EFFECTIVENESS OF VEHICLE ACTUATED SIGNALS FOR AT GRADE FOUR LEGGED INTERSECTIONS IN SRI LANKA: A COMPARISON STUDY AGAINST FIXED TIME TRAFFIC SIGNALS

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(08/8861H)

Degree of Master of Engineering

Department of Civil Engineering

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October 2011

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DECLARATION OF THE CANDIDATE

'I declare that, this is my own work and this thesis/dissertation does not incorporate without acknowledgement of any material previously submitted for a Degree or Diploma in any University or other institute of higher learning and to the best of my knowledge and belief, it does not contain any material previously published or written by another person except where the acknowledgement is made in the text'

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'I have supervised and accepted this thesis for the submission of the degree'

Signature of the supervisor: .... Date: / /2011

Prof. J.M.S.J. Bandara

This thesis is dedicated to my ever-loving family

ACKNOWLEDGEMENT

This dissertation would have not been possible without the guidance and the assistance of several individuals who in one way or another contributed and extended their valuable assistance in the preparation and completion of this study.

In the first place, I gratefully acknowledge my supervisor, Professor Saman Bandara, whose guidance, support and encouragement from the beginning to the final level enabled me to develop an understanding of the subject, which made him a backbone of this thesis.

I take this opportunity to express my sincere gratitude to the Transportation Division, Department of Civil Engineering and University of Moratuwa for the precious opportunity. In addition, I thank all the lecturers who delivered an interesting and interactive series of lectures and guidance throughout this Post Graduate Course and I specially thank our Course Coordinator Dr.Wasantha Mampearachchi.

"Reading makes perfect", one of the main sources of my reference is the University library. In fact, I was attracted towards traffic signals zone due to a nice book found in the university library, which is the *Australian Road Research Board (ARR) Report No 12*3 *'Traffic Signals: Capacity and Timing Analysis'* by R.Akcelik. I am dedicated to thank the Librarian and staff of library, University of Moratuwa for their extended cooperation and strategic organisation in exploring such a magnificent knowledge.

Financial support was one of the prerequisites at the beginning of this course; my heartfelt thanks go to my employer the Road Development Authority and the valued recommendations made by Eng.D.K.Rohitha Swarna, Director Engineering Services and Eng.S.Yogendra, Director Training.

I am grateful to my colleagues and particularly the staff of Planning Division of Road Development Authority for their collaboration in data collection, analysis and various aid to produce this thesis.

As a part time postgraduate course, it is not easy to complete this second degree without an immense input from the family. I, especially thank my wife and family members for their enthusiastic encouragement and countless collaboration to accomplish this dissertation.

Finally, I would like to thank everyone who was important to the successful realisation of this thesis, as well as expressing my apology that I could not mention personally one by one.

A.Kamalrajh

ABSTRACT

The goal of traffic engineers in recent years is trying their best to get the most out of the systems that they develop. By designing efficient systems, using the advancement of electronics and Information Technology (IT) the overall costs of transportation should be easier to manage.

Road Development Authority under the Ministry of Ports and Highways administrates over fifty traffic signals in the Western Province of Sri Lanka. All these fixed-time traffic signals are primarily located at major routes of the city of Colombo and other townships linked with Colombo in the Western Province.

In Sri Lanka, it is anticipated that the prevailing fixed-time traffic signals would be replaced by modern Vehicle Actuated signals, which would be the next generation of traffic signals possibly be introduced in near future. This study has been carried out to evaluate the efficiency of vehicle-actuated signals against prevailing fixed-time traffic signals, prior to their implementation in Sri Lanka.

This study was carried out to compare the efficiency of Vehicle Actuated Signals against prevailing fixed-time traffic signals in an urban area. Several signalised intersections were carefully studied with their geometric and traffic turning movements especially in Colombo (the capital of Sri Lanka) region and a simulation was programmed in Microsoft Excel in such a way to represent traffic turning movements of typical intersection in urban area. Various traffic volume combinations were selected among North-South and East-West through traffic and other turning movements (Left-turns, Right-turns & Heavy vehicles) were randomised within their permissible limits (found from the analysis of existing junctions) to characterise a real dynamic situation at an urban intersection. Numerous calculations for Cycle time, Vehicle-delay, Pedestrian-delay and Critical movements of different traffic combinations were computed with the help of a well-known Australian Software called ***SIDRA*** **[S**ignalised (and unsignalised) **I**ntersection **D**esign and **R**esearch **A**id, developed by Akcelik & Associates Pty Ltd].

The outcomes of analysis were tabulated against each different traffic combinations produced by Excel simulation and were compared in graphical and tabular forms for the efficiency of *fully-Actuated Signals* against *fixed-time Signals*.

It is found that the replacement of fixed-time traffic signals with fully actuated signals for stand-alone intersections shall not produce any major enhancement (reduction in delay) to the existing at grade four-legged intersections, which have three standard-approach lanes including right turn-bays with optimum length and two standard-exit lanes.

Moreover, it is sensible that semi-actuated signals would be a better alternative for signalised intersections, where major roads (continuous high demand) meet with minor roads (very stochastic or very low traffic demand).

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List of Abbreviations or Acronyms

AG – Articulated Goods vehicles

AR – All red

CAR – Cars

CD – Compact disk

E-W – East West direction

GOSL – Government of Sri Lanka

HCM – Highway Capacity Manual

HG – Heavy Goods vehicles

HV – Heavy Vehicles

LBU – Large Buses

LGV – Light Goods vehicles

LOS – Level of Service

LT – Left turn traffic

LV – Light Vehicles

MBU – Medium Buses

MCL – Motor Cycles

MG – Medium Goods vehicle

N-S – North South direction

pdf – Portable document format

Ped – Pedestrians

Pers – Persons

PFF - Peak Flow Factor

RA – Red_Amber

RT – Right turn traffic

SIDRA – **S**ignalised (and unsignalised) **I**ntersection **D**esign and **R**esearch **A**id

Sum - Summation

TH – Through Traffic

TWL – Three Wheelers

VAN - Vans

Veh – Vehicles

List of Annexure

A CD contains electronic version of the followings:

(i) SIDRA INTERSECTION software – 30 days trial version

(ii) SIDRA user manuals

(iii) Sample SIDRA INTERSECTION out puts in portable document format (pdf)

(iv) Classified traffic turning movement data – collected from Planning Division

(v) Microsoft Excel Tables of detailed analysis for random traffic generation and signal timing and performance measures

(vi) Thesis references and bibliography

1. INTRODUCTION

Traffic signal operations have been well developed and refined over the years as part of traffic engineering. The problem of interrupted traffic flow has been a major concern as it affects not only flow rates and delay but also safety for the travelling public. In recent times, traffic signals have developed from simple fixed-time control using electro-mechanical time clocks, which was the major method of traffic control up to 30 years ago, to highly sophisticated computerized traffic actuated controllers.

The main advantage of traffic-actuated control is the ability to be responsive to highly fluctuating traffic patterns thereby reducing the overall delay to traffic at an intersection. The advantages obtained from the efficient operation of a traffic signal come in several forms. At a properly controlled intersection accident rates will be lower, the traffic flow will be increased, and delays will be minimised all of which have direct economic benefits to a country. Additionally, environmental benefits will be realised such as lower air pollution, which is directly related idling and accelerating vehicles, and lower noise pollution caused by braking, accelerating and idling of heavy vehicles.

In Sri Lanka, traffic signal control of junctions is a major technique used to manage traffic on the National Road Network other than roundabouts. Therefore, it is important that traffic signals operate as efficient as possible. Last couple of decades, Road Development Authority under the Ministry of Ports and Highways administrates over fifty traffic signals in the Western Province of Sri Lanka, other than the signals maintained by Colombo Municipal Council. All these traffic signals are fixed-time traffic signals and primarily located at entrances and exits points of the capital city of Sri Lanka and other townships linked with Colombo. Outside the Colombo district, there is far less use of traffic signal systems to control daily traffic.

One of the major development in traffic engineering signal is the advancement of vehicle-actuated signals, which is spread popularly in developed countries and being adopted in other developing countries as well. Sri Lanka is in an era to switch from traditional fixed-time signals to vehicle-actuated signals. Adopting actuated signals for Sri Lankan intersections has carefully been examined and proved rationally. Suitability of a particular type of traffic signal, either fixed or actuated for a particular junction depends on several parameters such as environmental conditions, traffic conditions, geometry of intersection, etc.

In this study, several at grade intersections in the Colombo district have been analysed to verify the appropriateness of vehicle actuated signals. The main idea of this research is to compare the efficiency of vehicle-actuated signals based on their performance against the traditional fixed-time traffic signals.

The analysis of actuated signals in the absence of real-time data would not be possible without the assistance of reliable licensed software. Road Development Authority has recently purchased an Australian software called ***SIDRA***, which is purely used for the analysis of fixed-time and vehicle-actuated signals. A lengthy analysis was carried out with the help of Microsoft Excel to generate traffic turning movements and their possible combinations to represent generalised traffic in urban area such as Colombo, the capital city of Sri Lanka. Numerous traffic turning-movement combinations were generated and unrealistic combinations were eliminated. After filtering, a set of 110 combinations of left-turns, through and right-turns for all four legs of intersection were selected for the detailed analysis. All these traffic combinations were carefully entered in graphical user interface of *SIDRA*, and processed for detailed analysis. Outcomes of cycle times, critical movements, vehicle delays and pedestrian delays were tabulated against respective turning movement combinations. Various graphs were plotted for performance comparisons of fixed and actuated traffic signals to compare the efficiency of these systems.

1. INTRODUCTION

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2. REVIEW OF LITERATURE

2.1. Introduction to Traffic Signal

There are basically, two modes of operation for traffic signals, fixed-time and actuated. Both fixed-time signals and actuated signals are controlled by microprocessors called controllers. These controllers are the brains of a signalised intersection.

Fixed-time, also known as pre-timed, controllers are of limited use for isolated intersections and require no traffic detection. Pre-timed intersections provide a specified amount of time for every traffic movement whether there are vehicles at an intersection or not. Different cycles can be set throughout the day to accommodate for peak hour traffic, but the cycle will always service every movement with a predetermined amount of time. These singles are usually found where traffic signals are closely spaced and the traffic flow patterns are evenly distributed with high pedestrian traffic, such as in urban areas.

Actuated signalised intersections, on the other hand, are more common control method used for isolated intersections and widespread throughout in major cities of popular countries. These traffic-actuated controllers can range from semi-actuated, to fully traffic actuated, to traffic responsive volume-density. These signals are capable of varying the amount of time; they give to a movement based on the traffic they service. This is the case because actuated intersections have sensors that detect the presence or passage of vehicles. This feature allows any unused green time to go to the movement that needs it. In theory, actuated signals are more efficient because the signals can adapt to different traffic patterns.

2.2. Signal Control Strategies

Several different strategies are employed for the control of traffic signals ranging from non-actuated fixed timed to fully traffic responsive volume-density control. Here different vehicle actuated controllers are discussed in detail.

2.2.1 Vehicle Actuated Controllers

The following describes the general operation of traffic-actuated controllers as stated in the research report ARR No.123 by Akcelik (1981).

At vehicle-actuated signals, the green times, and hence the cycle time, are determined according to the vehicle demands registered by detectors. Phase sequence may be fixed or variable. In the case of fixed-sequence phasing, a phase can be skipped when there is no demand for it. The running phase waits in the *rest* position when no conflicting demand is present unless there is an automatic *call* for another phase.

For a given phasing system, efficient operation of vehicle-actuated signals depends on the values of various controller settings. The three basic controller settings which determine the length of the green period are *minimum green, vehicle interval* (the terms gap time, vehicle extension, unit extension, etc. are also used) and *maximum extension* (or maximum green) settings. Modern controllers have additional settings which differ according to the type of controller (Pak-Poy et al 1975; Staunton 1976). The location, number and other characteristics of detectors affect the choice of vehicle-actuated settings also. It is therefore difficult to give general-purpose rules for choosing the values of vehicle-actuated controller settings, which can be used for any controller and location. However, some suggestions are presented below which are based on a limited amount of research published in the literature (Grace, Morris and Pak Poy 1964; Webster and Cobbe 1966; Morris and Pak-Poy 1967; Pak-Poy et al 1975; Staunton 1976).

The green period allocated to a movement comprises a minimum green period and a green extension period which is subject to an upper limit (maximum green setting). In modern controllers, the minimum green period comprises a fixed period and an additional variable period which is determined by the number of vehicle actuations (after the first vehicle) during the red period. The *fixed minimum green* and *vehicle increment* settings must be chosen to be sufficiently long for the clearance of vehicles waiting *in one lane* between the detection point and the stop line. These settings must be chosen with care so as to avoid unduly long green times which result in loss of efficiency, especially when detections in more than one lane contribute to the length of the variable minimum green period.

As illustrated in *Figure 2-1* and *Figure 2-5* (Staunton 1976), detection of each additional vehicle extends the green period by the vehicle interval setting. The controller starts timing a new vehicle interval at each vehicle actuation. The green period terminates when the time between successive vehicle actuations exceeds the vehicle interval setting (*gap change)* or the total green extension time equals the maximum extension setting *(maximum change).*

The choice of the vehicle interval setting is critical in determining the length of the green period and hence the efficiency of operation. A basic control mode uses a fixed vehicle interval setting whereas modern controllers provide facilities for automatic reduction of vehicle interval *(gap reduction)* according to the traffic flow rate on the running phase (inappropriately called 'density'), the number of vehicles waiting on the red phase, waiting time on the red phase, or combinations and variations of these algorithms (e.g. 'volume-density' and 'headway-density' controllers). A summary of various gap reduction and other vehicle-actuated controller types can be found in Staunton (1976). Various recommendations for controller settings in simple (basic), volume-density, headway-density, and a 'variable-maximum' controller are given by Pak-Poy et al (1975). These are based on the use of delay criterion only. In an earlier publication, Morris and Pak-Poy (1967) considered the effect of vehicle interval on the percentage of stopped vehicles also. Some of the findings of this and other published work (referred to above) on the effect of vehicle interval setting are as follows.

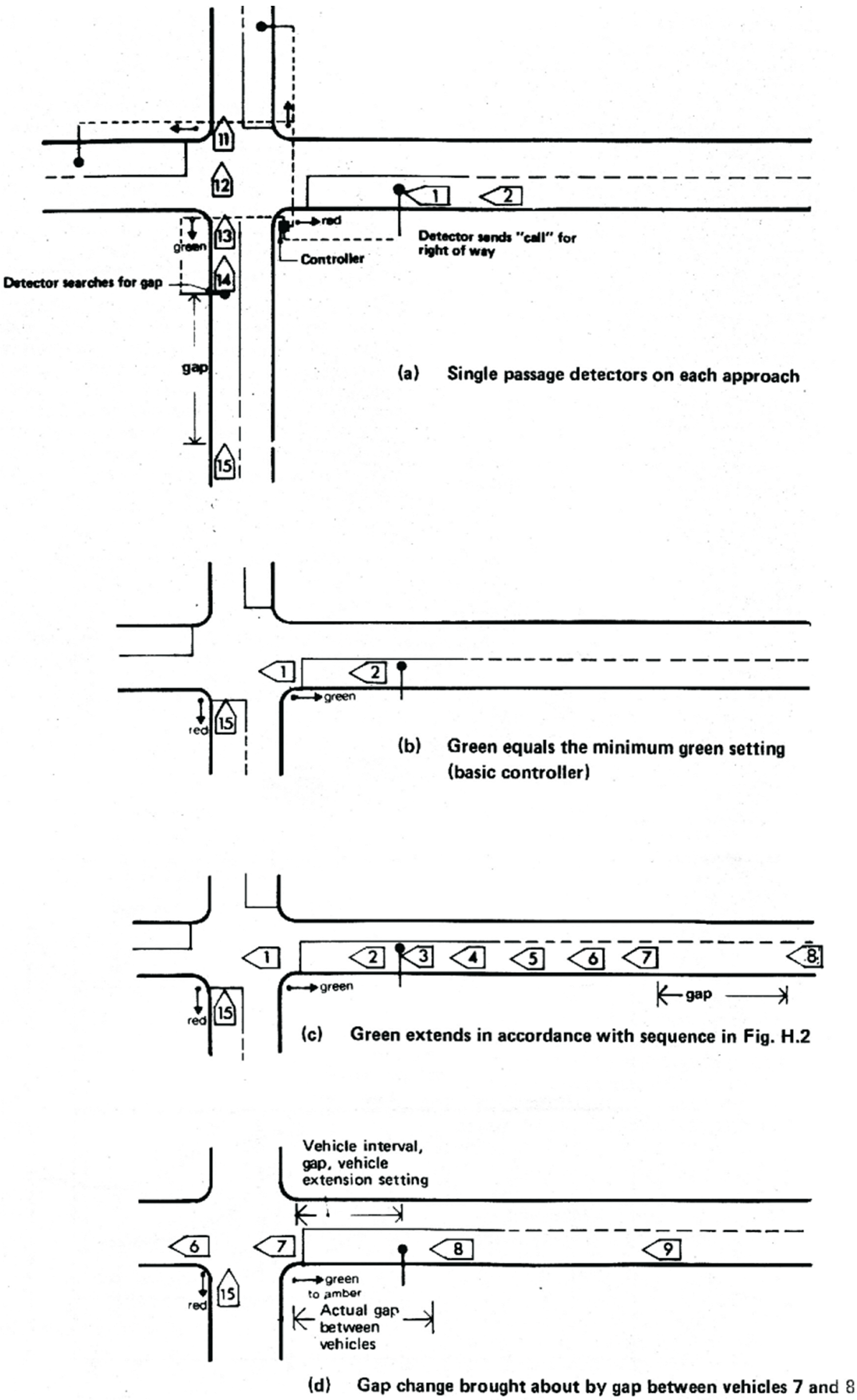

Figure 2-1: Vehicle-actuated control by vehicle interval (Staunton 1976)

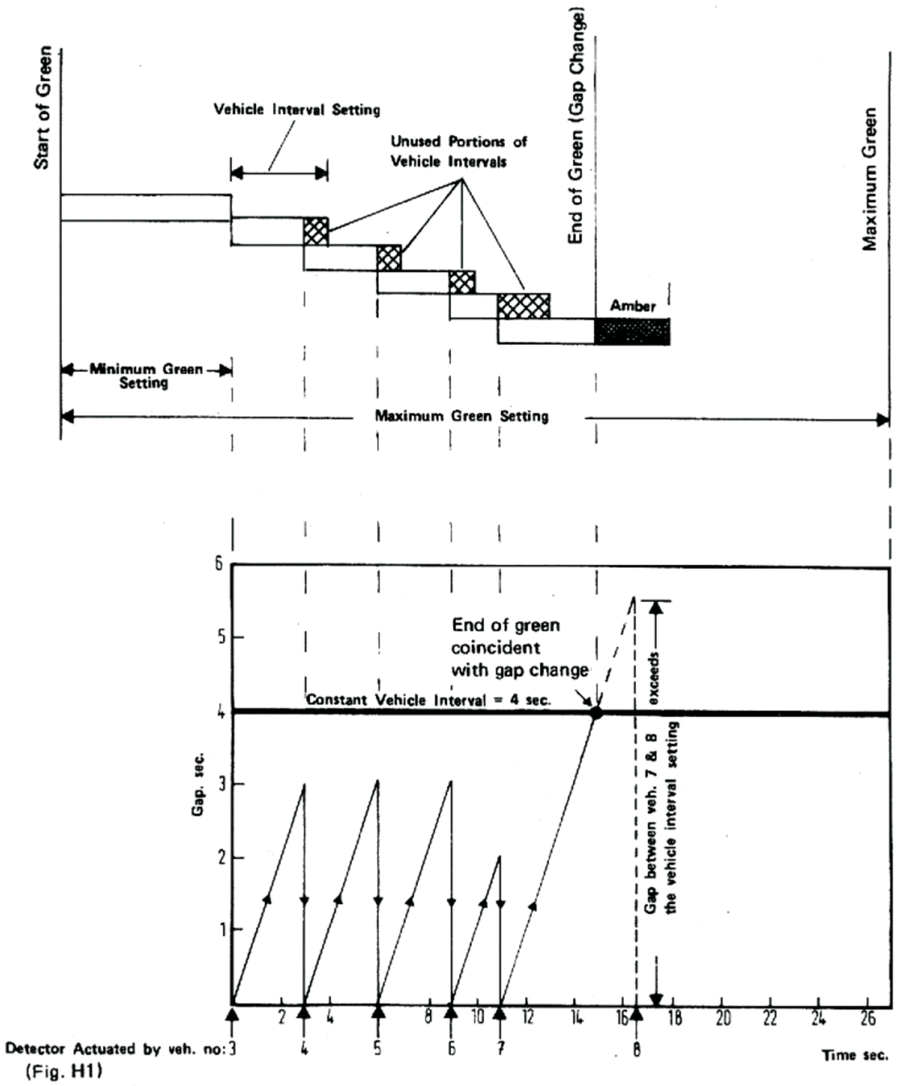

Figure 2-2: Extension sequence in a basic vehicle-actuated controller (Staunton 1976)

- There is an optimum value of the vehicle interval setting which minimises delay.
- The optimum vehicle interval becomes smaller and its effect becomes more critical as traffic flows increase *(Figure 2-3* and *Figure 2-4);* hence for relatively high flows, it is important that the vehicle interval setting chosen is not too large.
- The effect of increasing the vehicle interval is to increase the cycle time and to reduce the percentage of vehicles stopped *(Figure 2-5);* using large values of vehicle interval setting is advantageous from this viewpoint especially for low flows, where delays are not very sensitive to the value of the vehicle interval as seen in *Figure 2-4* (note that the results in *Figure 2-3* to *Figure 2-5* were obtained with no maximum setting control).

Based on these findings, and considering the importance of vehicle stops in relation to fuel consumption, cost and safety, in particular at high speed locations as emphasised by Staunton (1976), a compromise value of 3 or 4s appears to be a good choice as a *fixed* vehicle interval setting. However, the effect on the actual number of stops rather than the proportion stopped, especially under heavy flow conditions, needs to be assessed. In practice, the intersection geometry and traffic composition as well as the number and type of lanes (turning or through) per phase at a particular site (and at different times) must also be taken into account in choosing the value of vehicle interval setting.

