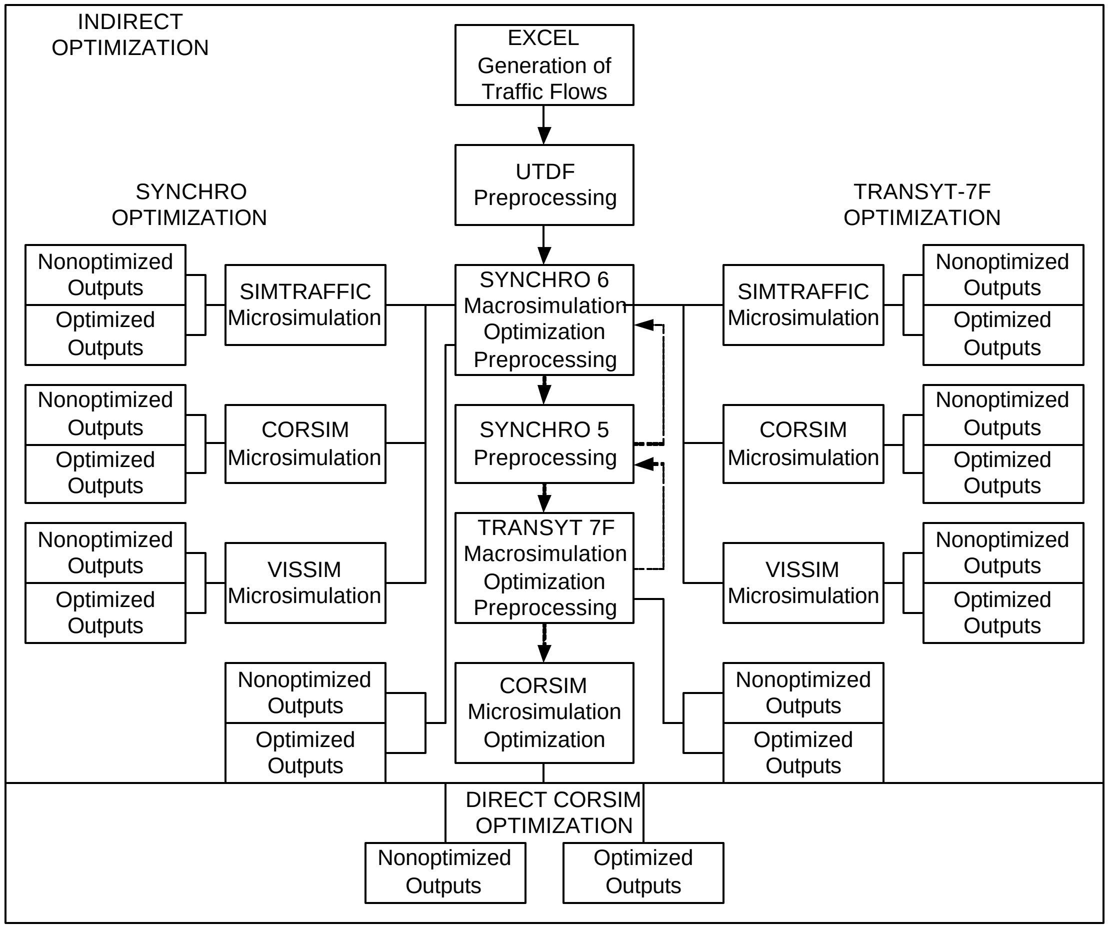


Figure 3.4 -Method to Assess Reliability of Timing Plans in Simulation Tools

The Optimized (OP) files were also saved for further analysis. This process was repeated for all 21 scenarios. Two Synchro files were developed (NO and OP) for all scenarios except the base scenario. Optimization in Synchro was done under the following constraints:

- Range for cycle lengths: 60 to 140 sec, with 1 sec increments
- Never allow uncoordinated signals
- No double cycling
- Extensive offset optimization

- No lead/lag phasing optimization
- Search for the best timing plans
- 60-minute analysis period

The next step was to export the OP and NO Synchro files into micro-simulation tools. Powerful Synchro transfer features were used to accomplish this. Synchro 6 has a direct connection to SimTraffic 6, its microsimulation counterpart. Synchro 6 also offers simple data transfer to CORSIM software. The transfer of data to VISSIM had to go through a series of UTDF files (volume, timings, phasing, lanes, layout). VISSIM uses a TEAPAC preprocessing tool called PRESYNCHRO to read the UTDF files and to build its own network with all relevant volumes, timings, and phasing data.

Although Synchro 6 was used as a major transfer tool between macro- and microsimulation tools, its previous version, Synchro 5, was used to export-import to TRANSYT-7F. Synchro 6 does not support export to TRANSYT-7F. Therefore, NO Synchro files were first saved as Synchro 5 files and then exported to TRANSYT-7F. These files were then used for macroscopic TRANSYT-7F analysis and optimization. The first new set of OP files was generated from the TRANSYT-7F optimization. Then these files with TRANSYT-7F signal timings were imported back into Synchro 5. Once again, the new Synchro 5 (TRANSYT-7F optimized) files were saved as Synchro 6 files and they were ready for new transfers to the three micro simulation tools. The TRANSYT-7F optimization was done based on input parameters from the Synchro 5 files, so its limitations were inherently the same as the listed Synchro 6 optimization constraints.

For the evaluation of the Direct CORSIM optimization, the GA procedure within TRANSYT-7F provides candidate plans to CORSIM. CORSIM executes runs with the GA plans and delivers performance measures. These, in turn, are used to generate new timing plan candidates through the GA procedure (Hale 2005). The entire optimization process continues until convergence, or until a defined number of iterations is complete. This is a new feature of TRANSYT-7F and its authors claim that it is the best way to optimize timings for CORSIM performance. This optimization is used in this study to investigate and compare its performance with a traditional indirect-optimization approach.

For the Direct CORSIM optimization, the base-case timings for two initial optimizations were imported into the other 20 CORSIM files describing 20 scenarios. These files were saved as NO CORSIM files. Each of the 20 files was then optimized twice using Direct CORSIM optimization. GA optimization performance largely depends on the initial timing plans (Hale 2005). For this reason, the authors have executed optimization for two initial timing plans. The first was the TRANSYT-7F macroscopically optimized timing plan through the Hill Climbing procedure. The second was the optimized timing plan through the Direct CORSIM optimization.

3.3.5 Simulation Runs

Ten simulations were performed for OP and NO Synchro 6 files within each microsimulation software for each scenario. The same was repeated for OP and NO TRANSYT-7F files. Each microsimulation was an hour long with 5 minutes of seeding time. The 10 runs for each scenario used the same starting random seed numbers for the corresponding microsimulations. Ten runs have been shown to be enough for any of the microsimulation tools if the traffic demand (based on the HCM V/C ratio) does not exceed or come close to 1.0 (Tian et al. 2002). Since all three simulation tools had V/C ratios of less than one for most scenarios, they capture reliable performance measures from the microsimulation outputs. In total, more than 2460 one-hour simulation runs were accomplished for the evaluation of indirect optimization.

Forty optimization runs were conducted for the Direct CORSIM optimization: two for each scenario based on two initial timing plans. Each run took about 6-7 hours. The parameters used in the GA optimizations were:

- Cycle length, splits and offsets were optimized (no phasing optimization)
- Lower cycle length was 60 sec, upper cycle length was 90 sec, original cycle length was either 60 sec (optimal base cycle length from Direct CORSIM optimization) or 66 sec (optimal base cycle length from TRANSYT-7F Hill Climbing optimization)
- The Optimization objective function was Total Delay
- All default values were used for GA parameters, except for maximum number of generation which was increased from 20 to 100 and population size, which was increased from 10 to 20

The next step was to check the reliability of timing plans optimized through Direct CORSIM optimization. This was necessary because Direct CORSIM optimization evaluated alternative signal timings for only one simulation random seed number. However, simulation results are valid only if they hold for several random seed numbers of the same simulation.

For each scenario, 100 one-hour CORSIM runs were performed for three cases: NO timing plans and two sets of OP timing plans with different initial timings. In total, 6,100 one-hour CORSIM runs were performed.

3.3.6 Results of the Assessment of Simulation Tools

The following subsections present the results of the evaluation of macroscopically and microscopically (Direct CORSIM) optimized timing plans when used in microsimulation. The results are divided into two sections: uniform changes in traffic demand and uniform changes in traffic distribution.

3.3.6.1 Uniform Changes in Traffic Demand

Figures 3.5 – 3.12 show changes in PI impacted by uniform changes in traffic demand. Each of the macro- and microsimulation outputs are presented for OP and NO signal timings for all 10 scenarios. In addition to PI, optimized and non-optimized Cycle Lengths (CL) are shown. Cycle lengths are optimized, along with other signal timing parameters, either in Synchro or in Transyt-7F. Although these two optimization tools produce similar cycle lengths most of the time, sometimes these cycle lengths differ significantly. Figure 3.5 compares the changes in PI for Synchro and TRANSYT-7F. Figure 3.6 shows OP and NO PIs for Direct CORSIM optimization. The next six figures (Figure 3.7 to Figure 3.12) show the changes in PIs for the microsimulation runs and the macro optimization tool used to optimize signal timings for those simulation runs.

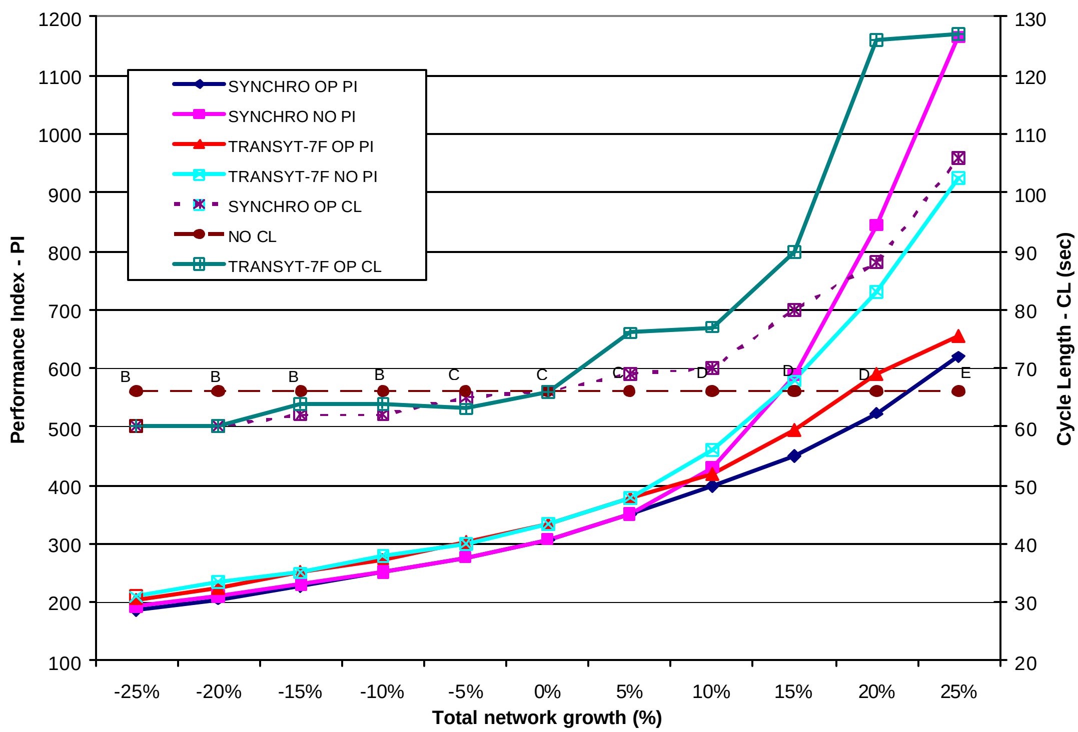


Figure 3.5 - PI and CL vs. Network Growth for Synchro and Transyt-7F

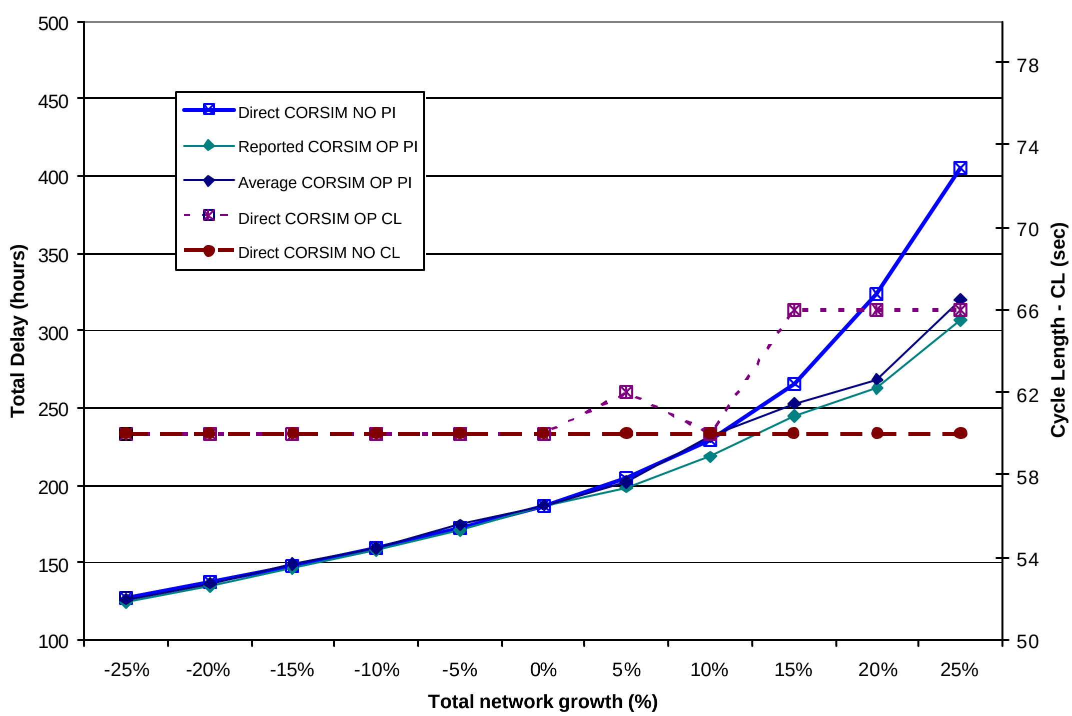


Figure 3.6 - Total Delay vs. Network Growth for Direct CORSIM Optimization

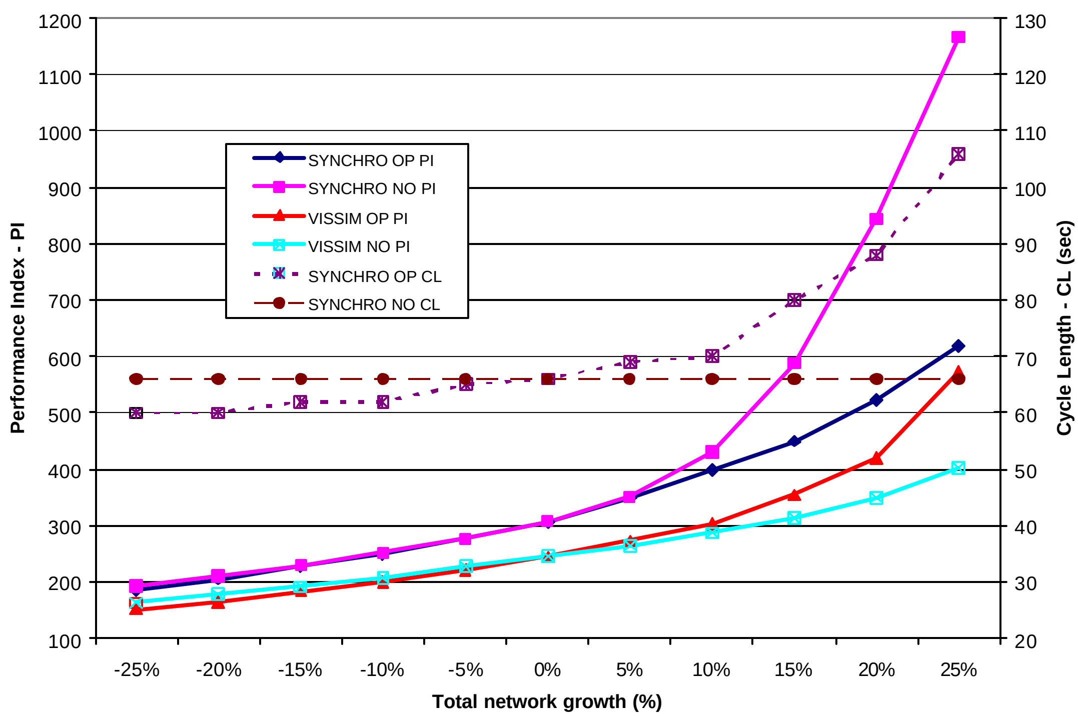


Figure 3.7 - PI and CL vs. Network Growth for VISSIM and Synchro

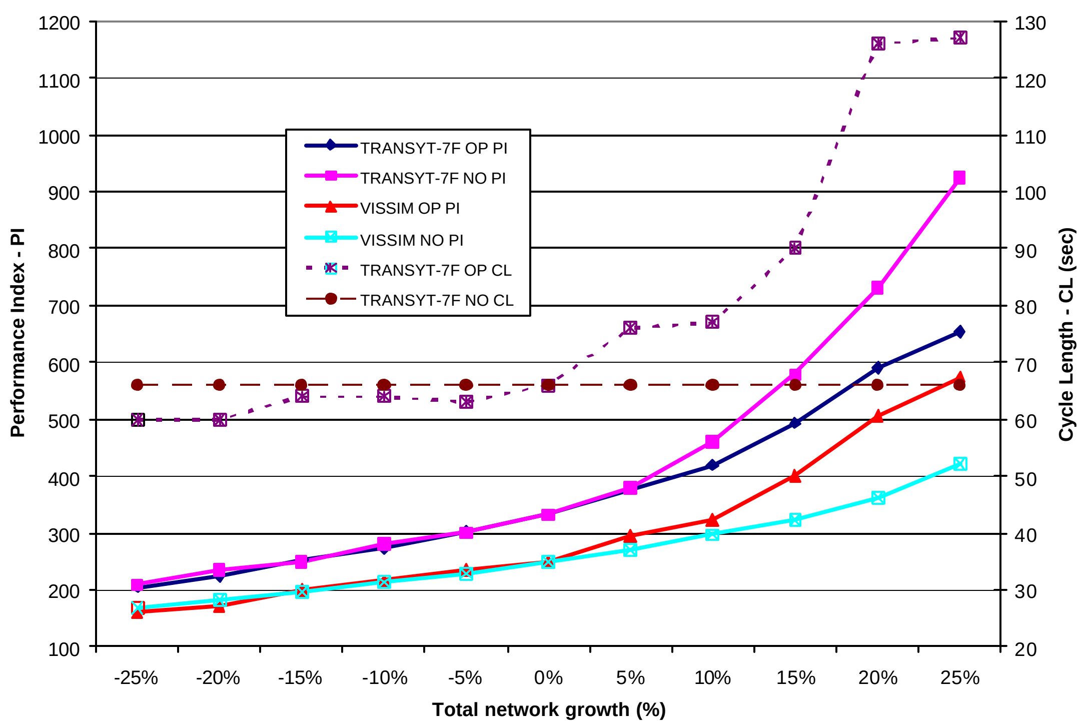


Figure 3.8 – PI and CL vs. Network Growth for VISSIM and Transyt-7F

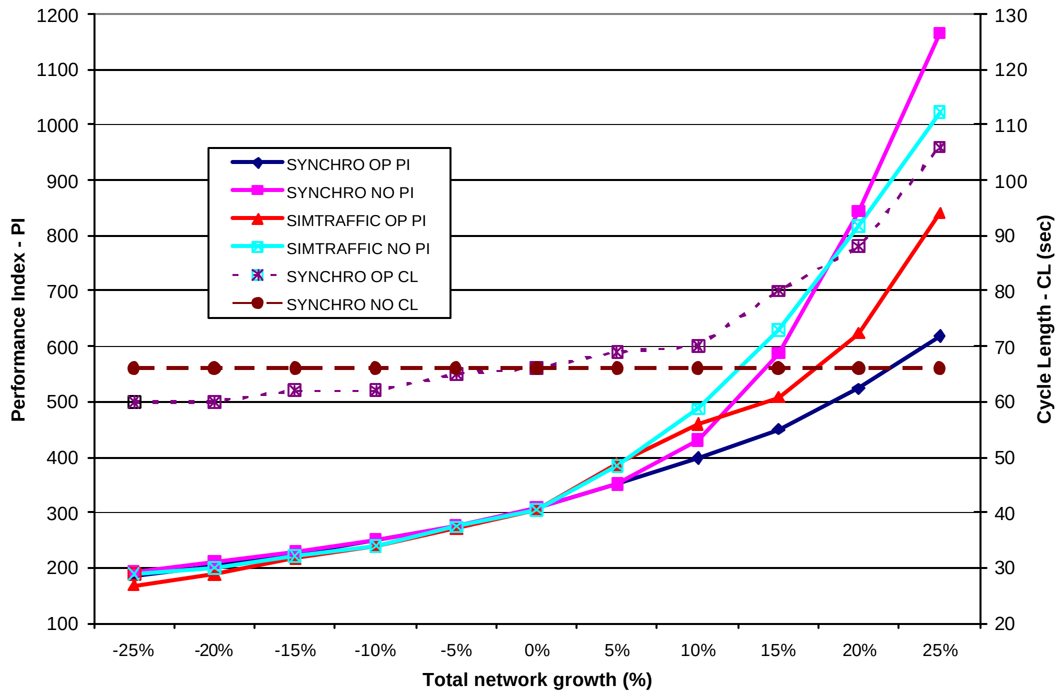


Figure 3.9 - PI and CL vs. Network Growth for SimTraffic and Synchro

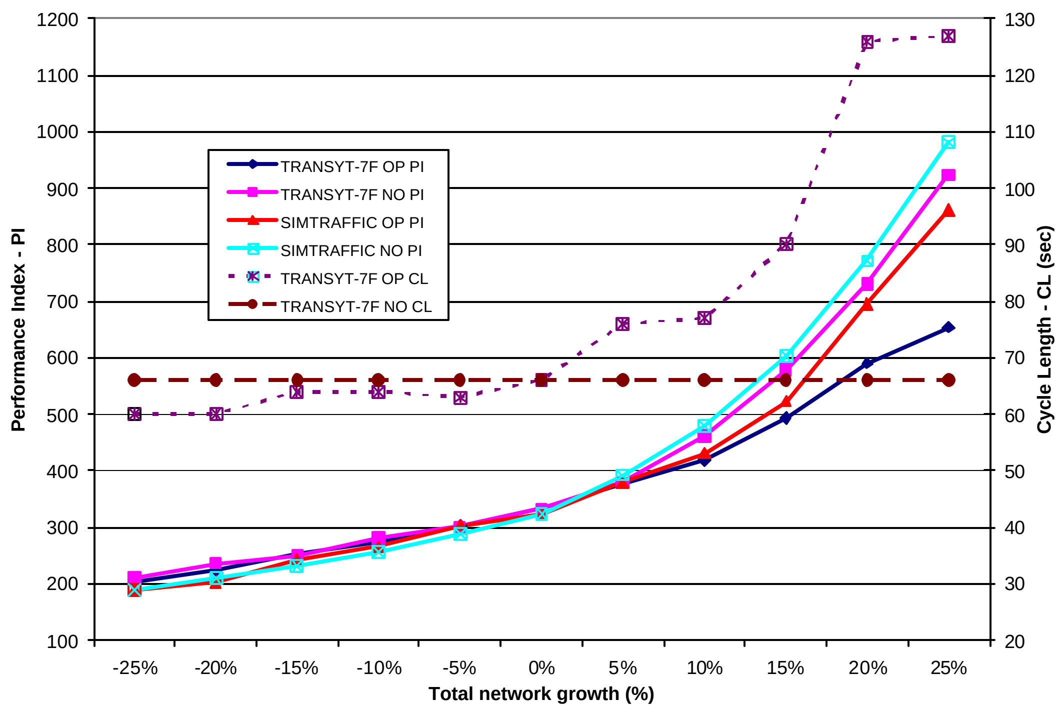


Figure 3.10 - PI and CL vs. Network Growth for SimTraffic and Transyt-7F

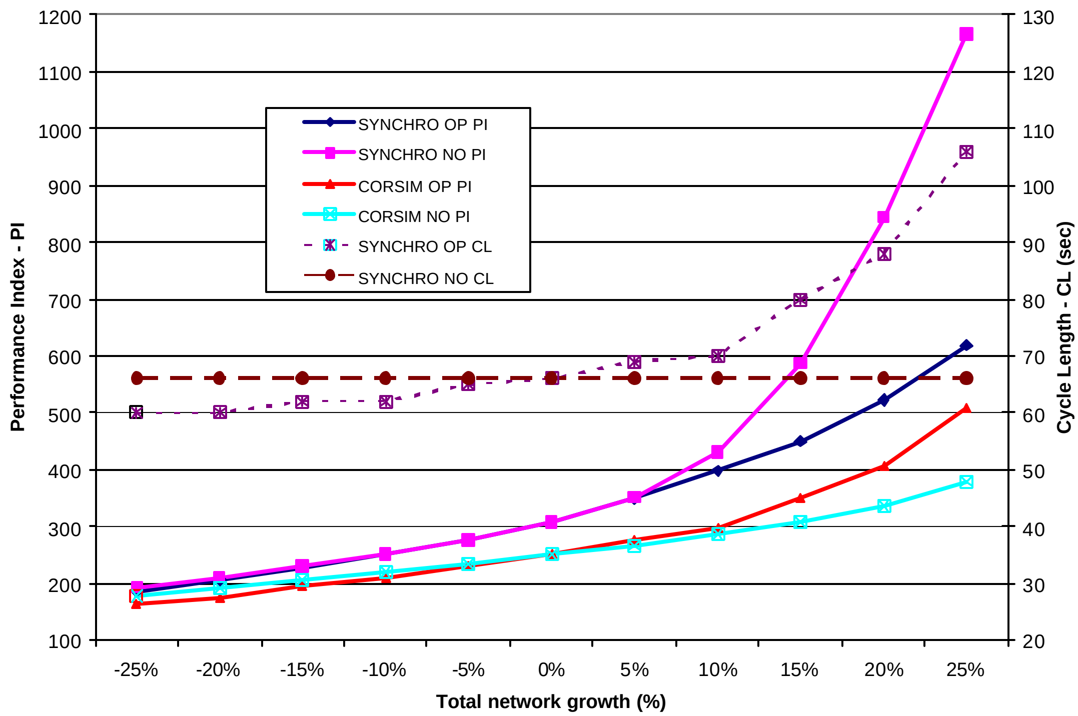


Figure 3.11 - PI and CL vs. Network Growth for Corsim and Synchro

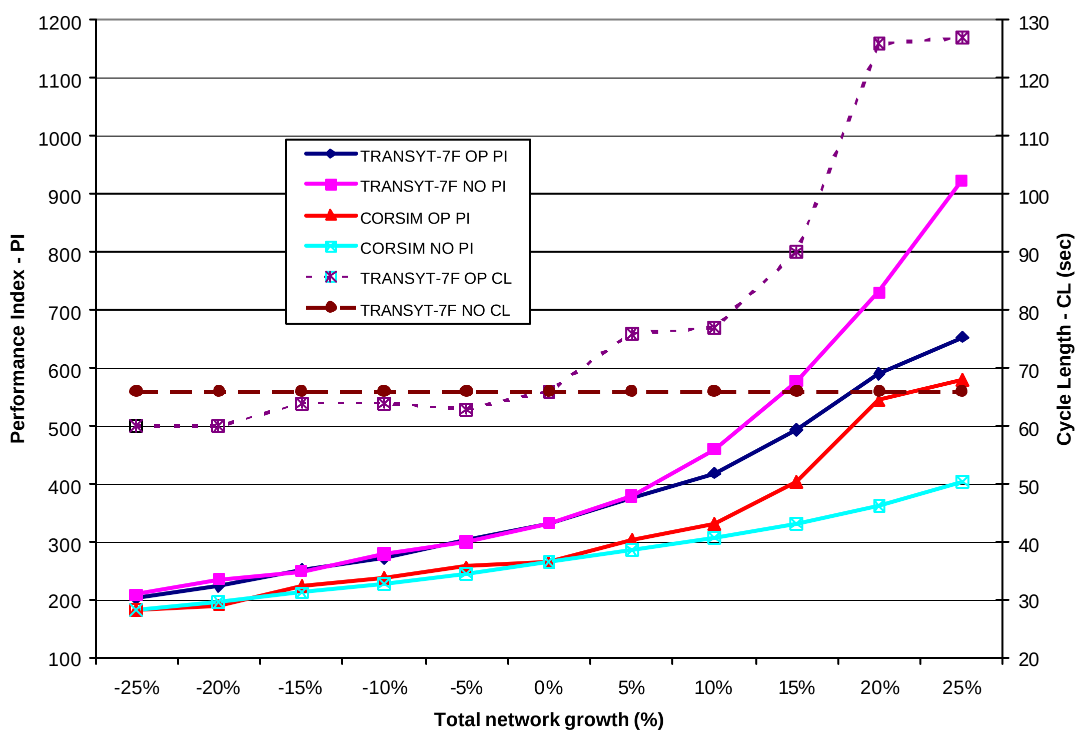


Figure 3.12 - PI and CL vs. Network Growth for Corsim and Transyt-7F

The macroscopically obtained PIs from Figure 3.5 are similar. Both Synchro and TRANSYT-7F OP timings yield lower PIs than the base NO timings. However, the difference between OP and NO timings is much higher as demand grows.

There is a correlation between traffic demand and cycle length. This is because the cycle length is a timing parameter that is used to increase throughput in the network. When traffic demand increases, increasing the throughput is often the only way to deal with congestion.

The results of the Direct CORSIM optimization are shown in Figure 3.6. The optimal solutions are based on total delay (hours) because the PI is not available as an objective function in the GA optimization. However, since total delay is a major contributor of the PI, as defined in this study, it can be considered an equivalent performance measure. For most of the scenarios, the signal timings optimized through Direct CORSIM optimization do not yield lower PIs than the NO timings. Figure 3.6 also shows that reported total delays for the OP timing plans (based on a single simulation run) are lower than the average total delays obtained through 100 simulation runs with an OP timing plan.

The Figure 3.7 – Figure 3.12 show various trends in PIs for the microsimulation tools. The OP and NO PI values represent averages from 10 simulation runs. In general, VISSIM and CORSIM simulation tools do not justify use of the optimized signal timings (NO timings are better). These microsimulation models, when used with default parameters, report increased PIs for signal timings optimized by any of the macroscopic tools. PIs from SimTraffic justify use of the optimized signal timings from any of the two macroscopic optimization tools.

3.3.6.2 Uniform Changes in Traffic Distribution

Figures 3.13-3.20 show changes in PI impacted by the changes in turning movement proportions. Both the left and right turning movement proportions were changed at the same time. This approach had the opposite effect on the degree of optimization. Increase in left turns is a limiting factor for the capacity of left turn phases, which, if not satisfied, causes additional delays. At the same time, an increase in right turns is a relaxing factor because the capacity of right turns is not limited due to right-turns-on red. The proportion of through movement increases and the through phase is under pressure when both turning movements decrease since the total traffic demand is kept constant.

Each of the macro- and microsimulation outputs are presented for OP and NO signal timings for all 10 scenarios. Figure 3.13 compares the PI changes for Synchro and TRANSYT-7F. The PIs from both optimization tools show that they can find better OP timings than the base NO timings. For the NO timing plans, TRANSYT-7F better handles increases in through movements (turning movements are reduced) than Synchro 6. However, when turning movements increase, Synchro 6 yields lower PIs than TRANSYT-7F. The only case where the OP PI is not lower than the NO PI is for the 20% decrease in turning proportions with TRANSYT-7F. For this scenario, TRANSYT-7F's PI (excessive fuel consumption) is minimal, although the PI used in the study (Delays + 10 x Stops) is not. This inconsistency is the reason for the discrepancy in microsimulation outputs based on TRANSYT-7F optimized signal timings for this scenario.

