Impacts of Traffic Signal Control Strategies

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Royal Institute of Technology
Stockholm, Sweden 2006
To my mother, who brought me up, and to my son, whom I live for
Acknowledgements

This dissertation is a result of the research project entitled “Development of models for impact analysis of road traffic facilities (EMV)” funded by the Swedish Road Administration (SRA). Professor Karl-Lennart Bång, Division of Transport and Logistics (ToL), has been my supervisor and has provided very valuable advice and guidance throughout my work for which I am deeply grateful.

This work is a product of assistance and support of many people. I would like to express special thanks to the following colleagues:

Andrew Cunningham and Jeffery Archer, with whom I have written two papers, which are incorporated in this thesis. Their contribution has mainly been in software programming of the evaluated signal control strategies.
Carlos Moran and Johan Wahlstedt, who have assisted with field data reduction. Carlos has also contributed with driver behavior analysis.
Stefan Eriksson, Lennart Leo, and Björn Bergman, who helped out with field measurements.

In addition, many thanks to Albania Nissan, who supported me with her humor and generosity; and to other colleagues working at our division, who helped me with various details, especially Brigitt Högb erg, Katarina Fogelström and my project colleague Karin Aronsson.

Last but not least, I would like to thank my opponent (examiner) in the final seminar, Frank Montgomery, Director of Learning and Teaching, Institute for Transport Studies, University of Leeds, for his valuable advice and suggestions regarding this work.

Stockholm, December 2006

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Abstract

Traffic signals are very cost effective tools for urban traffic management in urban areas. The number of intersections in Sweden controlled by traffic signals has increased since the seventies, but efforts to study the traffic performance of the employed strategies are still lacking. The LHOVRA technique is the predominant isolated traffic signal control strategy in Sweden. Past-end green was originally incorporated as part of LHOVRA (the “O” function) and was intended to reduce the number of vehicles in the dilemma zone. Coordinated signal control in Sweden is often fixed-time with local vehicle actuated signal timing adjustments and bus priority. This research study was undertaken to increase the knowledge of the traffic performance impacts of these strategies.

The aim was to evaluate the following control strategies using Stockholm as a case study:

1. The LHOVRA technique with a focus on the “O” function;
2. Fixed time coordination (FTC);
3. Fixed time coordination with local signal timing adjustment (FTC-LTA);
4. FTC-LTA as above + active bus priority (PRIBUSS);
5. Self-optimizing control (SPOT).

Field measurements were used for study of driver behavior and traffic impacts as well as for collecting input data needs for simulation. The results from low speed approaches showed a higher proportion of stopped vehicles after receiving green extension. Moving the detectors closer to the stop line, and/or making the detectors speed dependent were suggested as measures to solve these problems. The VISSIM simulation model calibrated and validated with empirical data was used to study traffic performance and safety impacts of the LHOVRA technique as well as to test the suggested improvements. The simulation experiment results from these design changes were shown to reduce accident risk with little or no loss of traffic performance.

TRANSYT was used to produce optimized fixed signal timings for coordinated intersections. HUTSIM simulations showed that local signal timing adjustment by means of past-end green was beneficial when applied to coordinated traffic signal control in the study area. Both delays and stops were reduced, although not for the main, critical intersection which operated close to capacity.

To study the impacts of strategies for coordinated signal control with bus priority, extensive field data collection was undertaken during separate time periods with these strategies in the same area using mobile and stationary techniques. A method to calculate the approach delay was developed based on the observed number of queuing vehicles at the start and end of green. Compared to FTC-LTA, the study showed that PRIBUSS reduced bus travel time. SPOT reduced both bus and vehicle travel time.

Future research efforts for the development of signal control strategies and their implementation in Sweden should be focused on strategies with self-optimization functionality.

Key words: Driver behavior, Dilemma zone, Incident reduction, Signal control, Lhovra, Spot, Pribuss, Bus priority, Field measurement, Simulation, Calibration, Traffic performance, Stopped delay, Traffic safety.
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1 EXECUTIVE SUMMARY

Traffic signals are one of the most powerful tools for urban traffic control available to city authorities. Their correct installation can improve both traffic flow and the safety of all road users. In comparison to other traffic improvements, signals are also relatively low capital intensive. In Sweden, the number of intersections controlled by traffic signals has increased since the seventies, but the efforts to study the traffic performance of the employed strategies are still lacking.

The LHOVRA technique is the dominating isolated traffic signal strategy in Sweden. It was originally developed in order to increase safety and to reduce lost time and the number of stopped vehicles at signalized junctions along high-speed roads (70 km/h or more). The “O” function in LHOVRA incorporated past-end green (PEG) with the intention of reducing the number of vehicles in the dilemma zone and thereby reducing the number of red light drivers and rear-end collisions. LHOVRA has been shown to be effective in such environments.

More recently, the “O” function has also been adapted to allow vehicle-actuated past-end green in fixed time coordinated systems. In Stockholm, the timing of the signals is normally performed manually taking into consideration local signal timing adjustment based on detector inputs controlling the termination of the green signal by using PEG.

Delay at signalized intersections may constitute a significant part of bus journey times in the urban environment. Giving buses priority at traffic signals can be an effective measure to reduce this delay. Bus priority in Swedish urban traffic signal systems, called PRIBUSS, normally operates with fixed time plan selection. Weighting bus priority has also been introduced in Swedish market using self-optimizing strategy SPOT.

Due to lack of knowledge of traffic performance impacts of these techniques in urban environments, a major research study was undertaken funded by the Swedish Road Administration (SRA). The aim was to evaluate the following control strategies using Stockholm as case study:

1. LHOVRA technique with the focus on the “O” function;
2. Fixed time coordination (FTC);
3. Fixed time coordination with local signal timing adjustment (FTC-LTA);
4. FTC-LTA as above + active bus priority (PRIBUSS);
5. Self-optimizing control (SPOT).

The methodology for this study included field data collection using mobile and stationary techniques, offline signal timing calculations with TRANSYT, and microscopic simulation modeling using the HUTSIM and VISSIM models.

This thesis is divided in four parts and a synthesis, as follows:

Part I Overview of urban traffic signal control strategies and evaluation methods.
Part II Impacts of isolated signal control with the LHOVRA technique.
Part III Impacts of fixed time coordination with local signal timing adjustment.
Part IV Impacts of strategies for coordinated signal control with bus priority.
Part V Synthesis and Conclusions.
The study in Part I aimed to provide a general idea of:

- The Urban Traffic Signal Control (UTSC) strategies;
- The methods for measuring the impacts of UTSC strategies.

Study and experience have shown that

1. Field measurements are very time consuming and expensive;
2. Analytical methods can be applied for fixed-time control of isolated intersections and then adjusted for vehicle-actuated control and coordinated systems;
3. Simulation offers a useful offline test environment in which changes in detector positions and signal controller logic can be made quickly without jeopardizing the safety of road users.

Field observations were used to study driver behavior in Part II. Field measurements were also used for studying the traffic impacts as well as for collecting input data needed for simulation. Simulation models calibrated and validated with empirical data were used to study traffic performance and safety impacts of existing control strategies and to test some suggested improvements.

The study in Part II showed that, depending on the actual speed distribution in the approaches, the effectiveness of the “O” function varied considerably. Many ideas to improve this function were suggested including: moving the detectors closer to the stop line and making the detectors speed dependent.

A micro-simulation experiment based on field data was performed to test design changes. VISSIM was selected mainly due to the possibility to define behavior in relation to the onset of amber “reaction-to-amber”. Particular attention has been given to the “reaction-to-amber” function in VISSIM, which can be assigned to allow the vehicle/driver only one decision to stop or go when there is a signal change from green to amber.

The simulation experiment considered four different scenarios, two different detector positions and two different signal controller logic programs for the “O” function. The detector positions are:

(a) 130 and 80 meters (standard recommendation);
(b) 110 and 65 meters (suggested).

The different signal controller logic programs are:

(i) Standard “O” function implementation (i.e. the norm);
(ii) Speed dependent “O” function implementation with a minimum speed threshold of 56.5 km/h, below which past-end green will not be applied (suggested).

The results confirmed the ideas and showed that:

- A distance closer to the stop line (110 and 65 meters) has positive effects on safety in terms of a reduction in red-light violations and measures of Time To Collision (TTC) and positive effects on performance in terms of the proportion of the green time to the cycle time;
- A speed dependent “O” function with a greater distance to the stop line (130 and 80 meters) reduces wasted green time and the number of stops after receiving PEG;
- A speed dependent “O” function with a shorter distance to the stop line (110 and 65 meters) reduces red light violations and the number of conflicts;
Furthermore, it has been shown that a speed dependent “O” function may have the ability to eliminate conflicting situations completely if PEG is only given to those vehicles that really need it.

To evaluate local traffic actuated signal-timing adjustments by means of PEG the following control strategies were studied in Part III:
1. Fixed time coordination without local signal timing adjustment (FTC);
2. Fixed time coordination with local signal timing adjustment (FTC-LTA).

The TRANSYT model was used purely to generate optimized signal timings for input to micro-simulation using the HUTSIM software. The study gave the following results:

In the main intersection
- Local traffic adjustment with manual FTC increased total delay by 9%;
- Signal timings determined using TRANSYT reduced the average intersection delay by 11% compared to manual signal settings. Local traffic adjustment had little effect (reduced total delay by a further 1%).

In the studied area
- Local traffic adjustment with manual Fixed Time Coordination (FTC) had little effect (reduced total delay by 1%);
- Signal timings determined using TRANSYT reduced the average intersection delay by 9% compared to manual signal settings. Local traffic adjustment (LTA) reduced total delay by a further 5%.

The study highlighted the effectiveness of TRANSYT in producing optimized fixed signal timings for coordinated intersections. HUTSIM simulations showed that local signal timing adjustment by means of past-end green, originally designed to improve safety and traffic performance of high-speed isolated intersections, was beneficial when applied to coordinated traffic signal control in the study area. Both delays and stops were reduced, although not for the main, critical intersection which operated close to capacity.

Comparative results from simulation and field measurements (part IV) are available for the main intersection with the manual FTC with LTA under non-peak traffic. The HUTSIM simulation results of stops and delay were about 14% higher than the corresponding field results. It should be noted that HUTSIM does not totally reflect the actual on-street situation. In reality, parked vehicles, bicycles and pedestrians often reduce traffic performance. However, since all cases have been assessed using HUTSIM, all the results are comparable.

To study the impacts of strategies for coordinated signal control with bus priority, extensive field data collection was undertaken during separate time periods with these strategies in the same area using mobile and stationary techniques. The aim was to evaluate the following control strategies using Stockholm as a case study:
1. Fixed time coordination with local signal timing adjustment (FTC-LTA);
2. FTC-LTA as above with active bus priority (PRIBUSS);
3. Self-optimizing control (SPOT) with active weighted bus priority.

A method to calculate the approach delay was developed, based on the observed number of queuing vehicles at the start and end of green. Compared to FTC-LTA, the study showed that:
In the main intersection
- Bus travel time was reduced by 14% using PRIBUSS and 12% using SPOT;
- Travel time for all vehicles increased by 24% using PRIBUSS and was increased by 30% using SPOT.

In the network
- Bus travel time was reduced by 11% using PRIBUSS and 28% using SPOT;
- Travel time for all vehicles did not increase using PRIBUSS, and was reduced by 6.5% with SPOT.

In the whole network, SPOT performed best at all times of the day with some exceptions at the oversaturated main intersection. A general comparison for both buses and vehicles with buses given a weight of 20 or 25 vehicles resulted in a higher reduction in travel time with SPOT than with PRIBUSS.

Comparable results from simulation and field measurements (parts III & IV) are available for the Main Intersection stops and delay with the manual FTC with LTA during non-peak traffic. The HUTSIM simulation results of stops and delay are about 14% higher than the corresponding field measurements. It should be noted that HUTSIM does not totally reflect the real on-street situation. In reality, parked vehicles, bicycles, and pedestrians often reduce traffic performance.

Modern micro-simulation techniques calibrated with observed driver behavior parameters now provide an opportunity to develop and test more advanced signal control strategies. The work conducted in the present study has demonstrated the potential of this method. Furthermore, the availability of emulators linking simulation software to signal controller hardware facilitates the very complex tasks involved in this process.

The basis for the development of signal control strategies and their implementation should be to provide strategies with self-optimization functionality. This implies an ability for the traffic engineer to apply traffic policy objectives to the relative weight of different traffic elements and impacts.
2 INTRODUCTION

2.1 BACKGROUND
There is no doubt that time separation of traffic conflicts using traffic signals is one of the most powerful tools for urban traffic control available to city authorities. Their correct design and operation can improve both traffic performance and the safety of all road users (TRL, 1996). In comparison to other traffic improvements, signals are also relatively low-capital-intensive. In recent years, advances in informatics and telecommunications have led to a new generation of low cost controllers and self-optimizing systems that have made modern signaling even more cost-effective. Based on results from experimental investigations in Sweden and abroad (Davidsson, 1990) self-optimizing systems have the possibility of reducing traffic costs by 10-20%. However, due to budgetary problems and generally low interest on the part of politicians and decision makers in investing in technology for improved traffic performance in urban street networks, the level of new investments in this area is very low in spite of the fact that investments in traffic signals are most cost-effective.

The number of intersections controlled by traffic signals has increased in Sweden since the seventies. According to the Swedish Road Administration (SRA), there are approximately 3,200 traffic signal installations in Sweden, two thirds of which are signalized intersections. The majority of the signalized intersections operate as isolated intersections, employing gap extension signal group based control, using the so-called LHOVRA strategy. Fixed time coordination of intersections in urban areas constitutes less than 50 per cent of all the signalized intersections in Sweden (Davidsson 1990).

Design and operation of effective, safe and environmentally friendly intersections require a high level of knowledge about the relationships between intersection design, traffic flow, environment, and impacts on traffic performance, safety and emissions (Bang, 1997). However, the efforts to study impacts of control strategies on traffic performance are still very limited. The Swedish Road Design Manual VU94 produced by the SRA, also lacked a chapter dealing with traffic signal control issues in its first version (Vägverket, 1994).

Bang et al (1978) developed procedures for capacity analysis of traffic signals as a part of the Swedish Capacity Manual. Computer aids for the capacity calculation procedures were later developed by the SRA (1981) and Hagring (2000).

Swedish isolated traffic signal control is normally traffic actuated using a specially devised strategy called LHOVRA (Vägverket, 1991), which includes an incident reduction function designed to reduce the number of vehicles in the “Dilemma zone”. This is defined as an area in the approach to the stop-line where a driver on seeing amber may not be able to stop in advance of the stop line with an acceptable deceleration rate, or to clear the intersection during the change interval (Bang et al 1964) and (Huang & Pant, 1994). Reducing the number of vehicles in the dilemma zone and thereby reducing red light driving and rear-end collisions is achieved by detecting vehicles at the beginning of the dilemma zone and postponing a decided change to amber using past-end-green (PEG). Al-Mudhaffar (1998) developed a concept that improves this function by integrating the PEG in the maximum green time. This reduces the delay without impairing safety. The LHOVRA technique belongs to the generation of the VA control lacking data collection for evaluation and self-adjustments. Accordingly, there is not enough knowledge regarding the impacts of LHOVRA.
A major Swedish study of self-optimizing isolated signal control was undertaken by Bang (1976) with the aim of developing strategies inspired by a method for real time optimization proposed by Miller (1963). The system, called TOL, showed good results in simulation and full-scale field trials compared to traditional Swedish vehicle actuated control. Kronborg et al (1997) combining real time optimization with the Scandinavian tradition of signal groups developed a modified Self Optimizing Signal (SOS) control strategy. It seems that it now is a good time to develop the popular Swedish LHOVRA technique.

Coordinated traffic signal control systems in Sweden normally operate with fixed time or traffic actuated signal plan selection with the purpose of obtaining “green waves”, reducing delays and the number of stops along signalized routes. Some municipalities use the offline TRANSYT software for this purpose (Vincent et al, 1980). In Stockholm, the timing of the signals is normally performed manually taking into consideration local signal timing adjustment based on detector inputs controlling the termination of the green signal by using PEG. This function was originally intended for use at isolated junctions on high-speed roads as a means of reducing the number of vehicles in the dilemma zone. The effects of PEG with regard to capacity and delay on congested urban networks have received little attention, particularly in low speed urban networks. The Swedish way of coordinated design needs to be modernized and self-optimized signal control is an alternative to be studied.

Urban signal control in major Swedish cities also normally incorporates active bus priority - PRIBUSS (GFK, 1991) that aims to display a green signal at the arrival of the bus at the stop line. Decentralized systems for self-optimized signal control of urban signal networks, e.g. the Italian SPOT system (Mizar, 2001), are also being introduced on the Swedish market. Due to a lack of knowledge of traffic performance impacts of these techniques, a major research study was undertaken from 2000 – 2005, funded by the Swedish Road Administration. The aim was to evaluate different strategies using Stockholm as a case study. This thesis is based on that study.

2.2 OBJECTIVE
Traffic signal control is a very cost-effective method for the improvement of urban traffic systems in terms of performance, safety and environment. The level of knowledge regarding the impacts of different types of systems and control strategies is insufficient. The same is true of methods of evaluation and assessment of such systems as shown in the previous section. The overall aim of the thesis was to reduce this knowledge gap through the development and evaluation of urban traffic signal control strategies.

2.3 SCOPE
The main focus of this thesis is on strategies applicable to Swedish traffic conditions, street network configurations, and traffic signal hardware. The scope includes the following items:

- To review current urban traffic signal control strategies and methods to measure and assess their impacts on traffic performance;
- To evaluate impacts of the LHOVRA incident reduction “O” function and possibilities to improve its effectiveness;
To evaluate the use of PEG for local signal timing adjustment within the Fixed Time Coordination (FTC);

To evaluate the traffic performance of the coordinated traffic signal control strategies with bus priority (PRIBUSS and SPOT) compared to Fixed Time Cooperation with local timing adjustment (FTC-LTA).

2.4 LIMITATIONS
The limitations in this thesis are as follows:

- The impacts of strategies focus on the ones used in Sweden;
- The study does not include the impacts on pedestrians.

2.5 STRUCTURE OF THE THESIS
This thesis is divided in four parts and a synthesis, as follows:

Part I Overview of urban traffic signal control strategies and evaluation methods.

Part II Impacts of isolated signal control with the LHOVRA technique.

Part III Impacts of fixed time coordination with local signal timing adjustment.

Part IV Impacts of strategies for coordinated signal control with bus priority.

Part V Synthesis and Conclusions.

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Part I

Overview of Urban Traffic Signal Control Strategies and Evaluation Methods
ABSTRACT

The number of intersections controlled by traffic signals has increased in Sweden since the seventies, but efforts to study the traffic performance of employed strategies are still lacking.

This part of the thesis, by literature study, aims to give a general idea about
- The Urban Traffic Signal Control (UTC) strategies;
- The methods for measurement of the impacts of the UTC strategies.

Some reflections regarding the area of strategies comparison are:
- Terms are understood in different ways;
- Every new strategy emerges where a new system is needed;
- Some studies become a marketing tool for a product;
- Replacing an entire system is not easy. This is due to:
  - Current engineers and experts are masters of the old system,
  - The new system demands new knowledge that has to be obtained and developed,
  - The new system needs high cost equipment and installation,
  - It can sometimes be more profitable to improve an existing strategy than to replace it.

Some reflections regarding the area of the measurement methods are:
- Field measurements are a very time-consuming and expensive;
- Analytical methods are applied for the fixed time isolated intersections and then adjusted for the vehicle-actuated systems and the coordination systems;
- Simulation offers a useful offline test environment in which changes like detector positions and signal controller logic can be made quickly without jeopardizing the safety of road users.

Therefore, it is recommended to use the simulation models for the vehicle-actuated and self-optimizing strategies and to use field measurements to both study driver behavior and to assess simulation input data needs.
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1 OVERVIEW OF TRAFFIC SIGNAL CONTROL STRATEGIES

The U.S. Highway Capacity Manual HCM 2000 (TRB 2000) states that “A traffic signal essentially allocates time among conflicting traffic movements that seek to use the same space”. This definition concurs with the Swedish definition of traffic signal control (Bang et al 1978) as “time separation between conflicting traffic movements with the use of signals”.

Traffic signals are thus used in at-grade intersections to reduce conflicts to a minimum by time-sharing of the right of the way. This greatly enhances safety, but may reduce the capacity of the intersection. And it is well known that using traffic signals often runs the risk of being considered a panacea for all traffic problems. Many countries have therefore developed criteria and warrants for signal installation depending on the traffic flow, visibility and registered accidents (Webster and Cobbe, 1966; Bang, 1978).

1.1 HISTORICAL REVIEW

The first traffic signal was installed in London in 1868 and used semaphore “arms” together red and green gas lamps. Unfortunately it exploded, putting an end to this sort of control for 50 years. However, in 1918 the first three-colored light signals were installed in New York and in 1925 they began to be used in Great Britain (Webster & Cobbe, 1966).

At the beginning of the 1930s, an attempt to make the signals more “intelligent”, or vehicle responsive, was made in America, using microphones at the side of the road, requiring drivers to sound their horns. This was obviously not too popular and the first traffic detectors – electrical and pneumatic – were invented.

Traffic signals are now used throughout the world, using the three light signals of green, red and amber. Also, by convention, these are normally arranged vertically with red at the top and the green light at the bottom. This also helps people who are colorblind – both drivers and pedestrians – to identify the differences between the lights.

In Sweden, the first traffic signal was installed in Stockholm 1925 at the Kungsgatan – Drottninggatan intersection, with two lights and manual regulation. Three-light traffic signals and automatic fixed time control were first used in 1930. The 1950s were a peak period for implementation of traffic signals in Sweden using the traditional two detector methods (Bang, 1975). The LHOVRA strategy was developed in the 1970s in order to increase safety and to reduce lost time and the number of stopped vehicles at signalized intersections along high-speed roads (70 km/h or more), which are frequent around the major cities Stockholm, Gothenburg, and Malmö (Davidsson 1990).

1.2 OVERVIEW OF TRAFFIC SIGNAL CONTROL METHODS

A traffic signal controller allocates right-of-way at an intersection through a sequence of green signals. Each approach or separate movement is allocated to a phase. Inter-green times are specified between conflicting phases. Sets of non-conflicting phases (with some exceptions for turning traffic) are grouped into stages. Stages are arranged to follow a set order. A complete series of stages is called a cycle, see Figure 2.1 (page 7).
The methods to control the traffic signals can be divided in two main categories:

- Isolated traffic signal control, in which the signal timing decisions are based solely on the traffic demand in the approaches to the intersection.
- Coordinated traffic signal control, in which the signal timing decisions are based also taking into consideration other adjacent traffic signals to which the intersection controller is connected in order to facilitate passage of the signalized system, see Figure 2.2.

Bang (TFK, 1982) has divided the methods to operate the intersections as follows:

1.2.1 Isolated Traffic Signal Control

a. Fixed time signal control (FT): Predetermined, fixed signal timing (called time plan) calculated to minimize overall intersection delay for the traffic demand during the studied period as described in Section 2.3.1 below. Separate time plans can be developed for different periods during the day, e.g. morning peak, mid-day, afternoon peak, night, for which the signal timing plan is designed.

b. Vehicle actuated control (VA): Variable green time allocations and cycle time based on detection of the traffic demand in the signalized approaches or groups of lanes (signal groups). The decision to extend green light or not is based solely on the conditions for the actual approaches or signal groups served by the ongoing green. See Section 2.3.2 below.

c. Self-optimized real-time control: Variable green time allocation and cycle time based on real-time optimization of traffic performance with regard to the conditions for all the signalized approaches in the intersection. See Section 2.3.3 below.

In addition to these methods the signals could also be controlled manually (usually requires police officers) or be put in flashing amber mode (out of operation)

1.2.2 Coordinated Traffic Signal Control

a. Fixed time coordination: Operation of all signalized intersections belonging to the coordinated system with pre-determined, fixed time parameters (cycle time, green times, offsets) in a number of time plans designed for given traffic situations as explained above. The selection of time plan is also fixed following a pre-determined time schedule based on historic traffic demand variations.

b. Fixed time coordination with traffic actuated time plan selection: Same as a) above but with the selection of different time plans based on traffic data from selected detectors in the system.

c. Fixed time coordination with local signal timing adjustment (LTA). The LTA is based on traffic detector inputs from selected approaches in each intersection, and is designed to provide adaptation to short-term traffic variation through small adjustments of signal timing within the framework of the fixed time coordination.

d. Traffic actuated time plan calculation: The time plans are recalculated at regular intervals based on the information collected from selected detectors located in strategic positions.

e. Dynamic coordination. Calculation of all signal timing events and parameters in real time based on input from traffic detectors in a signalized intersection.
Figure 2.1 Traffic signal cycle time, stages, and phases (TRL, 2002)

Figure 2.2 Green waves in coordinated system (TFK, 1982)
Coordination of traffic signals can include several intersections along a road or in an area. The objective of signal coordination is usually to minimize the travel time for all vehicles in the system. This is also advantageous from an environmental point of view.

The traffic signal coordination can be achieved from a special unit for central control, or at a local level with some form of linking between individual intersections. Fixed time control is used today mainly in coordinated systems where a constant offset of the green light between adjacent intersections is desired.

### 1.3 STRATEGIES FOR ISOLATED TRAFFIC SIGNAL CONTROL

The development of traffic signal control has been influenced by the fast development of computer technology that has made it possible to use more complex strategies for both isolated and coordinated traffic signal control. Such strategies enable the use of self-optimizing strategies with performance functions aimed at minimizing the total vehicle delay, the number of stopped vehicles, or a general cost function combining the effects of delay and stopped vehicles. A remaining problem, which makes complex strategies expensive to implement is the need for accurate detection of the movements and discharge of all vehicles in the system.

In isolated signal control it is normally assumed that the arrival of vehicles in the approaches to the intersection is random with a negative exponential time headway distribution.

\[ f(h) = qe^{-qh} \]

where:
- \( h \) = Headway
- \( q \) = Traffic flow = \( \frac{1}{h} \)

A brief review of the main methods for isolated traffic signal control listed in Section 1.2.1 is presented below as a background for further analysis of some of these strategies.

#### 1.3.1 Fixed Time Control

With fixed time signals, the green and cycle times are predetermined and have fixed duration. Fixed signal timing calculated to minimize overall intersection delay for the traffic demand during the studied period.

Computations of delay, which were carried out for a variety of flows, saturation flows and signal settings, and from the results a formula was deduced for the average of any signal on an approach to an intersection. Webster (1966) found that

\[ d = \frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)} - 0.65 \left( \frac{c}{q^2} \right)^{1/3} * x^{(2-5\lambda)} \]
where:
\[
d = \text{Average delay per vehicle on the particular arm}
\]
\[
q = \text{Traffic flow}
\]
\[
c = \text{Cycle time}
\]
\[
g = \text{Effective green time}
\]
\[
\lambda = \text{Proportion of the cycle which is effectively green (i.e. } g/c \text{)}
\]
\[
x = \text{The degree of saturation (i.e. } q/\lambda s \text{)} (s = \text{saturation flow})
\]

To determine the minimum cycle time, the arms with the highest ratio of flow to saturation flow are selected from each phase. By differentiating the equation for the overall delay, Webster (1966) found that minimization of the overall delay at an intersection with respect to the cycle time could be represented by

\[
c_o = \frac{aL + b}{1 - \sum y_i} = \frac{aL + b}{1 - Y} \text{ sec}
\]

where:
\[
y_1, y_2, \ldots, y_n = \text{The maximum ratios of flow to saturation flow for phases 1,2,\ldots,n,}
\]
\[
Y = \sum y
\]
\[
L = \text{The total lost time per cycle.}
\]
\[
a & b \text{ are constants.}
\]

For a certain balance of flows the values of \(a\) & \(b\) are 1.5 & 5 respectively:

\[
c_o = \frac{1.5L + 5}{1 - Y} \text{ sec}
\]

This cycle time \(c_o\) is the “optimum cycle time”, which under light traffic conditions could be very short. From a practical point of view, including safety considerations, it may be desirable to consider it as lying between 25 and 120 seconds.

It has been found that for cycle times within the range three-quarters to one and a half times the optimum value, the delay is never more than 10 to 20 percent above that given by the optimum cycle (Webster, 1966). Some examples of the variation of delay with cycle time are shown in Figure 2.3.
Figure 2.3 Effect of variation of cycle time on delay (Webster, 1966)

For setting the green times, it was found that the ratio of the effective green times should equal the ratio of the $y$ values, i.e.

$$\frac{g_1}{g_2} = \frac{y_1}{y_2}$$

where $g_1$ and $g_2$ are the effective green times of phases 1 and 2 respectively.

If $c_o - L$ is the total effective green time in the cycle, the above rule gives

$$g_1 = \frac{y_1}{Y} (c_o - L)$$

$$g_2 = \frac{y_2}{Y} (c_o - L)$$

etc.

1.3.2 Vehicle Actuated Control (VA)

Some form of vehicle-actuated control (VA) is applied at virtually all isolated signalized intersections in Sweden because of its adaptability to short-term traffic variations. VA-control
requires detectors to be installed in all signalized approaches to detect vehicle passages and/or vehicle presence. This information is used for the following purposes:

1. To register demand for green light for vehicles arriving during red signal in the approach;
2. To register demand for green time extension for vehicles arriving during green light in the approach;
3. To register presence of vehicles within the detection area in the approach after the termination of green, i.e. overflow of a queue to the next signal cycle.

Extensions in VA-control normally occur with a predetermined extension interval \((f)\), as long as the time interval between vehicles passing these detectors is shorter than \((f)\), subject to minimum and maximum green time \(g_{\text{min}}, g_{\text{max}}\) restrictions. In traditional VA-control, \(g_{\text{max}}\) is constant and \(g_{\text{min}}\) is variable as a function of the number of vehicles arriving during red time. The signals return to a state of all-red at no demand in order to make a swift change to green possible in any phase when the next vehicle or pedestrian actuates a detector.

The efficiency of two-phase VA-control is primarily a function of the parameters minimum green \(g_{\text{min}}\), maximum green \(g_{\text{max}}\) and extension interval \((f)\). Low value of \(g_{\text{min}}\) reduces the average delay \((d)\) at load factors (volume-capacity ratio) below 0.4. For load factors above 0.7, there is a tendency at multi-lane intersections to obtain demands for very long periods of green. If restrictions of \(g_{\text{max}}\) are not applied, this results in deteriorated control efficiency and for load factors above 0.85 worse performance than at FT.

1.3.2.1 LHOVRA Technique

The traditional Swedish signal control technique is based on what has been termed the “time-gap method”. Using this method, decisions are made regarding status changes in a particular signal group based on the demand for green. This in turn depends on whether a vehicle passes a predefined detector position(s) with a time-gap that is greater or shorter than that specified in the controller logic.

A special form of VA called LHOVRA has been developed in Sweden with the purpose of increasing safety and reducing lost time and the number of stopped vehicles at signalized intersections along high-speed roads (70 km/h or more) (Vägverket, 1991). During the trial period, the accident rate at the test intersections was reduced from 0.7 accidents per million incoming vehicles to 0.5 (Brüde & Larsson 1988). After its initial success, LHOVRA has been widely used in Sweden and the other Scandinavian countries in urban areas at a lower speed (50 km/h). In LHOVRA, \(g_{\text{min}}\) is constant and independent of the number of vehicles arriving during red time.

The LHOVRA acronym describes the following functions (se Part II for more details):

- L = Truck, bus and platoon priority
- H = Main road priority
- O = Incident reduction
- V = Variable amber time
- R = Variable red time
- A = All red turning
The use of detectors for each of these functions is shown in figure 2.4 below.

<table>
<thead>
<tr>
<th>Functions</th>
<th>Use of detectors for LHOVRA functions on a 70 km/h approach road</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>V</td>
<td>V</td>
</tr>
<tr>
<td>R</td>
<td>R</td>
</tr>
<tr>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>

Figure 2.4 The use of detectors for the different LHOVRA functions on a typical 70 km/h approach road (Vägverket, 1991)

The incident reduction or “O” function is designed to reduce the number of vehicles in the dilemma zone and thereby reduce the number of red light drivers and rear-end collisions. This is achieved by detecting vehicles at the beginning of the option zone and postponing the decision to change to amber (Vägverket, 1991). The LHOVRA technique manual depending on many factors, suggests that the practical dilemma zone is between 130 and 50 meters before the stop line. This function is performed with detectors placed at 130 and 80 meters from the stop line to detect and follow a vehicle inside the dilemma zone.

1.3.2.2 Signal Group Control
One important difference between LHOVRA and other strategies like SCOOT and SPOT, which are based on stage control, is that LHOVRA control is based on signal group control, see Figure 2.5 below. This gives Scandinavian control an advantage, as signal group control is more flexible than stage control.

Figure 2.5 Example of signal group control (TFK, 1982)

With stage control, the signal groups in the intersection are divided into a number of stages. Each signal group must belong to at least one stage. The stages are put in a sequence. There is a minimum green time for each stage.
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With signal group control, no stages are defined, only primary phase pictures. The controller is free to form secondary phase pictures from the primary phase pictures. The minimum green times are only given for the signal group.

1.3.3 Self Optimizing Control

VA-control provides flexibility with regard to random short-term traffic variations, which significantly reduces the average delay as compared to fixed-time control (FT), particularly at low to medium traffic demand. However, since the initiation of a green phase and its extension only depends on the registered traffic demand in the studied approach, disregarding possible queuing in conflicting traffic movements, VA does not provide any form of optimal control.

1.3.3.1 Miller’s algorithm

Miller (1963) suggested a simple self-optimizing strategy based on the total vehicle delay considering the impacts on traffic in all approaches to an intersection. In Miller’s strategy the decision to extend a phase is made at regular intervals by the examination of a control function. This function represents the difference in vehicle-seconds of delay between the gain made by the extra vehicles that can pass the intersection during an extension and the loss to the queuing vehicles in the cross street resulting from that extension.

\[
\Phi = \begin{bmatrix}
\delta_N + \delta_S - q_N\frac{s_N}{1-s_N} - q_S\frac{s_S}{1-s_S}
\end{bmatrix} \left[ r_{NS} + t_{NS} \right] - h \left[ n_w + n_E + \sum_{i=1}^{k_w} q_{W_i} + \sum_{j=1}^{k_e} q_{E_j} \right]
\]

where:
\( \Phi \) = Control function;  
\( q \) = Traffic flow;  
\( r_{\text{SS}} \) = Effective red time;  
\( l_{\text{SS}} \) = Lost green time  
\( \delta \) = Number of additional cars that can pass the intersection if the green is extended by \( h \) seconds;  
\( h \) = Time interval between the calculations of the control function;  
\( s \) = Saturation flow  
\( n \) = Number of queuing cars, buses, etc in approaches with red light that will suffer an increased delay of \( h \) seconds if the prevailing green is extended;  
\( k \) = Number of time intervals (see below)

if \( \Phi \geq 0 \) no change

\[ k = \text{For the respective direction is the minimum integer that fulfill the relation.} \]

\[ n + \sum_{i=1}^{k} q_i - \sum_{i=2}^{k} s_i \leq 0 \]

The first term of equation represents the gain in travel time to the additional cars that can pass the intersection if the green phase is extended by \( h \) seconds. The second term of equation represents the loss due to extra delay to the traffic in the cross approach, which has a red light at the time of the calculation.

