TECHNICAL UNIVERSITY OF CRETE
DEPARTMENT OF PRODUCTION ENGINEERING AND MANAGEMENT

USE OF AYTOMATIC INCIDENT DETECTION WITHIN COORDINATED TRAFFIC SIGNAL TIME DETERMINATION

<u>DIPLOMA THESIS</u>

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CONTENTS

ABBREVIATIONS

AID	Automatic Incident Detection
AIDA	Automatic Incident Detection Algorithm
BOStrab	Betriebsordnung für Straßenbahnen
HGV	Heavy Good Vehicles
LOS	Level of Service
p.c.u	passenger car units
StVO	Straßenverkehrs-Ordnung

INTRODUCTION

The Automatic Incident Detection Algorithm (AIDA) implemented in Munich on several main arterial, is one of the few existing automatic incident detection algorithms for urban areas. At this project, the AIDA is integrated into a traffic optimization tool. This is achieved by using the available information from the existing detectors regarding a random incident. The purpose of this thesis is to calculate the offset times of traffic signals of consecutive intersections within parts of the road network where traffic coordination is implemented. In addition, the thesis develops some suggestions for the traffic improvement in a non-incident case.

The first three chapters are introductory in order to give the basis for the development of the above ideas. The first chapter includes the basic principles of intersection signalization and signal coordination, which is the field of all optimization actions. In the second chapter the traffic signal optimization algorithms as the Combination method, TRANSYT, SCOOT and VERON are described, whereas in the third chapter the incident characteristics and AIDA are presented. Furthermore, the last three chapters represent the innovative section of the thesis presented here. More specifically, in chapter four, two new traffic coordination algorithms are described. The first one takes into consideration only the speeds of the vehicles moving in the main road while the second one takes into consideration the saturation flow as well as the delays in an intersection as in TRANSYT and SCOOT algorithms. In chapter five suggestions for the traffic optimization in case of non-incident are given and finally in the last chapter some remarks and comments on the applicability of the presented ideas are made.

CHAPTER 1

BASIC PRINCIPLES OF INTERSECTION SIGNALIZATION AND SIGNAL COORDINATION

1.1 General

Traffic signals are an important tool for managing road traffic, particularly as it is rarely possible nowadays to provide sufficient road space despite increasing traffic demands. Therefore attempts are made to improve the circulation of traffic in a town by reducing traffic jams and by increasing the steady flow of traffic, which results in an increase in the average speed of vehicles.

As traffic signals directly influence the traffic flow, in so far as traffic streams with shared but conflicting paths are alternately stopped or let through, they should be designed, built and operated with particular care.

The design of traffic signals includes choosing the method of control, describing the control phasing, calculating signal timings and designing the geometrical layout of the intersection, street or the relevant part of a network, including the traffic management[23] measures that should accompany them.

The individual components e.g. constructing the intersection, organizing the incoming lanes[15] into traffic lanes, managing pedestrians and cyclists, and signaling individual traffic streams should be co-ordinated in such a way as to ensure a safe flow of traffic whatever demands and operating conditions might occur. The intersection layout, traffic management and signaling should be considered as a single entity.

In urban areas, traffic signal control decisively influences traffic management within the entire road network. It is therefore an important tool within the framework of a co-ordinated traffic concept, through which measures for public transport pre-emption, for directing pedestrians and cyclists safely, for platooning motor vehicle streams on particular routes are integrated. Implementing comprehensive traffic management ideas is termed Traffic System Management. It involves systematically influencing and controlling the type and quantity of traffic flowing into a town, the role of the public transport system, the management of commercial traffic and of parked vehicles, or air pollution caused by traffic. The term expresses the fact that managing complex traffic problems is a supervisory task and it requires considerable organizational effort to co-ordinate the many and diverse measures used. Traffic signal control occupies an important place in Traffic System Management.

1.2 Traffic Signals and Signal Sequences

In Germany, traffic signals for motor vehicles have the signal sequence GREEN-AMBER-RED-RED and AMBER (simultaneously)-GREEN. In special cases, where the traffic signals are operated only at intervals, the signal sequence OFF-AMBER-RED-OFF is acceptable. A green arrow can be shown on the far left hand side of the crossing for those turning left when the oncoming traffic is stopped at the red light. It can suffice in special cases to indicate a priority time to those turning right by means of a single signal phase[19] with a green arrow. Motor vehicle signals apply to all other forms of traffic required to use the carriageway unless they are signaled separately.

Traffic signals for pedestrians have the signal sequence GREEN-RED-GREEN.

Cyclists do not usually require special signals and be directed together with motor vehicle traffic or pedestrians. Where separate signals are provided for cyclists, the same signal sequence is used as for motor vehicles.

Public transport systems (urban railways, trams, bus services) are provided with special traffic signals with a special sequence as defined in BOStrab (Operating regulations for light rail) unless they are signaled together with motor vehicle traffic signals.

In case of danger, a flashing amber light may be installed to warn. Where symbols are used, only black symbols on an illuminate amber background of type given in the StVO (Street and Highway Regulation) are permitted.

1.3 Criteria for the Use of Sets of Traffic Signals

As a rule, traffic signals are installed to increase traffic safety or to improve the quality of traffic flow.

Because the demands of individual groups of road users can be partly contradictory, and because partly conflicting aims can occur, no quantitative criteria for justifying the installation of traffic signals can be given in these guidelines. However, there are recommended procedures for establishing priorities using, amongst others, the following criteria:

- the number and severity of accidents,
- the line of sight on access roads to intersections,
- requirements for the safety of pedestrians and cyclist,
- the volume of vehicular traffic on the main road and on the side roads,
- the management of public transport,
- the traffic flow for pedestrians and cyclists,
- directing motor vehicles within the road network,
- protecting road networks from overloading,
- damage to the environment.

The special requirements of police and emergency rescue vehicles can also justify the installation of traffic signals.

Finally, it should be noted that traffic signal has also an influence on motor vehicle fuel consumption, exhaust and noise emissions and on motor vehicle speeds within urbanished areas [8, 21].

1.4 Signal Coordination

In situations where signals are relatively closely spaced, it is necessary to coordinate their green times[10] so that vehicles may move efficiently thought the set of signals. It serves no purpose to have drivers held at one signal watching wasted green at a downstream signal, only to arrive there just as the signal turns red.

In some cases, two signals are so closely spaced that they should be considered to be one signal. In other cases, the signals are so far apart that they may considered independently. However, vehicles released from a signal often maintain their grouping for well over 300 meters. Common practice is to coordinate signals less than 800 meters apart on major streets and highways[11] [18].

For the coordination of two or more intersections[13] a master controller is usually used in order to coordinate the local controllers at each signalized node.

In that way, not only the traffic signal indications between different nodes are coordinated, but also pre-determined traffic signal indication changes can be done at each node from the point where the master controller has been installed [9].

1.4.1 Factors Affecting Coordination

There are four major areas of consideration for the engineer considering signal coordination:

1. Benefits
2. Purpose of signal system
3. Factors lessening benefits
4. Exceptions to the coordinated scheme

It should be noted first that except for the most complex coordination plans require that all signals have the same cycle length. While some signals might hold stopped vehicles for longer than they have to for strictly local purposes, the overall effect will be beneficial. If the overall effect is not beneficial, then the coordination serves no purpose.

In order to better understand some of the discussion, refer to Figure-1. This figure illustrates the path (trajectory) that a vehicle take as time passes. At $t=t_1$, the first signal turns green. After some lag, the vehicle starts and moves down the street. It reaches the second signal at some time $t=t_2$. Depending upon the indication of the signal, it either continues or stops.

The difference between the two green initiation times is referred to as the signal offset, or simply as the offset. In general, the offset is defined as the difference between green initiation times, measured in terms of the downstream green initiation. In Figure-1.1 it is t_2 minus t_1.

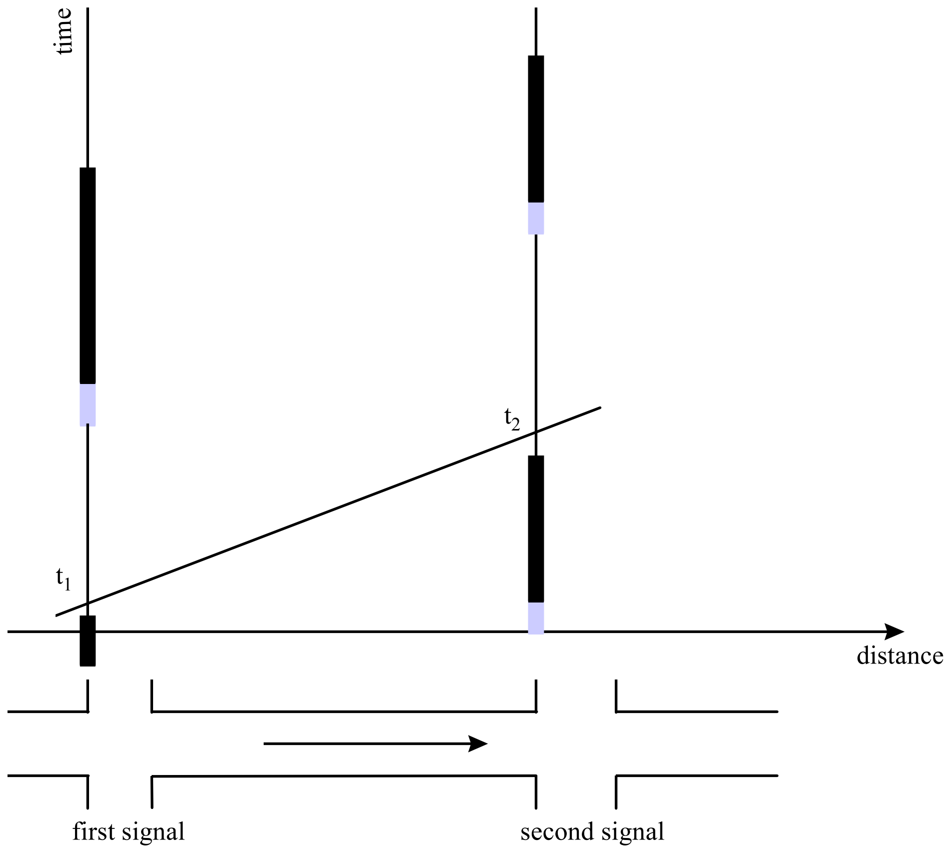

Figure-1.1: Time space diagram

1.4.1.1 Benefits

The prime benefit of coordination is improvement of service provided, usually measured in terms of stops and delay.

It is common to consider the benefit of a coordination plan in terms of a “cost” or “penalty” function: a weighted combination of stops and delay, plus perhaps other terms:

cost =A×(total stops)+B×(total delay)+other terms **(Eq.-1.1)**

The object is to make this disbenefit as small as possible. The weights A and B are coefficients to be specified by the engineer or analyst.

The coefficients may be selected on the basis of a judgment of how important the two are to the public. For example, perhaps one stop is as bothersome as 5 seconds of delay, so that A=5B.

The values of A and B may also be selected so as to reflect the estimated economic cost of each stop and delay. The amounts by which various timing plans reduce the cost shown in Eq.-1 can then be used in a cost-benefit analysis to evaluate alternative plans.

In practice, numeric values of the improvement in stops and delay are usually obtained only with timing plans done with signal-optimization computer packages, such as the TRANSYT program. For those done manually, the engineer usually tries to make the number of vehicles stopped as small as possible, or tries to minimize delay. This is usually acceptable.

The conservation of energy and the preservation of the environment have grown in importance over the years. Given that vehicles must (or will) travel, fuel conservation and minimum air pollution are achieved by keeping vehicles moving as smoothly as possible at efficient speeds. This can be achieved by a good signal-coordination timing plan.

Another benefit of signal coordination is the maintenance of a preferred speed. The signals can be set so as to encourage certain speeds: vehicles going much faster than this design speed will only have to stop frequently.

The fact that vehicles can be sent through successive intersections in moving platoons[20] is also benefit. In a well-formed platoon, the time headway between vehicles is generally somewhat shorter than can be achieved when they start from a stop. This is true despite their greater speed, leading to a more efficient use of the intersection.

It also possible with good coordination to stop fewer vehicles. On short blocks with heavy flows, this is particularly important, for if all vehicles are stopped, the queue that results may overflow the available storage (the space available to store vehicles).

1.4.1.2 Purpose of Signal System

Usually the physical layout of the street system and the major traffic flows determine the purpose of the signal system.

First, one must consider the type of system, one-way arterial[1], two-way arterial, one way, two-way, or mixed network. Although the existing system is a good starting point, it may sometimes happen that -for any reasons- the best solution is still not satisfactory. The engineer will then have to consider changing some streets. This must be done with sensitivity to capacities in both directions and many other issues.

Next, one must consider the movements to be progressed. On a two-way arterial, one or both directions may be progressed (that is, given the advantage of the coordination). If both are to be progressed, there will generally have to be some compromise between the two.

It is necessary to set an objective: for what purpose are the signals to be coordinated? The common objectives include maximum bandwidth ("windows" of green for traveling platoons), minimum stops, and minimum combination of stops and delay.

Last, it is necessary to recognize that there are described below under "exceptions".

1.4.1.3.Factors Lessening Benefits

Among the factors limiting the benefits of signal coordination are the following:

- Inadequate roadway[22] capacity[3]
- Existence of substantial side frictions, including parking, loading, double parking, and multiple driveways
- Complicated intersections, involving multiphase control
- Wide variability in traffic speeds
- Very short signal spacing
- Heavy turn volumes, either into or out of the street.

Heavy turn-out volumes may impede platoons or destroy their structure by the loss of vehicles from the middle of the platoon. Left-turn volumes may interfere with platoons heading in the other direction.