It is apparent that basic vehicle-actuated control with a fixed vehicle interval setting will not produce efficient operating conditions (delays and stops) during both light and heavy flow periods. Gap reduction features of modern controllers are useful for this reason. However, there are difficulties in finding and maintaining optimum adjustment *Figure 2-3* of sophisticated equipment in practice as pointed out by Staunton (1976). Some modern controllers, which has 'waste' and 'headway' settings in addition to the normal 'gap' (vehicle interval) setting. With this controller, when the time between successive vehicle actuations exceeds the headway setting, the excess time (waste increment) is accumulated, in addition to normal gap.

Some variations in the operation of specific types of traffic-actuated controllers are described as follows:

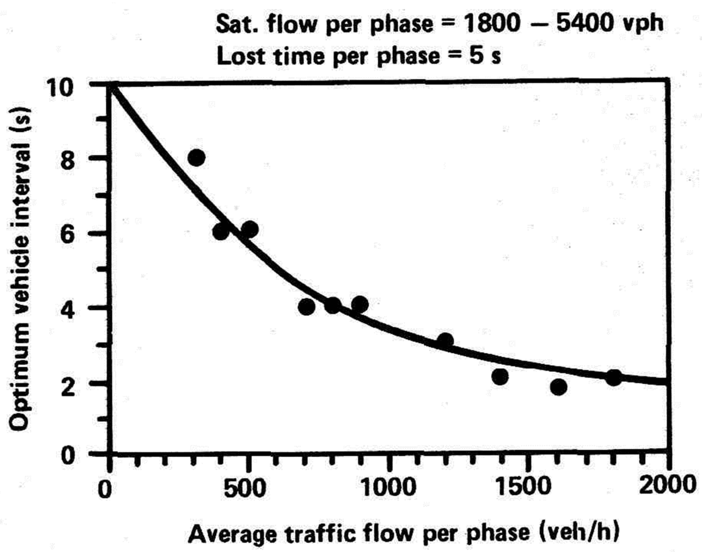

Figure 2-3: Vehicle interval with average traffic flow per phase (Morris and Pak-Poy 1967)

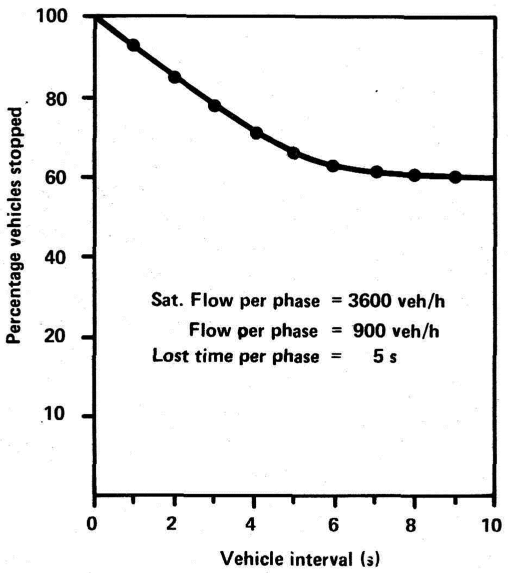

Figure 2-5: Vehicles stoppage with vehicle interval
(Morris and Pak-Poy 1967)

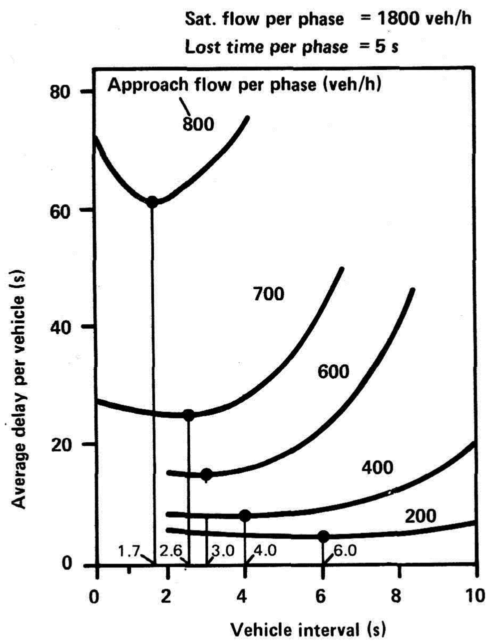

Figure 2-4: Average delay with vehicle interval
(Morris and Pak-Poy 1967)

2.2.2 Semi-Actuated Controllers

Semi-actuated controllers are used at intersections where the minor road has traffic volumes significantly lower than the major road. The priority of operation is to minimize the interruption of traffic on the major road while still providing adequate service to the minor road.

Vehicle detectors are required only on the minor road. The detectors will input a call for green time as well as calls for vehicle interval extensions up to a pre-set maximum limit.

The major road has a pre-set recall to its green phase. No detector call is required and the green will always revert to major phase when the minor road has been serviced. The major road will have a pre-set minimum green time, and will continue to rest in green on that phase until a call has been placed by the minor road.

2.2.3 Fully-Actuated Controllers

Fully actuated controllers are used where both roads at an intersection have relatively equal volumes. These controllers are particularly efficient where the traffic flows are random and uneven. The priority of operation is to *minimize the total delay* by minimizing stops on all phases.

Vehicle detectors are required on all legs of the intersection. These detectors will input calls for initial minimum green time as well as vehicle interval extensions.

Any or all phases of the signal can be set for automatic recall to green which will give the corresponding phase the minimum green time without a vehicle actuation. If all phases have automatic recall set, then the signal will cycle through all the phases, giving each phase a minimum green time, even if no traffic is present on a particular leg. If recall is not set on any phase then the signal will service only those phases that have traffic actuations and skip the others. In the absence of any traffic, such as in night-time operation, the controller can either rest on the last served green phase or rest on red on all phases. If recall is set on only one phase, the signal will revert to green on that phase once during every cycle. In the absence of traffic, the signal will rest on green on this phase. This is a common setting for many intersections, as one of the legs is usually considered slightly more major than the others.

2.2.4 Volume-Density Controllers

Volume-density controllers are similar in operation to fully actuated controllers; however, contain more advance features for analysing the traffic volumes on the green phase being served and the traffic density on the red phase being held. This information is then processed and the timing patterns altered for a more efficient operation. These controllers are the most efficient means of operation signals at isolated intersections.

- The most important feature of volume-density controllers is the ability to reduce the green vehicle extension interval depending on the density of opposing traffic. As the measured density of the opposing traffic increases the vehicle extension interval for green time is reduced linearly (known as ***gap reduction***) to some pre-set minimum extension time. *Figure 2-6* illustrates the gap reduction process.

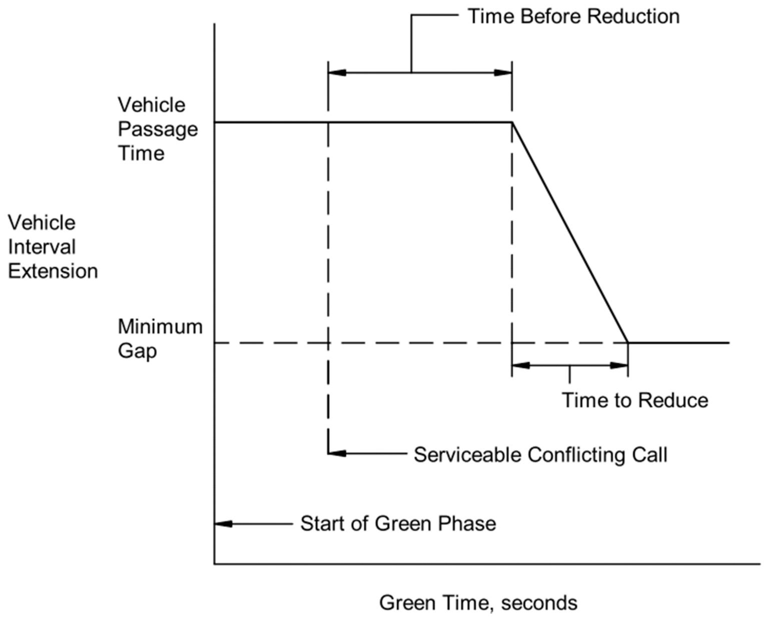

Figure 2-6: Gap reduction process (Shaflik 1995)

- The controller has the ability to increase the minimum green time depending on the number of vehicles queued behind the stop line. *Figure 2-7* illustrates the variable initial timing process.
- In order to function properly these controllers must obtain information early enough to react to the fluctuating traffic patterns. Detectors must be place well in advance of the stop lines for such information to be useful.

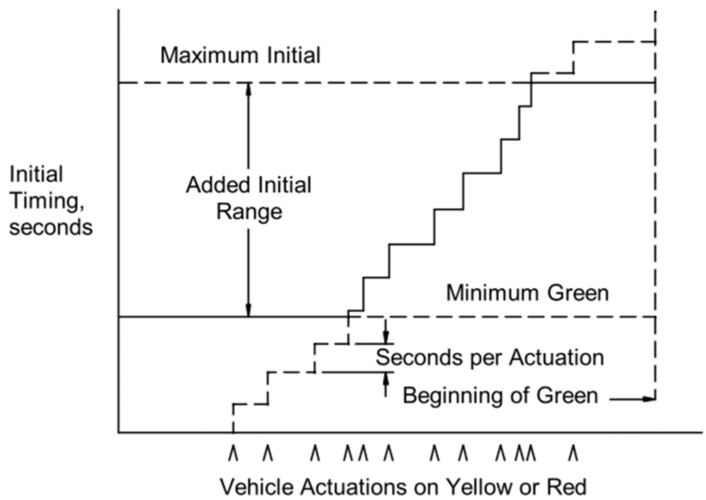

Figure 2-7: Variable initial timing process (Shaflik 1995)

A special version of the volume-density controller, known as a ***'Modified-Density Controller'***, has many of its features but requires less information from the intersection. Traffic flow statistics are obtained from the detector actuations of the previous cycle with the assumption that the present cycle being served has the same characteristics as the cycle preceding it. This may be acceptable for situation where traffic flow is relatively deterministic, however, is not efficient where intersection have highly unpredictable and random flows.

2.3. Detectors

One of the advantages to actuated control is the ability to adjust timing parameters based on actual vehicle or pedestrian demand. Since this vehicle or pedestrian demand varies at different times of the day, a detector is placed in the path of approaching vehicles or at a convenient location for the use of pedestrians.

The actual operation of the signal is highly dependent on the operation of these detectors. Some of the more common detector types are briefly described below.

- **Loop Detector**

This is the most common detector type. It is a loop of wire imbedded in the pavement carrying a small electrical current. When a large mass of metal passes over the loop, it senses a change in inductance of its inductive loop sensor by the passage or presence of a vehicle near the sensor.

- **Microwave Radar Detector**

A detector that is capable of sensing the passage of a vehicle through its field of emitted microwave energy. The principles of operation involve microwave energy being beamed on an area of roadway from an overhead antenna, and the vehicle's effect on the energy detected.

- **Video Detection**

A detector that is responds the Video image or changes in the Video image of a vehicle.

- **Magnetic Detector**

  A detector that senses changes in the earth's magnetic field caused by the movement of a vehicle near its sensor.

- **Magnetometer Detector**

  A detector that measures the difference in the level of the earth's magnetic forces caused by the passage or presence of a vehicle near its sensor.

- **Infrared Detector**

  A detector that senses radiation in the infrared spectrum.

- **Light-Sensitive Detector**

  A detector that utilizes a light-sensitive device for sensing the passage of an object interrupting a beam of light directed at the sensor.

- **Pneumatic Detector**

  A pressure-sensitive detector that uses a pneumatic tube as a sensor.

2.4. Overview of Traffic Signal Design

2.4.1 Definitions and notations

A number of definitions and notations need to be understood in signal design. They are discussed below.

2.4.2 Cycle

A signal cycle is one complete rotation through all of the indications provided in a traffic signal.

2.4.3 Phase (P or Ø)

A part of a signal cycle, that is allocated to selected traffic movement(s) receiving the right-of-way simultaneously. Furthermore, a phase is the green interval plus the change and clearance intervals that follow it. Thus, during green interval, non-conflicting movements are assigned into each phase. It allows a set of movements to flow and safely stops the flow before the phase of another set of movements start.

2.4.4 Cycle length/time (*C*)

Cycle length is the time in seconds that it takes a signal to complete one full cycle of indications. It indicates the time interval between the starting of green for one approach until the next time the green starts. It is generally denoted by *C*.

2.4.5 Interval

It indicates the change from one stage to another. There are two types of intervals *change interval* and *clearance interval*. Change interval is also called the amber (yellow) time (***A***) indicates the interval between the green and red signal indications for an approach. *Clearance interval* is also called *all red* (***AR***) is included after each amber (yellow) interval indicating a period during which all signal faces show red and is used for clearing the vehicles off the intersection (common right-of-way).

2.4.6 Amber time (*A*)

This is the time allocated for drivers to stop safely or to proceed safely depending on the location of the vehicle with respect to the stop line.

$$A = \tau + \frac{V}{2f(v)} \qquad \textbf{\textit{(Equation 2-1)}}$$

Where, τ - reaction time, *V* – speed of a vehicle, *f(v)* – Deceleration rate

2.4.7 All red time (*AR*)

The time required for a vehicle that enters the intersection at the end of amber period to clear the intersection. This period is considered one of the lost times.

$$AR = \frac{W+l}{V} \qquad \textbf{\textit{(Equation 2-2)}}$$

Where W – Total width of intersection, *l* – Length of vehicle, *V* – speed of a vehicle

2.4.8 Red Amber (*RA*)

The indication shows that, the green signal indication is going to start next. This period is considered one of the lost times.

2.4.9 Green interval or Display Green (G_i)

It is the green indication for a particular movement or set of movements and is generally denoted by G_i. This is the actual duration the green light of a traffic signal is turned on.

$$G_i = g_i + i + A \qquad \textbf{\textit{(Equation 2-3)}}$$

Where, i – starting delay (generally 1.0 s)

2.4.10 Effective Green time (g_i)

This is the time during which a given traffic movement or set of movements may proceed; it is equal to the cycle length minus the effective red time

$$g_i = C - r_i \qquad \textbf{\textit{(Equation 2-4)}}$$

2.4.11 Red interval (r_i)

It is the red indication for a particular movement or set of movements and is denoted by r_i. This is the actual duration the red light of a traffic signal is turned on.

2.4.12 Lost time (l)

It indicates the time during which the intersection is not electively utilised for any movement. For example, when the signal for an approach turns from red to green, the driver of the vehicle, who is in the front of the queue, will take some time to perceive the signal (usually called as reaction time) and some time will be lost here before he moves.

2.4.13 Inter Green Period (I)

This is the time between the end of green indication to a phase and the beginning of the green period of the following phase. Inter Green period is generally denoted by ***I***. This time can be calculated using the following formula;

$I = A + AR + RA$ ***(Equation 2-5)***

2.4.14 Signal design procedure

The signal design procedure involves six major steps. They include:

1. Phase design
2. Determination of amber (yellow) time and clearance time (all red)
3. Determination of cycle length/time
4. Apportioning of green time
5. Pedestrian crossing requirements
6. The performance evaluation of the above design

2.4.14.1 Phase design

The objective of phase design is to separate the conflicting movements in an intersection into various phases, so that movements in a phase should have no conflicts. If all the movements are to be separated with no conflicts, then a large number of phases are required. In such a situation, the objective is to design phases with minimum conflicts or with less severe conflicts.

There is no precise methodology for the design of phases. This is often guided by the geometry of the intersection, flow pattern especially the turning movements, the relative magnitudes of flow. Therefore, a trial and error procedure is often adopted. Even though, there are methods given in the class note of *PCEC 34: Traffic & Highway Capacity Design-Traffic Signal Design* (University of Moratuwa) and guidelines given in *chapter-16* of HCM (2000). However, phase design is very important because it affects the further design steps.

2.4.14.2 Determination of cycle length

An optimum cycle time that minimises mean delay to critical lanes can be found using the following empirical relationship proposed by Webster and Cobbe (1966) based on stochastic delay analysis.

$$C = \frac{1.5L+5}{1-\frac{\sum_{i=1}^{n} q_i}{S}}$$ ***(Equation 2-6)***

Where, L – Total lost time per cycle, qi – Critical lane flows, S – Saturation flow, n – number of phases

2.4.14.3 Apportioning of green time

Portion of effective green time for a particular phase can be calculated using the following formula.

$$g_i = (C - L).\frac{q_i}{\sum_{i=1}^{n} q_i} \qquad \textbf{\textit{(Equation 2-7)}}$$

Where, qi – Critical lane flow for a particular phase, C – Cycle time, L – Total lost time (s)

2.4.14.4 Pedestrian crossing requirements

If there is a pedestrian demand, it is necessary to ensure that sufficient time is offered for pedestrian to cross the appropriate approach.

The time required in seconds for a pedestrian to cross a particular approach, t_i could be calculated assuming a walking speed of 1.0 – 1.5 m/s

ε – Seconds need to be allowed for pedestrians to enter the crossing depending on the pedestrian demand and space available for pedestrian.

$$\varepsilon = 4s, if\ pedestrian\ demand\ per\ cycle\ \leq 10ped$$
$$\varepsilon = 7s, if\ pedestrian\ demand\ per\ cycle\ > 10ped$$

Also a checking has to be done, that $\varepsilon + t_i < G_i + A_i$

If this is not satisfied increase G_i to G_i^1 such that $\varepsilon + t_i = G_i + A_i$
Finally, the adjusted cycle time C^1 (increased) has to be recalculated based on G_i^1 as mentioned above.

It is a general practice that the pedestrians shall not be allowed against an arrow signal and the minimum pedestrian green time shall be 4s.

2.4.14.5 The performance evaluation

In general, vehicle delay is used to measure the performance of a signalised intersection. If the signalised intersection is considered a stationary incident of duration τ with no filtering then the delay is given by the following formula;

$$d_i = \frac{\tau^2 .q_1.q_3}{2q_1q_3} \qquad \textbf{\textit{(Equation 2-8)}}$$

Where q_1- arrival rate, q_3 – departure rate.

At a traffic signal $\tau = C.(1 - \frac{g}{C})$, $q_1 = q$ and $q_3 = S$

Therefore, delay per cycle lane = $\frac{C^2\left(1-\frac{g}{C}\right)^2.qS}{2(S-q)}$ ***(Equation 2-9)***

Total arrival per cycle = qC

Therefore, the mean delay per vehicle for a given lane is $d_i = \frac{C.\left(1-\frac{g}{C}\right)^2}{2(1-\frac{q}{S})}$ ***(Equation 2-10)***

Where, C – Cycle time, S – Saturation flow, g – Effective green time

2.5. Introduction to *SIDRA* software

The *SIDRA INTERSECTION* software is an advanced micro-analytical tool for evaluation of alternative intersection designs in terms of *capacity, level of service and a wide range of performance measures including delay, queue length* and *stops for vehicles* and *pedestrians*, as well as *fuel consumption*, *pollutant emissions* and *operating cost*.

Akcelik & Associates Pty Ltd spent over 30 years developing *SIDRA INTERSECTION* software to provide a powerful tool that helps to save time and effort, and enables production of effective solutions for local traffic conditions.

For the last 30 years, this company has been contributing to intersection modelling for traffic engineers and planners through the many unique innovations delivered in powerful and world-renowned software SIDRA INTERSECTION. Detailed information could be obtained by visiting www.sidrasolutions.com.

Original version *SIDRA 1* was developed by Rahmi Akçelik during 1975-1979 (Akçelik 1979). The word *SIDRA* is an acronym for **S**ignalised (and unsignalised) **I**ntersection **D**esign and **R**esearch **A**id. The first released of *SIDRA INTERSECTION* was in 1984 and it is a popular professional tool for traffic engineers and planners worldwide currently.

2.5.1 Traffic signal timing concept in SIDRA

Determine signal timings using fixed-time / pre-timed and actuated signal analysis methods for any intersection geometry allowing for simple as well as complex phasing arrangements involving overlap movements by selecting one of many cycle time options available. Use advanced signal timing options such as green split priority for coordinated movements, equal and unequal target degrees of saturation, cycle time optimisation for fixed-time / pre-timed signals or coordinated actuated signals, and maximum green optimisation for actuated signals.

2.5.2 Signal model features available in SIDRA

Analyse the effects of turn on red and semi-actuated signals. Make use of features such as undetected movements, dummy movements and phase transition data. Use the graphical input method for easy specification of phasing and timing data including known phase times.

2.5.3 Cycle Time and Green Split Options

SIDRA INTERSECTION offers different options to specify the desired method of cycle time calculation (subject to minimum and maximum cycle time constraints):

Practical Cycle Time: A cycle time and green times that satisfy the practical (target) degree of saturation for critical movements are determined.

Optimum Cycle Time: A cycle time that optimises a selected performance measure is determined for fixed-time/ pre-timed or coordinated actuated signals. One of a large number of options can be chosen as the performance function to determine the best cycle time.

User-Given Cycle Time: The green times using the given cycle time are determined for fixed-time/ pre-timed or coordinated actuated signals.

User Given Phase Times: The phase times given for the selected sequence are used. In this case, the phase times are added to determine the cycle time.

The *Green Split Priority* option is available for the allocation of longer green times to coordinated movements while keeping other movements at their target practical degree of saturation levels.

Parameters for *Actuated Signal* timing analysis include *Maximum Green Setting*, *Gap Setting* and *Effective Detection Zone Length*. Actuated signal timings can be optimised by varying the *Maximum Green Settings*.

2.5.4 What can *SIDRA INTERSECTION* Do?

Using SIDRA, various analyses can be performed and the information shall be obtained as follows;

- Analyse a large number of intersection types including signalised intersections (fixed-time / pre-timed and actuated), signalised pedestrian crossings, single point interchanges (signalised), roundabouts, roundabout metering, two-way stop sign control, all-way stop sign control, and give-way / yield sign-control
- Obtain estimates of capacity and performance characteristics such as *delay, queue length*, *stop rate* as well as *operating cost, fuel consumption* and *pollutant emissions* for all intersection types
- Analyse many design alternatives to optimise the intersection geometry, signal phasing and timings specifying different strategies for optimisation;
- Handle intersections with up to eight (8) legs, each with one-way or two-way traffic, one-lane or multi- lane approaches, and short lanes, slip lanes, continuous lanes and turn bans as relevant
- Determine signal timings (fixed-time / pre-timed and actuated) for any intersection geometry allowing for simple as well as complex phasing arrangements
- Carry out a design life analysis to assess impact of traffic growth
- Carry out a parameter sensitivity analysis for calibration, optimisation, evaluation and geometric design purposes
- Design intersection geometry including lane use arrangements taking advantage of the unique lane-by-lane analysis method of *SIDRA INTERSECTION*
- Design short lane lengths (turn bays, lanes with parking upstream, and loss of a lane at the exit side)

- Analyse effects of heavy vehicles on intersection performance
- Analyse complicated cases of shared lanes and opposed turns (e.g. permissive and protected phases, slip lanes, turns on red)
- Analyse oversaturated conditions making use of the time-dependent delay, queue length and stop rate models used in SIDRA INTERSECTION.