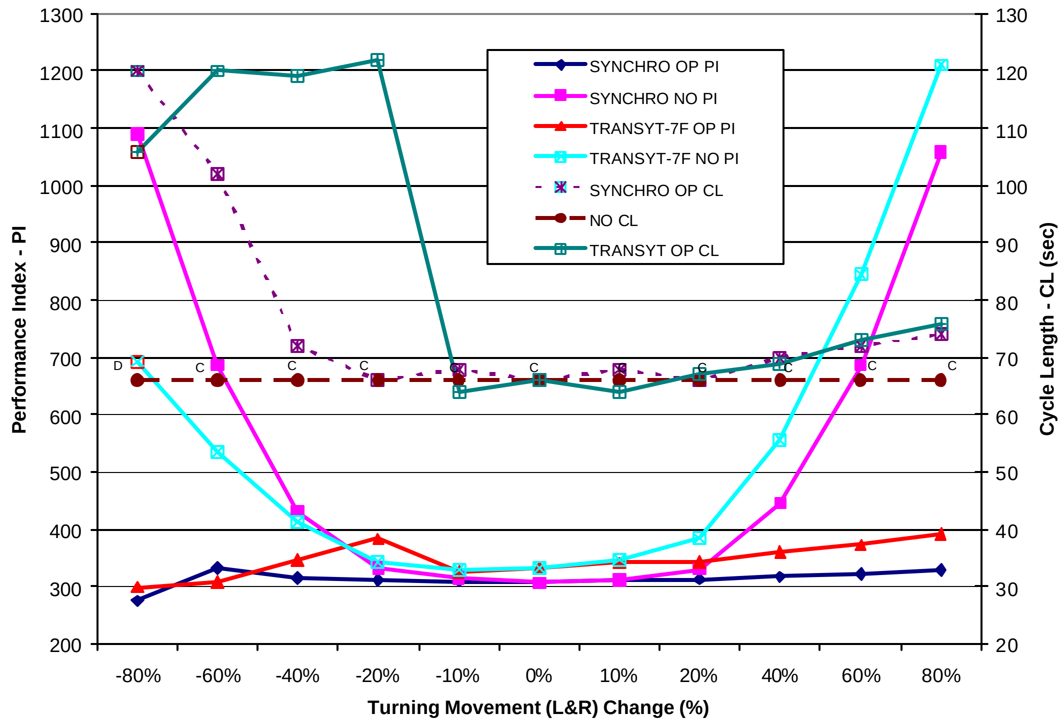


Figure 3.13 - PI and CL vs. TM Change for Synchro and Transyt-7F

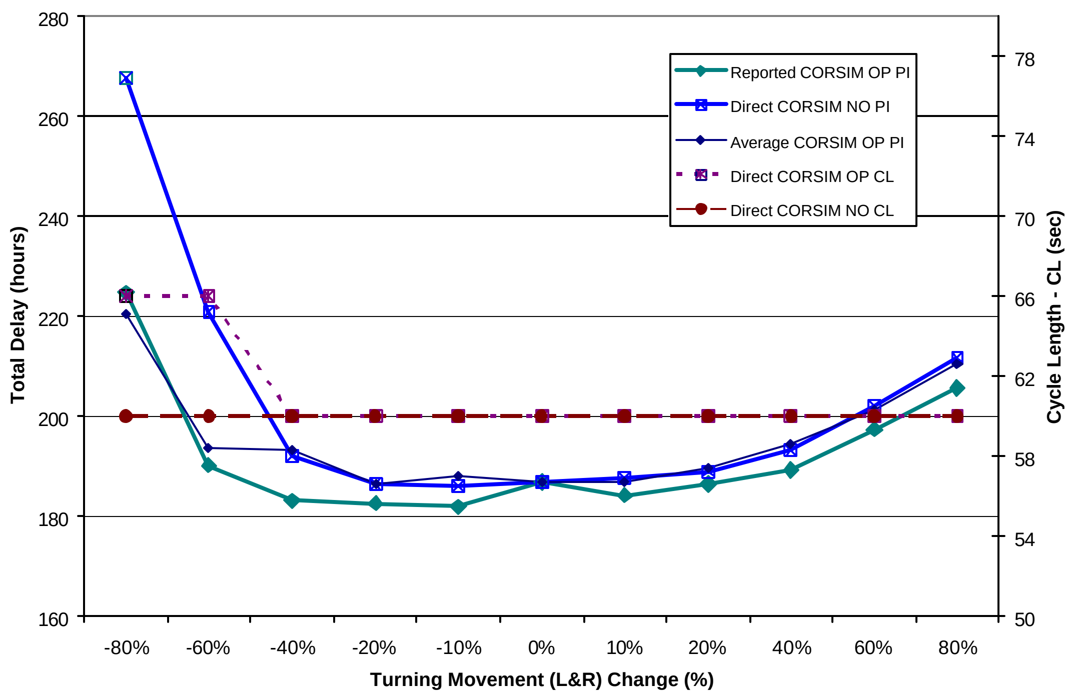


Figure 3.14 - Total Delay vs. TM Change for Direct CORSIM Optimization

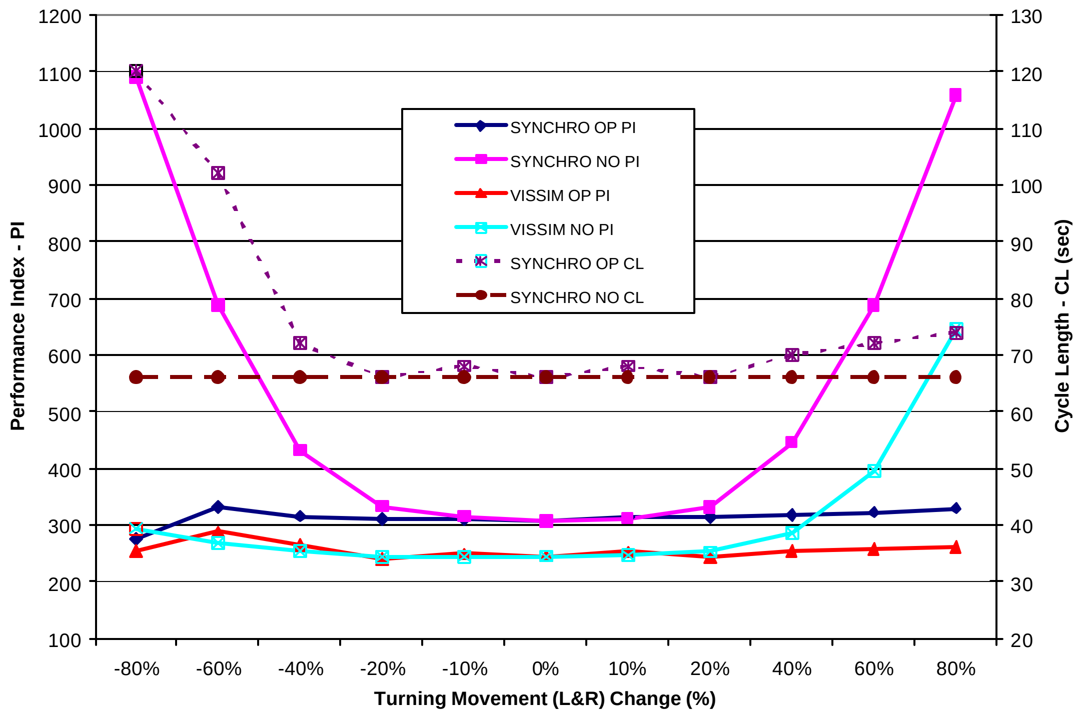


Figure 3.15 - PI and CL vs. TM Change for VISSIM and Synchro

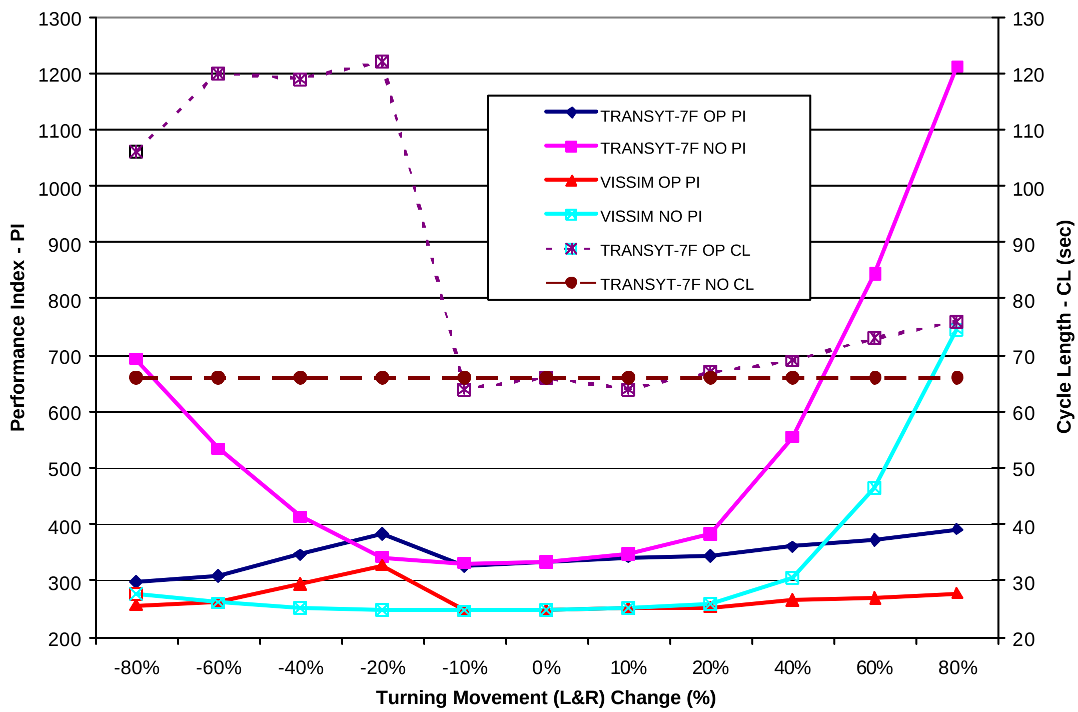


Figure 3.16 - PI and CL vs. TM Change for VISSIM and Transyt-7F

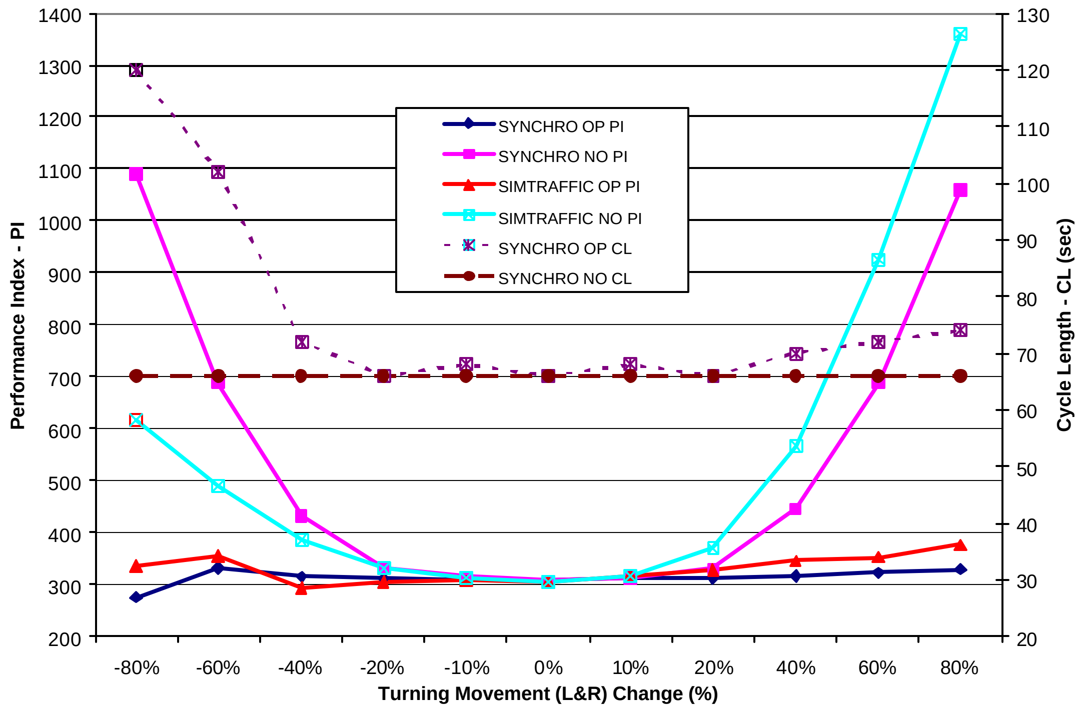


Figure 3.17 - PI and CL vs. TM Change for SimTraffic and Synchro

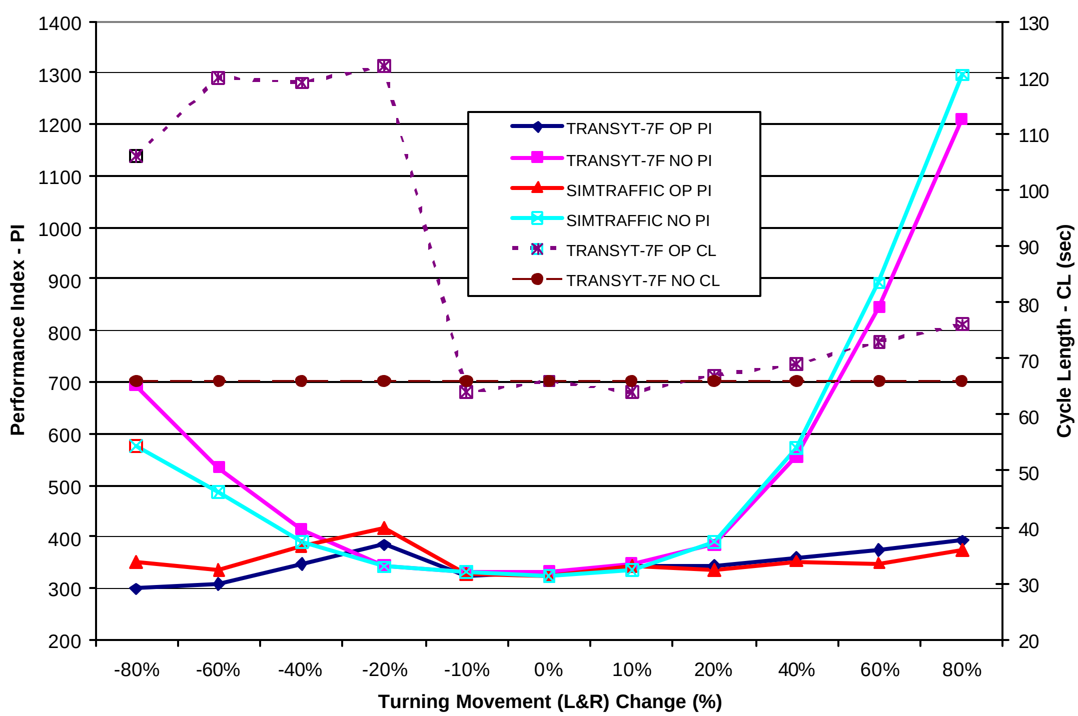


Figure 3.18 - PI and CL vs. TM Change for SimTraffic and Transyt-7F

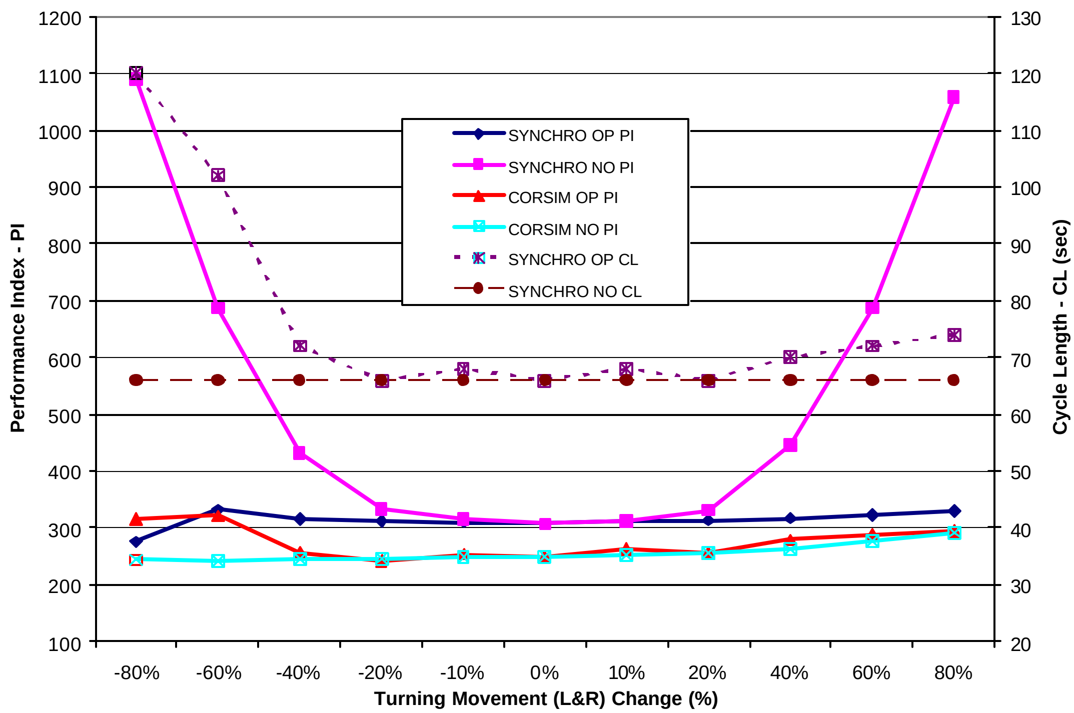


Figure 3.19 - PI and CL vs. TM Change for CORSIM and Synchro

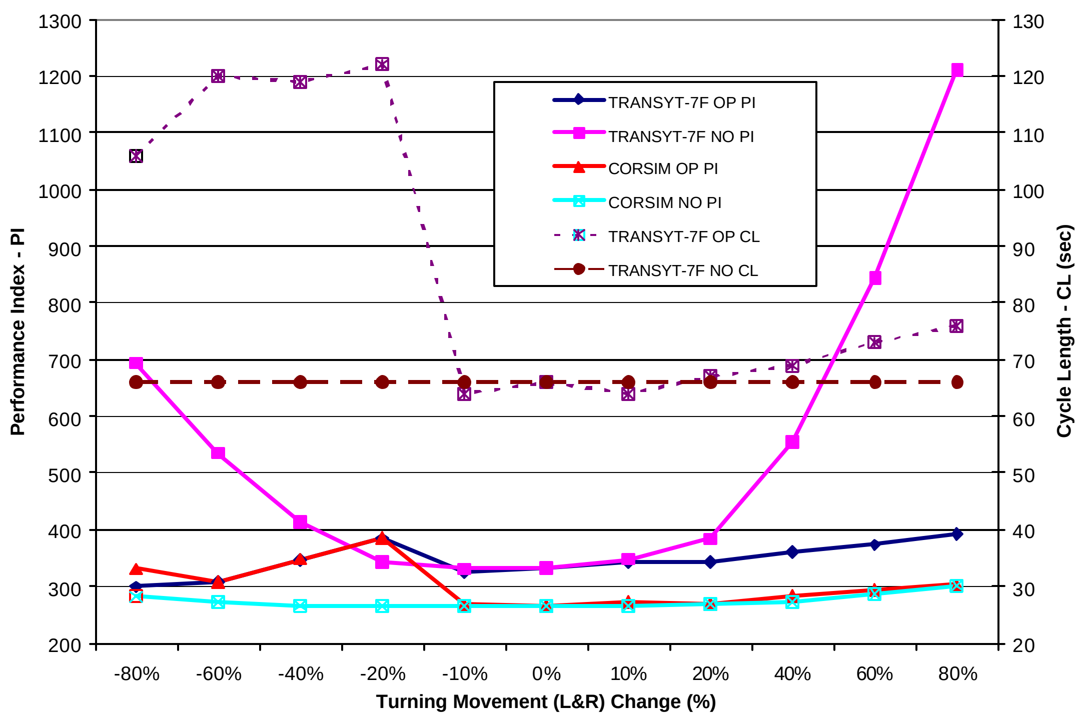


Figure 3.20 - PI and CL vs. TM Change for CORSIM and Transyt-7F

Figure 3.14 shows the results for Direct CORSIM optimization timing plans for changes in turning movements. The GA optimization procedure failed to produce reliable OP timing plans. The reported optimized total delays are lower than the averaged ones, showing the benefits of optimization where there are none. Except for the scenarios of 80% and 60% decrease in turning movements, the average delays of OP signal timings are not significantly different (at a 95% level of confidence) from the NO delays.

The Figure 3.15 to Figure 3.20 show changes in PI from the microsimulation outputs and macro optimization software used to optimize signal timings for the simulation runs. Once again, the microsimulation tools behave quite differently. In general, the PI results for each of the microsimulation tools are similar, regardless of which macro optimization tool is used. Among the microsimulation tools, CORSIM is the one that, when used with its default parameters, yielded to PIs that did not justify use of the optimized signal timings. Results from VISSIM justified optimization only when turning movements are increased. SimTraffic's results always justified the optimization. Use of an OP timing plan is justified when the simulation PI generated by this plan is smaller than the simulation PI generated by NO timing plans.

3.3.7 Discussion

The following paragraphs discuss the results of the evaluation of macroscopically optimized timing plans in micro simulation tools.

3.3.7.1 Uniform Changes in Traffic Demand

PI results for OP timing plans from VISSIM (Figures 3.7 and 3.8) and CORSIM (Figures 3.11 and 3.12) did not justify Synchro and TRANSYT-7F optimizations when they were compared to NO timing plans. These findings are a consequence of the fact that VISSIM and CORSIM have much higher vehicle throughputs than those used by Synchro 6 or TRANSYT-7F in their analytical traffic models. When demand increases, both Synchro 6 and TRANSYT-7F require an increase in cycle length to increase throughput. However, the throughput is not critical for the same link flows in VISSIM or CORSIM, so increasing cycle length is unnecessary and increases delay. Therefore, the PI for OP signal timings is worse than the PI for NO signal timings.

The differences between the capacities of SimTraffic, CORSIM, VISSIM and the HCM method are described by Tian et al. (2002). A set of similar experiments is conducted in this study for a single intersection approach to estimate the maximum throughputs for all three types of simulation software. The results show that maximum throughput rates for the three micro simulations are 0.52, 0.53, and 0.37 veh/sec/lane for VISSIM, CORSIM, and SimTraffic, respectively. For the sake of comparison, the adjusted saturation flow rate from Synchro 6 (based on 1900 veh/hour/lane HCM ideal saturation flow rate) of 1770 veh/hour/lane is equivalent to 0.49 veh/sec/lane. These findings explain the behavior of VISSIM and CORSIM outputs. They also explain why SimTraffic is sometimes more conservative than macrosimulation tools (Figure 3.9 and 3.10).

Although throughput rate may be the most important reason for discrepancies between macro- and microsimulations, it is not the only reason. Adjustment of saturation flow rates in Synchro and TRANSYT-7F is tried in order to reduce the discrepancies. The saturation flow rate in Synchro was adjusted to replicate empirically measured VISSIM and CORSIM throughput rates. Although this action reduced the optimal cycle length, the optimized timings still yielded a higher (worse) PI than NO timings.

3.3.7.2 Uniform Changes in Traffic Distribution

The turning movement changes modeled in this study impact the cycle lengths from the macro optimization in two ways. When turning proportions decrease, optimized cycle length increases to provide enough capacity for a higher number of through movements. Conversely, when turning proportions increase, the cycle length maintains an almost constant value (with only a slight increase). The major benefits of the OP timing plans in these cases come from optimizing phase splits.

When turning movement proportions decrease, through traffic at the intersections increases. These cases are equivalent to an increase in traffic demand only for the selected through movements. The explanation of the results in Figures 3.5-3.12 already shed some light on this problem. The throughput rates for VISSIM and CORSIM are much higher than the adjusted saturated flow rates for Synchro and TRANSYT-7F. Therefore, the increase in cycle length suggested by the macro optimization tools is not justified for these two microsimulations. On the other hand, the outputs for SimTraffic, whose throughput rate is lower, justify changes in cycle lengths due to the increase in through demand.

The results are quite different among microsimulation tools for an increase in turning movements. While SimTraffic and VISSIM outputs justify optimization of signal timings, CORSIM does not. VISSIM results report lower benefits from the optimization than Synchro or TRANSYT-7F. SimTraffic's results in both cases show higher benefits of optimization. Besides the differences in throughput rates for micro simulations, these differences in results should also be associated with default lengths of various vehicle types in each microsimulation.

It was observed that various microsimulations can handle different numbers of vehicles for the same left turn storage lengths. For example, while SimTraffic and VISSIM cannot handle more than 12 vehicles, CORSIM handles 14. For low turning movements these differences are not essential. However, when left turn demand approaches capacity, even one extra vehicle becomes critical. Since the traffic demand is kept continuous throughout the analyzed peak hour, the oversaturated left turn movements propagate queues and blockage of through lanes. This explains why CORSIM simulation did not justify use of the optimized timing plans. Its left-turn capacity was higher than those of other simulation tools.

3.3.7.3 Direct CORSIM Optimization

Although total delays (Figures 3.6 and 3.14) on optimized curves are either the same or lower than nonoptimized delays, better timing plans could have been found if more attention were given to Direct CORSIM optimization. There are three factors that account for this.

First, it seems that 100 generations were not enough to reach optimal solution for such a complex optimization. The timing plans might be better if the authors divided the optimization process into a few steps, e.g., first optimizing the cycle length, then optimizing splits and offsets (Hale 2005).

Second, in order to get the optimal solution, the initial solution should be as close as possible to the sought optimal solution. This is a paradox of the Direct CORSIM GA optimization. The optimization process is supposed to find the optimal solution; it is not meant to be a procedure to fine-tune the optimal solution. The way it is designed in the Direct CORSIM procedure, GA optimization becomes more of a fine-tuning tool than a procedure for finding an optimal solution from a wide range of possible solutions. Therefore, better signal timings may be found if the many signal timing sets were tried as the initial points in the optimization procedure. However, even if this were the case, there is no guarantee that a global optimum would have been reached.

Third, even when TRANSYT-7F (Direct CORSIM optimization through TRANSYT-7F) reported a better (than initial) solution, it is not always better when the stochastic nature of microscopic simulation runs is considered. The TRANSYT-7F manual (Hale 2005) does not explain the process of checking the quality of the optimal solution. It seems that the performance of each timing plan is evaluated only for one simulation run. The optimal timing plan is often slightly better than the original timing plan. However, when the performance of the OP timing plan is evaluated for many simulations runs (various random seeds), it may not be better than the original timing plan. An illustration of this discrepancy can be seen in Figure 3.6 for the scenario of a 10% increase in traffic demand. In this case, the optimal solution reported a decrease in delay for about ten hours while the average delay, obtained from 100 runs, was higher than the delay for NO signal timings.

3.3.8 Summary of Selection of Appropriate Simulation Tools

The reliability of signal timing optimizations for two types of macroscopic optimization software, Synchro and TRANSYT-7F 10, was evaluated in the three most popular microsimulations in the U.S.: SimTraffic, CORSIM and VISSIM. Default simulation parameters were used to perform the evaluations. In addition, Direct CORSIM optimization, a new feature of the TRANSYT-7F software, was used to perform a GA stochastic optimization within CORSIM. The following conclusions are reached in this part of the study.

The results show that OP signal timings for various changes in traffic flows can be justified in microsimulation software (SimTraffic). However, this justification highly depends on the adjustments of the microsimulation parameters. It seems that SimTraffic is calibrated in such a way to justify optimization schemes from macroscopic optimization tools. The other two microsimulations need proper calibration before they can be used for assessment of various signal timing plans. However, the real question is whether macroscopic optimization and simulation software can be adjusted to field-calibrated microsimulations. The answer to this question will have to wait for the findings from the ageing of timing plans on real world arterials.

The reliability of the stochastically optimized signal timings was evaluated through hundreds of CORSIM runs. The GA optimization process is very slow and highly dependant on the timing plan used as a starting point for the optimization. Above all, the GA optimization generates timing plans that are not reliable when evaluated through many CORSIM simulation runs. Except for a few extreme changes in traffic flows, the results showed that for the examined traffic conditions, Direct CORSIM optimization did not generate significantly better timing plans than the existing plans. GA seems to be a powerful but unexplored optimization tool. If the GA optimization is to be used as an everyday tool to optimize signal timings, more guidance on how to use the optimization is needed. The areas where this procedure lacks explanation are: design of the optimization process (multilevel versus all-in-one), sensitivities of both parameters and results.

The findings from the evaluation of macroscopically optimized timing plans in microscopic simulation tools can be summarized as:

- Only SimTraffic simulation is calibrated in such a way that justifies use of the macroscopically developed optimized timing plans.
- Other microsimulation tools (VISSIM and CORSIM) need a comprehensive calibration and validation of the underlying traffic models that are running under these simulators.
- Calibration and validation of these microsimulators should be conducted with a clear objective to justify use of the microscopically optimized timing plans.

Based on these results, Synchro and SimTraffic are selected to be used for evaluating the deterioration of pretimed and actuated-coordinated control regimes. However, SimTraffic simulation cannot be used for evaluating the deterioration of the SCOOT adaptive control regime. This is because there is not an interface that connects SCOOT and SimTraffic. For this reason, the deterioration of the SCOOT control has to be done with VISSIM, the software that interfaces SCOOT. As described in previous parts of this chapter, the method for determining the deterioration of adaptive control relies on the deterioration of pretimed signals. Therefore, VISSIM settings need to be calibrated in such a way to justify the use of optimized timing plans from SYNCHRO. In short, a combination of Synchro and SimTraffic tools is used to estimate the ageing of pretimed and actuated traffic control while a combination of VISSIM, Synchro and SCOOT is used to estimate the ageing of SCOOT adaptive control.

3.4 Major Modeling Process

Figure 3.21 shows a flowchart describing the modeling efforts for assessment of deterioration of the three traffic control regimes. There were two aspects of assessing the ageing of timing plans: deterministic and stochastic.

The deterministic aspect was described in the subsection of this chapter titled "Traffic Demand and Distribution Scenarios." Essentially, one base scenario and 20 various deterministic changes of traffic flows were modeled in Excel. Optimized timing plans were then developed in Synchro for these traffic flows. Ten simulations were run for both optimized and nonoptimized (base) timing plans. The whole

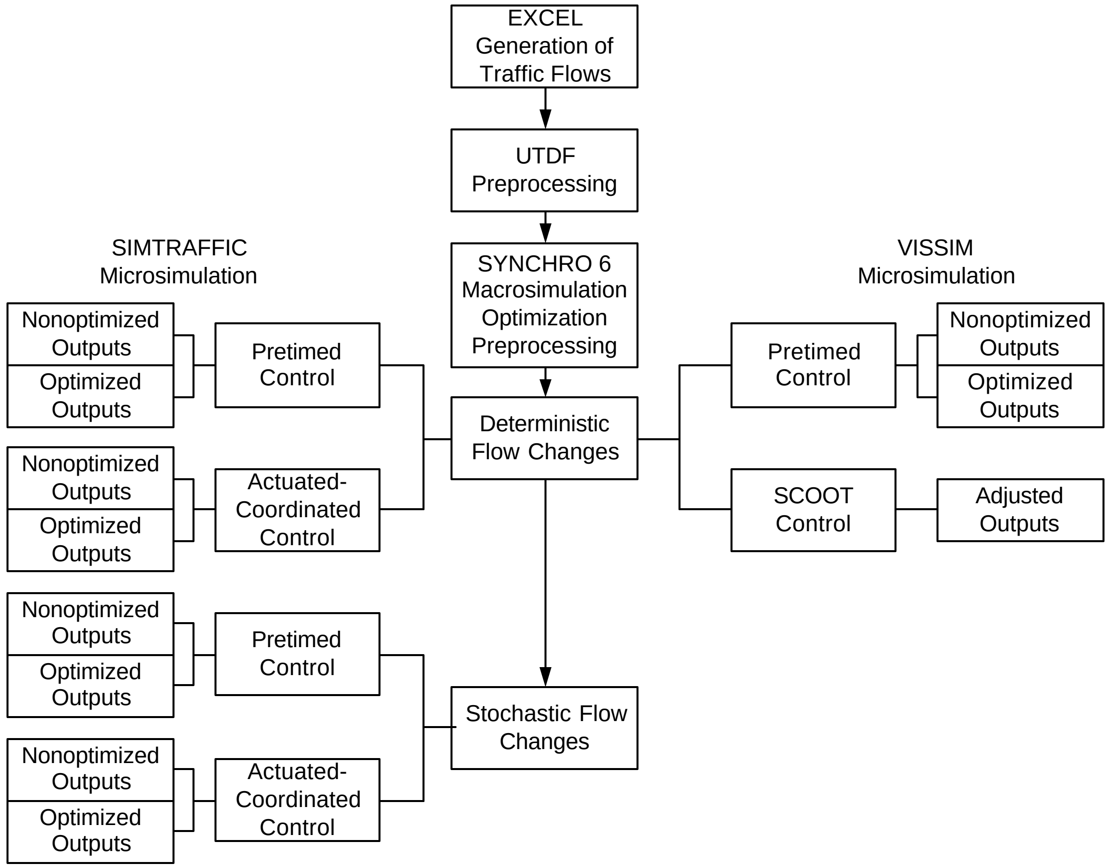


Figure 3.21 – Modeling Process for Evaluating Ageing of Traffic Control Regimes

procedure is done for pretimed and actuated control in SimTraffic and for pretimed and SCOOT control in VISSIM. Simulating SCOOT in SimTraffic is impossible since there is no interface between them. An assessment of actuated control in VISSIM has not been done due to problems in calibrating the VISSIM model for actuated traffic control from Synchro. More about VISSIM and SCOOT calibration is given in the following section of the chapter.

Stochastic traffic demand and distribution scenarios were developed to assess the ageing of timing plans. This approach was used to model the highly irregular nature of changes in traffic flows, which are more realistic than the uniform deterministic changes applied in the first approach. For this purpose, 100 link flow scenarios were generated in the Excel worksheet. Stochastic traffic demand was modeled through Monte Carlo simulation with uniform distribution of network entry input flows. Similarly, stochastic traffic distribution was modeled through Monte Carlo simulation with uniform distribution of turning movement proportions. However, unlike the deterministic scenarios, each entry point flow and turning movement proportion was allowed to have a different value. Also differing from the deterministic scenarios, the upper and lower limits were in percentages from the mean values, ± 20% for entry input demand and ± 60% for turning movement proportions. Uniform distribution was selected over normal distribution to model the change of mean values and not around the mean value.

Stochastic experiments were done only for SimTraffic simulations. The reason for this is the amount of computer time necessary to perform VISSIM simulations for SCOOT control. SCOOT is a real-time traffic control system and its process of optimizing signal timings cannot be speeded up. This obstacle prevents VISSIM from running faster than real time. For 21 deterministic scenarios, it was necessary to perform 210 VISSIM-SCOOT simulations. Considering that only one simulation can be run at a time (only one SCOOT computer was available), it took 21 12-hour working days to run these 210 simulations (10 1-hour simulations each day). Stochastic simulations with SCOOT would require half the time and would not be essential in determining the potential ageing of the SCOOT control.

3.5 Preparing VISSIM-SCOOT Simulations

The reliability of Synchro timing plans in SimTraffic has been checked. It was found that the change in SimTraffic performance measures justify use of Synchro's optimizations. Simulation runs for the Synchro-SimTraffic combination could be run without any further adjustments. However, before VISSIM simulations could be run, it was important to calibrate VISSIM to support use of the optimized pretimed plans from Synchro. In addition, SCOOT settings based on incoming traffic from VISSIM had to be calibrated. The following subsections describe validation efforts for VISSIM with pretimed control, the SCOOT-VISSIM connection, and validation of SCOOT settings.