1.4.1.4 Exceptions to the Coordinated Scheme

It is misleading to thing that all signals may easily be coordinated. Very often an intersection is sitting right in the middle of the traffic coordinated system, requiring four phases and a 120-sec cycle[4] length. Rather than require the entire system to be 120 sec, the traffic engineer elects to think of two separate systems, one on each side of this troublesome intersection. Perhaps the engineer can set the progression at 60 or 90 sec, so that the problem intersection may be at some multiple of the system.

Another situation that arises is that there is one intersection which cannot handle the volumes delivered to it at any cycle length[5]. This is refereed to as a critical intersection. Some engineers choose to detach this intersection from the system. Others build the progression around it, delivering vehicles to it in a way that does not cause storage problems in upstream blocks. This last approach requires changing the purpose for which the progression is being designed.

1.5 The Time-Space Diagram and Ideal Offset

The time-space diagram is simply the plot of signal indications as a function of time for two or more signals. The diagram is scaled with respect to distance, so that one may easily plot vehicle position as a function of time. Figure-1.1 is a time-space diagram for two intersections.

The standard convention are used in Figure-1.1 a green signal indication is shown by a blank or simple line ( ), amber by a shaded line ( ), and red by a solid line ( ).

Offset has already been defined as the difference between the green initiation times at two adjacent intersections. More precisely, it is the green initiation time (of the phase of interest) at the downstream intersection minus the green initiation time (of the phase of interest) at the upstream intersection. It is usually expressed as a positive number between zero and the cycle length.

The “ideal offset“ is defined as the offset that will cause the specified objective to be best satisfied. For the objective of minimum delay, it is the offset that will cause minimum delay.

More often, the ideal offset is exactly the offset such that, as the first vehicle of a platoon just arrives at the downstream signal, the downstream signal turns green. It is usually assumed that the platoon was moving as it went through the upstream intersection. If so, the ideal offset is given by

$$t(ideal)=\frac{L}{V} \qquad \textbf{(Eq.-1.2)}$$

where: t(ideal)=ideal offset (sec),

L= block length (m)

V= vehicle speed (m/sec)

If the vehicle were stopped, and had to accelerate after some initial start-up delay, the ideal offset could be represented by (Eq.-1.2) plus some term representing the start-up time at the first intersection.

(Eq.-1.2) will generally be used without an added term for start-up (which might add 2 or 4 sec). Usually, this will reflect the ideal offset desired for maximum bandwidth, minimum delay, and minimum stops. Even if the vehicle is stopped at the first intersection, it will be moving in most of the system.

Note that the penalty for deviating from the ideal offset is usually not equal in positive and negative deviations.[18]

CHAPTER 2

TRAFFIC SIGNAL OPTIMIZATION ALGORITHMS

2.1 Presentation of Some Basic Optimization Algorithms

At this chapter, a brief presentation of some of the existing traffic signal optimization algorithms, is presented.

The simplest method of synchronizing traffic signals is by the use of fixed-time plans in which all the signals run on a common cycle with each stage appearing at a fixed point in the cycle. Before a system of this kind can be put into operation it is necessary to have a method for selecting the signals to be used. In general it has been assumed that the aim is to maximize the bandwidth (the proportion of the cycle for which the vehicle unimpeded by other traffic and traveling at a predetermined speed on each section of the main road, could enter and pass through the system without meeting any of the lights red). A serious disadvantage of this aim is that the bandwidth that can be obtained is almost always insufficient to deal with the amount of traffic that can, and does, pass through the system. Some work has also been done on setting the signals to minimize delay or stops.

The **Combination method** which was presented by Hillier in 1965, is an advance of these methods for setting signals because it applies a rigorous optimization process to a reasonable model of traffic. The method assumes that cycle time, green times, flows and saturation flows are known and then chooses the offsets of the signals to minimize delay over the network. The technique can be applied on an area basis subject to some restraints on the type of network. Its most serious restriction is the assumption it makes that the delay between two signals depends solely on the relative settings of the two signals, which is a good approximation only in heavily loaded conditions.

Another method of automatically determining signals plans is the **TRANSYT** (TRAffic Network StudY Tool) method which first presented in 1967 but up to now several new versions have been proposed. The principle idea of TRANSYT is that the overall impedance to traffic is measured by a performance index that can be chosen with any desired balance between journey time and numbers of stops. The optimization process minimizes the performance index by altering the points within the signal cycle at which each stage starts. In this way, both signal offsets and green times are included in the optimization procedure. Thus, TRANSYT consists of two main elements. The traffic model which is used to calculate the performance index of the network for a given set of signal timings and a 'hill-climbing' optimization process that make changes to the settings and determines whether they improve the performance index or not. The performance index is defined as follows:

$$\text{Performance index} = \sum_{i=1}^{i=n} (d_i + Kc_i)$$

where d_i is the average delay in p.c.u-hours per hour on the i*th* link of the network

c_i is the average number of p.c.u stops per second on the i*th* link

K is a weighting factor.

Finally it is necessary add that the TRANSYT optimization is undertaken off line using historic data [19, 20].

SCOOT (Split, Cycle and Offset Optimization Technique) is also a method of coordination and it could be said that SCOOT is the on-line equivalent of TRANSYT. The first research phase ended in 1975 and integrated in 1981. This method adjusts the signal timings in frequent, small increments to match the latest traffic situation. Data from vehicles detectors are analyzed by an on-line computer which contains programs that calculate and implement those timings that are predicted to minimize congestion.

SCOOT optimization method consists of three main elements, the 'split' optimizer, the offset optimization and the cycle time optimization. As far as the first is concerned, a few seconds before each stage change at every SCOOT intersection is scheduled to occur, the signal optimizer estimates whether it is better to make the change earlier, as scheduled or later. Any decision by the optimizer may alter a scheduled stage change time by no more than a few seconds. The signal optimizer implements whichever alteration will minimize the over saturation degree on the approaches to that junction. It essential that each intersection is treated by the split optimizer independently of other intersections. The second main element, the offset optimizer, estimates once every cycle time, whether or not to alter all the scheduled stage change at the junction. This is accomplished by comparing the sum of the PIs (see above in TRANSYT description) on all adjacent streets for the scheduled offset with offsets that occur a few seconds earlier or later. Whichever alteration gives the minimum PI is implemented by amending the stage change times which are stored for that junction.

Finally, the last SCOOT optimization is the cycle time optimization. As stated above, all signals within a sub-area are operated by SCOOT on a common cycle time. Where SCOOT calculates that there is an advantage, some junctions can be operated on one half of the common cycle time of the sub-area; this is referred to as 'double-cycling' and is of particular value for signal controlled pedestrian crossing [13, 20].

An other offset time optimization method which is implemented on-line is **VERON** method which proposed by Böttger(1972) and integrated as an on-line technique by Pajic(1985). The traffic flow profiles are used in the same way as in SCOOT model. The only difference is that VERON uses smoothed data instead of current data that SCOOT use.

The optimization model of VERON is the same with TRANSYT model where the objective function is function of the sum of the inflow-neighboring-streets waiting times and the number of stops. More specifically, the objective function is:

$$ZF(n,Z,T_0)=GW(n,Z)*W(T_0)+GH(n,Z)*H(T_0)$$

where,

W is the sum of the waiting times

H is the umber of stops

GW is the weighing factor of the waiting time

GH is the weighing factor of stops

the indices mean:

n is the number of intersection

Z is the number of the inflow street and T_0 the inner offset time of the red-beginning at the zero point of the system time [28].

2.2 Comments

As one can see, from the above presentation of the algorithms different methods for the offset time optimization have developed. These methods are composed of two parts. The first part is a traffic model that describes the traffic situation and calculates the waiting times and the number of stops. The second part, is an optimization model that use an objective function which is function of the delays and stops and calculates the best offset time. The optimum offset time is the time that minimizes the objective function of the optimization model [28].

Apart from that, the former two of the algorithms (Combination method and TRANSYT) use off-line data which are taken from an average flow pattern of traffic past a point in the road network. In contrast, the later two (SCOOT and VERON) use on-line data for their calculation.

CHAPTER 3

INCIDENTS CHARACTERISTICS AND AUTOMATIC INCIDENT DETECTION

3.1 Incident Definitions

Traffic Incidents can be defined as any event which causes a reduction in capacity. They could be distinguished between predictable and unexpected incidents; the former consists of, for example, planned roadworks and regular parking infringements, whereas the latter include accidents and vehicle breakdowns [26, 23].

A frequently occurring incident is the congestion ,too. Congestion is defined as the inability of the transportation network to accommodate travel demand. The adverse effects of traffic congestion include undesirable low-speeds, erratic stop and go driving, delay and unpredictable travel times, increased transportation cost, adverse environmental impact.

Congestion can be a recurring or a non-recurring event. Recurring congestion is caused by combined effect of heavy traffic volume and inadequate capacity. This type of congestion is predictable and follows well-defined temporal and spatial patterns, in peak periods in many urban areas or in geometric bottlenecks[2]. Non-recurrent congestion is caused by traffic incidents, which reduces the road capacity and creates perturbation in the traffic [23, 26, 29].

3.2 Incident Related Delay and Its Characteristics

The occurrence of the traffic incidents results in capacity reduction of the roadway segment upstream the incident site, therefore the segment is not capable of serving the demand passing the roadway, thus the excess demand volume is stored in the freeway and traffic backup forms. The resulting incident related congestion continues to extend upstream the incident site until lanes are re-opened, capacity is restored, and incident related queues are dissipated over time and traffic flow returns to normal conditions. The effect of such a situation is an incident related delay.

The formation of traffic incidents delay is illustrated in Figure-3.1, which presents the cumulative input-output curves for estimating of freeway incident delay. The horizontal axis represents time and the vertical cumulative traffic volume (the increasing traffic volume by the addition of vehicles during the incident). The initial traffic flow rate (prior the incident) is represented by the slope of the line AC. When an incident occurs, roadway capacity is reduced and the flow past the incident slows down due to capacity reduction. The capacity reduction depends on the segment affected by the incident, i.e. number and lane width and the severity of the incident.

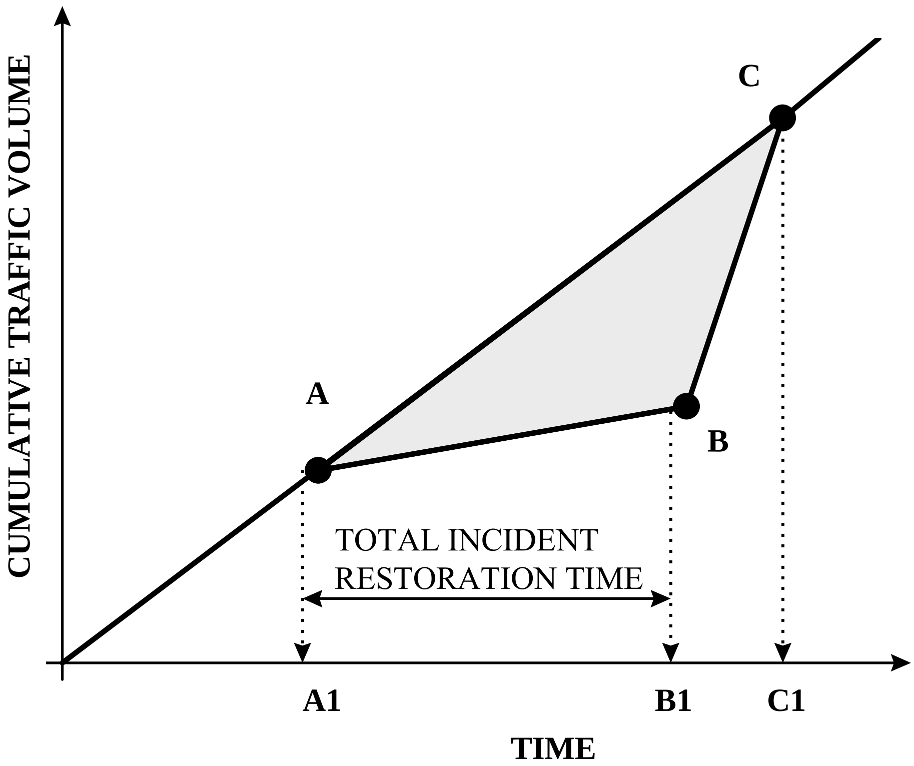

Figure-3.1: Incident related delay calculation diagram

In Figure-3.1, the slope of line AB represents the flow rate accommodated by the roadway segment for the time period between the moment of incident occurrence until the moment the incident has been removed and roadway capacity has been restored.
Right after incident removal, traffic flow rate increases (slope of line BC) until queued and delayed traffic passes the incident site and traffic returns to normal conditions. It is important to mention that the slope of line BC (i.e. traffic incident removal) equals to

the capacity of the roadway segment, therefore the segment operates under capacity until the queue is dissipated and facility returns to normal operational conditions. The area of the triangle ABC represents the incident related delay, therefore a simple analytical method for estimating incident delay is the calculation of the ABC area.

From the Figure-3.1 it is evident that incident delay depends on the following factors:

1. The demand flow at the time of the incident
2. The remaining capacity of the freeway after the incident occurrence, and
3. The total incident duration and the queue dissipation rate after the incident removal and clearance [23].

The model presented in Figure-3.1 corresponds to a simple case where the roadway is partially blocked, i.e. at least one lane is open, no traffic diversion plans have been implemented, i.e. the flow rate before the incident occurrence is equal to the flow rate after the incident removal, and the slope of line BC is constant.

In Figure-3.2 an analytical model for estimating incident delay under more complex conditions has been proposed. This model covers cases such as: (a) short-term closure of the affected freeway, (b) the bottleneck flow rate is increased through effective management of on-coming traffic, and (c) the demand flow rate S_2 is reduced to S_5 by diverting traffic upstream of the incident site.