2.5.5 How Does SIDRA Intersection Work

The operation of the SIDRA INTERSECTION system is shown in the *Figure 2-8* as a Flow diagram.

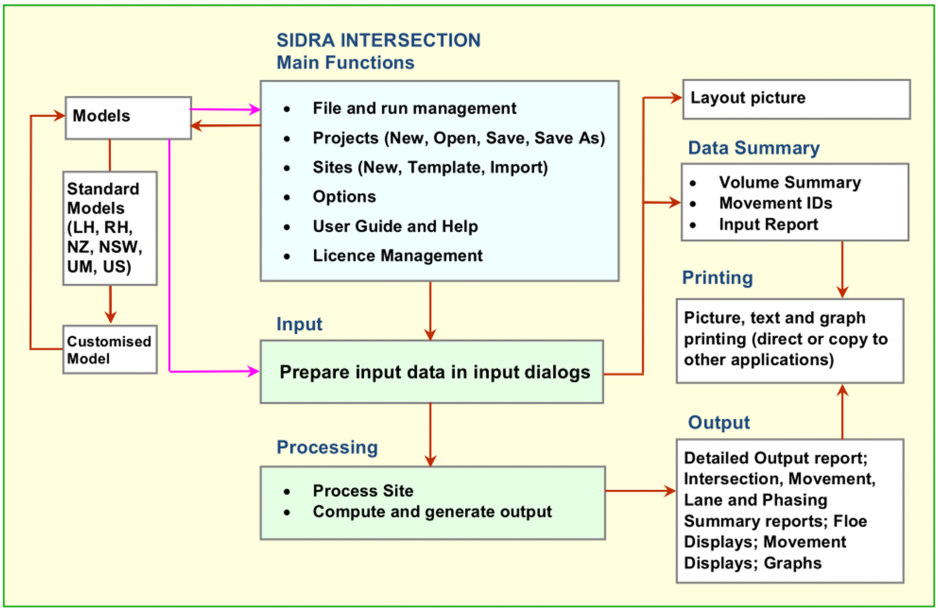

Figure 2-8: Operation of the *SIDRA INTERSECTION system*
(SIDRA Akcelik & Associates 2010)

2.5.6 Actuated signals-method used in SIDRA

The methods used in *SIDRA INTERSECTION* for actuated signal timing and performance estimation are based on results of research reported by *Akçelik (1994b, 1995a-d), Akçelik and Chung (1995a,b), Courage, et al (1996), Akçelik, Chung and Besley (1997a) and TRB (2000).*

Basic parameters in actuated signal operations are summarised in *Figure 2-9 (Akçelik, Besley and Roper 1999; AUSTROADS 2003).* The relationships among basic parameters with presence detection can be summarised as follows:

$$h = t_o + t_s = 3.6\ L_h / v = 3.6\ (L_v + L_s) / v_s \qquad \textit{(Equation 2-11)}$$

$$t_o = h - t_s = 3.6\ (L_p + L_v) / v \qquad \textit{(Equation 2-12)}$$

$$t_s = h - t_o = 3.6\ (L_s - L_p) / v \qquad \textit{(Equation 2-13)}$$

$$L_h = L_v + L_s = h.v / 3.6 \qquad \textit{(Equation 2-14)}$$

Where;
h = headway (seconds)
L_v = vehicle length (m)
L_h = vehicle spacing (m)
L_s = space (gap) length between vehicles (m)
L_p = detection zone length (m)
t_o = occupancy time (seconds)
t_s = space (non-occupancy) time (seconds)
v = vehicle speed (km/h)

The performance models for actuated signals use the same general modelling framework used in *SIDRA INTERSECTION* for all intersection types.
For ***Fully Actuated Signals***, i.e. when all movements have been specified as Non-coordinated (isolated) and no movement has been specified as Non-Actuated, the equations for actuated signals are used with ***no platooned arrival effects***.

If the program determines that some movements have been specified as Non-actuated or Signal Coordination data (non-isolated) have been given for some movements in the *Movement Data dialog (refer sample input data in annex)*, it will decide that this intersection is a case of *Coordinated-Actuated Signals* or *Isolated Semi-Actuated Signals*. If some movements have been specified as Coordinated, this is likely to be a case of Coordinated-Actuated Signals. If all movements have been specified as Non-coordinated, this is a case of Isolated Semi-Actuated Signals. The method assumes that coordination can be specified for some movements at semi-actuated signals in order to model the effects of platooned arrivals. In these cases:

- the equations for fixed-time (pretimed) signals will be used for Non-actuated movements,
- the equations for actuated signals will be used for actuated movements (i.e. the movements not specified as Non-actuated), and
- For coordinated movements (Non-actuated or not), platooned arrival effects will apply to performance calculations (parameters PF_1, PF_2, f_{p1} and f_{p2}) as well as opposed turn and short lane models.

The HCM Delay and HCM Queue options are used for the HCM versions of SIDRA INTERSECTION. These can be accessed by cloning a model (Model tab) and using Model Defaults - Model General for the cloned model.

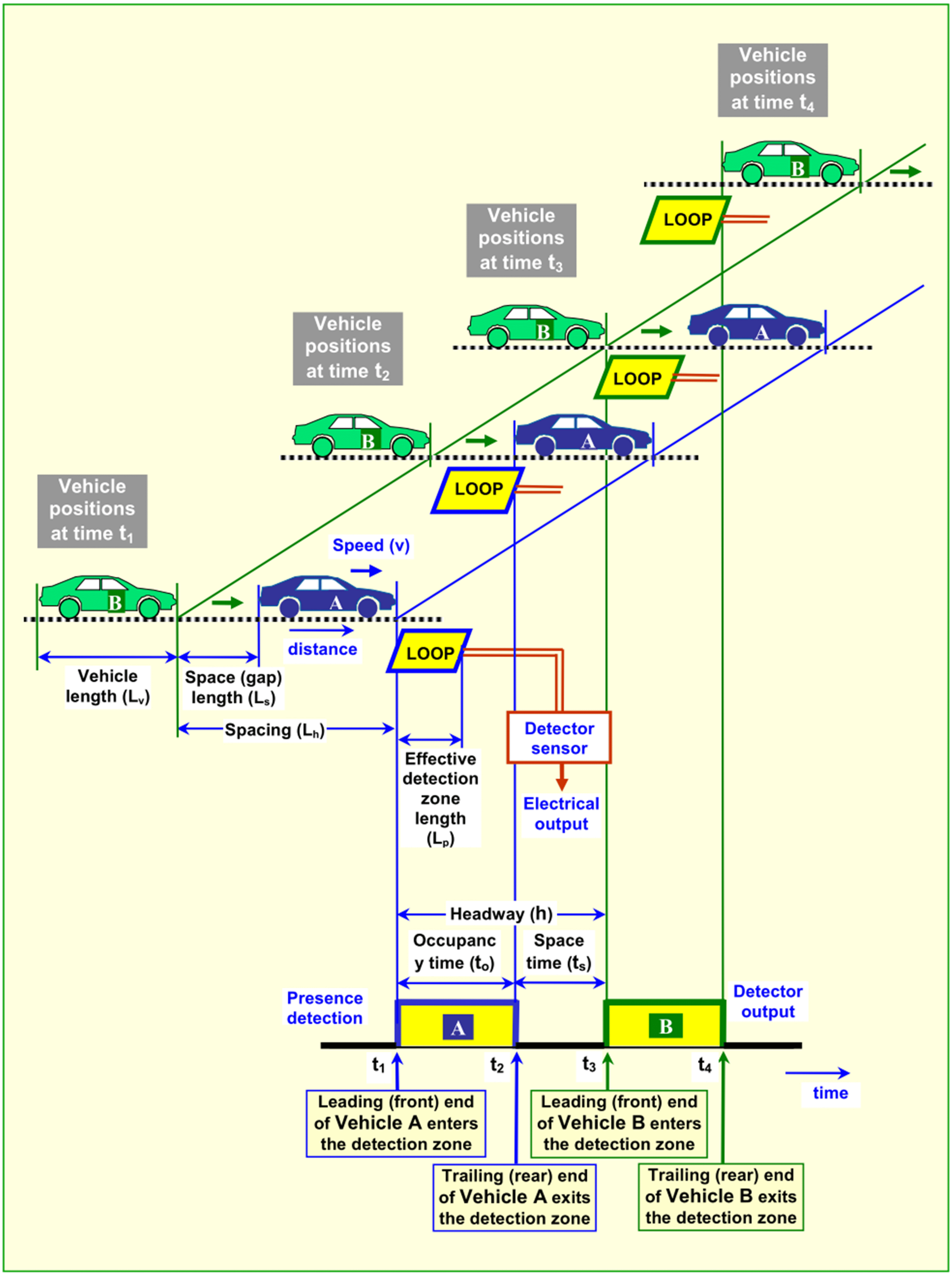

Figure 2-9: Basic parameters in actuated signal operation (SIDRA Akcelik & Associates 2010)

For pedestrian movement performance calculations, the same method as for fixed-time / pre-timed signals is used. However, pedestrian green times are different for actuated and fixed-time / pre-timed signals. *In the case of actuated signals, pedestrian movements are allocated the minimum pedestrian time* whereas full vehicle green time is available for pedestrians (except clearance time losses) in the case of fixed-time / pre-timed signals.

SIDRA INTERSECTION uses maximum green setting (G_{max}), gap setting (e_s) and effective detection zone length (L_p) parameters for actuated signal control timing and performance estimation purposes. These parameters can be specified as global parameters (i.e. to apply to all actuated movements in the Site) in the Sequence Data dialog (refer sample input in annex). Standard default values of these parameters are given in *Table 2-2*.

Table 2-1: Default values of actuated signal settings in SIDRA(Akcelik & Associates 2010)

	Max. Green Setting (s)	Gap Setting (s)	Effective Detection Zone Length (m or ft)
Major movement	50	2.5	4.5 m (15 ft)
Minor movement (1)	20	2.0	4.5 m (15 ft)

(1) Applies to arrow-controlled (protected) turns only.

For driving on the left-hand side of the road (Australia, New Zealand, Japan, UK, etc):

- Major movements are the through and left-turn movements, and
- Minor movements are arrow-controlled (protected) right-turn movements

For the purpose of actuated signal timing calculations in *SIDRA INTERSECTION*, the term "minor movement" refers to arrow-controlled turn (protected) movements, and does not include opposed (filter, or permitted) turn movements. The through ("major") movement settings are used for opposed (filter, or permitted) turn movements in shared or exclusive lanes. In the case of permitted and protected turns (green circle and green arrow), the through movement parameters are used for the permitted (green circle) period and the minor movement parameters are used for the protected (green arrow) period.

In some signal controllers, a maximum green extension setting (G_{emax}) is used. For the purpose of *SIDRA INTERSECTION*, the maximum green setting represents the sum of minimum green time and the maximum green extension setting ($G_{max} = G_{emax} + G_{min}$).

In accordance with the general definition, a right-turn movement in the case of driving on the left-hand side of the road from the stem of a *T-junction* or from a one-way street may also be treated as a "minor movement", resulting in a default maximum green setting, which is too short. Where this is not desirable due to the nature of this movement, the maximum green setting should be set for this movement in the Movement Timing dialog.

The maximum green setting, gap setting and effective detection zone length parameters can be specified in the Sequence Data dialog (refer sample input in annex). Any maximum green times given in the *Movement Timing dialog* for individual movements will override the values given in the *Sequence Data dialog*.

The gap setting and effective detection zone length parameters are used as global values only. The gap setting is a *space-time* value (e_s) as used with presence detection, i.e. headway time less detector occupancy time. For the purpose of signal timing and performance calculations, *SIDRA INTERSECTION* will convert this setting to a *headway time* value (e_h) for each movement. The corresponding parameters in *Figure 2-11* are headway (h) and space-time
($t_s = h - t_o$ where t_o = occupancy time).

In the *Detailed Output* report, the *Progression and Actuated Signal Parameters* table lists the space-time and headway time values of gap settings.

SIDRA INTERSECTION converts the gap settings from space-time values to headway values using the following formula:

$$e_h = e_s + t_{ou} = e_s + 3.6\,(L_v + L_p) / v_{ac} \qquad \textbf{\textit{(Equation 2-15)}}$$

where t_{ou} is the detector occupancy time (s/veh) during the unsaturated part of the green period (i.e. after queue clearance), L_v is the average vehicle length (m/veh), L_p is the effective detection zone length (m) which is typically within r ± 0.5 m of the detector loop length, and v_{ac} is the approach speed (km/h) which is used as the departure speed of vehicles after queue clearance.

SIDRA INTERSECTION calculates the average vehicle length from:

$L_v = L_{hj} - 2.0$ ***(Equation 2-16)***

Where L_{hj} is the average spacing in a stationary queue, or jam spacing (m/veh), which is calculated as an average value considering all lanes used by the subject movement.

Equation 2-6 assumes a jam space length (gap distance between vehicles) of Lsj = 2.0 m. The average jam spacing is calculated as a flow-weighted average of the jam spacing values of light vehicles (LVs) and heavy vehicles (HVs) considering all lanes used by the subject movement:

$L_{hj} = (1 - p_{HV})\, L_{hjLV} + p_{HV}\, L_{hjHV}$ ***(Equation 2-17)***

where p_{HV} = proportion of heavy vehicles, L_{hjLV} and L_{hjHV} are the average jam spacing per vehicle (m/veh) for light and heavy vehicles, respectively (parameters L_{hjLV} and L_{hjHV} are specified as input in the *Movement Data dialog*).

For example, L_{hjLV} = 7.0 m, L_{hjHV} = 13.0 m, and p_{HV} = 0.05 (5% HVs) gives L_{hj} = 7.3 m. The average vehicle length is L_v = 7.3 - 2.0 = 5.3 m. For e_s = 2.5 s, L_p = 4.5 m and v_{ac} = 60 km/h, the occupancy time is t_{ou} = 3.6 x (5.3 + 4.5) / 60 = 0.6 s, therefore e_h = 2.5 + 0.6 = 3.1 s.

2.5.7 Actuated Signal Timing Method

For traditional vehicle-actuated control, unequal degrees of saturation are likely to result with lower degrees of saturation allocated to the minor movements as seen in *the example given in Figure 2-10* (Akçelik 1994b, 1995a,d, 1997a; Akçelik, Chung and Besley 1997a; Courage, et al 1996).

SIDRA INTERSECTION uses a simple method for calculating average green and cycle times at actuated signals, making use of estimates of degrees of saturation at actuated signals (x_a) as target degrees of saturation in the practical cycle time and green split equations. The method is similar to the method for the analysis of fixed-time (pre-timed) signals, and is consistent with the method described in HCM 2000 (TRB 2000).

An iterative timing calculation method is used by calculating the target degree of saturation for actuated signals (x_a) from the following equations:

(i) For initial calculations when the red time (r) is not known:

$$x_a = 1.5\ y^{0.5} e_h^{-0.1} \qquad \textit{subject to } 0.40 \le x_a \le 0.95 \qquad \textbf{(Equation 2-18)}$$

(ii) For subsequent iterations when the red time (r) is known:

$$x_a = 0.78\ y^{0.5} e_h^{-0.1}\ r^{0.18} \qquad \textit{subject to } 0.40 \le x_a \le 0.95 \qquad \textbf{(Equation 2-19)}$$

Where y = flow ratio (arrival flow rate / saturation flow rate), e_h = gap setting as a headway value (seconds), and r = effective red time (seconds).

The use of actuated signal degrees of saturation (x_a) from the above equations results in unequal degrees of saturation for critical movements reflecting the results of real-life actuated signal operations, and contrasts with the equal degree of saturation (EQUISAT) method used for fixed-time (pre-timed) signals.

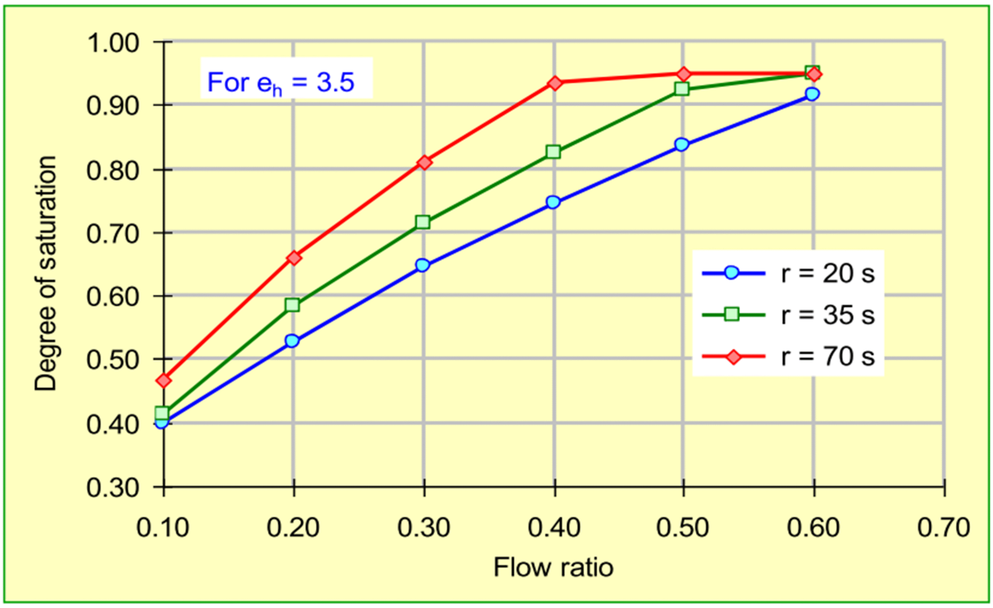

Figure 2-10: Degrees of saturation at vehicle-actuated signals (SIDRA Akcelik & Associates 2010)

The actuated signal timings are very sensitive to the maximum green time settings particularly under heavy demand conditions. Very long cycle times may result if large values of maximum green time settings are used, resulting in significant capacity losses due to short lane, permitted (filter) turn and lane blockage effects. Optimisation can be done by trying to optimise maximum green time settings using the *Sensitivity Analysis option* in the Demand & Sensitivity dialog.

The actuated signal timing calculations use the same method as for fixed-time signals with the following differences:

1. For Fully-Actuated Signals, i.e. when all movements have been specified as Actuated and Non-coordinated, the actuated signal degrees of saturation (x_a) are used for all movements instead of the practical degree of saturation (x_p) in calculating the required green time. In this case:

- The maximum cycle time constraint is not used, and maximum green times determine the value of largest possible cycle time.
- Green split priority method will not apply
- Cycle time cannot be specified by the user unless green splits are also user-specified
- Optimum cycle time calculations cannot be carried out.

2. With some movements specified as *Non-Actuated* (Fixed-Time / Pretimed) for *Coordinated-Actuated* or *Isolated Semi-Actuated* signals:

- User-specified target degrees of saturation (x_p) will be used for Non-Actuated movements, and the actuated signal degrees of saturation (x_a) will be used for Actuated movements.
- The maximum cycle time constraint applies
- Green split priority method applies. Priority movements should be specified as the coordinated movements in the case of Coordinated Actuated Signals, and as the *Non-Actuated* movements in the case of *Semi-actuated* Signal Control
- Cycle time can be specified for the program to determine green splits
- Optimum cycle time calculations can be carried out (*Optimum Cycle Time* facility)

3. If all movements are specified as Actuated with some or all movements specified as *Coordinated*, all aspects of case (2) apply with the exception that:

- Actuated signal degrees of saturation (x_a) are used for all movements.
- The maximum cycle time constraint does not apply.

In cases (i) to (iii) above:

- The cycle time increment is forced to 1 second (the user-specified value will be ignored). The calculated cycle time is rounded to the nearest integer value (not rounded up).
- No green time adjustments are applied to balance movement degrees of saturation
- Allowance is made for occasional pedestrian calls using a simple method, which reduces the pedestrian minimum green time used in timing calculations according to the probability of no pedestrian arrivals (Akçelik 1995a).

2.5.8 Delay

SIDRA INTERSECTION output includes estimates of average delay and the corresponding Levels of Service (LOS) for movements, lanes, approaches and the intersection in Detailed Output, Intersection Summary, Movement Summary, and Lane Summary reports, Movement Displays and Graphs as appropriate.

Delay to a vehicle is the difference between interrupted and uninterrupted travel times through the intersection as seen in *Figure 2-11* which shows the delay experienced by a through vehicle stopping and starting at traffic signals (time-distance and speed-time diagrams representing the acceleration and deceleration manoeuvres of the vehicle are shown).

The average delay predicted by *SIDRA INTERSECTION* is for all vehicles, queued and unqueued. Based on this definition, the total (aggregate) delay (vehicle-hours per hour) is the product of average delay and the total demand flow rate. The total delay for a movement (or lane group) is the sum of total delays for all lanes that are used by the movement (or belong to the lane group) allowing for the proportion of movement demand flow in each lane.