3.5.1 VISSIM Calibration for Pretimed Traffic Control

Data between Synchro and VISSIM are shared through the UTDF format. Saving data from Synchro in the UTDF format is one of Synchro's standard features. Link traffic flows, signal phasing and timing data, network layout and lanes are transferred by TEAPAC's pre-processor "Pre-Synchro" from the UTDF files to VISSIM. As a result, links, connectors and nodes are built in the VISSIM environment. Traffic flows are converted into VISSIM's traffic inputs and routing decisions automatically. Also, signal timing and phasing data from Synchro are saved as separate NEMA files for each signal in the network. In short, all basic data are accurately transferred to VISSIM.

However, underlying car-following and driver behavior models in VISSIM are not adjusted to Synchro's model. It is a question of whether they can be properly adjusted at all. Preliminary runs (described in previous sections of this chapter) showed that the major problem is throughput. Throughput is largely influenced by adjusted saturation flow rate at the intersections.

Saturation flow rate is one of the most important parameters in the field of traffic control. Basically, this parameter represents a service rate at the queue channel which, in this case, is an approach to the signalized intersection. By adjusting saturation flow rate, we effectively adjust the service rate of the signal's phase. When signal timings are calibrated in the field, the values for this parameter are collected by observing how many vehicles pass the stop line during a given green interval. Saturation flow rate depends on driving habits of the local population, geometry of the intersection, grade of the terrain, weather, pavement conditions, etc. The Highway Capacity Manual (HCM) (HCM 2000) defines ideal saturation flow rates for certain road types. These rates are then adjusted for the aforementioned factors to get actual saturation flow rates at the intersection approaches.

Synchro complies with the HCM and uses the same approach to calculate adjusted saturation flow rates. Default VISSIM parameters are, however, set in such a way that saturation flow rates measured from its simulations are much higher than those from Synchro. As a consequence, "VISSIM's traffic control," with the same parameters as "Synchro's traffic control," serves more vehicles in the same amount of time.

This difference has a crucial impact when the optimization is performed. Since it has smaller saturation flow rates, Synchro assumes that larger cycle lengths are needed to increase vehicle throughput. It is well known (Tarnoff and Ordonez 2004) that larger cycle lengths increase delays. Consequently, with optimized timings from Synchro, additional cycle length and increase of delays is transferred to VISSIM. VISSIM's traffic, on the other hand, does not see any benefits from the longer cycle length. Instead, it sees all the delays caused by the longer cycle length. For this reason, VISSIM's performance under optimized timing plans from Synchro is often worse than with non-optimized plans. As a rule, these nonoptimized timing plans usually have shorter cycle lengths.

VISSIM's saturation flow rate can be changed by adjusting the parameters Additive Part of Desired Safety Distance (BX_ADD) and Multiplicative Part of Desired Safety Distance (BX_MULT) contained within the Wiedemann 74 car Following Model (PTV AG 2005). The VISSIM Manual provides a lookup table that lists saturation flow rate values for major combinations of BX_ADD and BX_MULT. However, it is not possible to properly calibrate VISSIM by simply adjusting these parameters to match Synchro's saturation flow rate from VISSIM's lookup table. The approach that gives results is comparison of PIs from VISSIM and Synchro. This means adjusting BX_ADD and BX_MULT in such a way to make the PI from VISSIM's simulation output as close as possible to Synchro's output (for base traffic conditions). If these two PIs are similar, it is expected that total performance of traffic models contained in these two simulation tools will be similar. This means that the models would yield similar results for any traffic conditions. Table 3.2 provides values for Delay, # of Stops, and # of vehicles that have exited network for various values of BX_ADD and BX_MULT. The last column in Table 3.2 represents the Sum of Squared Differences (SSD). The differences between Delay, Stops, and

Table 3.2 – Calibration of VISSIM Saturation Flow Coefficients

BX_ADD	BX_MULT	Delay (Hours)	Stops	Vehicles	SSD
3	3.5	194.4	26810	11889	3687544
3	4	201.7	27060	11883	2789549
3	4.5	211.3	28167	11878	317219
3.2	4	213.5	28465	11841	71928
3.2	4.5	218.6	29152	11890	178255
3.3	3.9	216.4	29008	11862	77720
3.3	4	219.4	29675	11857	893612
3.3	4.1	218.5	29143	11862	170965
3.3	4.2	219.1	29149	11870	175723
3.4	3.8	214.6	27930	11874	640190
3.4	3.9	215.1	29017	11870	82611
3.4	4	222.2	29263	11854	284788
3.4	4.1	218.4	28639	11865	8580
3.4	4.2	224.7	30528	11851	3233650
3.5	3.5	213.4	28393	11857	114283
3.5	3.7	218.8	28929	11868	39812
3.5	*3.8*	*217.5*	*28695*	*11881*	*1316*
3.5	3.9	221	29208	11869	228641
3.5	4	219.9	29174	11873	197235
3.5	4.1	224	29858	11872	1272457
3.5	4.2	231.4	31449	11843	7394349
3.5	4.5	244.5	34418	11835	32355675
3.6	3.8	225.5	30634	11872	3625282
3.6	3.9	227.3	29817	11856	1182145
3.6	4	232.4	31284	11865	6523170
3.6	4.1	239.7	34966	11836	38889793
3.7	3.7	226.7	30264	11873	2353205
3.7	4.2	242.1	33098	11851	19080493
3.8	3.5	228.3	30515	11852	3187011
4	3.5	236.1	31987	11845	10609357
4	4	253.5	36249	11828	56538767
4	4.5	275.1	41396	11804	160435646
SYNCHRO		227	28730	11880	0

Vehicles values and corresponding Synchro values are taken, squared, and summed. The minimal SSD means that VISSIM's PI for corresponding BX_ADD and BX_MULT is the closest to Synchro's PI.

Therefore, the values 3.5 and 3.8 are chosen for BX_ADD and BX_MULT, respectively. VISSIM's results for pretimed traffic control, presented in the next chapter, validate selection of these values.

3.5.2 SCOOT – VISSIM Connection

The SCOOT-VISSIM interface is developed to evaluate an integrated traffic system including vehicle traffic, bus, rail transit, pedestrian and bicyclists controlled by SCOOT in the VISSIM simulation environment. This section describes the major elements and data flows of the SCOOT-VISSIM interface.

VISSIM consists of two parts: traffic simulator and signal state generator. The former simulates traffic flow, while the latter determines signal phases and timings. The traffic simulator sends detector values to the signal state generator each second. Based on received detector data, the signal state generator determines optimal signal timing parameters and sends them back to the traffic simulator. Figure 3.22 shows the internal structure of VISSIM's simulation system.

Signal Controller Junction (SCJ) defines the signal controller that controls the signal phases of the junction (intersection) during the simulation. SCJ also allows the external signal control system to supplement its own signal generator. This is done by setting the optional Vehicle Actuated Programming (VAP) module to model detector-based signal control. Each signal controller has its own individual VAP module. This

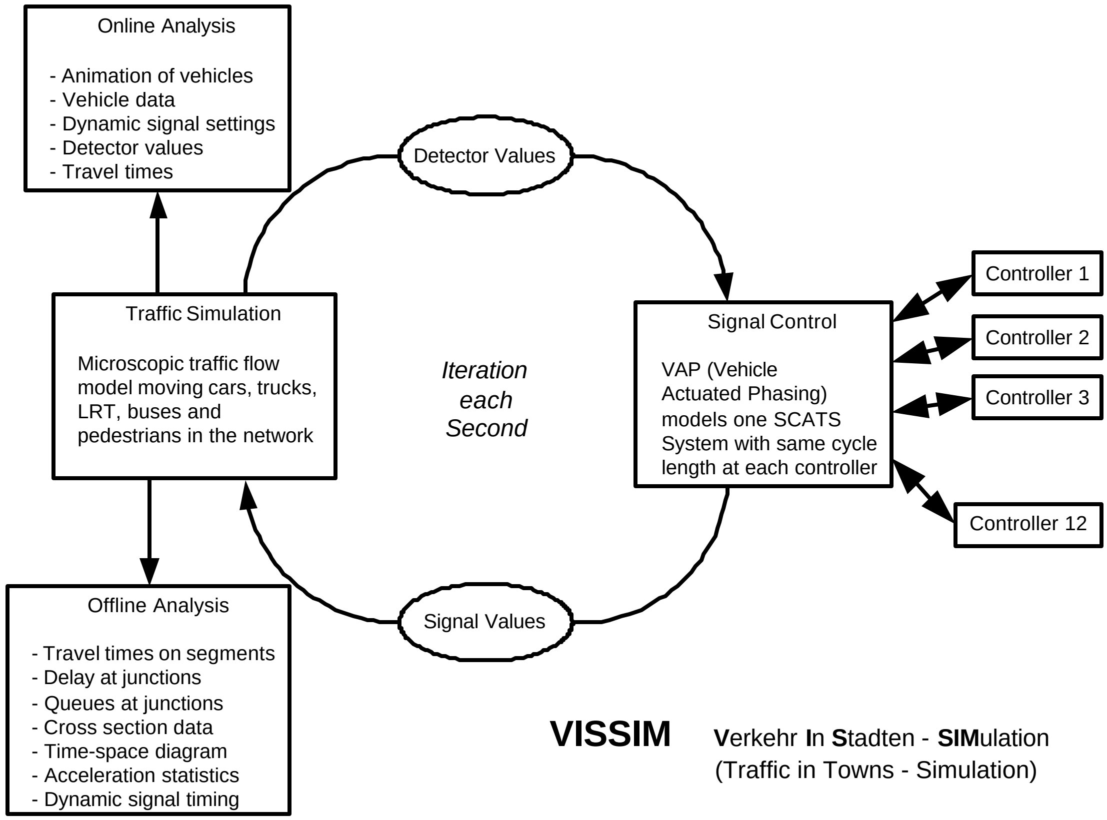


Figure 3.22 - Internal Structure of VISSIM Simulation Software

characteristic makes SCOOT-VISSIM interface different from the Dynamic Link Library (DLL) of the SCOOT-CORSIM interface (Hansen and Martin 1998). The SCOOT-VISSIM interface includes two parts: the VAP modules for signal controllers, and the communication module, both developed in VISUAL C++ (Feng and Martin 2003). The functions of the VAP module are:

- Get detector data from the SCJs
- Exchange virtual detector data and new SCOOT signal timings between the SCJs and the communication module
- Implement new traffic signal lights

The functions of the communication module are:

- Exchange detector data between each VAP and SCOOT
- Retrieve new SCOOT signal timings
- Distribute SCOOT signal timings to the corresponding VAP

Every SCJ in the VISSIM network has one VAP module to control the exchange. Figure 3.23 shows the architecture and data flow of the SCOOT-VISSIM simulation environment.

3.5.3 SCOOT Validation

SCOOT's working principles were described in the previous chapter. Basically, the quality of SCOOT's performance is mostly affected by its ability to model arrivals and service rate at each intersection approach. If these two variables are known, SCOOT can be adjusted to perform efficiently.

The arrival of vehicles at a stop bar is estimated by the SCOOT traffic model. This model estimates when vehicles detected upstream will arrive at the stop bar. The model takes into account the journey time necessary to travel from the detectors to the stop bar and dispersion of the vehicles in the platoon. SCOOT does not allow adjustment of the platoon dispersion model. The only parameter that can be changed to adjust the accuracy of vehicle arrivals is Journey Time – JNYT.

Service rate at the signal approach is represented by saturation flow rate or SCOOT's equivalent saturation occupancy. This parameter is the most important parameter for SCOOT validation. It impacts both the platoon of vehicles that will depart from the approach, as well as the number of vehicles that will potentially

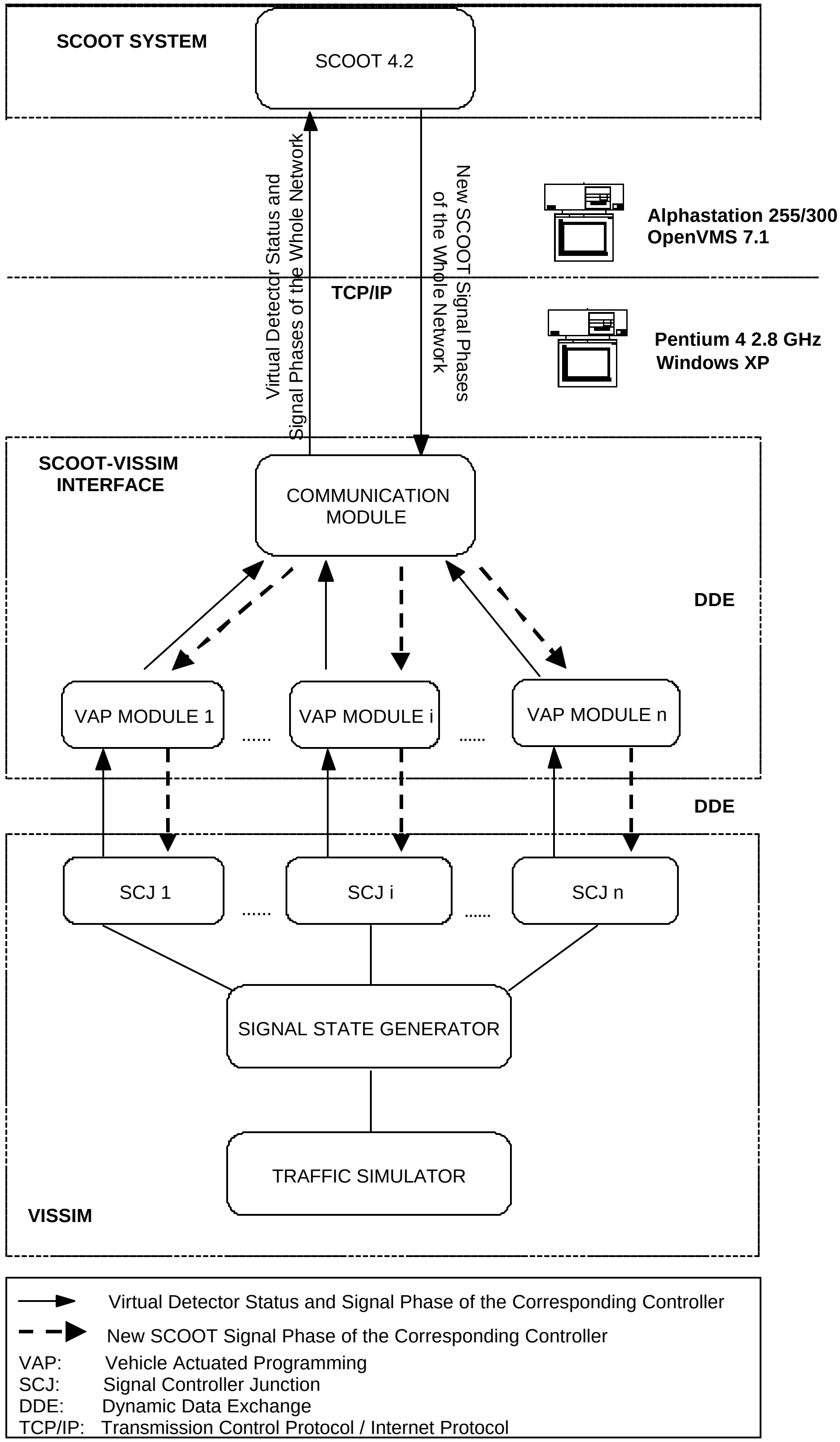


Figure 3.23 – VISSIM-SCOOT Simulation Environment

remain in the queue. Saturation Occupancy - STOC is a SCOOT parameter that is used to adjust this service rate.

There are a few other parameters important for SCOOT validation. Start Lag Time (SLAG) represents the time from the appearance of green on the street to the time when the traffic queue effectively starts moving. Similarly, End Lag Time (ELAG) is the time from the appearance of amber on the street to the time of effective cessation of traffic movements. Maximum queue clear time (QCMQ) represents the time necessary for the maximum traffic queue to clear the intersection. This parameter helps SCOOT to recognize when traffic queues are causing congestion upstream of the detectors.

All of these parameters help traffic engineers to adjust SCOOT to best represent real traffic conditions. If, for example, the STOC is too small or the JNYT is too short, SCOOT's model will estimate vehicles in queue when there are none. Similarly, if the STOC is too high or the JNYT is too long, SCOOT's model will not be able to estimate vehicles in queue when they are there.

Validation of SCOOT is the process in which vehicle estimates from SCOOT's models are compared with real-world vehicles. This process follows calibration of SCOOT parameters in which all of the aforementioned parameters are adjusted. SCOOT validation in this study was done by comparing the number of vehicles in queue from the SCOOT model with the number of vehicles in queue from the traffic simulation. Traffic simulation here plays the role of real-world traffic conditions.

Figure 3.24 shows screenshots from both SCOOT and VISSIM. The upper part of Figure 3.24 shows SCOOT's link validation facility, which reports the number of

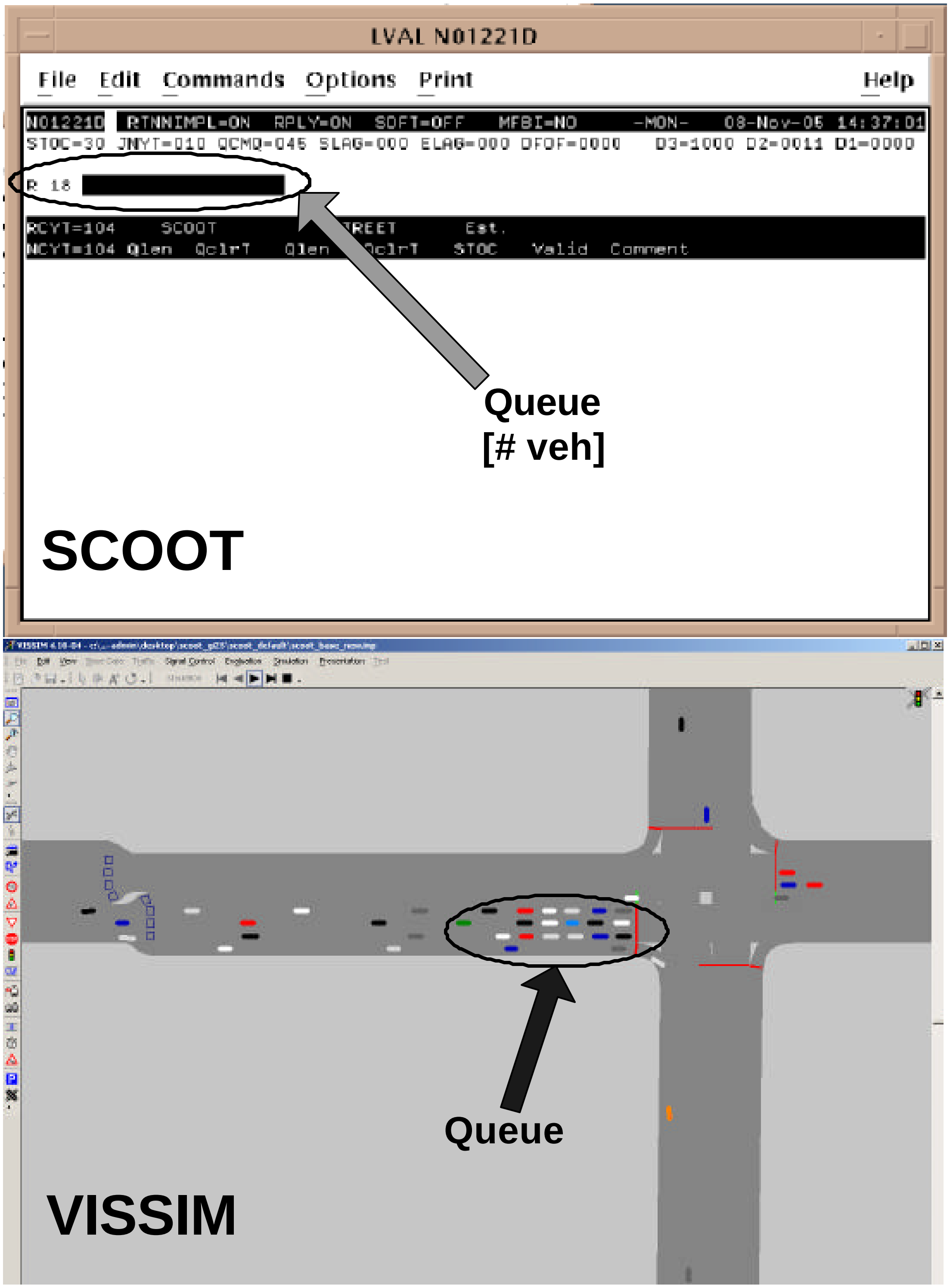


Figure 3.24 – Queue Formation in SCOOT and VISSIM

vehicles in queue. The black bar in the upper left corner varies as vehicles join or leave the queue. R stands for red light and the number 18 represents the current number of vehicles in queue. The lower part of the figure shows queue formation at an intersection approach from VISSIM. At the time instance shown in Figure 3.24, the number of vehicles in queue from VISSIM (16 vehicles standing and two just about to join the queue) closely matches SCOOT's queue estimation.

Figure 3.25 shows the coefficient of determination between queues in SCOOT and VISSIM at the end of the red light intervals. The data consists of parallel queue measures taken at the two busiest approaches for each of the nine intersections in the network. Traffic conditions during the data collection were the same as for the base scenario in modeling the deterioration of traffic control regimes. The coefficient of determination (R^2) shows that there is a high correlation between number of vehicles in queue for SCOOT and VISSIM. An underlying least square model shows that SCOOT estimates around 18 vehicles when 20 vehicles are standing in real queue. The results of the validation show that, in general, SCOOT has been validated properly.

The SCOOT estimation error is most likely result of imperfections in modeling the dispersion of the traffic platoon and imperfections in converting the SCOOT's LPUs into the actual number of vehicles. A coefficient of determination of 0.84 is pretty high. The results from real-world SCOOT installations rarely have such high correlations between real queues and queues from the SCOOT model (Jayakrishnan et al. 2001).

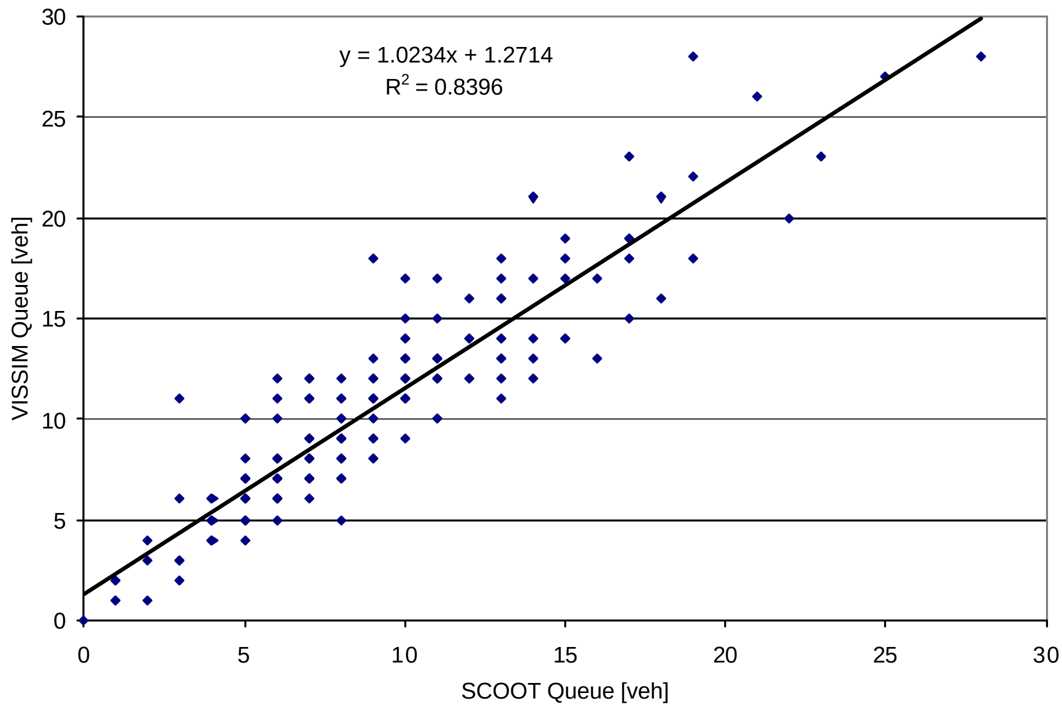


Figure 3.25 – Correlation between Traffic Queues in SCOOT and VISSIM

3.6 Summary of Research Methodology

The first part of this chapter defined ageing (deterioration) of traffic control regimes. It is important to understand that ageing in this study means the process of deterioration of a traffic control regime due to long and steady changes in underlying traffic conditions (demand and distribution). All common cyclical (diurnal and weekly) traffic fluctuations and variations are not considered by this approach.

A method is proposed to measure the extent of the signal timing plans' ageing. This method relies on the existence of optimized pretimed traffic plans for any experienced change in traffic flows. Existence of these plans is a crucial factor in determining ageing of adaptive traffic controls.

The second part of the chapter discussed design of simulation experiments. An arterial network with basic features of modeled traffic controls are introduced in this section. A scenario of base traffic conditions is presented, along with 20 other scenarios for various traffic demands and distributions. Experiments with these variations were done both for selection of appropriate simulation tools and assessment of deterioration of traffic control regimes.

The third part of the chapter is very important. It describes efforts to select proper simulation tools for assessing the deterioration of traffic control regimes. Thousands of simulations are performed to find the best combination of macro- and microsimulation tools. Finally, the results showed that Synchro-SimTraffic is the best combination. However, it was also concluded that VISSIM simulation has to be calibrated for pretimed traffic control. VISSIM is the only simulator that is connected to SCOOT and that can be used to assess the ageing of the SCOOT control.

The fourth part of the chapter describes the major modeling process. Pretimed and actuated controls are modeled (using the Synchro-SimTraffic combination) for both deterministic and stochastic changes in traffic flows. Pretimed and SCOOT adaptive controls are modeled only for deterministic changes in traffic flows.

The last part of the chapter describes tasks for preparing for the VISSIM and SCOOT simulations. First, VISSIM is calibrated in such a way to justify use of optimized timing plans from Synchro. The basic features of VISSIM- SCOOT connection are then presented. Finally, SCOOT validation is performed. The results confirmed that SCOOT settings are fairly calibrated to represent ‘real’ conditions from the VISSIM simulation.

CHAPTER 4

RESULTS

This chapter presents the results of the modeling. The results are presented in the form of graphs and tables with minimal discussion of their meanings. A major discussion of the findings is given in the next chapter. The chapter is divided into four sections. The first part of the chapter presents a general finding about the methodology used in this study. The second section deals with the reliability of the ageing measure used in Bell's work (Bell 1985). The third part of the chapter presents the results for ageing of pretimed and actuated-coordinated traffic control regimes for deterministic and stochastic changes in traffic flows. The final part of the chapter shows the results of assessing the ageing of SCOOT adaptive control.

4.1 Justification of the Adopted Approach for Measuring Ageing

Figure 4.1 shows the relationship between the PI and uniform increase in traffic demand over the entire network. The purpose of Figure 4.1 is to validate the assumed idealized concept of ageing shown in Figure 3.1. The findings show almost no difference in PI for optimized and non optimized traffic control for a 5% increase in traffic demand. As traffic demand grows, the difference in PI becomes greater.

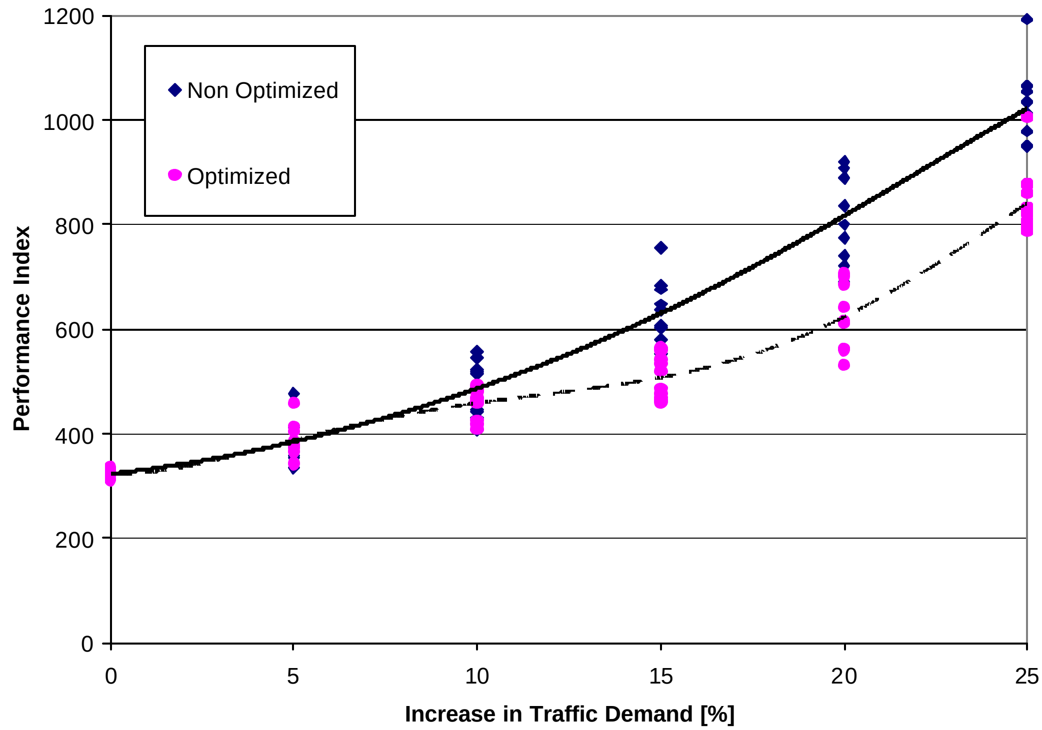


Figure 4.1 - Modeled Impact of the Uniform Increase in Traffic Demand on PI

4.2 Reliability of Bell's Ageing Measure

Figures 4.2 – 4.5 show the relationship between the benefits of updating signal timings and CF – Average Absolute Difference in Link Flows. The figures show this relationship for a pretimed traffic control regime for four different types of changes in link flows. The CF parameter is calculated using Equation [2.1]. The benefits of updating signal timings are calculated according to Equation [3.1]. Figures 4.2-4.5 show a high coefficient of determination (R^2) between the CF parameter and the benefits of updating signal timing plans.