The estimates incident delay comes from the calculation of the shaded region area of the general conditions diagram. This analytical method for estimating incident delay is simple and effective, however its implementation requires the estimation of the flow rates (S_i's in Figure-3.2) and the various time components (T_i's in Figure-3.2) which are not easily quantifiable, it is unable to capture the dynamic nature of traffic operations and it does not provide the capability of estimating other incident related impacts e.g. environmental impact, cost.

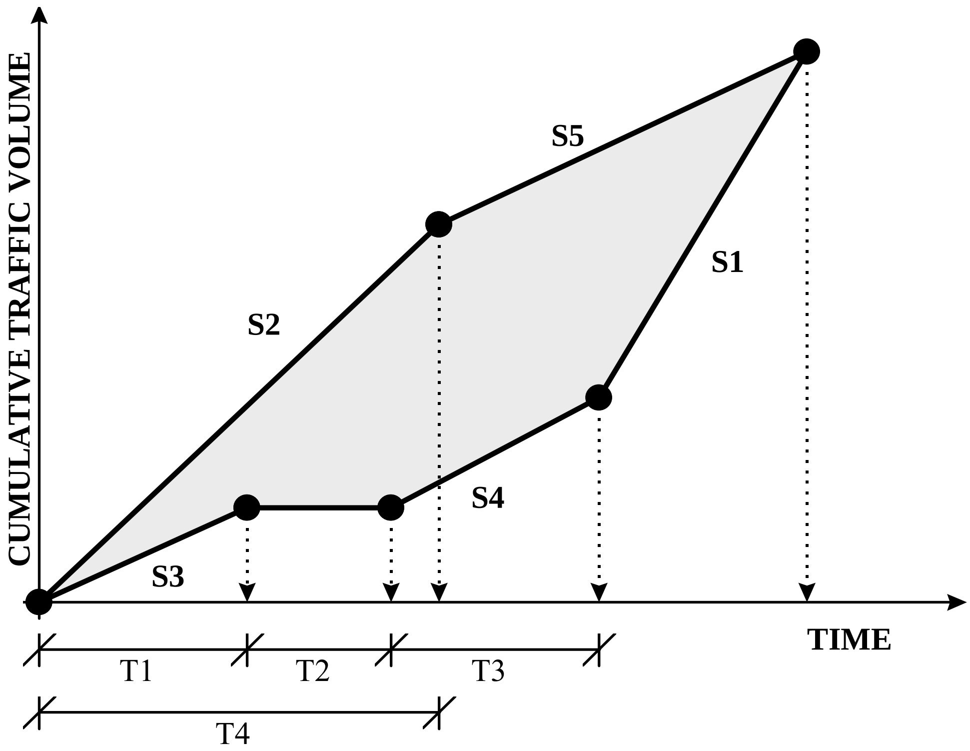

Figure-3.2: General condition incident delay estimation

S1= Capacity Flow Rate

S2= Initial Flow Rate

S4= Adjusted bottleneck Flow Rate

(management of on-coming traffic)

S5= Revised Demand Rate

(Traffic diversion implemented) [23]

T1= Incident Duration until first change

T2= Duration of total closure

T3= Incident Duration under adjusted flow

T4= Time elapsed under initial demand

3.3 Automatic Incident Detection

As it referred before, an incident can increase dramatically the delays in a segment of the road network. So, fast and reliable motorway incident detection[6] is instrumental in reducing traffic delay and increasing safety. In particular, with the information from incident detection, optimal control strategies guide the traffic flow toward smooth operation by preventing additional vehicles from entering upstream of the incident and by communicating traffic information to the travelers. In addition, incident detection constitutes the cornerstone for prompt incident management and safety improvement near the incident location.

Automatic incident detection (AID) involves two major elements: A traffic detection system that provides the traffic information necessary for the detection (actual automatic gathering information systems are almost exclusively based on inductive loop detectors[12]) and the incident detection algorithm which are used in analyzing these data to obtain information on incidents, their cause and consequences. Local presence detectors embedded in the motorway pavement are used extensively to obtain traffic data, primarily on occupancy and volume. Wide-area machine-vision detectors and other detector types can also be used for data collection. Incident detection algorithms can detect capacity reducing incident, and safety reducing incidents [24, 7].

3.4 Automatic Incident Detection Algorithm (AIDA)

The AIDA algorithm is one of the few, today's existing, algorithms for the incident detection in urban areas. It has been developed to detect incidents and identify the traffic situation (Level of Service (LOS)) on the Ingolstädter Straße, as urban arterial of the city of Munich, in order to inform the re-routing system on the motorway network about the adjacent urban traffic situation and to avoid additional traffic entering this area, if it is either already affected by a disturbance[8] or disturbances are expected in case of increased traffic flow. The AIDA algorithm is based on DRIVE I MONICA results with extensive further developments, in particular with respect to the identification of the current and predicted traffic situation for the route recommendation Ingolstädter Straße.

3.4.1 Network

The incident detection, devoloped in the framework of the Munich COMFORT project, mainly covers the Ingolstädter Straße between Neuherbergstraße and Mittlerer Ring. The adjacent area consists of residential buildings mixed with buildings for commercial purposes.

The traffic load on the Ingolstädter Straße, with at least two lanes per direction, is about 17,000 up to 20,500 vehicles per day and travel direction and can be classified as high. Thereby, the big part of HGV (Heavy Good Vehicles) (between 5.4% and 10.8%) is remarkable.

Figure 3.1 shows the test site with the locations of measurement sites and detectors

3.4.2 Hardware: Data Detection and Transmission in the Urban Area

3.4.2.1 Description of the Detectors

With regard to the economy and the preservation of the cityscape, inductive loops detectors are considered as the most suitable data collection facilities. To distinguish the kind of vehicles (passenger car and HGV) and to get the speeds, double inductive loops have been installed.

3.4.2.2 Location of the Detectors

The detectors that the relevant urban road network has been equipped have been placed at: Ingolstädter Straße in in-town direction as well as their flow-out-area Frankfurter Ring, Middle Ring Road and Leopoldstraße.

In contrast to the usual installation for a traffic actuated control, the detectors are located downstream the intersections. This has been done because of the following reasons:

- Downstream the intersections there are less lanes, because there are no turning lanes. This causes less detectors and therefore shows a more cost-saving solution.
- The outflow area of a crossing is rarely affected by a traffic jam. This leads to more reliable data and thus to a more reliable incident detection.

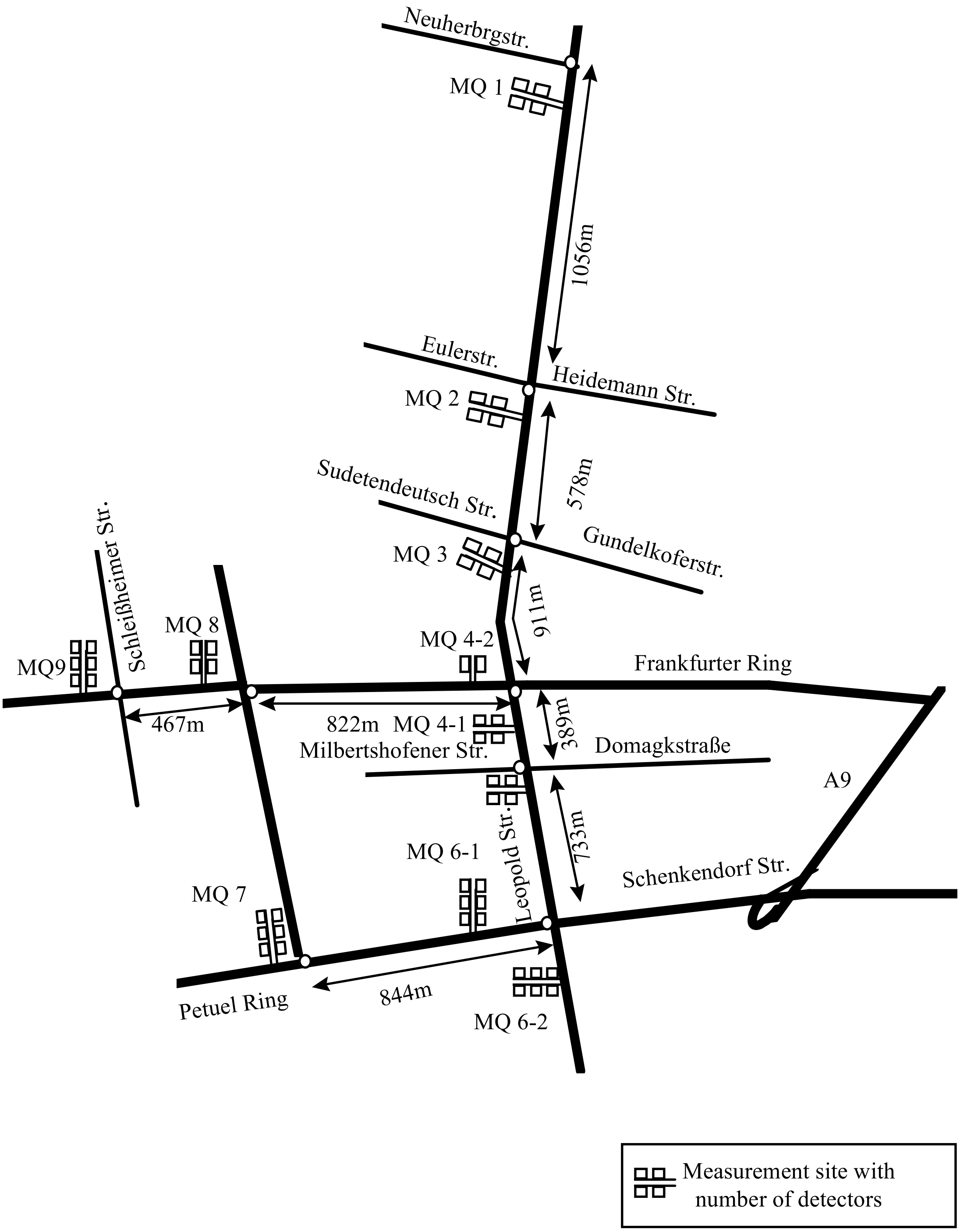

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Figure 3.1: Road Network and Detectors Locations

- If traffic-dependent control will be realized later on, the data can be used additionally for

  -optimizing the control

  -network control and for

  -the usage for an O/D-estimation(origin/destination)

3.4.3 Measurements

The data which were collected by the detectors, are edited and cyclically aggregated in the control unit of the traffic lights. The data format for the AIDA-research computer includes the following information:

a) measuring sites values
   - date
   - time stamp (‘real time’ in sec., derived by the radio clock)
   - number of measuring sites
   - error identification 0 or 1
   - cycle time (in case of breakdown 120 sec.)
   - number of lanes

b) detector values (lane-actuated)
   - green period (end and duration of green period of the observed direction)
   - total number of vehicles
   - total number of HGV
   - harmonized averaged speeds of all passenger cars
   - harmonized averaged speeds of all HGV
   - time of occupancy.

3.4.4 Output

Within AIDA, different possibilities can be selected and visualized for each measuring site.

i) time series of the incident criteria at the levels 1 and 2

ii) the traffic situation of the fundamental-diagram for any day during the measuring period

iii) the time series of the capacity-reserve ΔQ of an junction, in dependence of the current traffic signal program.

iv) finally, a diagnosis can be asked for each link, in which for each of the four levels the indication for a detected incident, congestion or the time series for the capacity usage is shown. For the levels 1 and 2, incidents are shown (upstream and down stream). At level 3, the current speed has fall down n-times below the threshold for congestion, before a congestion alarm is set off. For algorithm 4, the capacity of the junction is shown for the link between the two measuring sites. The capacities are determined by using the cycle and green times.

3.4.5 Methodological Approach

The AIDA program for incident detection consists of the following modules:

Plausibility Examination

Examination of the data input with regard to plausibility, removing (replacing) of invalid data out of the further calculation.

Algorithm for incident detection/ traffic situation analysis

- incident detection, step 1:	early diagnosis of disturbances via a link process[16]
- incident detection, step 2:	incident detection during several measurement periods via a process at measuring site level[18]
- traffic situation analysis:	analysis of the current traffic situation in the fundamental diagram and congestion detection
- analysis of capacity utilization:	analysis of the current capacity utilization at the junctions, determination of the actual reserve of capacity with regard to currency applied signals plans

3.4.5.1 Examination of Plausibility

The following examinations of plausibility per traffic inductive loop are carried out:

* $B_{r,i}$, $Q_{r,i}$ and $V_{r,i} \geq 0$

* $B_{r,i} \leq B_{max,i}$

* $Q_{r,i} \leq Q_{max,I}$

* $V_{r,i} \leq V_{max,i}$

where $B_{r,i}$, $Q_{r,i}$ and $V_{r,i}$ express the occupancy (in seconds), the traffic volume[24] (vehicles per cycle) and the speed (average speed per cycle) at the i*th* measuring site, respectively.

3.4.5.2 Incident Detection, Step 1

This is based on a link process and uses the cut-off at the measuring site downstream as a criterion: there is more traffic flowing in at the upstream measuring site than flowing out at the downstream measuring site. This is the so-called incident detection, step1. The situation changes as soon as the consequences affect the neighboring measuring sites. Therefore step 1 of the incident detection serves the early diagnosis of incidents.

The process considers the turning streams Q_a and Q_z at the observed junctions. An incident is defined by the following condition:

$$\frac{Q_1(t) + Q_1(t-1)}{2} > Q_2(t) + Q_z - Q_a + Qs \qquad \textbf{(eq.1)}$$

The traffic volume at the measuring site down stream is compared with the mean traffic volume of the current and previous signal plan cycle.

The correction value Q_a considers the parts of the stream Q_1, which turn before measuring site Q_2 and do not pass it for this reason. The correction value Q_z considers in the streams, which flow in addition to Q_1 from the neighboring junction arms in the measuring site $Q_.$ Q_z and Q_a are determined, based on existing historical time series for traffic volume, for each junction.