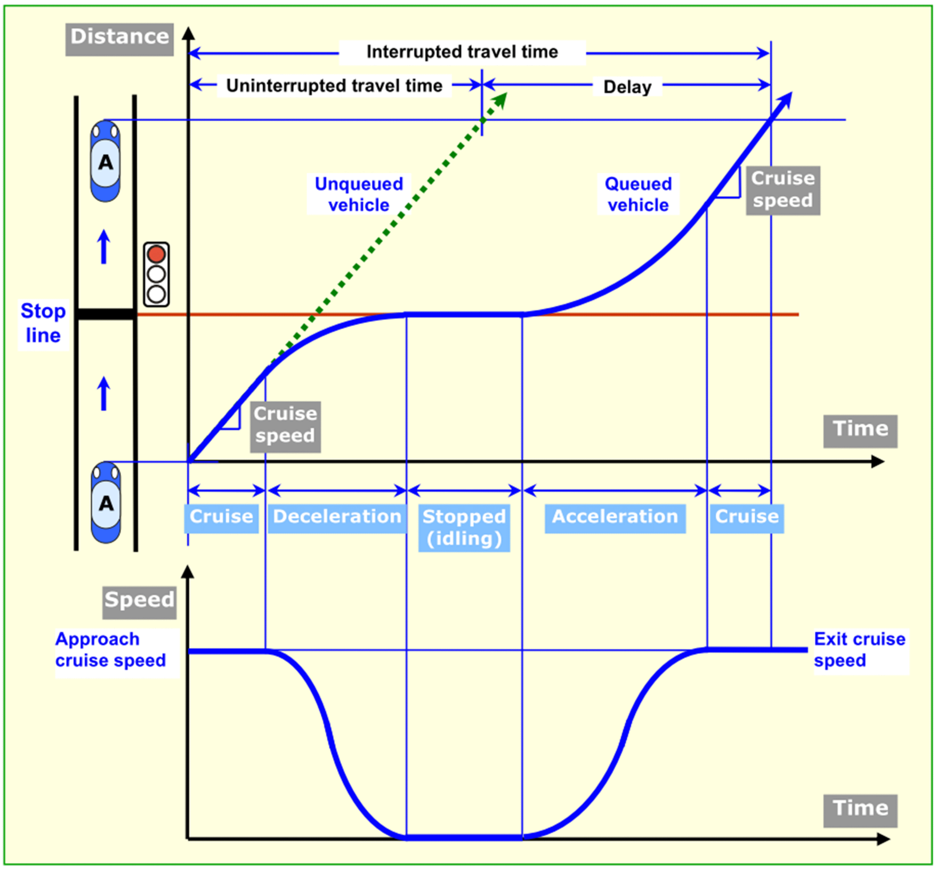

Figure 2-11: Delay definition, experienced by vehicles at traffic signals (SIDRA Akcelik & Associates 2010)

SIDRA INTERSECTION also gives the total delay in terms of person delays (person-hours per hour) using the Vehicle Occupancy (persons/vehicle) factor. Total delay values are given for vehicles, pedestrians and all persons separately.

The total vehicle delay (veh-h/h) and the total pedestrian delay (ped-h/h) are not added together directly, and the total intersection delay (including delay experienced by vehicles and pedestrians) is given in terms of total person delay (pers-h/h) only.

The average *intersection delay* is based on persons, i.e. determined by dividing the total intersection delay (pers-h/h) by the total flow in persons/h. Since different level of service criteria are used for vehicles and pedestrians, the intersection level of service is provided separately for vehicles and pedestrians based on their respective average delays, and an intersection level of service based on person delay is not provided. Similarly, the average delay used in Optimum Cycle Time, Demand Analysis (Design Life / Flow Scale) and Sensitivity Analysis runs is based on *person delay*.

2.5.9 Delay Measurement

SIDRA INTERSECTION delay is the average delay to vehicles arriving during a given flow period including the delay experienced after the end of the flow period which is possible under heavy (especially oversaturated) traffic conditions. This corresponds to the path-trace (instrumented car) method of measuring delays. An alternative delay measurement method is the queue-sampling method, which involves counting the number of vehicles in the queue at regular intervals, e.g. every 5 seconds.

Delays obtained using the *path-trace method* agree with the *queue sampling method* of measurement for low to medium degrees of saturation (v/c ratios), but the difference between the two methods is significant for oversaturated conditions (degree of saturation > 1). More detailed information is found in Akçelik (1981, 1988b, 1990a,b, 1996a,b); Akçelik and Chung (1994b); Akçelik and Rouphail (1993, 1994); Brilon and Wu (1990); Rouphail and Akçelik (1992).

Figure 2-12 shows the delays experienced by individual vehicles (horizontal lines) and the queue counts (vertical lines) for a deterministic oversaturation model to explain the concepts involved. The delay experienced by the last vehicle *departing* during the current flow period, which arrives at point C (time T1) and departs at point E (time T_f) is d_1. The delay experienced by the last vehicle arriving during the current flow period, which arrives at point C (time T_f) and departs at point D (time T_2) is d_2.

In *Figure 2-12*, the total delay for vehicles arriving during the current flow period (duration T_f) is represented by the triangular area ACD. This includes the total delay experienced after the current flow period (area CDE). End of oversaturation is at point F (achieved due to a lower arrival rate after the current flow period).

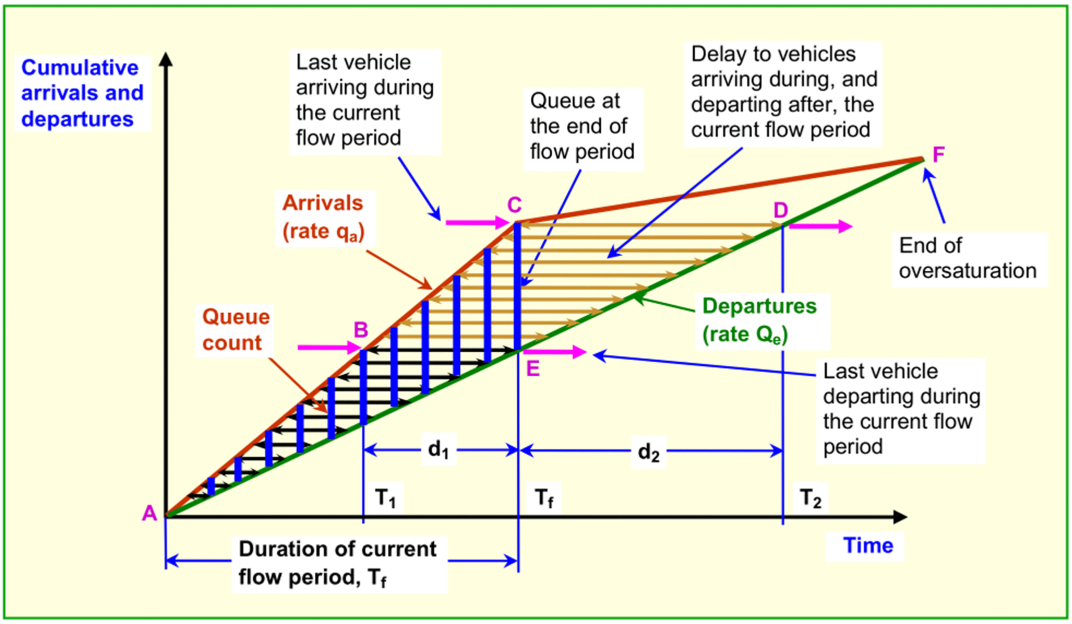

Figure 2-12: Delays experienced by vehicles in oversaturated conditions (SIDRA Akcelik & Associates 2010)

The queue sampling method counts the number of vehicles in the queue during T_f. The corresponding total delay is represented by the area ACE. The average delays are obtained by dividing the total delays (areas ACD and ACE) by the number of vehicles arriving during the current flow period (q_a T_f). This shows the significant difference in delays accounted for by the path-trace and queue-sampling methods in the case of oversaturation (queue build up with a residual queue at the end of the current flow period).

2.5.10 Delay definitions

The following are useful delay definitions, which form the basis of the *SIDRA INTERSECTION* method. The definitions are presented with the help of *Figure 2-13* which depicts a vehicle turning left at an intersection where the approach and exit cruise speeds are the same (v_{ac} = v_{ec}), and the approach and exit negotiation speeds are the same (v_{an} = v_{en}):

(i) ***Intersection control delay* (d_{ic}):** This is sum of *stop-line* and geometric delays (d_{ic} = d_{SL} + d_{ig}), thus it includes all deceleration and acceleration delays experienced in negotiating the intersection. In earlier versions of *SIDRA INTERSECTION*, this was referred to as the ***overall delay with geometric delay***.

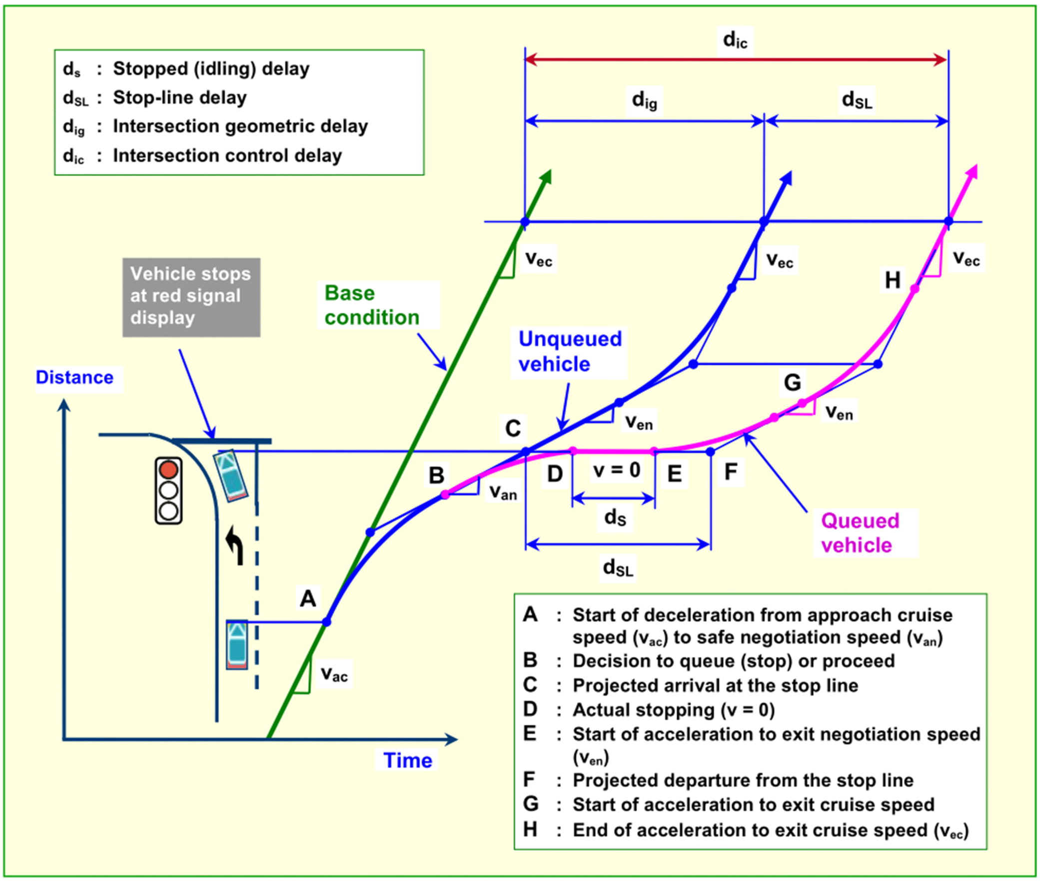

Figure 2-13: Graphical representation of various delays used in SIDRA (SIDRA Akcelik & Associates 2010)

The delay to a vehicle, which decelerates from the approach cruise speed to a full stop (due to a reason such as a red signal, a queue ahead, or lack of an acceptable gap), waits and then accelerates to the exit cruise speed is considered to include the delay due to a deceleration from the approach cruise speed down to an approach negotiation speed and then to zero speed, idling time, acceleration to an exit negotiation speed along the negotiation distance, travelling the rest of the negotiation distance (if any) at the constant exit negotiation speed, and then acceleration to the exit cruise speed. As seen in *Figure 2-13*, this delay is the ***intersection control delay***.

(ii) ***Stop-line delay*** (d_{SL}): This is calculated by projecting the time-distance trajectory of a queued vehicle from the approach and exit negotiation speeds to the stop line (or give-way / yield line), which is shown as the time from C to F in *Figure 2-13*. The stopline delay is equivalent to queuing delay plus main stop-start delay, and is represented by the first two terms of the delay model ($d_{SL} = d_q + d_n = d_1 + d_2$).

(iii) ***Geometric delay*** (d_{ig}): This is the delay experienced by a vehicle going through (negotiating) the intersection in the absence of any other vehicles, which is of particular interest for satisfactory modelling of the performance of roundabouts and sign controlled intersections. Intersection geometric delay is due to a deceleration from the approach cruise speed down to an approach negotiation speed ($v_{ac} \rightarrow v_{an}$), travel at that speed (v_{an}), acceleration to an exit negotiation speed ($v_{an} \rightarrow v_{en}$), travel the rest of exit negotiation distance at constant exit negotiation speed (v_{en}) and then acceleration to the exit cruise speed ($v_{en} \rightarrow v_{ec}$). Thus, this delay includes the effects of the physical (geometric) characteristics of the intersection (negotiation radius and distance, and the associated speeds), as well as the effects of basic control features (e.g. a stop sign vs a give-way / yield sign).

(iv) ***Queuing delay*** (d_q): This is part of the stop-line delay that includes stopped delay and queue move-up delay but does not include the main stop-start delay ($d_q = d_s + d_{qm} = d_{SL} - d_n$). The queue move-up delay is not shown in the example given in *Figure 2-13*.

(v) ***Stopped delay*** (d_i or d_s): This is the stopped (idling) time at near-zero speed. It is the delay excluding all deceleration and acceleration delays (i.e. not including any geometric, stop-start and queue move-up delays), thus it is equivalent to queuing delay less queue move-up delay ($d_s = d_q - d_{qm}$).

(vi) ***Queue move-up delay*** (d_{qm}): This is the delay associated with queue move-ups, i.e. acceleration from zero speed to queue move-up speed and deceleration to zero speed ($0 \rightarrow v_{qm} \rightarrow 0$)

(vii) ***Main stop-start delay*** (d_n): This is associated with deceleration from the *approach negotiation* speed to zero speed and acceleration back to the exit negotiation speed ($v_{an} \rightarrow 0 \rightarrow v_{en}$).

A more general method is used in *SIDRA INTERSECTION* that allows for different approach and exit cruise speeds ($v_{ac} \neq v_{ec}$) and different approach and exit negotiation speeds ($v_{an} \neq v_{en}$). When the approach and exit cruise speeds are different, the *base condition* involves an acceleration ($v_{ac} < v_{ec}$) or deceleration ($v_{ac} > v_{ec}$). Similarly, when the approach and exit negotiation speeds are different, the *geometric delay* derivation involves an acceleration ($v_{an} < v_{en}$) or deceleration ($v_{an} > v_{en}$).

The Lane Delays table in the Detailed Output of SIDRA report gives a detailed breakdown of delay types (i) to (vii).

The delay to a vehicle which decelerates from the approach cruise speed to a full stop (due to a reason such as a red signal, a queue ahead, or lack of an acceptable gap), waits and then accelerates to the exit cruise speed is considered to include the delay due to a deceleration from the approach cruise speed down to an approach negotiation speed and then to zero speed, idling time, acceleration to an exit negotiation speed along the negotiation distance, travelling the rest of the negotiation distance (if any) at the constant exit negotiation speed, and then acceleration to the exit cruise speed. As seen in Figure 2-13, this delay is the intersection control delay (overall delay with geometric delay).

A key construct used in developing the *SIDRA INTERSECTION* delay definitions given above was a clarification of whether the delay estimated by a traditional analytical delay model includes any acceleration and deceleration delays. The *SIDRA INTERSECTION* method assumes that the analytical model delay is a *stop-line delay* that includes the *main stop-start delay* to queued vehicles, and does not include the *geometric delay*.

In *Figure 2-13*, the main stop-start delay is represented by $d_n = t_{CD} + t_{EF}$, i.e. the sum of times C to D and E to F, the stopped delay is represented by $d_s = t_{DE}$, and therefore the stop-line delay is $d_{SL} = t_{CF}$ (in this example, the stopped delay is the same as the queuing delay because there is no queue move-up delay).

In determining control delay for individual movements, control delay values for the lanes used by the movement are not aggregated directly. The stop-line delay values for the lanes used by the movement are aggregated first, and then the geometric delay for the movement is added. Geometric delay and other statistics for movements combined using the same movement number are the flow-weighted average values for individual origin-destination movements.

The use of control delay (overall delay with geometric delay) is the recommended method for consistency in comparing alternative intersection treatments. *Stop-line delay* given in the *Lane Delays* table in the Detailed Output report is recommended only for comparison of *SIDRA INTERSECTION* results with those from software packages that estimate delay without the geometric delay, or when the survey method used produces a delay that does not include the geometric delay.

The delay models used by *SIDRA INTERSECTION* when the HCM Delay option is applicable (Model Defaults - Model General) differ from the standard *SIDRA INTERSECTION* models although the model structures are similar.

For ***continuous movements***, an uninterrupted travel delay (difference between zero-flow travel time and the uninterrupted travel time at given flow level) is calculated. The value of this is usually small. However, the geometric delay is significant for continuous left and right-turn movements (hence, the control delay will be much higher than the uninterrupted travel delay).

For signal coordination (platooned arrivals), the HCM *progression factor* method is used for delay prediction. In *SIDRA INTERSECTION*, an additional progression factor is used for the prediction of queue-related performance statistics. The colour code used for movements in the Control Delay display under Movement Displays is based on the Level of Service (LOS) values as indicated by the legend of the display. This varies according to the LOS method used. For continuous movements, grey colour indicates that LOS is not allocated to these movements.

2.5.11 PEDESTRIANS

Pedestrians can be modelled at fixed-time (pretimed) or actuated signalised intersections, mid-block signalised crossings, and single point urban interchange facilities allowed by *SIDRA INTERSECTION*. This section explains various aspects of pedestrian movements at these facilities. Pedestrians at unsignalised (Zebra) crossings are also discussed below.

Pedestrian movements are defined as crossing in front of the selected approach road. There will be one pedestrian movement per approach for Full Crossings, and two pedestrian movements (one for each carriageway) for Staged Crossings. The user has the option of specifying different volumes for each stage at a staged crossing

2.5.11.1 Pedestrian Saturation Flow Rate and Performance Measures

For pedestrian movements, a fixed saturation flow of 12,000 ped/h is used as a default. This can be changed in the Movement Data dialog. The pedestrian saturation flow rate represents a number of pedestrians in each row of the queue (with increasing pedestrian volumes, pedestrians tend to queue side by side in increasing numbers, thus increasing the saturation flow rate). The pedestrian saturation flow rate (s_p in ped/h) can be estimated from:

$s_p = 3600\ n_p / h_{sp1}$ ***(Equation 2-20)***

Where, n_p is the number of pedestrians in a row in the queue, and (h_{sp1} is the queue discharge headway between two pedestrian rows (seconds) :

$h_{sp1} = 0.8 + L_{hjp} / 1.5$ ***(Equation 2-21)***

Where L_{hjp} is the pedestrian queue spacing (default value is 1.0 m, or 3 ft. for the HCM version).

For pedestrian queue length estimation, *SIDRA INTERSECTION* estimates the number of pedestrians in a row ("number of pedestrian lanes") from:

$n_p = s_p\ h_{sp1} / 3600$ ***(Equation 2-22)***

For the *SIDRA INTERSECTION* default values of s_p = 12,000 ped/h and L_{hjp} = 1.0 m, $h_{sp1} \approx 1.5$ s and n_p = (12,000 / 3600) x 1.5 = 5 pedestrians in a row.

As a result of the large saturation flow rate value, pedestrian degrees of saturation and queue clearance times are small. For this reason, *SIDRA INTERSECTION* estimates the pedestrian performance measures (delay, queue length and effective stop rate) ignoring the second term of the performance equations (i.e. no overflow effect), and sets the flow ratio to zero (equivalent to assuming a very large saturation flow rate). Therefore, the pedestrian performance measures include the red time effect only.

In *SIDRA INTERSECTION*, appropriate values of approach distance, speed, queue-space parameters are used for pedestrian movements. The approach distance is 10 m (or 30 ft.), and the walking speed is 1.3 m/s (or 4.3 ft/s) for pedestrian movements. These parameters are used for the purpose of pedestrian travel statistics. For pedestrian green time calculations, a crossing speed of 1.2 m/s (or 4.0 ft/s) is used which is discussed in detail below.

2.5.11.2 Pedestrian Effect on Capacity of Vehicle Movements at Signalised Intersections

SIDRA INTERSECTION offers two methods of modelling the effect of pedestrian movements on vehicle capacity at signalised intersections:

- The program can assign increased start loss to the vehicle movement that is subject to pedestrian interference (the preferred method), or
- Saturation flow rate can be reduced by the program using a factor calculated as a function of the conflicting pedestrian volume.

To utilise either of these methods, the pedestrian movement must be specified as an opposing movement in the Priorities dialog. The desired method for modelling the effect of pedestrians can then be selected in the Movement Data dialog.

2.5.11.3 Pedestrian Performance Measures in SIDRA INTERSECTION Output

Various output produced by *SIDRA INTERSECTION* give the results according to approach roads and pedestrians separately. Generally, *SIDRA INTERSECTION* text output and movement displays present performance measures (delay, stop rate, etc) for vehicles, pedestrians and all persons separately.

Various *total* values given in persons per hour combining the results for vehicles and pedestrians (e.g. total delay, total effective stops, etc.) are calculated using the Vehicle Occupancy (persons/vehicle) factor specified in the Volumes input dialog. The total vehicle and total pedestrian values are not added together directly, and the total values for the intersection (as experienced by both vehicles and pedestrians) are given in terms of total person values only. The average intersection values are also based on persons, i.e. determined by dividing the total intersection value (e.g. total delay in person-hours per hour) by the total intersection flow in persons/h.

Pedestrian volumes specified as input include pedestrians crossing the road in both directions (two-way volumes). In calculating the queue length, half the pedestrian volume is used as relevant to pedestrians waiting to cross in each direction (equal directional split is assumed). Pedestrian volume is not used in equations for calculating the average delay or average effective stop rate. This assumes that pedestrian queues will always clear. For calculating the total delay and total stops, flow rates based on original volumes are used so that the delays and stops experienced by pedestrians in both directions are accounted for.

When *Staged Crossings* are specified, two pedestrian movements are generated representing two stages of the crossing. The user has the option of specifying different volumes for each stage at a staged crossing. The associated average values of various performance statistics (delay, proportion queued, etc) do not represent performance of particular pedestrians along the route for the entire staged crossing. In such cases, summation of the statistics such as total delay, total cost, etc as statistics corresponding to points in the system is representative of the system statistics, and the total pedestrian volumes (repeated for two movements representing a staged crossing) match the associated total performance statistics. Similar issues arise in network modelling.

In *SIDRA INTERSECTION* output, practical spare capacity values and lane information are not given for pedestrian movements.