1.3.3.2 TOL
Bang (1976) further developed Miller’s theory for practical implementation including the time costs for delay as well as vehicle operating costs for stops, and bus priority functionality (see Figure 2.6). The system was called “Traffic Optimization Logic (TOL)”, and was subjected to extensive testing using both simulation and field trials in cooperation with Ericsson and the City of Stockholm.
In the TOL method, the extension of the green light is based on calculations at regular intervals \((h)\) of a control function \((\Phi)\). This function represents the gain or loss in community cost resulting from extension of the prevailing green light by \(h\) seconds.

The method is exemplified below for the simple case of an intersection between two one-lane approaches, A and B. Assuming that lane A has green light for the moment, the decision to extend the green is based upon the evaluation of the control function \(\Phi_A\).

\[
\Phi_A = r_A \left( a_v \delta_{Av} + a_b \delta_{Ab} + a_p \delta_{Ap} \right) + b_v \delta_{Av} + b_b \delta_{Ab} - h \left( a_v n_{A_v} + a_b n_{A_b} + a_p n_{A_p} \right) - \left( b_v \Delta n_{A_v} + b_b \Delta n_{A_b} \right)
\]
where:

\[ r_A = \text{Time interval (red and inter-green) until phase A will get green light again if it is terminated immediately; } \]
\[ a = \text{Cost of delay per second} \]
\[ \delta = \text{Number of additional cars, buses, pedestrians etc that can pass the intersection if the green is extended by } h \text{ seconds; } \]
\[ b = \text{Vehicle operating cost to bring a vehicle to a complete stop and to resume normal speed} \]
\[ h = \text{Time interval between the calculations of the control function; } \]
\[ n = \text{Number of queuing cars, buses, etc in approaches with red light that will suffer an increased delay of } h \text{ seconds if the prevailing green is extended; } \]
\[ \Delta n = \text{Number of extra vehicles, buses, etc., that will be forced to stop if the prevailing green is extended by } h \text{ seconds; and } \]
\[ v, b, p = \text{Index of vehicle (average of all types), bus and pedestrian.} \]

The first term of equation \[ r_A \left( a_v \delta_{Av} + a_h \delta_{Ah} + a_p + \delta_{Av} \right) \] represents the gain in travel time to the additional cars, buses etc that can pass the intersection if the green phase is extended with \( h \) seconds. This traffic gains \( r_A \) seconds which is the time interval until phase A will get a green light again if it is terminated immediately. The next two terms \( \left( b_v \delta_{Av} + b_h \delta_{Ah} \right) \) represent the gain due to reduced number of stops. The negative terms of equation \[ -h \left( a_v n_{Bv} + a_h n_{Bh} + a_p n_{Bp} \right) - \left( b_v \Delta n_{Bv} + b_h b_v \Delta n_{Bp} \right) \] correspondingly represent the loss due to extra delay and number of stops to the traffic in approach B, which has a red light at the time of the calculation.

Phase A is extended until \( \Phi_A < 0 \) subject to restrictions of maximum green time. During phase B, the control function \( \Phi_B \) is evaluated in a similar manner.

The results of the studies of the TOL strategy compared to conventional Fixed Time (FT) and Vehicle Actuated (VA) Control showed significant reductions in average delay and proportion of stopped vehicles, see Figure 2.7.
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The Miller and the TOL methods do not guarantee that an overall optimal control is obtained, since they are based on a large number of short-term optimizations. It was, however, a step forward compared with existing VA-control, which does not involve any direct optimization (Bang, 1976).

1.3.3.3 MOVA

The British Transport Research Laboratory (TRL) continued to develop self-optimizing strategies between 1982 and 1988, leading to the MOVA strategy for isolated intersections (Vincent, 1988). MOVA uses maximum green times for each stage, but they are normally set so high that they are not reached. There is also a maximum value for the cycle length, which is needed to avoid excessive pedestrian waiting times. MOVA decides the green split within this maximum cycle. In the optimization, MOVA uses a microscopic traffic model. The position of each vehicle is predicted between the IN detector and the stop line.

Every half-second, MOVA calculates whether the total delay will be minimized if the current stage continues to be green during 0.5s, 1.0s, 1.5s ... up to a programmed level or if it goes to red, as explained in Figure 2.8. MOVA compares area 1A in the figure below with area 1B and area 2. If area 1A is the largest, the current stage continues; if not it goes to red. Stopped vehicles are also taken into account in the calculation (Kronborg, 1992).
During over-saturated conditions, i.e. one or more approaches are left with a significant queue at the end of green, MOVA recognizes this automatically and changes strategy. Instead of the Miller algorithm, a heuristic capacity-maximizing algorithm is used.

Both MOVA and TOL are stage-based, which reduces flexibility compared to the signal group control normally applied in Swedish isolated signal controllers (Kronborg 1992). MOVA can use alternative stages, which does not, however, provide the full flexibility of signal group control for secondary traffic movements (e.g. separately controlled right turn movements). The demand is expressed as demand for a stage, not for a signal group. This also creates problems. As there are normally no detectors close to the stop line the cruising speed between the X detector and the stop line must be estimated. This has to be done with a margin for slower vehicles, introducing a late change from green to yellow.

1.3.3.4 SOS

SOS - Self Optimizing Signal control - is a control strategy for isolated intersections (Kronborg et al, 1997) which combines the Scandinavian tradition of signal group control with mathematical optimization of the same type as originally developed by Miller (1963).
The main function of the SOS strategy is to decide when to end each phase picture. During normal non-over-saturated situations SOS carries out its principal task from the moment when the queue gathered at red is discharged until all traffic in the approach is discharged. During this period, SOS seeks the optimal moment to change from green to amber. The controller takes care of the rest including green demand, phase picture sequence etc.

This means that SOS works in parallel with the controller software. SOS only intervenes by giving stop orders on the signal group level to the controller. SOS is not used for optimization during extremely low traffic conditions, i.e. at night when there are no more than a few vehicles in the intersection at the same time.

SOS consists of the following modules:
- Detectors;
- Intersection and traffic model;
- Optimization algorithm;
- Over-saturation algorithm;
- Advanced incident reduction;
- Special features;
- Traffic data base;
- Operator interface.

The most important parts of SOS are the traffic model, the optimization, and the over-saturation algorithm. Before the optimization, SOS calculates the most likely phase picture sequence for the forthcoming cycle. This information is used in a Miller-type optimization to minimize a mathematical function of delay costs, stop costs and other cost elements. If the intersection is over-saturated a special algorithm is used. Special attention is paid to rear-end collision risks and red driving risks using an advanced heuristic incident reduction. This function tries to minimize the number of vehicles in the option zone. Results from the trial of SOS are presented in Table 2.1 below.

### Table 2.1 Comparison between LHOVRA and SOS control (Kronborg et al, 1997)

<table>
<thead>
<tr>
<th>Results compared with a fine-tuned LHOVRA control</th>
<th>Traffic 14 - 15</th>
<th>Traffic 7.15 - 8.15</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SOS-default</td>
<td>SOS-safety</td>
</tr>
<tr>
<td>Vehicle delay</td>
<td>-3.5%</td>
<td>0%</td>
</tr>
<tr>
<td>Number of stopped vehicles</td>
<td>-8.6%</td>
<td>-13.2%</td>
</tr>
<tr>
<td>Number of vehicles in the option zone</td>
<td>-32.7</td>
<td>-55.3%</td>
</tr>
<tr>
<td>Total socio-economic cost</td>
<td>-6.6%</td>
<td>-6.5%</td>
</tr>
</tbody>
</table>

### 1.3.4 Other Methods for Isolated Traffic Signal Control

In addition to traditional real-time optimization, new methods like fuzzy control and neural networks are entering the field of adaptive traffic signal control. So far, the most published applications of these two new methods are mainly theoretical, but active research is being conducted in this area.
1.3.4.1 Fuzzy Logic

The fuzzy signal control has been developed in the context of fuzzy inference. The fuzzy statement protocol is a fruitful technique for modeling the knowledge and experience of a human operator. Thus, traffic signal control is a suitable task for fuzzy control (Niittymäki, 2002).

Modeling of human reasoning: An inference process is described by rules. The inference of ordinary rules and fuzzy rules is quite different. Reasoning described by normal rules forms a hierarchical exclusive structure where only one branch in the decision tree is followed. With fuzzy rules the structure is flat and all the rules are processed equally. Contradictory rules are not excluded, but combined with other rules. This approach makes it possible to freely combine contradictory and mutually incompatible arguments.

The fuzzy inference is more a calculation than a reasoning process. The outcome of this calculation is a distribution of truth-values for each possible output. There is no unambiguous way of reducing this distribution output to a single value and therefore several "defuzzification" methods exist.

The general-purpose fuzzy inference object can be used by other objects of the model as a passive component, i.e. it is updated only when used by some other object. A fuzzy inference object can also be an active component, which is then updated per every simulation cycle. In this case, the fuzzy object has to be associated with task specific additional functionality to perform the interactions with the rest of the model.

Signal group oriented fuzzy control: Here a new approach is proposed, which is to combine the existing multi-agent control scheme with a more intelligent negotiation and reasoning mechanism, i.e. with fuzzy inference. The target of combining fuzzy inference with signal group technique is to keep the flexibility of signal group oriented control while improving fine-tuning of the signal.

The proposed principle is not limited to fuzzy control only as any type of control algorithms could be applied. The improvement of the control scheme is not achieved because of the algorithm only, but due to the better access to traffic measurements through the simulation model. The traffic simulation model is an essential part of the proposed control scheme, as will be explained later.

The open fuzzy extender objects provide a generic inference engine for the signal group agents and the inference process can be completely defined by the user. The signal group agents supply the relevant traffic inputs to the inference object and obtain the extension length. The inference objects provide a framework where contradictory objectives and incompatible inputs can be weighted against each other.

Application results: past efforts in the comparison with traditional fixed time control were conducted in simulation environments. The results are based on single cases, see Table 2.2.
**Table 2.2 Summary of earlier experience of isolated fuzzy traffic signal control (Niittymäki, 2002)**

<table>
<thead>
<tr>
<th>Study</th>
<th>Application</th>
<th>Study method</th>
<th>Results of fuzzy control performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pappis and Mamdani 1997</td>
<td>Fuzzy two stage signal control of one-way intersection</td>
<td>Simulation: comparison to ideal adaptive control with optimal cycle</td>
<td>Equal or slightly better</td>
</tr>
<tr>
<td>Kim 1994</td>
<td>Isolated fuzzy signal control</td>
<td>Simulation: comparison to fixed signal timing</td>
<td>5.33% better efficiency</td>
</tr>
<tr>
<td>Sayers et al. 1995-98</td>
<td>Isolated fuzzy signal control</td>
<td>Simulation: comparison to VA-control</td>
<td>Better flexibility, promising results</td>
</tr>
<tr>
<td>Trabia et al. 1999</td>
<td>Two stage fuzzy control</td>
<td>Simulation: comparison to VA-control</td>
<td>9.5% improvement in average delays, 1.3% improvement in stopping percentage</td>
</tr>
</tbody>
</table>

Niittymäki (2002) has tested fuzzy control methods in several intersections. The proposed controller consists of traffic and control models, and it is justified that this kind of on-line simulation or simulation based traffic control is a working method. According to statistical tests in before and after studies, fuzzy control has proven to be a possible control method in real isolated traffic signal control (Niittymäki, 2002).

Kosonen and Bang (2001) introduced a fuzzy control system based on a multi-agent idea. The system, called HUTSIG, is closely related to the microscopic traffic simulator HUTSIM. The latter is used both for off-line evaluation of the signal control scheme and for on-line modeling of the traffic situation during actual control. Indicators are derived from the simulation model as input to the control scheme. In the presented control technique each signal operates individually as an agent, negotiating with other signals about the control strategy. The agents make decisions based on fuzzy inference that allows various aspects such as fluency, economy, environment and safety to be combined.

### 1.3.4.2 Chaotic Neural Networks

It is known that Chaos phenomenon exists in various dynamical systems. Urban traffic systems have a typical chaotic characteristic. Chaos theory should be a kind of effective method to deal with the problem. In recent years, a great deal of research has been carried out into chaotic neural networks (CNN). A Multistage Self-Organizing Algorithm Combined Transiently CNN for Cellular Channel Assignment has been developed and a CNN with reinforced self-feedbacks was proposed.

Dong et al (2005) developed a multi-layer chaotic neural network involving feedback (ML-CNN) based on Hopfield networks and chaos theory. It was used to optimize urban traffic signal timing. The ML-CNN has a characteristic of escaping from a local minimum of the energy function, so that it can find a global minimum more easily as compared with Hopfield’s model.
Figure 2.9 shows an example of an ML-CNN’s framework that can be used in an intersection with standard four-signal phases. Compared with Hopfield networks, several major characteristics of the networks consist of:

- it is a three-layer network including an input layer, an output layer and a hidden layer;
- all the outputs in the output layer are returned to the input layer;
- the hidden layer consists of many chaos neurons with self-feedback.

Simulation research was carried out at an intersection in China. The results indicate that urban traffic signal timing using ML-CNN could reduce the average delay per vehicle at intersection by 25.1% compared to conventional timing methods.

1.4 STRATEGIES FOR COORDINATED TRAFFIC SIGNAL CONTROL

In isolated signal control, it is normally assumed that the arrival of vehicles in the approaches to the intersection is random with a negative exponential time headway distribution.

In an urban street network, the arrival rates are often influenced by the queue discharge from upstream traffic signals, which create vehicle platoons moving along the approach links. The shorter the distance between the signalized intersections, the less dispersed are these platoons when they arrive at the downstream traffic signal’s stop line, see Figure 2.10.

If adjacent signalized intersections are coordinated in such a way that they operate with the same cycle time and with constant split and offset, it is possible to set these signal timing parameters on a one-way street in such a way that the platoon from the upstream intersection will arrive at the downstream stop-line when this signal is green (called “green wave”), see Figure 2.2 (page 7).
Figure 2.10 Greater benefit of coordination when intersections are close (TRL, 2002)

For two-way streets, green waves can be accomplished manually by an experienced traffic engineer subject to signal spacing and cruising speed requirements using time-space diagrams for different time plans, each suited to a typical traffic situation during the day (e.g. morning peak, mid-day, evening peak and low traffic).

For two-way streets, the relationship between speed ($v$), cycle time ($c$) and distance ($D$) is

$$D = v \cdot \frac{c}{2}$$

or with different speeds in both directions of the street ($v_1$ and $v_2$)

$$\frac{1}{v_1} + \frac{1}{v_2} = \frac{2}{v}$$

$$D = \frac{v_1 \cdot v_2 \cdot c}{v_1 + v_2}$$

When cycle time has been determined for every time plan, the green times for the phases are calculated for every intersection by using the formula

$$g = \lambda (c - L)$$

where:

$g$ = Effective green time
$c$ = Cycle time
$\lambda$ = Portion of green time
During the night, the coordination is usually broken down into smaller groups of linked intersections or to isolated control. Based on the knowledge of the traffic patterns, sometimes an estimated O/D-matrix, and some fresh traffic data, 3-5 fixed reference plans are produced by hand or with some rudimentary computer-aided systems. However, the difficulties in designing efficient coordination increase rapidly when the number of traffic signals grows and when the coordination involves networks. Also, the problem of creating "green waves" when there is only limited spare capacity in the system is hard to resolve manually.

The traffic flows used when producing the time plans for coordinated control will change over time making these time plans less well suited for the traffic situation for which they were originally designed. These errors will cause a gradual deterioration of efficiency in the system. Road authorities often fail to maintain and update the signal settings, and it is not unusual that the same coordination is still in operation 10-15 years after its design. The consequences for road users in terms of delays and stops are significant. Other negative effects are that fuel consumption and emissions increase, as well as drivers’ irritation.

1.4.1 Off-Line Program TRANSYT

Several computer aids are available supporting the development of time plans for signal coordination of arterial streets or street networks. The TRANSYT program (Vincent et al, 1980) (Traffic Network Study Tool) has become one of the most widely used programs of its type in the world. TRANSYT is an off-line program for calculating optimum coordinated signal timings in a network of traffic signals. After the first program was developed in 1967, a number of versions have been produced, all of which have two main elements: a traffic model and a signal optimizer, see Figure 2.11.

![TRANSYT structure](image)

**Figure 2.11** TRANSYT structure including traffic model and signal optimizer (TRL, 2002)
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The traffic model represents traffic behavior in a highway network in which most intersections are controlled by traffic signals. The model predicts the value of a "Performance Index" for the network for a given fixed time plan and an average set of flows on each link. The Performance Index measures the overall cost of traffic congestion and is usually a combination of the total delay and the number of stops made by vehicles. The optimization process adjusts the signal timings and, using the traffic model, calculates whether the adjusted timings reduce the Performance Index. By successive adoption of beneficial timings an optimum is reached.

\[ PI = \sum_{i=1}^{N} (W \cdot \text{delay}) + \sum_{i=1}^{N} \left( \frac{K}{100} \right) \cdot \text{stops} \]

where:
- \( N \) = Number of links
- \( W \) = Average cost of delay (per PCU hour)
- \( K \) = Cost per stop (per 100 PCU stops)

TRANSYT assumes that all major intersections in the network are signal or priority controlled, that all the signals have a common cycle time, and that all signal stages and their minimum periods are known. For each distinct traffic stream, it is assumed that, for traffic flowing between intersections, or turning at intersections, the flow rate, averaged over a specified period, is known and assumed to be constant.

1.4.2 AUT/TRANSYT

Fully adaptive on-line systems have been developed in recent years, e.g. PRODYN, SCOOT and SCATS. The experimental results from these and several other systems encouraged Swedish road authorities and research institutes to engage in the development of AUT-Automatic Updating of TRANSYT reference plans. AUT is not a fully adaptive system working in real time; instead it works with a specific number of reference plans that are updated every 24 hours. The AUT control strategy involves time settings being automatically adjusted to variations in the traffic. Using TRANSYT together with two modules, TRAF (deals with traffic data) and AVT (User Friendly TRANSYT), the system is able to work automatically (Hammarström, 1999). The basic idea is that the road authority decides the type of effect - minimum exhaust emissions, minimum fuel consumption, minimum travel costs etc for which the signal settings are to be optimized. The result of the test area with AUT showed a reduction in fuel consumption of 15% (Hammarström, 2000).

1.4.3 Centralized Optimization System SCOOT

The Split Cycle Offset Optimization Technique (SCOOT) (Hunt et al, 1981) was developed by TRRL and other organizations in the UK. This is a fully adaptive system that uses the data from the detectors placed in the intersection to optimize the traffic signal settings. This system was created in order to provide fast responses to change in traffic conditions. There is a special emphasis on the fact that these responses are carried out in a stable manner. The system searches for the minimization of a function that has terms for the stops, delay and congestion for all signalized intersections in the coordinated network. The weight assigned to every component can be modified in order to bias the optimization process towards some defined links.

SCOOT has three key principles (Wood, 1993):
- Cycle flow profile (CFP): This aspect is similar to the one used in TRANSYT, but in this case they are updated every few seconds. In the case of TRANSYT, the accuracy depends on the data about the average flows, saturation flows, cruise time and some other variables. In the case of SCOOT this process is automatic and carried out in real-time.

- Update of online model and queue estimation: The queue estimation is carried out in the same way as in TRANSYT (see Figure 2.12). The calculation of the queue is carried out every few seconds while the data collected from the sensors is collected every second. The detectors are placed upstream and downstream of the intersection.

- Incremental optimization: SCOOT uses an elastic coordination that stretches or shrinks the coordination plans to match them to the detected CFP. This process is carried out to evaluate the effects of modifying up or down the length of the cycle in a series of frequent but small increments. For some intersections the decisions are not suitable but that is compensated with the large majority.

![Figure 2.12 Principles of the SCOOT traffic model (Hunt et al, 1982)](image_url)
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SCOOT Version 3.1 contains the active bus priority facilities developed in the EC project PROMPT, with capabilities for bus detection either through transponder/detector communications, as used in London or via Automatic Vehicle Location (AVL) systems. A key element of system optimization is bus journey time prediction from the point of detection to clearance of the stop line, particularly where a green time extension is implemented or causing succeeding stages to occur early (a recall). Extensions can be awarded centrally, or the signal controller can be programmed to implement extensions locally on a street (a local extension). Overestimation can cause unnecessary signaling disruption while underestimation can cause many buses to miss priority. With AVL, accurate prediction is further complicated because the bus could be polled at any location on a link (Hounsell et al, 1996).

SCOOT Version 4.5 enhanced bus priority logic - differential priority - by providing priority on the basis of criteria other than vehicle class alone, such as adherence to schedule. If a ‘degree of lateness factor’ can be determined and provided to SCOOT, SCOOT is then able to provide different levels of priority according to how late a bus is.

Austin & Martin (1996) describe the effect of removing pelican crossings from SCOOT control and applying different strategies, such as operating with and without vehicle actuation and varying the maximum period devoted to vehicles. The new version SCOOT MC3 includes the Pedestrian User Friendly Intelligent (Puffin) facilities (Modeling of Variable inter-green). Puffin crossings are relatively new and are intended to become the UK standard for signal controlled pedestrian facilities at stand-alone crossings and junctions. Unlike pelicans, there is no flashing amber period and, instead; the length of the red to vehicles is variable depending on the time that pedestrians take to cross the road (SCOOT, 2006a).

While SCOOT has been proven to be effective in achieving significant savings in delay, its delay-minimization objective may not be the most effective technique for networks with a high level of congestion (Wood et al, 1994). A number of new tools have been developed for use in heavily congested situations. These include a 'gating' facility, which can be used to automatically meter the flow of traffic into a congested area. It has been tested and evaluated in Southampton where significant benefits have been obtained. Another tool is the MONACO program, which can be used in conjunction with SCOOT to monitor, analyze and diagnose congestion in a UTC network and to trigger the introduction of congestion strategies automatically (Bretherton, 1995).

Early results showed that SCOOT achieved an average saving in delay of about 12% when compared with up-to-date TRANSYT fixed-time plans. Research by Bell and Bretherton (1986) suggests that SCOOT is likely to achieve an extra 3% reduction in delay for every year that a fixed-time plan "ages". Since SCOOT is designed to adapt automatically to compensate for ageing and incident effects, it is reasonable to expect that, in many practical situations, SCOOT will achieve savings in delay of 20% or more (SCOOT 2006b).

1.4.4 Hybrid Centralized Optimization System SCATS

The Department of Main Roads of New South Wales, Australia, has developed the Sydney Coordinated Adaptive Traffic System (SCATS) (Akcelik, 1996). Development began in 1970 and from 1975 considers local microprocessors. In 1980, SCATS was already installed in over 30 cities around the world.
SCATS combines theoretical models and library plans. Cycle time and splits are calculated for each critical intersection based on real traffic demand. The timing plans for sub-areas made up of a critical intersection and its adjacent surrounding signalized intersections are then selected to be compatible with the signal timing required for their critical intersection. The optimization criterion for SCATS minimizes vehicle stops and delays, particularly stops in the radial roads. However, in some cases, the function can be specially designed towards some specific objectives like maximizing the throughput of the street network.

SCATS can thus be described as a hybrid system that contains plan selection/generation system combined with local adaptation (Wood, 1993). The architecture of the network consists of groups of signals. Every such “subsystem” contains a critical intersection and its neighbors. The subsystems are grouped together and controlled by a regional computer. The whole system can be expanded by the addition of extra regional computers. Usually, a central computer monitors the activity of the regional computers. Every regional computer can modify the subsystems dynamically, but an intersection can only be allocated to a single subsystem.

SCATS uses two levels of control: “strategic” and “tactical” (Lowrie, 1982).

- The strategic level calculates signal timings based on typical traffic conditions. It calculates the common cycle for a subsystem using the flow and occupancy data from the detectors. It can consider up to 10 intersections and gives priority to the critical intersection defined apriority. The strategic level also uses some specific detectors located at critical points in the network;

- The tactical control optimizes individual intersections within the restrictions imposed by the strategic control. Some automatic stage flexibility is available at the local controllers where stages for which there is currently no demand can be skipped. Some stages (like main roads) are unable to be skipped by default. The tactical level uses the same type of detectors as in strategic control but they are placed in different locations.

The strategic detector information is used by the algorithms to calculate, on a cycle by cycle basis, the phase split plan, internal offset plan, external offset plan and cycle length to apply to the subsystem for the next cycle together with an incremental modification to the splits and offsets.

Reactive/Case-based control logic usually calculates four or eight split plans for each intersection. It nominates a phase serving the highest traffic demand as a “stretch phase.” The SCATS algorithm computes phase splits based on required green time for each phase during the peak period. It also computes a stretch cycle length based on green times of the selected stretch phase (Harsh et al, 2006).

The event response is also considered to be gradual. To avoid oscillating offsets, it is calculated each cycle, but only implemented when at least three out of the previous five cycles have suggested a change to that offset. Also, when a detector is activated for a certain duration, an alarm raises in the operator room. Then the operator can use video cameras to check whether it is available.

The performance result relies on the skills of the traffic engineers who set up the system. A study in Paramatta showed that there was no significant reduction in journey time. There was,
however, a reduction in the number of stops in the central area (9%) and on the arterial roads (25%) (Wood, 1993).

1.4.5 Decentralized Optimization System SPOT

The idea of SPOT is decentralized, socio-economic optimization of the signal settings in real time for each individual intersection based on predictions of arrival flow profiles from upstream intersection arms (originally proposed by Bang, 1975). The main costs are caused by vehicle delays and vehicle stops. In order to give priority to buses and trams, higher unit costs are used for these vehicles.

This technique was developed in Italy and has been fully operational since 1985 in a network of 40 signalized intersections (Mizar, 2001). SPOT/UTOPIA uses a hierarchical-decentralized control strategy based on local controllers connected to other local controllers and a central controller. The central principles of Utopia are the provision of priority to selected public vehicles at signalized intersections and improve the mobility of private vehicles.

SPOT is now used in several cities in Italy and also in the Netherlands, USA, Norway, Finland and Denmark (Arveland, 2005). Early results from the Turin implementation showed an increase of 15.9% in the average travel speed of private vehicles, while public transport speeds rose by an average 19.9%. These improvements were based on a comparison with the performance of the fixed-time traffic control system that previously operated in the city. More recent results from field trials staged in Turin, Gothenburg, and Leeds confirm the effectiveness of the UTOPIA strategies (Mizar, 2001).

1.4.6 Comparative Discussion Regarding SCOOT, SCATS & SPOT

The signal control strategies SCOOT, SCATS & SPOT are discussed below, focusing on the following aspects:

- Control concept;
- Public transport;
- Congestion;
- Pedestrians;
- Detection and data collection;
- Traffic safety and incidents.

1.4.6.1 Control Concept

SCOOT uses pre-determined time plans. The intersections in the area are grouped into “sub-areas” with intersections operated on a common cycle length. In response to short-term changes in traffic demand, a cycle optimizer varies the cycle length of each sub-area by a small amount, usually a few seconds. The most heavily loaded intersection determines the change in cycle length for each subsystem to obtain an optimum solution for the entire controlled area.

SCATS is a hybrid system that contains plan selection and online generation of improved time plans combined with local traffic adaptation. The architecture of the network consists in groups of signals forming subsystems including a critical intersection. The subsystems are grouped together and controlled by a regional computer. SCATS aims at optimal control of each subsystem, but with less flexibility regarding signal-timing parameters than SCOOT.
SPOT permits high flexibility in signal settings from a base reference plan. It falls back to a fixed plan if the local control fails. UTOPIA can work at a local level if the central computer fails, coordinated with the nearby intersections. SPOT uses distributed and local control. It works at local level determining optimal signal settings in real time for each individual intersection. The central level supplies overall constraints or other area-wide strategies.

The studied adaptive strategies have different concepts for controlling signal timings. The selected adaptive control strategies use different approaches to adaptive signal control. SCOOT is a responsive control strategy. It responds to varying traffic by changing cycle length and phase splits in small increments. SCATS, a semi responsive (case-based) strategy, adapts to varying traffic by selecting a timing plan from an offline-stored library of plans that best suits current traffic demand. SPOT, on the other hand, is a predictive control strategy. It detects current traffic demand and predicts future arrivals at an intersection to determine signal timing for each individual intersection.

The decentralized optimization process in SPOT enables faster changes of signal timing than what is possible in SCOOT with its wide sub-area signal timing optimization, or SCATS with its subsystem optimization focused on signals surrounding a critical intersection. However, because of its decentralized mode of optimization, SPOT does not guarantee that optimal traffic performance is obtained on a sub-area or network basis.

1.4.6.2 Consideration of Public Transport Vehicles
SCOOT Version 3.1 contains active bus priority facilities (developed in the EC project PROMPT), with capabilities for bus detection, either through transponder/detector communication as used in London, or via Automatic Vehicle Location (AVL) systems.
SCOOT Version 4.5 enhanced bus priority logic by providing priority on the basis of criteria other than vehicle class alone, such as adherence to schedule.

SCATS: Public transport priorities are available in a passive way. Bus lanes and routes can be defined. Some implementations consider a change to a particular stage when a vehicle is detected.

SPOT: The central principles of Utopia are the provision of priority to selected public vehicles at signalized intersections. UTOPIA can work linked with transport public location system in order to predict arrival time to intersections. It provides information to users of the public transport system.

1.4.6.3 Congestion
SCOOT considers congestion detected as the portion of time during the signal cycle that the detector is fully covered (= occupancy). Congestion is treated as a variable that starts from zero and increases. This ensures smooth changes in the signal control.

SCATS considers congestion by special detectors for queues. If queues are detected, SCATS may 1) extend the cycle time, 2) activate background “gating” plans, and 3) in the case of main roads increase capacity on individual intersections restricting greens on side roads.

SPOT considers congestion by queue management on a local level through an excess queuing parameter. A high weight can be specified at a central or local level in order to avoid blocking the upstream intersection.
1.4.6.4 Consideration to Pedestrians
The new version SCOOT MC3 includes the Pedestrian User Friendly Intelligent (Puffin) facilities (Modeling of Variable inter-green).

In SCATS some modules are applicable in the case of pedestrians like an extra stage with a red arrow and pressure pads to detect their presence.

SPOT does not consider special or unique features for pedestrians.

1.4.6.5 Detection and Data Collection
SCOOT uses detectors upstream and downstream. Includes a detector failure routine and a back-up plan consisting of fixed time plans. SCOOT includes a special module (ASTRID) to generate a database of the data collected. Provides information in the form of flow, delays, vehicle stops and congestion.

SCATS detectors are placed immediately before the stop line. The number of detectors is not fixed. In the case of Sidney SCATS controls 1,700 intersections with 17,000 detectors. Failures in local detectors will cause the junctions to only operate at the command of the regional computer. If a strategic detector fails, then the information is collected from the upstream detector. If the connection with the regional computer is lost, then the local controller will continue with the last plan. With SCATS, data can be collected from the detectors for the maintenance of the program.

SPOT uses detectors upstream and downstream too. Falls back to a fixed plan if the local control fails. UTOPIA can work at a local level if the central computer fails, coordinated with the nearby junctions. SPOT collects amount of data enough to estimate OD matrices. A high weight can be specified at a central or local level in order to achieve traffic management.

1.4.6.6 Consideration to Traffic Safety and Incidents
SCOOT has no special safety considerations except for sufficient inter-green, minimum inter-green and conflicting matrix. SCOOT considers predefined routes in order to give green waves to emergency vehicles (an operator of the system must activate this sequence). Includes an (under development) incident identification module (INGRID).

In SCATS, safety issues are not specially considered. In some specific applications like in Sidney, turning stages are often included for safety reasons. Longer inter-greens are used than is common in SCOOT.

SPOT includes no special safety features. It considers incidents at a local control level and can switch to fixed plans if considered necessary. It provides a wave of green and supplies predicted journey time adding the routing of the emergency vehicles.

1.5 CONCLUSIONS
Some reflections regarding the area of strategies are:

Terms are understood in different ways. In his literature survey, Hagring (2000) shows that saturation flow is calculated in different ways and differences are significant. Delay is not calculated with the same formula in HCM, SIDRA, or CAPCAL (see next chapter).
Every new strategy emerges where a new system is needed. The new strategy is developed and compared with other already existing and probably worse system. This means that comparison has rarely been made with a common reference point (benchmark).

Some studies become a marketing tool for a product. In Australia, SCATS is the best system (Luk, 1989). In Turin SPOT/UTOPIA is the best system (Mizar, 2001). TRL’s reports present SCOOT as the best system, even though Wood (1996) recommended studying the effects of decentralizing on this technique.

Replacing an entire system is not easy. This is due to:

- Current engineers and experts are masters of the old system;
- The new system demands new knowledge that has to be obtained and developed;
- The new system needs installation and new equipment that usually has a high cost;
- It is not easy to switch systems, for example from SCOOT to UTOPIA, while is easier to develop SPOT/UTOPIA. But it can sometimes be more profitable to improve an existing strategy than to change the whole system.

In Sweden the initial situation is also specific:

LHOVRA is used today for isolated intersections. The LHOVRA technique belongs to the generation of the VA control that lacks the ability to collect data and use it for evaluation and self-adjustments. Accordingly, there is not enough knowledge regarding the impacts of LHOVRA. A major Swedish study of self-optimizing isolated signal control was undertaken in 1976 with the aim of developing strategies inspired by a method for real-time optimization. The system, called TOL, showed good results in simulation and full-scale field trials compared to traditional Swedish vehicle actuated control. A modified Self Optimizing Signal control strategy (SOS) was developed in 1997, but it is not used in the market. SOS combines real time optimization with the Scandinavian tradition of signal groups. Some efforts to develop LHOVRA have been made, but not enough to produce a new concept. SPOT is also being tested and was recently used in two Swedish cities.

Time-planned traffic control with bus priority is used in coordinated intersections. Some municipalities use the offline TRANSYT software for this purpose. In Stockholm, the timing of the signals is normally performed manually taking into consideration local signal timing adjustment based on detector inputs controlling the termination of the green signal by using PEG. The effects of PEG with regard to capacity and delay on congested urban networks have received little attention, particularly in low speed urban networks. The Swedish method of coordinated design needs to be modernized. Self-optimized signal control is an alternative that is also worth studying in-depth.