The value Q_s describes the tolerance area, within which the traffic volume downstream can vary without leading to an alarm. It is given exogenously as a relative value part of Q_2: Q_s=a*Q_2. The value “a” will be calculated empirically during the trial and can be adjusted in the computer program.

3.4.5.3 Incident Detection, Step 2

The step 2 of the AIDA algorithm operate at measuring site level and use as criteria typical changes in the measurement data caused by an incident:

- higher occupancy and lower traffic volume at the measuring site upstream
- lower occupancy and lower traffic volume at the measuring site downstream.

If the congestion affects the measuring site upstream the incident, the values of traffic volume and occupancy do no longer change significantly during the incident, so that this Algorithm does no longer lead to an alarm. At this moment the third part of the Algorithm, the congestion detection, intervenes. However, the congestion detection can already appear during the alarm of step 2.

3.4.5.4 Congestion Detection, Traffic Situation Analysis

In the framework of the traffic situation analysis, the actual traffic situation is described with the help of fundamental diagrams. The fundamental diagrams are determined empirically for each observation points as V/Q-diagrams (Traffic Speed/Volume - diagrams) and are divided in Level-of-Service-areas (LOS). Thereby the traffic volume is given in passenger car-units. Figure-3.2 illustrates an exemplary Speed-Volume diagram.

3.4.5.5 Capacity Utilization Analysis

For the decision Algorithm of the dynamic re-routing, the incident detection as well as

the question “if- and if yes, how much- additional traffic can be led into the Ingolstädter Straße because of the actual traffic situation”, are of a substantial importance. Within the urban area, the road network capacity determined mainly by the capacities of the intersections. The theoretical capacity is calculated by using the maximum capacity utilization and green period at the traffic light for the observed travel direction at the cycle time of the current signal program. As the additional criterion for the control logic

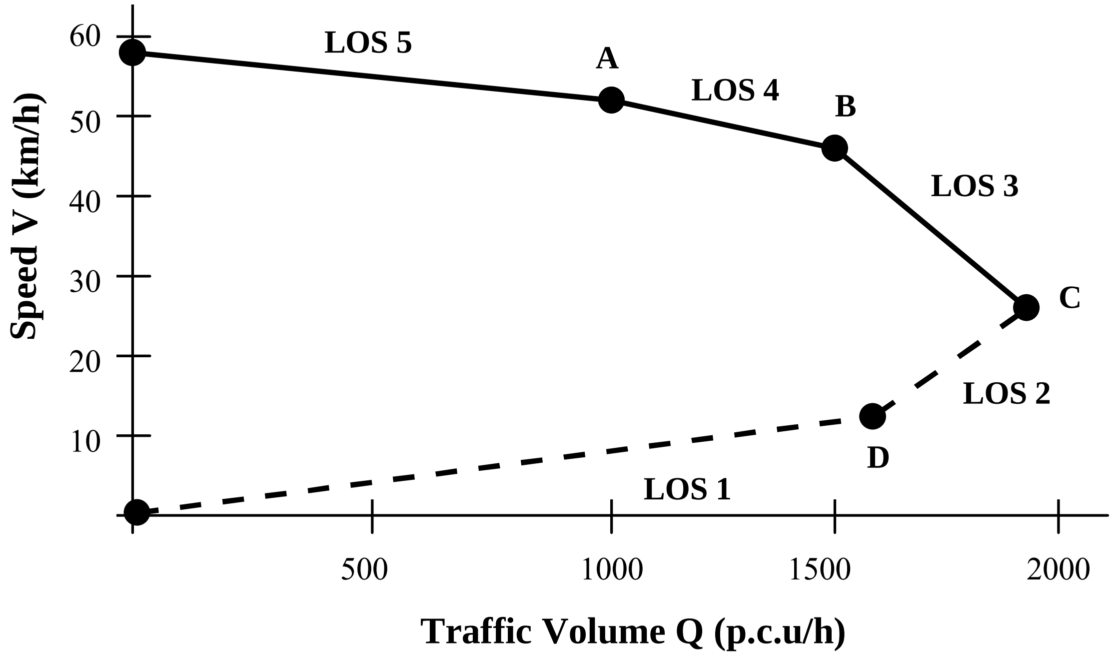


Figure 3.2: Speed-Volume diagram

the capacity utilization is determined out of actual measurements. The capacity reserves at the intersections result from the difference of the maximum capacity and the real traffic volume. Therefore, the traffic volume, which is counted for each cycle per measuring site, is used as data input. This traffic volume is assigned to each upstream intersection with consideration of historical flow-ins.

Figure 3.3 illustrates a lost-capacity diagram. This diagram shows clearly the overloading of the facility between 6.30 and 8.30 am.

3.4.6 Assessment

To make the assessment category "operational" the analysis should refer to the reliability of the algorithms. Generally, the following indicators have been considered:

* detection rate: the ratio of incidents detected out of all incidents that occur during a specified time period

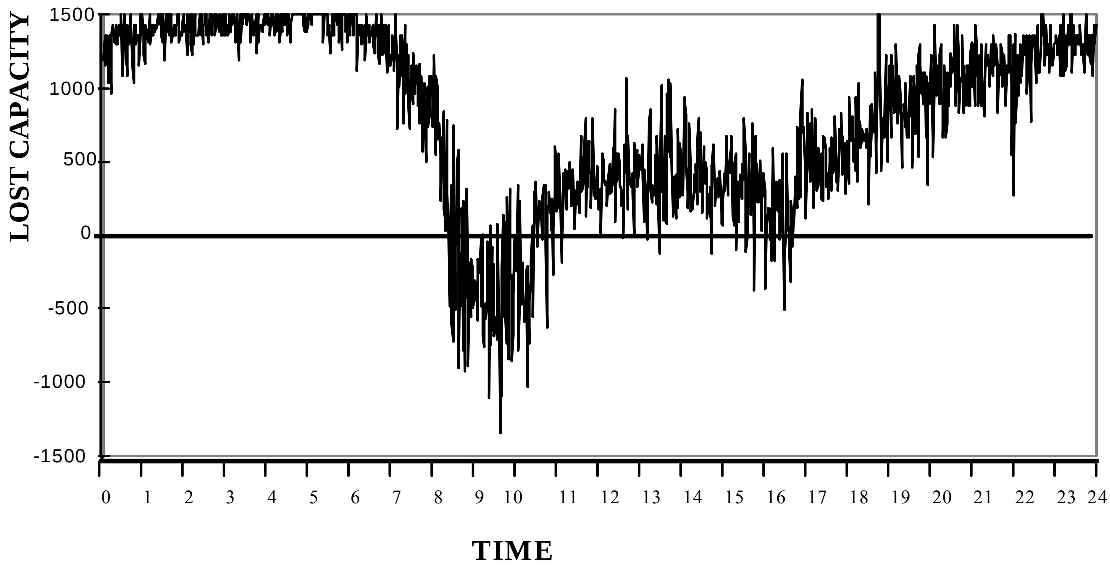


Figure-3.3: Lost Capacity Diagram

* false alarm rate: the ratio of false alarms out of all decisions (incident and non-incident) made by the system during a specific time period.
* reaction time: the time interval from the occurrence of the incident until the time that the incident is detected [23].
* delay between the end disturbance and the alarm cancellation
* localization of the incident/disturbance

Thus, in the framework of the field trial eight incidents were observed. With the exception of the police control, carried out 1st February, the incidents are instabilities of traffic flow (congestion). The following table gives an overview about the indicated incidents of the steps 1, 2 and 3 with AIDA

The evaluations criteria for the algorithms reliability are given separately for Algorithm 1 and for Algorithm 2 (upstream and downstream).

As shown in Table-3.1, the Algorithm 3 gives a very accurate description of the traffic situation (it has alarm rate and no identification rate equal to zero), and it can be considered as a very useful tool for further study of the traffic conditions as it will be seen below [22].

	indicated incident	correct identification	false identification	no identification
Algorithm 1	9	1	8	7
Algorithm 2	up down	up down	up down	up down
	38 7	1 6	37 1	7 2
Algorithm 3	8	8	0	0

Table-3.1

CHAPTER 4

TRAFFIC COORDINATION ALGORITHMS

As it appeared in the last chapter, the state of the art for the traffic signal optimization uses the calculation of delays.

In this chapter two algorithms for the optimization of the traffic coordination are presented. The first algorithm does not take into account the delays and uses only the vehicle speed of cars moving on the main street in order to calculate the offset time in an adjacent pair of intersections. The other one uses a “splitting up” and a “delays” optimization to calculate the offset time and optimize the traffic situation. All the algorithms are related with the A.I.D.A as shown Figure-4.1.

The A.I.D.A receives input data from the detectors and decides whether there is an incident or not. If there is one (according to the three steps as presented in Chapter 3), the alarm is turned on and the implementation of the algorithm begins. This algorithm

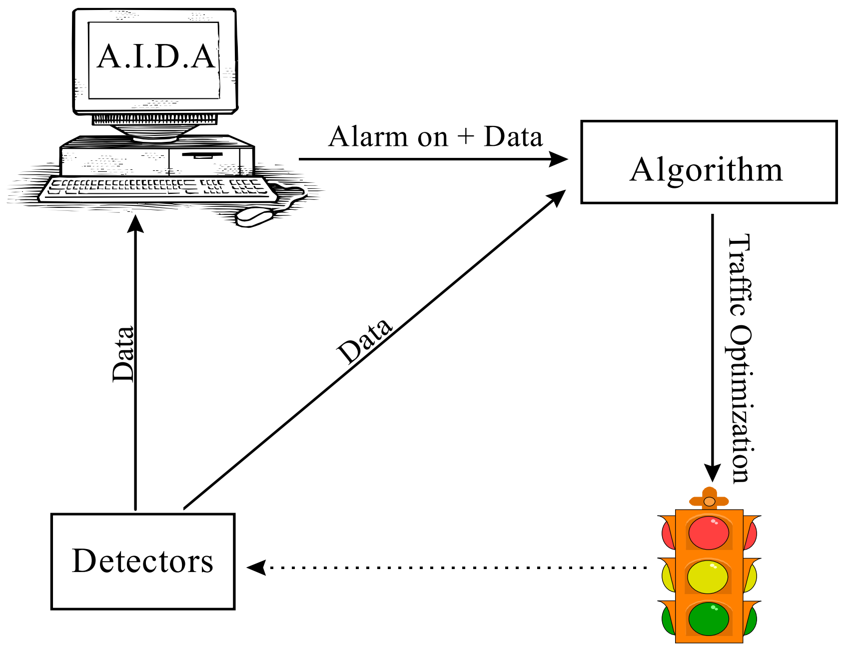


Figure-4.1

lasts as long as the alarm is on. The input data that the algorithms use, come from A.I.D.A (L.O.S-Diagram) as well as from the detectors.

4.1 Algorithm-1

For the offset time calculation using this algorithm some assumptions have been made. First of all, it is supposed that there is a main road with heavy traffic volumes and some secondary streets without serious traffic and of course, there are appropriate installations, according to the A.I.D.A.

Figure-4.2 illustrates the road network showing the locations of the inductive loop detector and Figure-4.3 shows the flowchart of the Algorithm-1 as well.

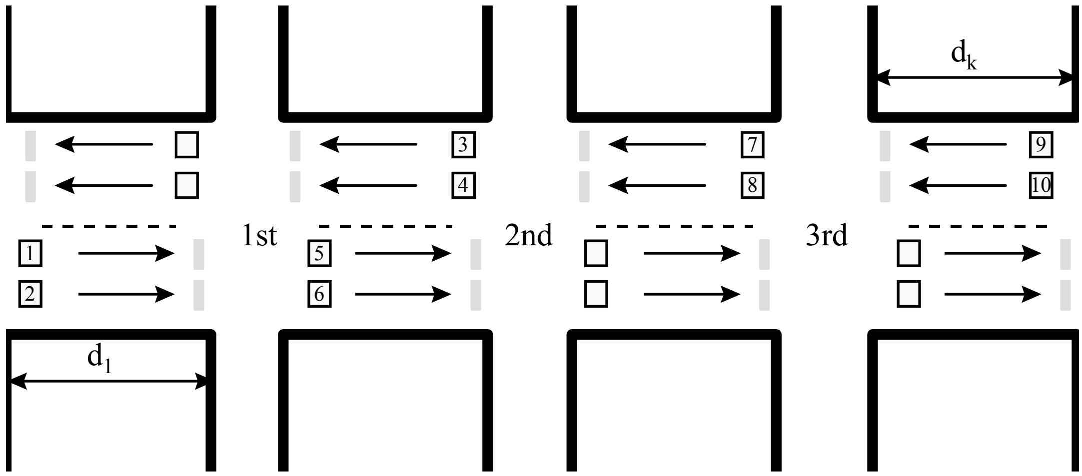


Figure-4.2: Road network with the locations of the detectors

4.1.1 Description of Algorithm-1

As shown in the flowchart in Figure-4.3, the first condition that is examined is whether an incident has been detected by A.I.D.A or not (alarm on or alarm off). If this condition is false the implementation of the program stops and a pre-determined fixed traffic coordination program is implemented. If it is true, the algorithm uses as input the number of intersections that are affected by the incident. For one-way streets these are only the downstream intersections and for two-way on streets these are all the intersections that participate in the traffic coordination.

Then, the position of the intersection is determined. So if $i=1$ or $i=k$ the intersection is "external" (1st or 3rd intersection as Figure-4.2 shows) and it is supposed that the offset time depends mainly on the measures of the detectors 1 and 2 or 9 and 10. The new offset time will be $\psi(1) = \frac{d_1}{V_{(1)Fund.}}$ (or $\psi(k) = \frac{d_k}{V_{(k)Fund.}}$) where d_1 and d_k are the distances as shown in Figure-4.2. If the intersection is internal that is $1<i<k$ (2nd intersection as show Figure-4.2) the offset time depends on vehicle speeds in both directions that are the measures taken from detectors 5 and 6 or 7 and 8. Thus, the offset time $\psi(i) = A \cdot x(i) + B \cdot y(i)$, where $0 \leq A \leq 1$, $0 \leq B \leq 1$ and $A+B=1$ are weighting factors that can change during the day in order to adapt to with traffic demand which can differ in each direction.