2.5.11.4 Minimum Green Times for Pedestrian Movements

For pedestrian movements at signalised intersections or signalised mid-block crossings, "*pedestrian minimum green time*" represents the minimum time required for both Walk and Flashing Don't Walk displays, but excluding any overlaps with terminating intergreen displays (see *Figure 2-14*). In the case of parallel pedestrian and vehicle movements at signalised intersections, the pedestrian minimum green time also represents the minimum green display required for parallel vehicle movements in order to satisfy the Walk and Flashing Don't Walk display requirements for the pedestrian movement.

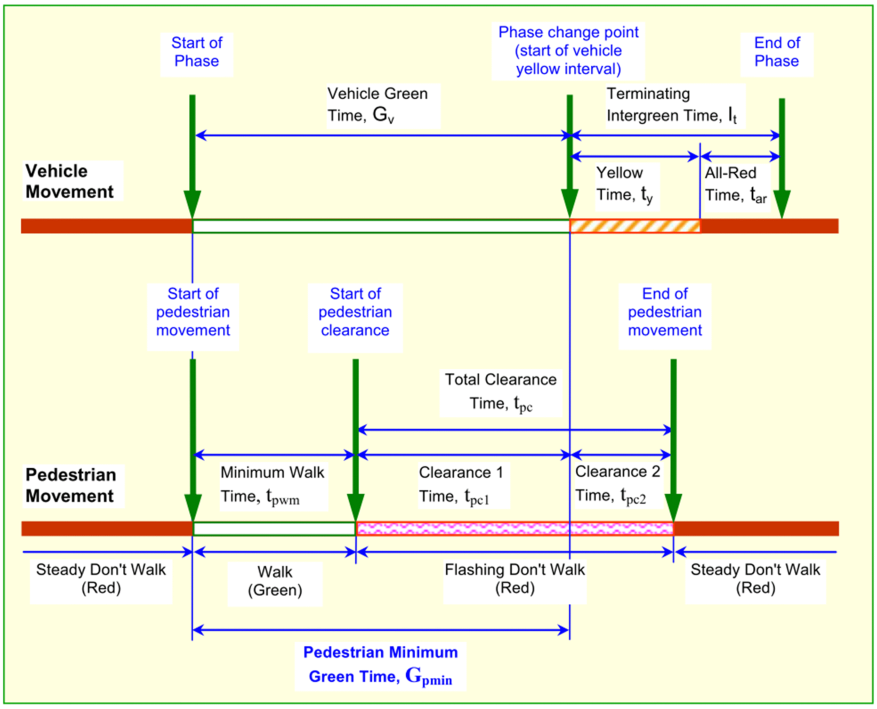

Figure 2-14: Walk and clearance times for pedestrian movements (SIDRA Akcelik & Associates 2010)

Minimum green times required for the purpose of pedestrian movements may be specified by the user in the Phasing & Timing - Pedestrian Movement Timing Data dialog, or calculated by the program using the crossing distance data calculated by the program or specified by the user in the Pedestrians input dialog.

The following method is used in *SIDRA INTERSECTION* for calculating pedestrian minimum green times and end gain values (see *Figure 2-14*). The parameters used in this method can be specified in the Phasing & Timing - Pedestrian Movement Timing Data dialog. Default values of these parameters are listed in *Table 2-2*.

Table 2-2: Default parameter values for calculating pedestrian timing data

Parameter	SIDRA INTERSECTION standard	HCM versions
Minimum Walk Time, t_{pwm}	5 s	7 s
Crossing speed, v_{pc}	1.2 m/s	4.0 ft/s (1.2 m/s)
Minimum Clearance Time, t_{pcm}	5 s	5 s
Clearance Time Overlap ("Clearance 2" time), t_{pc2}	2 s	3 s
Pedestrian Start Loss, t_{ps}	2 s	2 s
Pedestrian End Gain, t_{pe}	3 s	4 s

Firstly, the total pedestrian clearance time (duration of the Flashing Don't Walk interval), t_{pc} (in seconds) is calculated from:

$t_{pc} \approx \max (L_{pc} / v_{pc}, t_{pcm})$ ***(Equation 2-23)***

(approximated up to the nearest integer value)

where

t_{pcm} = minimum pedestrian clearance time (in seconds),
L_{pc} = pedestrian crossing distance (in metres), and
v_{pc} = pedestrian crossing speed for signal timing purposes (in metres per second).

Figure 2-15 shows the distribution of pedestrian crossing speeds at signalised intersections and midblock signalised crossings in Melbourne, Australia (Akcelik & Associates 2001, Bennett, Felton and Akçelik 2001). It is seen that the default crossing speed of 1.2 m/s used in *SIDRA INTERSECTION* (and widely used in traffic engineering practice) corresponds to 15th percentile speed (15% of pedestrians had crossing speeds slower than 1.2 m/s) for midblock crossings and about 4th percentile speed for crossings at signalised intersections (when all data were combined, 1.2 m/s represented 10th percentile speed). The average crossing speeds for Melbourne sites were 1.5 m/s for midblock crossings and 1.8 m/s for crossings at signalised intersections (1.6 m/s for all data combined).

The crossing distance is set to the appropriate pedestrian crossing distance value (D_c for full crossing, D_{ca} or D_{ce} for staged crossing) calculated by the program or specified by the user in the Pedestrians dialog.

As seen in *Figure 2-14*, the total clearance (Flashing Don't Walk) time consists of "Clearance 1" and "Clearance 2" times, $t_{pc} = t_{pc1} + t_{pc2}$. The "Clearance 1" interval is the first part of the Flashing Don't Walk interval, which occurs before the terminating intergreen time. The "Clearance 2" interval is the second part of the Flashing Don't Walk interval, which follows the "Clearance 1" interval and overlaps with part of the terminating intergreen time.

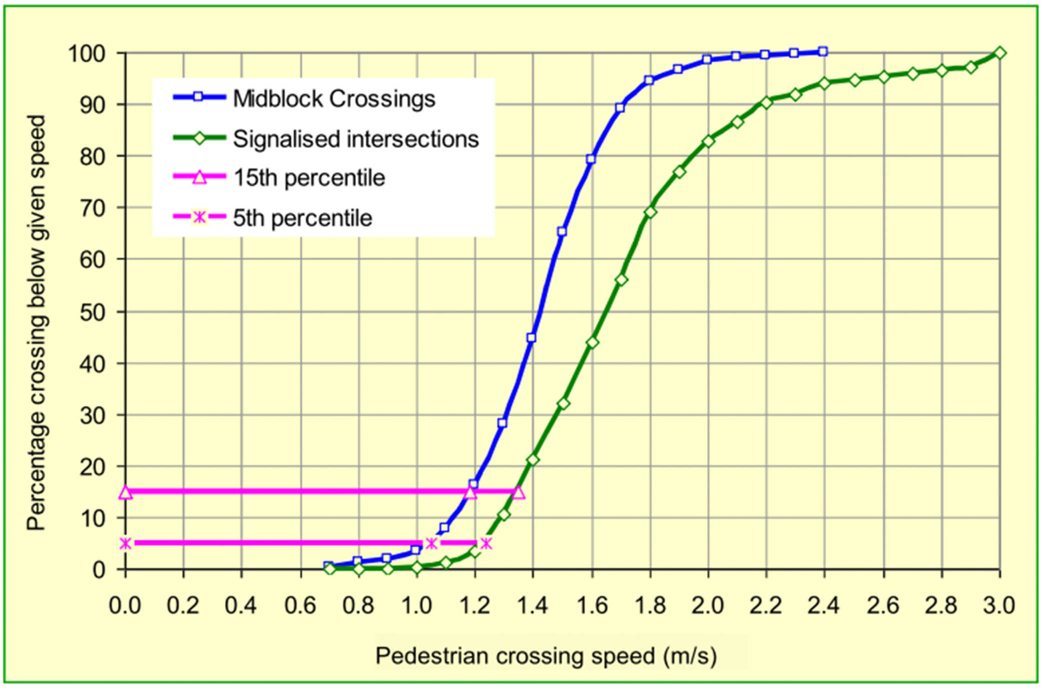

Figure 2-15: Pedestrian crossing speeds at signalised intersections and midblock crossings (SIDRA Akcelik & Associates 2010)

In the case of parallel pedestrian and vehicle movements at signalised intersections as seen in *Figure 2-14*, the "Clearance 1" interval overlaps with the green display for parallel vehicle movements. In this case, the Clearance 2 interval may overlap with part or all of the vehicle yellow interval, or with the vehicle yellow and part or all of all-red interval depending on the duration of t_{pc2}.

The "Clearance 1" time is determined from:

$t_{pc1} = t_{pc} - t_{pc2}$ ***(Equation 2-24)***

Where

t_{pc} = total clearance time from *(Equation 2-23)*, and

t_{pc2} = "Clearance 2" time from *Table 2-2*.

The pedestrian minimum green time, G_{pmin} (in seconds) is given by:

$G_{pmin} = t_{pwm} + t_{pc1}$ ***(Equation 2-25)***

Where

t_{pwm} = minimum Walk time from *Table 2-2*, and

t_{pc1} = "Clearance 1" time from *(Equation 2-24)*.

SIDRA INTERSECTION uses the following more complicated formula to avoid possible numerical error cases:

$G_{pmin} = \max [(t_{pwm} + t_{pc1}), G_{mt}, t_{ps}, (t_{pc1}-t_{pe})]$ ***(Equation 2-26)***

G_{mt} = a temporary value of the minimum displayed green time (vehicle default, or the minimum green which allows for a minimum effective green time of 3 seconds, $G_{mt} = t_{ps} - t_{pe} + t_{pc1} + 3$),

t_{ps} = start loss for pedestrians (unused part of the Walk time), and

t_{pe} = end gain for pedestrians (the initial part of the Clearance 1 time used by pedestrians), and

t_{pc1}= Clearance 1 time.

In *Figure 2-14*, the displayed pedestrian Walk time equals the minimum Walk time ($t_{pw} = t_{pwm}$), and the displayed vehicle green time equals the "pedestrian minimum green time" ($G_v = G_{pmin}$). If the pedestrian Walk display is extended in line with the parallel vehicle green display ($G_v > G_{pmin}$), the displayed pedestrian Walk time is larger than the minimum walk time:

$t_{pw} = G_v - t_{pc1} > t_{pwm}$ ***(Equation 2-27)***

All parameters in the pedestrian minimum green, negative end gain, lost time and effective green equations above are in seconds. Parameters t_{pwm}, t_{pc2}, t_{pg} and t_{ps} can be specified set in the Phasing & Timing - Pedestrian Movement Timing Data dialog (see *Table 2-2*).

If the user specifies the minimum green time for a pedestrian movement, the Clearance 1 time will be recalculated as follows:

$t_{pc1} = G_{pmin} – t_{pwm}$ ***(Equation 2-28)***

User-specified minimum green time must be sufficient to satisfy both (t_{pwm}+ t_{pcm}) and (t_{pwm}+ t_{pe}).

Example: Calculate the pedestrian minimum green time and end gain values for a crossing distance of L_{pc} = 15 metres using (i) the *SIDRA INTERSECTION* standard and (ii) HCM default values.

Using v_{pc} = 1.2 m/s and t_{pcm} = 5 s from *Table 2-2* (same for the *SIDRA INTERSECTION* standard and HCM default cases), the total clearance time required is t_{pc} = max (15/1.2, 5) ≈ 13 s.

(i) Using the SIDRA INTERSECTION standard default values:

From *Table 2-2*, the minimum Walk time is t_{pwm} = 5 s and the "Clearance 2" time is t_{pc2} = 2 s. Therefore, "Clearance 1" time is t_{pc1} = 13 - 2 = 11 s, and the pedestrian minimum green is G_{pmin} = 5 + 11 = 16 s.

(ii) Using the HCM default values:

From *Table 2-2*, the minimum Walk time is t_{pwm} = 7 s and the "Clearance 2" time is t_{pc2} = 3 s. Therefore, "Clearance 1" time is t_{pc1} = 13 - 3 = 10 s, and the pedestrian minimum green is G_{pmin} = 7 + 10 = 17 s.

2.5.11.5 Minimum Green Time Adjustment for Pedestrian Volume

For actuated signals, *SIDRA INTERSECTION* uses a method that accounts for the effect of occasional pedestrian calls. This method reduces the normal minimum green time required for pedestrians (G_{pmin}) according to the probability of no pedestrian demand during the average signal cycle. This may have a significant effect on the operation of the intersection when the pedestrian volumes are low, especially for those crossing a major road where the parallel vehicle movement has a low volume.

For this purpose, the probability of no pedestrian arrivals during the average signal cycle (p_{op}) is calculated from:

$$p_{op} = e^{-q_p (c - t_{pwm}) / 3600} \qquad \textbf{\textit{(Equation 2-29)}}$$

where q_p is the pedestrian flow rate (ped/h), c is the average cycle time (s), and t_{pwm} is the minimum duration of the pedestrian Walk signal display (*see Table 2-2*).

The adjusted minimum pedestrian green time is calculated as:

$G'_{pmin} = (1 - p_{op})\, G_{pmin}$ ***(Equation 2-30)***

Where G_{pmin} is the minimum pedestrian green time from *(Equation 2-26)* or a user-specified value, and $(1 - p_{op})$ is the probability of pedestrian arrivals during the average signal cycle.

(Equation 2-29) is based on the use of a simple negative exponential distribution of pedestrian arrival headways and the premise that pedestrian calls are not recorded during the Walk display (Akçelik 1995a). Probabilities of no pedestrian arrivals during the signal cycle as a function of the pedestrian flow rate and the average cycle time calculated from *(Equation 2-29)* for a minimum Walk time of t_{pwm} = 6 s are shown in *Figure 2-16*.

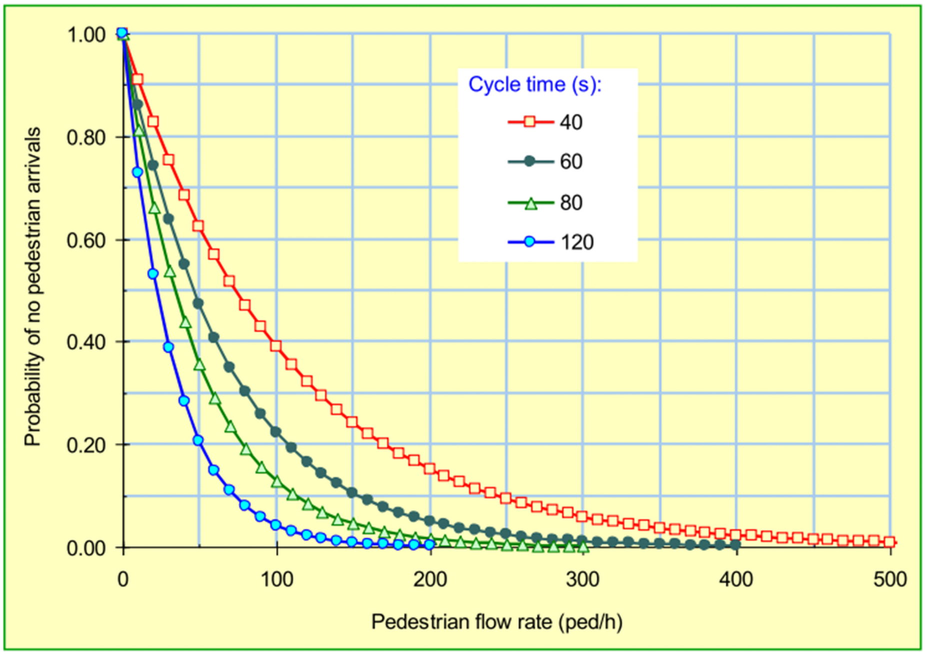

Figure 2-16: Probabilities of no pedestrian arrivals during the signal cycle (SIDRA Akcelik & Associates 2010)

2.5.11.6 Effective Green and Red Times for Pedestrian Movements

Estimating pedestrian performance characteristics (delay, queue length, etc.) and level of service, the general *SIDRA INTERSECTION* method of determining effective green and red times is applied to pedestrian movements (see *Figure 2-17*). The default values of pedestrian movement parameters used in this method are given in *Table 2-2*.

A pedestrian *start loss* parameter (t_{ps}) is used to represent the pedestrian reaction (and caution) at the start of the Walk interval. A pedestrian *end gain* parameter (t_{pe}) is used to represent the pedestrians who arrive after the end of the Walk interval and continue crossing during the first part of the (Flashing Don't Walk) interval.

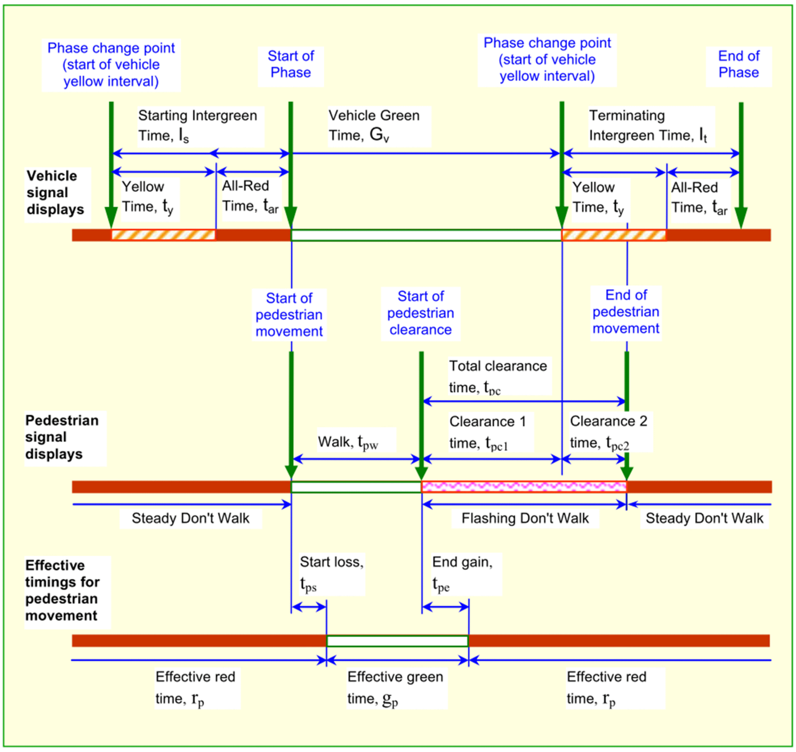

Figure 2-17: Effective green and red times for pedestrian movements (SIDRA Akcelik & Associates 2010)

The lost time for a pedestrian movement is calculated from:

$$l_p = I_s + t_{ps} + t_{pc1} - t_{pe} \qquad \textbf{\textit{(Equation 2-31)}}$$

Where I_s is the starting intergreen time, t_{ps} is the pedestrian start loss, t_{pc1} is the Clearance 1 time, and t_{pe} is the pedestrian end gain.

The effective green and red times (g_p, r_p) for a pedestrian movement is given by:

$$g_p = t_{pw} - t_{ps} + t_{pe} \qquad \textbf{\textit{(Equation 2-32)}}$$

$$r_p = c - g_p \qquad \textbf{\textit{(Equation 2-33)}}$$

Where t_{pw} is the pedestrian walk time, t_{ps} is the pedestrian start loss, t_{pe} is the pedestrian end gain, and c is the cycle time ($c = r_p + g_p$).

Note that the above relationships give $l_p + g_p = I_s + G_v$.

Example

If the total duration of the Flashing Don't Walk interval is t_{pc} = 15 seconds, starting intergreen time is I_s = 5 seconds and the displayed vehicle green time is G_v = 40 seconds, what are the values of effective green and lost time for the pedestrian movement using (i) the SIDRA INTERSECTION standard and (ii) HCM default values? Assume that the pedestrian Walk display is extended in line with the parallel vehicle green display.

(i) Using the SIDRA INTERSECTION standard default values:

From *Table 2-2*, the "Clearance 2" time t_{pc2} = 2 s, the start loss t_{ps} = 2 s and the end gain t_{pe} = 3 s. Therefore, "Clearance 1" time is t_{pc1} = 15 - 2 = 13 s, the Walk time is t_{pw} = 40 - 13 = 27 s. The lost time and effective green time for the pedestrian movement are $l_p = I_s + t_{ps} + t_{pc1} - t_{pe}$ = 5 + 2 + 13 - 3 = 17 s, and $g_p = t_{pw} - t_{ps} + t_{pe}$ = 27 - 2 + 3 = 28 s. Check: $l_p + g_p$ = 17 + 28 = 45 s (= $I_s + G_v$ = 5 + 40).

(ii) Using the HCM default values:

From *Table 2-2*, the "Clearance 2" time t_{pc2} = 3 s, the start loss t_{ps} = 2 s and the end gain t_{pe} = 4 s. Therefore, "Clearance 1" time is t_{pc1} = 15 - 3 = 12 s, the Walk time is t_{pw} = 40 - 12 = 28 s. The lost time and effective green time for the pedestrian movement are $l_p = I_s + t_{ps} + t_{pc1} - t_{pe}$ = 5 + 2 + 12 - 4 = 15 s, and $g_p = t_{pw} - t_{ps} + t_{pg}$ = 28 - 2 + 4 = 30 s. Check: $l_p + g_p$ = 15 + 30 = 45 s (= $I_s + G_v$ = 5 + 40).

3. DATA COLLECTION

Traffic turning movement data (Classified Count) and intersection geometric drawings were collected from the Planning Division and Highway Designs Division of Road Development Authority for the following junctions, in order to scrutinize the prevailing pattern and behaviour of traffic turning movements in Colombo district.

Most of these classified turning movements data were directly downloaded from data logger and converted into Microsoft-Excel using an interface software called ***KNOwin,*** for German Classified Counters, available at Planning Division-I (Traffic Data Collection-Subdivision of the *Planning Division of Road Development Authority*). Sample outputs from this software and geometric drawings are given in appendix as raw data in electronic version on the attached *Compact Disk (CD).*

3.1. Selected At Grade Intersections in Colombo District

Vehicle-turning movements of about thirty (30) intersections were examined and the following six (06) intersections were selected for evaluating permissible limits of different vehicle-turning movements for random traffic generation. All these intersections have twelve (12) possible vehicle-turning movements.