Urban signal control in major Swedish cities normally also incorporates active bus priority – PRIBUSS. A decentralized system for self-optimized signal control of urban networks, SPOT, is also being introduced on the Swedish market.

The need for better knowledge about traffic performance and safety impacts of these techniques is the reason behind this study.
2 METHODS FOR EVALUATION OF TRAFFIC SIGNAL CONTROL STRATEGIES

This chapter is an overview of the methods to measure or estimate the performance impacts of the signalized intersections. Definitions of traffic performance impacts can be found for example in the Highway Capacity Manual HCM 2000 (TRB, 2000) and the Indonesian Highway Capacity Manual IHCM (Bang, 1997). General terminology used in this thesis is listed in the glossary.

Travel time is broadly defined as “the time necessary to traverse a route between any two points of interest.” (FHWA, 1998). Travel time can be directly measured by traversing the route(s) that connects any two or more points of interest. Travel time is composed of running time, or time in which the mode of transport is in motion, and stopped delay time, or time in which the mode of transport is stopped (or moving sufficiently slow as to be stopped, i.e., typically less than 8 km/h). Figure 3.1 illustrates the concepts of running time and stopped delay time.

![Figure 3.1 Running Time and Stopped Delay Time (FHWA, 1998)]

There are three main methods of estimating of traffic performance measures:
- Empirical methods (field measurements);
- Analytical methods;
- Simulation models.
2.1 EMPIRICAL METHODS

Traffic performance is highly influenced by driver behavior. Quantitative tools to understand human behavior and values have only partially been developed in the past few decades. A great deal of both basic and applied research has been conducted in this area and it is only lately that we have come to realize the benefits in practical applications (Goulias, 2000).

The requirements for higher data quality led to new modeling approaches of increasing complexity in transportation research. Some of these approaches require new kinds of data. Moreover, the increasing complexity of these models often also implies that more detailed data are required, leading to increased demands on respondents (Arentze, 2000).

Anyhow to collect data, it is necessary to decide these demands. Some of these demands determine the methods to measure or to collect data, for example the travel time, and from the other side determine the suitable time.

2.1.1 Time Elements

There are several time elements that must be considered in establishing the scope for travel time data collection activities (FHWA, 1998):

- Months of the Year: Travel time data are commonly used to represent “typical” or “average” annual conditions, and should be collected during months that have typical or average traffic volume patterns. As with defining other time elements in the scope, traffic volume patterns from automatic traffic recorder (ATR) stations can be used to determine typical or average months for data collection;

- Days of the Week: Traditionally, data collection efforts for many transportation agencies have been focused on the middle weekdays (i.e., Tuesday, Wednesday, and Thursday). Monday and Friday are typically excluded from data collection (depending on study budgets), and these days’ high variation from conditions during the middle of the week would necessitate a much larger sample of weekdays;

- Time Periods: The time periods define the ranges in the time of day that travel time data will be collected. Like other elements of the study scope, the time periods will likely be determined by the study objectives. For travel time studies that are focused on identifying congestion trends and problems, three time periods are commonly considered: Morning Peak Period, Off-Peak Period, & Evening Peak Period.

2.1.2 Travel Time Measurement Techniques

Several data collection techniques can be used to measure or collect travel times. These techniques are designed to collect travel times and average speeds on designated roadway segments or links. Because these techniques differ from point-based speed measurement, the resulting travel time and speed data are much different than spot speeds. A general overview of the various techniques is provided in the following paragraphs (FHWA, 1998), which are:

2.1.2.1 Test Vehicle Techniques

Test Vehicle Techniques (often referred to as “floating car”) are the most common travel time collection methods and consist of a vehicle(s) that is specifically dispatched to drive with the traffic stream for the express purpose of data collection. Data collection personnel inside the
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test vehicle control the speed of the vehicle according to set driving guidelines (“average car”, “floating car”, or “maximum car”). A passenger in the test vehicle can manually record travel times at designated checkpoints using a clipboard and stopwatch, or computer instrumentation may be used to record vehicle speed, travel times or distances at preset checkpoints or intervals. An electronic distance-measuring instrument (DMI) attached to the vehicle’s transmission can be coupled with a portable computer to record speeds and distances traveled up to every half-second or less. A GPS receiver coupled with a portable computer can be used to record the test vehicle’s position and speed at time intervals as short as every second.

2.1.2.2 License Plate Matching Techniques
License plate matching techniques consist of collecting vehicle license plate characters and arrival times at various checkpoints, matching the license plates between consecutive checkpoints, and computing travel times from the difference between arrival times. License plate matching for travel times can be performed in a number of different ways. The manual method involves recording license plate characters using voice tape recorders. A portable computer-based method relies on human observers to transcribe the license plate numbers into the computer in the field. Video cameras or camcorders can be used to collect license plates with automatic transcription into a computer or manual entry at a later date.

2.1.2.3 ITS Probe Vehicle Techniques
ITS probe vehicle techniques utilize passive instrumented vehicles in the traffic stream and remote sensing devices to collect travel times. The ITS probe vehicles can be personal, public transit, or commercial vehicles and often are not driven for the express purpose of collecting travel times. ITS probe vehicles also typically report travel time data to a transportation management center (TMC) in real-time.

2.2 ANALYTICAL METHODS

The theory of traffic signals focuses on the estimation of delays and queue lengths that result from the adoption of signal control strategy at individual intersections, as well as on a sequence of intersections (Rouphail et al, 1997). Although delays at signals have been studied on the road, most of the laboratory’s work was done initially by simulating traffic behavior at signals on a special-purpose computer and later on a general-purpose computer (Webster, 1954). In the simulation technique, artificially generated traffic of a random nature was used to feed the computer. It was assumed that once the signals turn green, traffic discharges during the green period at a constant rate (the saturation flow) as long as the queue exists (Webster, 1966). And belong to the same reference; some later theoretical work, which takes account of bunching in the arrival pattern, has been carried out by Miller (1963a).

The methods of delay estimation are usually applied for the fixed time isolated intersection and then adjusted for vehicle-actuated systems and coordinated systems.

Delay at an intersection can occur for two reasons:

- Traffic Delay (DT) due to traffic interaction with other movements in the intersection;
- Geometric Delay (DG) due to deceleration and acceleration when making a turn in the intersection.

According to Papacostas and Prevedouro (1993), traffic delay can be further divided into travel delay and stopped delay. Travel delay for an individual vehicle is the difference between the time a vehicle passes a point downstream of the intersection where it has
regained normal speed, and the time it would have passed that point had it been able to continue at its approach speed. Stopped delay for an individual vehicle is the time duration of "substantially standing still" while waiting in queue at a signalized intersection approach.

2.2.1 Delay Estimation for Fixed Time Traffic Signal Control

According to Rouphail et al (FHWA, 1992), delay models contain both deterministic and stochastic components of traffic performance. The deterministic components are estimated based on the following assumptions:

- A zero initial queue at the start of the green phase,
- A uniform arrival pattern at the arrival flow rate (q) throughout the cycle,
- A uniform departure pattern at the saturation flow rate (S) while a queue is present, and at the arrival rate when the queue vanished,
- Arrivals do not exceed the signal capacity, defined as the product of the approach saturation flow rate (S) and its effective green to cycle ratio (g/c).

A simple diagram describing the delay process is shown in Figure 3.2 below:

![Figure 3.2 Deterministic components of delay models (FHWA, 1992)](image)

Two delay components are estimated for each lane group (Papacostas and Prevedouro, 1993): the uniform delay \(d_1\) and the overflow delay \(d_2\). The former accounts for uniform
arrivals, while the latter accounts for random arrivals. The two delay components are estimated as follows:

\[ d_1 = 0.38C \frac{(1 - \frac{c}{b})^2}{1 - \frac{c}{b}} X \]  
\[ d_2 = 173X^2 \left\{ X - 1 + \frac{(X - 1)^2 + 16X}{c} \right\} \]

The equations above give the stopped delay. The total delay for each lane group is estimated as follows (Papacostas and Prevedouro, 1993):

\[ TD_i = PF_i (d_1 + d_2) \]

where:
\[ TD_i = \text{Total delay for lane group } i \]
\[ PF_i = \text{Progression factor for lane group } i \]

The progression factor accounts for the process of vehicle arrivals in conjunction with the signal indications. When most arrivals take place while red is displayed for the lane group analyzed (poor progression), the delays are likely to be higher than average (PF > 1.0). Random arrivals result in average conditions (PF = 1.0). On the other hand, when most arrivals take place while green is displayed (good progression), the delays are likely to be lower than average (PF < 1.0).

The average delay for an approach \( j \) is calculated as:

\[ D_j = DT_j + DG_j \]

where:
\[ D_j = \text{Mean delay for approach } j \text{ (sec/pcu)} \]
\[ DT_j = \text{Mean traffic delay for approach } j \text{ (sec/pcu)} \]
\[ DG_j = \text{Mean geometric delay for approach } j \text{ (sec/pcu)} \]

The average traffic delay for an approach \( j \) can be determined from the following formula (based on Akcelik, 1988):

\[ DT = c \times 0.5 \times \frac{x \left( 1 - GR \right)}{(1 - GR \times DS)} \times 3600 \times \frac{NQ_j \times 3600}{C} \]

where:
\[ DT_j = \text{Mean traffic delay for approach } j \text{ (sec/pcu)} \]
\[ GR = \text{Green ratio (g/c)} \]
\[ DS = \text{Degree of saturation} \]
\[ C = \text{Capacity (pcu/h)} \]
\[ NQ_j = \text{Number of pcu that remain from the previous green phase} \]
The calculation results are not valid if the capacity of the intersection is influenced by "external" factors such as blocking of an exit due to downstream congestion, manual police control etc.

The average geometric delay for an approach \( j \) has been empirically estimated as follows (Bang, 1997):

\[
DG_j = (1-psv) \times pT \times 6 + (psv*4) \tag{[3:6]}
\]

where:

- \( DG_j \) = Mean geometric delay for approach \( j \) (sec/pcu)
- \( psv \) = Proportion of stopped vehicles in the approach
- \( pT \) = Proportion of turning vehicles in the approach

### 2.2.2 Effect of Upstream Signals

Roughail (1989) presented a set of analytical models for estimating progression adjustment factors (PAFs) to delays at signalized, coordinated intersection approaches. He stated in a later study (FHWA, 1992) that at a point located downstream of a traffic signal, two different types of observations are made:

a- Vehicles pass the signal in “bunches” that are separated by a time equivalent to the red signal (platoon effect), and

b- The number of vehicles passing the signal during one cycle does not exceed some maximum value corresponding to the signal throughput (filtering effect).

Pacey (1956) has studied and modeled the phenomenon of platoon diffusion or dispersion. There, the effect of vehicle bunching weakens as the platoon moves downstream, since vehicles in it travel at varying speeds, spreading over the downstream road section. Hillier and Rothery (1967) analyzed vehicle delays at pre-timed signals using the observed traffic profiles.

The TRANSYT model of platoon dispersion is a well-known example using the estimation of deterministic delays in a signalized network. Figure 3.3 shows the TRANSYT model of dispersion.

The OUT profile is deduced from the arrival pattern (IN profile), the signal times and the saturation flow. As a platoon moves down a link, it disperses. Vehicles at the front will tend to spread ahead; the end of the platoon will also spread out forming a longer down-out tail. This effect is modeled in TRANSYT with a standard formula derived from on-street measurements.

\[
q^I_{(k+1)} = F \cdot q_k \cdot p + (1-F) \cdot q^I_{(k+1-1)}
\]

where:

- \( q^I_{(k+1)} \) = Traffic flow in step \( k \) of IN-profile
- \( q_k \) = Traffic flow in step \( k \) of UT-profile
- \( p \) = Share of UT-profile which arrives at link
- \( t \) = 0.8 of the average travel time in the section where the dispersion occurs
- \( F \) = \( 1/(1+0.35t) \) = adjustment factor

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Figure 3.3 The TRANSYT model of dispersion (TRL, 2002)

The default dispersion parameter can be modified (normally 35) for situations where the rate of dispersion is significantly greater or less than normal.

2.2.3 Delay Estimation for Vehicle-Actuated Control

In vehicle-actuated control (VA), the detectors provide information to the signal control device. This device consequently determines the green time needs and different extensions. Under these conditions, delay cannot be calculated by analytical methods. The calculation will require the sequence and distribution of the reproduced green times. This is only possible under some special circumstances. Empirical data (the Swedish model – Bang, 1978) have shown that vehicle actuated control gives low value average delay under low traffic conditions compared to fixed-time (FT) control. This difference decreases as the flow increases. At a high saturation level, the phases frequently use maximum green time, which can make VA less efficient than FT. The effectiveness of vehicle-actuated control is greatly dependent highly on the strategy used.

The previous discussion applies to the vehicle movements that can clear the intersection without any secondary conflicts. Such conflicts appear in the case of the normal two phase where left-turning vehicles are discharged in the same phase as opposing straight ahead flow, and when turning vehicles cross the path of pedestrians on the side street crosswalks.

Some of the current delay models use a single adjustment for all types of actuated control and are not sensitive to changes in actuated controller settings. Roupail et al (1997) used TRAF-NETSIM and field data to evaluate a generalized delay model developed to overcome some of these deficiencies.
The best-known models for analysis of traffic signal control can be found in SIDRA (Akcelik, 1996) and HCM (TRB, 2000). The Swedish method is well documented and the computer aid model CAPCAL (Vägverket, 1995) is under continuous development.

### 2.2.4 CAPCAL

A major Swedish development effort regarding traffic impact models was undertaken in the 1970s resulting in the Capacity & Delay Report TV: 131 (Vägverket, 1977), and the traffic signal modeling in this report was undertaken by Bang (1978). TV131 includes a module for estimation of saturation flows on a lane-by-lane basis that incorporates special consideration to secondary vehicle-vehicle conflicts as well as vehicle-pedestrian conflicts.

CAPCAL was originally developed in the late seventies as a computer aid for intersection capacity analysis as documented in TV131. The software has been subsequently developed and enhanced with other measures of performance (Vägverket, 1995).

The following traffic performance measures are used in CAPCAL:
- Degree of saturation;
- Proportion of stopped vehicles;
- Queue length;
- Interaction delay;
- Geometric delay.

The effects of vehicle actuation and of signal coordination ("bunched" arrival flow) are the weakest points of all analytical, deterministic calculation procedures since they basically model fixed-time control. Not only is the empirical basis for different correction factors weak, but the definition of the factors to be corrected for is also vague, to put it mildly (Vägverket, 1995). "Vehicle actuation", for example, includes a large number of different types of systems and their implementations. These cannot easily be characterized by a few variables and the differences between them may well be larger than the systematic difference between fixed time and vehicle actuation.

In TV131, the effect of coordination is ignored while a table of correction factors is supplied for the effects of two types of vehicle actuation, regarding queue lengths, stops and delay (Vägverket, 1977). The two types are "older", e.g. with a fixed phase sequence, and "modern", e.g. a signal group controller with pedestrian push buttons, return to all-red etc. The correction factors range from 1.0 to 0.5 for both types. The estimates were based mainly on a literature survey.

For the present version of CAPCAL, the correction factors for a "modern" vehicle actuation were recalculated using a simulation model, resulting in lower values for very low degrees of saturation in particular. As the traffic volumes approach zero, the probability of arriving during all-red increases and the average delay actually converges to zero. For normal flow conditions, the differences in relation to TV131 are small.

### 2.2.5 SIDRA

SIDRA (Signalized & unsignalized Intersection Design & Research Aid), was developed by Akcelik in ARRB Transport Research Ltd (Akcelik, 1996).
The general form of the model for delay, queue length and effective stop rate in SIDRA is a two-term formula which can be expressed as \( P = P_1 + P_2 \). The first term \( P_1 \) represents non-overflow cases, which occur under low demand conditions, and includes the effect of randomness in arrival (demand) flow rates under such conditions. This differs significantly from previous types of two-term models where the first term does not include any randomness effects (e.g. Akcelik 1980 & 1988; and Akcelik & Rouphail, 1994).

The second term \( P_2 \) is an incremental term associated with overflow delay, overflow queue and queue move-up rate. Overflow conditions (cycle failures) can occur when the average demand is below capacity (temporary cycle over saturation due to random variations in arrival flow rates) or when average demand is above capacity (permanent over-saturation that lasts for a period of time). This model structure is considered to be essential for improved quality of performance predictions, especially for actuated signals where overflow queues are negligible at high degrees of saturation.

### 2.2.6 HCM 2000

The U.S. Transportation Research Board’s Highway Capacity Manual (HCM) (TRB, 2000) has been developed and published in a number of new editions over the last 50 years. HCM 2000 deals with techniques for estimation of capacity, traffic performance and level of service for most types of road traffic facilities. A corresponding manual exists for public transport facilities including bus and rail traffic facilities.

The structure of the HCM 2000 procedures for determination of saturation flow at signalized intersections is similar to those of TV-131 and CAPCAL, as reported by Hagring (2000). HCM 2000 focuses on delay measures for fixed time signal control, but also gives advice on signal timing procedures for vehicle-actuated controllers and the impact on delay of signal progression.

The values derived from the formulas in HCM 2000 represent the average control delay experienced by all vehicles that arrive in the analysis period, including delays incurred beyond the analysis period when the lane group is oversaturated. Control delay includes movements at slower speeds and stops on intersection approaches as vehicles move up in queue position or slow down upstream of an intersection.

HCM 2000 uses many adjustment factors:
- The progression adjustment factor, \( PF \), applies to all coordinated lane groups, including both pre-timed control and non-actuated lane groups in semi-actuated control systems;
- The calibration term \( k \) is to incorporate the effect of controller type on delay;
- The incremental delay adjustment factor, \( I \), incorporates the effects of metering arrivals from upstream signals.

### 2.3 SIMULATION MODELS

Traffic simulation models use numerical techniques on a digital computer to create a description of how traffic behaves over extended periods of time for a given transportation facility or system (TRB, 2000). Traffic simulation models focus on the dynamic of traffic flow. They can represent a range of situations from a single facility to an entire network. Some implicit assumptions include interdependencies between the traffic objects (vehicle
headways, origin destination flow patterns), processing capabilities of the physical places (saturation flow rates, capacities, delay relationships), and processing logic (signal timing controls).

Simulation offers a useful offline test environment in which changes like detector positions and signal controller logic can be made quickly without jeopardizing the safety of road-users.

2.3.1 Description of Simulation Models
Simulation models have several strengths and shortcomings. HCM 2000 (TRB, 2000) describes these strengths and shortcomings, listed below:

Simulation Model Strengths
- Other analytical approaches may not be appropriate;
- Can experiment off-line without using on-line trial-and-error approach;
- Can experiment with new situations that do not exist today;
- Can yield insight into what variables are important and how they interrelate;
- Can conduct potentially unsafe experiments without risk to system users;
- Can study the effects of changes on the operation of a system;
- Can vary demand over time and space;
- Can model unusual arrival and service patterns that do not follow more traditional mathematical distributions.

Simulation Model Shortcomings
- There may be easier ways to solve the problem;
- Simulation models require considerable input characteristics and data, which may be difficult or impossible to obtain;
- Simulation models may require verification, calibration, and validation, which, if overlooked, make such models useless or not dependable;
- The simulation model may be difficult for analysts to use because of lack of documentation or need for unique computer facilities;
- Some users may apply simulation models and not know or appreciate model limitations and assumptions.

The simulation models have common features. These are described below:

2.3.1.1 Common Features
In the report of Smartest Project (2000) there are descriptions of different simulation models, their characteristics, applications and limitation. An important part of this report is summarized below:

Nearly all the models use a time stepping approach where the vehicles are moved around the road network using a fixed time step, typically at one-second intervals. Only three models (FLEXSYT-II, SIGSIM and SIMNET) use an event-based approach where the states of objects in the network are changed at discrete times in response to events on an event list. Simple car following, lane changing and gap acceptance laws are used to govern vehicle movements along road links. Both signalized and non-signalized intersections can be modeled. Queues of traffic form at intersections and can extend all the way to upstream intersections where they can block movements.
Part I: Overview of Urban Traffic Signal Control Strategies and Evaluation Methods

Virtually all the simulators can model both coordinated and adaptive traffic signal systems. Two major approaches are used for modeling traffic signal operation:

- Algorithms for changing the signal settings are an integral part of the simulator;
- Signal control as an external module with a well-defined communications interface to the simulator.

Most of the models have the capability of displaying an animation of the vehicles moving around the network as the simulation progresses. Very few (AIMSUN2, FLEXSYT-II, HUTSIM, MELROSE, Paramics and VISSIM) have a graphical network builder, which can considerably reduce the amount of time required to input the network details.

Most of the models provide outputs that allow efficiency indicators to be measured. These usually include travel times, travel time variability, queue lengths and vehicle speeds.

2.3.1.2 Calibration and Validation

The calibration and validation of the various models is varied. Model calibration and validation refers to the process by which the analyst confirms that the model does in fact provide a reasonable approximation of reality (TRB, 2000). The primary objective of this activity is to adjust the parameters in the model so that simulation model results correspond to real-world situations. Adjustment of these parameters is called calibration. Calibration data is used as an input to the model. An example of calibration data is a vehicle’s characteristics such as its acceleration and deceleration rates.

Validation occurs when the output of the model is statistically compared with the baseline case observed in the field. Typical baseline case parameters are speed, delay, and queue length. Validation data is not a direct input to the model. It is used to check the output of the model.

2.3.1.3 Analyzing Output

Once the model is calibrated and validated, the user can conduct a statistical analysis of the simulation model results for the baseline case with calibrated parameters. If the selected simulation model is stochastic in nature, simulation model results produced by a single run of the model represent only point estimates in the sample population. Typical goals of data analysis using output from stochastic model experiments are to present point estimates of the performance measures and to form confidence intervals around these estimates (TRB, 2000).

2.3.2 Simulation Model Requirements

The following criteria are considered by HCM 2000 (TRB, 2000) in selecting a model: model capabilities, data availability, ease of use, required resources, model applications and past performance, and validation and calibration. Demand specification for the simulation model needed for this study are divided into four approaches:

1. The simulation model should include the ability to model:
   - Different type of strategies, including LHOVRA and bus priority,
   - Incidents,
   - Further expansion beyond current application.

2. The input providing the ability to be calibrated with
   - Acceleration/deceleration functions for different vehicle types,
• Desired speed distribution,
• Saturation flow,
• Headway distributions,
• Car-following model (gaps and stopping distance),
• Critical gaps in yielding and lane distribution model.

3. The output should provide results in terms of
   • Efficiency: travel time, delays, stops, queue lengths, speed and public transport reliability,
   • Safety: headway, interaction with pedestrians, overtaking, and the indicators to the accidents.

4. The model interface should be user friendly allowing for:
   • The possibility to adjust it for study needs which depend also on the contacts with the appropriate people who can do that,
   • Graphical user interface for input, editing and for presentation of results,
   • The ability to change the design of the intersections with few input change,
   • Execution times several times faster than real time.

2.3.3 Selecting the Equivalent Simulation Models

Each simulation model, depending on the application, has its own strengths and weaknesses. It is important to relate relevant model features to the needs of this study and determine which model satisfies these needs to the greatest extent.

After studying different available simulation models, it was decided to use HUTSIM and VISSIM depending on the criteria above and the experience at our institute and the possibility to adjust these models for study needs, which is also dependent on contact with the appropriate people who can make such adjustment. HUTSIM was already used and adjusted to LHOVRA technique. VISSIM was also used and during the attempt to adjust it to LHOVRA too.

2.3.3.1 HUTSIM

HUTSIM is a micro-simulation tool developed especially for traffic signal simulation. It can be connected with real signal controllers, which makes it possible to test and evaluate real control strategies. Recently the scope of HUTSIM has been enlarged towards general urban traffic simulation (Sane and Kosonen, 1996). HUTSIM can be used for:

   • Evaluation and testing of signal control strategies;
   • Evaluation of different traffic arrangements;
   • Development of new control systems;
   • Evaluation of telematics applications.

The limitations of the model are the time needed to construct large models at detail level and the need for a powerful PC to model heavy traffic.

Validation and Calibration: HUTSIM calibrated with acceleration/deceleration rates for different vehicle types, car-following gaps and stopping distance and critical gaps in yielding and lane switching. And it is validated with delays, stops, queues and saturation flows of different lane types.
2.3.3.2 VISSIM

VISSIM models transit and traffic flow in urban areas as well as interurban motorways on a microscopic level. It is a decision support system for traffic and transport planners. Alternative scenarios of complex intersections and control strategies are evaluated using VISSIM before the situation is actually built respectively implemented (PTV, 2001). VISSIM can be used to:

- Define optimal vehicle actuated signal control strategies,
- Test various layouts and lane allocations of complex intersections,
- Test the location of bus bays and test the feasibility of complex transit stops,
- Find appropriate lane allocations of weaving sections on motorways.

The traffic flow model of VISSIM is a discrete, stochastic, time step based microscopic model, with driver-vehicle-units as single entities. The model contains a psychophysical car following model for longitudinal vehicle movement and a rule-based algorithm for lateral movements (lane changing).

Validation and Calibration: The psycho-physical traffic flow model of Wiedemann and the lane-changing algorithm are the kernel of VISSIM and have continuously been calibrated and validated on motorways by time-consuming manual analysis of roadside and moving observer films in the 1970s and 1980s. Lately, the data of moving vehicles equipped with radar sensors plus automatic video detection has been applied to improve the model at stop-and-go conditions and at various types of intersections.

2.3.4 Simulation Model Development

The study is focused on modeling the effects considering LHOVRA and other control methods with different forms of self-optimized time setting that can be applied in Sweden. The work comprises the following steps:

- Calibration of model based on the field measurements,
- Validation of the simulation model,
- Simulating traffic control strategies for vehicle actuated isolated intersections (for example LHOVRA) with different control strategies/functions for different types of intersection configuration and saturation levels,
- Standardizing the traffic performance output values of the signal traffic control of isolated intersections.

2.3.5 The Required Data to Develop the Simulation Models

In order to calibrate and validate the simulation model for signal controllers, data of the following types are needed, for a small number of intersections and coordinated systems:

1. Basic data to describe the network and the traffic conditions:
   - Geometric design,
   - Signal control and detector functions,
   - Traffic distribution and traffic composition,
   - Traffic demand.

2. Calibration data for traffic behavior in simulation models:
   - Gap acceptance,
   - Car following,
• Acceleration and retardation,
• Driver behavior in the dilemma zone,
• Yielding behavior,
• Lane changing behavior,
• Lane selection.

3. Validation data for the simulation model considering the traffic effects
   • Travel time and delay,
   • Number of stops,
   • Queue length,
   • Saturation flow.

2.4 CONCLUSIONS

Compared with empirical and analytical models, simulation models predict performance by stepping through time and across space, tracking events as the system state unfolds. Time can be continuous or discrete, and system state is a technical term that effectively describes the status or current condition of the system. Empirical models predict system performance on the basis of relationships developed through statistical analysis of field data, whereas analytical models express relationships between system components on the basis of theoretical considerations as tempered, validated, and calibrated by field data (TRB, 2000).

Some of the estimations in this section are:
   • Field measurements are a very time-consuming and expensive method;
   • Analytical methods can be applied for fixed-time control of isolated intersection and then adjusted for the vehicle actuated and for the coordinated systems. (For the present version of CAPCAL, the correction factors for a "modern" vehicle actuation were recalculated using a simulation model);
   • Simulation offers a useful offline test environment in which changes like detector positions and signal controller logic can be made quickly without jeopardizing the safety of road users.

Therefore, it is recommended to use the simulation models for the vehicle-actuated and self-optimizing strategies and to use field measurements as necessary as well as for collecting simulation input data. For the purposes of this study, it was also decided to use both field measurements and simulation models calibrated and validated with empirical data to study the impacts of isolated and coordinated control systems.
Part II

Impacts of Isolated Signal Control with the LHOVRA Technique
ABSTRACT

The LHOVRA technique is the predominant isolated traffic signal strategy in Sweden. It was originally developed to increase safety and reduce lost time and the number of stopped vehicles at signalized junctions along high-speed roads (70 km/h or more). During its trial period, the accident rate at the test intersections reduced from 0.7 accidents per million incoming vehicles to 0.5. At present there is a lack of knowledge regarding the impacts on traffic performance of urban environment of different LHOVRA functions. Past-end green was originally incorporated as part of LHOVRA (the “O” function) and was intended to reduce the number of vehicles in the dilemma zone and thereby reduce the number of red light drivers and rear-end collisions.

The aims of this part of the thesis were to evaluate and improve the “O” function. Field studies at three intersections with the same speed limit have shown that, depending on the actual speed distribution in the approaches, the actual dilemma zone varied considerably, and thereby the effectiveness of the “O” function. Many ideas to improve this function were suggested including moving the detectors closer to the stop line and making the detectors speed dependent.

A micro-simulation experiment based on field data was performed to test design changes. VISSIM was selected mainly due to the possibility to define behavior in relation to the onset of amber “reaction-to-amber”. Particular attention has been given to the “reaction-to-amber” function in VISSIM, which can be assigned to allow the vehicle/driver only one decision to stop or go when there is a signal change from green to amber.

The simulation experiment considered four different scenarios, two different detector positions and two different signal controller logic programs for the “O” function. The results confirmed the ideas and showed that:

- A distance closer to the stop line (110 and 65 meters) has positive effects on safety in the form of a reduction in red-light violations and measures of Time To Collision (TTC) and positive effects on performance in the form of the proportion of the green time to the cycle time;
- A speed dependent “O” function with the greater distance to the stop line (130 and 80 meters) reduces wasted green time and the number of stops after receiving PEG;
- A speed dependent “O” function with the closer distance to the stop line (110 and 65 meters) reduces red-light violations and the number of conflicts;
- Furthermore, it has been shown that a speed dependent “O” function may have the ability to eliminate conflicting situations completely if past-end green is only given to those vehicles that really need it.

But it is still the standard “O” function, which has the lowest average delay time. However, the results show that there is no scenario that is better in all aspects.

There are reservations to how far the results of this study can be generalized to real-world scenarios given the fact that the this experiment is based on simulation. Work will now proceed to test other models based on newly collected field data using this methodology in order to test their robustness.
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1 INTRODUCTION

1.1 BACKGROUND
The LHOVRA technique is the predominant traffic signal control strategy in Sweden. The strategy was originally developed to increase safety and reduce lost time and the number of stopped vehicles at signalized junctions along high-speed roads (70 km/h or more). During its trial period, the accident rate at the test junctions reduced from 0.7 accidents per million incoming vehicles to 0.5 (Brüde & Larsson, 1988). After its initial success, LHOVRA has been widely used in Sweden and the other Scandinavian countries also in urban areas with a lower speed limit of 50 km/h.

LHOVRA is not software, but a modular toolbox of traffic engineering functions based on the traditional Swedish signal group based control. The traffic engineer decides which functions are suitable for use at specific junctions.

At present, knowledge regarding the impacts on traffic performance of urban environment of different LHOVRA functions is insufficient. It is therefore important to develop a traffic performance and safety model for these impacts. Some attempts have been made to develop the “L” function in the project “Red Driving” to achieve super LHOVRA (Kronborg, 2003), but nothing important has been done to improve the “O” function.

Past-end green was originally incorporated as part of LHOVRA (the “O” function) and was intended for use at isolated junctions on high-speed roads as a means of reducing the number of vehicles in the option zone and thereby reduce the number of red light drivers and rear-end collisions.

The author studied this function in his master’s thesis (Al-Mudhaffar, 1998), which resulted in the idea of improving the function by integrating the past-end green (PEG) in the maximum green time. This reduces the delay without impairing safety.

1.2 OBJECTIVES OF THIS PART
The objectives of this part of the thesis are as follows:

- To evaluate the traffic performance and traffic safety impacts of the LHOVRA technique;
- To improve the effectiveness of the Incident Reduction Function “O” function;
- To evaluate the suggested ideas for improving the “O” function.

1.3 LIMITATIONS
The limitations in this part of the thesis are as follows:

- The impacts of the LHOVRA technique focus on the “O” function;
- The study does not include the effects on pedestrians.


1.4 METHODOLOGY
The following steps were included in the research methodology for this part of thesis:

- Literature study of driver behavior, dilemma zone and the “O” function in the LHOVRA technique;

- Data collection for the purposes of
  - Empirical determinations of driver behavior at the signalized intersections and the incident reduction function,
  - Input data for simulation model;

- Analysis of driver behavior and the effectiveness of the incident reduction function;

- A study to improve the applicability of the “O” function;

- Calibration and validation of the VISSIM simulation model, and using it for evaluating the suggested ideas to improve the “O” function.
2 DRIVER BEHAVIOR, DILEMMA ZONE AND THE LHOVRA TECHNIQUE

“The recognition of driver behavior is particularly critical in the design of the traffic signal clearance or change interval” (Wortman, 83). One of the most important things to be discussed as regards driver behavior at signalized intersections, is the choice of whether or not to stop the instant the traffic light changes to amber when the vehicle is in the dilemma zone. The characteristics of the driver form the basis of such a decision. Perception-reaction time and the driver’s estimated deceleration rate underlie the decision of the driver to stop or to continue through the intersection. Since the judgment of the driver is not always entirely correct, incidents tend to occur.

This chapter first discussed driver behavior at signalized intersections in relation to the dilemma zone, and goes on to look at how the LHOVRA technique can use this definition to design the green time extension known as “past end green” and the possibilities and limitations of this design. The discussion focuses on possible ways to improve traffic performance parallel with safety aspect of this incident reduction or “O”-function.

2.1 DRIVER BEHAVIOR AT SIGNALIZED INTERSECTIONS

Driver behavior on highways has been studied extensively but the situation is not the same for the approaches of the signalized intersections. Two relevant studies from the USA are summarized below:

The first is “Evaluation of driver behavior at signalized intersections” by Wortman and Matthias (1983), where a study of six intersections has been conducted to evaluate and determine the approach speeds of vehicles, the average deceleration rates of stopping vehicles, and the distance of vehicles from the intersection at the onset of the amber interval. The distance from the intersection was measured for both stopping vehicles and those that proceeded through the intersection. The posted speed limits for these intersections ranged from 30 to 50 mph (about 48 to 80 km/h).

The results of the study, shown in Table 2.1, indicate that the mean deceleration rates from the six sites ranged from 7.0 to 13.9 ft/s² (from 2.1 to 4.2 m/s²), with a mean value for all observations of 11.6 ft/s² (3.5 m/s²). The observed mean perception-reaction time for each of the sites ranged from 1.16 to 1.55 s. The mean for all approaches was approximately 1.3 s while the 85th percentile times ranged from 1.5 to 2.1 s”.