Finally, it is examined whether the calculated offset time lies between pre-determined bounds in order to avoid undesirable changes and big fluctuations that could cause serious traffic problems. That is why when $lb \leq \psi(i) \leq rb$, ψ (i) is kept unchanged but when it is ψ (i) $\leq lb$ the lb value is kept and when $rb \leq \psi$ (i), the rb is kept.

After the calculation of the new offset times at each intersection the loop closes and the program is repeated again.

4.2 Algorithm-2

The state of the art for setting traffic signals in an urban street network involves the determination of cycle time, splits of green time, and offsets. All the existing methods use a sequential procedure for calculating the traffic control variables. A common cycle time is established first, then green splits are calculated for each intersection, and, finally, offsets among the signals are determined [10].

This algorithm follows, more or less, the same path and it should be seen in relation to the TRANSYT, SCOOT and VERON algorithms. It consists of two steps. Firstly, it is estimated whether it is better to change the current phase to the next or not.

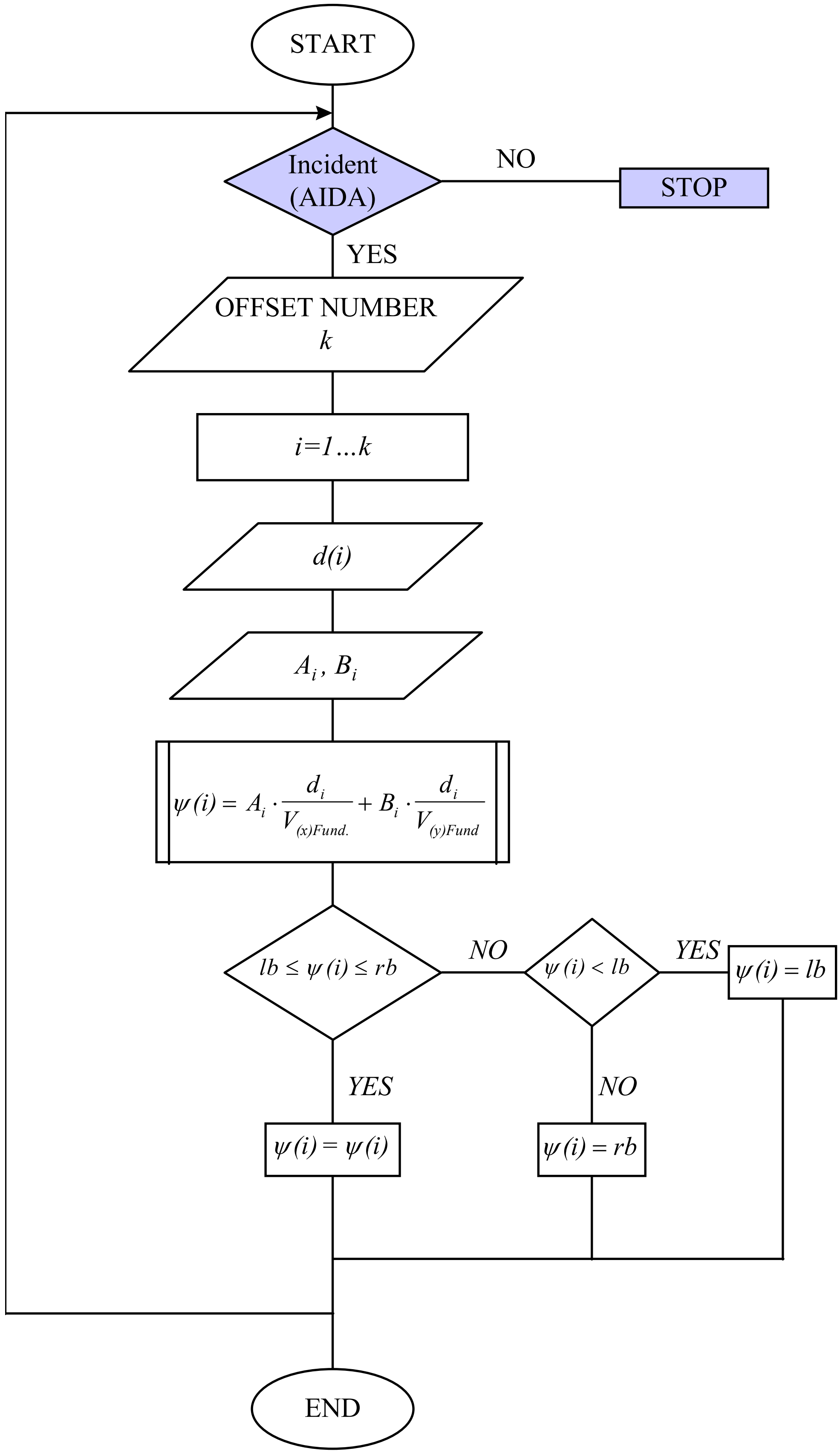


Figure-4.3: Flow chart of Algorithm-1

Secondly, follows the offset time optimization which comes from the minimization of the delay. The minimum delay optimization is designed to minimize total arterial system delay as function of the offset time at each signalized intersection [6].

The difference between Algorithm-2 and SCOOT focuses on the way the moment of the platoon splitting and the delays are calculated.

Figure-4.4 depicts the algorithm as a flowchart starting from Split Optimization Module and continuing with the Offset Time Optimization Module.

4.2.1 Split Optimization Module

Before the detailed, step-by-step explanation of the algorithm, it would be helpful to explain the symbols and the variable names that are used.

TSR :Total Saturation Rate

maxTSR :maximum Total Saturation Rate

t :instant of the change of the green phase to red

j :the number of the current inflow street into the intersection

$SF_{j(geom)}$:Saturation Flow dependent on the geometry of the specific street in p.c.u/hour

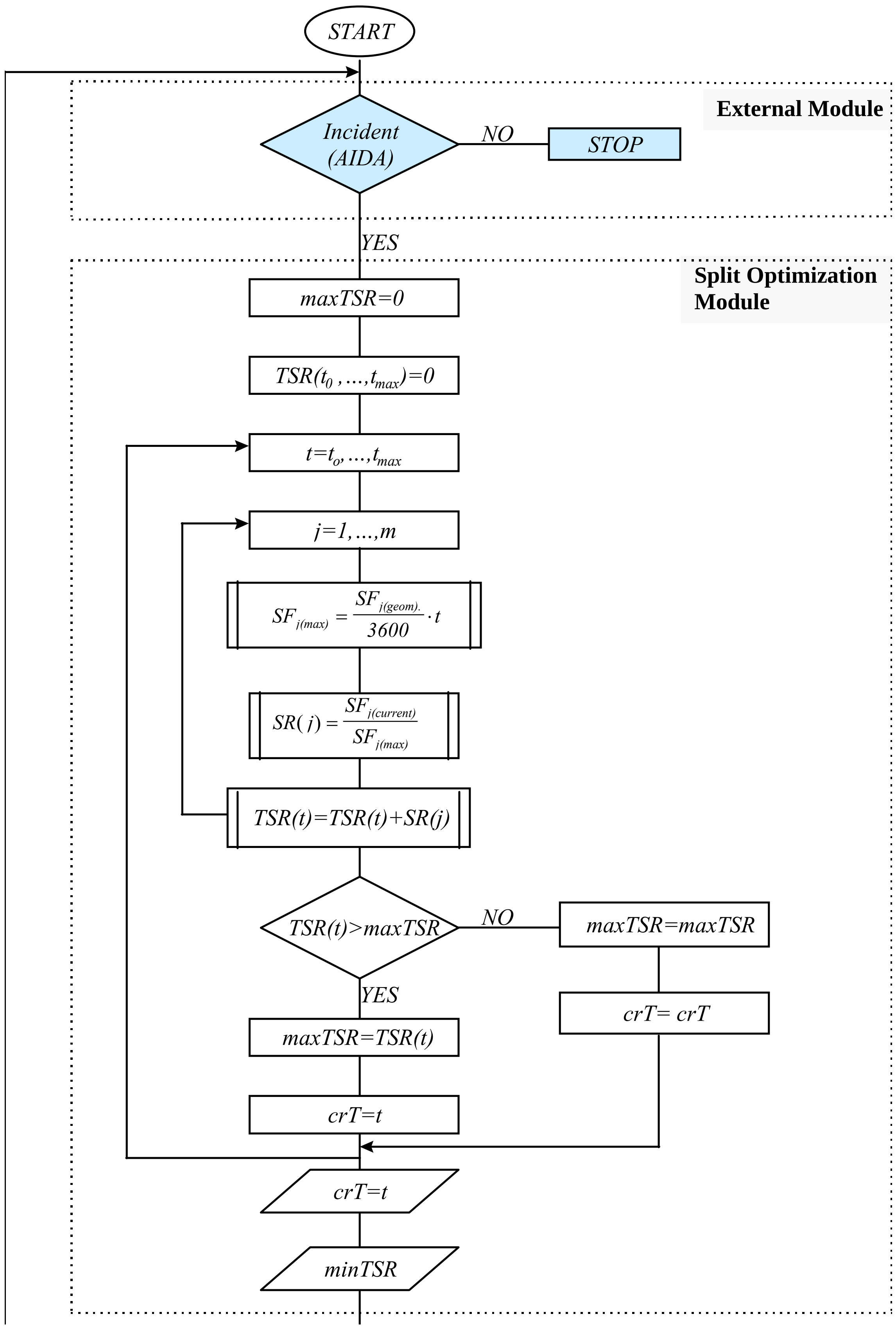
$SF_{j(max)}$:the maximum Saturation for the duration of t seconds

$SF_{j(current)}$:the current Saturation Flow in a specific inflow street of the intersection

crT :the optimum time instant for the phase change from green to red. Actually it is the time that minimizes the over-saturation rate

The algorithm starts with the initialization of some variables. So *maxTS* is set equal to 0, a random small number, in order to be smaller than the first calculated *maxTSF*. To initialize the total saturation flow, $TSR(t_0)$ is set equal to zero.

In the next step, the time is set to run from an initial value t_0 to a maximum value t_{max}. This has to be done in order to restrict the splitting-up time between two bounds (t_0 and t_{max}), thus avoiding undesirable traffic effects caused by unreasonable time values. Further, *j* runs from 1 up to *m*, where *m* is the number of the inflow streets in the specific intersection. This allows the calculations at each inflow street for every *t* as described below.


START
External Module
Incident (AIDA)
NO
STOP
YES
Split Optimization Module
maxTSR=0
$TSR(t_0, ..., t_{max})=0$
$t=t_o, ..., t_{max}$
$j=1,...,m$
$SF_{j(max)} = \frac{SF_{j(geom).}}{3600} \cdot t$
$SR(j) = \frac{SF_{j(current)}}{SF_{j(max)}}$
TSR(t)=TSR(t)+SR(j)
TSR(t)>maxTSR
NO
maxTSR=maxTSR
crT= crT
YES
maxTSR=TSR(t)
crT=t
crT=t
minTSR

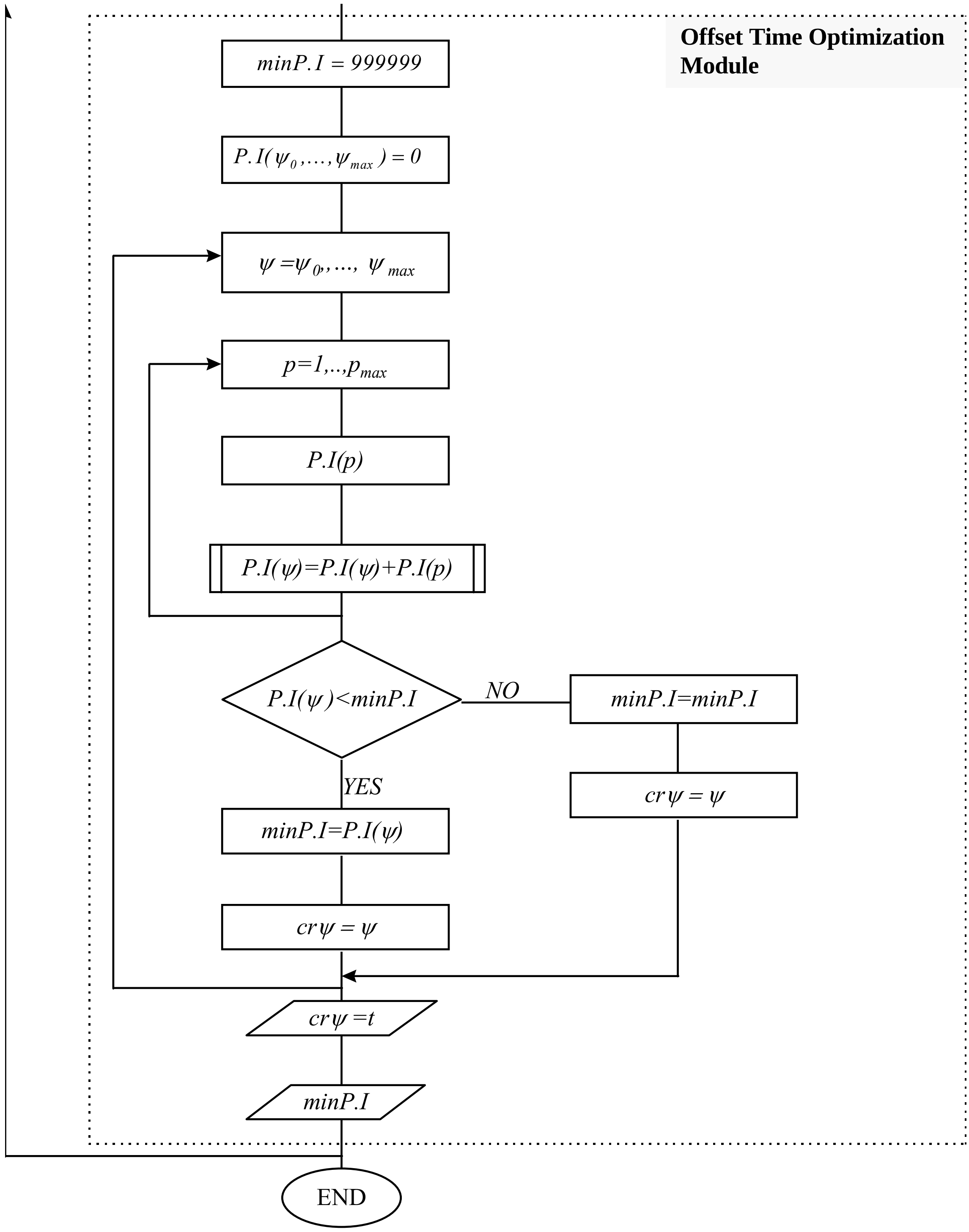


Figure-4.4: Flow chart of Algorithm-2

The calculation of the maximum saturation flow $SF_{j(max)}$ follows for every inflow street which is function of $SF_{j(geom)}$. Because of $SF_{j(geom)}$ is given in p.c.u/h it is divided by 3600 to transform it into seconds. Multiplying the fraction $\frac{SF_{j(geom)}}{3600}$ by the splitting-up (green time) time results in the maximum saturation flow of the *j*-inflow street for the *t* seconds of length of the green time.