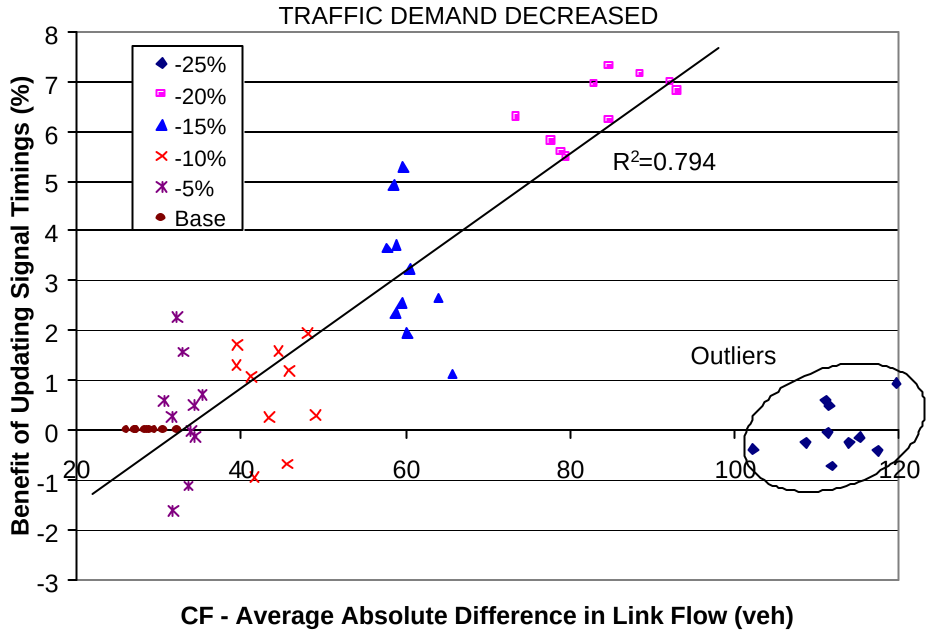


Figure 4.2 - Benefits of Updating vs CF for Decreased Traffic Demand

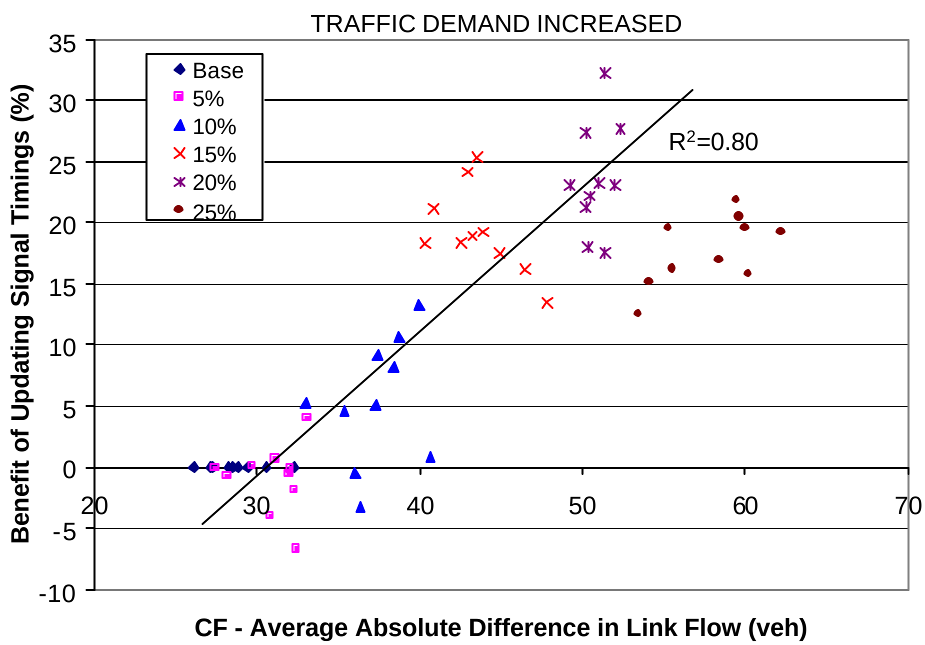


Figure 4.3 - Benefits of Updating vs CF for Increased Traffic Demand

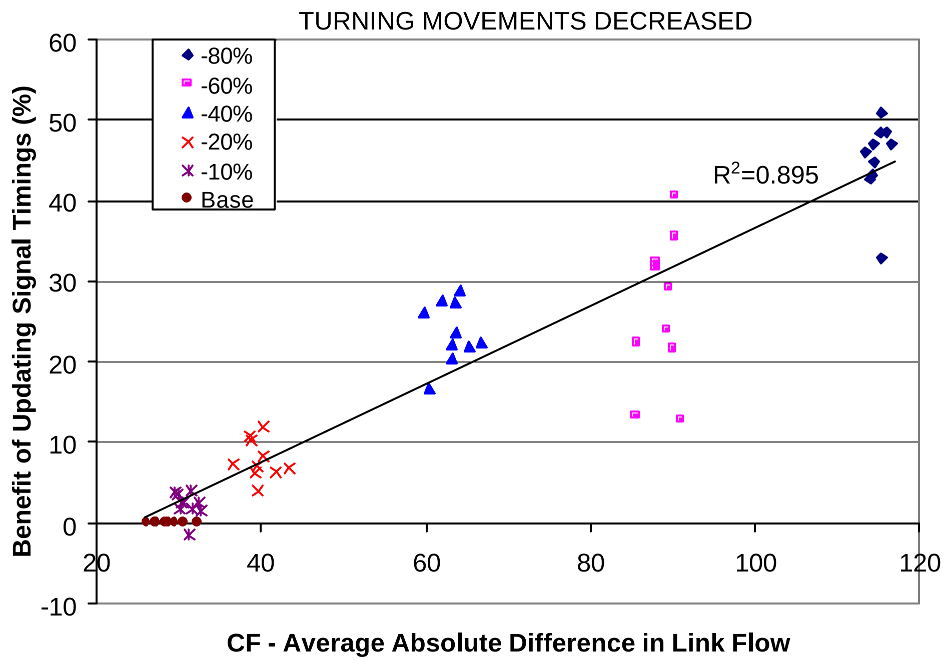


Figure 4.4 - Benefits of Updating vs CF for Decreased Turning Movements

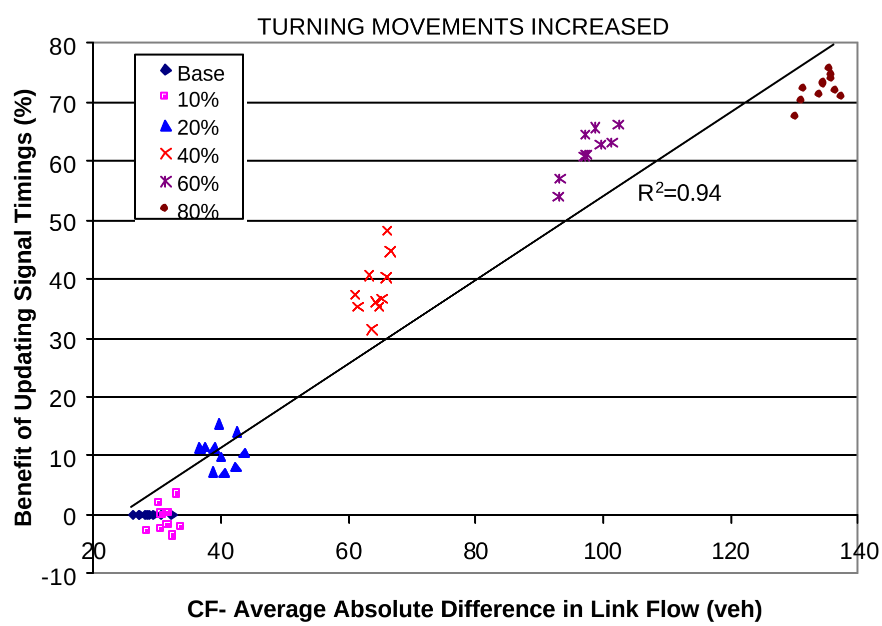


Figure 4.5 - Benefits of Updating vs CF for Increased Turning Movements

Figure 4.2 shows how the benefits of updating timing plans are related to a decrease in traffic demand. For an average decrease of 20 vehicles per link, around 2% of delay and stops could be saved by retiming traffic signal timing plans.

Figure 4.3 shows the impact of the increase in traffic demand on the benefits of updating signal timings. The figure shows that the benefits increase up to a certain point. On the section where benefits increase, around 10% benefits are estimated for an average increase of 10 vehicles per link.

Figure 4.4 shows the relationship between the benefits of updating and the decrease in turning movement proportions. A decrease in turning movements (and increase in through movements) equivalent to the CF ageing measure of 10 yield around a 5% increase in benefits of retiming traffic signal timing plans. The results show that the benefits could reach 50%.

The relationship between the benefits of updating signal timings and the uniform increase in turning movements is shown in Figure 4.5. An increase in turning movements (and decrease in through movements) equivalent to the CF ageing measure of 10 yields around a 10% increase in the benefits of retiming traffic signal timing plans. The results show that the benefits of updating can be as high as 75%.

Figure 4.6 shows the same relationship between the benefits of updating and the average change in network traffic demand. The average change in network traffic demand represents a measure used to express random changes in traffic demand and turning movements which occur at each link, independently of other links (traffic flows are still balanced). In other words, Figure 4.6 represents all of the changes randomly applied in the network from the previous four figures. Unlike Figures 4.2 –

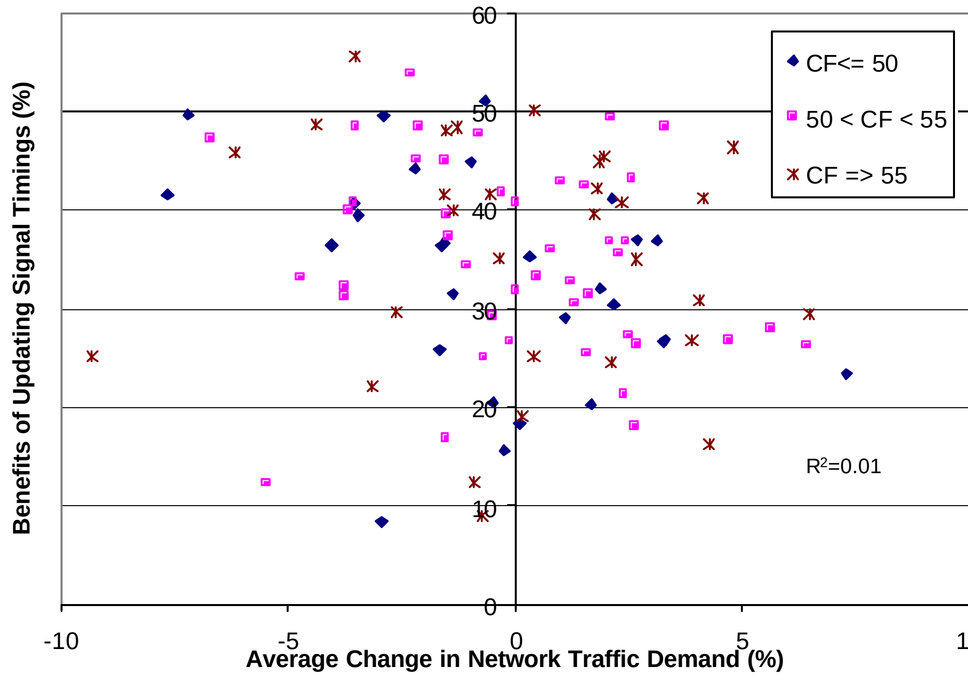


Figure 4.6 - Benefits of Updating Timings vs Average Change in Traffic Demand

4.5, Figure 4.6 shows no correlation between the benefits of signal timings and the CF measure.

4.3 Ageing of Pretimed and Actuated Traffic Controls

This section is divided into two parts. The first part deals with changes in traffic control performance as a result of deterministic changes in traffic demand and distribution. The second subsection deals with stochastic or random changes and their impact on traffic control.

4.3.1 Deterministic Traffic Demand and Distribution Inputs

The first part of this section show the performances of optimized and non-optimized timing plans for various changes in traffic flows. Figure 4.7 shows the

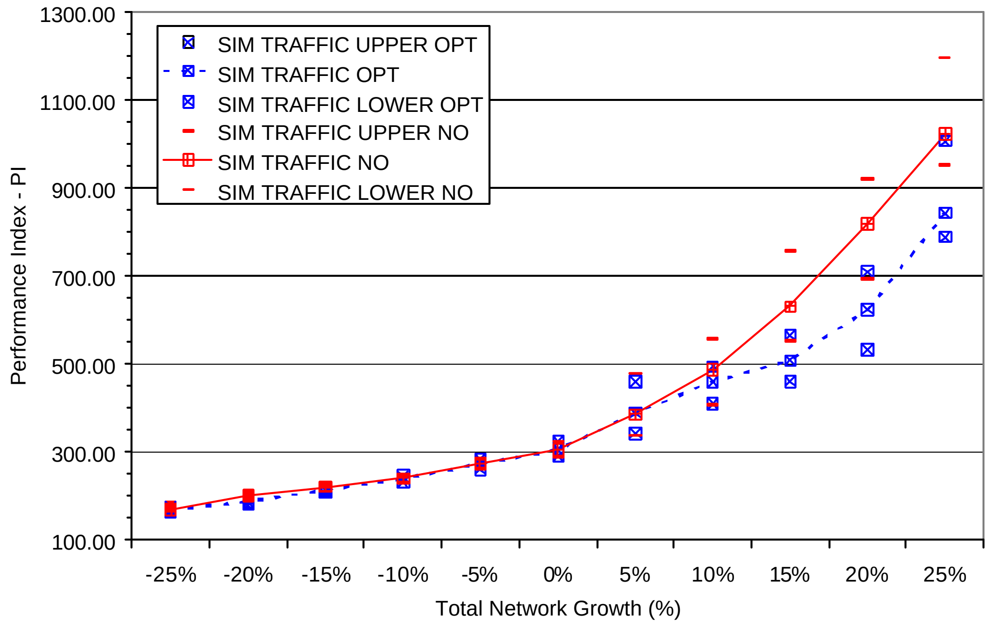


Figure 4.7 – PI vs Total Network Growth for Pretimed Traffic Control

performance index of optimized and nonoptimized timing plans for scenarios representing various levels of traffic demand. For most of the scenarios, optimized timing plans generate smaller a PI (fewer delays and stops) than nonoptimized timing plans. The difference between plans is more accentuated for an increase in traffic demand.

Figure 4.8 shows the performance of optimized and nonoptimized timing plans for scenarios representing various proportions of turning movements at intersection approaches. Optimized timing plans are better (smaller PIs) than nonoptimized timing plans for almost all scenarios. The only exception is a 10% reduction in turning movements where optimized and nonoptimized timings do not yield to statistically different PIs.

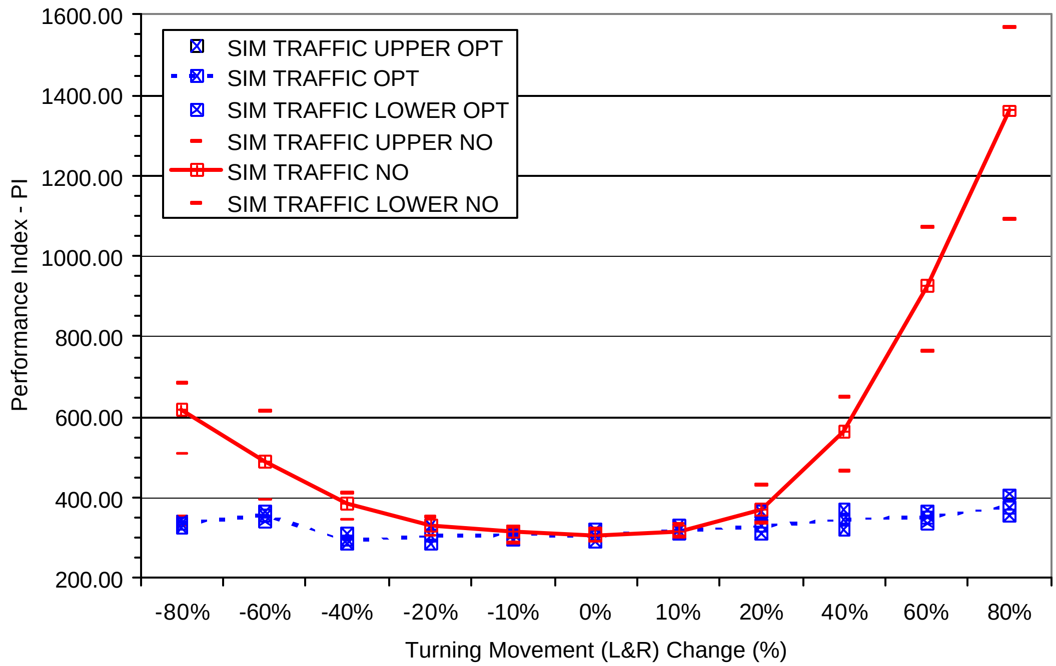


Figure 4.8 - PI vs Change in Turning Movements for Pretimed Traffic Control

Figures 4.9 and 4.10 show the performances of optimized and nonoptimized timing plans for actuated-coordinated traffic control. Figure 4.9 shows the PI's dependence on changes in traffic demand. Much like the results for pretimed traffic control, these results prove the supremacy of optimized plans over nonoptimized plans. The optimized timing plans are not better only for scenarios with a ± 5% change in traffic demand (Table 4.3). In these cases, changes in traffic volumes are so small that they cannot impact the performance of nonoptimized timing plans.

Figure 4.10 shows that optimized timing plans are not better than non-optimized plans for all scenarios where turning movement proportions are reduced. This inconsistency does not come from the fact that optimized timing plans do not

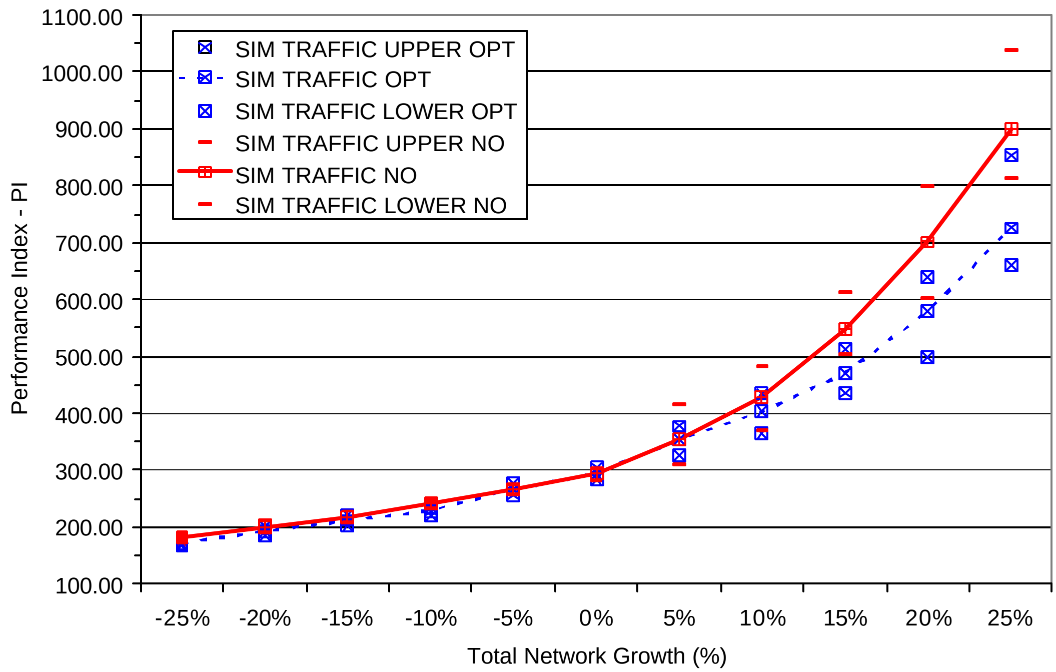


Figure 4.9 - PI vs Total Network Growth for Actuated Traffic Control

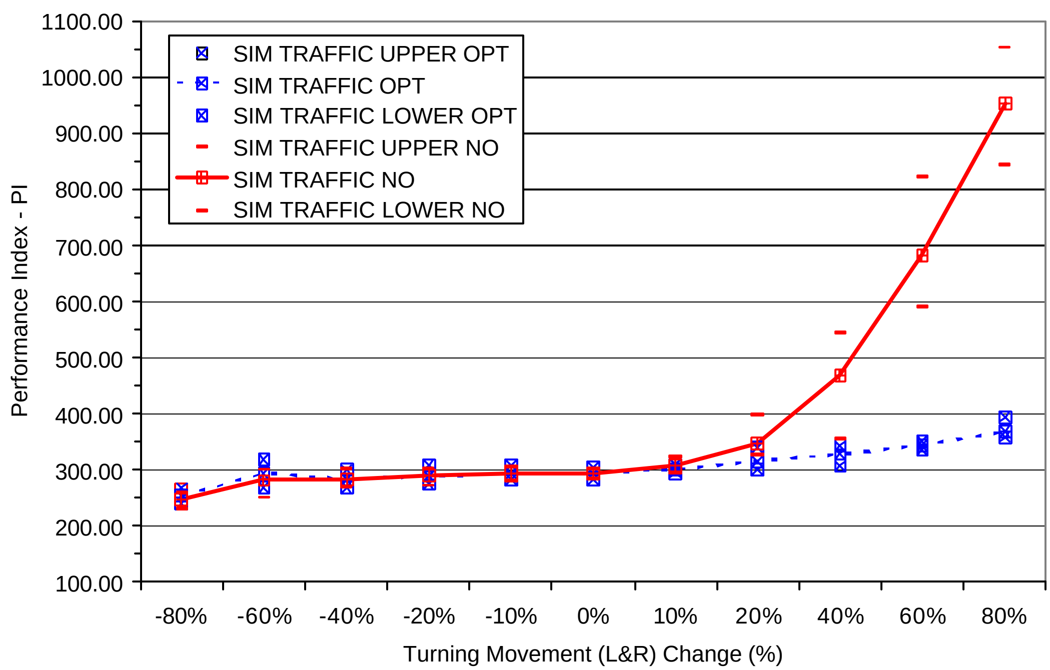


Figure 4.10 - PI vs Change in Turning Movements for Actuated Traffic Control

work well. It comes from the fact that nonoptimized timing plans for actuated control can perform very well for these scenarios.

Statistical testing is necessary to investigate the validity of the differences between optimized and nonoptimized plans. For this purpose, hypothesis tests about the difference between the means of two populations with independent samples are used. Two populations represent all possible PI values from simulation runs for non-optimized (first population) and optimized (second population) signal timing plans. The objective is to test whether the mean of the PIs from the nonoptimized simulation runs is higher than the mean of the PIs from optimized simulation runs.

We set up the null hypothesis mathematically as:

$$H_0: \mu_1 - \mu_2 = 0 \qquad [4.1]$$

With an alternative hypothesis as:

$$H_a: \mu_1 - \mu_2 > 0 \qquad [4.2]$$

Where

μ_1 – Mean PI for population of non-optimized PIs

μ_2 – Mean PI for population of optimized PIs

The tests statistic for the small-sample case ($n_1 = n_2 = 10$) is:

$$t = \frac{(\overline{x_1} - \overline{x_2}) - (\overline{m_1} - \overline{m_2})}{\sqrt{s^2\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}} \qquad [4.3]$$

Where:

x_1 – Mean PI for sample from population of non-optimized PIs

x_2 – Mean PI for sample from population of optimized PIs

s^2 – Estimate of common variance σ^2

In the case of two independent random samples of sizes n_1 and n_2, the *t* distribution will have degrees of freedom:

$$df = n_1 + n_2 - 2 \qquad [4.4]$$

However, the variances of the two populations are not equal ($\sigma_1^2 \neq \sigma_2^2$). If we assume that the populations are normally distributed (which is mostly true), we estimate the standard deviation of the sampling distribution (s_{x1-x2} - estimate of σ_{x1} - σ_{x2}) as:

$$s_{\bar{x}_1 - \bar{x}_2} = \sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}} \qquad [4.5]$$

In this case, the *t* distribution is still used but with degrees of freedom calculated as:

$$df = \frac{[1/n_1 + (s_2^2/s_1^2)/n_2]^2}{[1/n_1^2(n_1 - 1)] + [(s_2^2/s_1^2)^2/n_2^2(n_2 - 1)]} \qquad [4.6]$$

Where:

n_1 – sample size of the first population (population consists of PIs for non-optimized signal timings)

n_2 – sample size of the second population (population consists of PIs for optimized signal timings)

Tables 4.1- 4.4 show the results of hypothesis testing for pretimed traffic control and changes in traffic demand for a 95% level of confidence. The results show that nonoptimized signal timings are greater than optimized for most traffic demand scenarios. If traffic demand changes by ± 5%, this change will not have a

Table 4.1 – Testing for Pretimed SimTraffic Control – Traffic Demand

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-25%	NO	168.87	2.76	1.31	-0.03	17.78	1.73	UPHOLD
	OP	168.88	3.08					
-20%	NO	200.62	5.00	2.33	18.93	17.89	1.73	REJECT
	OP	187.70	5.40					
-15%	NO	221.00	4.83	1.72	11.93	13.54	1.76	REJECT
	OP	214.01	2.51					
-10%	NO	240.79	5.35	2.36	2.75	17.99	1.73	REJECT
	OP	238.89	5.21					
-5%	NO	273.47	7.52	3.24	1.10	17.90	1.73	UPHOLD
	OP	272.58	6.97					
Base	NO	305.37	10.07					
5%	NO	384.88	39.51	16.05	-1.68	17.22	1.74	UPHOLD
	OP	387.89	31.84					
10%	NO	486.89	52.60	19.25	14.28	14.48	1.75	REJECT
	OP	458.87	30.63					
15%	NO	630.39	63.18	23.76	56.23	15.37	1.75	REJECT
	OP	507.81	40.69					
20%	NO	817.20	83.19	32.63	75.84	16.51	1.74	REJECT
	OP	623.47	61.03					
25%	NO	1022.98	72.23	30.60	73.25	17.77	1.73	REJECT
	OP	841.79	64.37					

Table 4.2 – Testing for Pretimed SimTraffic Control – Traffic Distribution

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-80%	NO	616.91	51.59	16.43	154.70	9.26	1.83	REJECT
	OP	336.46	6.22					
-60%	NO	490.38	66.85	21.25	65.20	9.19	1.83	REJECT
	OP	355.94	6.93					
-40%	NO	385.97	20.86	7.00	77.81	11.22	1.80	REJECT
	OP	293.94	7.38					
-20%	NO	330.50	18.58	7.79	20.99	17.67	1.73	REJECT
	OP	304.29	16.20					
-10%	NO	314.70	10.76	4.13	7.86	15.96	1.75	REJECT
	OP	307.56	7.40					
Base	NO	305.37	10.07					
10%	NO	316.01	9.79	3.91	-2.26	16.92	1.74	UPHOLD
	OP	318.01	7.56					
20%	NO	369.08	27.77	10.89	27.03	16.50	1.74	REJECT
	OP	329.20	20.35					
40%	NO	566.37	57.01	18.74	114.06	10.44	1.81	REJECT
	OP	345.57	16.17					
60%	NO	926.37	96.07	30.53	232.25	9.18	1.83	REJECT
	OP	352.44	9.69					
80%	NO	1361.55	131.23	41.82	340.45	9.28	1.83	REJECT
	OP	376.94	16.37					

Table 4.3 – Testing for Actuated SimTraffic Control – Traffic Demand

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-25%	NO	181.42	3.19	1.35	18.92	17.78	1.73	REJECT
	OP	171.58	2.85					
-20%	NO	199.24	6.15	2.57	8.92	17.63	1.73	REJECT
	OP	192.84	5.31					
-15%	NO	218.52	5.68	2.40	9.69	17.76	1.73	REJECT
	OP	211.80	5.05					
-10%	NO	240.87	5.00	2.10	18.72	17.68	1.73	REJECT
	OP	228.74	4.37					
-5%	NO	266.23	6.62	2.82	0.32	17.83	1.73	UPHOLD
	OP	265.99	6.00					
Base	NO	294.29	6.81					
5%	NO	354.09	27.90	9.83	0.32	13.09	1.77	UPHOLD
	OP	353.65	13.68					
10%	NO	427.70	37.52	14.04	14.96	15.20	1.75	REJECT
	OP	402.64	23.72					
15%	NO	548.24	37.40	14.64	45.49	16.47	1.75	REJECT
	OP	470.39	27.29					
20%	NO	701.12	65.40	25.58	53.78	16.45	1.75	REJECT
	OP	579.47	47.61					
25%	NO	901.02	66.93	28.20	73.74	17.72	1.73	REJECT
	OP	725.89	58.93					

Table 4.4 – Testing for Actuated SimTraffic Control – Traffic Distribution

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-80%	NO	248.55	9.24	4.36	-5.03	17.82	1.73	UPHOLD
	OP	253.24	10.22					
-60%	NO	281.44	16.63	7.34	-10.59	17.99	1.73	UPHOLD
	OP	294.27	16.18					
-40%	NO	282.65	9.74	4.54	-0.82	17.89	1.73	UPHOLD
	OP	283.44	10.52					
-20%	NO	290.28	8.77	4.48	1.58	17.08	1.74	UPHOLD
	OP	288.79	11.11					
-10%	NO	293.32	7.76	3.52	0.49	17.98	1.73	UPHOLD
	OP	292.91	8.00					
Base	NO	294.29	6.81					
10%	NO	308.33	8.82	3.26	8.59	14.85	1.75	REJECT
	OP	301.39	5.36					
20%	NO	347.41	22.72	8.50	24.23	15.20	1.75	REJECT
	OP	315.82	14.35					
40%	NO	468.29	55.52	18.01	74.01	9.93	1.81	REJECT
	OP	327.84	12.66					
60%	NO	682.20	84.91	26.90	145.90	9.07	1.83	REJECT
	OP	343.76	5.25					
80%	NO	953.07	62.42	20.01	292.19	9.49	1.83	REJECT
	OP	368.61	10.28					

significant impact on the performance of the traffic control. This is also the case for low levels of traffic demand (-25%). In this case, traffic volumes are so low that the benefits of retiming signals are negligible.

Figures 4.11 – 4.14 show the relationship between the benefits of updating timing plans and four different types of changes in link flows for pretimed and actuated-coordinated control regimes. These benefits are calculated according to Equation [3.1].

Figure 4.11 shows how the benefits of updating timing plans are related to a decrease in traffic demand. The benefits are higher for actuated traffic control than for pretimed control. For both of these control regimes, one could save around 2% in delay and stops by retiming traffic signals for every 10% decrease in traffic demand. Figure 4.12 shows the impact of the increase in traffic demand on the benefits of

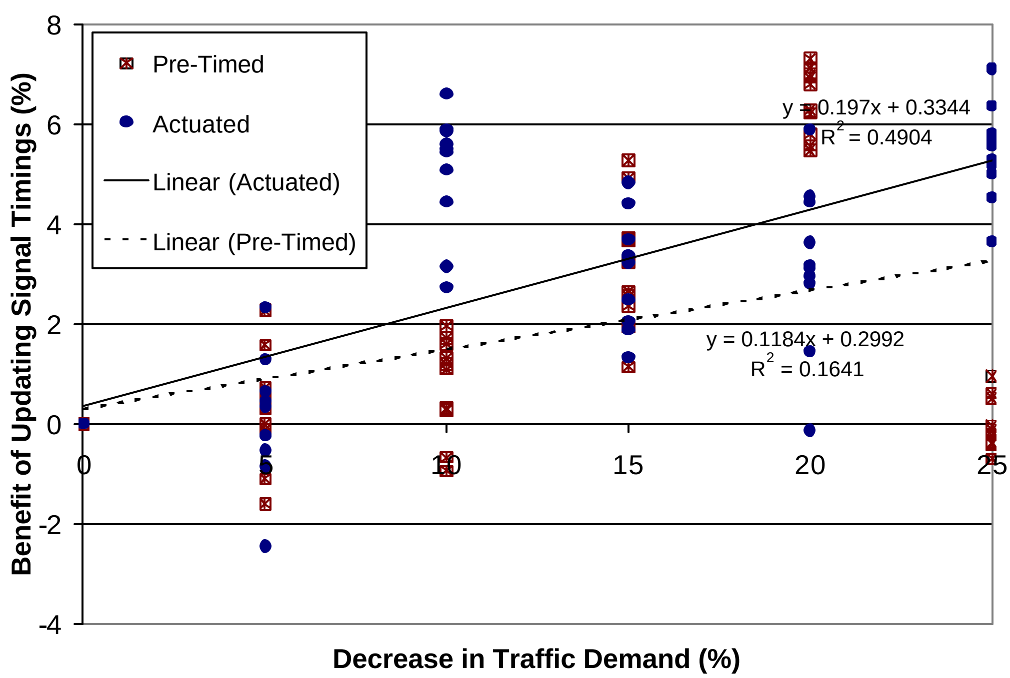


Figure 4.11 - Benefits of Updating Timings vs Decrease in Traffic Demand

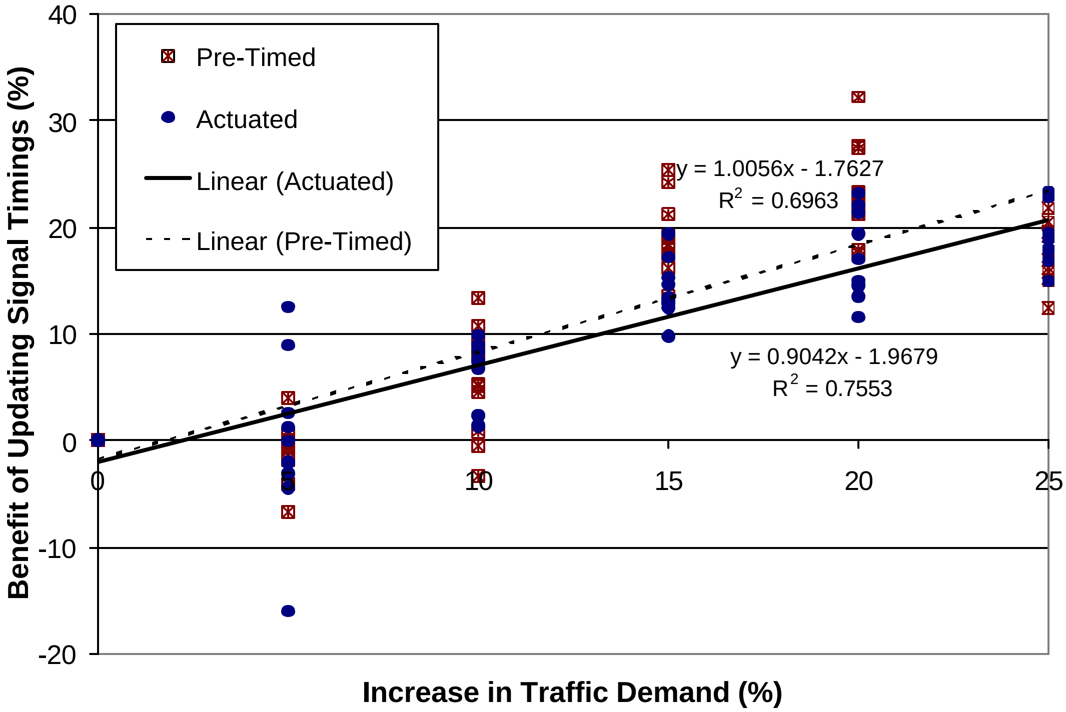


Figure 4.12 - Benefits of Updating Timings vs Increase in Traffic Demand

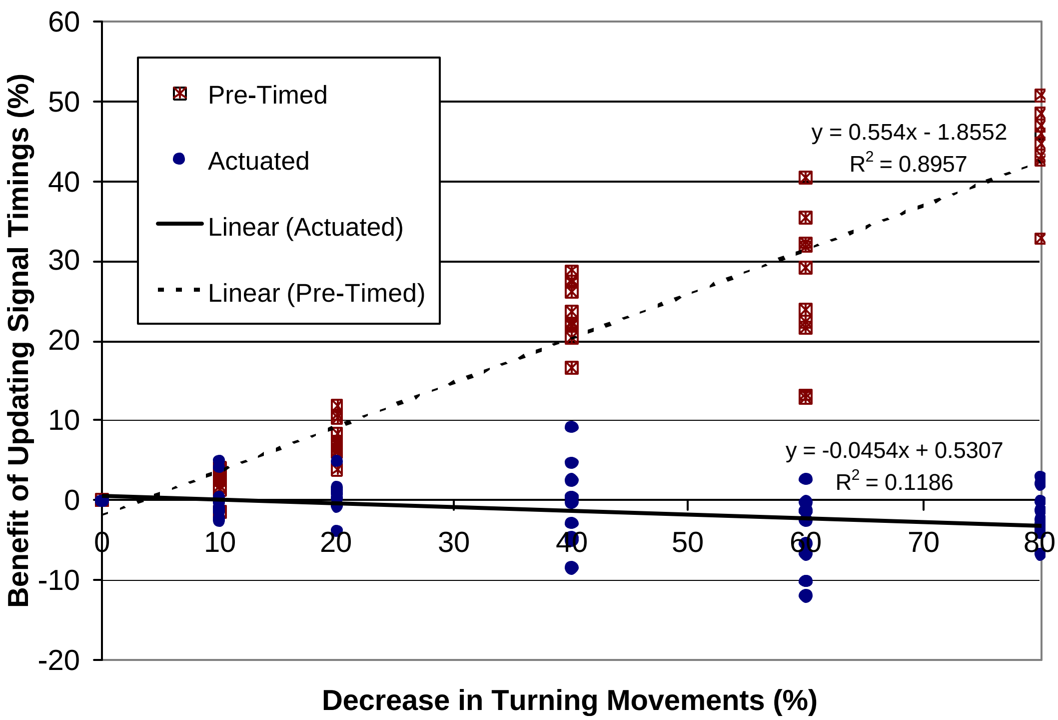


Figure 4.13 - Benefits of Updating Timings vs Decrease in Turning Movements

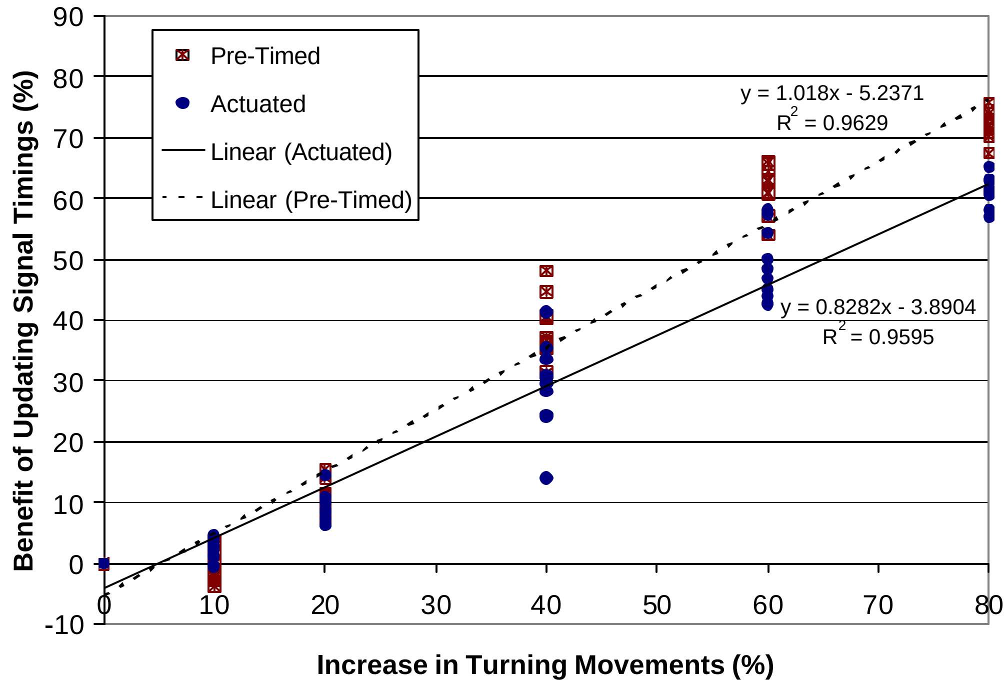


Figure 4.14 - Benefits of Updating Timings vs Increase in Turning Movements

updating signal timings. The figure shows that the benefits increase and that the increase is higher for pretimed control than for actuated-coordinated control.The results show a 10% increase in benefits for each 10% increase in uniform traffic demand over the entire network.