The results of this study indicated that, at all six study locations, the mean approach speeds of the last vehicles through the intersection were higher than the first vehicle to stop. This result is very important and indicates that the decision to stop is related to the speed of the vehicle as an aspect of the driver behavior.

Another finding was that the distance from the intersections at the start of amber was considerably lower for the last vehicle through the intersection. In fact, the mean distance differences ranged from approximately 100 to 150 ft (30.5 to 45.7 meters). Although these values are representative for intersections with an approximate difference in posted speed
limits of 34 km/h, the variation in vehicle approach speeds was much less. This variation was less than 10 km/h for the last vehicle through the intersection and less than 13 km/h for the first vehicle to stop. The results of this study are shown in more detail in Table 1.

Table 2.1 Distance from intersection at beginning of amber interval (Wortman, 83)

<table>
<thead>
<tr>
<th>Intersection Approach</th>
<th>Last Vehicle Through Intersection</th>
<th>First Vehicle to Stop</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Distance (ft)</td>
<td>Standard Deviation</td>
</tr>
<tr>
<td>University Drive</td>
<td>118.9</td>
<td>64.0</td>
</tr>
<tr>
<td>Southern Avenue</td>
<td>142.9</td>
<td>60.2</td>
</tr>
<tr>
<td>Day</td>
<td>152.6</td>
<td>72.8</td>
</tr>
<tr>
<td>Night</td>
<td>149.6</td>
<td>92.2</td>
</tr>
<tr>
<td>US-60</td>
<td>136.3</td>
<td>46.9</td>
</tr>
<tr>
<td>First Avenue</td>
<td>102.7</td>
<td>33.7</td>
</tr>
<tr>
<td>Sixth Street</td>
<td>146.5</td>
<td>49.3</td>
</tr>
<tr>
<td>Broadway Boulevard</td>
<td>114.1</td>
<td>46.4</td>
</tr>
<tr>
<td>Day</td>
<td>135.4</td>
<td>54.5</td>
</tr>
</tbody>
</table>

The posted speed limit for a signalized intersection in Sweden is generally 50 or 70 km/h. Results have been calculated for these speeds and are presented in Table 2.

Table 2.2 Approximate distance from intersection at beginning of amber interval for the approaches with a posted speed limit of 50 or 70 km/h (calculated from Wortman, 83)

<table>
<thead>
<tr>
<th>Posted speed limit (km/h)</th>
<th>Last vehicle through the intersection (m)</th>
<th>First vehicle to stop (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean distance</td>
<td>Standard deviation</td>
</tr>
<tr>
<td>50</td>
<td>31</td>
<td>10</td>
</tr>
<tr>
<td>70</td>
<td>42</td>
<td>15</td>
</tr>
</tbody>
</table>

The second is a recent study entitled “Field Testing and Implementation of Dilemma Zone Protection and Signal Coordination at Closely Spaced High-Speed Intersections” by Paot et al (2005), where a study to develop a technique for the field evaluation of dilemma zones at high-speed intersections was carried out at three closely spaced intersections in Ohio.

Two camera detectors {at 300ft (91.44m) and 600 ft (182.88m)} were placed on both approaches of each intersection at Roosevelt Blvd. The number of vehicles in the dilemma zone was determined by counting the number of vehicles that ran red, abruptly stopped at red, and accelerated through amber.

The base case consisted of observations where no green extension was provided when the signal changed to amber. The test cases consisted of green extensions of 1 sec, 2 sec, 3 sec, 4 sec and 5 sec when the vehicles were detected at 300 ft (91.44m) or 600 ft (182.88m), but not both. The traffic data was collected during off peak hours in the morning (9 am – 11 am) and at night (8 pm – 10 pm). In all, 288 hours of data were collected on the six intersection approaches.

The analysis revealed that accelerating through amber was the major conflict for all intersections, followed by running red and stopping abruptly respectively. All three types of
conflict can be successfully used to identify vehicles that experience dilemma zone problems at signalized intersections. The use of these measures in implementing dilemma zone protection techniques in the future is important, since currently no other direct measures are available for identifying dilemma zone problems at signalized intersections.

Overall, this study has shown that, for the roadway segment of Roosevelt Blvd between Highland and Armco, which has a speed limit of 45 m/h (72.41 km/h), dilemma zone protection can be effective by placing detectors at 300 ft (91.44 m) and providing a green extension of 3 sec on most, but not all, approaches. The effectiveness was more evident during the morning period than during the night period, indicating that drivers’ speed behavior may be different during daylight and night conditions.

### 2.2 DILEMMA ZONE

The dilemma zone area at high-speed signalized intersections is defined as a place in an isolated high-speed signalized intersection where a high potential for an accident exists in a roadway section close to the stop line. In these situations, a driver on seeing an amber light may not be able to stop in advance of the stop line with an acceptable deceleration rate, or clear the intersection during the change interval (Vägverket, 1991). Figure 2.1 explains the dilemma zone concept.

![Figure 2.1 Schematic representation of a dilemma zone](image)

Coming to the determination of the dilemma zone, Alexander and Bång (1964) in their thesis define the limits of the dilemma zone according to the following conditions:

\[
x_{\text{max}} \leq \tau \cdot v_0
\]

For the maximum distance for a passing car, and

\[
x_{\text{min}} \geq \delta_2 \cdot v_0 + \frac{v_0^2}{2r}
\]

For the minimum stopping distance.
where:
\( v_0 \) = Approaching speed in m/s
\( x \) = Approaching distance in m.
\( \delta_2 \) = Reaction time for braking + time to start braking
\( r \) = Required retardation
\( \tau \) = Time length of the amber light

Taking the 90th percentile of the drivers’ reactions to amber to determine the dilemma zone as the area of the approach between a point where 90% of the drivers will stop on amber, and a point where 90% of the drivers will go (i.e. 10% will stop), the boundaries of a dilemma zone for 56 km/h are between 77 and 31 meters, and between 100 and 46 meters for 72 km/h.

Most methods for assessing the safety of intersections do not include an assessment of signal operations. Reducing the number of vehicles in the dilemma zone for an intersection approach should reduce the number of drivers who do not receive dilemma zone protection and thereby reduce the probability of crashes for those approaches (Zimmerman & Bonneson, 2004). Various types of traffic control devices or techniques have been used to reduce dilemma zone problems. These include advance-warning signs with or without flashers, vehicle detectors, and amber interval timings. A proper amber interval may provide reasonable amber timing to minimize the dilemma zone problem.

Parsonson (1978) proposed that at signalized intersections where approach speeds are 56 km/h (35 mph) or higher, the traffic engineer needs to select a detector-controller configuration that will (a) detect an approaching vehicle before it enters the dilemma zone and either (b) extend the green signal to provide safe passage through the zone or else (c) end the green signal when the vehicle is still upstream of the dilemma zone and thereby provide adequate stopping distance.

The Swedish control strategy LHOVRA defines the dilemma zone concept as primarily concerning the fact that: “…drivers who encounter change to amber in the zone make different decisions on whether or not to stop”. It is also stated that the dilemma zone, on approaches with a speed of 70 km/h, is between 97 and 53 meters upstream of the stop line. If a vehicle’s speed is 70 km/h and the length of the amber period is 5 seconds, a driver can proceed without committing a red light infringement from a point 97 meters upstream of the stop line. With a short reaction time (e.g. 0.75 seconds), and a heavy deceleration rate (e.g. 0.5 g), a driver can choose to stop from a point 53 meters from the stop line. Within these two distances (97 - 53 = 44 meters), drivers can choose either to drive on or to stop, and this section can be described as the dilemma zone. The stop distance is calculated by the formula: Stop distance = reaction distance + deceleration distance, as explained in Figure 2.2.
Theoretical dilemma zone in an approach with 70 km/h. (Vägverket, 1991)

The decision to stop or continue is a result of the differing behavior of drivers. Different behavior results in different decisions at the same point in an approach. The dilemma zone is a place with rear-end collision potential, mainly because of the differing propensities of drivers to brake or continue through a red light. For rear-end collisions to occur, at least two vehicles must be traveling close together in the approach (i.e. within the described dilemma zone), when a more cautious driver decides to brake and a more aggressive driver decides to continue.

2.3 THE LHOVRA TECHNIQUE

The LHOVRA technique was suggested as a control strategy for rural roads with a speed of 70 km/h at the end of the 1970s. After its initial success, LHOVRA has been widely used in Sweden and the other Scandinavian countries in urban areas also at a lower speed (50 km/h).

The LHOVRA acronym describes the following functions:

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Swedish Name</th>
<th>English Translation</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>Lastbil prioritering</td>
<td>Truck, bus and platoon priority</td>
</tr>
<tr>
<td>H</td>
<td>Huvudväg prioritering</td>
<td>Main road priority</td>
</tr>
<tr>
<td>O</td>
<td>Olycka reduktion funktion</td>
<td>Incident reduction</td>
</tr>
<tr>
<td>V</td>
<td>Variabel gul</td>
<td>Variable amber time</td>
</tr>
<tr>
<td>R</td>
<td>Rödkörning control</td>
<td>Red driving control (variable red time)</td>
</tr>
<tr>
<td>A</td>
<td>Allrödvänding</td>
<td>All red turning</td>
</tr>
</tbody>
</table>

The technique assumes an increased knowledge of the composition and characteristics of the flow on the different approaches to the intersection. This knowledge is gathered using
detectors through the approach. Figure 2.3 shows the use of detectors for each of the functions included in LHOVRA.

<table>
<thead>
<tr>
<th>Functions</th>
<th>Use of detectors for LHOVRA functions on a 70 km/h approach road</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>V</td>
<td>V</td>
</tr>
<tr>
<td>R</td>
<td>R</td>
</tr>
<tr>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>

**Figure 2.3** The use of detectors for the different LHOVRA functions on a typical 70 km/h approach road (Vägverket, 2002)

As with most vehicle actuated signal control systems, the duration of green time is variable. When there is demand, green will always be given for a minimum time-period, see Figure 2.4. If there is a continued demand after this minimum period, the duration of the green signal can be extended up to maximum green time limit.

In a situation where two or more signals groups with non-conflicting traffic flows are co-ordinated within a signal phase, a signal group that has shown green for the minimum period but has no continued demand will be allowed to stay green provided that one of the other groups in the phase has continued demand. This is referred to as *passive* green as opposed to *active* green (VV, 1991; 2002).

**Figure 2.4** Diagram illustrating the different signals and signal status during one cycle for a typical signal group used to control vehicles
The review of LHOVRA functions below is taken from the English version of the manual (report 1987:57 VV, Division of Road Planning and Design. Pages 45-46)

2.3.1 L Function
This function is normally active with a cyclic signal group sequence and can be used on a primary road where truck priority is desired. From the cost aspect, this function is the most expensive, since it requires detectors placed far out in the approaches.

2.3.2 H Function
The H function can be used on a major/primary road where priority is desired. It should be combined with O function to avoid any tendency towards continuous maximum time utilization.

This function gives priority to the road defined as the primary road. It does not consider specific safety effect aspects.

2.3.3 O Function
The O function should normally be used. The most difficult aspect in its application is to determine the practical dilemma zone for the particular type of approach. It is important that the O function interval timing allows the last extending vehicle to clear the dilemma zone safely before the change of light begins, see Figures 2.5 and 2.6.

The basis of the function is that the last extending vehicle maintains a certain constant speed. This is not the case in approaches exposed to friction, i.e. where the flow in the approach is disturbed for some reason. Friction occurs in approaches with mixed lines. Here the O function interval timing must be studied extra closely in connection with final adjustment. For the primary road’s approaches, measuring and minimizing the portion of vehicles passing D80 (detector at 80m) at green/amber can do this. The applications describe interval times for ideal cases without friction. In practice, the interval times may sometimes need to be increased above their theoretical values.
Figure 2.5 O Function: Incident reduction (Vägverket -VV, 2002)

Figure 2.6 Example of using PEG in the time setting
2.3.4 V Function
The V function should be used in sub-approaches without secondary conflict and cycle traffic. Detector faults in D80 must generate a continuous pulse that in turns gives continuous full green/amber time. See Figure 2.7.

In approaches with higher permitted speeds than 50 km/h, the first detector (in the direction of travel) controlling V function (D80) must have increased sensitivity. This should be such that a detector pulse is always generated when a motorcycle passes at 90 km/h.

Figure 2.7 V Function - Variable Amber (VV, 2002)
2.3.5 R Function

The R function should be used together with the O function. An R function alone should be used restrictively owing to the learning risk. Learning is considering to be excluded in a combination using the R function. In simultaneous R and O function, intentional red light infringements can only occur when the maximum permitted green display is exceeded and change to green/amber consequently takes place without regard to the vehicle’s position in the approach.

![Figure 2.8 Red-driving control activated (VV, 2002)](image1)

**Figure 2.8** Red-driving control activated (VV, 2002)

**Control of red driving for left turning traffic:** The system works in the same way, considering a safe time designed by the situation. Figure 2.8 shows how this function affects the diagram of phases.

![Figure 2.9 Red-driving control for turning traffic (VV, 2002)](image2)

**Figure 2.9** Red-driving control for turning traffic (VV, 2002)
This function provides drivers driving through red with the possibility to reduce the risk of collision. It does not, by any means, reduce the numbers of cars driving through red.

2.3.6 A Function

This function aims to reduce as far as possible the number of instantaneous green-amber–red-green cycles and to ensure that the approaching vehicle is far enough away if they occur. The function detects the tailing vehicle in the all red situation and prevents unnecessary changes that will disrupt smooth driving. This function is designed to not cause stress to drivers as far as possible and mainly occurs when all red situations are likely to happen.
3  EVALUATION OF THE LHOVRA TECHNIQUE BY FIELD STUDY

This chapter covers data collection and reduction methods and results:

3.1  FIELD DATA COLLECTION

A vast amount of data was selected for the purpose of this thesis. It was important to define the needs and to select the sites carefully to have the reliable data.

3.1.1  The Required Data

The data required in this thesis were to achieve two main purposes:

1. Empirical determination of the driver behavior at the moment of facing amber light. Data to analyze the effectiveness of the incident reduction function is also needed;

2. Input data for simulation model, data of the following types are needed:
   - Basic data to describe the network and the traffic conditions,
   - Calibration data for traffic behavior in simulation models,
   - Validation data for the simulation model considering the traffic effects.

These data are:
- Car following model parameters,
- Stop distance at the stop line,
- Acceleration and retardation,
- Probability to drive or stop at change from green to amber,
- Arriving traffic generation (time gap),
- Flow and turning flows through the time,
- Traffic composition,
- Queue length,
- Lane distribution,
- Saturation Flow.

3.1.2  Site Selection

Measurements for the purpose of the empirical study were carried out at five typical intersections in urban areas with independent signal control with the LHOVRA technique. The factors taken into account were the speed, separated left-turning signal control, significant flow to measure queues and a clear view of the intersection from a potential camera position. The measured places are located in the Stockholm region. The intersections selected for the study present different speeds and different signal types for left-turning movements, as shown in Table 3.1.
### Table 3.1 Design speed and signal control type for measured intersections

<table>
<thead>
<tr>
<th>Intersections name</th>
<th>Number of lanes per approach</th>
<th>Speed (km/h)</th>
<th>Existence of separated left turning lane</th>
<th>Separated left turning signal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Main appr.</td>
<td>Secondary approach</td>
<td>Main appr.</td>
<td>Secondary approach</td>
</tr>
<tr>
<td>Vällingbyv.– Ångermanneg</td>
<td>1</td>
<td>1</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Spånga.– Bällstav.</td>
<td>1</td>
<td>1</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Ängbyplan</td>
<td>2</td>
<td>1</td>
<td>70</td>
<td>50</td>
</tr>
<tr>
<td>Islandstorg</td>
<td>2</td>
<td>1</td>
<td>70</td>
<td>50</td>
</tr>
<tr>
<td>Täby – Ytterbyv.</td>
<td>2</td>
<td>2</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

### 3.1.3 Methods for Data Collection

An example taken from Spånga (Figures 3.1 and 3.2) shows the location of the equipment connected to Data-logger as follows:

- Two digital cameras on the telemast (to film the flow and driver behavior in one approach) are also connected to the data-logger to record the time from the data-logger in order to synchronize the times of all the registrations;

- Controllers to ensure all changes of the traffic lights were recorded;

- Pneumatic tubes which were disposed in the same location of the detectors belonging to the LH0VRA technique. These detectors are located:
  - 100 m from the stop line for O, and A functions,
  - 55 m from the stop line for O, V and R functions,
  - near the stop line for V and R functions.
Figure 3.1 Equipment used at the Spånga intersection

Figure 3.2 Pneumatic tubes’ location in the approach to the Ängbyplan intersection
At this point, the specified information source for every variable is to be defined. Information about flow, heavy traffic percentage, queue length, and stop distance will be gathered mainly from the videos. The videos identify which vehicles are involved and their behavior in the approach. Information must then be gathered for every vehicle from the log file. Passing time at the detectors will be recorded from the logs. The data-logger, which has 32 channels, is shown in Figure 3.3. Figure 3.4 shows an example of log file data.

![Data-logger TMS 07b](image)

**Figure 3.3** Data-logger TMS 07b

![Log file data](image)

**Figure 3.4** Log file data shows the mode, detector number, date, and passage time of every axle-pair as well as other installation events

A post-processing of the log file with Axle Passage Interpreter (API) Program (Archer, 2003) is given information relative to the speed and the flow and its composition. This information could be classified by cycles in one way and by cars in another way.
### 3.1.4 Execution of Data Collection

The recorded times with video film and data-logger are summarized in Table 3.3 below:

**Table 3.3 Recorded times with video film and data-logger at every site**

<table>
<thead>
<tr>
<th>Nr</th>
<th>Site</th>
<th>Date</th>
<th>Recorded time</th>
<th>Begin</th>
<th>End</th>
<th>Focus on</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Islandstorget</td>
<td>29 May</td>
<td>04:00</td>
<td>15:00</td>
<td>19:00</td>
<td>Approach from City + Intersection</td>
</tr>
<tr>
<td>2</td>
<td>Islandstorget</td>
<td>30 May</td>
<td>03:30</td>
<td>06:00</td>
<td>09:30</td>
<td>Approach from City + Intersection</td>
</tr>
<tr>
<td>3</td>
<td>Islandstorget</td>
<td>30 May</td>
<td>00:30</td>
<td>09:30</td>
<td>10:00</td>
<td>Focus on distance</td>
</tr>
<tr>
<td>4</td>
<td>Islandstorget</td>
<td>29 May</td>
<td>03:00</td>
<td>10:00</td>
<td>13:00</td>
<td>Approach from City</td>
</tr>
<tr>
<td>5</td>
<td>Vällingby</td>
<td>29 Aug.</td>
<td>03:30</td>
<td>13:00</td>
<td>16:30</td>
<td>Approach from center + Intersection</td>
</tr>
<tr>
<td>6</td>
<td>Vällingby</td>
<td>29 Aug.</td>
<td>03:30</td>
<td>13:00</td>
<td>16:30</td>
<td>Approach from City + Intersection</td>
</tr>
<tr>
<td>7</td>
<td>Vällingby</td>
<td>29 Aug.</td>
<td>04:30</td>
<td>07:00</td>
<td>12:30</td>
<td>Approach from center + Intersection</td>
</tr>
<tr>
<td>8</td>
<td>Vällingby</td>
<td>29 Aug.</td>
<td>04:30</td>
<td>07:00</td>
<td>12:30</td>
<td>Approach from City + Intersection</td>
</tr>
<tr>
<td>9</td>
<td>Täby</td>
<td>24 Sep.</td>
<td>02:30</td>
<td>16:00</td>
<td>18:30</td>
<td>Approach from center</td>
</tr>
<tr>
<td>10</td>
<td>Täby</td>
<td>25 Sep.</td>
<td>04:00</td>
<td>07:00</td>
<td>11:00</td>
<td>Approach from center</td>
</tr>
<tr>
<td>11</td>
<td>Täby</td>
<td>25 Sep.</td>
<td>04:00</td>
<td>07:00</td>
<td>11:00</td>
<td>Intersection</td>
</tr>
<tr>
<td>12</td>
<td>Täby</td>
<td>24 Sep.</td>
<td>02:30</td>
<td>16:00</td>
<td>18:30</td>
<td>Intersection</td>
</tr>
<tr>
<td>13</td>
<td>Ängbyplan</td>
<td>9 Oct.</td>
<td>04:30</td>
<td>13:30</td>
<td>18:00</td>
<td>Approach from center</td>
</tr>
<tr>
<td>14</td>
<td>Ängbyplan</td>
<td>9 Oct.</td>
<td>04:30</td>
<td>07:30</td>
<td>11:00</td>
<td>Intersection</td>
</tr>
<tr>
<td>15</td>
<td>Ängbyplan</td>
<td>9 Oct.</td>
<td>04:30</td>
<td>13:30</td>
<td>18:00</td>
<td>Intersection</td>
</tr>
<tr>
<td>16</td>
<td>Ängbyplan</td>
<td>9 Oct.</td>
<td>03:00</td>
<td>07:30</td>
<td>10:30</td>
<td>Approach from center</td>
</tr>
<tr>
<td>17</td>
<td>Spånga</td>
<td>7 Oct.</td>
<td>04:30</td>
<td>07:30</td>
<td>12:00</td>
<td>Intersection</td>
</tr>
<tr>
<td>18</td>
<td>Spånga</td>
<td>7 Oct.</td>
<td>04:30</td>
<td>07:30</td>
<td>12:00</td>
<td>Approach from center</td>
</tr>
<tr>
<td>19</td>
<td>Spånga</td>
<td>7 Oct.</td>
<td>04:30</td>
<td>14:30</td>
<td>18:00</td>
<td>Intersection</td>
</tr>
<tr>
<td>20</td>
<td>Spånga</td>
<td>7 Oct.</td>
<td>04:30</td>
<td>14:30</td>
<td>18:00</td>
<td>Approach from center</td>
</tr>
</tbody>
</table>
3.2 METHOD OF DATA REDUCTION

The decision of the driver to stop or go, at the moment of facing the beginning of the amber light, is dependent on a number of factors including the current speed of the vehicle and the distance to the stop line, but also certain personal and physical characteristics such as reaction time, level of driving experience, driving style (aggressive – defensive), sense of urgency, etc. As a result, three events may happen at the stop line: the vehicle stops or passes either during amber or red.

Another important aspect is the effectiveness of the “O” function. This is ascertained by determining the used portion of the extended green time and thereby its necessity. In this case, only vehicles passing during past-end green are registered.

The method of following these alternatives is described in the following steps:

1. The exceeding green time over the maximum green time is observed from data-logger registration;
2. Vehicles passing during past-end green, with vehicles before and after, are registered by type, passage time, and speed at the beginning of the dilemma zone;
3. These vehicles and their behavior in the dilemma zone are video-monitored as far as the detector near the stop line;
4. When the vehicle passes the stop line, its speed and the signal light is registered;
5. The judgment of the need for past-end green is made based on:
   - If the vehicle passes the stop line and at what speed,
   - During which signal this happened,
   - How much of the PEG has been used.

3.3 RESULTS

This section will focus on the results that give new knowledge about driving behavior in Sweden. The necessary input data for simulation is documented in the related chapter on simulation.

3.3.1 Driving Behavior and Dilemma Zone

Results from four intersections were analyzed: three with 50 km/h posted speed and one with 70 km/h. Unfortunately, the data collected from the Ängbyplan intersection are not reliable, because the results showed that the green extension at the detected approach is related to the opposite approach, making it very difficult to differentiate the kind of the green extension.

Returning to the formulas to determine the dilemma zone mentioned in section 2.2 and considering them in the diagram of the distribution of the stop-or go diagram, two lines denoting the start and end of the dilemma zone have been introduced, see Figure 3.5. The diagonal straight line indicates the distance that can be covered by a vehicle at its current speed during the normal amber period, i.e. the start of the dilemma zone. The second (curved) line represents the normal braking distance for a light vehicle given current speed and hard deceleration rate during the entire braking event, i.e. the end of the dilemma zone. These lines separate the majority of stop or go decisions, and identify an area of behavioral indecision. This area of variant behavior represents the main safety risk, particularly when a vehicle that has decided to stop precedes a vehicle that decides to go.
Part II: Impacts of Isolated Signal Control with the LHOVRA Technique

The go-decisions that are above the line denoting the distance that can be covered during the amber period (4 or 5 seconds) would normally result in red-light violations were it not for the common yet inappropriate behavior of drivers to accelerate at the onset of amber.

The studied distribution of the stop-or-go behavior of the drivers at the moment of facing amber depends only on their speed and distance to the stop line. The diagram below shows behavior at one of the approaches of the Islandstorget intersection with 70 km/h posted speed, where the amber time is 5 seconds. The perception time is 0.75 seconds and the deceleration time is 4.9 m/s², which are according to LHOVRA handbook (Vägverket, 2002). Figure 3.5 explains the result of the driving behaviors in a 70-km/h approach.

![Diagram of driving behaviors](image)

**Figure 3.5 Driving Behaviors in a 70-km/h approach – Islandstorget (IS)**

Considering 96% of the vehicles, the diagram shows a dilemma zone between 110 and 53 m.

For 50 km/h posted speed intersections, the following Figures (3.6 – 3.8) show that driving behavior differs as a result of differences in the real speed:
Figure 3.6 Driving Behaviors – Vällingby (VA)

Figure 3.7 Driving Behaviors – Spånga (SP)

Figure 3.8 Driving Behaviors – Täby (TA)
A summary of the results of these three intersections with 50-km/h posted speed is shown in Table 3.4. Results show that, at facing change to amber, the mean approach speeds of the vehicles passing through the intersection were higher than those of the vehicles that stopped before the stop line.

Another clear result was that the distance from the intersections was considerably lower for the last vehicle through the intersection than for the first vehicle to stop. In fact, the mean distance differences ranged from 14.4 meters in Vällingby to 50.1 meters in Täby, although these values are for intersections with large variations in vehicle approach speeds. This variation in the mean speed was about 23 km/h for the last vehicle through the intersection and about 20 km/h for the first vehicle to stop.

Table 3.4 The distribution of the stop or go behavior at facing change to amber in relation to speed and distance

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Pass</th>
<th>Stop</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Speed (km/h)</td>
<td>Distance (m)</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>Max</td>
</tr>
<tr>
<td>Vällingby</td>
<td>23.4</td>
<td>39.0</td>
</tr>
<tr>
<td>Spånga</td>
<td>37.8</td>
<td>72.0</td>
</tr>
<tr>
<td>Täby</td>
<td>46.0</td>
<td>79.4</td>
</tr>
</tbody>
</table>

The dilemma zone from the measured fields was estimated by taking the area with a different decision of driving behavior. The comparison of the practical dilemma zone and the real speed range within this zone from the above diagrams is summarized in Table 3.5. The results show that there is a large variation in the mean speeds of all the vehicles in the approaches. This results in a large variation in the dilemma zones which begin from 70 meters (from the stop line) in Täby and from only 22 meters (from the stop line) in Vällingby.

Table 3.5 Practical dilemma zone and the real speed range in three intersections with 50 km/h posted speed

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Mean speed (km/h)</th>
<th>Speed range (km/h)</th>
<th>Dilemma zone (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vällingby</td>
<td>23.3</td>
<td>15 to 28</td>
<td>10 to 22</td>
</tr>
<tr>
<td>Spånga</td>
<td>36.3</td>
<td>27 to 57</td>
<td>20 to 63</td>
</tr>
<tr>
<td>Täby</td>
<td>44.6</td>
<td>30 to 68</td>
<td>23 to 70</td>
</tr>
</tbody>
</table>

3.3.2 Driver Decision Modeling

Depending on the two parameters of speed and distance, the decision model of the stop probability (used in the VISSIM simulation model) is a binary logistic model. The formula is
\[ P = \frac{1}{1 + e^{-(\alpha - \beta_1 \text{speed} - \beta_2 \text{distance})}} \]  

The parameters could be estimated using the SPSS program. Table 3.6 shows the values of alpha, beta1 and beta2 for three intersections.

**Table 3.6 Values of alpha, beta1 and beta2 for three intersections**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Measured value at Islandst. (70 km/h)</th>
<th>Measured value at Täby (50 km/h)</th>
<th>Measured value at Spånga (50 km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha )</td>
<td>- 0.534</td>
<td>- 2.489</td>
<td>- 28.53</td>
</tr>
<tr>
<td>( \beta_1 )</td>
<td>- 0.880</td>
<td>- 0.112</td>
<td>- 0.600</td>
</tr>
<tr>
<td>( \beta_2 )</td>
<td>+ 0.080</td>
<td>+ 0.155</td>
<td>+ 1.014</td>
</tr>
</tbody>
</table>

Using the values of alpha, beta1 and beta2 from the program SPSS, the results of the stop probability when facing change to amber are shown below in Figures 3.9 and 3.10.

The diagrams of the two 50 km/h approaches show little difference depending mostly on the difference in the practical dilemma zone of these approaches as shown in Figure 4.2 above. But it is still clear that the behavior belongs to the same category compared with the diagram of the 70 km/h approach, where the curve is much softer, i.e. has a much wider dilemma zone.
**Figure 3.9** Stop probability at facing change to amber in approach with 70 km/h

**Figure 3.10** Stop probability at facing change to amber in two approaches with 50 km/h
3.3.3 Effectiveness of the “O” Function

It is obvious from Table 3.5 that the speeds of the vehicles in Vällingby were low and the functionality of PEG is doubtful. Further study of the number of stops after receiving the green extension may give better understanding of the functionality of PEG in the low speed approaches. Table 3.7 shows the results from three intersections with 50 km/h recommended speed. The table shows the share of the stopped vehicles to the observed cycles receiving PEG (named as V / C) and the share of the stopped vehicles to the observed vehicles receiving PEG (named as V / V).

Table 3.7 The share of the stopped vehicles after receiving Past End Green

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Flow veh/h/lane</th>
<th>Observed cycles</th>
<th>Cycles with PEG</th>
<th>Vehicles received PEG</th>
<th>Stopped veh after receiving PEG</th>
<th>Share % V / C</th>
<th>V / V</th>
<th>Detector distance to stop line m</th>
<th>Mean speed at facing amber km/h</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vällingby</td>
<td>348</td>
<td>318</td>
<td>71</td>
<td>109</td>
<td>25</td>
<td>35.1</td>
<td>22.9</td>
<td>41</td>
<td>23.3</td>
</tr>
<tr>
<td>Spånga</td>
<td>274</td>
<td>276</td>
<td>33</td>
<td>33</td>
<td>3</td>
<td>9.1</td>
<td>9.1</td>
<td>53</td>
<td>36.1</td>
</tr>
<tr>
<td>Täby</td>
<td>717</td>
<td>419</td>
<td>97</td>
<td>506</td>
<td>6</td>
<td>6.2</td>
<td>1.2</td>
<td>85 – 55</td>
<td>44.6</td>
</tr>
</tbody>
</table>

The table shows that the detector location is already adjusted to the actual approach speeds, where Vällingby with 23.3 km/h mean speed has the detector at 41 m, while Täby with 44.6 km/h mean speed has the first detector at 80 m from the stop line.

It is obvious from this table that the share of the vehicles that stopped after receiving the PEG extension decreases (from 22.9% in Vällingby to 9.1% in Spånga) as the average speed of the vehicles at the time of receiving the PEG increases (from 23.3 in Vällingby to 36.1 km/h in Spånga). In the case of Täby, platoons are another reason for fewer stopped vehicles compared to the number of vehicles received PEG.

3.3.4 O-Function and the Flow Through the Time

The O-function consists of an extra green extension (PEG) after the end of maximum green time. This extension allows the vehicles in the dilemma zone to be evacuated without the stress to take the decision of “pass or brake”. While these vehicles are evacuating, some others may arrive and, depending on the time gap, extend the PEG again. In cases where the traffic is intense, the gap between the vehicles becomes smaller and then the PEG does not perform as it is supposed to do i.e. evacuate vehicles in the dilemma zone, but just a usual green extension. Thus, the PEG also has a time limit, and thereafter the light turns to amber. This time limit is mentioned in section 2.3.3.

The medium time headway between vehicles in a flow is inversely related to the flow. Then it is expected that a high flow will have a larger number of ”O” functions activated. For these data linear regression was used to estimate the relationship between the flow and the number of cycles in which the ”O” function is activated. All the results were shown to be significant.

The following diagrams (Figures 3.11 - 3.13) illustrate how the "O” function was activated and the flow varied and the relationship that this could have with the flow.
Figure 3.11 O-Function activations and Flow through-time – Täby

Figure 3.12 O-Function activations and Flow through time – Spånga

Figure 3.13 O-Function activations and Flow through time – Vällingby
### 3.3.5 Other Results

**Stop gap at the stopline**

To measure the stop gap between vehicles queued at the stop line the following method was used:

1. Number and type of vehicles in the queue are counted;
2. The end of the queue is located approximately in the video film using some other reference points as the pneumatic tubes, lines etc;
3. The length of the vehicles in queue is estimated using the average values from the street design manual ARGUS (Vägverket, 1987);
4. \[ \text{Stop gap} = \text{length of the queue} - \text{length of the vehicles in queue} \]

According to the data from Islandstorget the average stop distance is 3 m with 0.9 m in standard deviation. The observed cases were 28. Table 3.8 shows the result of the stop distance.

**Table 3.8** Stop distance at the stop line of the signalized approach (from Islandstorget)

<table>
<thead>
<tr>
<th>Stop gap (m)</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>St dev</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.9</td>
<td>3.0</td>
<td>4.7</td>
<td>0.9</td>
</tr>
</tbody>
</table>

**Deceleration**

Deceleration data corresponds to data collected at the Ängbyplan intersection. The methodology is based on video analysis. The position of every vehicle at the moment of facing the amber was estimated based on the data provided by the video camera mounted on a tower and by the pneumatic tubes. If the vehicle stops, then its stop position can be defined. Another camera recorded the vehicles from behind. With this focus the moment when the vehicle starts stopping is estimated. Deceleration is calculated using the following kinematical relationship.

\[
a_r = \frac{v^2 - v_0^2}{2(s - s_0)} = -\frac{v_0^2}{2(s - s_0)}
\]

The data collection corresponds to Ängbyplan in the interval 7:30-7:50. The total number of observations was 20 and the average speed was 66.5 km/h. Table 3.9 shows the results of deceleration from Ängbyplan.

**Table 3.9** Results of deceleration from Ängbyplan

<table>
<thead>
<tr>
<th>Deceleration (m/s²)</th>
<th>Min</th>
<th>Mean</th>
<th>Max</th>
<th>St dev</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.4</td>
<td>2.5</td>
<td>4.6</td>
<td>0.9</td>
</tr>
</tbody>
</table>

The average deceleration is near to the default value in VISSIM and the max deceleration near to the assumption in the LHOVRA handbook.