In the next step, the saturation rate is calculated. It is obtained by dividing the current saturation flow $SF_{j(current)}$ by the pre-calculated maximum saturation flow $SF_{j(max)}$. As far as the $SF_{j(current)}$ is concerned it is a prediction of how many vehicles are going to pass over the stop line during the green period considering the distance from the detector

up to the stop line. The calculation of the current saturation flow $SF_{j(current)}$ will be elaborated in the Section 4.3.

From the addition of all saturation rates for every *j*, results the total saturation rate *TSR(t)* for the time instant *t*. If *TSR(t)* is grater than the maximum total saturation rate *maxTSR*, *maxTSR* becomes equal to *TSR(t)* and the crucial time *crT* becomes equal to the current *t.* Otherwise, both the *maxTRS* and *crT* stay unchanged.

Then the loop closes and it is repeated for the next time value. The final output will be the *crT* that came out of the maximum total saturation flow *maxTSR*. Obtaining this output, the first module of the algorithm, the Split Optimization module is finished and the Offset Time Optimization Module follows.

4.2.2 Offset Time Optimization Module

As done in the presentation of the former module, it would be helpful to explain the symbols and the variable names that are used before the detailed, step-by-step explanation of this module.

MinP.I :is the minimum performance index

P.I :is the performance index

ψ :is the offset time

p :is the number of the current traffic phase

$cr\,\psi$:is the crucial offset time

After these definitions, the step-by-step presentation can start. The module starts with the appropriate initializations. The minimum Performance Index is set equal to

999999, a random big number, that will help in the first comparison in order to set the first P.I as *minP.I.* The initial P.I for each ψ is set equal to zero.

Then the offset time variable ψ is first set to ψ_0 and it will be incremented at the end of each cycle, ending with $\psi_{\max}$. The variable p is then assigned, which runs from 1 up to p_{max} where p_{max} is the maximum number of the phases at the specific intersection.

Further, the *P.I* for each phase is calculated. The way that the delay (that is the *P.I*) of each phase is calculated will be explained in section 4.4. Then, all the individual *P.I* are summed up and the total *P.I* for the offset time ψ is calculated. After the calculation of the total *P.I,* it is compared with the *minP.I* and the smaller value as well as the offset time that minimizes the total *P.I*(ψ) are kept. This happens for every ψ and finally the optimum offset time $cr\psi$ is obtained.

At this point the traffic signal optimization in the specific intersection has been integrated and the algorithm is repeated again. The same algorithm is implemented for every intersection that participates in the traffic coordination and is affected by the incident, improving the traffic situation in the examined part of the network.

4.3 Saturation Flow Calculation

It could be said that this is a "microscopic" way of Current Saturation Flow calculation and is relayed on the examination of each individual vehicle and its interaction with other vehicles and impact of the roadway geometry.

So, more or less, the speeds of the vehicles are known as it will be shown in Chapter 6. The exact positions of the detectors are known, too, and the time that a vehicle needs to cover the distance from the detector up to the stop line can be easily calculated. Apart from that, the instant as well as the kind of vehicle (personal car or HGV) passing over the detector are known. Thus, changing at will the green time, one can calculate the number of vehicles that are able to pass during the current green time.

The following example gives a schematic description of this idea. It is supposed that the detector gives the information as depicted in Figure-4.5. The vertical axis expresses the type of vehicles that are HGV or personal cars and the horizontal axis gives the arrival time of the vehicles in seconds. It is supposed that a vehicle needs to cover the distance D, as shown in Figure-4.6, 10 sec. In that case, if green time lasts 50 sec., only the vehicles that passed the detector up to the 40*th* second will be allowed to pass. According to this example, four personal cars and one HGV are going to pass. As one can see, if the green time would be increased 10 seconds, one more car would be able to pass.

Finally, changing the green time from a minimum value up to a maximum, the

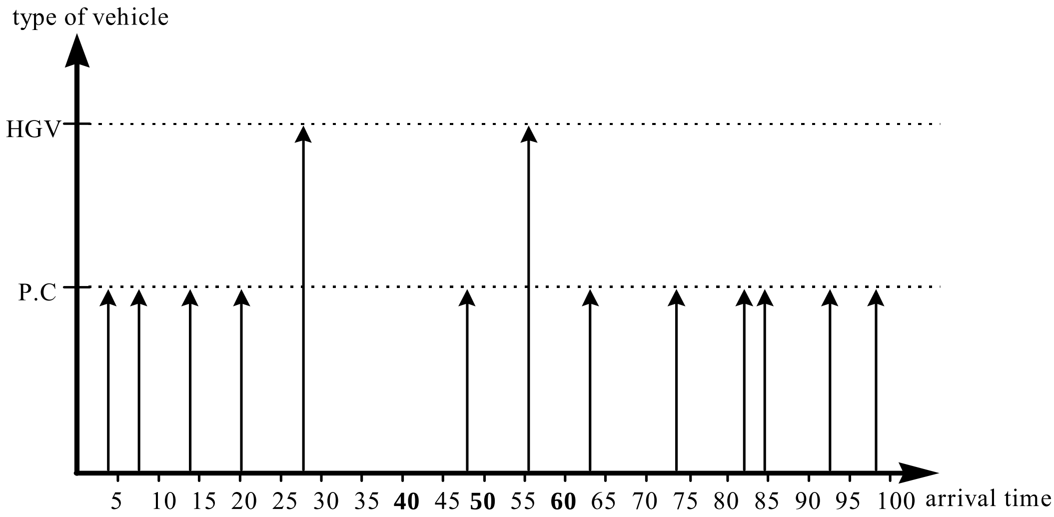


Figure-4.5

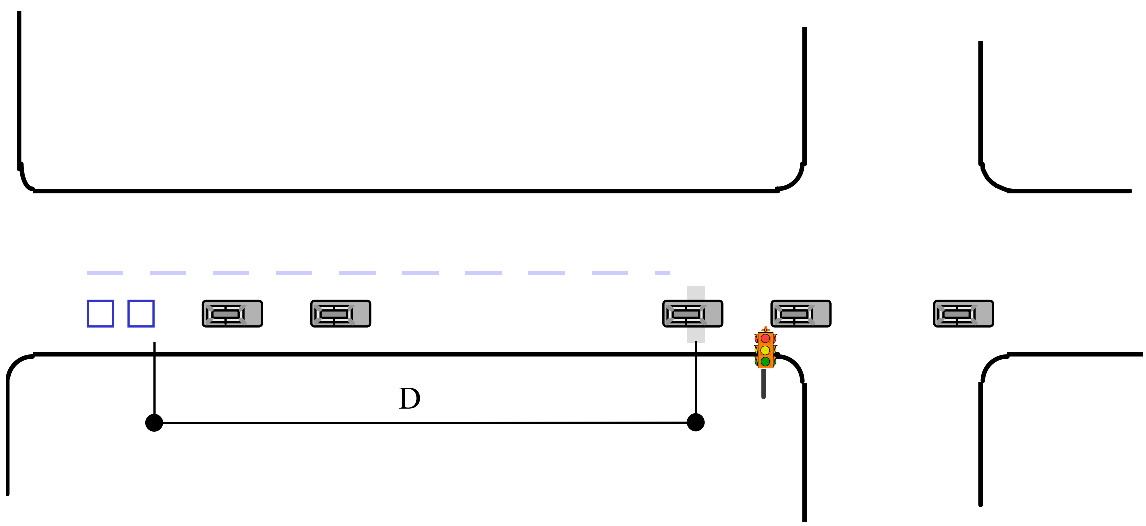


Figure-4.6

optimum current saturation flow can be calculated for every time step. In addition, calculating the maximum capacity flow for each time step the saturation rate is obtained as described before in the presentation of the algorithm.

4.4 Saturation Flow Calculation Using Queuing Theory

In the previous paragraph, the Saturation Flow was calculated supposing that the arrival instances were known. From the hardware point of view it does not seem difficult to determined.

However, sometimes this kind of information is not available. In this case arises the need to find another way for the calculation of the Saturation Flow. This can be achieved considering that the vehicle arrivals follow a random, or the Poisson, distribution, having arrival rate λ. The following requirements define required conditions for randomness of vehicle arrivals:

- Each vehicle is positioned by its driver independently of other vehicles.
- The number of vehicles passing a point in a given length of time is independent of the number that passes this point in any other length of time.

A counting distribution that satisfies the above requirements is the Poisson distribution, which is described by the following equation:

$$P(n/qt) = \frac{e^{-qt} \cdot (qt)^n}{n!} \quad \textbf{Eq-4.1}$$

where:

$P(n/qt)$	:is the probability of the arrival of n vehicles at a point during the time interval t when the average volume is q vehicles per unit of time
n	:is the number of vehicle arrivals
q	:is the average traffic volume (vehicle per unit of time)
t	:is the time interval (units of time)
e	:is the base of natural logarithms (2.718)
$n!$	:is the n factorial

In that way vehicles are considered as “customers” waiting to use a “server” such as a signalized intersection. In addition it could be supposed that the vehicle queue is stable which means that the arrivals can be regarded as a stationary stochastic process;

that is systems in which the expected arrival rate λ does not vary with the time t [11, 14, 27].

After these assumptions a more specific determination of λ follows. The arrival rate λ comes easily by dividing the traffic volume q counted, during the time interval t (time unit), by that interval (of course the time interval is given in seconds).

So, as Figure-4.7 shows, if someone wanted to calculate the Saturation Flow a few seconds after the time instant t_2, the value of λ_2 would be used as arrival rate. In the same way, if someone wanted to calculate the Saturation Flow at the instant t which is $t_{k-2} < t < t_{k-1}$, the value of λ_{k-2}, would be used, etc.

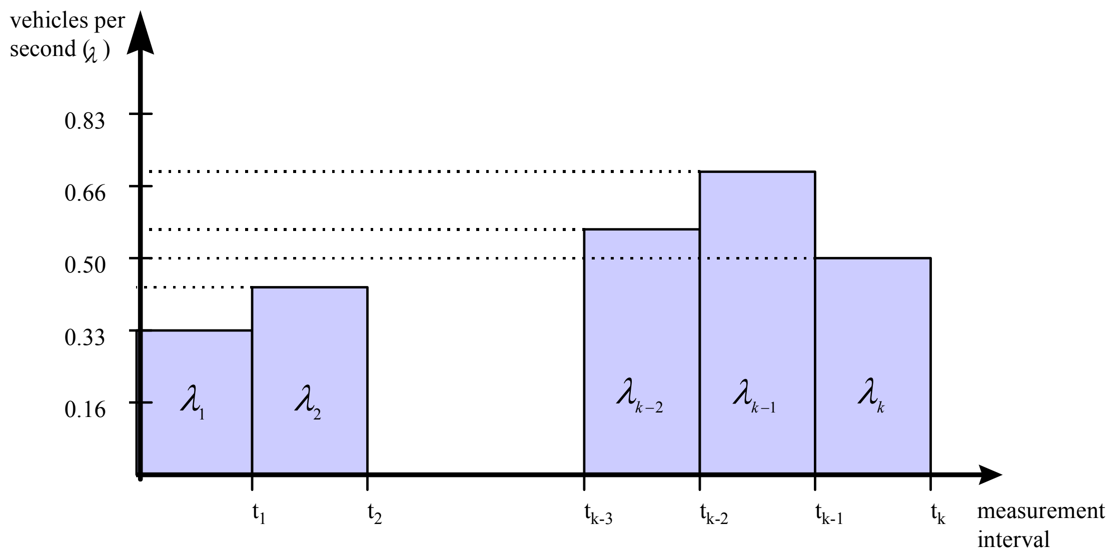


Figure-4.7: Arrival rate per cycle

The reason that λ is calculated in this way is the following: there are two conflicting requirements which must be satisfied simultaneously. The first one is that a steady value should be for λ otherwise a steady-state solution can not be implemented and the second one is that the traffic demand changes dramatically during the day i.e. there is a morning and an evening pick periods on weekdays, moderate-to-high flows during midday and much lower flows at other times. So, dividing the time in small time intervals which are equal to the current cycle length and calculating the arrival rate λ based on these time intervals, a relatively realistic and stable for sure, value of λ is obtained. In that way the above two conflicted requirements are compromised too.