Figure 4.13 shows the relationship between the benefits of updating and the decrease in turning movement proportions. A decrease in turning movements of 10% yields around a 5% increase in the benefits of retiming pretimed traffic control. However, the benefits of retiming actuated-coordinated control are negligible. The reasons for this difference are explained in the Discussion chapter of the dissertation.

Finally, the relationship between the benefits of updating and the uniform increase in turning movements is shown in Figure 4.14. An increase in turning movements of 10% yields around an 8% increase in the benefits of retiming traffic signals. The results show that the benefits of updating can be as high as 75%. The benefits obtained after updating pretimed control are again higher than those of actuated-coordinated control.

4.3.2 Stochastic Traffic Demand and Distribution Inputs

Figure 4.15 shows the results from the experiments with stochastic changes both in traffic demand and turning movements. The results from the stochastic experiments have shown that, regardless of the average change in network traffic demand, there is an average benefit of 35% in terms of delay and number of stops from updating pretimed signal timing plans. Similarly, the average benefit of retiming actuated-coordinated signal timing plans is around 27%.

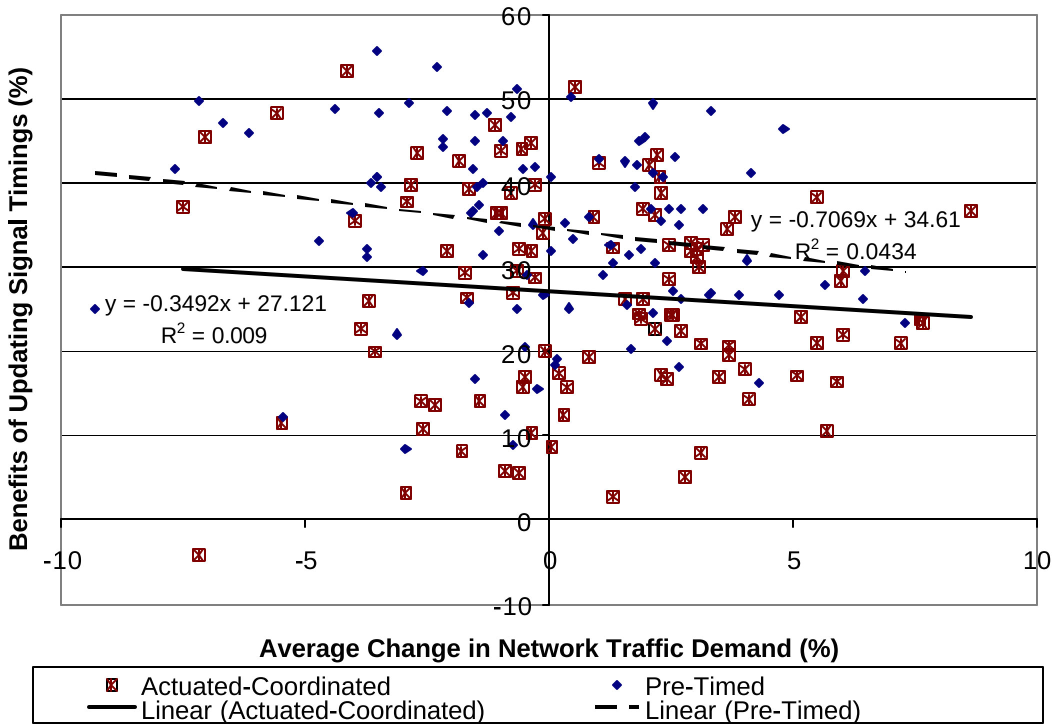


Figure 4.15 - Benefits of Updating Timings vs Change in Traffic Demand

4.4 Ageing of the SCOOT Adaptive Control Regime

The results for the ageing of SCOOT adaptive control are divided into two subsections. The first presents the results for ageing when only traffic demand has been changed (scenarios 1 to 11). The second part presents the findings when traffic demand is constant and the proportions of turning movements are changed. Both subsections first present the results for pretimed control in VISSIM, which is a "base case" used to compute the ageing of the SCOOT control.

4.4.1 Ageing of the SCOOT Control Regime for Changes in Traffic Demand

Figure 4.16 presents performance indexes from VISSIM for SCOOT-derived and pretimed signal timings. The results for two types of pretimed plans are presented in Figure 4.16: optimized and nonoptimized. VISSIM calibration in Chapter 3 explains how VISSIM's settings were adjusted to endorse signal timings developed in Synchro. The results presented in Figure 4.16 (curves VISSIM NO and VISSIM OP) show that optimized plans perform better than nonoptimized for the changed VISSIM settings.

The results from Figure 4.16 need formal statistical evaluation. The hypotheses presented in section 4.3.1 are used to test whether optimized performance is better than nonoptimized performance. The null hypothesis, stating that the mean of non-optimized PIs is smaller or equal to the mean of optimized PIs, is tested, as described earlier in 4.3.1. Table 4.5 shows the results of the hypothesis testing. One can see that

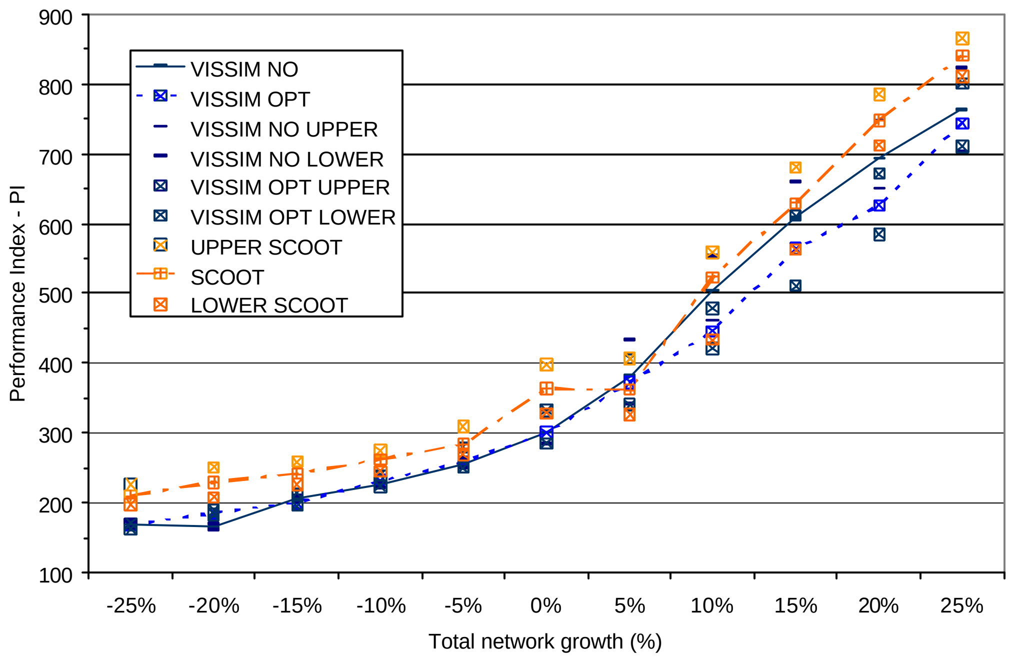


Figure 4.16 – PI vs Traffic Growth for SCOOT and Pretimed Traffic Control

Table 4.5 – Testing for Pretimed VISSIM Control – Traffic Demand

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-25%	NO	168.65	2.88	1.24	1.30	17.90	1.734	UPHOLD
	OP	168.00	2.67					
-20%	NO	165.38	2.62	0.99	-44.55	15.46	1.753	UPHOLD
	OP	185.18	1.70					
-15%	NO	205.39	2.96	1.85	8.23	14.50	1.761	REJECT
	OP	200.38	5.06					
-10%	NO	225.90	2.74	1.77	-9.28	14.12	1.761	UPHOLD
	OP	231.42	4.90					
-5%	NO	256.07	4.08	3.51	-2.60	11.74	1.782	UPHOLD
	OP	258.25	10.33					
Base	NO	300.34	15.47					
5%	NO	382.17	32.05	12.52	6.44	16.41	1.746	REJECT
	OP	371.98	23.24					
10%	NO	503.98	30.75	11.53	38.10	15.26	1.753	REJECT
	OP	446.13	19.57					
15%	NO	609.42	36.72	16.06	25.32	17.96	1.734	REJECT
	OP	564.04	35.10					
20%	NO	693.51	35.95	15.08	38.78	17.67	1.734	REJECT
	OP	626.16	31.33					
25%	NO	764.34	42.36	15.96	11.28	15.43	1.753	REJECT
	OP	744.18	27.46					

for the lower changes in traffic demand, optimized timing plans are not better than nonoptimized timing plans (scenarios of 25, 20, 10, and 5 % of decrease in traffic demand). However, when demand grows, it becomes clear that optimized plans outperform nonoptimized plans.

The signal timings imported from Synchro to VISSIM do not represent the best timing plans for VISSIM simulations. However, for lack of a better way to find optimal timing plans for VISSIM simulations, these plans are adopted to evaluate the ageing of SCOOT traffic control. The next sections will show that the quality of these plans does not play a crucial role in judging SCOOT's ageing.

The performance of SCOOT control in VISSIM is shown by the "SCOOT" curve in Figure 4.16. The SCOOT control performs worse than any pretimed controls for most of the traffic demand scenarios.

The validity of the differences between SCOOT and optimized signal timing plans is tested. For this purpose, hypothesis tests about the difference between the means of two populations with independent samples are used. Two populations represent all possible PI values from simulation runs for SCOOT (first population) and optimized (second population) traffic control regimes. There are two objectives. The first objective is to test whether SCOOT control performs worse than optimized pretimed control. Mathematically, we set up the null hypothesis as:

$$H_0: \mu_1 - \mu_2 = 0 \qquad [4.7]$$

With an alternative hypothesis as:

$$H_a: \mu_1 - \mu_2 > 0 \qquad [4.8]$$

Where

μ_1 – Mean PI for population of SCOOT PIs

μ_2 – Mean PI for population of optimized PIs

The tests statistic for the small-sample case ($n_1 = n_2 = 10$) is:

$$t = \frac{(\overline{x_1} - \overline{x_2}) - (\overline{m_1} - \overline{m_2})}{\sqrt{s^2\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}} \qquad [4.9]$$

Where:

x_1 – Mean PI for sample from population of SCOOT PIs

x_2 – Mean PI for sample from population of optimized PIs

s^2 – Estimate of common variance σ^2

and $\overline{m_1} - \overline{m_2} = 0$ [4.10]

Similar to previous tests:

$$s^2 = s_{\overline{x_1} - \overline{x_2}} = \sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}} \qquad [4.11]$$

and

$$df = \frac{[1/n_1 + (s_2^2/s_1^2)/n_2]^2}{[1/n_1^2(n_1 - 1)] + [(s_2^2/s_1^2)^2/n_2^2(n_2 - 1)]} \qquad [4.12]$$

Table 4.6 shows the results of testing this hypothesis for 11 scenarios of changes in traffic demand. SCOOT performs better than optimized pretimed control only for the scenario of a 5% increase in traffic demand.

Table 4.6 – Testing for SCOOT and Pretimed Control – Traffic Demand

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-25%	PT_OP	168.00	2.67	3.28	-51.65	10.27	1.81	UPHOLD
	SCOOT	209.86	10.04					
-20%	PT_OP	165.38	2.62	4.21	-69.89	9.72	1.81	UPHOLD
	SCOOT	229.55	13.06					
-15%	PT_OP	200.38	5.06	3.46	-49.80	13.56	1.76	UPHOLD
	SCOOT	241.82	9.71					
-10%	PT_OP	231.42	4.90	3.18	-38.59	14.12	1.76	UPHOLD
	SCOOT	262.18	8.77					
-5%	PT_OP	258.25	10.33	5.00	-24.92	17.63	1.73	UPHOLD
	SCOOT	283.16	11.96					
Base	PT_OP	300.34	15.47	9.31	-46.24	14.98	1.75	UPHOLD
	SCOOT	363.45	25.07					
5%	PT_OP	371.98	23.24	10.84	6.81	17.88	1.73	REJECT
	SCOOT	361.95	25.20					
10%	PT_OP	446.13	19.57	13.05	-47.69	13.82	1.76	UPHOLD
	SCOOT	523.19	36.34					
15%	PT_OP	564.04	35.10	15.97	-36.62	17.98	1.73	UPHOLD
	SCOOT	629.49	36.30					
20%	PT_OP	626.16	31.33	11.79	-79.56	15.39	1.75	UPHOLD
	SCOOT	748.35	20.22					
25%	PT_OP	744.18	27.46	10.74	-66.00	16.44	1.75	UPHOLD
	SCOOT	840.88	19.97					

However, the results of Table 4.6 still do not prove that SCOOT is ageing. In order to show the existence of ageing, another test was performed. The objective of the second test is to show how SCOOT performance degrades when compared with SCOOT performance for the base traffic condition. For the base conditions, SCOOT is worse than optimized pretimed control. The difference in mean PIs for SCOOT and optimized pretimed control is equal (Figure 4.16 and Table 4.6):

$$\overline{m_1} - \overline{m_2} = 363.45 - 300.34 = 63.11 \qquad [4.13]$$

If SCOOT does not age, this difference should remain the same or become smaller. Therefore, a null hypothesis is set up so that rejecting the hypothesis means that SCOOT ages.

Mathematically, we set up the null hypothesis as:

$$H_0: \mu_1 - \mu_2 = 63.11 \qquad [4.14]$$

With an alternative hypothesis as:

$$H_a: \mu_1 - \mu_2 > 63.11 \qquad [4.15]$$

Where

μ_1 – Mean PI for population of SCOOT PIs

μ_2 – Mean PI for population of optimized PIs

The tests statistic for the small-sample case ($n_1 = n_2 = 10$) is:

$$t = \frac{(\overline{x_1} - \overline{x_2}) - (\overline{m_1} - \overline{m_2})}{\sqrt{s^2\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}} \qquad [4.16]$$

Where:

x_1 – Mean PI for sample from population of SCOOT PIs

x_2 – Mean PI for sample from population of optimized PIs

s^2 – Estimate of common variance σ^2

and $\overline{m_1} - \overline{m_2} = 63.11$ [4.17]

Similar to previous tests:

$$s^2 = s_{\overline{x_1} - \overline{x_2}} = \sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}} \qquad [4.18]$$

and

$$df = \frac{[1/n_1 + (s_2^2/s_1^2)/n_2]^2}{[1/n_1^2(n_1 - 1)] + [(s_2^2/s_1^2)^2/n_2^2(n_2 - 1)]} \qquad [4.19]$$

Table 4.7 shows the results of testing the hypothesis that SCOOT does not age. The results show that SCOOT ages for only a few scenarios representing increases in traffic demand (10, 20, and 25%). However, upholding the hypothesis for other scenarios does not mean that SCOOT's performance is not changing unpredictably. The results are more a consequence of the way the hypothesis is set up than of the SCOOT's ability to provide changed but steady traffic performance. More discussion on these results is given in the Discussion chapter of the dissertation.

4.4.2 Ageing of SCOOT Control Regime for Changes in Traffic Distribution

Figure 4.17 shows the results of the experiments in which traffic distribution has been changed. First, the outputs from pretimed control are discussed. Optimized timing plans yield a lower performance index than nonoptimized timing plans. Again, the hypothesis that nonoptimized plans are better than optimized plans is tested. The results of the tests are shown in Table 4.8. Except for a 10% increase in left and right

Table 4.7 – Testing Ageing of SCOOT Control – Traffic Demand

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-25%	PT_OP	168.00	2.67	3.28	-26.22	10.27	1.81	UPHOLD
	SCOOT	209.86	10.04					
-20%	PT_OP	165.38	2.62	4.21	1.15	9.72	1.81	UPHOLD
	SCOOT	229.55	13.06					
-15%	PT_OP	200.38	5.06	3.46	-26.04	13.56	1.76	UPHOLD
	SCOOT	241.82	9.71					
-10%	PT_OP	231.42	4.90	3.18	-40.59	14.12	1.76	UPHOLD
	SCOOT	262.18	8.77					
-5%	PT_OP	258.25	10.33	5.00	-38.21	17.63	1.73	UPHOLD
	SCOOT	283.16	11.96					
Base	PT_OP	300.34	15.47	9.31	0.00	14.98	1.76	UPHOLD
	SCOOT	363.45	25.07					
5%	PT_OP	371.98	23.24	10.84	-49.67	17.88	1.73	UPHOLD
	SCOOT	361.95	25.20					
10%	PT_OP	446.13	19.57	13.05	8.63	13.82	1.76	REJECT
	SCOOT	523.19	36.34					
15%	PT_OP	564.04	35.10	15.97	1.31	17.98	1.73	UPHOLD
	SCOOT	629.49	36.30					
20%	PT_OP	626.16	31.33	11.79	38.47	15.39	1.75	REJECT
	SCOOT	748.35	20.22					
25%	PT_OP	744.18	27.46	10.74	22.93	16.44	1.75	REJECT
	SCOOT	840.88	19.97					

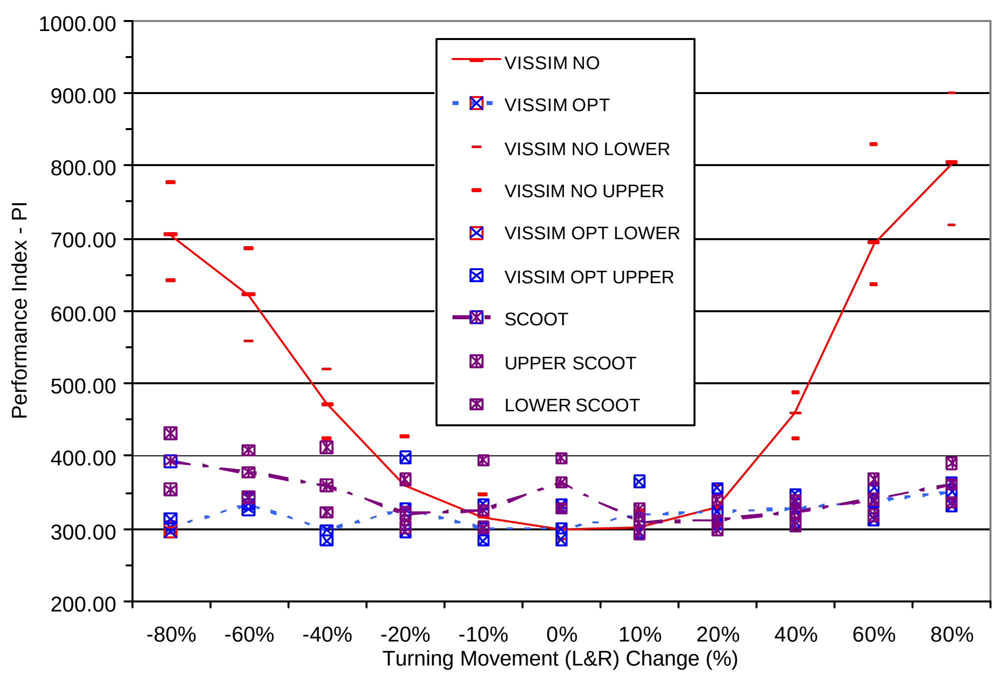


Figure 4.17 – PI vs Turning Movements for SCOOT and Pretimed Controls

Table 4.8 – Testing for Pretimed VISSIM Control – Traffic Distribution

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-80%	NO	704.90	37.56	12.02	259.95	9.44	1.833	REJECT
	OP	301.85	5.85					
-60%	NO	622.34	42.36	13.48	175.91	9.24	1.833	REJECT
	OP	333.47	4.87					
-40%	NO	472.25	36.16	12.19	111.33	11.42	1.796	REJECT
	OP	298.39	13.39					
-20%	NO	360.69	37.23	15.35	19.11	17.46	1.74	REJECT
	OP	327.21	31.16					
-10%	NO	316.36	19.01	7.77	12.78	17.32	1.74	REJECT
	OP	300.43	15.56					
Base	NO	300.34	15.47					
10%	NO	303.20	12.00	7.97	-12.57	13.86	1.761	UPHOLD
	OP	319.07	22.15					
20%	NO	328.95	15.48	7.06	4.55	17.97	1.734	REJECT
	OP	323.54	16.09					
40%	NO	458.74	18.35	7.19	109.33	16.49	1.746	REJECT
	OP	327.64	13.43					
60%	NO	694.58	56.63	18.43	185.66	10.06	1.812	REJECT
	OP	338.13	13.77					
80%	NO	804.61	64.84	20.83	222.42	9.57	1.812	REJECT
	OP	350.67	11.54					

turning movements, the optimized timing plans always yield a smaller performance index to the nonoptimized timing plans.

In general, the SCOOT control is worse than optimized pretimed control when turning movements are decreased (and through movements are increased). On the other hand, for increased turning movements, the SCOOT control yields PIs that are comparable to the ones from optimized pretimed control.

The hypothesis that SCOOT performance is worse than optimized performance is set as a null hypothesis. Table 4.9 shows the results of testing this hypothesis. The hypothesis is rejected (which means SCOOT is better than optimized control) for

Table 4.9 – Testing for SCOOT and Pretimed Control – Traffic Distribution

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-80%	PT_OP	301.85	5.85	8.87	-68.59	9.82	1.81	UPHOLD
	SCOOT	393.20	27.43					
-60%	PT_OP	333.47	4.87	7.28	-36.57	9.84	1.81	UPHOLD
	SCOOT	377.60	22.51					
-40%	PT_OP	298.39	13.39	9.70	-43.69	13.02	1.77	UPHOLD
	SCOOT	359.24	27.59					
-20%	PT_OP	327.21	31.16	11.84	4.18	15.67	1.75	REJECT
	SCOOT	320.78	20.75					
-10%	PT_OP	300.43	15.56	10.88	-17.00	13.34	1.77	UPHOLD
	SCOOT	325.52	30.69					
Base	PT_OP	300.34	15.47	9.31	-46.24	14.98	1.75	UPHOLD
	SCOOT	363.45	25.07					
10%	PT_OP	319.07	22.15	7.85	8.39	13.34	1.77	REJECT
	SCOOT	308.56	11.23					
20%	PT_OP	323.54	16.09	6.60	10.19	17.38	1.74	REJECT
	SCOOT	311.83	13.30					
40%	PT_OP	327.64	13.43	5.22	4.77	16.30	1.75	REJECT
	SCOOT	322.76	9.61					
60%	PT_OP	338.13	13.77	7.11	-2.00	16.94	1.74	UPHOLD
	SCOOT	340.52	17.79					
80%	PT_OP	350.67	11.54	7.35	-8.89	14.33	1.76	UPHOLD
	SCOOT	361.45	20.16					

scenarios with a 20% decrease in turning movements and 10%, 20%, and 40% increase in turning movements.

As for changes in traffic demands, the ageing of SCOOT is tested based on SCOOT performance for base traffic conditions. The same difference in mean PIs for SCOOT and optimized pretimed control ($\overline{m_1} - \overline{m_2} = 63.11$) is used to test the ageing of SCOOT control. Table 4.10 shows the results of testing the null hypothesis, which is set up in such a way that rejecting the hypothesis means that SCOOT ages. The results show that SCOOT ages only for the highest decrease in turning movements (80%).

Table 4.10 – Testing Ageing for SCOOT – Traffic Distribution

Scenario	Control	Mean PI	St. Dev	S_{12}	$T_{statistic}$	Df	T_{table}	H_0
-80%	PT_OP	301.85	5.85	8.87	21.21	9.82	1.81	REJECT
	SCOOT	393.20	27.43					
-60%	PT_OP	333.47	4.87	7.28	-15.72	9.84	1.81	UPHOLD
	SCOOT	377.60	22.51					
-40%	PT_OP	298.39	13.39	9.70	-1.62	13.02	1.77	UPHOLD
	SCOOT	359.24	27.59					
-20%	PT_OP	327.21	31.16	11.84	-45.19	15.67	1.75	UPHOLD
	SCOOT	320.78	20.75					
-10%	PT_OP	300.43	15.56	10.88	-25.78	13.34	1.77	UPHOLD
	SCOOT	325.52	30.69					
Base	PT_OP	300.34	15.47	9.31	-46.24	14.98	1.75	UPHOLD
	SCOOT	363.45	25.07					
10%	PT_OP	319.07	22.15	7.85	-58.74	13.34	1.77	UPHOLD
	SCOOT	308.56	11.23					
20%	PT_OP	323.54	16.09	6.60	-65.11	17.38	1.74	UPHOLD
	SCOOT	311.83	13.30					
40%	PT_OP	327.64	13.43	5.22	-66.53	16.30	1.75	UPHOLD
	SCOOT	322.76	9.61					
60%	PT_OP	338.13	13.77	7.11	-50.90	16.94	1.74	UPHOLD
	SCOOT	340.52	17.79					
80%	PT_OP	350.67	11.54	7.35	-43.17	14.33	1.76	UPHOLD
	SCOOT	361.45	20.16					

For all other scenarios, SCOOT does not perform worse than it performs for the base traffic conditions (for which it is calibrated). Results of SCOOT performance for all scenarios are provided in Table 4.11. The results show that when SCOOT ages, its ageing (Equation [3.5]) can be anywhere from 20 to 50 %.

4.5 Summary of Results

The results from the simulation experiments were presented in this chapter. The results validate the general methodology used to assess the ageing of traffic control regimes. However, when evaluating the ageing process, a universal measure of changes in traffic flows, as used in Bell's study (1985), is not a good indicator of

Table 4.11 – Overall SCOOT Performance

Scenario	Control	Mean PI	Difference	SCOOT vs. PT_OP	Ageing	γ (% Ageing)
Base	PT_OP	300.34	63.11	NA	NA	NA
	SCOOT	363.45				
-25% TD	PT_OP	168	41.86	Worse	No Ageing	NA
	SCOOT	209.86				
-20% TD	PT_OP	165.38	64.17	Worse	No Ageing	NA
	SCOOT	229.55				
-15% TD	PT_OP	200.38	41.44	Worse	No Ageing	NA
	SCOOT	241.82				
-10% TD	PT_OP	231.42	30.76	Worse	No Ageing	NA
	SCOOT	262.18				
-5% TD	PT_OP	258.25	24.91	Worse	No Ageing	NA
	SCOOT	283.16				
5% TD	PT_OP	371.98	-10.03	Better	No Ageing	NA
	SCOOT	361.95				
10% TD	PT_OP	446.13	77.06	Worse	Ageing	18.1
	SCOOT	523.19				
15% TD	PT_OP	564.04	65.45	Worse	No Ageing	NA
	SCOOT	629.49				
20% TD	PT_OP	626.16	122.19	Worse	Ageing	48.35
	SCOOT	748.35				
25% TD	PT_OP	744.18	96.7	Worse	Ageing	34.73
	SCOOT	840.88				
-80% TM	PT_OP	301.85	91.35	Worse	Ageing	30.91
	SCOOT	393.2				
-60% TM	PT_OP	333.47	44.13	Worse	No Ageing	NA
	SCOOT	377.6				
-40% TM	PT_OP	298.39	60.85	Worse	No Ageing	NA
	SCOOT	359.24				
-20% TM	PT_OP	327.21	-6.43	Better	No Ageing	NA
	SCOOT	320.78				
-10% TM	PT_OP	300.43	25.09	Worse	No Ageing	NA
	SCOOT	325.52				
10% TM	PT_OP	319.07	-10.51	Better	No Ageing	NA
	SCOOT	308.56				
20% TM	PT_OP	323.54	-11.71	Better	No Ageing	NA
	SCOOT	311.83				
40% TM	PT_OP	327.64	-4.88	Better	No Ageing	NA
	SCOOT	322.76				
60% TM	PT_OP	338.13	2.39	Worse	No Ageing	NA
	SCOOT	340.52				
80% TM	PT_OP	350.67	10.78	Worse	No Ageing	NA
	SCOOT	361.45				

changes in traffic flows.

Evaluation of ageing for pretimed and actuated traffic control regimes show that both of these regimes deteriorate with changes in traffic flows. The benefits of retiming signal timing plans, which come as a consequence of the deterioration process, show that updating pretimed traffic control is more beneficial than updating actuated traffic control. Assessment of SCOOT ageing shows that performance of this control regime highly fluctuates with changes in traffic flows. The ageing process is recognized and quantified based on performance for the initial condition. The SCOOT control performs worse than optimized pretimed control for most of the scenarios but it ages only for few extreme scenarios.

CHAPTER 5

DISCUSSION

This chapter discusses the results presented in the previous chapter. First, the general methodology used to assess the ageing of traffic control is discussed. Next, the ageing measure used by Bell is discussed, along with the reason for its failure to provide a reliable measure of aged traffic flows. The third part of this chapter discusses the results of assessed ageing of pretimed and actuated traffic control regimes. Specific reasons for the particular results are provided. The last part of the chapter discusses the ageing of the SCOOT control regimes.

5.1 Methodology for Assessing the Ageing of Traffic Control Regimes

The methodology developed to assess the ageing of different traffic control regimes for various changes in traffic flows works well for most cases. Figure 4.1 proves the idealized concept of ageing which was introduced at the beginning of the Methodology chapter (Figure 3.1). However, selection of appropriate tools for assessing ageing reveals some deficiencies in the concept. In order to have a clear distinction between performance of optimized and nonoptimized timing plans, one should use a microsimulation software that endorses use of these plans. This is often not the case. Only a few microsimulators endorse optimized timing plans from macroscopic optimization tools like Synchro and TRANSYT-7F.

Although adjustment of microsimulators to endorse the use of macroscopically optimized timing plans seems like the only feasible approach, this is not the case. Adjustment of the microsimulation is a tedious and complex process which, in the end, might not give good results. There are two reasons for this. First, traffic models embedded into microsimulations are much more complex and are founded on stochasticity of traffic flows. Therefore, it is a question of whether they can be properly adjusted at all. Second, the MOEs used to express the performance of macro and micro simulators are often not in accordance. If the macroscopically optimized timing plans were going to be endorsed by microsimulation tools, the MOEs should be adjusted as well.

The particular difficulties of this study are associated with an inability to validate use of Synchro's actuated-coordinated timing plans in VISSIM. Once validated to endorse pretimed plans for assessing the ageing of the SCOOT control, VISSIM could not be adjusted to endorse actuated-coordinated timing plans. Future research should go in the direction of developing VISSIM microscopic optimization (similar to Direct CORSIM optimization). If the microscopic VISSIM optimization had been available, it would be possible to compare pretimed, actuated, and SCOOT control with more accuracy.

Another deficiency in the methodology is associated with the SCOOT ageing concept. The ageing of SCOOT is based on the difference between SCOOT control and pretimed control for a base traffic condition. In this particular study, the difference for the base condition was quite high. The ageing of SCOOT was defined as a situation where the difference (between SCOOT and pretimed control) for some new traffic conditions exceeds the base difference. This did not occur many times for the given traffic conditions, making SCOOT look good. If this difference had been smaller for the base traffic conditions (which might happen for other network and traffic configurations), then SCOOT would age more.

5.2 Reliability of the Ageing Measure for Changed Traffic Flows

The concept of the CF ageing measure is proven to be an unreliable indicator of the possible benefits of updating signal timings. The CF concept used in Bell's study (1985) and in this research is based on the absolute differences in link flows for existing and aged traffic flows.