The perception time had also been estimated as the time between the beginning of the amber time when the car is in the dilemma zone and the time when the car begins to stop. The average is about 1 second while the LHOVRA handbook assumes that it is 0.75 seconds.
4 IMPROVING THE INCIDENT REDUCTION FUNCTION

The empirical study in the previous chapter has shown that results of using the “O” function were different depending on the real speed in the approach and the detector location as well as the location of the intersection, as in the case in Vällingby as an example of an intersection near the centre of the city. In this chapter a theoretical study of the “O” function will be done with focus on the approaches with 70 km/h posted speed.

4.1 O-FUNCTION AT A SPECIFIED RECOMMENDED SPEED

The LHOVRA technique manual, due to many factors, suggests that the practical dilemma (option) zone for the 70-km/h approaches is between 130 and 50 meters before the stop line. This function is performed with detectors placed at 130 and 80 meters from the stop line to detect and follow the vehicle within the dilemma zone.

Extension of green time by past-end-green is triggered by the existence of a vehicle in the zone. Preferably, however, a change should not occur when this vehicle is within the dilemma zone as long as the maximum past-end-green is not exceeded. This reduces the risk for rear-end collisions as well as change-induced stress in the approach and reduces the possibility of red light infringement. These objectives can be achieved by other functions in the LHOVRA technique, and using the “O” function for many objectives weakens the argument for the main condition of “past-end-green”.

The LHOVRA manual concurs with the fact that the function is not always useful. It is suggested that postponement of the change from green to amber when there is a vehicle in the dilemma zone, should not take place other than during a reasonable time-period. The function will only be of use as long as the approach is not operating near saturation flow, thereby reaching the limit for permitted maximum green display.

4.2 DISCUSSION POINTS

The extension for the detector at 130 meters is 4.0 or 3.5 seconds, and for the detector at 80 meters 2.0 seconds, to allow a vehicle traveling at 70 km/h to be able to proceed through the intersection, see Figure 5.1.

In criticizing the use of these extensions, it can be seen that 3.5 seconds through the differential distance 50 (130 –80) meters provides the vehicle with a possibility to get a new extension at the 80 meters detector if it moves at a speed of 56.6 km/h (15.7 m/s) or higher. That means that, after a 3.5-seconds extension, vehicles moving at 56.5 km/h or lower speeds are going to miss the second green extension and thereby find themselves at the start of amber in their own dilemma zone. At such speeds the vehicles could easily stop before the stop line if no extension has occurred. A similar problem faces vehicles with speeds from 56.6 until 66.0 km/h, which, after receiving two green extensions, they would not pass but find themselves within their own dilemma zone. The situation will be worse if the first extension is 4.0 seconds, which means that vehicles with speeds between 49.5 and 66.0 km/h risk being within their own dilemma zone, see Figure 5.2.
“Past end green” within a practical dilemma zone in an approach with (70 km/h)

**Figure 5.1** PEG for a vehicle traveling at 70 km/h in an approach with a 70 km/h limit

“Past end green” for a vehicle with (60 km/h)

**Figure 5.2** PEG for a vehicle traveling at 60 km/h in an approach with a 70 km/h limit
4.3 SUGGESTIONS FOR IMPROVEMENT
The practical suggestions presented here are dependent on the demands and requirements of the responsible authority and the possibilities of the application and controller. To ensure safety without compromising traffic performance, different aspects of the function can be improved in many ways that build upon each other.

4.3.1 Changing the Detector Location and Green Time Extension
From the above study it seems that choosing a dilemma zone between 130 and 50 is not so practical and impair safety at some speeds. It has to be reduced so that it starts at a distance of 110 m instead of 130 m. Also, detectors are to be placed at 110 m and 65 m. This solution may move the risky speeds to lower ones, but risk can only be totally eliminated by adjusting the green extensions to the detector’s position to 3.5 + 3.0 seconds.

4.3.2 Using Double Detector at Beginning of the Dilemma Zone
Detecting the vehicle at entry to the dilemma zone by a double detector to measure the speed of the vehicle gives the possibility to make the order of “past-end-green” conditional, thereby excluding the use of the function when it is not needed. If a detected vehicle has a speed of less than 70 km/h an extension is not needed because the vehicle can stop quite easily at this distance.

4.3.3 Considering the Headway Between Two Existing Vehicles in the Dilemma Zone
Depending on the possibility of the controller to remember the passage of vehicles at the detectors, the headway between vehicles is used as a new condition to determine if the extension should be happening.

If the detected vehicle has a speed of 70 km/h or higher, the order of “past-end green” must depend on the existence of another vehicle with a lower speed in the zone. The condition is: the distance remaining to the stop line has to be more than the minimum distance to stop. In other words, if the headway between the existing vehicle and the new approached one is less than (detector distance to the stop line - min possible stop distance of existence vehicle) / speed, then an order of extension must be given.

Using the critical values recommended by the LHOVRA manual, with a detector distance of 110 meters and an 0.5 second safety factor, the formula below becomes:

\[ h < \frac{110 - \left(0.75v + \frac{v^2}{2a} \right)}{v} + 0.5 = \frac{110}{v} \left(0.75 + \frac{v}{g}\right) + 0.5 \]

where:
- \( h \) = The headway between the two vehicles in the dilemma zone (s),
- \( v \) = The speed of the previous (low speed) vehicle in the dilemma zone (m/s),
- \( a \) = The max deceleration of the previous vehicle in the dilemma zone (m/s/s) and
- \( g \) = The gravity acceleration = 9.8 m/s/s

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5 EVALUATING THE SUGGESTED IMPROVEMENTS USING SIMULATION

The reasons for adopting a methodology that uses micro-simulation modeling for the purposes of this study is the simulation’s possibility to offer a useful offline test environment in which changes to detector positions and signal controller logic can be made quickly without jeopardizing the safety of road-users. There are however, a number of disadvantages and problems associated with the use of simulation. These are mainly related to the validity of the simulation model and the results that are generated. These points are discussed in more detail below.

5.1 SCOPE AND OBJECTIVES

The primary objective of this part concerns traffic safety and performance of signal controlled urban intersections and in particular the effectiveness of the existing and an improved incident reduction function in resolving driver dilemma situations. It is hypothesized that the two main suggested improvements would give improvements in safety with little or no loss in traffic performance in comparison to the existing incident reduction function. The suggested improvements to be tested are alternative detector positioning at 110 and 65 meters, instead of 130 and 80 meters, and speed dependent past-end green delay at the dilemma zone entry detector. A secondary objective of this part is to test the methodology described below for future research in this area.

5.2 METHOD

The methodology adopted in this study is centered on the use of micro-simulation modeling as a suitable environment to test the suggested improvements to the incident reduction function. Before details of the simulation experiment are discussed in more detail, it is necessary to consider the intersection test site, and collection and analysis of relevant empirical data that will serve as input to the simulation model. Thereafter, issues related to the use of simulation, choice of simulation software, and methods of model calibration and validation can be discussed. The methodology is also concerned with identifying measures of traffic performance and the choice and application of suitable safety indicators for the purposes of this study, and consideration of how output data from the simulation model will be generated and analyzed.

5.3 INTERSECTION TEST SITE

The intersection that has been chosen as a test platform for the purposes of this study is of relatively simple design by Swedish standards. This particular intersection is used as it has been the basis for the development and testing of the signal controller logic in VisVAP. The intersection is illustrated and described briefly below in Figure 5.1:
As the diagram illustrates, the intersection is controlled through the use of 7 signal groups, each represented by a single signal head. Approach 1 represents the focus area for the study of the “O” function and has a posted speed limit of 70 km/h. This approach has a separate lane and signal group for left-turn movements. Effectively, this approach is a mirror image of approach 3 from the opposite direction. Together, these approaches can be considered the primary or mainline road passing through the intersection. The other two approaches (2 and 4) have a posted speed limit of 50 km/h. Approach 2 also has a separate lane and signal group for right-turn movements. The main signal phases are also indicated in the diagram. Phase 2 is that which allows mainline traffic to pass through the intersection and gets most green time in accordance with the higher demand. Phase 3 shows the main phase for the secondary roads where conflicting left-turn movements exist on green. Phase 1 allows left-turn movements from the mainline approaches, and a right turn from approach 2.

5.4 CHOICE OF MODEL

Following a detailed review of existing micro-simulation software, VISSIM was selected as the model of choice. This particular simulation software tool makes it possible to model intersections with a very high level of detail. The reasons for selecting this particular simulation tool were many. Upon closer investigation, VISSIM proved superior to other microscopic models in the following key aspects:

- The ease with which models can be constructed and adjusted;
- The possibility to define behavior in relation to the onset of amber (“reaction-to-amber”);
- The realistic internal modeling of braking and acceleration in the car-following model;
- The possibility to define time dependent road-user input (e.g. flows and compositions).
5.4.1 VISSIM
VISSIM is a microscopic simulation tool that is time-based (as opposed to event-based) that uses models of driver behavior and vehicle performance for the purposes of modeling urban traffic and transit. Unlike many other simulation tools it is not strictly node and link based and there are no predefined intersection types; instead the user builds a network based on one or more aerial photographs or technical drawings, adding objects to the network at the exact point they are needed and where they will have the desired effect on the dynamic road-user objects. This solution is one that offers great modeling flexibility. The VISSIM simulation model has been validated against the data from various real world situations (Fellendorf & Vortisch, 2001). It is now widely used for the modeling of small to medium sized networks that require a high level of detail.

The flow of traffic in VISSIM is represented as the movement of individual driver/vehicle units. In theory, every driver has stochastically allocated behavioral characteristics and each vehicle has stochastically allocated performance capabilities. Consequently, when a driver is assigned to a vehicle, the driver’s behavior will be adapted to the vehicle performance characteristics.

5.4.2 Car-Following and Driver Behavior in VISSIM
VISSIM uses a psychophysical car-following model for longitudinal vehicle movement and a rule based algorithm for lateral movement, developed by Wiedemann (1974) at the University of Karlsruhe in Germany. Two separate car-following models exist: one (Wiedemann ’99) is recommended for inter-urban motorways and the other for urban town traffic (Wiedemann ’74). Basically, the movement of vehicles according to these models is based on behavioral assumptions regarding desired speed and gap-acceptance (PTV, 2000).

According to Wiedemann’s model, a driver/vehicle unit can be in any one of four states: free driving, approaching, following, or braking. In braking, for example, a medium to high deceleration rate will be applied if the distance to any preceding vehicle falls below a desired safety distance. Acceleration or deceleration is the result of speed, speed differences, distance, and the individual characteristics of the driver/vehicle unit. Wiedemann suggests that the ability to perceive and estimate speed and distance varies among the driver population; these psychological aspects and physiological restrictions are therefore combined in the psychophysical car-following model to provide a realistic and representative amount of variation.

In research that looked at traffic micro-simulation and driver models, the Wiedemann ’74 car-following model was studied in detail with regard to its implementation in the PELOPS micro-simulation program (Ludmann et al, 1996).

It has been reported that the Wiedemann model is being developed to operate with an update resolution of one second owing to the fact that this also represents an average driver reaction time. Using higher resolutions therefore implies that drivers are overly reactive to changes in speed and distance relative to preceding vehicles. To solve this problem, a “buffer-ring” was used to record values in a certain number of memory fields that was dependent on the range of simulation steps and driver reaction time. This solution gave drivers different individual reaction times dependent on the driving situation and allowed a flexible range of simulation steps from 1.0 to 0.01 seconds. The principles of the Wiedemann ’74 car-following model and the concept of the buffer-ring are illustrated in Figure 5.2.
PTV, who market VISSIM, state that the reaction time of driver/vehicle units in more recent versions of the software is subject to a “reaction smoothing” which effectively compensates for different update resolutions in a similar way to the buffer-ring suggested by Ludmann and colleagues. It is also stated that acceleration and deceleration may be slightly higher than that found in real world situations due to the implementation of this function.

Besides the car-following model, there are other behavioral sub-models with different parameters that can be adapted by the user for normal and forced lane-changing and lateral behavior. Different behavior sets can be defined and applied in VISSIM for different link-types and road-user types.

5.4.3 VisVAP
Another very significant factor in the decision to use VISSIM was the possibility to define the Swedish signal controller logic including various LHOVRA functions in the additional VisVAP module. VisVAP is an acronym for “visual vehicle actuated programming”; basically it is a separate software program that communicates with VISSIM by extracting information from detectors and returning control messages to the signal heads or other objects. In effect, VisVAP represents the signal controller. VisVAP (Figure 5.3) also has the advantage of being visual. Effectively, this means that the logic is openly defined in a series of flow charts that are hierarchically related and can be followed by signaling engineers. There is also a non-visual version of this software referred to as VAP.

The Swedish signal controller logic including various LHOVRA functions has now been defined and is still undergoing testing. Presently, the logic works at a one-second-time resolution due to restrictions in the software. Similarly, the safety times between signal changes are rounded up or down making some cycles slightly longer and some shorter. This, however, has no significant effect on the traffic performance or safety effects related to the “O” function.
Figure 5.3 Defining the signal controller logic and LHOVRA functions in VisVAP

5.5 SIMULATION MODEL CALIBRATION AND VALIDATION

Determining if the results emanating from simulation runs are accurate entails making sure the model has been calibrated within acceptable tolerance levels and that it has been validated in accordance with predetermined thresholds for key traffic and safety parameters. The methods of calibration and validation and the level of stringency are most often determined in accordance with the main goals and objectives of the simulation study.

A detailed description of deep calibration and validation is found in the report “Vehicle Actuated Signal Control and the LHOVRA Incident Reduction Function, Archer and Al-Mudhaffar, 2004”. Details about the criteria for calibration and validation in that study are described below.

5.5.1 Calibration Criteria

Simulation model calibration generally refers to the process through which the individual components of the simulation model are adjusted or tuned so that the model will accurately represent the data collected from field measurements. The components or parameters of a simulation model that require calibration include mainly those related to: traffic control operation, traffic flow characteristics and driver behavior. Validation on the other hand, is concerned with testing the accuracy of the model by comparing traffic or safety indicator parameters generated by the model with those collected in the field. Validation is directly linked to calibration in an iterative process where adjustments aim to improve the ability of a model to replicate qualitative and quantitative aspects of behavior and performance measured in the field (see e.g. Milam & Choa, 2000 and Chu et al, 2004).
Given the purpose of this particular study, which considers not only traffic performance but also certain safety issues related to rear-end conflicts, model calibration is based on the following variables:

- Number of signal cycles for signal group 1 on approach 1;
- Amount of green and amber time for signal group 1 on approach 1;
- Degree of bunching generated by the arrival process;
- Car-following behavior (particularly safety distance parameters) when in car-following mode (to ensure that the headway distribution between vehicles at the start of the dilemma zone is acceptable);
- Vehicle deceleration including average and maximum levels of deceleration and the deceleration profile;
- Reaction to amber for driver/vehicle units that find themselves in the dilemma zone.

Calibration is carried out using the above set of variables for two different time-periods, namely the off peak hour between 14:00 – 15:00 and the peak hour between 16:00 – 17:00. The number of simulation runs is calculated using a measure of the total distance traveled by all vehicles in the simulation network and the average network speed. A 95 per cent confidence interval and a 5 per cent allowable error are used as a suitable criterion for both variables.

The traffic control parameters (number of signal cycles and amount of green and amber time) should arguably be part of the validation process, however since the signaling logic is relatively new and untested it is more likely to require revision (i.e. re-calibration). Testing the signal logic during off-peak and peak conditions provides valuable information regarding the signal behavior relative to traffic demand on approach roads.

5.5.2 Validation Criteria

In ideal circumstances, calibration of the model parameters will improve the ability of the simulation model to replicate signal performance, traffic performance and safety indicator results that match field data and observations within an acceptable range of error. Typical traffic performance measures that are used in validation include traffic volumes, average travel times, average delay times, queue lengths and measures of traffic density. Generally, however there are few guidelines regarding the definition of an acceptable range of error. To compare the accuracy of the simulated values against observed and measured traffic variables statistical methods and models such as the GEH-statistic (a comparison of the allowable variance under the GEH formula for GEH = 5 with a variance of 5 per cent) are applied for traffic flow rates and turning percentages for all approaches, and the following data for approach 1 only:

- Vehicle throughput at stop line;
- Saturation flow at stop line;
- Headway distributions at 110 meters before stop line;
- Average and maximum speeds at 110 meters before stop line;
- Average and maximum queue lengths;
- Number of red-light violations;
- Number of safety-critical rear-end conflicts.
5.6 SIMULATION MODEL INPUT DATA

It was decided to simulate a 4-hour afternoon period between 14:00 and 18:00. This period includes the main afternoon peak hour between 16:00 and 17:00. The peak ensures a reasonable level of “O” function activity although the peak demand is rarely at more than half-capacity on the main approaches.

Data regarding the traffic flows and relative turning percentages were collected in an earlier study conducted at this site.

5.6.1 Traffic Flows, Compositions and Turning Percentages

Processing of this original site specific data was carried out in order to generate input data for the simulation model at 15-minute intervals. Turning percentages were also extracted from the original data. Significant differences were found in the first hour compared to the last three. Time dependent adjustments were therefore applied in the simulation model. The traffic compositions were also recorded in the original data at 98 per cent private cars, 1.5 per cent heavy goods vehicles and 0.5 per cent buses on the mainline approaches. On the secondary approaches the proportion of heavy good vehicles and buses dropped to 0.75 per cent and 0.25 per cent respectively. Figure 5.4 shows the traffic flows/demand for the four approaches.

Figure 5.4 Traffic flows/demand for the intersection approach roads

5.6.2 Speed Profiles

The accurate representation of desired speed profiles is critical to the performance of the simulation model. These profiles are usually recorded at a point upstream of the stop line and involve processing to select free-flow vehicles only. Since the behavior of vehicles in the dilemma zone is related to speed it is essential that the profile include a representative distribution of speeds. The actual speed profiles used are illustrated below. These were calculated from the recent collected data for an intersection with a similar traffic flow and posted speed limit. Figure 5.5 shows the speed profiles for the approaches with posted speeds of 50 and 70 km/h.
5.6.3 Driving Behavior in Dilemma Zone

Particular attention has been given to the “reaction-to-amber” function in VISSIM, which can be assigned to allow the vehicle/driver only one decision to stop or go when there is a signal change from green to amber. Default parameters in VISSIM are not sensitive to the design of the intersection or the posted speed. This function is described as a binary logistic regression function, whereby the probability of stopping is stochastically assigned to each driver/vehicle unit in the model based on speed and stop line distance values. The function estimates the probability (p) of stopping given the speed (v) and distance (x) from the stop-line and is critical to the objectives of this study. The formula used is stated below.

\[ p = \frac{1}{1 + \exp\left(-\alpha - \beta_1 \cdot v - \beta_2 \cdot x\right)} \]

The alpha constant and beta variable weightings are situation specific. Their values are critical to the accuracy of the function and determine the form of the curve. The default values suggested for this function have been found to be unsuitable for Swedish signal control. This is probably due to differences in the amount of amber time given as standard in German and Swedish traffic signal applications. The resulting function is illustrated graphically in Figures 5.6 and 5.7.

Figure 5.5 Speed profiles for the intersection approaches with posted speeds 50 and 70 km/h
Part II: Impacts of Isolated Signal Control with the LHOVRA Technique

**Figure 5.6** Graphical representation of the “Reaction-to-Amber” function in VISSIM

![Graphical representation of the “Reaction-to-Amber” function in VISSIM](image1.png)

**Figure 5.7** Graphical representation of the revised “Reaction-to-Amber” function from field measurement

![Graphical representation of the revised “Reaction-to-Amber” function from field measurement](image2.png)

Interestingly, this function also has implications regarding the number of red-light violations. If a vehicle decides to go from a greater distance, it will not manage to pass the stop line before the signal has changed to red, thus producing a violation. This function also emphasizes the need for an accurate speed profile representation. If there are too many speeds in the higher tail end of the speed distribution there will be too many red-light violations, and an imbalance at the other end will produce too many stops. The “O” function probably prevents violations in such cases in the real world by allowing a signal group to remain green.
5.6.4 Acceleration / Deceleration

Also of importance in relation to this function is the way in which vehicles brake and the amount of braking power they actually use. In a number of preliminary tests with VISSIM it was found that braking behavior was very similar to that found in the real world. Average deceleration rates were calculated from real-world data based on the braking distance and initial speed of vehicles as the braking began. An average deceleration rate of 2.47 metres/sec$^2$ with standard deviation 0.85 was found for a total of 20 cases from the field data. The testing with VISSIM showed a braking rate of 2.04 metres/sec$^2$ with standard deviation 0.43 for a total of 74 cases. The onset of deceleration is also realistic in that more and more braking power is applied until finally easing up before stopping, braking power up to a maximum limit is used as and when it is needed in the same way it is in reality. Thus, the braking curve is non-linear. The accurate representation of deceleration is critical for the measures of safety described later. Acceleration was also found to be realistically represented in VISSIM both from a standing start and for vehicles that were already moving.

5.6.5 Degree of Bunching

In many simulation models there are problems establishing model borders and the effects that the borders have or do not have in the simulation model when compared to the real world situation. Given that the studied intersection is quite isolated with some distance to other intersections and limited possibilities to overtake, the arrival pattern of vehicles may require adjustment in the simulation model. Normally, in simulation models vehicles are generated stochastically in accordance with a random seed number and a particular type of distribution (a negative exponential or Poisson distribution is often used to represent vehicle arrivals). The average headway generated between vehicles is then generally dependent on the vehicle demand that is modeled at the point of generation. In some cases, this simple mathematical function may not be representative of the real world vehicle arrival process if there are intersections upstream of the approaches and/or long approach roads with no possibilities for the overtaking of slow moving vehicles.

In order to check the degree of bunching, a threshold was determined to separate free-flow vehicles and those that are in a car-following state. A threshold of 3.0 seconds was chosen in accordance with the car-following model in the U.S. Highway Capacity Manual (HCM, 2000). The data from the Yxvägen site showed that approximately 53 per cent of all vehicles were in car-following mode during the first off-peak hour (14:00-15:00) on approach 1, and also that 19 per cent of all vehicles had a time-headway greater than 10 seconds. During the peak hour (16:00-17:00), the proportion of car-following vehicles increased to 74 per cent with only 4 per cent of all vehicles having a time-headway greater than 10 s.

5.6.6 Headway Distribution Data

An important variable that is very rarely used for the purposes of calibrating a simulation model is the headway distribution. In many models, reproducing the correct flows and speeds at different points within a model is usually accepted as a means of ensuring that vehicles are behaving in a representative manner, and at intersections parameters such as delays, queue lengths and saturation flows are commonly used.

This study focuses on an isolated intersection and the effects of an incident reduction function on safety in the form of rear-end-collisions and conflicts. It is therefore considered important to ensure that the distribution of headway between vehicles at a particular point on the mainline approach road is accurate (prior to the onset of braking). If the number of short headways in the model is overrepresented when compared to the real world situation, there is
likely to be a higher number of rear-end conflicts. Similarly, under-representation of shorter headways will result in too few conflicts.

The headway distribution is therefore important for calibration purposes, particularly where rear-end conflicts are considered. In some cases, the parameters of the car-following model (and even the lane changing model if there is more than one directional lane) may require adjusting in order to reproduce representative gaps at a given point in the simulation model. The headway distributions for the studied site for the first off-peak hour and the hour containing the main peak are shown in Figure 5.8.

Figure 5.8 Headway distributions for off-peak and peak hours at the studied Yxvägen site

5.6.7 Queue Lengths

There are a great many different methods for measuring vehicle queues. The important issue in a simulation study such as this is to ensure that the same method is applied in the model as in the real world situation. In addition, since the queue lengths are measured once per cycle before a change to green, it is important that the number of signal cycles and average amount of green time generated is correct in the simulation model runs. Average and minimum queue lengths were measured for the studied site by counting the number of stopped vehicles at the time the signals changed to green.

The average and maximum queue lengths were measured during the first off-peak hour (14:00-15:00 p.m.) and the peak hour (16:00-17:00 p.m.) at the stop line for signal group 1 on approach number 1. During the off-peak hour, the average queue length was 2.60 vehicles and the maximum was 6.00 vehicles. For the peak hour, the average queue length was 7.10 vehicles and the maximum was 18.00 vehicles.

5.6.8 Saturation Flow Rate

The saturation flow rate at a particular intersection stop line is normally an important calibration measure. For the purposes of this study, a saturation flow rate was calculated from the same field study site as that from which the speed profiles, “reaction-to-amber” function, and deceleration rates were derived. The stop line at the similar field study site and the actual test site had a similar passage of vehicles and proportion of vehicles traveling straight ahead and turning right. No left-turn was allowed in either case. The right turning vehicles obviously have an effect on the saturation flow, as they must slow down before the turn is made. The saturation flow rate from the field study site that was used to calibrate the simulation model stop line was found to be 1,850 vehicles per hour. The right-turning percentage at the test site stop line is never greater than 6 per cent and should therefore have only a minor effect on
vehicles traveling straight ahead. Studies at other more congested junctions reveal saturation flows in excess of 2,000 vehicles per hour during peak hour.

5.7 SIMULATION MODEL OUTPUT DATA
The output data from VISSIM can be configured to meet the requirements of the user. For the purposes of this study it was necessary to output two special files. The first file contained the exact signal status per update cycle in order to calculate green times, amount of past-end green and green extension etc. The second was a raw data file containing the speed and link position of each vehicle per update cycle and also details regarding the vehicle ahead and relative speed difference. This information was necessary in order to extract useful information concerning dilemma zone behavior, number of stops on amber etc. The second raw data file consisted of some 2.7 million rows of data.

5.7.1 Performance Data
There are no general norms for the evaluation of traffic performance at signal-controlled intersections. The most interesting and useful measures for the purposes of this study are listed below under suitable categorical headings:

- Network performance
  - Number of vehicles passing through the intersection,
  - Total distance traveled by all vehicles,
  - Total travel time for all vehicles,
  - Average network speed;
- Signal system functionality in relation to all signal groups
  - Number of signal cycles during test period,
  - Average green time / red times per signal group and cycle;
- Signal system functionality in relation to the studied signal group
  - Total green time for signal group,
  - Amount of green extension time,
  - Amount of excessive (wasted) green extension time,
  - Amount of past-end green time,
  - Amount of excessive (wasted) past-end green time,
  - Number of stop/go vehicles given past-end green (as percentage);
- Queue lengths from studied signal group stop line (average and maximum);
- Delay time from a position 130 meters from the stop line and 20 meters past in the straight-ahead and right-turn directions (average per vehicle). Also, the average number of stops per vehicle;
- Dilemma zone behavior
  - Number of stop/go vehicles on change to amber (relative percentages).

5.7.2 Safety Data
While there are no general norms for the evaluation of traffic safety there are many recognized parameters that can be used to evaluate performance. For the purposes of safety evaluation, there are very few recognized measures in general.

A common and useful measure of safety is the number of red-light violations. In fact, it is quite useful to know the exact signal status of the last vehicle passing the stop line before the end of the cycle actually received. This will be included as a suitable measure for the purposes
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of this study. Given that the “O” function also tries to prevent rear-end conflicts and collisions it is useful to have a measure of the number and seriousness of such conflicts if they should occur during the simulation.

For this purpose, a useful measure has been proposed by Minderhoud and Bovy (2001). They suggest two measures based on the original Time-to-Collision (TTC) notion introduced by Hayward in the early 1970s. Briefly, TTC is “…the time that remains until a collision between two vehicles would have occurred if the collision course and speed difference are maintained.” The TTC concept has been applied beneficially as a proximal safety indicator (i.e. has a strong correlation to the number of accidents and their outcome severity) in many safety analyses. The first measure suggested is the Time Exposed TTC which measures the time duration of exposition to safety critical TTC-values over a predefined threshold. This measure is summed for all vehicles passing a selected stretch of road during a designated time period.

The second measure is Time Integrated TTC, which uses the integral of the time to collision profile in order to express the level of safety. This measure is more qualitative, representing the area of the TTC profile under the stated threshold thereby providing information as to the seriousness of a conflict situation. This measure is also summed for all vehicles passing a selected stretch of road during a designated time period. The revised TTC values proposed by Minderhoud and Bovy are illustrated below in figure 5.10. Interestingly, these TTC values are recommended for use with simulation studies.

Figure 5.10 Graphical representation of the Time Exposed and Time Integrated TTC safety Indicators (Minderhoud and Bovy, 2001)

The choice of safety measures for the purposes of this study are thus the following:

- Number of red-light violations and last passages on past-end green, green extension and amber in relation to the studied signal group;
- TTC data for vehicles in dilemma zone at signal change to amber and for those vehicles receiving past end green
  - Number of TTC occurrences in relation to the studied signal group approach,
  - Total Time Exposed TTC in relation to the studied signal group approach,
  - Time Integrated TTC in relation to the studied signal group approach.
5.8 ANALYSIS OF SIMULATION OUTPUT DATA

The output data from many micro-simulation software packages can be configured to meet the requirements of the user. However, if the output required by the user is not available directly in the desired format, it is usually possible to extract raw data from a simulation run and thereafter to perform post-processing via statistical software packages or specially developed software.

For the purposes of this study, it was necessary to generate two specially formatted files from VISSIM and to post-process the content of these files in order to extract useful information relating to aspects of traffic performance and safety. The first file contained the exact status of each signal group per update cycle (0.10 seconds). This was necessary in order to calculate e.g. green time, and the amount of past-end green and given green extension time. The second file contained raw data including the speed and position of each vehicle per update cycle and details regarding any preceding vehicle and the relative speed difference.

Owing to the overwhelming quantity of data, a special analysis program ("VISSIM O-Function Analysis") was written by Archer in order to generate the necessary traffic performance and safety results consistently. This program also provides the option of running batch jobs to process and assimilate results for the data files for each different simulation scenario.

The information in both files was used to generate a result file containing the following information for a particular signal group:

- A signal status event list showing the start and end time for each status shown during all signal cycles;
- Amount of unused green extension time and past-end green time given to vehicles;
- Signal status seen by last vehicle to pass the stop line before the end of the cycle;
- Saturation flow and discharge rates;
- Behavior of vehicles that faced a signal change to amber while in the dilemma zone and for those vehicles that received past-end green including:
  - speed and distance to stop line,
  - stop or go behavior,
  - average and maximum braking rates for stopping vehicles,
  - actual signal status for continuing vehicles,
  - preceding vehicle details (if any),
  - Time Exposed and Time Integrated TTC,
  - minimum distance to a preceding vehicle;
- Number/proportions of Stop/Go decisions on change to amber and on past-end green;
- Number of red-passes for vehicles receiving past-end green and on past-end green;
- Number of TTC-cases for vehicles receiving past-end green and on past-end green;
- Time Exposed TTC for vehicles receiving past-end green and on past-end green;
- Time Integrated TTC for vehicles receiving past-end green and on past-end green.

5.9 SIMULATION EXPERIMENT

The simulation experiment considered four different scenarios, two different detector positions and two different signal controller logic programs for the “O” function. The detector positions are:

(a) 130 and 80 meters (standard recommendation);
(b) 110 and 65 meters (suggested).
The different signal controller logic programs are:

(i) Standard “O” function implementation (i.e. the norm);
(ii) Speed dependent “O” function implementation with a minimum speed threshold of 56.5 km/h, below which past-end green will not be applied (suggested).

The simulation experiment is run for each of the four scenarios for the same 4-hour period representing the period between 14:00 and 18:00. Using the same random seed number for each scenario implies that exactly the same vehicle arrivals will be generated on the intersection approaches. The results presented here represent the values for one particular random seed for each of the four scenarios. As an assurance that the results are representative of actual conditions and that no unwanted random effects were included, a further four runs were made for each scenario using a specific set of random seed numbers.

The results of these simulation runs were found to be compliant with the set of results presented below. The results for each of the five runs for each scenario have not been added or averaged due to the random variation caused by the overly sensitive activation of the incident reduction function given the short time period for the simulation. A perhaps fairer and more robust method would have been to test each of the four scenarios with one different random seed number over a simulation period corresponding to a much longer test period (e.g. 50-100 hours). This method will be applied in the following studies. However, for the purposes of this study, the results are considered sufficiently representative.

5.10 RESULTS
The results are presented for the network performance and then traffic performance and safety.

5.10.1 Network Performance
The results for overall network performance are summarized in Table 5.1 below. The results show no obvious effects on performance for the different conditions in each of the four simulation scenarios.

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Standard “O” Function</strong></td>
<td><strong>Speed Depd. “O” Function</strong></td>
</tr>
<tr>
<td>Total no. of vehicles / (St Dev)</td>
<td>6660 (22)</td>
</tr>
<tr>
<td>Total dist traveled km / (St Dev)</td>
<td>2923 (16)</td>
</tr>
<tr>
<td>Total travel time h / (St Dev)</td>
<td>84.7 (4.3)</td>
</tr>
<tr>
<td>Av network sp km/h / (St Dev)</td>
<td>34.5 (2.3)</td>
</tr>
</tbody>
</table>

5.10.2 Traffic Performance
The results of the traffic performance are presented in the following sections:

5.10.1.2 Cycle and Green Times
As expected, the number of signal cycles was found to increase when its detector positions were at 110 and 65 meters instead of 130 and 80 meters, as the average cycle time decreased
by 5.8% (from 46.9 to 44.2 s), while the average green time decreased by only 2.3% (from 34.6 to 33.8 s). This results in a positive effect by increasing the proportion of the green time to the cycle time by 3.7% (from 73.8 to 76.5%). Using the speed dependent “O” function has slight effects on the average cycle times. Table 5.2 shows the results of cycle and green times for the intersection in the four simulation scenarios.

Table 5.2 Cycle and green times for the intersection in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of cycles / (St Dev)</td>
<td>307 (6.1)</td>
</tr>
<tr>
<td>Av cycle time s / (St Dev)</td>
<td>46.9 (1.0)</td>
</tr>
<tr>
<td>Av green time s / (St Dev)</td>
<td>34.6 (0.8)</td>
</tr>
<tr>
<td>Av green / av cycle %</td>
<td>73.8</td>
</tr>
</tbody>
</table>

5.10.1.2 Signal System Functionality (all groups)

The results for signal system functionality related to each signal group are also summarized in Table 5.3. The results for the green times were found to decrease when its detector positions were at 110 and 65 meters instead of 130 and 80 meters for all the signal groups. This can be attributed to the slightly earlier starting of the green extensions with a detector that is further from the stop line compared to a shorter distance (20 meters difference). Using the speed dependent “O” function has different effects on the average green times.