1. Orugodawatta Junction
2. Kirulapona Junction
3. Punchikawatta Junction
4. Thimbirigasyaya Junction
5. Narahenpita Junction
6. Grandpass Junction

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4. METHODOLOGY

4.1. Selection of Intersection Geometry

Several existing and designed geometric drawings were investigated and a typical geometry was selected for a four-legged intersection with denoted dimensions as shown in *Figure 4-2 & Figure 4-2*, for detailed analysis. Generalised names were assigned for all four legs in the directions of cardinal points as *Northern Leg* in North direction, *Eastern Leg* in East direction, *Southern Leg* in South direction and *Western Leg* in West direction.

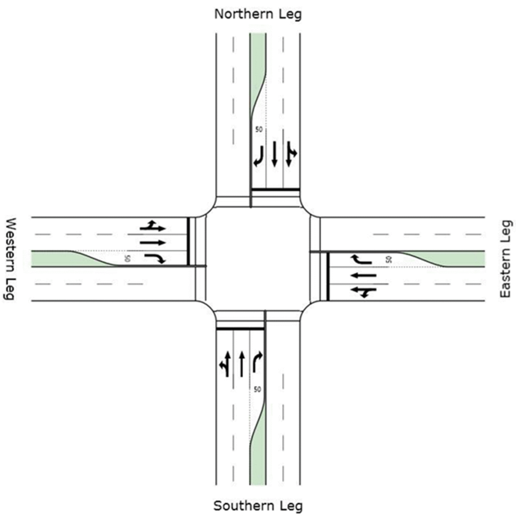

Figure 4-1: Typical four-legged intersection with right-turn bay (SIDRAAkcelik & Associates 2010)

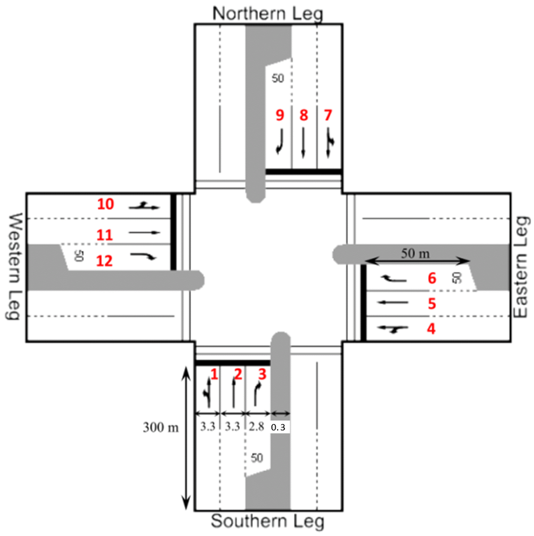

Figure 4-2: Typical four-legged intersection with selected dimensions for detailed analysis (SIDRA Akcelik & Associates 2010)

Three approach lanes and two exit lanes were allocated for each legs, based on the study of prevailing intersections. Most commonly used typical lane width of 3.3m was chosen for Left-turn (LT) and Through (TH) traffic. Right-turn (RT) pocket (Turn-bay) was selected with 2.8m wide and 50m long for RT as it is most commonly seen in Sri Lankan urban areas.

A length of 300m was decided on intersection's leg in cardinal points directions, to offer minimal effect from other signalised intersections in the immediate vicinity. Centre median for RT bay was chosen as 0.3m.

4.2. Selection of Signal Phase Arrangement

Four-phase signal arrangement was selected for the analysis, to avoid any conflicts among turning movements for a four-legged intersection.

There are two commonly used phasing arrangements in practice, for a four-phase signal. In order to analyse the performance of signalised intersections, operate under different phasing arrangements, these two different phasing arrangements were considered for the analysis and named as Type-A and Type-B as shown in *Figure 4-3 and Figure 4-4* respectively. Allowed turning movements from each approach legs in a particular phase are represented by lines with arrowheads, while banned movements are marked with stop-line at the head of each turning movements. Pedestrians' movements are also denoted in the same manner but they are across each approach leg with double arrowhead.

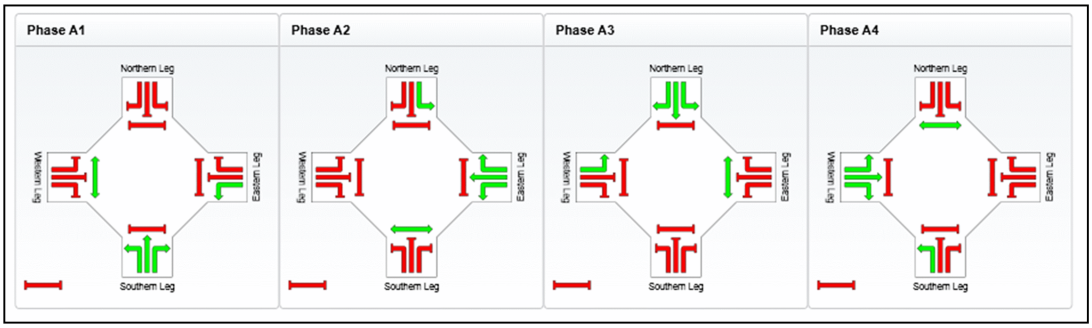

Figure 4-3: Signal Phasing arrangement Type-A for detailed analysis (SIDRA Akcelik & Associates 2010)

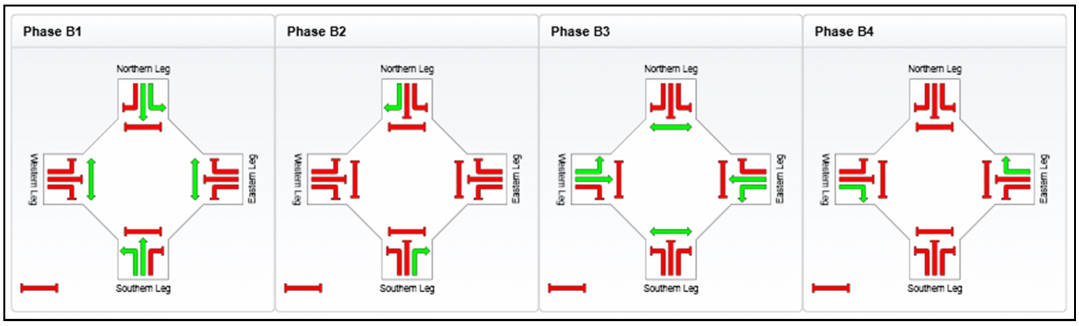

Figure 4-4: Signal Phasing arrangement Type-B for detailed analysis (SIDRA Akcelik & Associates 2010)

4.3. Generation of Random Traffic

Out of about thirty (30) intersections' turning movement data, around ten intersections were selected with four legs and different number of approach and exit lanes. These ten intersections were carefully examined and intersections, which did not have all twelve (12) vehicle-turning movements, were eliminated. As shown in *Figure 4-5,* intersections that have four-legs with all twelve turning movements, were only chosen for the detailed study.

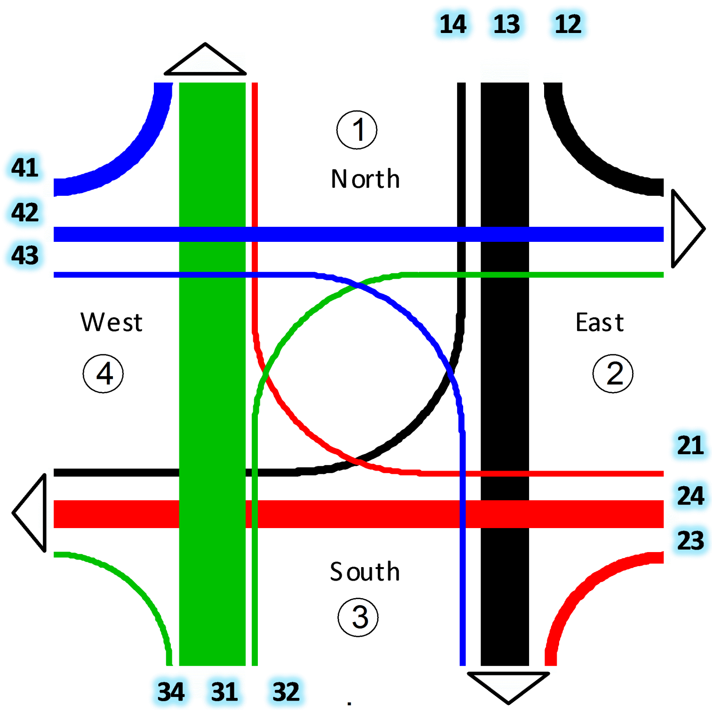

Figure 4-5: Four-legged intersection with different traffic demand-flow lines (KNOwin 1995)

Categorisation of Light-Vehicles (LV) and Heavy-Vehicles (HV) was accepted in a traditional way as defined in the traffic counter/logger interface software as (MCL, TWL, CAR, VAN, MBU, LBU, LGV, MG1 & MG2) are LV and (HG3, AG3, AG4, AG5 & AG6) are HV (Refer *Table 10-1*).

Where,

(1). MCL – Motor Cycles
(2). TWL – Three Wheelers
(3). CAR - Cars
(4). VAN - Vans

(5). MBU – Medium Buses
(6). LBU – Large Buses
(7). LGV – Light Goods vehicles
(8). MG1 – Medium Goods vehicle 1
(9). MG2 - Medium Goods vehicle 2
(10). HG3 – Heavy Goods vehicles 3
(11). AG3 – Articulated Goods vehicles 3
(12). AG4 – Articulated Goods vehicles 4
(13). AG5 – Articulated Goods vehicles 5
(14). AG6 – Articulated Goods vehicles 6

Categorised Light-Vehicles (LV) and Heavy-Vehicles (HV) for selected junctions were extracted onto separate spreadsheets, using a recorded Macro-programme for all twelve turning movements.

Then, turning movements on Left (LT), Through (TH) and Right (RT) directions were separated in a comprehensive table for all four directions (North, East, South & West), as shown in *Annex Table 10-2*. In addition, maximum and minimum values of turning movements for every individual turns were found using *MAX()* and *MIN()* formulae in Microsoft Excel.

All these maximum and minimum turning movement values were extracted from selected sites individually and another table was created to compare the individual turning movement values among all selected intersections as shown in *Annex Table 10-3*.

Generalised maximum and minimum values for LT, TH and RT for all four legs were determined and rounded off. These generalised and approximated traffic turning-movement values, as given in *Table 4-1, Table 4-2* and *Table 4-3* were used to generate random traffic (Light and Heavy vehicles whilst satisfying all conditions found) at a typical Sri Lankan intersection in urban area.

Table 4-1: Conditions for turning movements to generate random traffic

Condition-1 to generate Random Traffic					
Turning Movement Numbers	Min	Max	Rounded up to 100	For Random Generation	
1+7	0	240	300	300	N-S LT
2+8	6	1846	1900	1900	N-S TH
3+9	4	181	200	200	N-S RT
4+10	6	215	300	300	E-W LT
5+11	2	801	900	900	E-W TH
6+12	1	305	400	400	E-W RT
1+4+7+10	25	427	500	500	Total LT
2+5+8+11	64	1986	2000	2000	Total TH
3+6+9+12	5	419	500	500	Total RT

Table 4-2: Conditions for LV & HV maximum and minimum values and ratios

Condition-2 to generate Random Traffic				
Turning Movement	LV	HV	Practical Ratio w.r.t TH	Approx.Ratios rounded up to nearest 5
LT Min	0	0	0.00	0
LT Max	**228**	**19**	**21.83**	**25**
TH Min	0	0	0.02	0
TH Max	**1090**	**12**	**27.83**	**30**
RT Min	0	0	0.00	0
RT Max	**272**	**15**	**25.60**	**30**

Table 4-3: Conditions used in Microsoft Excel to generate random HV

Condition-3 for Heavy Vehicles - HV						
	N-S			E-W		
LT	20	20-LT	<=20	12	12-LT	<=12
TH	9	11	<=20	12	15-TH	<=15
RT	8	8-RT	<=8	15	15-RT	<=15
Applied in RANDBETWEEN() Formula in MsExcel						

In order to get possible traffic turning movement combinations; a set of step values were adopted, i.e 1, 10, 50, 100, 500, 1000, 1100 for Northern TH traffic and respective Southern through traffic were calculated based on step ratios (minimum to maximum), which were already computed from real turning movement data, i.e 0.02, 0.10, 1.00, 5.00, 10.00, 15.00, 20.00, 25.00, 28.00, 30.00. The same procedure was followed for E-W through traffic as shown in *Table 4-4* and *Table 4-5*.

Table 4-5: Selection of Through-traffic combinations for N-S direction

	North	1	10	50	100	500	1,000	1,100	Volumes		
	Ratios	0.02	0.10	1.00	5.00	10.00	15.00	20.00	25.00	28.00	30
1	South	1	1	1	5	10	15	20	25	28	30
10	South	1	1	10	50	100	150	200	250	280	300
50	South	1	5	50	250	500	750	1,000	1,250	1,400	1,500
100	South	2	10	100	500	1,000	1,500	2,000	2,500	2,800	3,000
500	South	10	50	500	2,500	5,000	7,500	10,000	12,500	14,000	15,000
1,000	South	20	100	1,000	5,000	10,000	15,000	20,000	25,000	28,000	30,000
1,100	South	22	110	1,100	5,500	11,000	16,500	22,000	27,500	30,800	33,000

*Used ROUNDUP() formula for rounding off & only shaded values are selected for TH- traffic assignment

Table 4-4: Selection of Through-traffic combinations for E-W direction

	East	1	10	50	100	300	500	800	Volumes		
	Ratios	0.05	0.10	1.00	5.00	10.00	15.00	20.00	25.00	28.00	30
1	West	1	1	1	5	10	15	20	25	28	30
10	West	1	1	10	50	100	150	200	250	280	300
50	West	3	5	50	250	500	750	1,000	1,250	1,400	1,500
100	West	5	10	100	500	1,000	1,500	2,000	2,500	2,800	3,000
300	West	15	30	300	1,500	3,000	4,500	6,000	7,500	8,400	9,000
500	West	25	50	500	2,500	5,000	7,500	10,000	12,500	14,000	15,000
800	West	40	80	800	4,000	8,000	12,000	16,000	20,000	22,400	24,000

*Used ROUNDUP() formula for rounding off & only shaded values are selected for TH- traffic assignment

Only realistic combinations of N-S and E-W were selected (as highlighted on tables) in order to make another set of combinations amount N-S and E-W directions.

Out of all possible combinations between N-S and E-W, a set of 110 ultimate combinations were elected and other turning movements for LT, RT and HV were randomised whilst satisfying the conditions obtained.

Formulated spread sheet was used to generate generalised random traffic by pressing F9-function key from computer keyboard and a set of trial turning movements was selected and copied onto another spread-sheet only with values (without formulae) to avoid further generation of random traffic in the trial set.

4.4. Preparation for Data entering and analysis

Traffic turning movements were arranged such a way to facilitate entering traffic data in the analytical software *SIDRA* to avoid any chances of typing error. Actually, all turning movements from different signalised intersections selected for detailed analysis were rearranged to match the turning movement of SIDRA from 1 to 12 as shown in *Figure 4-6 Figure 4-7*.

<u>South</u>		**<u>North</u>**	
34 → 1	(SIDRA) - LT	12 → 7	(SIDRA) - LT
31 → 2	(SIDRA) - TH	13 → 8	(SIDRA) - TH
32 → 3	(SIDRA) - RT	14 → 9	(SIDRA) - RT
P1 →	(SIDRA)	P5 →	(SIDRA)
<u>East</u>		**<u>West</u>**	
21 → 4	(SIDRA) - LT	41 → 10	(SIDRA) - LT
24 → 5	(SIDRA) - TH	42 → 11	(SIDRA) - TH
23 → 6	(SIDRA) - RT	43 → 12	(SIDRA) - RT
P3 →	(SIDRA)	P7 →	(SIDRA)

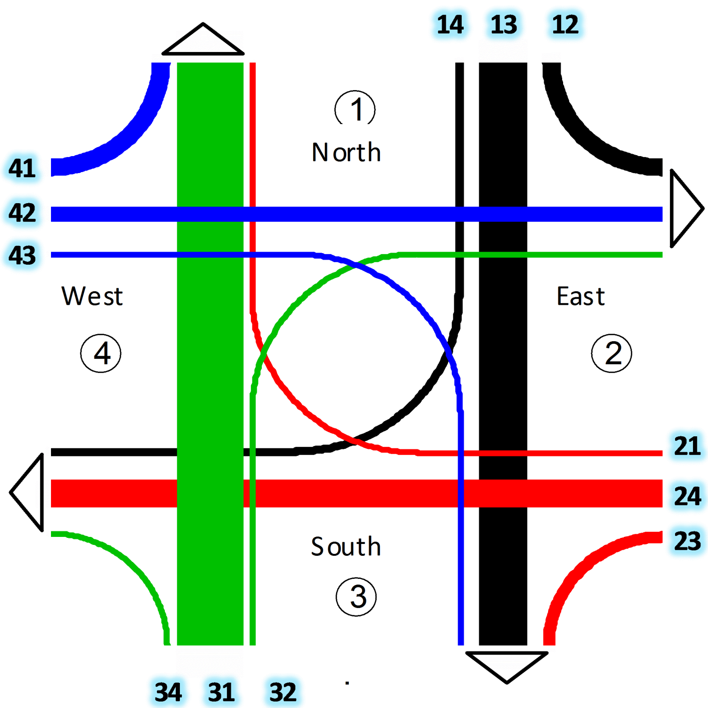

Figure 4-6: Four-legged intersection with traffic-turning movements' numbers (KNOwin 1995)

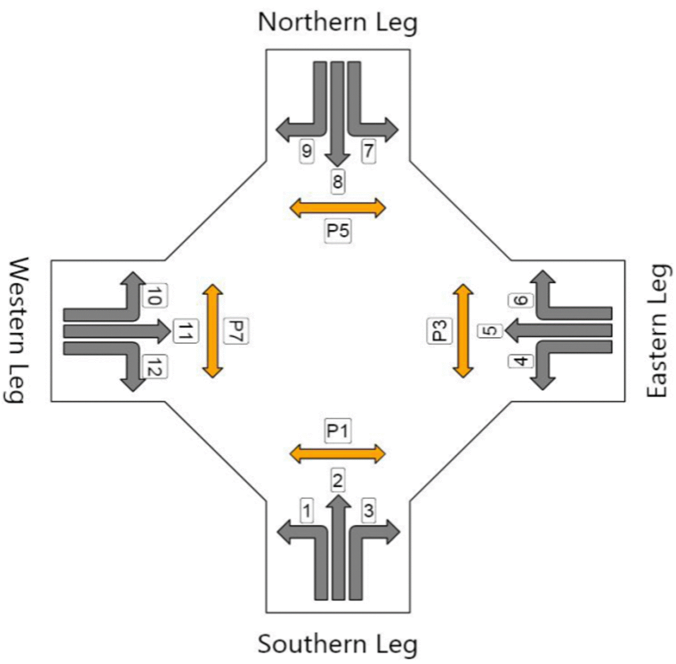

Figure 4-7: SIDRA typical four-legged intersection with turning movements' numbers (SIDRA Akcelik & Associates 2010)

Matching collected turning movement numbers with SIDRA turning movement numbering

SIDRA software was used to calculate all necessary information for each traffic signal site (110 different combinations from 00001 to 00110). Then, a comprehensive table was developed by extending the trial spreadsheet by individually entering selected information such as Cycle time (s), Intersection Control Delay, Pedestrian Delay and Critical Movements against every 110 trial signal sites.

Various graphs were plotted for Cycle time, Through traffic ratios in N-S and E-W direction, Intersection Control Delay [(Veh-h/h) Overall delay with geometric delay], Pedestrian delay (Ped-h/h) against total intersection traffic and through traffic demand, to evaluate most efficient system.

4.5. Other Important Parameters

In SIDRA software interface, it has eleven sub-interfaces for input section, they are; Intersection, Geometry, Volumes, Path Data, Movement Date, Priorities, Gap Acceptance, Pedestrians, phasing & Timing, Model Settings and Demand & Sensitivity. Several parameters and their entered values are given below.

- **Signal Analysis Method**
    - Either Fixed-time/pretimed or Actuated
- **Basic signal parameters**
    - Unit time for volumes = 15 min
    - Peak flow period = 15 min
    - Basic Saturation Flow = 1,900 pcuph
    - Target Level of Service (LOS) = C (minimum)
    - Extra bunching = 0%
    - Peak Flow Factor (PFF) = 95%
    - Vehicle occupancy = 2 pers/veh
    - Flow scale = 100%
    - Growth rate = 3.5%/year
    - Approach cruise speed = 40 km/h
    - Exit cruise speed = 50 km/h
    - Approach travel distance = 300m
    - Negotiation radius for Left-Turns = 10 m
    - Queue space for Light Vehicles (LV) = 5.0 m
    - Queue space for Heavy Vehicles (HV) = 10.0 m

- Vehicle length for Light Vehicles (LV) = 4.5 m
- Vehicle length for Heavy Vehicles (HV) = 8.0 m
- HVE (gap acceptance) = 2.0
- Arrival Type – 3 Isolated (not coordinated)
- Pedestrian effects, Extra start loss = 5 s
- Critical gap for RT = 4.5s
- Follow-up headway for RT = 2.5s
- End departures = 2.2veh
- Existing flow effect = 0%

- **Pedestrian data**
  - Volume per 15min = 225 ped
  - Peak Flow Factor = 95%
  - Flow Scale (constant) = 100%
  - Approach Travel distance = 10 m
  - Downstream distance = 10 m
  - Walking speed = 1.3 m/s
  - Queue space = 1.0 m
  - Saturation flow = 1500 ped/h
  - All pedestrians are assumed to cross the road fully not partially.
  - Crossing speed = 1.25 m/s
  - Minimum walk time = 5s
  - Minimum Clearance time = 5s
  - Clearance Time Overlap = 2s
  - Start Loss = 2s
  - End Gain = 3s
- **Cycle Time Options**
  - Optimum Cycle Time was chosen
  - Lower value = 10s
  - Upper value = 180s
  - Increment = 5s
- **Actuated Signal Data**
  - Maximum Green Time for Major Movement = 50s
  - Maximum Green Time for Minor Movement = 20s

- Gap Setting for Major Movement = 2.5s
- Gap Setting for Minor Movement = 2.0s
- Effective Detection Zone Length = 4.5 m

- **Model Settings**
  - Level of Service Method = 'Delay (HCM) & Degree of Saturation'
  - Level of Service Target = LOS C
  - Performance Measure = Delay
  - Percentile Queue = 95%
  - Hours per Year = 480h
- **Gap Acceptance**
  - HV Method for Gap-Acceptance = 'Include HV Effect for all percentages'
  - Gap-Acceptance Capacity = 'SIDRA Standard (Akcelik M3D)'
- **Vehicle Operating Cost**
  - Cost Unit = Rs.
  - Pump Price of Fuel = 73.50 Cost Unit/L
  - Fuel Resource Cost Factor = 0.50
  - Ratio of Running Cost to Fuel Cost = 3.0
- **Vehicle Mass**
  - Light Vehicel Mass = 1,400.0 kg
  - Heavy Vehicle Mass = 11,000.0 kg
- **Time Cost**
  - Average Income = 32.00 Cost Unit/h
  - Time Value Factor = 0.60
- **Demand & Sensitivity**
  - Sensitivity
    - Maximum Green – Lower (50.0%), Upper (120.0%) & Increment (5.0%)

.