When used to identify the relationship between deterministic changes in traffic flows and the benefits of updating timing plans, the CF correlates well with the benefits of retiming (Figures 4.2 to 4.5). However, deterministic changes in traffic flows are very rare on real road networks. When several simultaneous random variations are introduced in traffic flows, the CF fails to correlate the benefits of retiming signal plans. One can clearly see from Figure 4.6 that various CF measures are scattered all over the graph, covering various ranges of signal retiming benefits. Figures 4.2 to 4.5 show that the same CF values for various deterministic experiments yielded different benefits of signal retiming. This finding shows that the CF, as a unique measure of changes in traffic flows, cannot be used to correlate these changes with the benefits of signal retiming as in Bell's study (1985).

The major reason for the CF's inability to reflect ageing of traffic signal timing plans lies in its definition. By definition, CF equally accounts for changes in left, through and right movements. In reality, however, an additional vehicle in the left turn lane could cause much more congestion than the through or right moving vehicle. Therefore, the benefits of updating signals for the additional left turn vehicle could be much higher than for the other two cases.

5.3 Ageing of Pretimed and Actuated Traffic Control Regimes

The next subsections discuss the results of the ageing of pretimed and actuated traffic controls presented in section 4.3 of the previous chapter.

5.3.1 Deterministic Traffic Demand and Distribution Scenarios

5.3.1.1 Decrease in Traffic Demand

5.3.1.1.1 *Pretimed Traffic Control.* The results for the impact of a decrease in traffic demand on the pretimed traffic control regime are shown in Figures 4.7, and 4.11 and Table 4.1. These results were also previously shown in Figure 4.2. The results show that optimized timing plans yield a smaller PI than nonoptimized timing plans. The exceptions to this are reductions in traffic demand by 5% and 25%. For a 5% reduction in traffic demand, nonoptimized timing plans are probably good enough so that no benefits are achieved by optimization. For a decrease of 25%, different underlying reasons cause no benefits of updating signal timings. The outliers shown in Figure 4.2 coincide with no benefits of updating pretimed control regimes for a 25% decrease in traffic demand. These outliers represent a case where traffic demand decreases further, but there are no benefits of retiming pretimed signals. This is because of the limitations of the optimization set by the lowest possible cycle length.

In other words, the benefits of optimizing will be zero if the demand falls so low that it requires an optimal cycle length that is lower than the minimum cycle length.

5.3.1.1.2 *Actuated-Coordinated Traffic Control.* The results for the impact of a decrease in traffic demand on the actuated traffic control regime are shown in Figures 4.9 and 4.11 and Table 4.3. For this traffic control regime, there are benefits of updating signal timings for any scenario except for a 5% decrease in traffic demand. Unlike pretimed traffic control, this control regime still yields benefit from retiming their settings for a 25% decrease in traffic demand. This comes from the fact that during some cycles, green splits will be utilized better with a lower traffic demand. Pretimed traffic control has no such flexibility, so it cannot adjust further unless the cycle length is reduced.

5.3.1.2 Increase in Traffic Demand

5.3.1.2.1 Pretimed Traffic Control. The impacts of an increase in traffic demand on the pretimed traffic control regime are shown in Figures 4.7, and 4.12 and Table 4.1. The results show that optimized timing plans yield a smaller PI than the nonoptimized timing plans for all scenarios but the one where the traffic demand increases by 5%. This result for the 5% scenario is again a consequence of the fact that the changes in traffic flows are so small that no benefits are achieved by optimization. Table 4.1 and Figure 4.12 support such findings. Figure 4.12 also shows that the benefits of updating signal timings do not grow after a certain point (in this case, after a 25% increase in traffic demand). This finding is simply a consequence of the fact that once the saturation point in the network is reached, the benefits of retiming signals will no longer increase. Still, major attention should be given to the part of the graph where the relationship between the benefits and demand is linear. This addresses ageing pretty well for most traffic systems because the traffic demand does not increase more than 20% over 3 to 5 years.

5.3.1.2.2 *Actuated-Coordinated Traffic Control.* The impacts of an increase in traffic demand on the actuated traffic control regime are shown in Figures 4.9, and 4.12 and Table 4.3. The results are very similar to those of pretimed traffic control. Both pretimed and actuated traffic control regimes are incapable of coping with increased traffic demand without increasing cycle lengths.

5.3.1.3 Decrease in Turning Movements

5.3.1.3.1 *Pretimed Traffic Control.* Figures 4.8 and 4.13 and Table 4.2 show the impacts of a decrease in turning movement proportions on the performance of the pretimed traffic control regime. The benefits of retiming traffic signals for these changes come from reallocation of the green splits. More green time is needed for through movements to compensate for the increase in traffic resulting from the decrease in left and right turning movements. Table 4.2 shows that all decreases in turning movement proportions endorse updates of signal timings. According to Table 4.2, updating signal timings for all scenarios of decrease in turning movements yield significant benefits.

5.3.1.3.2 *Actuated-Coordinated Traffic Control.* Figures 4.10 and 4.13 and Table 4.4 show the impacts of a decrease in turning movement proportions on the performance of the actuated traffic control regime. Figure 4.10 shows no difference between optimized and nonoptimized actuated signal timings when turning movement proportions are decreased. Table 4.4 formally shows that the differences are not significant. Similarly, Figure 4.13 shows no benefits of optimization of signal timing plans for these scenarios. One could conclude that the traffic control does not perform well for these changes in traffic flows. However, the opposite is true. For the pretimed control (Figures 4.8 and Table 4.2), the benefits of retiming traffic signals come from reallocation of green splits and the increase in cycle length for the heavy through movements (80% decrease in turning movements). However, actuated-coordinated traffic control does not need retiming of splits and cycle length. The benefits of retiming actuated control are negligible because, with very low traffic demand for left turns during certain cycles, left phases are skipped and all green time is used for heavy through movements. For pretimed control, left phases cannot be skipped and an increase in cycle lengths is required to accommodate those through movements.

5.3.1.4 Increase in Turning Movements

5.3.1.4.1 Pretimed Traffic Control. Figures 4.8 and 4.14 and Table 4.2 show the impacts of an increase in turning movement proportions on the performance of the pretimed traffic control regime. The benefits of retiming traffic signals for these changes also come from reallocation of the green splits. More green time is needed for left turns (right turns on red are allowed) to compensate for an increase in traffic for the left turn movements. Table 4.2 shows that a small increase of 10% in turning movement proportions produces no benefits of updating signal timings. Updating signal timings for other scenarios yield significant benefits.

5.3.1.4.2 Actuated-Coordinated Traffic Control. Figures 4.10 and 4.14 and Table 4.4 show the impacts of an increase in turning movement proportions on the performance of the actuated traffic control regime. Table 4.4 shows that all increases in turning movement proportions endorse updates of signal timings. Figure 4.14 shows that the benefits of updating are smaller for actuated-coordinated control than for pretimed traffic control. The reason for this difference is that the actuated-coordinated control performs better for higher increases in turning proportions. Actuated traffic control will use all gaps in through traffic to pass more left-turning vehicles more, if possible. The pretimed control has no such flexibility. Therefore, once the signal timings are updated, the benefits of updating are higher for pretimed control. In general, whenever flexibility of actuated traffic control can be used to cope with changes in traffic conditions, the benefits of updating settings for this control are lower than for pretimed control.

5.3.1.5 Discussion on Other Findings for Deterministic Scenarios

Two other major findings can be taken from the results presented in Figures 4.2 – 4.14. First, the benefits of updating are higher for optimization of traffic flows with increased demand than for optimizing the traffic flows with decreased demand. This means that it is better to optimize signal timings for a traffic demand that is higher than the current traffic demand. The disbenefits of such an optimization are smaller than the disbenefits of having outdated signal timing plans when the traffic demand increases. This finding confirms the findings of Park et al. (2000).

Second, it seems that the benefits of updating timing plans when only turning movements have been changed (split optimization) are higher than the benefits of updating timing plans when only traffic demand has been changed (cycle length optimization). This finding emphasizes the importance of actuated traffic control. Actuated traffic signals account for the majority of changes in turning movement proportions by adjusting phase splits according to turning movement demand during each cycle. According to the results of this study, if updating of splits is unnecessary and traffic demand at the links is uniformly increased, the benefits of updating signal timings are limited to no more than 2-3% per year for up to a 5% increase in overall traffic demand.

5.3.2 Stochastic Traffic Demand and Distribution Scenarios

The results for stochastic changes in traffic demand and turning movements presented in Figure 4.15 are quite different from those for uniform deterministic changes. For example, for a 0 to 5% increase in traffic demand, the minimum benefits of updating the timing plans are around 15% for pretimed control, or 5% for actuated-coordinated control. These benefits are much higher than the benefits for a deterministic increase in traffic demand (Figures 4.7 and 4.9). This difference is associated with the different natures of deterministic and stochastic experiments. While all traffic demand inputs are equal and turning proportions do not change in the deterministic approach, all of these factors are changed randomly in the stochastic approach. So, while the stochastic experiments require optimization of all signal timing parameters (splits, offsets, and cycle length), the deterministic experiments, whose results are presented in Figure 4.7 and 4.9, require mostly optimization of cycle length. Therefore, the benefits of updating signal timings for stochastic experiments are consequently higher.

Surprisingly, the results from 14 examples of successful retiming projects (ITE 2005) show approximately 30-40% of average benefits in delay and stops when timing plans are updated. Although this is merely a coincidence, these results prove that the estimations of the benefits from this study are within the range of what others found in the field.

5.4 Ageing of the SCOOT Control Regime

5.4.1 General Discussion of SCOOT Performance

In general, SCOOT performs worse than optimized pretimed plans for almost all scenarios of changed traffic demand and distribution. Moreover, SCOOT often performs worse than nonoptimized pretimed plans.

Before discussing possible reasons for SCOOT performing worse than pretimed plans, here are some of the assumptions that might affect SCOOT's performance:

- The peak-hour traffic demands used in this research were all constant during the peak hour. SCOOT is known as a signal optimization strategy that delays the onset of congestion and recovery from congestion. Most of SCOOT's SCOOT benefits likely come from these two (prepeak and postpeak) periods. However, since traffic demand was kept constant, there were no opportunities for SCOOT to benefit from these two common peak periods. Alternatively, well prepared pretimed plans are an ideal traffic control for constant and heavy traffic demand.

- SCOOT version 4.2 is used in this study. Version 4.5 and especially version MC3 are likely to enable better SCOOT performance than the old version. Newnew features (added to with version 4.5 and version MC3) that can make a strong impact on SCOOT performance are complex links (with supplementary detectors) and the ability to estimate saturation occupancy online (SOFT). Had these features been used in this study, SCOOT might have performed better.
- The SCOOT network has been validated and calibrated to the best of the author's knowledge. However, SCOOT is a complex system. Many years of expertise are needed before one can get the most out of SCOOT. The purpose of this study was not to adjust SCOOT in such a way to get the best results. That was beyond the scope. The purpose was to build and validate SCOOT configurations and evaluate their performance under various traffic changes without adjusting SCOOT at all. Therefore, due to the experimental design of this study, the results and findings of SCOOT performance should not be used to qualify the general performance of SCOOT or compare SCOOT performance with any traffic condition or and network different from those used in this study.

There are several other possible reasons for the fact that SCOOT performs worse than nonoptimized pretimed plans:

- SCOOT control settings perform worse than nonoptimized timing plans
- The PIs of pretimed control in VISSIM could be overestimated. Therefore, SCOOT could have been better than nonoptimized timing plans if VISSIM had been adjusted to endorse timing plans from Synchro in the same way that SimTraffic is adjusted

However, one should not focus on the fact that SCOOT is sometimes worse than nonoptimized performance of pretimed control. Comparison of SCOOT performance and performance of optimized pretimed plans is essential for determining the ageing of the SCOOT control. However, before discussing the ageing of SCOOT, it is necessary to discuss the fact that SCOOT performs worse than optimized pretimed control (Figure 4.16 and Figure 4.7).

There are two major reasons that cause SCOOT to perform worse than optimized pretimed control. Each of these reasons can be divided further. The reasons are:

- SCOOT signal optimization strategies are not the best for current traffic conditions
- SCOOT does not model traffic properly and, therefore, signal timing adjustments are inadequate

The next subsections describe the failures of SCOOT optimization strategies and SCOOT's inability to model traffic properly.

5.4.1.1 Failures of SCOOT Optimization Strategies

There are two reasons why the SCOOT optimization strategies might not be the best for traffic changes modeled in this study. The first reason is associated with small unnecessary fluctuations of SCOOT signal timings. The second reason is related to selection of the best cycle lengths, which provide good traffic progression in the network.

5.4.1.1.1 *Fluctuations of SCOOT Signal Timings*. SCOOT is an adaptive traffic control system that adjusts to changes in traffic demand and distribution in real time. Over years, a time span characteristic for any ageing process, this system should not have problems in adjusting to traffic changes. In fact, SCOOT usually needs just 15 – 20 minutes (the time which was used for VISSIM simulation warm-ups) to adjust from initial timing settings to the timing settings "appropriate" for surrounding traffic conditions. The approach that SCOOT uses to determine the "appropriate" timing settings is another issue.

The simulation experiments have shown that SCOOT has no problem in adjusting to existing traffic conditions in the way the SCOOT model finds it is the best. However, SCOOT control will not remain constant after timings are adjusted to surrounding traffic conditions. The SCOOT control will continue to fluctuate by changing cycle lengths, offsets and splits to respond to short time variations in traffic. These fluctuations of SCOOT signal timings are counter effective over a longer time period (1 or 2 hours) with constant traffic demand. They yield more costs than benefits. All experiments conducted in this research assume constant traffic demand over a 1-hour peak period (different levels, but always constant along the time period). However, even the constant traffic demand is modeled with some variations in micro simulators. These variations will cause 5-minute hourly traffic rates to not be constant during the whole hour. These various 5-minute hourly traffic rates are large enough to trigger a fluctuation in SCOOT control. SCOOT changes its cycle lengths every 2.5 to 5 minutes. Other timing settings are changed even more frequently. In this way, SCOOT timings oscillate around their mean values, which are close to the ones defined by pretimed optimized traffic control.

These fluctuations in SCOOT's signal timings, although small, affect the smoothness of existing traffic progression by causing additional delays and stops. The consequences of such changes are termed "transients". When changes in SCOOT's signal timing settings are triggered by small and negligible changes in traffic flows, the transients mean more delays and stops than benefits. The increase in stops and delays during each cycle might be negligible, but their accumulation at the end of the simulation period is large enough to overcome delays and stops produced by optimized pretimed plans.

5.4.1.1.2 *Selection of Cycle Lengths.* Shelby et al. (2005) investigated the impact of selection of cycle lengths on traffic performance on arterial streets. The authors compared "system friendly" cycle lengths with strictly flow proportionate approaches in selecting cycle length referred to as the "90% rule". The "90% rule" represents a way of selecting cycle lengths used by most of the existing adaptive control systems in the USA. Systems like SCOOT (Hunt et al., 1981), SCATS (Lowrie, 1982) and OPAC (Gartner, 1983) all use the "90% rule" when selecting cycle lengths for multi-intersection network configurations. The rule means that a critical intersection will govern the cycle length of the whole network by keeping the degree of saturation at 90%. If an intersection becomes more than 90% saturated (and reallocation of splits cannot reduce this value), then the common cycle length for the whole network is increased by a few seconds. Similarly, the cycle length decreases if saturation falls below 90%. Although this follows the trends in traffic demand, it does not always yield the best cycle lengths. For certain network configurations, only a few cycle lengths provide good progression for both directions of major network arterials. If the selected cycle length is different from the cycle lengths which give the best progression, the traffic performance is suboptimal.

Shelby et al. (2005) used the notion of a “resonant cycle” for those cycle lengths that accommodate good two-way arterial progression. They found that the “resonant cycles” yield benefits of up to 40% fewer delays when compared with the cycles selected by the “90% rule”. The authors also found that, even if resonant cycles are not present (no cycles which provide good two-way progression), well-chosen cycle lengths may still outperform the cycles chosen by using the “90% rule”. These findings seriously question SCOOT’s strategy for optimizing network cycle length.

It has been noted in this study that the SCOOT cycle lengths are greater, on average, than the cycle length used in optimized pretimed plans. Larger cycle lengths increase the capacity of the network (only up to a certain point), but they also increase delay for all vehicles in the network. The almost constant difference between SCOOT’s performance and the performance of optimized pretimed control, shown in

Figure 4.16, shows that SCOOT's inadequate selection of cycle lengths could be one of the reasons for this discrepancy.

5.4.1.2 Failures of SCOOT Traffic Modeling

The section "SCOOT Validation" describes the fundamentals of SCOOT traffic modeling and the necessary adjustments that need to be made for SCOOT to accurately estimate vehicles at intersection approaches. It has been noted during the simulations that SCOOT keeps some green times longer or shorter than necessary. Green times for each phase depend on cycle length, but they are mostly adjusted based on cyclical demand for each phase.

Initial validation efforts have shown that SCOOT has been properly validated (Figure 3.25). Several experiments were done to check SCOOT's accuracy in estimating approaching vehicles at the intersections for changed traffic conditions. SCOOT estimates approaching vehicles based on average travel time (from detector to stop line), the assumed platoon progression factor, etc. All these factors are dependant on the intensity of traffic flows, which can be represented by the scenarios of changes in traffic demand used in this study. Scenarios representing changes in traffic distribution are not essential for checking SCOOT's ability to accurately estimate approaching vehicles. Therefore, 11 scenarios of changes in traffic demand are used to model traffic. Queue measures for both SCOOT (modeled) and VISSIM ("real") are taken. For each of nine intersections, SCOOT and VISSIM queues are measured during 10 minutes for each of 11 scenarios. The same 10-minute intervals are used to measure queues for each intersection, keeping the measurements consistent.

Similar results to those presented in Figure 3.25 are obtained for all 11 traffic demand levels. After the data reduction, a set of the least square models are developed for the 11 scenarios to represent correlation between modeled and actual number of vehicles in queues (see APPENDIX). Figure 5.1 shows the results from these experiments. The base traffic demand is represented by the zero point on the X axis. The least square model for this traffic demand is developed from experiments done for initial validation of the SCOOT model. Figure 5.1 shows the number of vehicles estimated by the SCOOT model for 100 actual vehicles (VISSIM). SCOOT

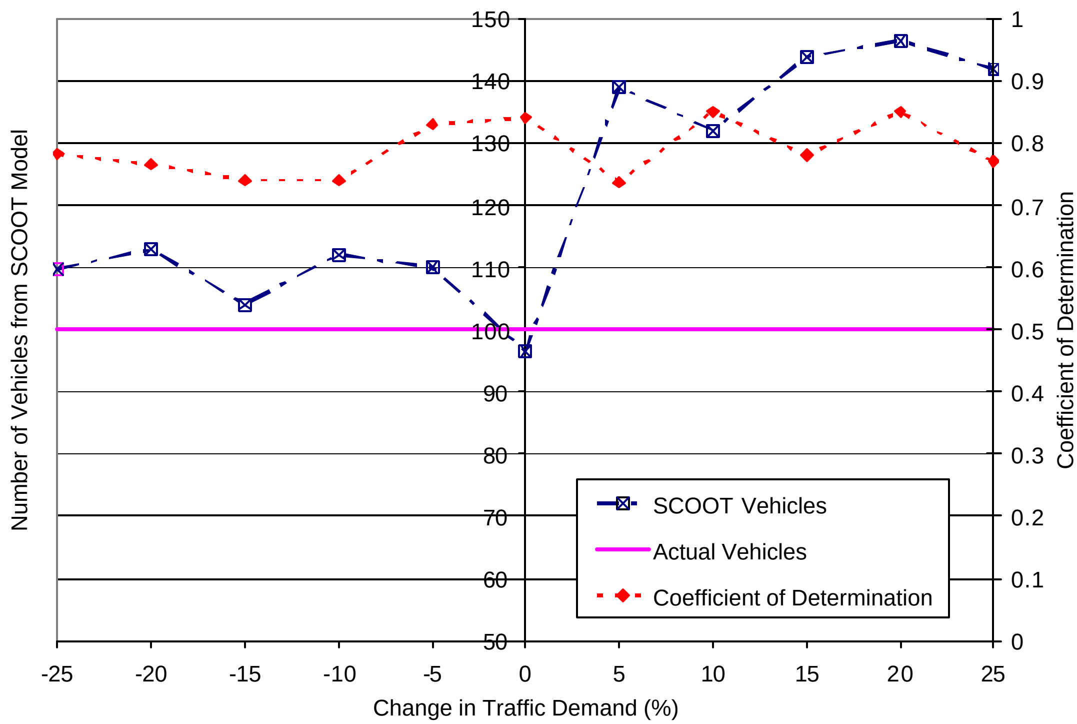


Figure 5.1 – Accuracy of SCOOT Traffic Model vs Traffic Demand

estimation for base traffic conditions is close to the actual number of vehicles. SCOOT estimates around 97 vehicles for every 100 actual vehicles. However, as traffic demand changes, the least square models become less accurate. When traffic demand increases, SCOOT tends to overestimate the number of vehicles. For example, for a 15% increase in traffic demand, SCOOT will estimate 145 vehicles for every 100 actual vehicles. For scenarios of decreased traffic demands, the error will be smaller. However, Figure 5.1 clearly shows that SCOOT's ability to model the approaching vehicles at the intersections degrades with changes in traffic demand. All of the least square models that have been developed from the queue observations are quite reliable, having coefficients of determination between 0.73 and 0.85.

The findings from these experiments show that SCOOT's traffic model mostly overestimates the number of approaching vehicles. The consequences of this overestimation are improper green splits and a tendency to increase cycle length in the region. When SCOOT overestimates the number of vehicles approaching the intersection during one phase, this information is conveyed to the SCOOT optimizers, which will consequently impose an unnecessary, longer green time for the phase. If a similar situation occurs for each phase, the cycle length will be increased. So, accuracy of the SCOOT model highly impacts the efficiency of the SCOOT traffic signal control. With that in mind, the results presented in Figure 5.1 prove the existence of SCOOT ageing, which has its roots not in traffic control, but in SCOOT modeling of traffic flows.

5.4.2 SCOOT Ageing for Changes in Traffic Demand

Figure 4.16 and Tables 4.6 and 4.7 show the impacts of changes in traffic demand on the performance of SCOOT and pretimed traffic control regimes. Table 4.6 shows that SCOOT is coincidentally better than optimized pretimed control only for the scenario of a 5% increase in traffic demand. For all other scenarios, SCOOT is significantly worse than the pretimed traffic control.

Table 4.7 shows that SCOOT does not age for most of the scenarios. However, these findings are more an indication of the difference between SCOOT and pretimed performance for base traffic conditions than a proof that SCOOT does not age. In fact, the difference between PIs for SCOOT and pretimed control is pretty high for base traffic conditions. Since the difference is so high, it is only exceeded a few times for increases in traffic demand of 10, 20, and 25% (for 15%, the amount that the difference is exceeded is not significant). However, when the difference is exceeded, it contributes to high coefficients of SCOOT ageing of 18, 48, and 35% for respective increases in traffic demand (Table 4.11).

5.4.3 SCOOT Ageing for Changes in Traffic Distribution

Figure 4.17 and Tables 4.9 and 4.10 show the impacts of changes in traffic distribution on the performance of SCOOT and pretimed traffic control regimes. Table 4.9 shows that for several changes in turning movement percentages (-20, 10, 20, and 40%), the SCOOT control is better than optimized pretimed control. This is not a coincidence. SCOOT shows quite consistent performance for most of the traffic distribution scenarios. This consistency and the fact that for these scenarios SCOOT control is often better than pretimed control are the consequences of two factors.

First, when total traffic demand in the network is kept constant, all increases in turning movements cause decreases in through movements. When through traffic is decreased, this is equivalent to a decrease in traffic demand for only the through movements. Figure 5.1 already showed that SCOOT accuracy is better for reduced traffic demand than for increased traffic demand. Therefore, SCOOT performs better for increased turning movements. Prove of this statement is visible in the left part of Figure 4.17. For high decreases in turning movements, the through traffic is increased so much that SCOOT again starts producing unreliable estimates of the traffic and consequently leads to poorer performance (Figure 4.17 – Scenarios -40, -60, -80%).

The second reason for improved SCOOT performance is that SCOOT behaves in a certain way as an actuated traffic control. The phases cannot be skipped (although the real SCOOT can make some phases demand-dependent), but left turn greens can be extended to satisfy varying traffic demand. This fact represents a benefit over pretimed control, which cannot vary green times for various left turn demand levels.

Unlike for changes in traffic demand, SCOOT is always better than nonoptimized pretimed timing plans for any change in turning movement proportions. This proves that the SCOOT control regime can respond very successfully to long term changes in turning movements.

Figure 4.8 shows that SCOOT does not age for most of the changes in turning movement proportions. The fact that SCOOT ages for a 80% decrease in turning movements is not surprising. A reduction of 80% in left and right turns for the base network traffic demand is equivalent to an increase of 22% for through traffic demand at each intersection approach. Figure 4.16 and Table 4.7 show that for such high increases in traffic demand, SCOOT does age. The ageing coefficient for this scenario is around 30%, which is comparable to previous results (Table 4.11).

One important issue has to be mentioned when discussing SCOOT ageing for changes in turning movements. Idealized detector placements are used to configure left turns in the SCOOT system. Usually, SCOOT uses filter links to determine green times necessary for left turns. This concept retrieves information on the number of left turning vehicles during the previous cycle when determining the duration of the left turn green time for the current cycle. Detectors are placed on the filter links, which are usually downstream links that the left-turn vehicles turn onto (Siemens 2003). When this concept is used, SCOOT does not provide proactive green time for left turn movements, but the green time is based on the demand from the previous cycle.

The SCOOT system configured in this study treats left turn movements as all other (through) movements. This means that the detectors are placed in the left turn pockets, well ahead of the stop line. This concept enhances split optimization of the left turns. However, placement of the detectors is more important from another aspect. SCOOT detectors, which are placed close to the upstream intersections, usually detect all vehicles approaching the intersection. When these vehicles come closer to the intersection, they split entering turning movement pockets (left and right). Therefore, the actual number of through vehicles that arrive at the intersection could be considerably lower than the number of detected vehicles. This inconsistency negatively impacts the accuracy of the SCOOT traffic model. The new SCOOT version MC3 allows installation of additional detectors to deduct turning vehicles. For older SCOOT versions (SCOOT 4.2 is used for this study), this problem is supposed to be solved by adjusting the saturation occupancy parameter STOC. STOC is supposed to be increased enough so that it accounts for vehicles that turn left and right at the intersection. However, a regular STOC validation is complex enough that this additional adjustment brings a higher potential for error. Often, validation of STOC for such conditions is almost impossible. Even when possible, such validation is highly dependent on the variability of turning movement proportions.

In order to avoid such problems, the SCOOT detectors in this study are placed unconventionally. Both left turn and through detectors are placed at the entrances to the left and right turning pockets (Figure 3.24). This distance is approximately 450 ft from the stop bars. The through detectors are placed after the entering points into left-turn pockets. In this way, only through vehicles are detected by “through” detectors and validation of STOC can be completed with higher accuracy. Description of this concept is important because, if the detectors had not been placed this way, SCOOT would not show such good results when assessing SCOOT ageing for the traffic distribution scenarios.

5.5 Summary of Discussion

Discussion of the results was presented in this chapter. The general methodology used to measure the ageing of various traffic control regimes is viable.

However, there are a few deficiencies. First, the results of the ageing process are quite dependent on adjustments of traffic model parameters and MOEs from macro and microsimulation tools. Second, the ageing of SCOOT is assessed in a way that largely depends on the difference between SCOOT and pretimed performance for the base traffic conditions. This approach shows that SCOOT performs better than it really does.

Further, the ageing measure introduced by Bell (1985) is discussed. This measure does not uniquely correlate changes in traffic flows with changes in performance of a traffic control regime. Therefore, the experimental results reported in Bell's study should be taken with reserve.

The ageing of pretimed and actuated controls is discussed in the next section of the chapter. In general, actuated-coordinated control always performs better than pretimed control. Therefore, the benefits of retiming actuated-coordinated control are always smaller than for pretimed control.

SCOOT performance is discussed in the last part of the chapter. Several reasons are discussed for SCOOT performance being worse than the pretimed performance. The reasons are associated with deficiencies in both SCOOT optimization strategies and the way SCOOT models traffic at the intersection approaches. The results show that accuracy of the SCOOT traffic model degrades with changes in traffic demand.

Bell's ageing results (Bell, 1985) from real networks (around 3% per year, with up to four years between updates) are frequently used when the disbenefits of not optimizing signal timings need to be estimated. The results from this research show that Bell's estimate should not be used for any situation. There are two reasons why the estimate of 3% should not be used anywhere.

First, the disbenefits of deterioration of traffic signal timings should not be associated with the time elapsed from the last retiming process. Time itself is not a factor that influences deterioration of traffic control. Instead, the disbenefits of deterioration of traffic signal timings should be associated with changes in traffic demand and distribution. Neither Bell's research nor this study has found a single measure of changes in traffic flows which could successfully correlate the disbenefits of deterioration. However, an increase or decrease in background traffic demand can be a simple qualifier of changes in traffic flows. The results from this study show that for every 5% increase in background traffic demand, the benefits of updating signal timings are 3%. These results coincide with Bell's results only if traffic demand increases 5% per annum.

Second, the results of from this study show that the benefits of updating signal timings can be much higher if the distribution of traffic within the network is significantly changed. The benefits of updating signal timings when left and right turning movements are increased can reach 50%. These distribution changes in traffic flows are not common for most urban networks, but they are not impossible. For such cases, Bell's results underestimate the benefits of updating traffic signal timings.

The highly stochastic results of the research presented in the dissertation have shown that the benefits of updating signal timings can be 35 – 45 %. Experiments that yielded to these results were random and not common for regular road networks. However, these results show that the ageing of timing plans can produce disbenefits that are much higher than 3% per year, depending on how much traffic distribution has changed (permanently) from the last retiming process.

CHAPTER 6

CONCLUSIONS

This chapter consists of two sections. In the first section, the conclusions of the research are presented. The second section provides the limitations of the research, as well as direction for future research.

6.1 Conclusions

This section presents the conclusions of an investigation on the ageing of pretimed, actuated-coordinated, and SCOOT traffic control regimes through the use of simulation. The research had two objectives. The first was to introduce and validate the theoretical concept of ageing of traffic control. The second objective was to estimate the ageing of the three traffic control regimes for a variety of deterministic and stochastic changes in traffic demand and distribution. The following conclusions are reached in this study.

6.1.1 Methodology for Assessing Ageing of Signal Timing Plans

The PI difference between updated (optimized) and not updated (non-optimized) timing plans is proven to be a reliable measure of change in traffic performance for aged traffic conditions. However, there were two problems associated with use of the PI measure:

- It does not work very well when traffic in the network is saturated or close to saturated. However, when traffic is saturated most traffic agencies, responsible for maintenance of traffic signals, act. The real benefits of retiming traffic signals come from conditions where ageing is unnoticed, with small and gradual changes in traffic conditions.

- In order to have a clear distinction between PI for optimized and nonoptimized timing plans, one should use a type of microsimulation software that endorses use of these plans. Only the Synchro-SimTraffic combination works fine. Other macro-micro combinations need adjustment of both traffic model settings and MOEs.

In addition to these problems, the CF ageing measure (used by Bell) is not proven to be a good measure of changes in traffic flows. The same CF value can yield several different benefits of updating timing plans. This measure can be used to assess the impact of a single deterministic change in traffic flow on traffic performance in the network. However, the CF ageing measure fails to correlate well with the benefits of updating timing plans if several traffic changes (stochastic variations of demand and distribution) are combined.

6.1.2 Ageing of Pretimed and Actuated-Coordinated Signal Timing Plans

Two different sets of experiments were conducted to estimate the benefits of updating signal timing plans. In the first, traffic demands and distributions were deterministically changed from the base conditions. The results show that:

- The impact of the turning movement changes on the ageing of the performance of traffic signals (up to 70%) is much higher than the impact of the traffic demand changes (up to 30%).
- Optimizing traffic signals for higher than existing traffic demand brings more benefits than costs, if the demand is expected to grow in future years.
- In a small coordinated grid network with constant turning movement proportions (average values), the annual benefits of retiming traffic signals are up to 3% for traffic growth of up to 5% uniformly over the entire network. These benefits are negligibly smaller for the actuated traffic control than for the pretimed traffic control.

The second set of experiments introduced the stochastic nature of ageing through the random changes of traffic demands and turning movement proportions. The results show that:

- The average benefits of retiming signal plans are around 35% for pretimed control and 27% for actuated control, when traffic inputs at the network and turning proportions are randomly changed.
- The results show that these benefits do not depend on average change in network traffic demand, due to the highly stochastic nature of the experiments.
- The benefits of retiming estimated in this study are in accordance with reported benefits from the field (ITE 2005).

6.1.3 Ageing of the SCOOT Traffic Control Regime

SCOOT ageing was evaluated only through deterministic changes in traffic demand and distribution. Around 210 hours of SCOOT simulations were performed. The results show that:

- SCOOT performs worse than pretimed control for most of the changes in traffic demand. This performance has roots in deficiencies of both optimization strategies and traffic modeling in SCOOT.
- When idealized detector placements are used, the SCOOT control performs better or equal to pretimed control for most of the changes in traffic distribution. Older SCOOT versions are not likely to outperform pretimed control if detectors are placed in traditional SCOOT positions.
- The traffic model within SCOOT fails to accurately estimate approaching vehicles for any change in traffic demand. The accuracy is weakened more by an increase in traffic demand than by a decrease in traffic demand.
- The ageing concept for SCOOT, which is based on the difference between SCOOT control and pretimed control for the base traffic condition, may underestimate the ageing of SCOOT.
- SCOOT performance varies - sometimes showing that SCOOT ages, and sometimes showing that it does not. However, in most of these cases, SCOOT is significantly worse than optimized pretimed plans.
- When SCOOT does age, the disbenefits of ageing are moderate. For high traffic demand growths of 20 and 25%, the results show that if the SCOOT

control had been replaced by optimized pretimed timing plans, delays and stops would be reduced by 11 to 16%.