4.5 Queue Length and Delays Calculation

For the queue length and the calculation of the consequent delays, the same syllogism as in Section 4.3 is used.

The average speed of the vehicles, the positions of the detectors and the instants the cars are detected as well as the kinds of vehicles passing over the detector are known. Changing at will the offset time ψ, one can calculate the number of vehicles that are going to accumulate in front of the stopping line and the consequent delays. More specific, the total delay will be the number of vehicles left from the last cycle plus the number of the arriving vehicles during the red period.

So let it be supposed that the number of the remaining cars from the previous cycle are q_0 and during the current cycle three cars have passed over the detector at the instants t_1, t_2, and t_3, respectively. Then the total delay for the given offset time ψ is given by the Equation 4.1, where $\bar{l}$ is the average length per vehicle including the distance between cars and U expresses the speed of the vehicles which is given by the Level of Service Diagram.

$$\text{delay} = q_0 \cdot \psi + \left(\psi - \frac{D - q_0 \cdot \bar{l}}{U} - t_1\right) + \left(\psi - \frac{D - (q_0 + 1)\cdot \bar{l}}{U} - t_2\right) + \left(\psi - \frac{D - (q_0 + 2)\cdot \bar{l}}{U} - t_3\right)$$

Eq.-4.1

The first term of the Eq.-4.1 expresses the delay because of the previous cycle vehicles, the second term is the delay for the first vehicle queuing back in the current cycle, the third term expresses the delay for the second vehicle, etc.

For better understanding Figure-4.8 shows the distance-related variables.

By generalizing Equation-4.1 for k arrivals the general formula for the delay calculation comes out, which is:

$$delay = (q_0 + k)\cdot \psi - \sum_{i=1}^{i=k} t_i - \frac{k \cdot (D - q_{0} \cdot \bar{l}) - \frac{(k-1)\cdot k}{2} \cdot \bar{l}}{U} \qquad \textbf{Eq.-4.2}$$

<u>or</u>

$$delay = (q_0 + k)\cdot \psi - \sum_{i=1}^{i=k} t_i - \frac{2 \cdot k \cdot (D - q_0 \cdot \bar{l}) - (k-1)\cdot k \cdot \bar{l}}{2 \cdot U} \qquad \textbf{Eq.-4.3}$$

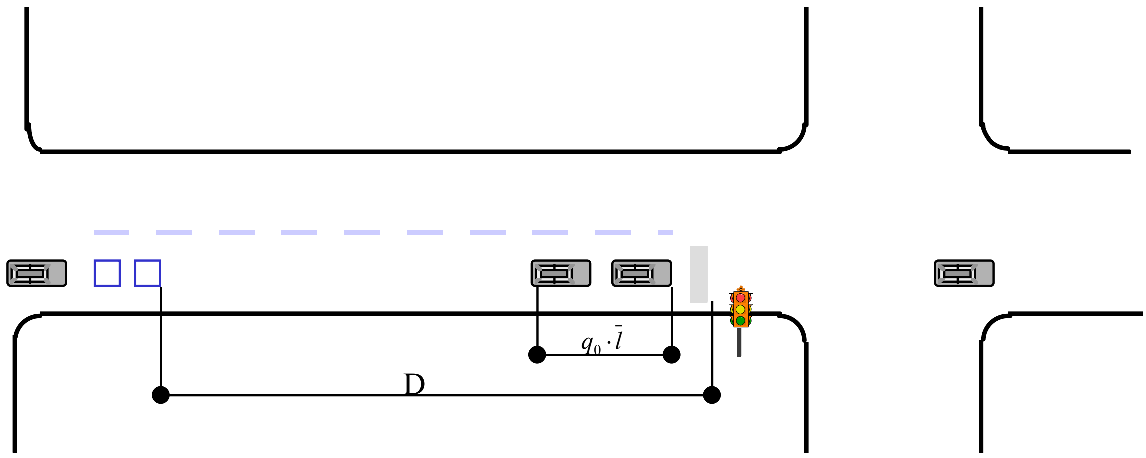


Figure-4.8

Thus, after having the formula of the delays and changing the offset time ψ from $\psi_{\min}$ up to $\psi_{\max}$ the offset time that minimize the delays results and this is the optimum offset time.

4.5.1 Calculation of the Initial Queue q_0

As mentioned before, the delays calculation starts supposing an initial queue of q_0 vehicles which have left from the previous cycle. These vehicles did not succeed in passing during the previous green time. An approximate number of these vehicles can be easily calculated.

Let it be supposed that k vehicles queued up during the previous cycle. According to the default values of the 1985 Highway Capacity Manual for ideal conditions, the discharge headway is about 2 sec/vehicle [12].

Thus, knowing the number of the queued up vehicles and the discharge headway one can calculate the vehicles that are able to pass over the stop line for the given green phase and further more the number of the vehicles that are not able to pass.

In case of the first implementation of the algorithm there is not any available information for the existing queue so one can suppose that the initial queue q_0 is equal to zero.

4.6 Queue Length and Delays Calculation Using Queuing Theory

Analogous to Section 4.4, not knowing the exact arrival instances the queue length and the consequent delays can not be calculated. To overcome this difficulty one could use the queuing theory considering, as it mentioned before, Poisson arrivals.

Thus, an arrival rate λ for each phase is calculated. This λ comes from the addition of the vehicle that passed over the detector during the last phase interval divided by that interval. So for each phase at the intersection, there is a traffic profile which uses the arrival rate as in Figure-4.7.

Using these profiles and changing at will the offset time someone can calculate the queue length and the delays. The problem that arises is how the formula for the delay calculation should be now. For better understanding of this thought, an example is examined.

An intersection is considered which has a phase with arrival rate λ=5 vehicles per minute. This means that on average every 12 seconds one vehicle passes over the detector (of course the time between two arrivals follows the exponential distribution). Figure-4.9 shows the schematic representation of this thought.

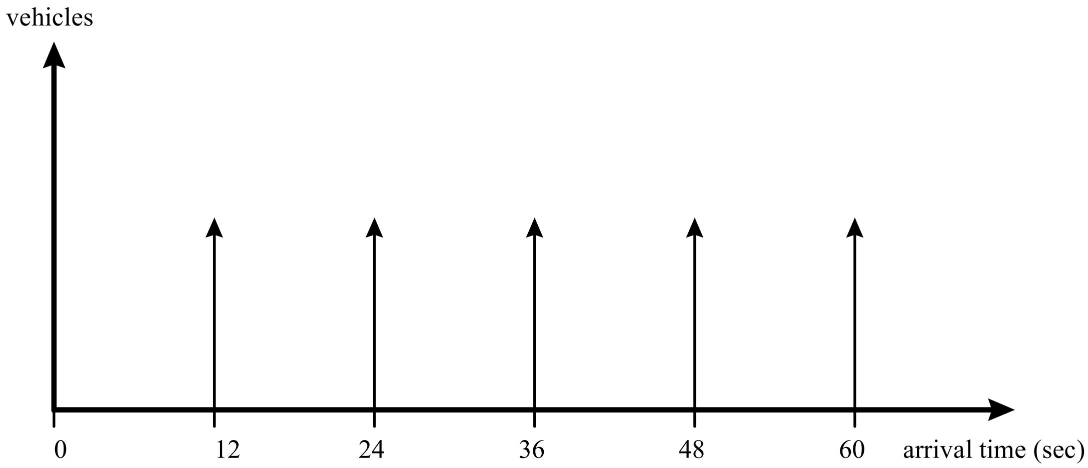


Figure-4.9: Traffic profile with λ=5 vehicles per minute

Using the symbols of Section 4.5 the total delay for two queued passenger vehicles will be

$$\text{delay} = q_0 \cdot \psi + (\psi - \frac{D - q_0 \cdot \bar{l}}{U} - \lambda) + (\psi - \frac{D - (q_0 + 1) \cdot \bar{l}}{U} - 2\lambda) \qquad \textbf{Eq.-4.4}$$

By generalizing Equation-4.4 for *k* arrivals and after some mathematical operations, the general formula comes out for the calculation of delays of Poisson arrivals:

$$delay = (q_0 + k) \cdot \psi - \frac{k \cdot (k+1)}{2} \cdot \lambda - \frac{2 \cdot k \cdot (D - q_0 \cdot \bar{l}) - (k-1) \cdot k \cdot \bar{l}}{2 \cdot U} \qquad \textbf{Eq.-4.5}$$

CHAPTER 5

SUGGESTION FOR THE TRAFFIC OPTIMIZATION IN CASE OF NON-INCIDENT

5.1 Cycle Time Selection Algorithm

This algorithm can be used for a better selection of the cycle time, following the fluctuations of the traffic flow during the day. It is a traffic optimization algorithm which can be used independently of other optimization algorithms. Figure-5.1. shows the description of the algorithm as a flow chart.

At this point the explanation of the symbols is given:

$SF(j)$: is the variable of the current saturation flow

$SF_{uTHR}(j)$: is a constant equal to the upper threshold of the saturation flow

$SF_{dTHR}(j)$: is a constant equal to the lower threshold of the saturation flow

$cycle(j)$: is the current length of the cycle time

z_u : is a counting variable which counts how many times the condition $SF>SF_{uTHR}$ has become true

z_u : is a counting variable which counts how many times the condition $SF<SF_{dTHR}$ has become true

k_u : is a constant number

k_d : is a constant number

After these definitions, the analytical description of the algorithm is following. At the beginning, some input data for the execution of the algorithm are given. An example of the form of these input data is given in Table-4.1 where the values of $z_u(j)$ and $z_d(j)$ are determined during the calibration phase of the algorithm integration.

The first condition which is examined is whether the current saturation flow *(SF)* is greater than upper threshold of the saturation flow (SF_{uTHR}) for the current cycle time.

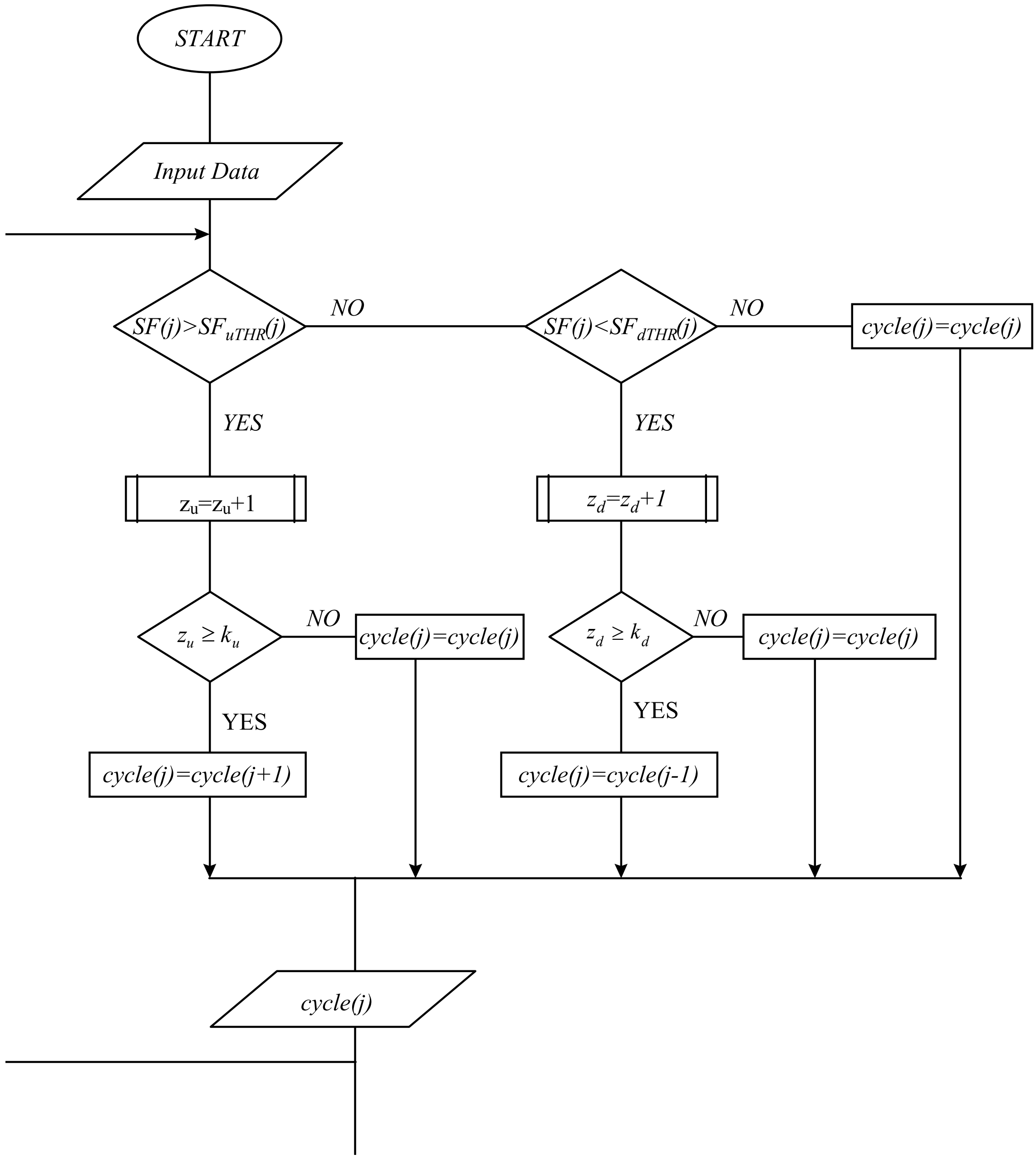


Figure-4.1: Cycle time selection algorithm

j	1	2	3	4	5
$cycle(j)$	52	72	90	100	120
Sf_{uTHR}	$SF_{uTHR}(1)$	$Sf_{uTHR}(2)$	$SF_{uTHR}(3)$	$SF_{uTHR}(4)$	$SF_{uTHR}(5)$
SF_{dTHR}	$SF_{dTHR}(1)$	$SF_{dTHR}(2)$	$SF_{dTHR}(3)$	$SF_{dTHR}(4)$	$SF_{dTHR}(5)$
k_u					
k_d					

Table-4.1: Example of the input data

If the condition $SF>SF_{uTHR}$ is "true" the counter z_u is increased plus one. After this increment z_u is compared with the constant number k_u and if $z_u \geq k_u$, the cycle changes and takes its next value, *cycle(j)=cycle(j+1)*, otherwise the cycle time stays unchanged.