5. ANALYSIS AND RESULTS

Graph-1: Fully Actuated Cycle time variation with Total Through-Traffic

- *Horizontal line with circled-tail and arrowhead* represents the practical Cycle time of 180s (The represented value is 1800, which is actually raised by 10 folds of actual value, for better illustration). In conjunction with this line, Cycle times of actuated Type-B signals are almost within the practical cycle time limit of Sri Lanka (180s) compare with actuated Type-A in order to achieve the level of service LOS 'C' or better. (Target level of service)
- Type-A actuated signals' results show unacceptable cycle times (exceptionally) due to either short-lane effect of significant difference between N-S and E-W through traffic.
- Cycle time of Type-B phasing arrangement lies between 150-180s range and which is independent from the ratio of N-S and E-W through traffic (whether they are equal or not).
- *Horizontal line with diamond tail and arrow-head,* represents that the N-S Through-Traffic is equals to E-W Through-Traffic (The represented value is 1000, which is actually one (1) raised by 1000 folds for better illustration). It shows that the cycle times are irrespective of different Through-Traffic combinations.

1

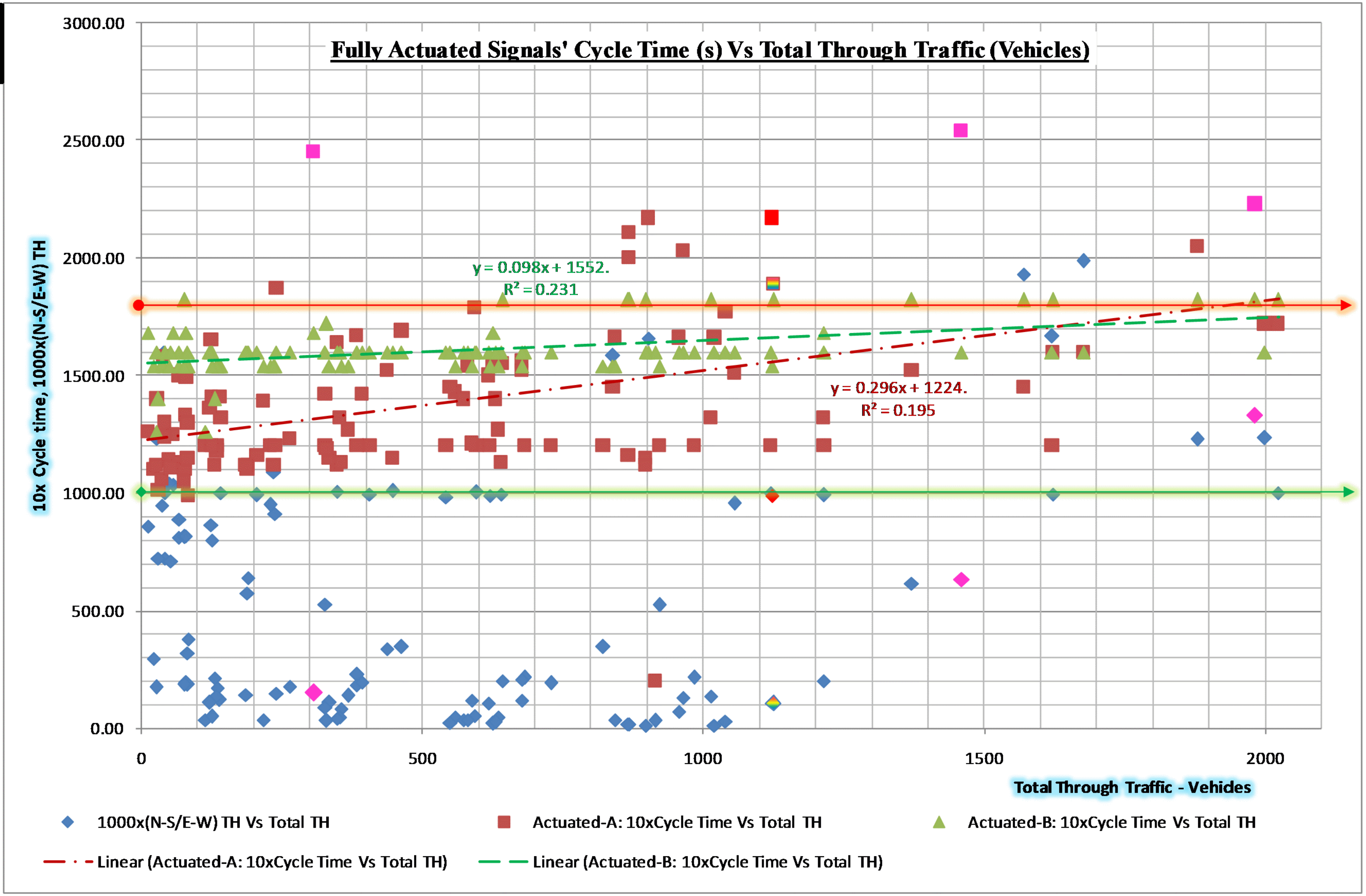

Figure 5-1: Fully actuated signals' Cycle time vs. Total through-traffic

Graph-2: Intersection Control Delay of Fully Actuated Signals

- The trend of Control delay for fully actuated Type-A and Type-B show that, up to a total through demand of 750veh Type-A would result lesser delay compare with Type-B, but beyond this point Type-B is better than Type-A.
- For total through traffic demand less than 500veh fully actuated Type-A signal absolutely will function efficiently within the a range of 100-140s cycle-time to achieve a level of service ‘C’ or better, where control delay would be lesser than 800Veh-h/h.

2

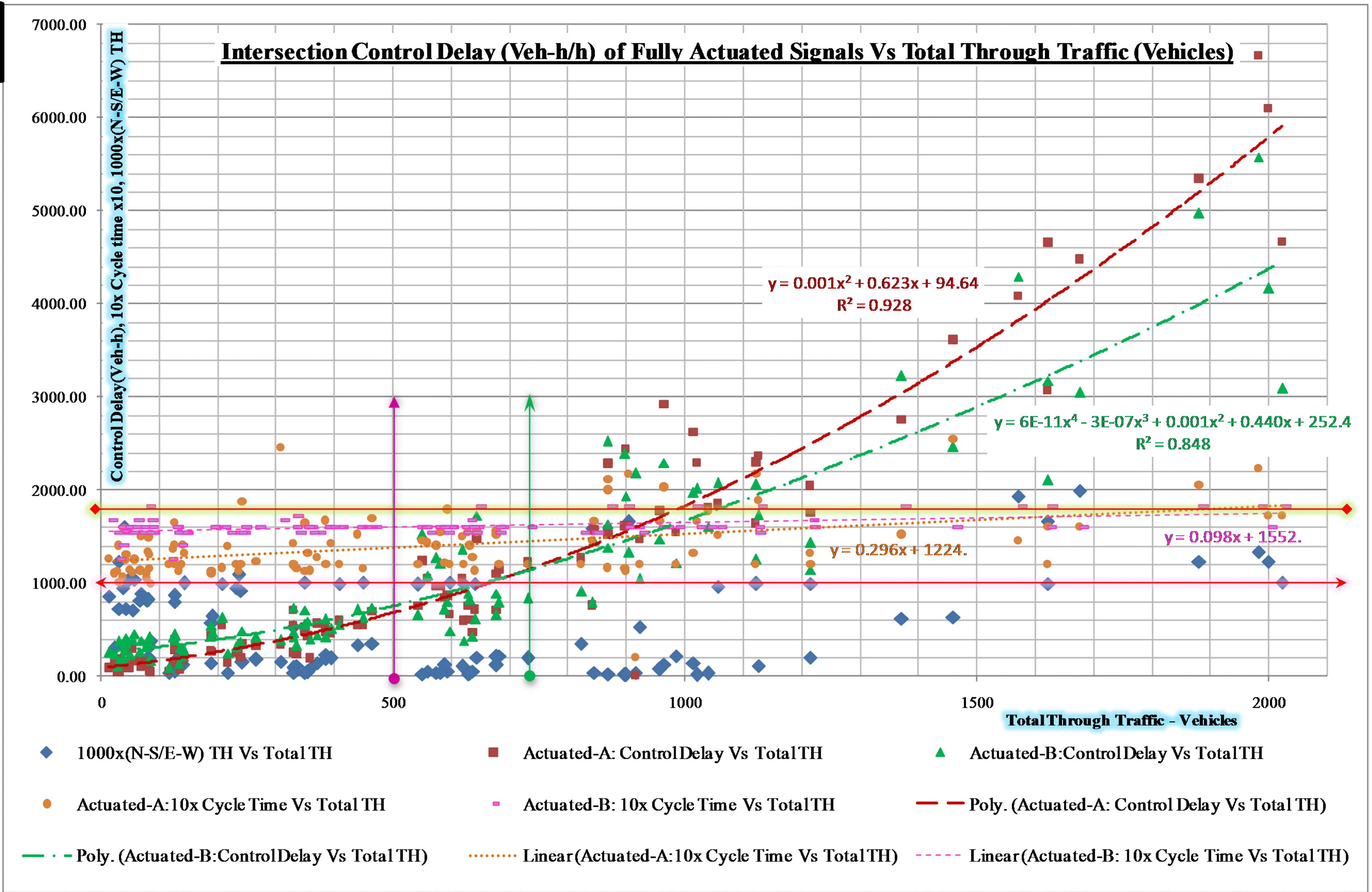

Figure 5-2: Intersection Control Delay of fully actuated signals vs. Total through-traffic

Graph-3: Intersection Control Delay of Fully Actuated Signals and Fixed-time Signals

- Fixed-time Type-B signals work efficiently (low delay) at maximum practical time of 180s when compare with other all types of signals considered for the analysis.(Actuated Type-A &B and Fixed-time Type-A)
- When comparing Fixed-time Type-A and Actuated Type-B there is no significant reduction in delay up to a total through traffic demand of 1250veh approximately. Beyond this demand Fixed-time Type-A would function better than Actuated Type-B.
- When total through traffic demand is very low in a range of 200-300veh Actuated Type-A signal would produce better result compare with other all three types. But not significant less than 500veh

3

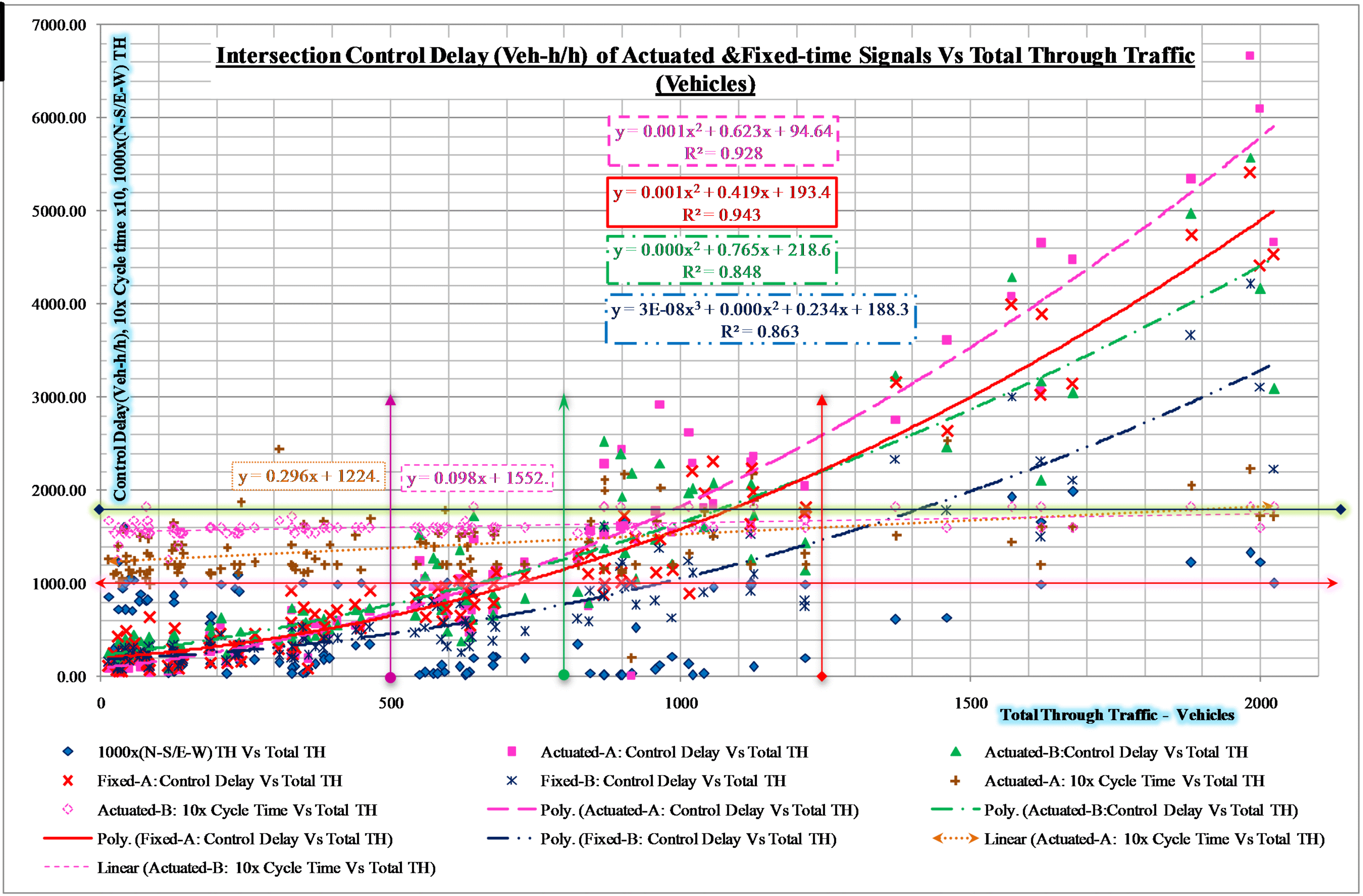

Figure 5-3: Intersection Control delay of actuated & fixed-time signals vs. Total through-traffic

Graph-4: Pedestrian's Delay at different types of Signalised Intersections

- Beyond 750veh of total through demand, Fixed-time Type-A works better than Actuated Type-A for pedestrians.

4

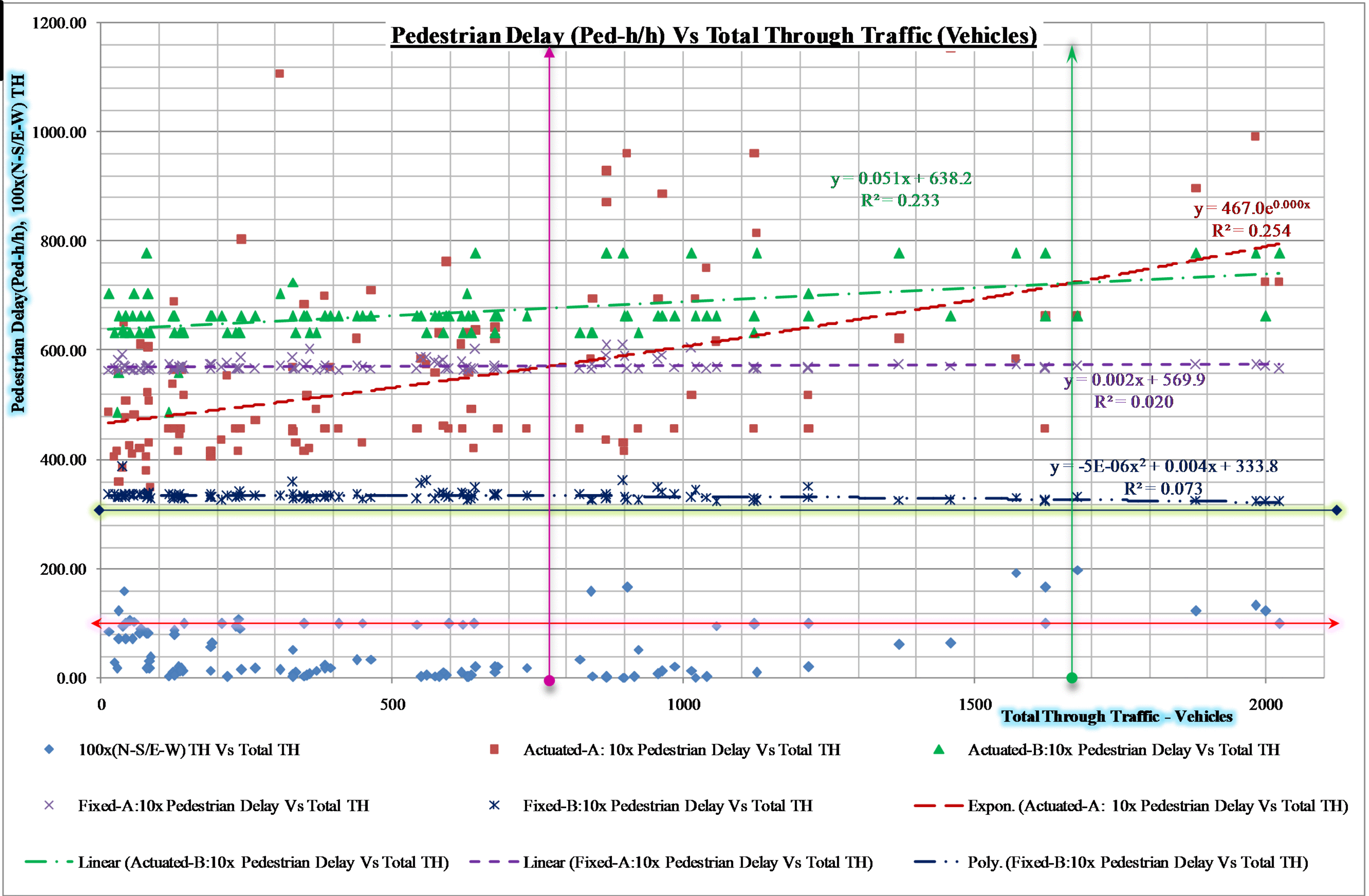

Figure 5-4: Pedestrian delay vs. Total through-traffic

Graph-5: Variation of Vehicle Control Delay with Total Intersection demand

- Fixed-time Type-B is the best, out of all other types when operate at maximum practical cycle-time of 180s for a target level of service 'C'.

5

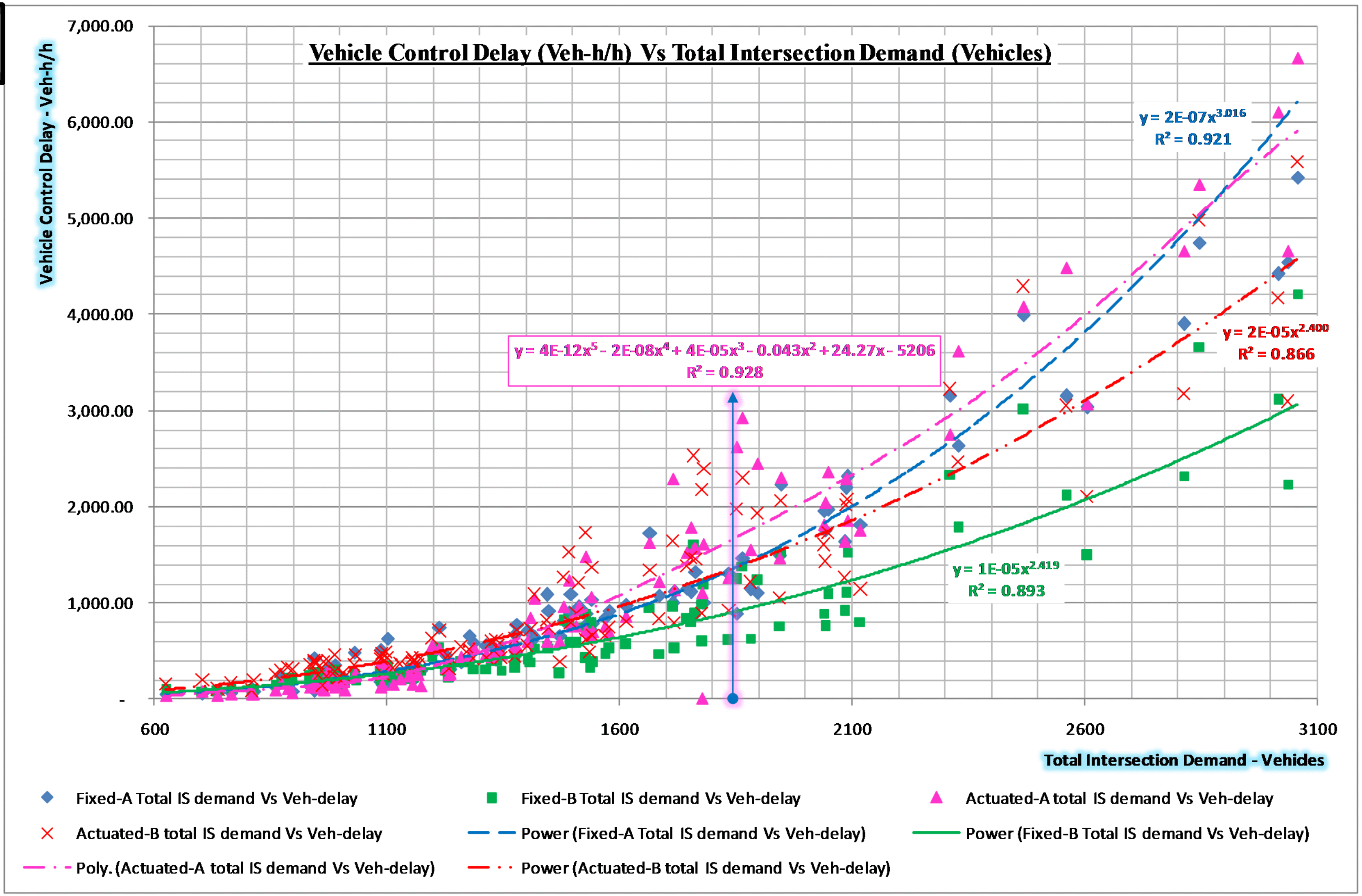

Figure 5-5: Vehicle Control delay vs. Total intersection demand

6. CONCLUSIONS

This research has presented a comparison study of fully actuated-signals against the traditional fixed-time traffic signals for four-legged levelled junctions having three approach lanes including right-turn bay and two exit lanes, based on the programme written in *SIDRA* software (Akcelik & Associates Pty Ltd 2010), recently procured by the *Road Development Authority of Sri Lanka*. In the absence of real-time traffic data, availability of limited vehicle turning movement data were used to simulate variability of vehicle arrivals in the dynamic and stochastic behaviour of traffic at Sri Lankan at grade intersections. It is believed that the presented results shall be utilised for various contributions to design and planning of vehicle-actuated signals for the future needs of Sri Lanka. Moreover, this research could be continued for further development connected to this subject.