6.2 Summary of Conclusions

The foundation of this research was set with three major hypotheses stating that each of the traffic control regimes does not age (deteriorate). The hypotheses were based on the case that signal timing parameters are not updated regularly. Based on the research findings, the hypotheses are:

1. $H_{0(1)}$ - Pretimed traffic control plans do not deteriorate with changes in traffic demand and distribution - REJECTED

2. $H_{0(2)}$ - Actuated traffic control plans do not deteriorate with changes in traffic demand and distribution - REJECTED

3. $H_{0(3)}$ - SCOOT adaptive traffic control does not deteriorate with changes in traffic demand and distribution - REJECTED

6.3 Future Research

There are several directions in which the future research on ageing of traffic control regimes should go. More investigation is needed on the real variations in traffic demand and distribution over several years. This would help to develop more realistic scenarios for modeling changes in traffic demand and distribution.

In addition, there is a need to investigate the possibility of developing a single ageing measure that would depend on changes in link flows and would be able to correlate the benefits of updating signal timing plans.

Major research efforts should be made in developing a stochastic optimization tool that would use outputs from the microsimulation to optimize signal timings. Such a tool, which uses the Genetic Algorithm (GA) approach, exists for CORSIM microsimulation (Direct CORSIM optimization). However, preliminary results from this study show that the GA needs more improvement before it can be used as an everyday tool to optimize signal timings. The areas where this procedure lacks explanation are design of the optimization process (multilevel versus all in one) and sensitivities of both parameters and results.

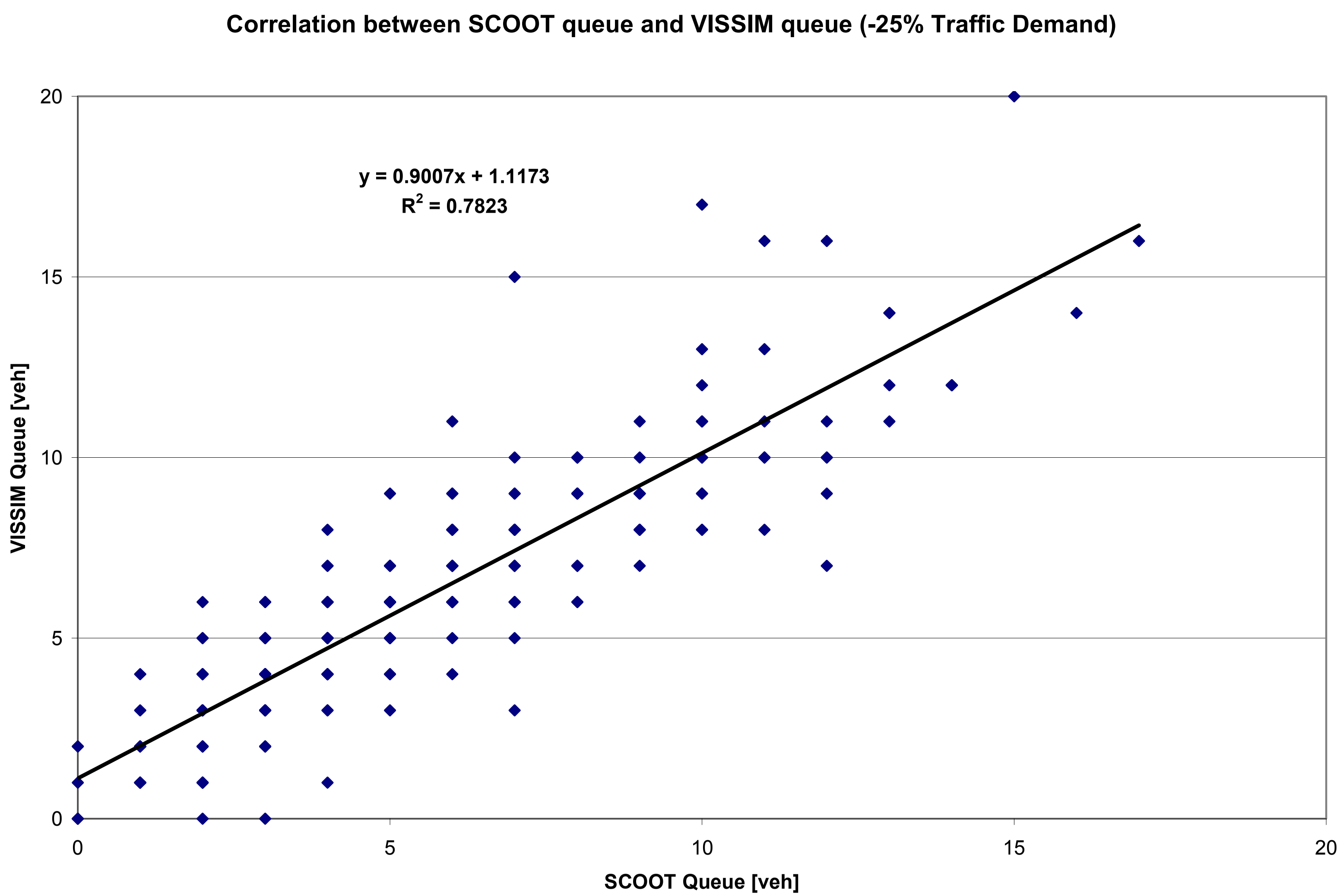
A similar stochastic optimization could be developed for VISSIM. The stochastic optimization in VISSIM would generate signal timing plans (pretimed or actuated) which are based on VISSIM performance outputs. This process would make performances of optimized and nonoptimized timings much more consistent. Development of such stochastic optimization in VISSIM would enable evaluation of all three control types (pretimed, actuated, and SCOOT) within a single microsimulation tool. Moreover, such stochastic optimization in VISSIM could allow use of custom-made performance indexes, which would include not only delays and stops, but also throughputs, queue lengths, etc.

Another direction in researching the ageing of signal timing plans is associated with prioritizing retiming of traffic signals in the network. If signal timings are supposed to be retimed on several corridors in the network – which corridor gets priority? The issue becomes more complex when different facilities are compared (e.g., 6-lane arterial versus 2-lane arterial). It would be interesting to research the trade-off between traffic demand (6 lanes have more traffic than 2) and the time that has elapsed since the last retiming process (e.g., the last retiming for the 6-lane road was 3 years ago, while the last retiming for the 2-lane road was 7 years ago).

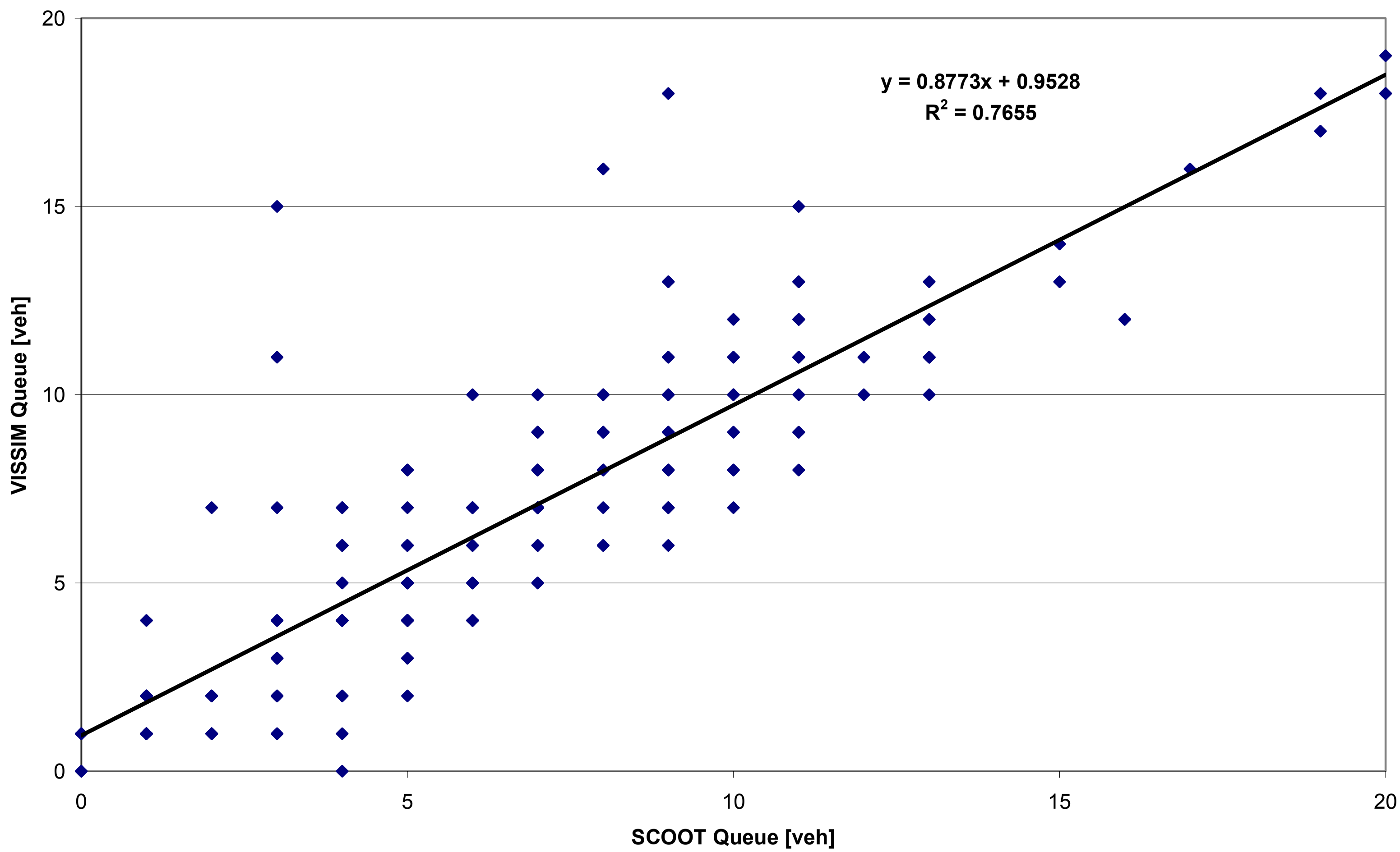
Regarding the accuracy of the SCOOT traffic model, this research provides results that are difficult to correlate with the underlying dynamics of traffic flows. Such correlation was unnecessary for this research and was beyond the research scope. More investigation is needed to thoroughly assess the accuracy of the SCOOT traffic model. A single intersection approach should be modeled with controlled changes in traffic speed, vehicle mix, platoon progression, link travel time, and other factors to investigate how each of these factors affects SCOOT accuracy to predict the number of vehicles in queue.

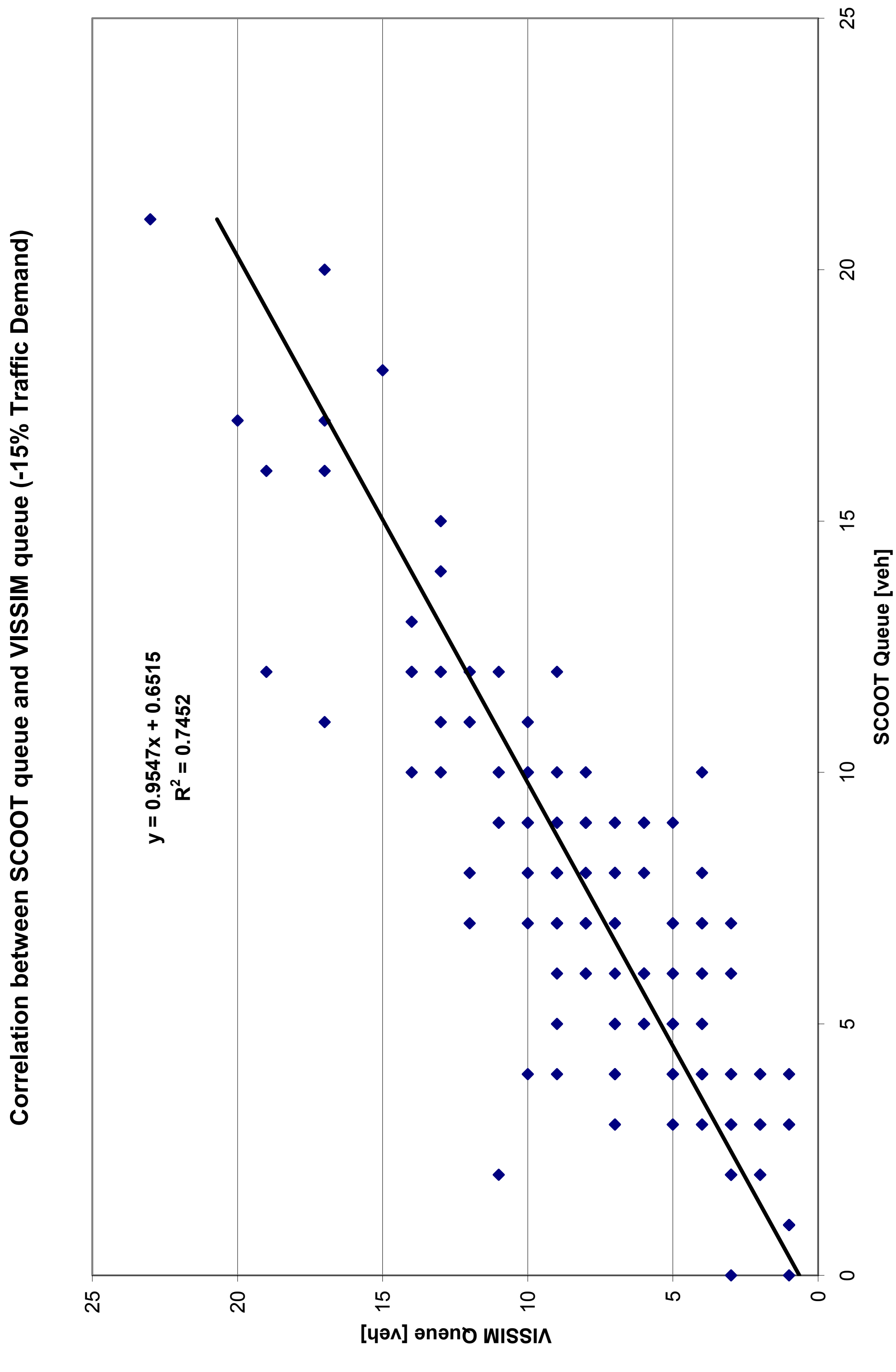
APPENDIX

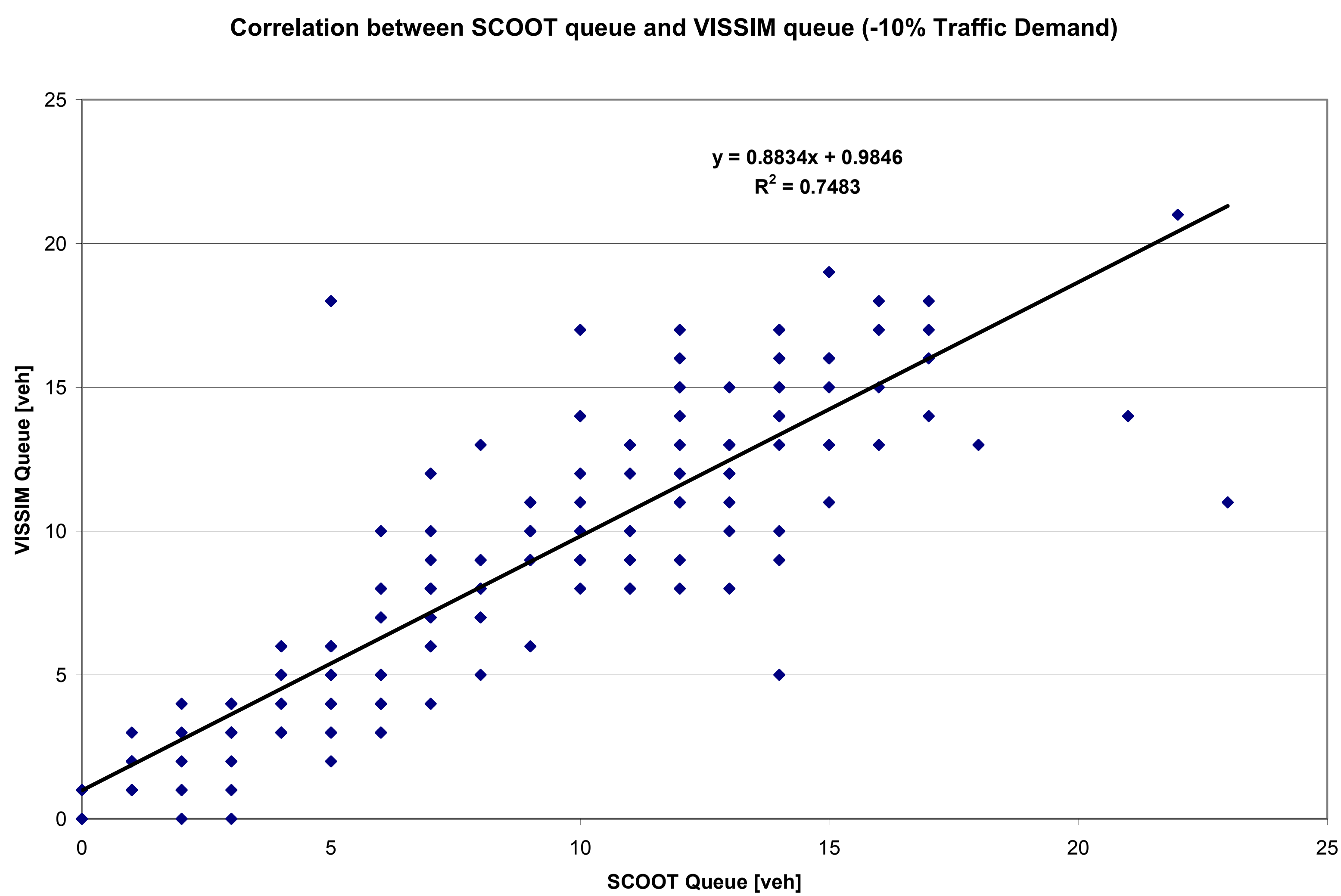
CORRELATIONS BETWEEN SCOOT AND VISSIM QUEUES

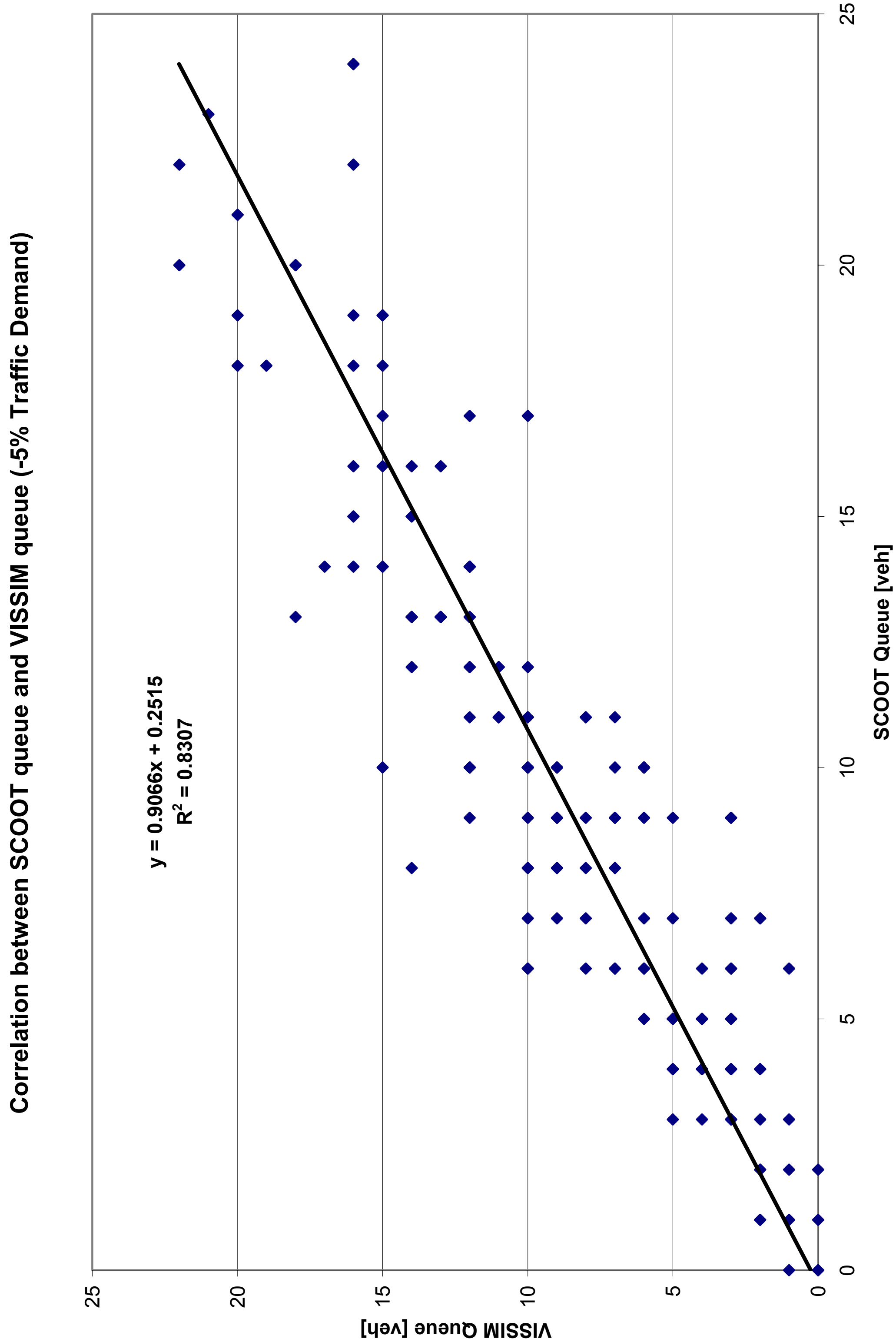

Correlation between SCOOT queue and VISSIM queue (-25% Traffic Demand)
y = 0.9007x + 1.1173
R2 = 0.7823
VISSIM Queue [veh]
SCOOT Queue [veh]
0
5
10
15
20

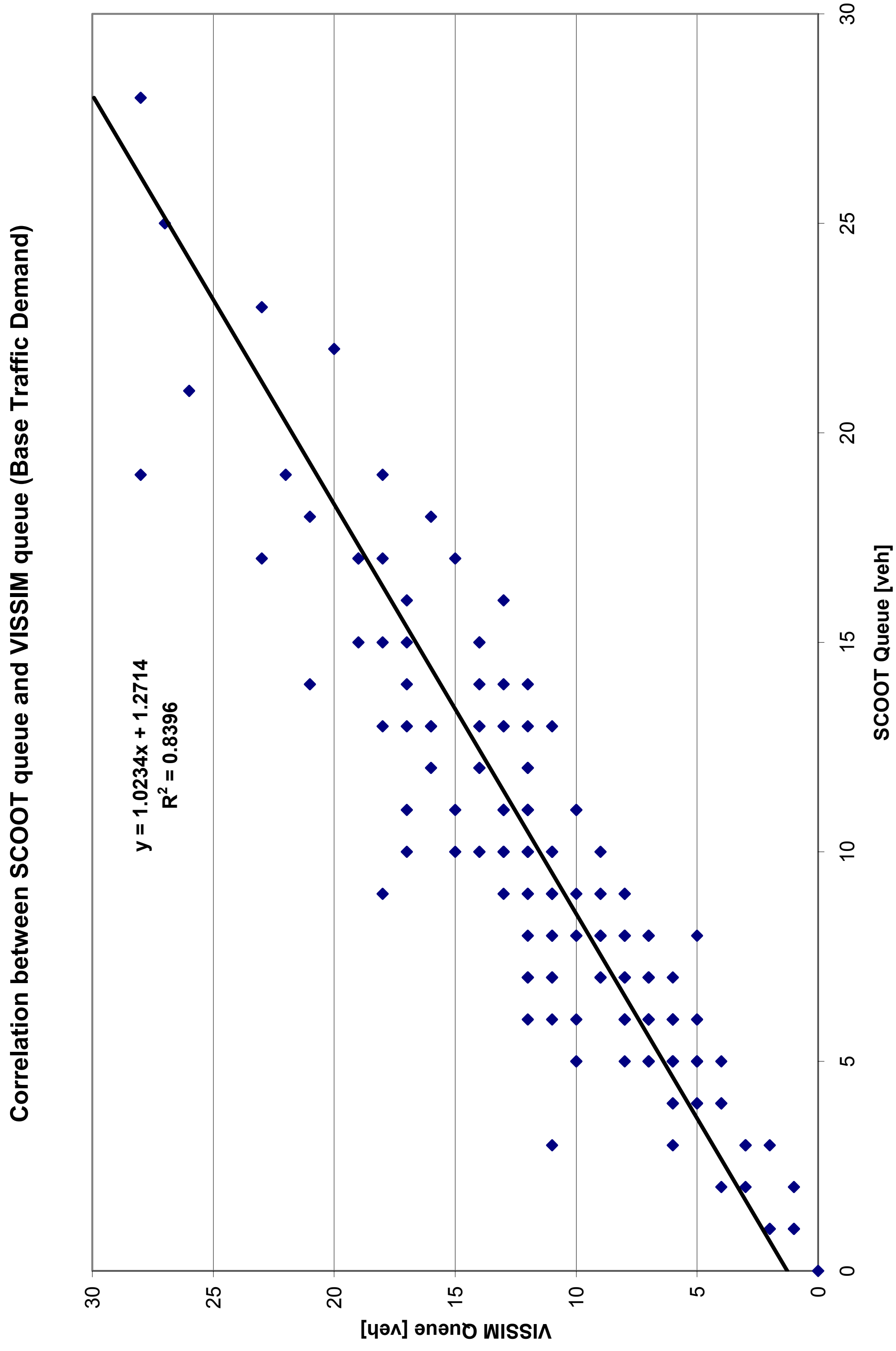
Correlation between SCOOT queue and VISSIM queue (-20% Traffic Demand)

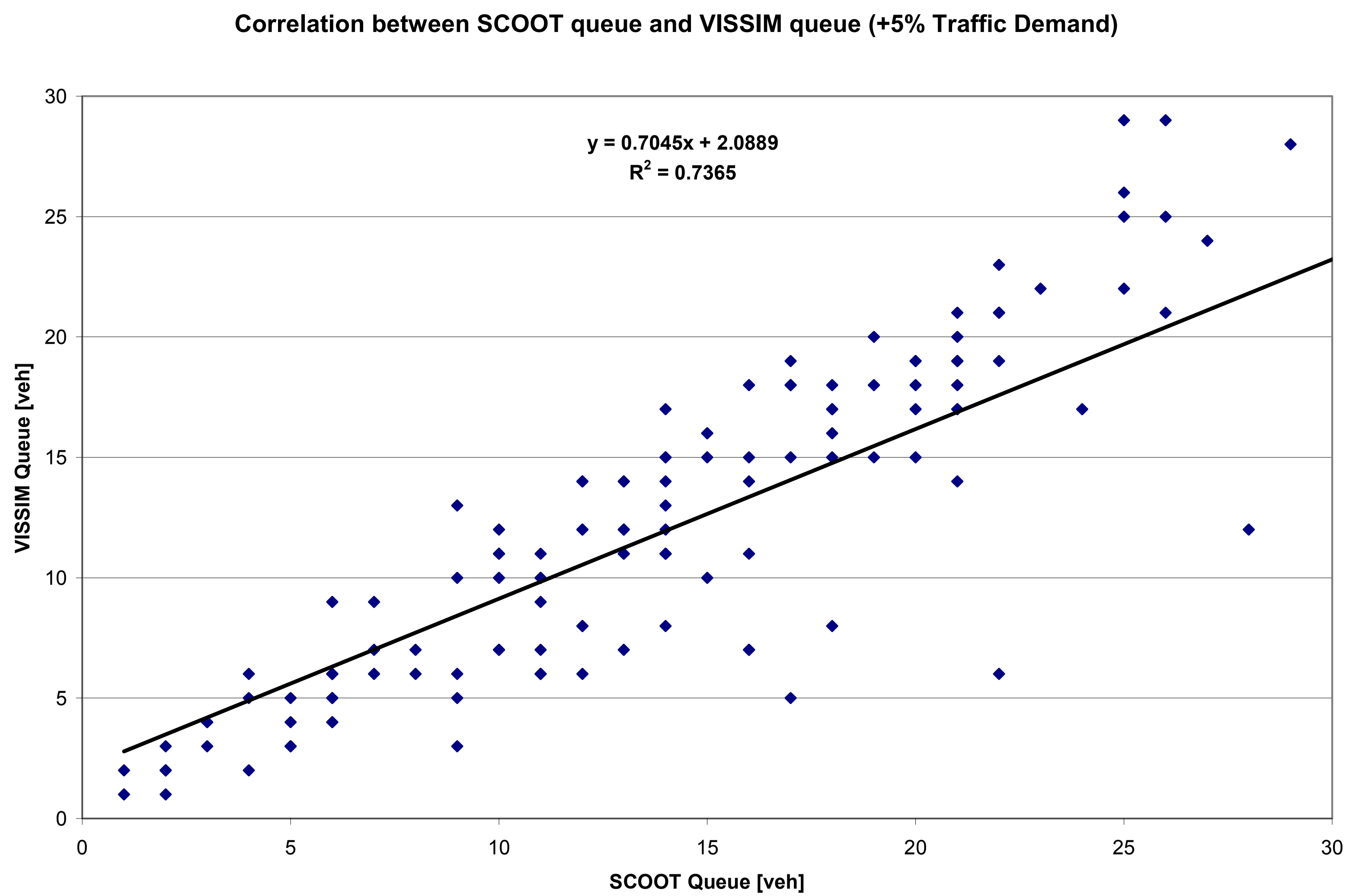



Correlation between SCOOT queue and VISSIM queue (-15% Traffic Demand)
y = 0.9547x + 0.6515
R2 = 0.7452
SCOOT Queue [veh]
VISSIM Queue [veh]
0
5
10
15
20
25

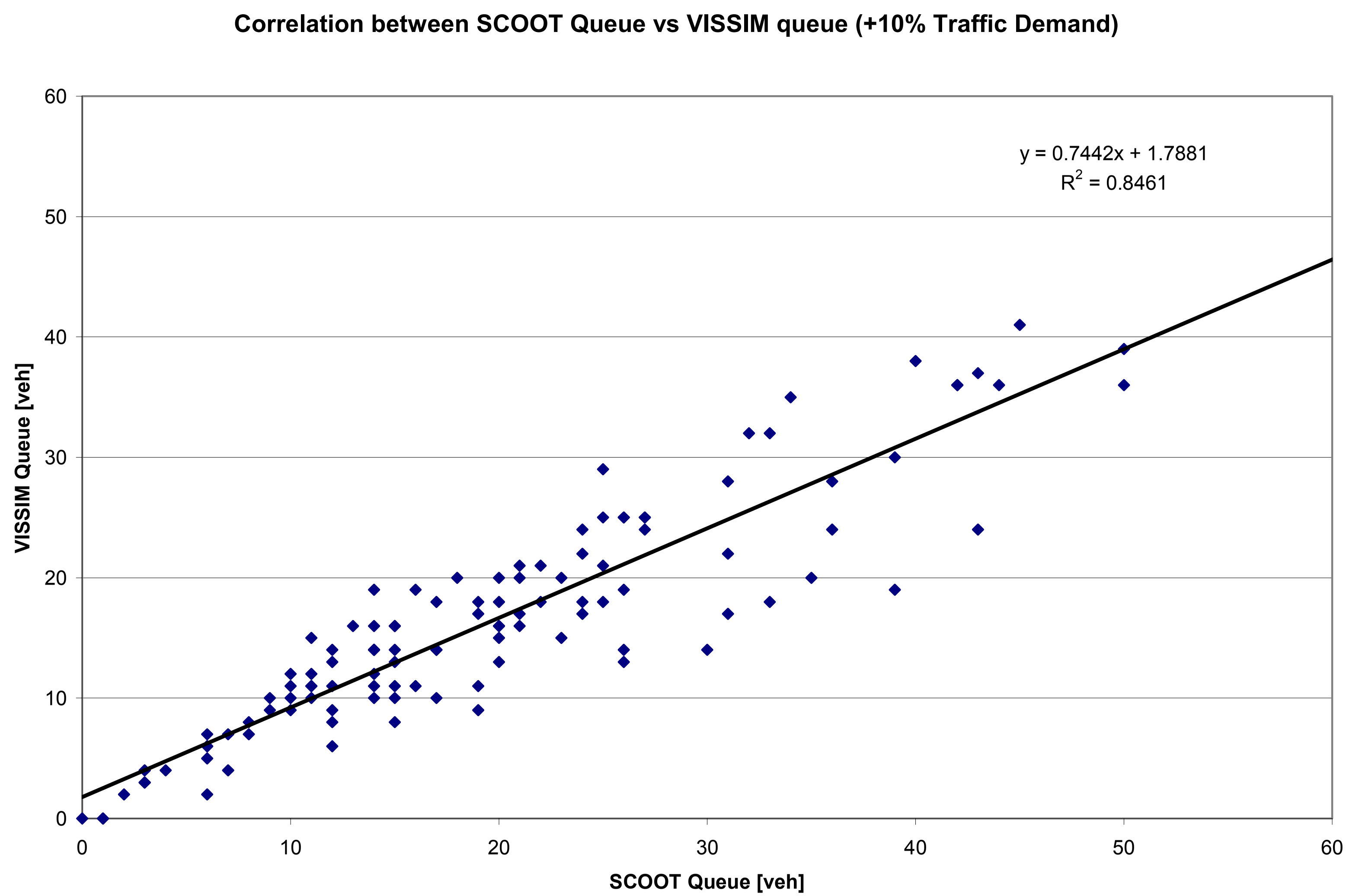

Correlation between SCOOT queue and VISSIM queue (-10% Traffic Demand)
y = 0.8834x + 0.9846
R2 = 0.7483
VISSIM Queue [veh]
SCOOT Queue [veh]
0
5
10
15
20
25