Table 5.3 Signal green time for all groups in the four simulation scenarios

<table>
<thead>
<tr>
<th>Signal group</th>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21.9</td>
<td>21.3</td>
</tr>
<tr>
<td>2</td>
<td>7.5</td>
<td>7.4</td>
</tr>
<tr>
<td>3</td>
<td>5.6</td>
<td>5.4</td>
</tr>
<tr>
<td>4</td>
<td>21.5</td>
<td>21.8</td>
</tr>
<tr>
<td>5</td>
<td>8.2</td>
<td>8.5</td>
</tr>
<tr>
<td>6</td>
<td>5.5</td>
<td>5.4</td>
</tr>
<tr>
<td>7</td>
<td>6.5</td>
<td>6.6</td>
</tr>
</tbody>
</table>

5.10.1.2 Signal System Functionality (studied signal group)

The results for signal system functionality related to the studied signal group (no. 1) are summarized in Table 5.4 below. The results here clearly show the effect on the amount of green time given of positioning the detectors at 110 and 65 meters instead of 130 and 80 meters. The data in the table clearly indicates that the total amount of past-end green issued was halved through the use of speed dependency. Also, the amount of excess (i.e. wasted) past-end green time was substantially reduced, implying a clear time enhancement in terms of traffic performance.

The speed dependent “O” function also appears to have reduced the relative percentage of stops in the dilemma zone when past-end green had been issued. This is more apparent at the more distant detector positions (130 and 80 meters) where no stops are recorded.
**Table 5.4** Signal system functionality data for the studied signal group in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total min. green time s / (St Dev)</td>
<td>6983 (47.2)</td>
</tr>
<tr>
<td>Total green ext time s / (St Dev)</td>
<td>992 (27.4)</td>
</tr>
<tr>
<td>Excessive gr ext time s / (St Dev)</td>
<td>87.6 (19.7)</td>
</tr>
<tr>
<td>Total PEG time s / (St Dev)</td>
<td>419.0 (24.0)</td>
</tr>
<tr>
<td>Excessive PEG time / (St Dev)</td>
<td>50.4 (14.9)</td>
</tr>
<tr>
<td>Stop percent on PEG</td>
<td>13.0</td>
</tr>
<tr>
<td>Go percent on PEG</td>
<td>87.0</td>
</tr>
</tbody>
</table>

**5.10.1.2 Delay Times**

The results for the average delay times, shown in Figure 5.5, increased with the speed dependent “O” function in both detector positions against the scenario with standard “O” function. The differences in number of stops per vehicle show a similar trend.

**Table 5.5** Delay time data for the studied signal group in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Av delay time per veh / (St Dev)</td>
<td>8.50 (0.52)</td>
</tr>
<tr>
<td>No of stops per veh / (St Dev)</td>
<td>0.31 (0.02)</td>
</tr>
</tbody>
</table>

**5.10.1.2 Dilemma Zone Behavior**

One of the most interesting results concerns the stop and go decision-making of drivers in the dilemma zone when a change to amber occurs. With the 110 and 65 meters detector positions, there was a clear, though marginal, increase in the percentage of vehicles that stopped. This is to be expected since the implementation of the “O” function prevents many of these cases from occurring in the first place. The overall drop in the number of cases that occurred was affected differently for the different detector positions when the “O” function was made speed dependent. Interestingly, there was a far greater reduction in the number of dilemma zone cases when the “O” function was used in the 110 and 65 meters detector positions’ than the more distant detector positions. This suggests that the theoretical dilemma zone was better matched to the practical detector positions (i.e. suits the roadway conditions). Table 5.6 shows the dilemma zone behavior data for the studied signal group in the four simulation scenarios.

**Table 5.6** Dilemma zone behavior data for the studied signal group in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of Cases / (St Dev)</td>
<td>95 (8.6)</td>
</tr>
<tr>
<td>Stop% on amber in DZ</td>
<td>61.05</td>
</tr>
<tr>
<td>Go % on amber in DZ</td>
<td>38.95</td>
</tr>
</tbody>
</table>
5.10.3 Traffic Safety

The results as regards traffic safety are presented below.

5.10.1.3 Number of Red-Light Violations and Last Passages in Cycle on Green and amber

A close examination of the last vehicle passes on each cycle and the signal that was actually displayed reveals some interesting results. The 110 and 65 meter detector positions showed significantly fewer last passes on amber and red (i.e. violations) and more on green (green extension or past-end green) than the more distant detector positions. Within the two detector positions, the effect of the speed dependent “O” function appeared to have little significance. However, a tendency to reduce red-light violations occurs with the speed dependent “O” for the 110 and 65 meter detector positions. Table 5.7 shows the red-light violations and last passages in cycle data for the studied signal group in the four simulation scenarios.

Table 5.7 Red-light violations and last passages in cycle data for the studied signal group in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Last passes on gr ext / (St Dev)</td>
<td>71 (11.2)</td>
</tr>
<tr>
<td>Last passes on PEG / (St Dev)</td>
<td>15 (6.6)</td>
</tr>
<tr>
<td>Last passes on amber / (St Dev)</td>
<td>125 (8.0)</td>
</tr>
<tr>
<td>Red-light violations / (St Dev)</td>
<td>9.7 (2.9)</td>
</tr>
</tbody>
</table>

5.10.1.3 Time-to-Collision Data for Vehicles in Dilemma Zone at Signal Change to Amber

A look at the Time-to-Collision data reveals a smaller number of TTC occurrences and a reduced amount of Time Exposed TTC for vehicles in the dilemma zone with the standard “O” function when the detectors are at 110 and 65 meters. At these detector distances, speed dependency appeared to make little difference on the number of occurrences and gave a slight improvement in Time Exposed TTC, but an increase in Time Integrated TTC. This suggests that the conflicts with speed dependency might be of a more serious nature. Interestingly, the speed dependent “O” function with the longer detector distances showed higher safety by the deterioration in all the studied factors. Table 5.8 shows the time-to-collision data for vehicles in the dilemma zone at signal change to amber in the four simulation scenarios.

Table 5.8 Time-to-Collision data for vehicles in dilemma zone at signal change to amber in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of TTC occurrence / (St Dev)</td>
<td>8.2 (0.7)</td>
</tr>
<tr>
<td>Time Exposed TTC / (St Dev)</td>
<td>18.1 (1.9)</td>
</tr>
<tr>
<td>Time Integrated TTC / (St Dev)</td>
<td>9.8 (1.0)</td>
</tr>
</tbody>
</table>
5.10.1.3 Time-to-Collision Data for Vehicles Receiving "O" function Past-end Green

The final and perhaps most interesting table shows what occurred with the vehicles that received past-end green as a result of the “O” function when they were in the dilemma zone when a change to amber occurred. The results are illustrative of the safety effect of this function and show that of the total cases (presented in Table 5.9) very few occur when the “O” function was actually in operation. Also, the Time Integrated TTC showed that the seriousness of the conflict situations was far less acute that in other rear-end conflict occurrences in the dilemma zone. Even more obvious is the finding that all conflict situations were avoided (i.e. 0 occurrences) if the “O” function was made speed dependent whatever the detector distances were.

Table 5.9 Time-to-Collision data for vehicles receiving "O" function past-end green for the studied signal group in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions</th>
<th>130 and 80 meters</th>
<th>110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of TTC occurrence / (St Dev)</td>
<td>1.2 (0.3)</td>
<td>0</td>
</tr>
<tr>
<td>Time Exposed TTC / (St Dev)</td>
<td>1.7 (0.2)</td>
<td>0.00</td>
</tr>
<tr>
<td>Time Integrated TTC / (St Dev)</td>
<td>1.3 (0.2)</td>
<td>0.00</td>
</tr>
</tbody>
</table>
6 SYNTHESIS AND CONCLUSIONS

The study combined empirical methods with simulation to analyze and improve the performance of the “O” function in the LHOVRA technique.

6.1 FIELD STUDY

The field study has shown that the actual dilemma zone varies considerably, and thereby the effectiveness of the “O” function, depending on the actual speed distribution and detector location in the approaches.

Field studies at three intersections with a 50 km/h speed limit showed that vehicles in the lower speed range when facing change from green to amber had a higher likelihood to stop before the stop line. These values concern all the three intersections, which all had large variation in vehicle approach speeds. The mean speeds of all the vehicles in the approaches show that there is a large variation between the intersections. This resulted in a large variation in parity and length of dilemma zones (beginning at 70 to 22 m) from the stop line.

The "O" function consists of an extra green extension (past-end green) after the end of maximum green time. This extension allows the vehicles in the dilemma zone to be evacuated without any stress to make the decision to “pass or brake”. While these vehicles evacuate, others may arrive and, depending on the time gap, extend the past-end green (PEG) further. In cases where traffic is intense, the gap between the vehicles becomes smaller. Then the PEG does not perform as it is supposed to do, i.e. evacuate vehicles in the dilemma zone, but just a normal green extension. Therefore, it was found to be advisable to integrate the PEG in the maximum green time, in order to reduce delay without impairing safety.

Field observations showed that speeds in some approaches were low, thereby making the functionality of PEG doubtful. Looking at the number of stops after receiving the green extension, it was clear that the low speed approaches had a higher share of stopped vehicles. The detectors’ locations had been designed depending on the actual approach speeds, but this was not enough to fit the real dilemma zone and thereby optimize the performance of using the “O” function.

6.2 SUGGESTIONS FOR IMPROVEMENT

The empirical study has shown that results of using the “O” function were different depending on the real speed in the approach and the detector location. A theoretical study of the “O” function was done with a focus on the approaches with a 70 km/h posted speed.

The LHOVRA technique manual, depending on many factors, suggests that the practical dilemma zone for the 70-km/h approaches is between 130 and 50 meters before the stop line. This function is performed with detectors placed at 130 and 80 meters from the stop line to detect and follow the vehicle within the dilemma zone.

Extension of green time by past-end-green is triggered by the existence of a vehicle in the zone. Preferably, however, a change should not occur when this vehicle is within the dilemma zone as long as the maximum past-end-green is not exceeded. This is not only because of the
risk of rear-end collisions, but also to reduce change-induced stress in the approach and to reduce the possibility of a red light infringement. These objectives can be achieved by other functions in LHOVRA technique, and using the “O” function for many objectives weakens the argument for the main condition of PEG.

The LHOVRA manual concurs with the fact that the function is not always useful. It is suggested that postponement of the change from green to amber when there is a vehicle in the dilemma zone, should not take place other than during a reasonable time-period. The function will only be of use as long as the approach is not operating near saturation flow, thereby reaching the limit for permitted maximum green display.

The position of the two detectors in the approach (at 130 and 80 meters) is dimensioned to allow vehicles traveling at 70 km/h to proceed through the intersection. Vehicles moving at lower speeds, which get determined extension, may find themselves in their own dilemma zone at the start of amber, while they could easily have stopped before the stop line if no extension had occurred. To ensure safety without compromising traffic performance, different aspects of the function can be improved in many ways by:

- Changing the Detector Location and Green Time Extension: The theoretical dilemma zone that starts at a distance of 110 m instead of 130 m is more suitable. So detectors should be placed at 110 m and 65 m. This solution may move the risky speeds to lower ones, which requires adjusting the green extensions;

- Using Double Detector at Beginning of the Dilemma Zone: Detecting the vehicle at entry to the dilemma zone, by a double detector to measure the speed of the vehicle, gives the possibility to make the order of “past-end-green” conditional and thereby excluding the use of the function when it is not needed;

- Considering the Headway Between Two Existing Vehicles in the Dilemma Zone: If the detected vehicle has a speed of 70 km/h or higher, the order of “past-end green” must depend on the existence of preceding vehicle with a lower speed in the zone.

6.3 SIMULATION

Table 5.10 below shows a summary of the traffic performance and safety results in the four simulation scenarios. It is clear that the results from the two hypothesized (suggested) improvements have revealed improvements in safety with little or no loss to traffic performance in comparison to the existing incident reduction function. These improvements are:

- A distance closer to the stop line (110 and 65 meters) has positive effects on safety in form of a reduction in red-light violations and measures of Time to Collision (TTC) and positive effects on performance in form of the proportion of the green time to the cycle time;

- A speed dependent “O” function with the greater distance to the stop line (130 and 80 meters) reduces wasted green time and the number of stops after receiving PEG;
• A speed dependent “O” function with the closer distance to the stop line (110 and 65 meters) reduces red-light violations and the number of conflicts;

• Furthermore, it has been shown that a speed dependent “O” function may have the ability to eliminate conflicting situations completely if past-end green is only given to those vehicles that really need it.

But it is still the standard “O” function, which has the lowest average delay time. However, the results show that there is no scenario that is better in all aspects, see Table 5.10.

Table 5.10 Summary of the traffic performance and safety results in the four simulation scenarios

<table>
<thead>
<tr>
<th>Detector Positions 130 and 80 meters</th>
<th>Detector Positions 110 and 65 meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Av cycle time s</td>
<td>46.9</td>
</tr>
<tr>
<td>Av green time s</td>
<td>34.6</td>
</tr>
<tr>
<td>Green t / Cycle t %</td>
<td>73.8</td>
</tr>
<tr>
<td>Total PEG time</td>
<td>419</td>
</tr>
<tr>
<td>Excessive PEG time s</td>
<td>50.4</td>
</tr>
<tr>
<td>Excessive PEG time %</td>
<td>12.0</td>
</tr>
<tr>
<td>Av delay time s per v</td>
<td>8.5</td>
</tr>
<tr>
<td>Stop percent on PEG</td>
<td>13.0</td>
</tr>
<tr>
<td>Go percent on PEG</td>
<td>87.0</td>
</tr>
<tr>
<td>No of TTC occurrence</td>
<td>1.2</td>
</tr>
<tr>
<td>Time Exposed TTC</td>
<td>1.7</td>
</tr>
<tr>
<td>Time Integrated TTC</td>
<td>1.3</td>
</tr>
<tr>
<td>Red-light violations</td>
<td>9.7</td>
</tr>
</tbody>
</table>

There are reservations to how far the results of this study can be generalized to real-world scenarios given the fact that this experiment is based on simulation. Work will now proceed to test other models based on newly collected field data using this methodology in order to test their robustness.
Part III

Impacts of Fixed Time Coordination with Local Signal Timing Adjustment
ABSTRACT

Urban traffic signal control systems in Sweden are often coordinated with fixed time plan selection that aim to produce “green waves”. Some municipalities use the offline TRANSYT software to optimize signal timing of such systems. In Stockholm, signal timing is normally performed manually taking into consideration local adjustments based on detector inputs controlling the termination of the green signal by using past-end green (PEG).

To evaluate local traffic actuated signal-timing adjustments the following control strategies were studied:
- Fixed time coordination without local signal timing adjustment (FTC);
- Fixed time coordination with local signal timing adjustment (FTC-LTA).

The TRANSYT model was used purely to generate optimized signal timings for input to micro-simulation using the HUTSIM software. The following delay results were obtained:

In the main intersection
- Local traffic adjustment with manual FTC increased total delay by 9%;
- Signal timings determined using TRANSYT reduced the average intersection delay by 11% compared to manual signal settings. Local traffic adjustment had little effect (reduced total delay by a further 1%).

In the studied area
- Local traffic adjustment with manual FTC had little effect (reduced delay by 1%);
- Signal timings determined using TRANSYT reduced the average intersection delay by 9% compared to manual signal settings. Local traffic adjustment reduced total delay by a further 5%.

A comparison of results from simulation and field measurements (part IV) for the main intersection with the manual FTC with LTA under non-peak traffic shows that HUTSIM simulation results of stops and delay were about 14% higher than the corresponding field results. It should be noted that HUTSIM does not totally reflect the actual on-street situation. In reality, parked vehicles, bicycles and pedestrians often reduce traffic performance. However, since all cases have been assessed using HUTSIM, all the results are comparable.
Part III: Impacts of Fixed Time Coordination with Local Signal Timing Adjustment

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1 INTRODUCTION

1.1 BACKGROUND

In an urban street network, arrival rates are often influenced by the queue discharge from upstream traffic signals, which create vehicle platoons moving along the approach links. If adjacent signalized intersections are coordinated in such a way that they operate with the same cycle time and with constant split and offset, it is possible to set these signal timing parameters on a one-way street in such a way that the platoon from the upstream intersection will arrive at the downstream stop line when this signal is green (called a “green wave”). The shorter the distance between the signalized intersections, the less dispersed are these platoons when they arrive at the stop line of the downstream traffic signal.

During the night, coordination is usually broken down into smaller groups of linked intersections or into isolated control. Based on the knowledge of the traffic patterns, sometimes an estimated O/D-matrix, and some fresh traffic data, 3-5 fixed reference plans are produced by hand or using some rudimentary computer-aided systems. However, the difficulties of designing efficient coordination increase rapidly when the number of traffic signals increases and when the coordination involves networks. Also, the problem of creating "green waves" when there is only limited spare capacity in the system is hard to solve manually.

The traffic flows used when producing the time plans for coordinated control will change over time making these time plans less well suited for the traffic situation for which they were originally designed. These errors will cause a gradual deterioration of the system’s efficiency. Road authorities often fail to maintain and update the signal settings, and it is not unusual for the same coordination to be still in operation 10-15 years after its design. The consequences for the road users in terms of delays and stops are significant. Other negative effects include higher fuel consumption and more emissions, as well as growing irritation on the part of drivers.

In Sweden urban traffic signal control systems are normally coordinated with fixed time plan selection that aims to produce “green waves”, reducing the number of stops along signalized routes, although urban areas are seldom like that in reality because of the congestion that dominates most of the day. Some municipalities use the offline TRANSYT software for this purpose (Vincent, et al, 1980). In Stockholm, the timing of the signals is normally performed manually taking into consideration local signal timing adjustment based on detector inputs controlling the termination of the green signal by using past-end green (PEG), which affects the split of green time, but not the cycle time, nor the offset between the intersections (Al-Mudhaffar & Cunningham, 2001). PEG may come in one or more directions in both main and cross streets. The effects of PEG have received little attention. One of the purposes of this work is therefore to reduce the knowledge gap as regards its effects on capacity and delay in congested urban networks.

1.2 OBJECTIVES OF THIS PART

The objective of this part of the thesis is to evaluate:
• Fixed time coordination without local signal timing adjustment (FTC);
• Fixed time coordination with local signal timing adjustment (FTC-LTA).

1.3 LIMITATION
The limitation in this part of the thesis is that the study does not include impacts on pedestrians.

1.4 METHODOLOGY
Traffic data from coordinated signalized intersections in an arterial street network in Stockholm were used as a basis for this study. The following steps were included in the research methodology:

• Data collection regarding geometrical layout, signal phasing, existing signal timing, traffic flow, cruise speed and other required input data for TRANSYT;
• Calculation of optimized signal timing for Fixed Time Co-ordination by TRANSYT;
• Evaluation of the traffic performance for different time settings and strategies using the HUTSIM simulation model.

1.5 STUDY AREA
The Kungsholmen area in Stockholm provides a good range of different street layouts, traffic conditions and public transport services typical to Swedish traffic situations. The study area includes the four most important signalized intersections in the selected site at Kungsholmen. These intersections are:

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fleminggatan - Pipersgatan</td>
<td>3321</td>
</tr>
<tr>
<td>Fleminggatan - Scheelegatan</td>
<td>3334</td>
</tr>
<tr>
<td>Fleminggatan - Carl Gustav Lin.</td>
<td>3338</td>
</tr>
<tr>
<td>Scheelegatan - Kungsgatan</td>
<td>3313</td>
</tr>
</tbody>
</table>

The Fleminggatan - Scheelegatan intersection (number 2 on the map in Figure 1) is located in the heart of this area and is the main intersection and has been specially studied. The predominant traffic flow is from west to east, particularly during the morning peak, as traffic approaches the city from one of Stockholm’s major motorways. All roads in the study area are undivided two-way streets with a speed limit of 50 km/h and flaring at some intersection approaches.
Part III: Impacts of Fixed Time Coordination with Local Signal Timing Adjustment

1.5.1 Traffic Flow
Approximately 35,000 vehicles a day pass through the most loaded intersection in the study area and about 21,000 vehicles pass through the main street Fleminggatan. During morning and evening peak hours the total volume of traffic (for each time period) is around 2,500 and 1,500 vehicles respectively.

1.5.2 Signal Control
Depending on the traffic flow, four principal timing plans for coordination operate over the course of the day:

1. From 07.00 to 10.00 the signals operate an 82 second cycle time;
2. From 10.00 to 14.00 the signals operate a 67 second cycle time;
3. From 14.00 to 18.30 the signals operate an 82 second cycle time;
4. From 18.30 to 22.00 the signals operate a 56 second cycle time.

From 22.00 to 07.00, each intersection reverts to isolated LHOVRA control.

As a result of the high traffic flows, almost all the available past-end green time is used and therefore, in practice, the effects of past-end green cannot be seen. However, during the midday off-peak hours 10.00 to 14.00 and at other off-peak times, the effect of using of past-end-green will be more obvious.

Figure 3.1 Study area in Kungsholmen
2 FIXED TIME COORDINATION WITH LOCAL TRAFFIC ADJUSTMENT

The coordinated traffic signal system in the Kungsholmen test area operated with four signal plans with clock-based plan selection. The fixed time coordination (FTC) had been developed manually, based on capacity analysis and time-distance diagrams. Within this framework, the technique using local signal timing adjustments was applied based on detector inputs controlling the termination of the green signal by using “Past-end green (PEG)” to try to reduce red light driving and the risk of rear-end collisions and to discharge platoons.

2.1 FIXED TIME COORDINATION (FTC)

All signal timings at Kungsholmen were produced by an experienced traffic signal engineer. Coordination between signals is based upon manually calculated time-and-distance diagrams, and no prior computerized optimization process took place. The establishment of effective green waves through several intersections was the primary objective of the current programming, see Figures 2.1 and 2.2.

Figure 2.1 Green waves in a manual time setting for both streets at the main intersection. The figure shows the 80 s cycle time that was used before it was adjusted to 82 s. (Björck, 2000)
2.2 INCIDENT REDUCTION “O” FUNCTION

The “O” or incident reduction function is a part of the LHOVRA strategy, which involves the use of past-end-green (PEG). The incident reduction function is designed to reduce the number of vehicles in the dilemma zone and thereby reduce the number of red light drivers and rear-end collisions. This is achieved by detecting vehicles at the beginning of the dilemma zone and postponing a decided change to yellow (SNRA, 1991).

The most difficult aspect in “O” function application is to determine the practical dilemma zone for the particular type of approach. It is important that the “O” function interval timing allows the last extending vehicle to clear the dilemma zone safely before the light begins to change, see Figure 2.3.

Figure 2.2 Green waves in a manual time setting with shorter cycle time (Björck, 2000)
Since its initial success, LHOVRA has been widely used in Sweden and the other Scandinavian countries in urban areas at a lower speed (50 km/h). More recently, the “O” function has also been adapted to allow vehicle-actuated past-end-green in fixed time coordinated systems, the intention being to improve traffic performance as well as increase safety.

In the original signal timing plans the approaches obtain the maximum allowable green time. Part of this time (up to 12 seconds) is considered as a PEG, which can be used to extend the green light. PEG affects the split of green time, but not the cycle time, nor the offset between the intersections. PEG may come in one or more directions in both main and cross streets. The examples in Figures 2.4 and 2.5 explain the use of the function within the fixed time coordination.
Figure 2.4 Traffic signal time setting in the main intersection with PEG in both streets (Björck, 2000)

Figure 2.5 Traffic signal time setting in secondary intersection with PEG in the main street (Björck, 2000)
3 CALCULATION OF OPTIMIZED SIGNAL TIMING USING TRANSYT

The aim here was to optimize signal timing for the local signal timing adjustment within the framework of the fixed-time coordinated signal control (FTC-LTA). The methodology used for this purpose was the TRANSYT (Vincent, 1980) model as described below.

3.1 THE TRANSYT MODEL

The software consists of a traffic model and an optimizer. The traffic model includes a detailed description of all links and nodes on a lane basis allowing calculation of delays (and stops) within the network for a given set of signal timings and traffic flow characteristics. The optimizer is relatively simple and uses the traffic model and hill-climbing techniques to calculate an economically based “Performance index” to search for an optimum solution regarding signal timings (cycle time, offset, and split), see Figure 3.1.

![Figure 3.1 TRANSYT traffic model and optimizer (TRL, 2002)](image)
The use of TRANSYT is part of a larger process that involves collecting data to input into the model, and output, which allows reviewing the results before implementing the signal timings on street.

Signal control terms (such as stage and inter-green) do not have exactly the same definitions as in the Swedish models. It is therefore very important to study the definitions of these terms in TRANSYT and find their interfaces to Swedish ones. Some of the more important definitions (based on TRANSYT 11 WORKSHOP, 2002) are listed below:

3.1.1 Model with Links and Nodes
The network is made up of nodes (intersections) and links (road between intersections). What one needs to do is to model these links and nodes to deduce the operational performance of the network under given traffic signal settings.

Links model streams of traffic and the user has to provide the link characteristics. The link may include many lanes. The required details are shown in Figure 3.2.

Nodes are used to describe intersection details to TRANSYT and the information required consists of the signal timing details including:

- Stage minimums;
- Inter-stage times.

3.1.2 Stage Change Time
The time of a change of stage (stage change time) is a time within the cycle at which the green signal on one stage is terminated and the change to the next stage green period is initiated.

A TRANSYT minimum stage green is defined as the common green time between start and termination of greens in that stage. In TRANSYT node data, the user is required to enter the minimum possible stage length, “minimum green + inter-green between the current stage and the previous stage, as explained in Figure 3.3.
3.2 INPUT DATA

The following types of required field data were collected as inputs for the TRANSYT network:

- Traffic flows and turning movements;
- Road layout – lane designation, flare lengths and distance between intersections (stop line to stop line);
- Traffic signal timings – inter-greens and minimum greens;
- Traffic signal stage sequence;
- Free flow cruise speeds (km/h);
- Stop line saturation flows.
Some of these data, which related to the traffic signal part of TRANSYT, are presented in Figure 3.4 and Tables 3.1, 3.2, and 3.3.

Figure 3.4 Node and link numbering at the Kungsholmen intersections
Table 3.1 Present traffic light time-setting, midday

<table>
<thead>
<tr>
<th>Crossing</th>
<th>Cycle time [s]</th>
<th>Red time [s]</th>
<th>Green time [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crossing 1</td>
<td>67</td>
<td>30</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td></td>
<td>38</td>
<td>20</td>
</tr>
<tr>
<td>Crossing 2</td>
<td>67</td>
<td>38</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td></td>
<td>29</td>
<td>31</td>
</tr>
<tr>
<td>Crossing 3</td>
<td>67</td>
<td>24</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50</td>
<td>12</td>
</tr>
<tr>
<td>Crossing 4</td>
<td>Vehicle actuated</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Crossing 5</td>
<td>67</td>
<td>20</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>48</td>
<td>14</td>
</tr>
</tbody>
</table>

Direction and sense of traffic flow

1. Only red time, without amber-red
2. Only green time, without amber

Table 3.2 Stage change time for each node

<table>
<thead>
<tr>
<th>Node</th>
<th>Stage 1 (s)</th>
<th>Stage 1 (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (3334)</td>
<td>7</td>
<td>44</td>
</tr>
<tr>
<td>2 (3313)</td>
<td>8</td>
<td>32</td>
</tr>
<tr>
<td>3 (3338)</td>
<td>4</td>
<td>50</td>
</tr>
<tr>
<td>4 (3340)</td>
<td>VA</td>
<td>VA</td>
</tr>
<tr>
<td>5 (3321)</td>
<td>9</td>
<td>52</td>
</tr>
</tbody>
</table>
Table 3.3 Start lag for each node

<table>
<thead>
<tr>
<th>Node</th>
<th>Stage 1 (s)</th>
<th>Stage 1 (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (3321)</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>2 (3334)</td>
<td>7</td>
<td>6</td>
</tr>
<tr>
<td>3 (3338)</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>4 (3313)</td>
<td>8</td>
<td>6</td>
</tr>
</tbody>
</table>

The CAPCAL program (VV, 1981) was used to calculate the saturation flow. The results are shown in Table 3.4.

Table 3.4 Saturation flow for the TRANSYT links

<table>
<thead>
<tr>
<th>Link</th>
<th>Saturation flow (f/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>2448</td>
</tr>
<tr>
<td>12</td>
<td>3366</td>
</tr>
<tr>
<td>13</td>
<td>2142</td>
</tr>
<tr>
<td>14</td>
<td>1530</td>
</tr>
<tr>
<td>21</td>
<td>1530</td>
</tr>
<tr>
<td>22</td>
<td>2142</td>
</tr>
<tr>
<td>23</td>
<td>2448</td>
</tr>
<tr>
<td>24</td>
<td>1530</td>
</tr>
<tr>
<td>31</td>
<td>2142</td>
</tr>
<tr>
<td>32</td>
<td>1530</td>
</tr>
<tr>
<td>33</td>
<td>1836</td>
</tr>
<tr>
<td>51</td>
<td>1836</td>
</tr>
<tr>
<td>52</td>
<td>1530</td>
</tr>
<tr>
<td>53</td>
<td>1530</td>
</tr>
</tbody>
</table>

3.3 RESULTS

The TRANSYT model was used purely to generate optimized signal timings for input to micro-simulation using the HUTSIM software. The same cycle time of 67 seconds, as in the existing manually produced signal plans, was used. Optimization was thus only performed for green split times and offsets. Table 3.5 summarizes the results.
Table 3.5 Stage change times and green times for nodes in FTC and TRANSYT optimizing

<table>
<thead>
<tr>
<th>Node no</th>
<th>Number of stages</th>
<th>FTC</th>
<th>TRANSYT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stage change time</td>
<td>Stage 1</td>
<td>Stage 2</td>
</tr>
<tr>
<td>1 (3321)</td>
<td>2</td>
<td>9</td>
<td>52</td>
</tr>
<tr>
<td>2 (3334)</td>
<td>2</td>
<td>7</td>
<td>44</td>
</tr>
<tr>
<td>3 (3338)</td>
<td>2</td>
<td>4</td>
<td>50</td>
</tr>
<tr>
<td>4 (3313)</td>
<td>2</td>
<td>8</td>
<td>32</td>
</tr>
</tbody>
</table>

The table shows that at three intersections, the green times allocated to different stages and the offset times are very similar in the FTC and TRANSYT optimized signal timing plans. The main differences occur for green and the offset times in intersection no. 1. The TRANSYT plan gives a 7-second longer green time and a 12-second early offset to the main street in this intersection, which helps to discharge all the queued vehicles in the eastbound arm before the flow streams from the main intersection.
4 EVALUATION OF THE DIFFERENT SIGNAL TIMING USING HUTSIM

The aim of this chapter was to evaluate the traffic performance impacts of local signal timing adjustment within the framework of the fixed-time coordinated signal control (FTC-LTA). The methodology used for this purpose was micro-simulation using the HUTSIM model as described below.

4.1 THE HUTSIM MODEL

HUTSIM is a time-based micro-simulation tool developed especially for evaluation of traffic signal control strategies. The simulator can be connected to real signal controllers modeling a number of intersections simultaneously, including reproducing most of their functions (Sane, & Kosonen, 1996). The calibration of HUTSIM includes acceleration/deceleration rates for different vehicle types, car-following gaps and stopping distance and critical gaps in yielding and lane switching. The model has also been validated for delays, stops, queues and saturation flows of different lane types (Kosonen, 1999). HUTSIM has been developed and validated for the LHOVRA strategy for isolated traffic signal control in close co-operation between the Finnish and Swedish researchers (Al-Mudhaffar, 1998). It has also been used for tests with fuzzy logic strategies (Kosonen & Bang, 2001).

4.2 SIMULATION APPLICATIONS

Off-line evaluation allows the designer to fine tune the level of signal performance to choose the best solution. HUTSIM is capable of modeling a number of intersections simultaneously, including reproducing most of the functions of real traffic signal controllers.

The following four cases have been evaluated using the HUTSIM model:

1. Manually set signal timing without local signal timing adjustment (FTC\text{man});
2. Manually set signal timing with local signal timing adjustment (FTC-LTA\text{man});
3. Optimized signal timing without local signal timing adjustment (FTC\text{opt});
4. Optimized signal timing with local signal timing adjustment (FTC-LTA\text{opt}).

Midday traffic flow conditions were used in this analysis in order to avoid congested periods when the phases are more likely to use the maximum green time in each cycle. Traffic flows in all simulations were derived from actual on-street counts. Each case was tested using the same trip matrix and all simulations were run for more than 10 hours to establish a “stable” set of results.

The HUTSIM simulation model of the Kungsholmen area is shown in Figure 4.1.
The traffic signal control logic in HUTSIM operates according to the phase control-timing file, which represents the control logic of the controller in the intersection. This logic has been developed to include the LHOVRA technique functions. Figure 4.2 shows an example of the control-timing file in HUTSIM (LastGrn in this file refers to PEG).
Part III: Impacts of Fixed Time Coordination with Local Signal Timing Adjustment

Figure 4.2 HUTSIM phase control-timing file with PEG (LastGrn) in node 3334
(The time is in a 0.1 second)

4.3 RESULTS
The average delay time results of HUTSIM simulations are summarized and compared in Table 4.1 below while Figure 4.3 shows only the results.
Table 4.1 HUTSIM simulations results: Average delay times (s/vehicle) and differences with comparison to FTC\text{man}.

<table>
<thead>
<tr>
<th>Intersection/Comparisons</th>
<th>FTC\text{man}</th>
<th>FTC-LTA\text{man}</th>
<th>FTC\text{opt}</th>
<th>FTC-LTA\text{opt}</th>
<th>FTC-LTA\text{opt} / FTC\text{opt}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Delay (1)</td>
<td>Delay (2)</td>
<td>Delay (3)</td>
<td>Delay (4)</td>
<td>% Difference (2) Compared to (1)</td>
</tr>
<tr>
<td>1 (3321)</td>
<td>13.4</td>
<td>12.5</td>
<td>10.6</td>
<td>9.6</td>
<td>9%</td>
</tr>
<tr>
<td>2 (3334)</td>
<td>16.9</td>
<td>18.5</td>
<td>15</td>
<td>14.9</td>
<td>9%</td>
</tr>
<tr>
<td>3 (3338)</td>
<td>9.6</td>
<td>9.4</td>
<td>8.7</td>
<td>7.3</td>
<td>9%</td>
</tr>
<tr>
<td>4 (3313)</td>
<td>21.3</td>
<td>19.5</td>
<td>19.5</td>
<td>18.8</td>
<td>9%</td>
</tr>
<tr>
<td>Av. (all)</td>
<td>12.6</td>
<td>12.5</td>
<td>11.5</td>
<td>10.9</td>
<td>9%</td>
</tr>
</tbody>
</table>

The following observations (highlighted in the table) can be made for all intersections:
- Using FTC-LTA\text{man} instead of FTC\text{man} had little effect (reduced total delay by 1%);
- FTC\text{opt} reduced the average intersection delay by 9% compared to FTC\text{man};
- FTC-LTA\text{opt} reduced total delay by a further 5% compared to FTC\text{opt}.