In case the condition $SF>SF_{uTHR}$ is false, the condition $SF<SF_{dTHR}$ is examined. So, if $SF<SF_{dTHR}$ is true, the counter *zd* is increased plus one. Then the new value of z_d is compared with the constant k_d and if $z_d \geq k_d$, the cycle changes taking its previous value, otherwise it stays unchangeable. If the condition $SF>SF_{uTHR}$ is false, the cycle time length does not change too.

After the completion of the above steps, the algorithm is repeated for the next cycle.

5.2 Variation of the Algorithm-2 in Case of Non-Incident

As shown in Sections 4.4 and 4.6 of Chapter-4, in case that the exact arrival instances are not known one can use the queuing theory to calculate the saturation flow as well as the delays. Therefore, it was supposed that the arrival rate λ is constant during the cycle time, because the steady-state condition should be satisfied, and a new λ was calculated for every cycle.

Since the same algorithm for the traffic optimization in case of non-incident is used, some assumption for the arrival rate should be made. It is easy for someone to understand that there is no need for a new λ calculation for every cycle because if there is no incident one can suppose that λ is constant for a longer time interval. Thus, a new arrival rate λ can be calculated every five cycles. Figure-5.2 represents this idea.

The advantage of this idea is that there is no need to calculate a new λ in every cycle, so unnecessary calculations and fluctuations are avoided.

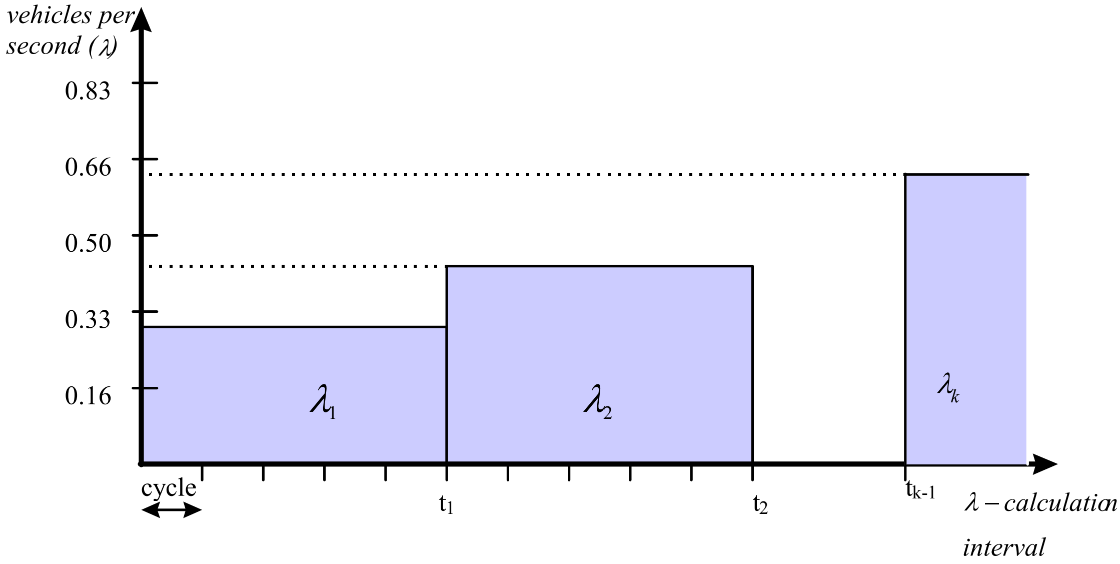


Figure-5.2: Arrival rate calculation at every five cycles

CHAPTER 6

REMARKS AND COMMENTS FOR THE PRESENTED ALGORITHMS

6.1 Comments on the Algorithm-1

As presented in Chapter 4, Algorithm-1 calculates the offset time between two adjacent intersections without taking the delays into consideration. That is why an assumption has been made. It is supposed that there is a main road with important traffic volumes and some secondary roads with relatively inconsiderable traffic.

As mentioned in Paragraph 4.1.1, after the calculation of the offset time, an examination is contacted in order to ascertain whether its value lays between the bounds *lb* (left bound) and *rb* (right bound). These bounds are determined using historical traffic data from all the roads that participate in the traffic coordination process.

This method is expected to have satisfactory effects as long as the above assumption is true. If not, the more the traffic volume in the secondary streets would increase, compared with the traffic volume in the main road, the more the error in the offset time calculation would have been increased. That means that if someone was able to calculate the total delay, one would be able to see a serious increment due to the serious error in the offset time calculation.

The advantage of this algorithm is that it requires installations only on the main street and not on the secondary streets. That makes its realization easier and more cost-saving.

In case of the Ingolstäder Straße, Algorithm-1 cannot be used because there are detectors only in one carriage way and the information which can be obtained is insufficient.

6.2 Comments on the Algorithm-2

As shown in the presentation of the Algorithm-2 in Chapter 4, the algorithm can be implemented in a “microscopic” form examining the arrival of every single vehicle separately. Alternatively, in a “macroscopic” form, an arrival rate λ is determined for every cycle, in case the algorithm is running because of an incident. In case of an absence of an incident, an arrival rate λ for about every 8 minutes (every 5 cycles) is calculated.

This algorithm should be more accurate than the first one because it takes into consideration the hole network, minimizing the overall over-saturation flow, as well as, calculating the total delay.

In case of the Ingolstädter Straße as it has been mentioned, there are detectors only in one carriage way and not enough installations in the side streets. To overcome this difficulty one could use the historical data that already exists for the streets without any installation (without current data). However this action would have negative effects on the calculations accuracy.

In addition, activating a traffic optimization program is a very serious decision which directly affects the traffic situation. Therefore, as in any dynamic control system (i.e., any control system reacting to the traffic flow), it requires an accurate assessment of the prevailing traffic condition in order undesirable effects to be avoided [5]. So in any case the balance point between the amount of historical and current data should be determined in order to have the total minimum cost (traffic-situation-related cost plus cost coming from the additional installations).

6.3 Comments on the Speed Measured by the Detectors

Up to now, the speed of the vehicles was used extensively for different calculations. However, there were not any comments for the “quality” of this piece of information which is obtained by the detectors; especially in case of an incident, the time-mean speed tends to be quite different from the speed indicated by the detector. For example, as Figure-6.1 shows, a vehicle passes over the detector with, lets say, 40 km/h. Then, due to a downstream incident, the driver breaks in order to pass safely through the incident area and the speed falls to 7 km/h. Finally, after the incident area the driver accelerates again.

It is obvious that the time-mean speed that was measured by the detector is not the actual vehicles-moving speed in this part of the network especially, if the incident is far away downstream of the measuring site.

The measured speed tends to be more realistic in case the incident is close downstream of

the detector because of the drivers anticipatory behavior [16]. The speed is even more realistic in the event of a heavy congestion where the detector is inside the congested area.

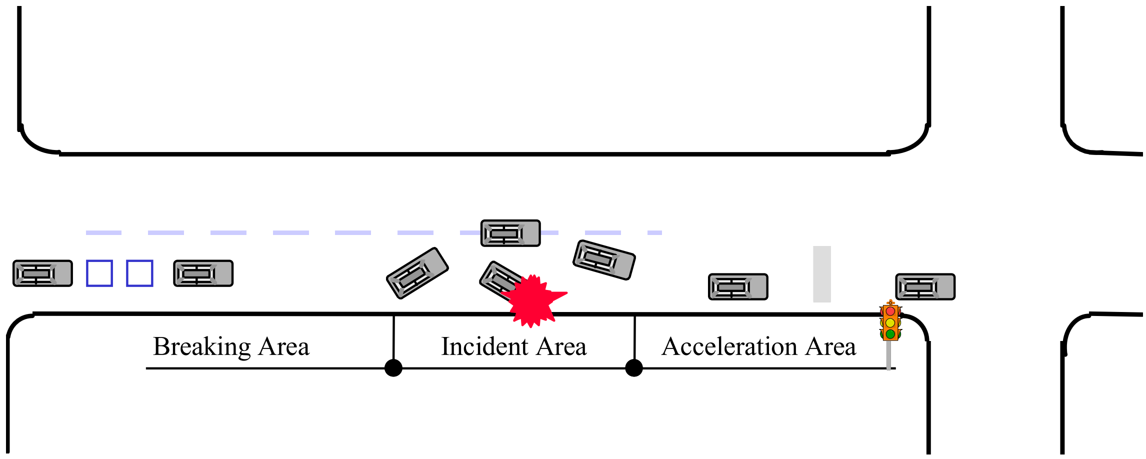


Figure-6.1: Incident downstream the detector

However, if nothing of the above happen, the only available information is the kind of the incident which is given by the three steps of the AIDA and the measured speed. In that case shapes of speeds distributions need to be obtained. The shapes of distributions can be obtained

(a) theoretically,

(b) by simulation,

(c) or by constructing histograms of observations with an assumed common mean [25].

Finally in case of the Non-Incident implementation of the Traffic optimization algorithms, the speed could be safely obtained by the fundamental diagram as it comes out of the third step of the AIDA.

CONCLUSION

The ideas that were presented in this thesis are expected to improve the traffic actuated signal control and to reduce the overall delay within a specific part of the traffic network in Munich and potentially in similar traffic networks were applicable in the future. The investigated Algorithm-1 is expected to have as satisfactory effects, as long as the assumption for the main and the secondary roads is satisfied. As far as the Algorithm-2 is concerned, better effects compared with Algorithm-1 are anticipated, due to its advanced sophistication, taking into consideration the saturation flow and the delays. The saturation flow and the delays calculations which are used by the Algorithm-2 are made with two different approaches. According Algorithm-1, every single vehicle is examined, while the alternative one utilizes the queuing theory, considering the existing system as a system with memoryless arrivals, memoryless departures and one server (M|M|1). The "customers", according to the latter, are the vehicles which are waiting to be "served" by the intersection (server), arriving with an arrival rate λ that was determined in the previous cycle.

Apart from that, suggestions for the traffic control optimization in case of non-incident have been made. Thus, firstly an algorithm is suggested, for the change of the cycle time, which allows a better manipulation of the cycle time changes during the day-traffic fluctuations. Furthermore, an additional algorithm based on the Algorithm-2 is presented for the offset time calculation. This additional algorithm could be applied in steady traffic conditions calculating a new arrival rate λ for every five cycles.

Key point in all the above calculations is the traverse travel speed of the vehicles. The local speed value given by the detectors can not be taken as traverse travel speed when there is an incident far away downstream of the detectors position. Therefore there is a need for some more effort to be made in order to obtain a better estimation of the vehicles traverse speed. This could be achieved through different methods, as theoretical study, simulation, or histograms construction based on historical data for the different sorts of incidents that are given by the AIDA.

GLOSSARY

1. **arterial road:** important main roads
2. **bottleneck (geometric):** a geometric deficiency in isolated sections of roadway facilities [23]
3. **capacity:** a generic expression pertaining to the ability of a roadway to accommodate traffic in given circumstances
4. **cycle:** one complete sequence of signal indications [18]
5. **cycle length:** total time for the signal to complete one cycle, given the symbol C (sec)
6. **detection:** includes all methods, techniques and actions needed to identify all spatial, temporal and severity characteristics of an incident [23]
7. **detector occupancy:** a measure of how long time the in-and exit-detectors are active [15]
8. **disturbance:** every derivation of one or more road users from the momentary driving intention [22]
9. **effective green time:** time during which a given phase is effectively available for stable moving platoons of vehicles in the permitted movements; this is generally taken to be the green time plus the change interval minus the lost time for the designed phase
10. **green time:** time within a given phase during which the "green" indication is shown
11. **highway:** main public road; main route
12. **inductive loop detectors:** detectors that are formed by two or three turns of a 12-gauge or 14-gauge wire placed in slots cut into the pavement and brought back to an amplifier/detector in the control cabinet [18]
13. **intersection:** the general area where two or more highways join or cross, within which are included the roadway and roadside facilities for traffic movements in that area
14. **interval:** period of time during which all signal indications remain constant
15. **lane:** marked division of a wide road for the guidance of motorist; line of vehicles within such a division
16. **link process:** the process that uses data from two adjacent measuring sites
17. **lost time:** time during which the intersection is not effectively used by any movement; these times occur during portion of the change interval (when the

intersection is cleared) and at the beginning of each phase as the first few cars in a standing queue experience start-up delays (sec or sec/phase)

18. **measuring site process:** the process that uses data from one measuring site
19. **phase:** part of a cycle allocated to any combination of traffic movements receiving the right of way simultaneously during one or more intervals
20. **platoon:** is a vehicle queue where all the vehicle speeds besides the first one are influenced by the slowest moving vehicle speed [4]
21. **progression speed:** the speed that ensures the continue movement of vehicles along the signalized street [9]
22. **roadway:** central part used by wheeled traffic
23. **traffic management:** is the use of traffic engineering and control not just to provide for a pattern of traffic but deliberately to seek to influence it to produce improvements in accessibility and environment [1].
24. **traffic volume (q):** is defined as the time rate of traffic flow and is evaluated by counting the number of vehicles that pass a point in a unit of time [2]

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