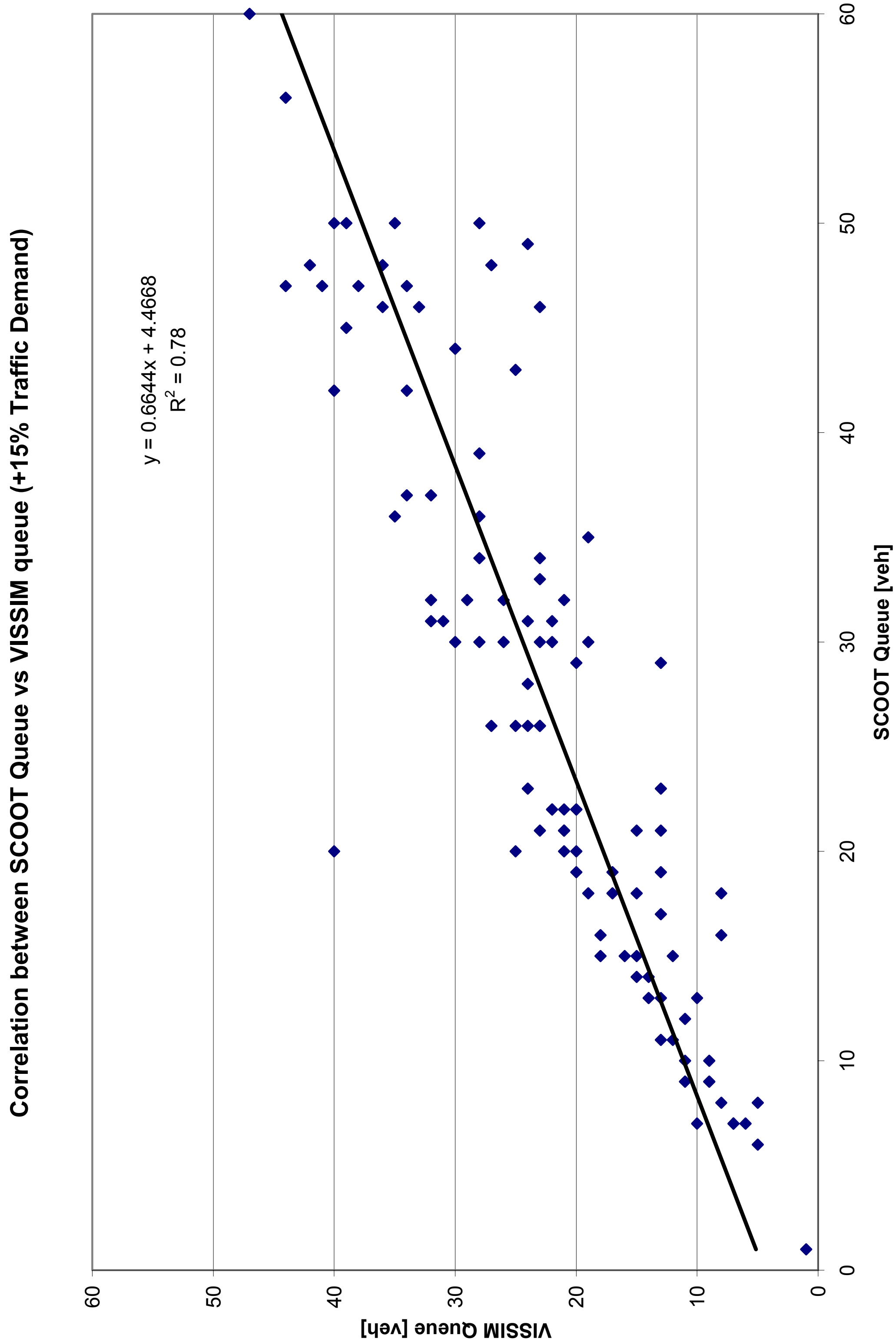

Correlation between SCOOT queue and VISSIM queue (-5% Traffic Demand)
y = 0.9066x + 0.2515
R2 = 0.8307
SCOOT Queue [veh]
VISSIM Queue [veh]
0
5
10
15
20
25
0
5
10
15
20
25


Correlation between SCOOT queue and VISSIM queue (Base Traffic Demand)
y = 1.0234x + 1.2714
R^2 = 0.8396
SCOOT Queue [veh]
VISSIM Queue [veh]
0
5
10
15
20
25
30

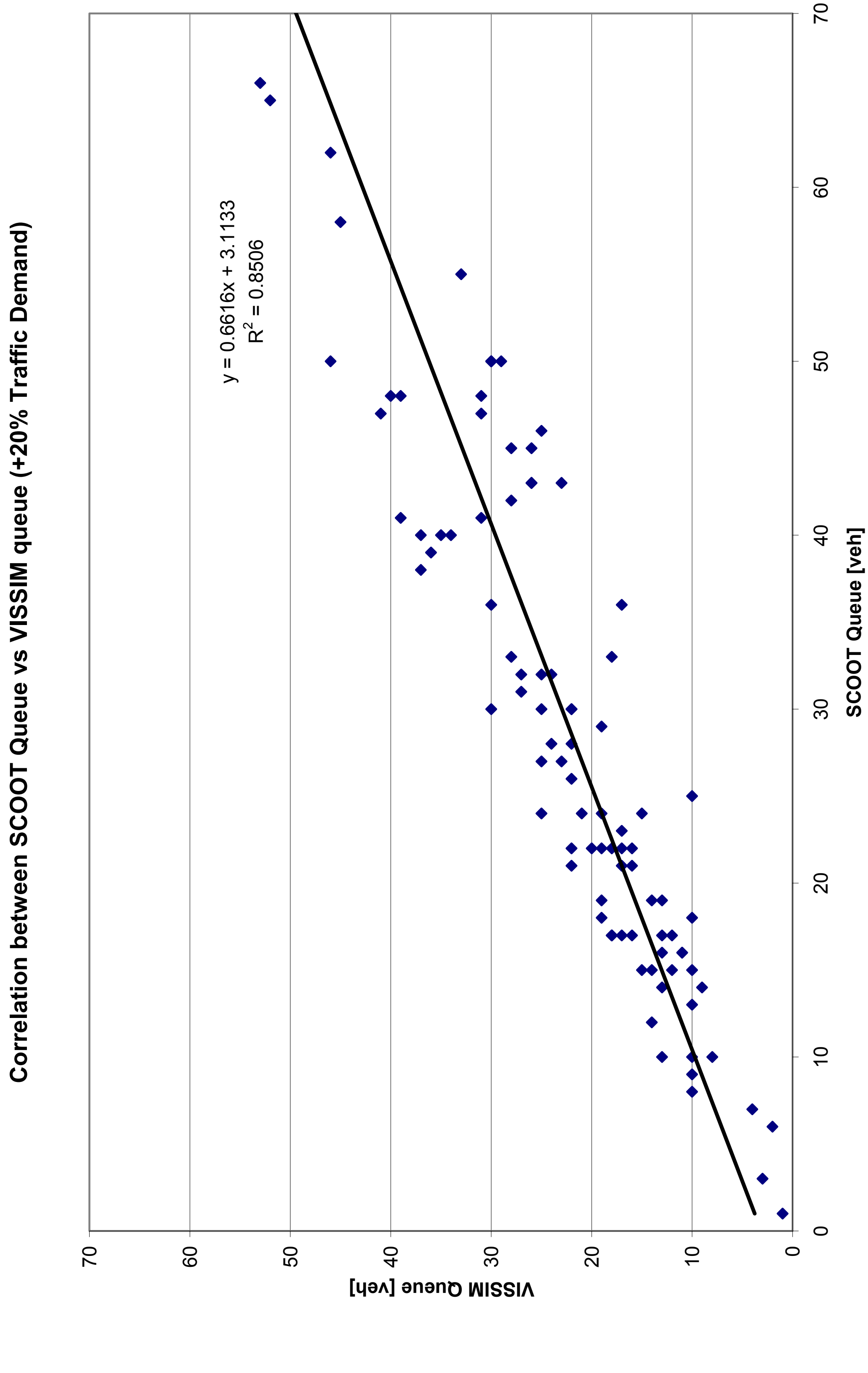


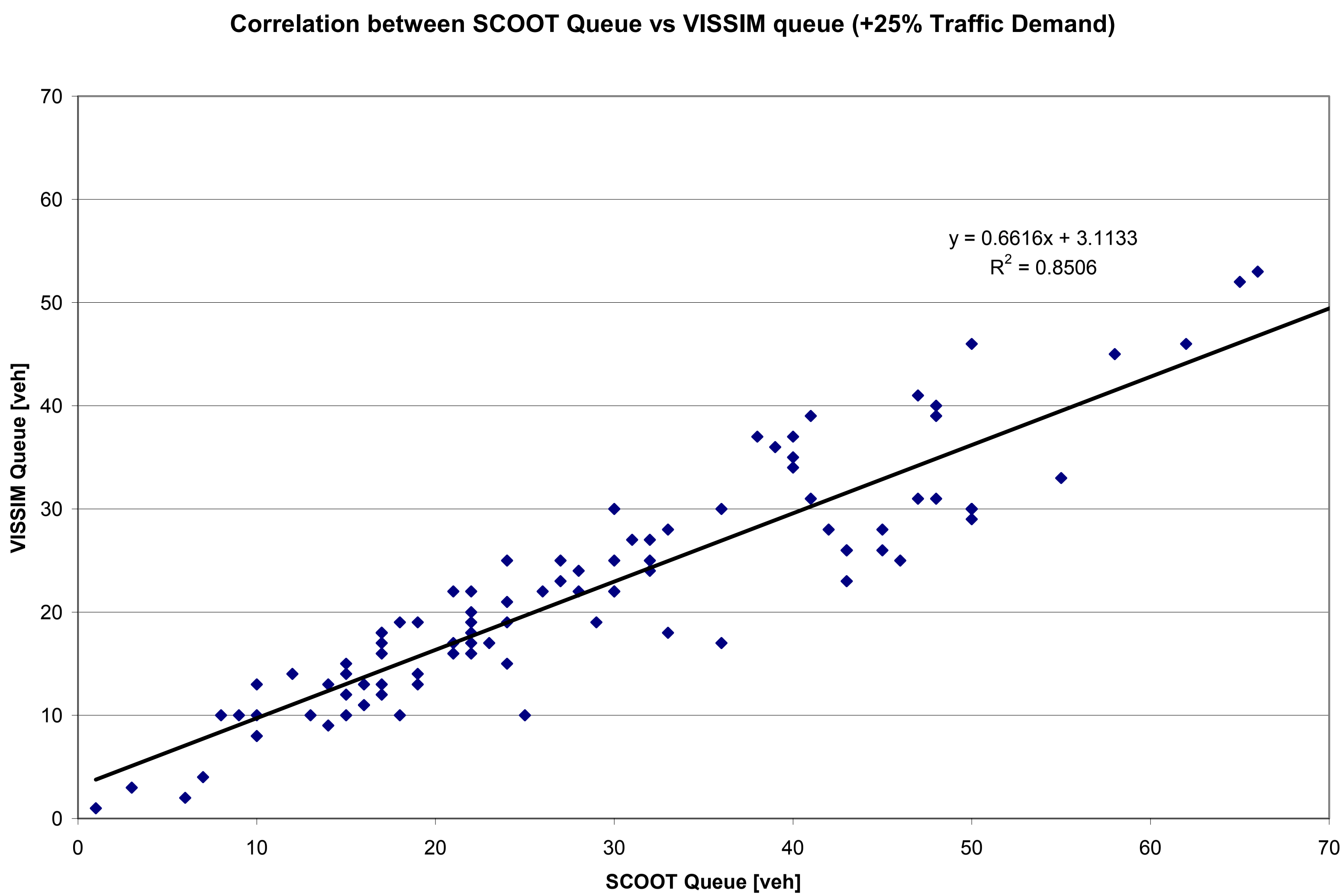
Correlation between SCOOT queue and VISSIM queue (+5% Traffic Demand)
y = 0.7045x + 2.0889
R2 = 0.7365
VISSIM Queue [veh]
SCOOT Queue [veh]
0
5
10
15
20
25
30

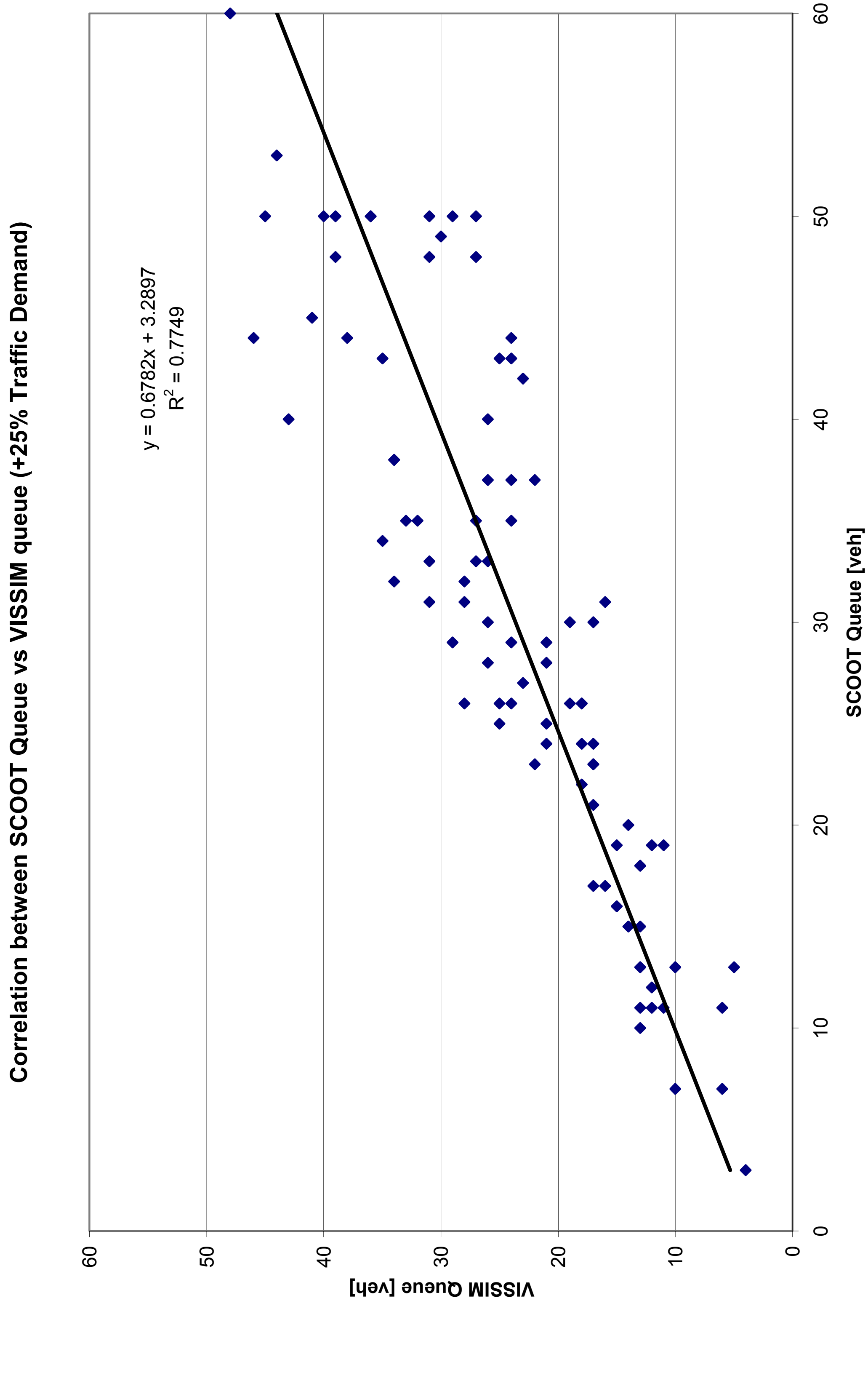

Correlation between SCOOT Queue vs VISSIM queue (+10% Traffic Demand)
y = 0.7442x + 1.7881
R2 = 0.8461
VISSIM Queue [veh]
SCOOT Queue [veh]
0
10
20
30
40
50
60



Correlation between SCOOT Queue vs VISSIM queue (+15% Traffic Demand)
y = 0.6644x + 4.4668
$R^2 = 0.78$
SCOOT Queue [veh]
VISSIM Queue [veh]
0
10
20
30
40
50
60


Correlation between SCOOT Queue vs VISSIM queue (+20% Traffic Demand)
y = 0.6616x + 3.1133
R2 = 0.8506
SCOOT Queue [veh]
VISSIM Queue [veh]
0
10
20
30
40
50
60
70


Correlation between SCOOT Queue vs VISSIM queue (+25% Traffic Demand)
y = 0.6616x + 3.1133
R2 = 0.8506
VISSIM Queue [veh]
SCOOT Queue [veh]
0
10
20
30
40
50
60
70


Correlation between SCOOT Queue vs VISSIM queue (+25% Traffic Demand)
y = 0.6782x + 3.2897
R^2 = 0.7749
VISSIM Queue [veh]
SCOOT Queue [veh]
0
10
20
30
40
50
60

REFERENCES

1. Abdel-Rahim, A., Taylor, W.C., and Bangia, A. (1998). "The Impact of SCATS on Travel Time and Delay." Paper prepared for the 8th ITS America Annual Meeting, Detroit Michigan.

2. Abraham, J. K. (2000). "SCATS Traffic Signal System." http://www.ci.troy.mi.us/TrafficEngineering/sindex.htm. Accessed in June, 2003.

3. Akcelik, R., and Besley, M. (1998). *SIDRA 5 - User Guide.*

4. Bell, M.C. (1985). "Ageing of Fixed-Time Traffic Signal Plans." *Report to Transport and Road Research Laboratory*. Department of Engineering, University Durham, UK.

5. Bell, M.C., and Bretherton, R.D. (1986). "Ageing of Fixed-Time Traffic Signal Plans." In *Proceedings of the Second International Conference on Road Traffic Control*, Institution of Electrical Engineers, London, U.K., 77-80.

6. Bloomberg, L., and Dale, J. (2000). "A Comparison of the VISSIM and CORSIM Traffic Simulation Models." Presented at the Annual Meeting of the Institute of Transportation Engineers, Washington, DC.

7. Boillot, F., Blossville, J. M., Lesory, J. B., Motyka, V., Papageorgiou, M., and Sellam, S. (1992). "Optimal signal control of urban traffic networks." Paper presented at the IEE Conference on Road Traffic Monitoring and Control at Stevenage, England, 75-79.

8. Bretherton, R.D., and Bowen, G.T. (1990). "Recent enhancements to SCOOT - SCOOT version 2.4." In *Proceedings for the 3rd IEE International Conference on Road Traffic Control*, London, UK, 95-98.

9. Bretherton, R. D. (1996). "Current Development in SCOOT: Version 3." *Transportation Research Record 1554,* Transportation Research Board, National Research Council, Washington, D.C., 48-52.

10. Bretherton, R. D., Wood, K., and Bowen, G. T. (1998). "SCOOT Version 4." *Traffic Engineering and Control*, Vol. 39, No. 7, 425-427.

11. Chandler, M.J.H., and Cook, D.J. (1985). “Traffic control studies in London: SCOOT and bus detection.” In *Proceedings for the PTRC Annual Summer Meeting* at Seminar M, P269, 111-128.

12. Chang, E. C. P., Lei, J.C., and Messer, C. J. (1988). *Arterial Signal Timing Optimization Using PASSER II-87 Microcomputer User's Guide*. TTI Research Report 467-1.

13. Chilukuri, B.R., Perrin, J., and Martin, P. T. (2004). “SCOOT and Incidents: Performance Evaluation in Simulated Environment.” *Transportation Research Record 1867*, Transportation Research Board, National Research Council, Washington, D.C., 224-232.

14. Courage, K.G., Landman, M., Wallace, C.E., and Jacks, C. (1979). *Signal Operations Analysis Package (SOAP).*

15. Dell'Olmo, P., and Mirchandani, P. B. (1995). “REALBAND: An approach for real-time co-ordination of traffic flows on networks.” *Transportation Research Record 1494*, Transportation Research Board, National Research Council, Washington, D.C., 106-116.

16. Federal Highway Administration (FHWA). (1995). “Improving Traffic Signal Operations - A Primer.” http://www.itsdocs.fhwa.dot.gov//JPODOCS/REPTS_TE/13466.pdf. Accessed February, 17, 2004.

17. Federal Highway Administration (FHWA). (1996). *Traffic Control System Handbook*. U.S. Department of Transportation.

18. Federal Highway Administration (FHWA). (1999). *CORSIM User’s Manual, Version 4.32*. Office of Safety and Traffic Operations R&D, Intelligent Systems and Technology Division, McLean, Virginia.

19. Federal Highway Administration (FHWA). (2002). “Guidelines for Transportation Management Systems Maintenance Concept and Plans.” FHWA-OP-04-011. http://www.itsdocs.fhwa.dot.gov//JPODOCS/REPTS_TE/13882.html. Accessed March 20, 2004.

20. Fehon, K. (2004). “Adaptive Traffic Signals – Are we missing the boat?” Paper presented at the ITE District 6 2004 Annual Meeting. Institute of Transportation Engineers, Washington, DC.

21. Fehon, K. (2005). “North American Development of Adaptive Traffic Control Signal System.” Paper presented at Annual ITE Meeting, Melbourne,

Australia, 2005. Web site: http://www.dksassociates.com/papers/AB05H13.pdf Accessed: October, 2005.

22. Feng, Y., and Martin, P. (2002). "SCOOT Bus Priority Evaluation in VISSIM Simulation Environment." *Report UTL-0802-017/1*. University of Utah Traffic Laboratory, Salt Lake City, Utah.

23. Fox, K., Clarke, R., and Kirby, H. (1994). "Using Artificial Intelligence to Reduce High Fuel Consumption in Congested Cities." Website: http://www.its.leeds.ac.uk/projects/primavera/capri.html Accessed October, 2005.

24. Fox, K., Franklin, P., and Montgomery, F. (1995). "PRIMAVERA Final Report." Website: http://www.its.leeds.ac.uk/projects/primavera/finrep.html Accessed October, 2005.

25. Gartner, N. H. (1983). "OPAC: A Demand Responsive Strategy for Traffic Signal Control." *Transportation Research Record 906*, Transportation research Board, National Research Council, Washington, D.C., 75-81.

26. Ghaman, R. (Undated). Quoted in "Newsletter of the ITS Cooperative Deployment Network" Website: http://www.nawgits.com/icdn/rt-tracs.html Accessed May, 2004.

27. Hale, D. (2005). *Traffic Network Study Tool, TRANSYT-7F*, United States Version T7F10. McTrans Center, University of Florida, Gainesville, FL.

28. Hansen, B. G. and Martin, P.T. (1999). "Online Testing of SCOOT Using CORSIM: Development of a CORSIM Run-Time Extension." *Report No. UTL-1198-017*, University of Utah Traffic Laboratory, Salt Lake City, Utah.

29. Head, K.L., Mirchandani, P. B., and Shepherd, D. (1992). "A hierarchical framework for real-time traffic control." *Transportation Research Record 1360*, Transportation research Board, National Research Council, Washington, D.C., 82-88.

30. Hereford Department of Transport, Worcester County Council, and Transportation Planning Associates. (1986). "Evaluation of Urban Traffic Control in Worcester." TCC division, Tollgate House, Bristol, UK.

31. Hu, K. (2000). "LADOT's adaptive traffic control system." Paper presented at the Workshop on Adaptive Signal Control Systems, 79th Annual Meeting, Transportation Research Board, Washington D. C.

32. Hunt, P.B., Robertson, D. I., Bretherton, R.D., and Winton, R.I. (1981). "SCOOT – a Traffic Responsive Method of Coordinating Signals." *Laboratory Report 1014*, Transport and Road Research Lab, Crowthorne, Berkshire, U.K.

33. Husch, D., and Albeck, J. (2003) (I). *Synchro 6, Traffic Signal Software – User Guide*. Trafficware Corporation, Albany, CA.

34. Husch, D., and Albeck, J. (2003). (II) *SimTraffic 6, Traffic Simulation Software – User Guide*. Trafficware Corporation, Albany, CA.

35. Institute of Transportation Engineers (ITE). (2005). *Benefits of Retiming Traffic Signals*. Institute of Transportation Engineers, Washington, D.C., 2005.

36. ITT Industries. (2002). *Evaluation of the SR-522 Seattle, Washington ACS Field Test*. US Department of Transportation, FHWA, Travel Management R&D, September 30, 2002.

37. Jayakrishnan, R., Mattingly, S. P., and McNally, M. G. (2001). "Performance Study of SCOOT Traffic Control System with Non-ideal Detectorization: Field Operational Test in the City of Anaheim." Paper presented at the 80th Annual Meeting of the Transportation Research Board, National Research Council, Washington D.C.

38. Jhaveri, C. H., Perrin, J., and Martin, P. T. (2003). "SCOOT Adaptive Signal Control: Evaluation of Its Effectiveness over a Range of Congestion Intensities." Preprint CD-ROM of 82nd TRB Annual Meeting. TRB, National Research Council, Washington, D.C.

39. Khoudour, L., Lesort, J.B., and Farges, J.L. (1991). "PRODYN - Three years of trials in the ZELT Experimental Zone." *Recherché - Transports - Securite*, English Issue, No. 6. Special Traffic Management, 89-98.

40. Kosonen, I., and Bargiela, A. (1999). "A Distributed Traffic Monitoring and Information System." *Journal of Geographic Information and Decision Analysis. Vol. 3. No. 1.* 31-40.

41. Lee, S., Messer, C.J., Oh, Y., and Lee, C. (2004). "Assessment of Three Simulation Models for Diamond Interchange Analysis." *Journal of Transportation Engineering Vol. 130, No. 3.* American Society of Civil Engineers, Reston, VA, 304-312.

42. Little, J. D. C., Kelson, M. D., and Gartner, N. H. (1981). "MAXBAND: A program for setting signals on arteries and triangular networks."

Transportation Research Record 795, Transportation Research Board, National Research Council, Washington, D.C., 40-46.

43. Liu, D., and Cheu, R.L. (2004). "Comparative Evaluation of Dynamic TRANSYT and SCATS-Based Signal Control Logic using Microscopic Traffic Simulations." Preprint CD-ROM of 83rd TRB Annual Meeting. TRB, National Research Council, Washington, D.C.

44. Lowrie, P.R. (1982). "The Sydney Coordinated Adaptive Traffic System – Principles, Methodology, Algorithms." In *Proceedings of International Conference on Road Traffic Signaling*, Institution of Electrical Engineers, London, U.K., 67-70.

45. Lowrie, P. R. (1992). *SCATS: A Traffic Responsive Method of Controlling Urban Traffic Control.* Roads and Traffic Authority, Sydney, New South Wales, Australia.

46. Luk, J.Y.K., Sims, A.G., and Lowrie, P.R. (1982). "SCATS – Application and Field Comparison with a TRANSYT Optimised Fixed Time System." In *Proceedings of International Conference on Road Traffic Signaling*, Institution of Electrical Engineers, London, U.K., 71-74.

47. Luk, J., Sims, A., and Lowrie, P. (1983). "Evaluating Four Methods of Area Traffic Control – The Parramatta Experiment." *Australian Journal of Instrumentation & Control, Vol. 39, No. 4*, Australian Road Research Board, Vermont South, Australia, 75-77.

48. Luyanda, F., Gettman, D., Head, L., Shelby, S., Bullock, D., and Mirchandani, P. (2003). "ACS-Lite Algorithmic Architecture – Applying Adaptive Control System Technology to Closed-Loop Traffic Signal Control Systems." *Transportation Research Record 1856,* Transportation research Board, National Research Council, Washington, D.C., 175-184.

49. MacGowan, J., and Fullerton, I. J. (1979). "Development and testing of advanced control strategies in the urban traffic control system." *Public Roads, Vol. 43, No. 2, 3, and 4*.

50. Martin, P.T., and Hockaday, L. M. (1995). "SCOOT - an update." *ITE Journal Vol. 65, Institute* of Transportation Engineers, 44-48.

51. Mauro, V., and DiTaranto, C. (1990). "UTOPIA." *Control, Computers, Communications in Transportation: Selected Papers from the IFAC Symposium*, Pergamon Press, 245 – 252.

52. Mazzamatti, M.V., Netto, D. V. V. F., Villanova, L. M., and Ming, S. H. (1998). "Benefits gained by responsive and adaptive systems in São Paulo." Presented at the IEE Road Transport and Information Control Conference, Publication No. 454. 21-23 April, London, UK.

53. Mirchandani, P.B. and Head, K. L. (2001). "RHODES: A Real-Time Traffic Signal Control System: Architecture, Algorithms and Analysis." *Transportation Research*. Part C: Emerging Technologies Vol. 9 No: 6.

54. Mystkowski, C. and Khan, S. (1999). "Estimating Queue Lengths Using SIGNAL94, Synchro3, TRANSYT-7F, PASSER II_90, and CORSIM." Presented at the 78th Annual Meeting of the Transportation Research Board, Washington, D.C.

55. Park, B.B., Rouphail, N.M., and Sacks, J. (2000). "Assessment of a Stochastic Signal optimization Method Using Microsimulation." Technical Report No 110, National Institute of Statistical Sciences, Research Triangle Park, NC, 2000. Website: http://www.niss.org/technicalreports/tr110.pdf. Accessed April, 2004.

56. Park, B.B., Rouphail, N. M., Hochanadel, J.P., and Sacks, J. (2001). "Evaluating Reliability of TRANSYT-7F Optimization Schemes." *Journal of Transportation Engineering Vol. 127, No. 4*. American Society of Civil Engineers, Reston, VA, 319-326.

57. Peck, C., Gorton, P.T.W., Liren, D. (1990). "The application of SCOOT in developing countries." In *Proceedings for the 3rd International IEE Conference on Road Traffic Control*, London, U.K., 104-109.

58. Powell, R. J. (1985). "SCOOT in Southampton." In *Proceedings for the the PTRC Annual Summer Meeting* at Seminar M, P269, 97-110.

59. Planung Transport Verkehr (PTV) AG. (2005). *VISSIM 4.10 User Manual.* Stumpfstraße 1, D-76131 Karlsruhe, Germany.

60. Quan, B.Y., Greenough, J.C., and Kelman, W.L. (1993). "The Metropolitan Toronto SCOOT demonstration project." Paper presented at the 72nd Annual Meeting of the Transportation Research Board at Washington, D.C.

61. Rakha, H., and Van Aerde, M. (1995). "REALTRAN: An Off-Line Emulator for Estimating the Effects of SCOOT." *Transportation Research Record 1494*, Transportation Research Board, National Research Council, Washington, D.C., 124-128.

62. Robertson, D.I. (1969). "TRANSYT: A traffic network study tool." *Laboratory Report 1014*. Transport and Road Research Lab, Crowthorne, Berkshire, U.K.

63. Robertson, D.I., and Hunt, P.B. (1982). "Method of estimating the benefits of coordinating signals by TRANSYT and SCOOT." *Traffic Engineering & Control Vol. 23, No.11*, Printerhall Limited, London, U.K., 527-531.

64. Robertson, G.D. (1987). "Handling congestion with SCOOT." *Traffic Engineering & Control Vol. 28, No.4*, Printerhall Limited, London, U.K., 228-230.

65. Rouphail, N. M., Park, B.B., and Sacks, J. (2000). *Direct Signal Timing Optimization: Strategy Development and Results.* Technical Report No 109, National Institute of Statistical Sciences, Research Triangle Park, NC. http://www.niss.org/technicalreports/tr109.pdf. Accessed April, 2004.

66. Sabra, Z., Wallace, C. E., and Lin, F. (2000). "Traffic Analysis Software Tools." *Circular Number E-CO14*. Transportation Research Board, National Research Council, Washington D.C.

67. Shelby, S. G., Bullock, D., and Gettman, D. (2005). *Resonant Cycles in Traffic Signal Control.* Preprint CD-ROM of 84th TRB Annual Meeting. TRB, National Research Council, Washington, D.C.

68. Siemens Traffic Controls Ltd. (2003). *SCOOT User Guide*. Version 4.2.

69. Stallard, C., and Owen, L. E. (1998). "Evaluating adaptive signal control using CORSIM." In Proceedings of the 1998 Winter Simulation Conference, 1147-1153.

70. Strong Concepts. (1995). *SIGNAL94 Tutorial/Reference Manual.*

71. Sunkari, S. (2004). "The Benefits of Retiming Traffic Signals." *ITE journal, Vol. 74, No.4*, Institute of Transportation Engineers, 26-30.

72. Swayampakala, R. K., and Graham, J.R. (2005). "Optimum Time Intervals for the Traffic Signal Re-Timing Process." Preprint CD-ROM of 84th TRB Annual Meeting. TRB, National Research Council, Washington, D.C.

73. Taale, H., Fransen, W. C. M., and Dibbits, J. (1998). *The second assessment of the SCOOT system in Nijmegen.* Presented at the IEE Road Transportation Information and Control Conference, Publication No. 454. 21-23 April, London, UK.

74. Tarnoff, P.J., and Ordonez, J. (2004). "Signal Timing Practices and Procedures – State of the Practice." *Publication FHWA-HOP-05-015*. Institute of Transportation Engineers.

75. Tian, Z.Z., Urbanik, T., Engelbrecht, R., and Balke, K. (2002). "Variations in Capacity and Delay Estimates from Microscopic Traffic Simulation Models." *Transportation Research Record 1802,* Transportation Research Board, National Research Council, Washington, D.C., 23-31.

76. Transport Research Laboratory (TRL). (2003). *SCOOT Workshop.* Material from the SCOOT Workshop held on 14th – 15th May 2003. Transport Research Laboratory, Crowthorne, United Kingdom.

77. Trueblood, M. (2005). "Should I Use CORSIM or SimTraffic?" HDR Engineering, Inc. Omaha, Nebraska. www.trafficware.com/downloads/crsmvst.pdf. Accessed May, 2005.

78. TYCO International Ltd. (2003). "SCATS Version 6 Booklet." http://www.traffic-tech.com/pdf/scatsbooklet.pdf. Accessed December 2003.