For the main intersection (no. 2):
- Using FTC-LTA\text{man} instead of FTC\text{man} increased total delay by 9%;
- FTC\text{opt} reduced the average intersection delay by 11% compared to FTC\text{man};
- FTC-LTA\text{opt} reduced total delay by a further 1% compared to FTC\text{opt}.

The percentage stopped vehicles results of HUTSIM simulations are summarized and compared in Table 4.2 below while Figure 4.4 shows only the results.
Table 4.2 HUTSIM simulations results: Percentage stopped vehicles and differences with comparison to FTC\textsubscript{man}.

<table>
<thead>
<tr>
<th>Intersection/Comparisons</th>
<th>FTC\textsubscript{man}</th>
<th>FTC-LTA\textsubscript{man}</th>
<th>FTC\textsubscript{opt}</th>
<th>FTC-LTA\textsubscript{opt}</th>
<th>FTC-LTA\textsubscript{opt} / FTC\textsubscript{opt}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stops (1)</td>
<td>Stops (2) Com. to (1) (2)</td>
<td>Stops (3) Com. to (1) (3)</td>
<td>Stops (4) Com. to (1) (4)</td>
<td>(4) Compared to (3) (4)</td>
</tr>
<tr>
<td>1 (3321)</td>
<td>38.5</td>
<td>34.3</td>
<td>-11%</td>
<td>28.8</td>
<td>-25%</td>
</tr>
<tr>
<td>2 (3334)</td>
<td>53.8</td>
<td>55.8</td>
<td>4%</td>
<td>49.8</td>
<td>-7%</td>
</tr>
<tr>
<td>3 (3338)</td>
<td>36.8</td>
<td>33.1</td>
<td>-10%</td>
<td>35.6</td>
<td>-3%</td>
</tr>
<tr>
<td>4 (3313)</td>
<td>63.6</td>
<td>63.4</td>
<td>0%</td>
<td>65.4</td>
<td>3%</td>
</tr>
<tr>
<td>Av. (all)</td>
<td>42.8</td>
<td>42.1</td>
<td>2%</td>
<td>41.2</td>
<td>-4%</td>
</tr>
</tbody>
</table>

Percentage stopped vehicles in different simulation senarios

![Percentage stopped vehicles in different simulation senarios](image)

Figure 4.4 Percentage stopped vehicles from HUTSIM simulations

The following observations (highlighted in the table) can be made for all intersections:

- Using FTC-LTA\textsubscript{man} instead of FTC\textsubscript{man} reduced total stopped vehicles by 2%;
- FTC\textsubscript{opt} reduced the average intersection stopped vehicles by 4% compared to FTC\textsubscript{man};
- FTC-LTA\textsubscript{opt} reduced total stopped vehicles by a further 5% compared to FTC\textsubscript{opt}.

For the main intersection (no. 2)

- Using FTC-LTA\textsubscript{man} instead of FTC\textsubscript{man} increased total stopped vehicles by 4%;
- FTC\textsubscript{opt} reduced the average intersection stopped vehicles by 7% compared to FTC\textsubscript{man};
- FTC-LTA\textsubscript{opt} increased total stopped vehicles by 2% compared to FTC\textsubscript{opt}.
5 SYNTHESIS AND CONCLUSIONS

After its initial success, LHOVRA has been widely used in Sweden and the other Scandinavian countries in urban areas at a lower speed (50 km/h). More recently, the “O” function has also been adapted to allow vehicle-actuated past-end-green in fixed time coordinated systems, the intention being to improve traffic performance as well as increase safety. In Stockholm, the timing of the signals is normally performed manually taking into consideration local signal timing adjustments based on detector inputs controlling the termination of the green signal by using PEG, which affects the split of green time, but not the cycle time, nor the offset between the intersections. PEG may come in one or more directions in both main and cross streets.

To evaluate local traffic actuated signal-timing adjustments the following control strategies were studied:
- Fixed time coordination without local signal timing adjustment (FTC);
- Fixed time coordination with local signal timing adjustment (FTC-LTA).

The TRANSYT model was used purely to generate optimized signal timings for input to micro-simulation using the HUTSIM software. The study showed that local traffic adjustment with the manual FTC had little positive effect, while using this adjustment with signal timings determined by TRANSYT reduced the average intersection delay significantly.

This study highlighted the effectiveness of TRANSYT in producing fixed signal timings for coordinated intersections. However, TRANSYT is heavily reliant on high quality input data, and the delay reduction of course depends on the quality of the original, manually developed signal timing plans.

HUTSIM simulations showed that local signal timing adjustment by means of past-end green, originally designed to improve safety and traffic performance of high-speed isolated intersections, was beneficial when applied to coordinated traffic signal control in the study area. Both delays and stops were reduced, although not for the main, critical intersection which operated close to capacity.

Comparative results from simulation and field measurements (part IV) are available for the main intersection with the manual FTC with LTA under non-peak traffic. The HUTSIM simulation results of stops and delay were about 14% higher than the corresponding field results. It should be noted that HUTSIM does not totally reflect the actual on-street situation. In reality, parked vehicles, bicycles and pedestrians often reduce traffic performance. However, since all cases have been assessed using HUTSIM, all the results are comparable.
Part IV

Impacts of Strategies for Coordinated Signal Control with Bus Priority
ABSTRACT

Delay in signalized intersections may constitute a significant part of bus journey times in urban environment. Giving buses priority at traffic signals can be an effective means to reduce this delay. Bus priority in Swedish urban traffic signal systems is normally associated with fixed time plan selection. Within this framework, local traffic-actuated signal timing adjustments are applied based on detector inputs to reduce the number of vehicles in the dilemma zone. Active bus priority is also applied to display a green signal when a bus arrives at the stop line. Due to lack of knowledge of the traffic performance impacts of these techniques, a major research study was undertaken, funded by the Swedish Road Administration. The aim was to evaluate the following control strategies using Stockholm as a case study:

1. Fixed time coordination with local signal timing adjustment (FTC-LTA);
2. FTC-LTA as above with active bus priority (PRIBUSS);
3. Self-optimizing control (SPOT) with active weighted bus priority.

Extensive field data collection was undertaken during separate time periods with these strategies in the same area using mobile and stationary techniques. From the traffic flow data it was found that maximum hour traffic flow was relatively high with about 1/14 of whole day traffic flow.

A method to calculate the approach delay based on the observed number of queuing vehicles at the start and end of green was developed to treat the observed data cycle by cycle. Compared to FTC-LTA, the study obtained the statistically significant following results:

The main intersection shows that:
- Bus travel time was reduced by 14% using PRIBUSS and 12% using SPOT;
- Travel time for all vehicles increased by 24% using PRIBUSS and was increased by 30% using SPOT.

The network shows that:
- Bus travel time was reduced by 11% using PRIBUSS and 28% using SPOT;
- Travel time for all vehicles did not increase using PRIBUSS and was reduced by 6.5% using SPOT.

In the whole network, SPOT performed best at all times of the day with some exceptions at the oversaturated main intersection. A general comparison for both buses and vehicles with buses given a weight of 20 or 25 vehicles resulted in a higher reduction in travel time with SPOT than with PRIBUSS.
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1 INTRODUCTION

1.1 BACKGROUND
Delay at signalized intersections may constitute a significant part of bus journey times in urban environments. Giving buses priority at traffic signals can be an effective means to reduce this delay (Al-Mudhaffar & Bang, 2006). Ordinary bus priority signal systems give priority to buses, thus improving the smoothness of bus transport. Public transport is now coming into focus as a new UTC has been implemented that estimate the effects on travel times for all road users.

‘Active’ priority systems involve bus detection in some form. Three different categories of detection can be identified (Hounsell et al, 2004):

- Infrastructure equipment only, which involves detecting buses;
- On-bus and local infrastructure, which involves the use of bus transponders or ‘tags’ and communication with inductive loops or beacon detectors on the approach to each equipped junction;
- On-bus and central infrastructure, which involves the use of on-board equipment for bus location and, usually, radio based communication between the bus and the control centre.

A new method of bus priority in fixed-time signals has been developed in Leeds in a collaborative project as part of the UTMC program known as SPRUCE. Developments in public transport priority have taken place in four areas (Jones, 1998):

- Selective vehicle detection;
- Traffic signal control;
- Vehicle discrimination techniques;
- Synergy with non-priority ITS measures.

Further progress has been made over the past decade by introducing traffic responsive control systems, where actual traffic conditions influence the timings of the traffic signals. One of the most advanced systems of this type is the Italian system SPOT/UTOPIA (Mizar, 2001) with weighted bus priority using automatic vehicle location (AVL).

In Stockholm, public transport priority in signal system, PRIBUSS (GFK, 1991), began to be implemented more than 15 years ago. Priority is given to applications that will make public transport more efficient and attractive, thereby meeting the objectives of the City of Stockholm. SPOT is installed at eleven intersections in Kungsholmen, while this study focuses on only six of them in the street Fleminggatan with the cross street Scheelegatan, where the video films were taken.

1.2 OBJECTIVES OF THIS PART
The objective of this part of the thesis is to evaluate different control strategies (with and without bus priority) using the following signal systems:

1. Fixed time coordination with local signal timing adjustment (FTC-LTA);
2. FTC-LTA as above with active bus priority (PRIBUSS);
3. Self-optimizing control (SPOT) with weighted bus priority.

1.3 LIMITATIONS
The limitations in this part of the thesis are as follows:
- The impacts of strategies focus on the ones used in Sweden;
- The study does not include the impacts on pedestrians.

1.4 METHODOLOGY
It was planned to use micro-simulation to make “controlled experiments” with different strategies, but this was unfortunately not possible due to lack of a functional emulator for the SPOT system, which was applied in Stockholm at the time of doing this study.

The following steps were included in the research methodology:
1. Collection of data on traffic flow and traffic performance for the following signal control strategies: manual FTC-LTA, PRIBUSS, SPOT;
2. Development of methodology for estimation of delay for signalized approaches based on observations of queue lengths at beginning and end of green;
3. Analysis and comparison of traffic performance for the main intersection (queue lengths, stops and stopped delay for all vehicles; travel time and stopped delay for buses);
4. Analysis and comparison of bus and vehicle travel times for the coordinated network.

1.5 STUDY AREA
The Kungsholmen area in Stockholm provides a good range of different street layouts, traffic conditions and public transport services typical to Swedish traffic situations. The network is located along three main roads; Fleminggatan, Scheelegatan and Norr Mälarstrand, close to the centre of Stockholm, see Figure 1.1.

Five intersections along the main street Fleminggatan have coordinated traffic signals with local traffic adjustment and bus priority. Two intersections on Scheelegatan and three on Norr Mälarstrand have coordinated traffic signals without bus priority. The Fleminggatan - Scheelegatan intersection (number 2 on the map in Figure 1.1) is located in the heart of this area and is the main intersection and has been specially studied. The predominant traffic flow is from west to east, particularly during the morning peak, as traffic comes into the city from one of Stockholm’s major motorways. Approximately 35,000 vehicles a day pass through the main intersection in the study area and about 21,000 vehicles pass through the main street Fleminggatan. All roads in the study area are undivided two-way streets with a speed limit 50 km/h and flaring at some intersection approaches.

Kungsholmen was also chosen by Stockholm authorities as a test site to implement the SPOT/UTOPIA, which was beneficial to the study described in this paper. The field trial network consisted of eleven signalized intersections where bus priority was provided at six intersections, five of them on Fleminggatan. The important “blue bus” number 1 along Fleminggatan received bus priority.
Figure 1.1 The field trial network & public transport routes on Fleminggatan

1.5.1 Traffic flow
Transport demand, both public and private, is high in the area. The number of buses with priority is very high during peak hours. The most loaded street has approximately 21,000 vehicles per day and one bus every 5 minutes in each direction during peak hours. Queues grow quickly and cause blocking back in the upstream intersections. The coordination of traffic signals creates a green wave in one direction during the morning peak and in the opposite direction during the afternoon peak.

There are no heavy goods vehicles in the area, but local delivery vehicles are more frequent. There are also many pedestrians and cyclists in the area.

1.5.2 Public transport
The main public transport routes are in the main street Fleminggatan. In total there are two main and several minor bus services that pass through the network. The buses have no dedicated lanes and the distance between bus stops ranges from 200 to 400 meters. The Fleminggatan-S:T Eriksgatan intersection, marked on the map by a circle, is not included in the coordination systems. The bus stop located to the left of the intersection is therefore not included in this study.
2 DESCRIPTION OF CONTROL STRATEGIES

Delay in signalized intersections may constitute a significant part of bus journey times in urban environments. Providing passive (e.g., reserved bus lanes) as well as active priority for buses at traffic signals can be an effective measure to reduce this delay. The thesis describes and evaluates methods for active bus priority, which requires detection and prediction of the arrival time of each bus at the next signalized stop line on the route. The control strategies are designed to change the current signal timing sequence to minimize bus delay, either by shortening the red time if the bus arrives during red, or extending the green time if required, to make sure that the bus will pass the intersection without having to stop.

In his thesis, Davol (2001) states that “There are three basic actions that a controller can perform in response to the detection of a transit vehicle: extension of the green interval in the current phase, ending another phase early to give an early green to the vehicle, and inserting an extra phase to allow the vehicle to pass before returning to the regular timing. The response used will depend on when in the cycle the vehicle is detected.” A review of the most common Transit Signal Priority strategies used in USA has shown that of all the different methods, early green and extended green are the ones most used (Ova and Smadi, 2001).

Urban traffic signal control systems in Sweden are normally coordinated with fixed time plan selection and associated with local signal timing adjustment based on detector inputs controlling the termination of the green signal by using PEG. Major Swedish cities also normally incorporate active bus priority (PRIBUSS) aimed at displaying a green signal on the arrival of the bus at the stop line. Decentralized systems for self-optimized signal control of urban signal networks, for example the Italian system SPOT, are also being introduced on the Swedish market. These strategies are described in more detail in this section.

2.1 PRIBUSS

PRIBUSS (Björck & Dahlgren, 1991) is a standardized signal toolbox providing several types of active bus priority schemes and compensation procedures, as explained in Figure 2.1:

- Extension;
- Re-taken start (return to green after start of yellow);
- Early green;
- Extra stage;
- Double early green;
- Double extra stage.
The actual position of the buses on prioritized routes is obtained using automatic vehicle location techniques (AVL). All blue buses on the main routes in Stockholm have equipment for location and communication. The test area for advanced priority functions is chosen so that the two routes (1 and 3) pass the test area. The position of the bus is based on a line description and dead reckoning. At certain calibration points, the Global Positioning System (GPS) onboard is used to make corrections.

The buses communicate via a dedicated radio channel with the public transport center where a forecast model provides arrival times for passengers waiting at bus stops to the nearest minute. However, the priority call is based on local radio communication between the bus and the controller (the so called LISA system), making predictions for the arrivals of PT vehicle at traffic signals. The priority is absolute: the bus should not be delayed at the signalized intersection.

Figure 2.2 shows an example of using PRIBUS at the main intersection and the compensation for the opposing phases, taken from EC track at Stockholm Street Administration (GFK).
Traffic signal succession at the intersection 3334 without using PRIBUSS

PRIBUSS: Extension without compensation for the opposing group

PRIBUSS: Extension with compensation for the opposing group

PRIBUSS: Extension after PEG with compensation for the opposing group

Figure 2.2 Traffic signal succession figures at intersection 3334 with PRIBUSS (Real time function from EC track)
2.2 SPOT

SPOT is a traffic signal control strategy developed by Mizar Automazione (2001) in Turin, Italy. It was first installed at intersections in Turin. SPOT is now used in several cities in Italy and also in the Netherlands, the USA, Norway, Finland and Denmark (Arveland, 2005). The idea behind SPOT is decentralized, socio-economic optimization of the signal settings in real time for each individual intersection based on predictions of arrival flow profiles from upstream intersection arms. The main costs arise out of vehicle delays and vehicle stops. In order to give priority to buses and trams, higher unit costs are used for these vehicles.

The decentralized optimization process in SPOT enables faster changes of signal timing than is possible in SCOOT (Hunt et al, 1981) with its area-wide signal timing optimization, or SCATS (Akcelik, 1996) with its sub-area optimization focused on signals surrounding a critical intersection.

SPOT requires detectors at the beginning of each link counting vehicles in each lane. SPOT is a special program that operates on a separate central processing unit (CPU) connected to the traffic signal controller by a special interface. The CPU can be on a single card or in a complete industrial PC. The SPOTs at each intersection exchange information with neighbors, with the AVL system for public transport, and with a database at the central level (Kronborg & Davidsson, 2000).

The new advanced Urban Traffic Control (UTC) system in Sweden minimizes the total time lost by private vehicles during their trips within the controlled area, subject to the constraint that public vehicles for which weighted priority has been requested shall not be stopped at intersections with traffic lights. The UTC in Stockholm is based on the SPOT/UTOPIA concept that has been designed by applying large-scale systems theory to the whole urban traffic control problem.

The selected system is reduced to a series of smaller and related problems, which can be classified into two different levels: the "local level" and the "area level". At the area level, the UTOPIA system is used to produce reference time plans. At the local level, problem formulation is based on the definition of two fundamental modules: the "local traffic model" and the "local control" (INCOME, 1997).

2.2.1 Theoretical Description

A more theoretical description of SPOT is given below.

2.2.1.1 Cost Function

The elementary component of the model is the "link". The link is a logical entity defined by grouping the carriageway turns which have the same favorable traffic light stages and which allow traffic movements with the same conflicts at the junction. Each link belongs to a carriageway and a carriageway can generate several links according to the possible carriageway turns, the signal setting and the junction traffic regulation.

For each quantity or parameter relating to the carriageway, a corresponding quantity or parameter is defined for the link together with a corresponding rule. As a result, the junction model used by the local control consists essentially of links corresponding to the incoming carriageways.
Optimization is performed on a time horizon for the next 120 seconds and is repeated every three seconds. The resulting optimal signal settings are actually in operation only for three seconds. The closed loop control thus obtained can be viewed as an "Open Loop Feedback Control" or as an application of a "Rolling Horizon "concept.

2.2.1.2 Reference Plan
The algorithm tries to adapt the control strategy to the reference plan, see Figure 2.3. The control strategy decided at the previous iteration is shifted on the horizon according to the elapsed time from the previous computation (normally 3 seconds). The algorithm operates on the public transport vehicles in the horizon. The control strategy thus obtained is then adapted to create a green window covering the predicted arrival times.

Figure 2.3 Principles of updating of SPOT control strategy

2.2.1.3 Network Communication
The communication network of the whole system has a hierarchical, distributed system architecture. The central system operates at the area level and the intersection control units are nodes of a network where each node can transmit messages to all the others. If a node wishes to send a message to another node to which it is not directly connected, it passes its message via other node(s). This allows a physical communication scheme where each SPOT is connected to the adjacent one(s) and some of them are connected to the area level.

2.2.2 Public Transport Detection in the Tested Area
Normal SPOT detection is based on in-counting and out-counting detectors, see Figure 2.4. In case of signal groups that should be given green on demand, there needs to be a demand detector closer to the stop line, or a push-button for pedestrians.
The first route given priority by the traffic control system was the number 1 bus service, which travels along the main route (Fleminggatan). The prediction model in the Public Transport Locator provides a forecast for a variety of traffic situations, e.g. bus stop before the stop line, reserved lanes, shared routes etc. Each individual service was predicted according to its own progress characteristics, reflecting the deviation from service headway.

The forecast model used in Stockholm was based on a filtering technique considering time of day and service number. In the case of bus priority (using dedicated stages), the length of the optional stage is set by the control strategy based on the distribution function given by the forecast of the bus arrival. The forecast is produced by the system and the first notification is available to the controller approximately 2 minutes before the vehicle reaches the stop-line. As the vehicle approaches the stop line, the forecast is updated. The last, and most crucial, location message from the system is given when the vehicle passes the stop line.

2.2.3 Results From the INCOME Project
The following average public transport journey time-savings as a result of priority at traffic signals were measured in INCOME (1997):
Part IV: Impacts of Strategies for Coordinated Signal Control with Bus Priority

- In London, average bus journey time-savings of 4%-10% were measured across some 20 junctions using Bus SCOOT compared to normal SCOOT control, with no significant impact on other traffic. This equates to average bus delay savings per junction of some 7%-20%;

- In Gothenburg, the introduction of SPOT with bus/tram priority achieved a 5%-15% improvement in journey times for buses/trams and a 5%-10% improvement for private traffic;

- In Turin, the introduction of UTOPIA, incorporating tram priority integrated with SPOT traffic responsive control produced journey time-savings for trams of 3%-16%. 
3 METHODS FOR DATA COLLECTION AND REDUCTION

This chapter describes the required data and the methods to collect and reduce it for the field study of the impacts of strategies with bus priority as well as the base case for comparison, manual coordinated signal control with local timing adjustment.

3.1 REQUIRED DATA
The following types of data were collected and analyzed for the coordinated network and for the main intersection:
- Traffic flow and composition;
- Signal timing characteristics (cycle time, green time);
- Traffic performance (travel time, queue lengths, number of stops, and stopped delay for overall motor vehicle traffic and travel time and stop delay for bus traffic).

For the main intersection, the data was collected and analyzed on a cycle-by-cycle basis. For the network, average values of the travel time were calculated for chosen time periods: 7:00-9:30, 12:00-14:00, and 15:30-18:30, and for all periods combined. The data collection was performed over four weekdays for each strategy.

The following methods were applied:
1. Floating-car surveys (network);
2. Real time data (ATR) for each bus and bus service on the main street (obtained from the Stockholm Public Transport Traffic Management System);
3. Stationary video recording (main intersection).

3.2 METHOD FOR FLOATING CAR SURVEY
Four cars were used to measure travel time in the network by registering the stop line passage time at every signalized intersection along the selected routes. The drivers were instructed to use “average car technique”, i.e. to follow the general speed of the traffic. The two routes used for the floating-car survey are shown in Figure 3.1.

![Figure 3.1 Routes 1 & 2 for floating car survey](image)
The collected data for each route and run were used to calculate travel time for the whole route in the area as well as for each route segment.

3.3 METHOD FOR STATIONARY VIDEO SURVEY
Stationary video recordings with two camcorders from an 17 m high platform were applied to measure traffic flows, queues and bus travel time as well as cycle and green times at the main intersection and its neighbors, as shown in Figure 3.2.

![Figure 3.2 Location of the camcorders at Kungsholmen](image)

Data reduction was performed on a lane-by-lane and direction basis for each cycle for the following time periods: 08.00-09.00, 12.00-13.00, and 16.00-17.00. Specially developed software for semi-automatic video analysis called SAVA (Archer, 2003) was used for this purpose.

3.4 METHOD FOR ATR DATA REDUCTION
Data from the ATR (Automatisk Trafikant Räkning, in English Automatic Passenger Counting) system were used to measure bus travel time including stops at the bus stops and traffic congestion. Since ATR data collection equipment was only fitted on 15% of the buses (random selection), the data reduction had to take account of the departure time of equipped buses in order to obtain comparable observations between different days and weeks.

The collected data was not regular, i.e. the number of registered bus departures differed from day to day and from time to time. And as the bus travel time is greatly dependent on the degree of congestion, comparing these data without matching the departure times is not sufficient. Therefore, the comparisons were made in two ways:
1. Among all bus departures for compared days;
2. Among all equivalent bus departures for compared days.

As there were insufficient data from registered departures at exactly the same time, it was necessary to have some margin of the time differences among departures but not more than 20 minutes.

3.5 METHOD FOR DETERMINATION OF INTERSECTION STOPPED DELAY BASED ON QUEUE OBSERVATIONS

Calculation of stopped delay for traffic-actuated signal control requires special consideration. Webster (1966) and Akcelik (1996) use adjustment factors of delay calculated for fixed signal timing.

Wilson (2002) divided the total measuring period (T) into sub-periods. Each sub-period incorporates a number of cycles and is defined by the property no vehicles waiting in a queue in front of the stop line at the beginning of the first cycle and the end of the last cycle of the sub-period. He draws a similar diagram of the vehicle-actuated signalized intersection. This method may also be used to calculate the delay time. The method is shown in Figure 3.3.

![Figure 3.3 Queue length (vehicles) as a function of time (Wilson, 2002)](image)

In this study, the number of queuing vehicles in each lane at the beginning and end of the green time, as obtained by video observation, was used for the delay calculation. Since the effective green times vary from cycle to cycle, the delay calculation model incorporates this feature, as illustrated in Figure 3.4.
Figure 3.4 Model of calculating stopped delay from queue lengths with explanation of the (Cr) & (Cg)

For the fixed cycle time, total delay is calculated as the area under the queue length line in Figure 3.4:

\[
d = \sum_{i=1}^{n} \left( \frac{C}{2} Qu_i - \frac{t_i Qu_i}{2} + \frac{C}{2} qu_i \right) = \sum_{i=1}^{n} \left( \frac{(C-t_i) Qu_i + C qu_i}{2} \right)
\]  

[3.1]

where:

- \(d\) = Total delay in pre-timed intersection for a number of cycle times (s)
- \(C\) = Fixed cycle time (s)
- \(t\) = Green time without queue (s)
- \(r\) = Effective red time (s)
- \(g\) = Effective green time (s)
- \(Qu\) = Queue at the start of g (veh)
- \(qu\) = Queue at the end of g (veh)
- \((C-t_i)\) = Saturated green time

The variable cycle time demands that every cycle time has to be treated separately as expressed in the formula below and explained in Figure 3.4:

\[
d = \sum_{i=1}^{n} \left( \frac{(C_{n-t_i}) Qu_i + C_{gi} qu_i}{2} \right)
\]  

[3.2]
where:

\[ C_n = \text{Variable cycle time begins with red signal (r to r C)} \]

\[ C_{gi} = \text{Variable cycle time begins with green signal (g to g C)} \]

The average delay \( \bar{d} \) is equal to the total delay divided by the number of vehicles passing the stop line during the measured time \( N \):

\[ \bar{d} = \frac{d}{N} \]
4 RESULTS

The flow volumes and distributions are changeable, largely depending on the different times of day. Data was therefore collected and reduced for different times. These distributions have affected the lengths of the cycles and green times in each direction. Therefore, the presentation of the results of the impacts of the different strategies were preceded by describing:

- The distribution of the traffic flow;
- The changes in the length of the green times and the cycle times.

The main intersection in the area (Fleminggatan-Scheelegatan) is the most congested one and it forms the bottleneck for both main and cross streets, which make it necessary to study it separately. The presentation is therefore in two parts:

- Main intersection traffic performance impacts
  - Bus travel time through the intersection,
  - Queue and delay in the intersection;

- Network traffic performance impacts
  - Bus travel time in the main street,
  - Vehicles travel time in the network.

4.1 TRAFFIC FLOW IN THE MAIN INTERSECTION

The traffic flow is never the same and the performance impacts will not be the same as they are dependent, among other things, on traffic volume. The study of the traffic conditions on the studied days aims to determine the differences in traffic volume and distribution and whether these differences affect the possibility to compare the impacts of the different strategies.

The video record focusing on the main intersection of Fleminggatan – Scheelegatan (Figure 4.1) gives a good possibility to count traffic flow and its distribution on the main street Fleminggatan as well as on the cross street Scheelegatan. The traffic flow has been transformed into passenger car units (pcu). Some results from the SPOT data are also shown here, representing traffic flow of mixed traffic. As the heavy vehicles ratio is between 4 and 6% then the traffic flow from SPOT data will be about 5% less than the pcu data.
4.1.1 Average Traffic Flow Distribution in Different Times of Day

The average traffic flows for three times of day are shown in Figure 4.2. The flow in the main street Fleminggatan towards east (city) was higher than the opposite flow (from city) but they had the same pattern all the time, while the flow in the cross street Scheelegatan towards the north (city) was higher than the opposite flow (from city) in morning traffic and lower in afternoon traffic.

![Traffic flow distribution over time](image)

**Figure 4.2** Average traffic flow distribution in the main intersection over time
Results from SPOT data show that the highest traffic flow was in the main street (direction towards city) between 08.00 and 09.00, while the highest traffic flow in the whole intersection was between 17.00 and 18.00.

4.1.2 Traffic Flow Volume with Different Strategies
In order to compare traffic performance results for the tested different strategies it was essential that the overall traffic flow and directional distribution did not change significantly during the study period. Figure 4.3 below shows the average traffic flow variation with time of day for the main intersection during the test periods for each strategy. The figure shows that the traffic flow conditions were very similar for all the tests.

![Traffic flow over time in the main intersection](image)

**Figure 4.3** SPOT data, traffic flow distribution with different strategies over a whole day

From these data it was found that the maximum hourly traffic flow was about 1/14 of the whole-day traffic flow.

Table 4.1 shows in more detail that morning and afternoon traffic flows were almost the same on the test days with FTC and PRIBUSS, while the traffic flow on the test days with SPOT was about 4% higher in the morning and about 2% higher in the afternoon than for other strategies. Midday traffic flow, on the other hand, was about 2% higher on the test days with FTC than the other two systems.

**Table 4.1** Traffic flows in the intersection with different strategies at different times of day

<table>
<thead>
<tr>
<th>Time</th>
<th>FIXED T</th>
<th>PRIBUSS</th>
<th>SPOT</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>08:00</td>
<td>2582</td>
<td>2578</td>
<td>2678</td>
<td>2613</td>
</tr>
<tr>
<td>12:00</td>
<td>2086</td>
<td>2039</td>
<td>2044</td>
<td>2057</td>
</tr>
<tr>
<td>16:00</td>
<td>2501</td>
<td>2505</td>
<td>2555</td>
<td>2520</td>
</tr>
</tbody>
</table>
4.2 CYCLE AND GREEN TIMES IN THE MAIN INTERSECTION

The signal timing was recorded during all trial periods. Green time in the main street represents the east-bound arm as it had a longer green time than the west-bound one. The approach towards the west has an early cut-off green time to facilitate the heavy left-turning movement from the opposing approach. This early cut-off was used in all the tested strategies. It was up to 7 seconds with fixed-time and PRIBUSS (an extra 6 seconds of PEG may come into the picture) and up to 16 seconds in SPOT.

4.2.1 Cycle Time

The differences in the range of the cycle time of the strategies were quite considerable. PRIBUSS had lower minimum and higher maximum cycle times as bus priority could be operated by extension or extra stage. SPOT had a wider range than both other strategies due to the adjustments in response to the fluctuation of the traffic flow. But the average of the cycle time was the same in fixed-time & PRIBUSS and only one second more in the SPOT system. This may give an early indication that the manual time setting of the coordinated fixed time has been done well.

![The cycle time in the main junction Flemingg.-Scheeleg.](image)

**Figure 4.4** Minimum, average, & maximum cycle times of the different strategies

4.2.2 Green Times

The average green times on the main and the cross streets were both the same in FIXED T & PRIBUSS, but the average green time in SPOT was 2 s lower on the main street and 3 s higher on the cross street. The differences in the range of the systems’ green times were quite considerable. Compared to fixed-time, PRIBUSS had lower minimum and higher maximum green times on the main street and lower minimum and the same maximum green times on the cross street. SPOT had a wider range than both other systems but compared to PRIBUSS, the maximum green time on the main street was the same and the minimum green time on the cross street was almost the same.
Part IV: Impacts of Strategies for Coordinated Signal Control with Bus Priority

Figure 4.5 Minimum, average & maximum cycle times of the main and cross streets

4.2.3 Ratio of Average Green Times to Average Cycle Time
Figure 4.6 below shows that the ratio of average green time in Fixed T and PRIBUSS was almost the same in spite of the big difference in the fluctuation in cycle length, while the ratio of average green time in SPOT was slightly different. The ratio of the average green time in SPOT was about 1% higher than FIXED T. There was also a difference in the split of this green time to the advantage of the cross street in the SPOT system by about 2%.

Figure 4.6 The ratio of the average green times to the average cycle time in different strategies

4.2.4 Analysis
The resulting average cycle times and green times in each phase for the main intersection are recorded in Table 4.2, which shows that the average timing was very similar for all tested strategies, the main difference being that SPOT allocated more green time to the cross street.
Each strategy had a different range of signal timing variation, with SPOT the largest and FTC-LTA the smallest, which was due to the different signal plans in different times of day.

Table 4.2 Minimum, average and maximum cycle and green times of the main and cross streets

<table>
<thead>
<tr>
<th>Strategy</th>
<th>Cycle time (s)</th>
<th>Main Street Green Time (s)</th>
<th>Cross Street Green Time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min</td>
<td>Av</td>
<td>Max</td>
</tr>
<tr>
<td>FTC-LTA</td>
<td>64</td>
<td>77</td>
<td>87</td>
</tr>
<tr>
<td>PRIBUSS</td>
<td>45</td>
<td>77</td>
<td>100</td>
</tr>
<tr>
<td>SPOT</td>
<td>42</td>
<td>78</td>
<td>115</td>
</tr>
</tbody>
</table>

4.3 TRAFFIC PERFORMANCE IMPACTS IN THE MAIN INTERSECTION

The most congested intersection in the area is located in the center of the coordinated systems in both directions. Therefore, it was necessary to study this intersection separately to get an idea about the traffic performance impacts of the strategies in the oversaturated situation.

Given that bus priority was one of the main aims of both PRIBUSS and SPOT, the traffic performance impacts were divided into:

- Bus travel time through the intersection;
- Queue and delay in the intersection.

For bus travel time, all the prioritized buses were recorded, and for number of vehicles in queue, data was collected and analyzed on a cycle-by-cycle basis.

4.3.1 Bus Travel Time Through the Main Intersection

Video recording was used to obtain bus travel time between the bus stops near the main intersection and the stop line of the adjacent intersection (Fleminggatan-Pipersgatan) as marked in Figure 4.7. This choice depends on the FTC manual time setting, which treated both intersections as one large intersection.

Figure 4.7 The distance defined for bus travel time through the main intersection
The Semi-Automatic Video Analyzer SAVA (Archer, 2003) was used to register passage time and the program MS Excel used to calculate the travel time.

The time stopped at bus stops is to be treated carefully. Sometimes it can be excluded to compare travel time, sometimes not. If the stop time is only due to passenger movements it will be possible to exclude it, but if it is due to congestion on the street, then it will be included in the travel time for comparison. In this case, it is also different also in different directions. Where the bus stop is after the intersection in direction 2 (from city), then the stop time is due to both passengers and congestion after the intersection. The stop time at the bus stop before the intersection in direction 1 (towards city) also depends on prioritizing method (PRIBUSS, among other things, uses a method known as “Double Recall”), which affects the stop time length as the bus stop is close to the intersection. Therefore, it is necessary to include the stop time at bus stops in the comparison.

However, the results in Figure 4.8 show that the average time in both directions had the same pattern. Looking only at travel time, it was reduced by 12.3% using PRIBUSS and 11.6% using SPOT, both compared to FTC. For travel time including bus stop time, this was reduced by 9.9% using PRIBUSS and by 8.7% using SPOT, both also compared to FTC.

Figure 4.8  Average bus travel time through the intersection with different systems

As shown in Figure 4.9 below, PRIBUSS performed better than SPOT in morning peak traffic, which may be due to the smaller queues on approaches towards the city, as the gating upstream regulated the volume of traffic arriving at the main intersection. In contrast, SPOT performed better than PRIBUSS in afternoon traffic, when the flow into the city was less than in morning traffic.
4.3.2 Queue and Delay at the Main Intersection

The method of calculating delay at the main intersection was to count the number of vehicles in the queue at the start of effective green time and the overflow (left-over) queue at the end of the effective green time. Accumulated arrivals are included in this calculation by the term $Qu$ in formulas [2] and [3], while $qu$ refers to the overflow queues.

The priority of bus service 1 caused some disturbance to other vehicles, especially in the cross direction, making it necessary to study the differences in traffic performance for both streets separately.

Results summarized in Table 4.4 below show that for the main street the delay decreased by 14% using PRIBUSS compared to FTC-LTA, while it increased by 9% using SPOT. In the cross street, PRIBUSS and SPOT caused higher delay compared to FTC-LTA. For both streets combined, PRIBUSS and FTC-LTA had very similar traffic flow and average number of queuing vehicles at start of green, but the overflow queue was higher for PRIBUSS, resulting in a 24% increase in delay. SPOT gave longer queues both at the start and the end of green (overflow), resulting in an increase in delay of 30%.

Table 4.4 shows the results regarding vehicle flow, bus flow and number of queuing vehicles at the start and end of green time (overflow). The same table also shows the average delay at the main intersection and the bus travel time including stop time at bus stops.
Table 4.4 Traffic flow, queue and delay in both streets and bus travel time in the main street

<table>
<thead>
<tr>
<th></th>
<th>Cycle time</th>
<th>Gr time</th>
<th>Flow</th>
<th>Queue/Start</th>
<th>Queue</th>
<th>Overflow</th>
<th>Overfl</th>
<th>Delay</th>
<th>Bus flow</th>
<th>Bus travel</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Main street</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FTC-LTA</td>
<td>77</td>
<td>42</td>
<td>1426</td>
<td>647</td>
<td>45</td>
<td>5</td>
<td>0.4</td>
<td>15.6</td>
<td>20.7</td>
<td>70.8</td>
</tr>
<tr>
<td>PRIBUSS</td>
<td>77</td>
<td>42</td>
<td>1433</td>
<td>565</td>
<td>39</td>
<td>8</td>
<td>0.6</td>
<td>13.5</td>
<td>21.3</td>
<td>63.8</td>
</tr>
<tr>
<td>SPOT</td>
<td>78</td>
<td>40</td>
<td>1472</td>
<td>781</td>
<td>53</td>
<td>14</td>
<td>1.0</td>
<td>16.9</td>
<td>22.0</td>
<td>64.6</td>
</tr>
<tr>
<td><strong>Cross street</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FTC-LTA</td>
<td>77</td>
<td>23</td>
<td>964</td>
<td>567</td>
<td>59</td>
<td>0</td>
<td>0.0</td>
<td>20.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PRIBUSS</td>
<td>77</td>
<td>23</td>
<td>941</td>
<td>653</td>
<td>69</td>
<td>47</td>
<td>5.0</td>
<td>33.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPOT</td>
<td>78</td>
<td>26</td>
<td>954</td>
<td>737</td>
<td>77</td>
<td>12</td>
<td>1.3</td>
<td>30.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FTC-LTA</td>
<td>77</td>
<td>65</td>
<td>2390</td>
<td>1214</td>
<td>51</td>
<td>5</td>
<td>0.2</td>
<td>16.6</td>
<td>20.7</td>
<td>70.8</td>
</tr>
<tr>
<td>PRIBUSS</td>
<td>77</td>
<td>65</td>
<td>2374</td>
<td>1218</td>
<td>51</td>
<td>55</td>
<td>2.3</td>
<td>20.6</td>
<td>21.3</td>
<td>63.8</td>
</tr>
<tr>
<td>SPOT</td>
<td>78</td>
<td>66</td>
<td>2426</td>
<td>1518</td>
<td>63</td>
<td>27</td>
<td>1.1</td>
<td>21.5</td>
<td>22.0</td>
<td>64.6</td>
</tr>
</tbody>
</table>

Looking at the impacts in *different directions*, it is clear that the vehicles take the advantage of bus priority in their direction at the expense of the cross direction. Delay with PRIBUSS was lowest in the main street and highest in the cross street, although the average cycle and green times were the same as with FTC, but slightly different from SPOT. SPOT had 2 s lower average green time in the main street but 3 s higher average green time in the cross street, while it had only 1 s higher average cycle time which made the ratio of the green time to the cycle time almost the same. These differences in the split of green time were to the benefit of the streams in the cross street in SPOT system.

Looking at the impacts at different *times of the day*, Figure 4.10 below shows that PRIBUSS was worst in morning traffic but the best in afternoon traffic, which was due to the effect of traffic flow distribution as the traffic volume towards the city was much higher in morning peak traffic and the gating upstream (which may be ceased with bus priority) had more effect on this intersection. This upstream gating (which was not used with SPOT) benefits the FTC system as the queues were blocked at other intersections upstream.

![Vehicle delay time for different systems and times of day](image)

*Figure 4.10* An overview of vehicle delay time at different times of day
4.3.3 Analysis
Bus priority in TRANSYT and SPOT gains from using a weighting factor. TRANSYT (TRL, 2002) gives the bus a weight of 20 pcu estimating that buses curried an average of 28 passengers, compared to 1.4 for other passenger cars (The result is an optimization of person delay as opposed to vehicle delay). Normal priority in SPOT (Peek, 2000) gives the bus a weight of about 25 private vehicles.

Comparing PRIBUSS and FTC-LTA results for the main intersection shows a delay reduction in bus travel time of 7 s/bus and a delay increase for other vehicles of 4 s/veh. On an hourly basis, bus travel time was reduced by 149 s/h and other vehicle delays increased by 9,500 s/h, resulting in a weighting factor of 1/64. Comparing SPOT to FTC-LTA in the same way, the results show a delay reduction in bus travel time of 6.2 s/bus and a delay increase for other vehicles of 4.9 s/veh. On an hourly basis, bus travel time was reduced by 136 s/h and other vehicle delays increased by 11,900 s/h, resulting in a weighting factor of 1/87. It should be borne in mind that the days operated with SPOT had a slightly higher traffic flow.

4.4 TRAVEL TIME IN THE NETWORK
The prioritized blue bus service 1 drives along main route 2 (Fleminggatan). The results are therefore presented in two parts:

- Bus Travel Time in the Main Route;
- Vehicle Travel Time in the Network.

4.4.1 Bus Travel Time in the Main Route
All travel time registrations were from buses traveling along the main street Fleminggatan where the buses are given priority in both the PRIBUSS and the SPOT systems. Data from the ATR system was used to measure bus travel time including stopped time at bus stops and due to traffic congestion. The comparison only included equivalent bus departures for comparable days since bus travel time is strongly related to congestion.

Figure 4.11 below shows that bus travel time was reduced by 11% using PRIBUSS and 28% using SPOT compared to FTC-LTA. The table shows that PRIBUSS has much lower stop time in traffic congestion with 40 s compared to 68 s with FTC, while SPOT had the lowest with only 5 s. Travel times with and without bus stops showed the same pattern. The results below represent both directions.
There are only 6 cases of matched equivalent departures across all three systems therefore it is advisable to make a pair-wise comparison to find more cases to compare. With Pair-Wise Comparisons, 10 matches could be found for every system pair. The results shown in Figure 4.12 below show that bus travel time along both directions of the main street was reduced by 7% using PRIBUSS and 20% using SPOT compared to FTC-LTA. Comparing bus travel time of the departures with SPOT and PRIBUSS, it was 10% lower with SPOT. It is obvious that SPOT gave buses better priority than PRIBUSS.

4.4.2 Vehicle Travel Time in the Network.
All floating car travel time registrations for the two routes were considered in this evaluation. The cars on route 2 drove along the same path as the prioritized buses, and could be affected positively by the bus priority strategy. Table 4.4 below shows that vehicle travel time was
reduced by 9% using PRIBUSS and 20% using SPOT. For route 1 the opposite is the case, where vehicle travel time increased by 6% using PRIBUSS and 4% using SPOT. For both routes combined, the results were almost the same for FTC-LTA and PRIBUSS, while SPOT reduced travel time by about 6.5% compared to both other strategies. The comparison of PRIBUSS and SPOT with FTC-LTA was shown to be statistically significant.

Table 4.4 Flow at the main intersection and travel times in the network

<table>
<thead>
<tr>
<th>Flow in the main intersection (veh/h)</th>
<th>Vehicle travel time in the network (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whole inters.</td>
<td>Cross street</td>
</tr>
<tr>
<td>FTC-LTAman</td>
<td>2390</td>
</tr>
<tr>
<td>PRIBUSS</td>
<td>2374</td>
</tr>
<tr>
<td>SPOT</td>
<td>2426</td>
</tr>
</tbody>
</table>

**Vehicles’ Travel Time at Different Times of Day**

Figure 4.13 below shows that results from PRIBUSS were better than FTC during morning and midday traffic conditions, while FTC was better during afternoon traffic. SPOT, on the other hand, performed best at all times of day.

![Figure 4.13 Vehicles’ travel time at different times of day](image)

**4.4.3 Analysis**

Both PRIBUSS and SPOT reduced travel time in the network, but the reduction in travel time varied between the prioritized buses and the other vehicles. To make a general comparison for both buses and vehicles together it is possible by weighting the bus in terms of a number of equivalent vehicles. In section 4.3.3, it is mentioned that TRANSYT gives the bus a weight of 20 pcu, while SPOT gives the bus a weight of 25 private vehicles. It may be useful to include both weighting factors in the following comparisons.

Giving the bus a weight of 20 vehicles, the reduction in travel time for all vehicles in the network was 1.2% with PRIBUSS and 8.4% with SPOT, both compared to FTC-LTA. Giving the bus a weight of 25 vehicles, the reduction in travel time for all vehicles in the network was 1.4% with PRIBUSS and 8.8% with SPOT, both compared to FTC-LTA.
5 SYNTHESIS AND CONCLUSIONS

The traffic flow conditions were very similar for all the tests. There were no significant differences although traffic flow was on average about 2% higher on SPOT days. The traffic flow distribution had some effect on the signal timing, as the cityward traffic volume was much higher in morning peak traffic. FTC-LTA blocked the queue upstream of the main intersection by deliberate gating. This gating ceased with bus priority. SPOT did not use this upstream gating. With its method of optimizing cycle and green times, SPOT proved to be insufficient to deal with the oversaturated situations.

The average cycle time was very similar for all tested strategies. The main difference was that SPOT allocated more green time to the cross street. Each strategy had a different range of signal timing variation, with SPOT the largest and FTC-LTA the smallest.

The traffic performance results from the field measurement are presented in two parts as below:

- In the main intersection;
- In the whole network.

5.1 THE MAIN INTERSECTION

Looking at the impacts in different directions, it was clear that the vehicles took advantage of bus priority in their direction at the expense of the cross direction. Delay with PRIBUSS was lowest in the main street and highest in the cross street, which was due to greater overflow queues. SPOT had higher queues but its timing flexibility reduced the overflow queues.

Performances during different degrees of saturation at different times of the day varied considerably. Bus priority imposed a higher penalty on other vehicles with PRIBUSS in the oversaturated situation. On average, FTC-LTA had the longest bus travel time, while it had the lowest average delay, while PRIBUSS and SPOT had almost the same average bus travel time.

Results for the main intersection showed that the reduction in bus travel time with PRIBUSS increased the delay for other vehicles by a weighting factor of 64 and the reduction in bus travel time with SPOT increased the delay for other vehicles by a weighting factor of 87, both in comparison to FTC-LTA.

5.2 THE NETWORK

The reduction in bus travel time with SPOT was twice as great as with PRIBUSS. SPOT had the lowest stop time in traffic congestion, which means smooth travel. This resulted in shorter delay for the other vehicles on the same route with almost the same pattern, while vehicle travel time increased on the cross route by using SPOT by only two thirds of the increase with PRIBUSS. For both routes combined, the results were almost the same for FTC-LTA and PRIBUSS, while SPOT reduced travel time significantly compared to the other two strategies.

SPOT showed a possibility to operate better in different degree of congestion, as it performed best at all times of day with some exceptions at the oversaturated main intersection.
A general comparison of both buses and vehicles, with buses given a weight of 20 or 25 vehicles, resulted in a greater reduction in travel time with SPOT than with PRIBUSS.

5.3 CONCLUSIONS
In the main part of the study, comparative experiments with different signal control strategies, were evaluated using a combination of field data collection methods including floating car and stationary video recording. Approach delay was calculated based on the observed number of queuing vehicles at the start and end of green (overflow). An important requirement for empirical evaluation is that the traffic demand and other conditions should remain generally unchanged during the whole study period, which was almost true in this study. It would also have been interesting to use micro-simulation to make “controlled experiments” with different strategies. This was not, however, possible due to the lack of a functional emulator for the SPOT system as it was applied in Stockholm at the time the study.

The field surveys showed that the PRIBUSS and SPOT strategies reduced bus travel time both in the network and at the main intersection, but at a rather high expense to other vehicle traffic on the main intersection cross street. In the network, SPOT performed better than PRIBUSS in both bus and vehicle travel time, while PRIBUSS performed better than FTC in bus travel time only.

Since manually designed signal time plans were used as the basis for FTC-LTA and PRIBUSS, the performance of these strategies might have been improved if the timing had been optimized using TRANSYT instead. Furthermore, the results indicate that the main intersection had a high degree of saturation, which made it very sensitive to local signal timing adjustments or bus priority; it was best served by FTC.

5.4 COMPARING RESULTS FROM SIMULATION WITH FIELD MEASUREMENTS
Comparable results from simulation and field measurements are available for the main intersection stops and delay with FTC-LTA_{man} during non-peak traffic. The field result for proportion of stops was 49.4%; the corresponding simulation result with HUTSIM was 55.8%. The field result for mean delay was 16.1 s/veh; the simulation result was 18.5 s/veh. It has been mentioned in part III of this thesis that HUTSIM does not totally reflect the actual on-street situation. In reality, parked vehicles, bicycles and pedestrians often reduce traffic performance.
Part V

Synthesis and Conclusions
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1 BACKGROUND

The number of intersections in Sweden controlled by traffic signals has increased since the seventies, but no efforts have been made to study traffic performance and safety impacts of the strategies used.

The LHOVRA technique is the predominant strategy for isolated traffic signal control in Sweden. It was originally developed to increase safety and reduce lost time and the number of stopped vehicles at signalized junctions along high-speed roads (70 km/h or more). LHOVRA has been shown to be effective in such environments. However, not enough is known about its impacts for the control of intersections in urban environments.

Urban traffic signal control systems in Sweden are often coordinated with fixed time plan selection with the purpose of obtaining “green waves” to reduce the number of stops along signalized routes. Some municipalities use the offline TRANSYT software for signal timing of such systems. In Stockholm, signal timing is normally performed manually taking into consideration local adjustments based on detector inputs controlling the termination of the green signal.

Self-optimized signal control is an alternative for both isolated and coordinated systems, but so far there are very few applications in Sweden. The introduction of such systems requires new knowledge and resources for equipment and installation. Therefore, it can sometimes be more profitable to improve an existing traffic signal strategy than to replace it.

2 METHODS FOR EVALUATION OF TRAFFIC SIGNAL CONTROL STRATEGIES

In part I of this thesis three main methods for estimation of traffic performance measures were defined:
1. Empirical methods (field measurements);
2. Analytical methods;
3. Simulation models.

Study and experience have shown that
1. Field measurements are very time consuming and expensive;
2. Analytical methods can be applied for fixed-time control of isolated intersections and then adjusted for vehicle-actuated and coordinated systems;
3. Simulation offers a useful offline test environment in which changes in detector positions and signal controller logic can be made quickly without jeopardizing the safety of road users.

Field observations were used to study driver behavior. Field measurements were also used for studying the traffic impacts and for collecting the input data needed for simulation. Simulation models calibrated and validated with empirical data were used to study traffic performance and safety impacts of existing control strategies and to test some suggested improvements.
3 DRIVER BEHAVIOR AND THE LHOVRA TECHNIQUE

The study in part II combined empirical methods with simulation to analyze and improve the performance of the “O” function within the LHOVRA technique. The field study showed that the actual dilemma zone varied considerably, and thereby the effectiveness of the “O” function, depending on the specific speed distribution and the location of the detectors in the approaches.

Field studies at three intersections with a 50 km/h speed limit showed that vehicles in the lower speed range, when facing a change from green to amber, had a higher likelihood of stopping before the stop line. These values concern all the three intersections, which all had large variations in vehicle approach speeds. The mean speed of all vehicles in the approaches show that there is a large variation between the intersections. This resulted in a large variation in the parity and length of the dilemma zones (beginning at 70 to 22 m) from the stop line.

The “O” function consists of an extra green extension known as Past-End Green (PEG) after the end of maximum green time. This extension allows the vehicles in the dilemma zone to be evacuated without the stress to make the decision to “pass or stop”. While these vehicles evacuate, others may arrive and, depending on the time gap, extend the PEG further. In cases where traffic is intense, the gap between the vehicles becomes smaller. Then the PEG does not perform as it is supposed to, i.e. evacuate vehicles in the dilemma zone, but is just a regular green extension. Therefore, it was found advisable to integrate the PEG in the maximum green time, in order to reduce delay without impairing safety.

Field observations showed that speed at some approaches was low, thereby making the functionality of PEG doubtful. Looking at the number of stops after receiving the green extension, it was clear that the low speed approaches had a higher share of stopped vehicles. The detectors’ locations had been designed depending on the actual approach speeds, but this was not enough to fit the real dilemma zone and optimize the performance of the “O” function.

4 IMPROVING THE INCIDENT REDUCTION FUNCTION

The empirical study showed that the results of using the “O” function were different depending on the real speed at the approach and the detector location. A theoretical study of the “O” function was conducted with a focus on approaches with 70 km/h posted speed.

Extension of green time by past-end-green is triggered by the existence of a vehicle in the zone. Preferably, however, a change should not occur when this vehicle is within the dilemma zone as long as the maximum past-end green is not exceeded. This reduces the risk for rear-end collisions as well as change-induced stress in the approach and reduces the possibility of red light infringement. These objectives can be achieved by other functions in the LHOVRA technique, and using the “O” function for many objectives weakens the argument for the main condition of “past-end green”.
The position of the two detectors in the approach (at 130 and 80 meters) is dimensioned to allow vehicles traveling at 70 km/h to proceed through the intersection. Vehicles moving at lower speeds, which get determined extension, may find themselves in their own dilemma zone at the start of amber, while they could easily have stopped before the stop line if no extension had occurred.

To ensure safety without compromising traffic performance, three suggestions were discussed:

- Changing the detector location and the green time extension;
- Using double detectors at the beginning of the dilemma zone;
- Considering the headway between two existing vehicles in the dilemma zone.

A micro-simulation experiment based on field data was performed to test the first two suggested improvements, which are moving the detectors closer to the stop line (110 and 65 meters instead of 130 and 80 meters) and making the detectors speed dependent (the minimum speed to get PEG is 56.5 km/h).

The results of the two suggested design changes were shown to reduce accident risk with little or no loss of traffic performance in comparison to the existing incident reduction function.

There are reservations as to how far the results of this study can be generalized to real-world scenarios given the fact that this experiment is based on simulation. More work is needed to test other models based on newly collected field data using this methodology in order to test their robustness.

5 LOCAL TRAFFIC ADJUSTMENT WITHIN FIXED TIME COORDINATION

After its initial success, LHOVRA has been widely used in Sweden and the other Scandinavian countries in urban areas with a (50 km/h) speed limit. More recently, the “O” function has also been adapted to allow vehicle-actuated past-end-green in fixed time coordinated systems, the intention being to improve traffic performance as well as to increase safety. In Stockholm, the timing of signals is normally performed manually taking into consideration local signal timing adjustment based on detector inputs controlling the termination of the green signal by using PEG, which affects the split of green time, but not the cycle time or the offset between the intersections. PEG may come in one or more directions in both main and crossing streets.

The following control strategies were studied in part III of this thesis in order to evaluate local traffic-actuated signal-timing adjustments:

- Fixed time coordination without local signal timing adjustment (FTC);
- Fixed time coordination with local signal timing adjustment (FTC-LTA).

The TRANSYT model was used purely to generate optimized signal timings for input to micro-simulation using the HUTSIM software. The results of the studied signal settings were shown that local traffic adjustment with manual FTC had little effect while using it with signal timings determined by TRANSYT reduced the average intersection delay considerably.
This study highlighted the effectiveness of TRANSYT in producing fixed signal timings for coordinated intersections. HUTSIM simulations showed that local signal timing adjustment by means of past-end green, originally designed to improve safety and traffic performance at high-speed isolated intersections, was beneficial when applied to coordinated traffic signal control in the study area. Both delays and stops were reduced, although not for the main, critical intersection which operated close to capacity.

6 IMPACTS OF COORDINATED SIGNAL CONTROL WITH BUS PRIORITY

Bus priority in Swedish urban traffic signal systems is normally associated with fixed time plan selection. Within this framework, local traffic-actuated signal timing adjustments are applied based on detector inputs with the aim of reducing the number of vehicles in the dilemma zone. Active bus priority is also achieved with the aim of displaying a green signal on the arrival of the bus at the stop line. The study in part IV was undertaken to obtain better knowledge about traffic performance impacts of the following control strategies, using Stockholm as a case study:

- Fixed time coordination with local signal timing adjustment (FTC-LTA);
- FTC-LTA as above with active bus priority (PRIBUSS);
- Self-optimizing control (SPOT) with active, weighted bus priority.

Extensive field data collection was undertaken during separate time periods with these strategies in the same area using mobile and stationary techniques. It would also have been interesting to use micro-simulation to conduct “controlled experiments” with different strategies. However, this was not possible due to the lack of a functional emulator for the SPOT system as it was applied in Stockholm at the time of the study.

The average cycle time was very similar for all tested strategies; the main difference was that SPOT allocated more green time to the crossing street. Each strategy had a different range of signal timing variations, with SPOT being the largest and FTC-LTA the smallest.

Traffic performance results were presented from the field measurements in two parts:

- In the main intersection;
- In the whole network.

A method to calculate the approach delay was developed based on the observed number of queuing vehicles at the start and end of green (overflow).

Results for the main intersection showed that the reduction in bus travel time with both SPOT and PRIBUSS increased the delay of other vehicles as compared to FTC-LTA. Performance under different degrees of saturation at different times of the day varied considerably. Bus priority with PRIBUSS imposed a higher penalty on other vehicles in the oversaturated conditions. On average, FTC-LTA had the longest bus travel time and the shortest delay, while PRIBUSS and SPOT had almost identical average bus travel times.

In the whole network, SPOT performed best at all times of the day with some exceptions at the oversaturated main intersection. A general comparison for both buses and vehicles with
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buses given a weight of 20 or 25 vehicles resulted in a higher reduction in travel time with SPOT than with PRIBUSS.

Manually designed signal time plans were used as the basis for FTC-LTA and PRIBUSS. The performance of these strategies might have been improved if the timing had been optimized using TRANSYT instead. Furthermore, the results indicated that the main intersection had a high degree of saturation, which made it very sensitive to local signal timing adjustments or bus priority; it was best served with FTC.

Comparative results from simulation and field measurements (parts III & IV) are available for the main intersection with the manual FTC with LTA under non-peak traffic. The HUTSIM simulation results of stops and delay were much higher than the corresponding field results. It should be noted that HUTSIM does not totally reflect the real on-street situation. In reality, parked vehicles, bicycles, and pedestrians often reduce traffic performance.

7 FUTURE RESEARCH AND DEVELOPMENT NEEDS

Traffic signal control, although being one of the most cost-effective methods to improve traffic performance and traffic safety in urban street networks, is an area where much research is needed to overcome the deficiencies of current practices in Sweden:

- The LHOVRA technique, which is the most common control method for isolated intersections in Sweden, is of the enhanced vehicle-actuated control type without self-optimization, automatic data collection, and processing capabilities for self-adjustment;

- Coordinated signal control systems in Sweden are generally fixed-time based with local green-time adjustments providing possibilities for adjustment to local flow variations and bus priority. Although the systems are normally equipped with many detectors, these are not used to support a more intelligent, self-optimizing control, taking into account all approaches and queues developing on the street network.

Modern micro-simulation techniques calibrated with observed driver behavior parameters now provide an opportunity to develop and test more advanced signal control strategies. The work conducted in the present study has demonstrated the potential of this method. Furthermore, the availability of emulators linking simulation software to signal controller hardware facilitates the very complex tasks involved in this process.

The basis for the development of signal control strategies and their implementation should be to provide strategies with self-optimization functionality. This implies an ability for the traffic engineer to apply traffic policy objectives to the relative weight of different traffic elements (by providing priority on the basis of criteria) and impacts (traffic performance, traffic safety, and emissions).
Glossary of Terms
Glossary of Terms

**Active Green** = Situation in which green is shown in accordance with an identified demand for green.

**Automatic vehicle location (AVL)** = AVL systems are used to help give buses priority. With AVL, buses are fitted with equipment to determine their location. A bus transmits its identifier and location by radio when it is polled (interrogated) from the central unit.

**Bus priority** = Bus priority is a facility whereby buses can receive priority benefits at controlled traffic signals to reduce delay. Buses can be given extensions where the green is extended, allowing the bus to continue, or recalls where a bus is detected on a red causing other stage times to be reduced in order to bring forward the green onto that link.

**Capacity** = The maximum sustainable flow rate at which vehicles or people can reasonably be expected to traverse a point or a uniform segment under given conditions.

**Congestion** = Congestion is defined as the time intervals in seconds (4 seconds in SCOOT) in a cycle during which the detector is continuously occupied. It is expressed as a proportion of the cycle time.

**Coordinated signal control** = The situation in which a signal controller operates in coordination with other signal controllers at the same or different intersections in the neighboring area. The goal of this coordination is to optimize traffic performance (number of stops, delay times etc).

**Cruise speed** = The speed of a vehicle’s travel from one stop line to the next without seeing a red signal.

**Cycle / cycle time** = The length of time in seconds for a complete stage sequence to run before it is repeated.

**Cyclic flow profile** = Shows the traffic demand over a traffic signal cycle.

**Degree of saturation** = Ratio of traffic flow to capacity.

**Delay** = The additional travel time experienced by a driver, passenger or pedestrian. Vehicle delay at an intersection can occur for two reasons: traffic delay and geometry delay.

**Demand** = An expression used to represent the number of vehicles on a link. The upstream detector measures demand as vehicles pass, creating a cyclic flow profile.

**Detectors** = Physical devices for detecting the presence or passage of road users at a particular location. Several detectors can be used to derive speed and headway information. Detectors are used to provide traffic demand input data to signal controllers.

**Dilemma zone** = The area upstream of the stop line in which a driver, on seeing the amber light, may not be able to stop in advance of the stop line with an acceptable deceleration rate, or clear the intersection during the change interval.

**Dispersion** = Describes the action of platoons spreading out as they move along a link.

**Effective green** = The time during a green period where traffic can flow at saturation flow through the junction.

**End lag** = The normal end delay for traffic, typically 3 seconds. This is the time between the stage terminating (lights turning amber) on-street and the end of effective saturation flow.
Fixed amber time = Amount of amber time that is always given during a signal cycle following the end of green time.

Fixed red time = Amount of red time that is always given during a signal cycle following the end of amber time.

Fixed time signal control = Form of signal control whereby all signal groups are given signal times that are fixed according to a predefined signal timing plan.

F-reduction = An important conditional part of the "O" function that ensures sufficient past-end green time is available for a vehicle to pass the stop line before it is given.

Gating = Action at a distance. It can be used to restrict traffic from entering or leaving a critical area.

Geometric delay = The component of delay that results when geometric features cause vehicles to reduce speed in negotiating a facility.

Green-extension time = Provision of additional variable green time in accordance with the prevailing traffic demand and remaining green time (maximum green time) available.

Isolated signal control = The situation in which a signal controller operates independently (i.e. in isolation) of other signal controllers in the neighboring area.

Incident = Any event which reduces the capacity of a network link to carry traffic.

Inter-green = The time in seconds between the FROM stage and the TO stage. In other words, the time between the end of green on one stage to the start of green on the next.

LHOVRA = An acronym (in Swedish) that represents a collection of special additional functions to enhance traffic performance and safety at vehicle actuated signal controlled intersections.

"L" function = A function that is used to give mainline priority to heavy goods vehicles and/or buses.

"H" function = A function that is used to give greater priority to mainline traffic.

"O" function = A function that is used to reduce incidents that result from varying stop/go behavior following the change from green to amber for drivers who find themselves in the so-called dilemma zone.

"V" function = A function that is used to give variable amber time over and above the amount of fixed amber time (i.e. an extension) where it is needed in order to reduce the amount of unnecessary amber time given in each cycle.

"R" function = A function that is used to reduce accident risk in relation to deliberate red-light driving and reduce the risk for conflicts between vehicles in opposing flows of traffic controlled by conflicting signal groups. This gives the impression of safer signal behavior.

"A" function = A function that is used to allow an immediate change to green from amber or red for a particular signal group if there is a recognized demand on the approach in question and no demand on any of the other conflicting approaches. This avoids confusion and saves turn-around cycle time by, for example, eliminating the need to display red-amber.

Link = Length of road from determined section / detector to the stop line.

Loop = Inductive loop used for detecting traffic often used as an alternative term for detector.
Glossary of Terms

**Maximum green time** = Amount of additional variable green time that is conditionally given during a signal cycle provided that there is continued traffic demand and within the determined green-time limit.

**Minimum green time** = Amount of fixed green time that is always given during a signal cycle provided that there is traffic demand (motorized or pedestrian).

**Network** = In TRANSYT, the layout of nodes and links in a region; the network of roads controlled by the system.

**Node** = A node is a set of traffic signals (equivalent to an intersection).

**Offset** = The offset on a link is the time between the start of green to the main feeder link at the upstream link to the start of green to this link at the downstream node.

**Overflow queue** = The queued (see queue) vehicles left over from the analysis period due to demand exceeding capacity.

**Passive Green** = Situation in which green is not shown in accordance with an identified demand for green on the current approach but as the result of a continued demand at another signal in the same signal phase. It is possible for the signal group to become active from a passive state through the use of the "O" function.

**Past-end green** = Time during which the normal signal change from green to amber is delayed as a result of the incident-reducing "O" function.

**Platoon** = A group of vehicles or pedestrians traveling together as a group, either voluntarily or involuntarily, as a result of signal control, geometrics, or other factors.

**Proportion of stopped vehicles** = Ratio of flow forced to come to a complete standstill before crossing the stop line due to the signal control.

**Red-amber time** = Amount of red-amber time that is always given during a signal cycle prior to the onset of green time.

**Saturation flow rate** = The equivalent hourly rate at which previously queued vehicles can traverse an intersection under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced.

**Signal controller** = Hardware device that physically controls a number of signal groups at a signaled intersection in accordance with predefined logic.

**Signal group** = Logical term for a collection of signal heads that display the same signal information at the same time.

**Signal head** = Hardware device that displays signal information to road users.

**Signal phase** = A combination of directional traffic flows that are simultaneously given green time at a signal controlled intersection. Phases can be in a predetermined phase order, cyclic, or acyclic.

**Signal timing plan** = Description of signal and detector functionality for all signal groups controlled by a signal controller at a particular intersection. This includes all signal times, extension times, past-end green time relative to particular signal groups and detectors, a description of additional LHOVRA functionality, a description of signal phases, and a signal conflict matrix that identifies conflicting signal groups (regulation of conflicting traffic flows) and the safety times between such groups.

**Split** = The proportion of green time given to a particular stage within a cycle.

**Stage** = Traffic signal stages consist of an inter-green followed by a green period.
Start lag = The normal start up delay for traffic, typically 2 seconds. This is the time between the lights turning green on-street and the start of effective saturation flow.

Stop rate = Average number of stops per vehicle (including multiple stops in a queue).

Stop time = A proportion of control delay when vehicles are at a complete stop.

Time Exposed TTC = A safety indicator that entails accumulating the total time spent in safety critical events, i.e. the time during which TTC values are below a predetermined TTC threshold in safety critical TTC events.

Time Integrated TTC = A safety indicator that entails accumulating the integral of the TTC profile below a predetermined TTC threshold as a measure of both the time spent in safety critical events and the level of severity.

Time-to-Collision (TTC) = A safety indicator that represents the shortest time-gap between two vehicles that are on a collision course given their speeds and distances relative to a common conflict point.

Traffic delay = The component of delay that results when the interaction of vehicles causes drivers to reduce speed below free-flow speed.

Traffic flow (Flow rate) = The equivalent hourly rate at which vehicles etc. pass a point on a lane etc (computed as number of passing vehicles divided by the time interval).

Traffic model = Models are used in SCOOT to help estimate the traffic on-street. Demand is measured at a detector using a cyclic flow profile. SCOOT uses this information to model the demand at the stop line (before the traffic gets there).

Transponder = In bus priority, transponders can be fitted to buses so that when a bus travels over a bus detector on-street or passes a beacon, a controller knows that a bus is approaching a signal.

VA = Vehicle Actuation is a method of traffic signal control in which the duration of the green signal is extended, up to a maximum, according to traffic flow.

Variable amber time = Amount of additional variable amber time that is conditionally given during a signal cycle provided that there is a recognized need in accordance with the “V” function.

Variable red time = Amount of additional variable red time that is conditionally given during a signal cycle provided that there is a recognized need in accordance with the “R” function.

Vehicle actuated signal control = Signal control in which signal groups can have variable signal times that are allocated in accordance with road user demand, a signal timing plan, an internal predefined logic, and a signal control strategy.

Queue = A line of vehicles waiting to be served by the system. Slow-moving vehicles joining the rear of the queue are usually considered as being part of the queue.
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