- Replacement of fixed-time(static) traffic signals with fully actuated-vehicle signals shall not produce any significant improvements (reduction in delay) to stand-alone at grade four-legged intersections, which have three approach lanes including right turn-bay with optimum length and two exit lanes, for urban areas.
- Pretimed signals with Phasing arrangement Type-'B' is the best option for four-legged levelled intersections having three approach lanes (including separate right-turn), when they operate at permitted practical cycle time (180s).
- Four-legged intersections, which are presently functioning with the Phasing arrangement Type-'A' (absence of Right-Turn pocket or exclusive Right-lane); opening of Right-turn pocket/bay with optimum length by road-widening and switch to pretimed signal Phasing Type-'B' would produce a long run benefits.
- Level of Service is 'C' or better is one of the targets.
- Replacement of Fixed-time signals with fully vehicle actuated signals shall not produce any significant improvements for Sri Lankan at grade intersections in urban area like Colombo.
- Semi-actuated signals would be a better option for signalised intersections. Further, remain with fixed-time signals for major roads (continuous high demand) and replace minor roads (highly fluctuating traffic demand) signals with vehicle actuated is the best solution.

- Phasing arrangement Type-B is the best for intersections which have separate right-turn bays/pockets or exclusive right-turn lanes.
- Intersections, which presently in operation of Type-A would be transformed to Type-B by introducing right-turn bays with *optimal length* for highly demanded direction for long-term benefits.
- These results tally with *Ben-Edigbe and Ibrahim (2010)* in their recent research paper found that '*optimised static signals (fixed-time) can produce good results and should also be considered especially at standalone intersections when traffic operations are at peak regularly'*.

7. RECOMMENDATIONS

Most jurisdictions installing traffic signals is the increased capital costs associated with any new equipment. Introduction of vehicle-actuated signals to Sri Lanka may lead to added costs of replacement of existing pretimed signal controllers, installation of new actuated signal controllers, placing appropriate detectors, regular monitoring and maintenance of the whole traffic signal systems.

In most situations, the traffic signal system is integrated with the street lighting system along the sides of the road. This provides an existing conduit system, which can easily be used for the detector lead-in cables at no extra cost. In situations where it is required to have additional auxiliary detectors, e.g at the stop line a total of 12 extra detectors would be required for an intersection with two-straight through lanes and a separate left turn lane. *At an average cost of $500 per inductive loop detector the total increase in cost to the signal installation would be $6000. This is minor in comparison to the approximately $100,000 that a complete installation costs at today's prices(Shaflik, 1995).* In fact, most installations require a set of two-detectors per lane. All that is required to have a stop line detector and a second detector located farther back (advanced detector) to increase the efficiency and safety of the actuated signal.

- Proper placement of vehicle detectors also significantly increase the performance of a traffic signal installation not only by increasing traffic flow, but also by reducing individual and total delay and by increasing safety to the travelling public.
- Since, the Government of Sri Lankan (GOSL) efforts to allocate funds to maintain the existing infrastructure and facilities of National Road Network; the decision has to be taken at government level to introduce new traffic management systems as vehicle-actuated traffic signals.
- Semi-actuated traffic signals could be incorporated to intersections, which are connected with minor roads where traffic demand is very stochastic or very low. In addition, semi-actuated signals shall be introduced to access roads and ramps, which are connected with major developing highways or expressways in Sri Lanka, to offer continuous traffic flow along these major routes.

- Coordinated signals could readily be adapted to certain adjoining intersections before introducing vehicle-actuated signals.

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10. APPENDICES

10.1. Traffic and Signal analysis tables

Table 10-1: Sample traffic turning movements with seperated HV and LV during a weekday

Narahenpita Junction	N-S Direction						E-W Direction						N-S Direction						E-W Direction						Sum		Total
	LT		TH		RT		RT		LT		TH		TH		RT		LT		LT		TH		RT				
Wednesday	12		13		14		21		23		24		31		32		34		41		42		43				
24/06/2009	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	
6:00- 6:15	38	0	107	1	38	0	80	0	22	0	83	0	126	1	9	0	25	0	0	0	19	0	18	0	565	2	676
6:15- 6:30	37	0	100	2	43	0	121	1	29	0	103	0	168	1	10	0	19	0	11	0	17	0	14	0	672	4	952
6:30- 6:45	48	0	167	4	47	0	156	0	33	0	135	0	235	1	10	0	36	0	35	0	32	0	13	0	947	5	1,209
6:45- 7:00	64	0	209	0	65	0	191	0	32	0	162	0	331	0	13	0	42	0	43	0	37	0	20	0	1,209	0	1,473
7:00- 7:15	90	0	227	2	82	0	211	0	25	3	151	0	460	1	39	0	49	0	57	0	61	0	15	0	1,467	6	1,623
7:15- 7:30	107	0	308	0	84	0	233	0	48	1	227	0	382	2	40	0	34	0	80	0	60	0	17	0	1,620	3	1,570
7:30- 7:45	108	0	274	0	69	0	205	0	55	0	178	0	400	1	45	0	53	0	64	0	97	0	21	0	1,569	1	1,656
7:45- 8:00	121	0	258	0	62	0	227	0	46	1	244	0	409	1	46	0	38	0	90	0	90	0	23	0	1,654	2	1,729
8:00- 8:15	157	0	330	0	121	0	209	0	66	0	169	0	406	0	42	0	61	0	80	0	54	0	34	0	1,729	0	1,749
8:15- 8:30	109	0	252	0	73	0	272	1	65	0	175	0	532	1	41	0	59	0	88	0	48	0	33	0	1,747	2	1,805
8:30- 8:45	156	0	330	3	107	0	185	0	40	0	145	0	564	0	46	0	53	0	84	0	60	1	31	0	1,801	4	1,580
8:45- 9:00	111	0	260	4	81	0	184	0	53	0	163	0	458	2	39	0	69	0	66	0	33	0	56	1	1,573	7	1,823
9:00- 9:15	126	1	358	3	94	0	148	1	50	0	125	0	623	2	49	0	64	0	75	0	60	0	44	0	1,816	7	1,379
9:15- 9:30	121	0	265	5	87	0	151	1	42	0	108	0	319	3	49	0	66	0	72	0	54	0	36	0	1,370	9	1,523
9:30- 9:45	116	0	281	1	80	0	149	2	54	0	221	0	344	4	43	0	51	0	66	0	83	0	28	0	1,516	7	1,603
9:45-10:00	135	0	291	6	64	0	158	2	74	0	177	0	401	4	49	0	53	0	81	0	67	0	41	0	1,591	12	1,462
10:00-10:15	117	1	228	6	87	0	163	3	68	0	135	0	328	2	51	0	69	0	105	0	69	0	30	0	1,450	12	1,441
10:15-10:30	114	1	260	3	105	0	130	3	63	0	56	0	352	4	50	0	52	0	96	0	87	2	62	1	1,427	14	1,423
10:30-10:45	157	0	209	1	83	0	176	1	60	0	83	0	295	1	50	0	47	0	78	0	84	0	98	0	1,420	3	1,358
10:45-11:00	142	0	212	3	97	0	112	1	47	0	118	0	247	4	45	0	61	0	84	0	127	3	55	0	1,347	11	1,317
11:00-11:15	127	0	205	0	119	2	118	1	30	0	85	0	294	4	49	0	60	0	81	0	90	1	51	0	1,309	8	1,472
11:15-11:30	139	0	311	7	86	1	135	1	61	0	102	0	308	1	37	0	50	0	88	0	101	0	44	0	1,462	10	1,418
11:30-11:45	112	0	295	1	86	0	123	1	35	0	74	0	378	3	50	0	55	0	88	0	92	0	25	0	1,413	5	1,459
11:45-12:00	135	0	283	3	81	0	153	0	70	0	61	0	319	3	30	0	59	0	108	0	119	1	34	0	1,452	7	1,520
12:00-12:15	135	2	282	1	81	0	153	3	62	0	104	0	345	2	57	0	43	0	68	0	105	1	76	0	1,511	9	1,575
12:15-12:30	179	1	353	3	75	0	145	0	37	0	96	0	403	5	47	0	49	0	64	0	87	0	31	0	1,566	9	1,575
12:30-12:45	134	0	361	3	55	0	127	0	45	0	84	0	378	1	62	0	51	0	119	0	124	0	31	0	1,571	4	1,598
12:45-13:00	153	0	333	1	75	0	128	1	35	0	103	1	430	6	62	0	59	0	86	0	86	0	39	0	1,589	9	1,549
13:00-13:15	141	0	353	0	87	0	157	0	30	0	97	0	383	5	40	0	47	0	74	0	102	0	33	0	1,544	5	1,591
13:15-13:30	144	1	349	2	58	0	153	1	48	0	105	0	403	4	43	0	49	0	102	0	94	0	35	0	1,583	8	1,583
13:30-13:45	135	0	378	0	76	0	140	0	42	0	120	0	386	2	31	0	41	0	101	0	97	0	34	0	1,581	2	1,479
13:45-14:00	165	0	387	0	73	0	104	0	36	2	91	0	311	4	49	0	52	0	59	0	105	0	41	0	1,473	6	1,468
14:00-14:15	178	3	370	0	83	0	75	1	61	0	81	1	341	1	56	0	26	0	81	0	74	0	36	0	1,462	6	1,453
14:15-14:30	167	2	388	0	79	0	107	2	46	1	82	0	311	1	44	0	36	0	75	0	64	0	48	0	1,447	6	1,548
14:30-14:45	157	1	406	0	80	0	103	1	34	1	52	0	415	3	45	0	55	0	81	0	87	0	27	0	1,542	6	1,334
14:45-15:00	138	3	325	4	69	0	101	0	43	0	80	0	266	1	54	0	55	0	64	0	96	0	35	0	1,326	8	1,276
15:00-15:15	136	0	286	3	64	0	86	0	31	0	85	0	310	0	49	1	35	0	82	0	73	0	35	0	1,272	4	1,319
15:15-15:30	126	1	285	1	75	2	124	0	32	2	95	0	300	2	32	0	37	0	77	0	97	0	31	0	1,311	8	1,273
15:30-15:45	127	0	331	2	37	0	73	1	25	2	82	0	320	3	41	1	43	0	80	0	77	0	28	0	1,264	9	1,367
15:45-16:00	136	1	356	2	76	0	147	2	31	0	105	0	239	1	50	0	32	0	83	0	80	0	25	1	1,360	7	1,481
16:00-16:15	134	0	368	3	47	0	123	0	41	0	98	0	327	1	46	0	50	0	87	0	104	0	52	0	1,477	4	1,503
16:15-16:30	165	0	379	2	52	0	136	1	32	2	76	0	363	1	42	0	43	0	79	0	94	0	36	0	1,497	6	1,341
16:30-16:45	174	0	284	0	50	0	114	1	34	1	74	0	324	4	52	0	32	0	75	0	86	0	36	0	1,335	6	1,680
16:45-17:00	165	0	589	0	92	0	99	0	11	0	68	0	378	3	43	0	56	0	92	0	63	0	21	0	1,677	3	1,577
17:00-17:15	132	0	451	0	56	0	100	0	43	1	46	0	430	5	48	0	45	0	83	0	103	0	34	0	1,571	6	1,808
17:15-17:30	154	1	407	0	86	0	96	2	28	0	62	0	614	4	54	0	46	0	98	1	110	0	45	0	1,800	8	1,706
17:30-17:45	184	0	456	1	67	0	85	0	63	2	34	0	439	6	56	0	44	0	84	0	141	0	44	0	1,697	9	1,547
17:45-18:00	172	0	408	1	68	0	110	0	74	2	35	0	301	3	47	0	43	0	100	0	142	0	41	0	1,541	6	1,466
18:00-18:15	164	0	392	1	50	0	123	1	38	4	39	0	326	3	48	0	44	0	77	0	117	0	39	0	1,457	9	1,455
18:15-18:30	153	0	424	3	54	0	85	0	40	2	36	0	311	1	63	0	66	0	77	0	102	0	38	0	1,449	6	1,426
18:30-18:45	130	1	362	5	48	0	125	0	34	2	80	0	298	3	47	0	46	0	86	0	110	0	49	0	1,415	11	1,382
18:45-19:00	103	1	400	8	77	0	99	0	41	5	57	0	228	11	49	0	53	0	112	0	91	0	47	0	1,357	25	1,534
19:00-19:15	131	3	325	5	86	1	92	0	41	1	69	0	339	2	67	0	56	0	86	0	137	8	85	0	1,514	20	1,358
19:15-19:30	117	2	364	4	96	0	75	0	35	0	64	0	314	1	30	0	35	0	70	0	85	8	58	0	1,343	15	1,237
19:30-19:45	124	2	332	1	87	0	70	1	22	1	72	0	225	2	35	0	36	0	78	0	82	0	67	0	1,230	7	1,243
19:45-20:00	123	3	333	6	107	0	56	0	25	0	48	0	262	6	35	0	48	0	88	0	65	0	38	0	1,228	15	1,042
20:00-20:15	83	4	244	7	103	4	52	0	14	1	20	0	278	2	30	0	40	0	89	0	49	0	22	0	1,024	18	978
20:15-20:30	94	1	243	3	96	0	51	0	12	0	20	0	243	2	35	0	29	0	45	0	67	0	37	0	972	6	914
20:30-20:45	71	2	242	2	86	0	42	0	14	1	19	0	246	6	26	0	33	0	42	10	38	0	34	0	893	21	780
20:45-21:00	62	3	223	3	77	0	46	1	6	0	29	0	157	2	17	0	24	0	64	0	50	0	16	0	771	9	683
21:00-21:15	72	1	197	2	37	0	31	1	20	0	18	0	141	2	21	0	20	0	48	0	50	0	22	0	677	6	692
21:15-21:30	55	2	198	6	34	0	34	0	17	0	25	0	104	2	25	0	36	0	55	1	62	0	36	0	681	11	606
21:30-21:45	49	0	189	0	36	0	36	0	8	2	23	0	87	1	29	2	26	0	50	0	42	0	26	0	601	5	649
21:45-22:00	29	0	278	0	27	0	23	1	8	1	16	0	108	2	38	0	11	0	41	0	40	0	26	0	645	4	0
Minimum Volume	29	0	100	0	27	0	23	0	6	0	16	0	87	0	9	0	11	0	0	0	17	0	13	0	565	0	0
Maximum Volume	184	4	589	8	121	4	272	3	74	5	244	1	623	11	67	2	69	0	119	10	142	8	98	1	1,816	25	1,823

Narahenpita Junction	For all Legs									N-S Direction											
	LT			TH			RT			LT				TH				RT			
Wednesday	Sum		Total	Sum		Total	Sum		Total	Sum		Difference		Sum		Difference		Sum		Difference	
24/06/2009	LV	HV		LV	HV		LV	HV		LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV
6:00- 6:15	85	0	85	335	2	337	145	0	145	63	0	13	0	233	2	19	0	47	0	29	0
6:15- 6:30	152	0	152	569	5	574	226	0	226	56	0	18	0	268	3	68	1	53	0	33	0
6:30- 6:45	181	0	181	739	0	739	289	0	289	84	0	12	0	402	5	68	3	57	0	37	0
6:45- 7:00	221	3	224	899	3	902	347	0	347	106	0	22	0	540	0	122	0	78	0	52	0
7:00- 7:15	269	1	270	977	2	979	374	0	374	139	0	41	0	687	3	233	1	121	0	43	0
7:15- 7:30	280	0	280	949	1	950	340	0	340	141	0	73	0	690	2	74	2	124	0	44	0
7:30- 7:45	295	1	296	1,001	1	1,002	358	0	358	161	0	55	0	674	1	126	1	114	0	24	0
7:45- 8:00	364	0	364	959	0	959	406	0	406	159	0	83	0	667	1	151	1	108	0	16	0
8:00- 8:15	321	0	321	1,007	1	1,008	419	1	420	218	0	96	0	736	0	76	0	163	0	79	0
8:15- 8:30	333	0	333	1,099	4	1,103	369	0	369	168	0	50	0	784	1	280	1	114	0	32	0
8:30- 8:45	299	0	299	914	6	920	360	1	361	209	0	103	0	894	3	234	3	153	0	61	0
8:45- 9:00	315	1	316	1,166	5	1,171	335	1	336	180	0	42	0	718	6	198	2	120	0	42	0
9:00- 9:15	301	0	301	746	8	754	323	1	324	190	1	62	1	981	5	265	1	143	0	45	0
9:15- 9:30	287	0	287	929	5	934	300	2	302	187	0	55	0	584	8	54	2	136	0	38	0
9:30- 9:45	343	0	343	936	10	946	312	2	314	167	0	65	0	625	5	63	3	123	0	37	0
9:45-10:00	359	1	360	760	8	768	331	3	334	188	0	82	0	692	10	110	2	113	0	15	0
10:00-10:15	325	1	326	755	9	764	347	4	351	186	1	48	1	556	8	100	4	138	0	36	0
10:15-10:30	342	0	342	671	2	673	407	1	408	166	1	62	1	612	7	92	1	155	0	55	0
10:30-10:45	334	0	334	704	10	714	309	1	310	204	0	110	0	504	2	86	0	133	0	33	0
10:45-11:00	298	0	298	674	5	679	337	3	340	203	0	81	0	459	7	35	1	142	0	52	0
11:00-11:15	338	0	338	822	8	830	302	2	304	187	0	67	0	499	4	89	4	168	2	70	2
11:15-11:30	290	0	290	839	4	843	284	1	285	189	0	89	0	619	8	3	6	123	1	49	1
11:30-11:45	372	0	372	782	7	789	298	0	298	167	0	57	0	673	4	83	2	136	0	36	0
11:45-12:00	308	2	310	836	4	840	367	3	370	194	0	76	0	602	6	36	0	111	0	51	0
12:00-12:15	329	1	330	939	8	947	298	0	298	178	2	92	2	627	3	63	1	138	0	24	0
12:15-12:30	349	0	349	947	4	951	275	0	275	228	1	130	1	756	8	50	2	122	0	28	0
12:30-12:45	333	0	333	952	8	960	304	1	305	185	0	83	0	739	4	17	2	117	0	7	0
12:45-13:00	292	0	292	935	5	940	317	0	317	212	0	94	0	763	7	97	5	137	0	13	0
13:00-13:15	343	1	344	951	6	957	289	1	290	188	0	94	0	736	5	30	5	127	0	47	0
13:15-13:30	319	0	319	981	2	983	281	0	281	193	1	95	1	752	6	54	2	101	0	15	0
13:30-13:45	312	2	314	894	4	898	267	0	267	176	0	94	0	764	2	8	2	107	0	45	0
13:45-14:00	346	3	349	866	2	868	250	1	251	217	0	113	0	698	4	76	4	122	0	24	0
14:00-14:15	324	3	327	845	1	846	278	2	280	204	3	152	3	711	1	29	1	139	0	27	0
14:15-14:30	327	2	329	960	3	963	255	1	256	203	2	131	2	699	1	77	1	123	0	35	0
14:30-14:45	300	3	303	767	5	772	259	0	259	212	1	102	1	821	3	9	3	125	0	35	0
14:45-15:00	284	0	284	754	3	757	234	1	235	193	3	83	3	591	5	59	3	123	0	15	0
15:00-15:15	272	3	275	777	3	780	262	2	264	171	0	101	0	596	3	24	3	113	1	15	1
15:15-15:30	275	2	277	810	5	815	179	2	181	163	1	89	1	585	3	15	1	107	2	43	2
15:30-15:45	282	1	283	780	3	783	298	3	301	170	0	84	0	651	5	11	1	78	1	4	1
15:45-16:00	312	0	312	897	4	901	268	0	268	168	1	104	1	595	3	117	1	126	0	26	0
16:00-16:15	319	2	321	912	3	915	266	1	267	184	0	84	0	695	4	41	2	93	0	1	0
16:15-16:30	315	1	316	768	4	772	252	1	253	208	0	122	0	742	3	16	1	94	0	10	0
16:30-16:45	324	0	324	1,098	3	1,101	255	0	255	206	0	142	0	608	4	40	4	102	0	2	0
16:45-17:00	303	1	304	1,030	5	1,035	238	0	238	221	0	109	0	967	3	211	3	135	0	49	0
17:00-17:15	326	2	328	1,193	4	1,197	281	2	283	177	0	87	0	881	5	21	5	104	0	8	0
17:15-17:30	375	2	377	1,070	7	1,077	252	0	252	200	1	108	1	1,021	4	207	4	140	0	32	0
17:30-17:45	389	2	391	886	4	890	266	0	266	228	0	140	0	895	7	17	5	123	0	11	0
17:45-18:00	323	4	327	874	4	878	260	1	261	215	0	129	0	709	4	107	2	115	0	21	0
18:00-18:15	336	2	338	873	4	877	240	0	240	208	0	120	0	718	4	66	2	98	0	2	0
18:15-18:30	296	3	299	850	8	858	269	0	269	219	0	87	0	735	4	113	2	117	0	9	0
18:30-18:45	309	6	315	776	19	795	272	0	272	176	1	84	1	660	8	64	2	95	0	1	0
18:45-19:00	314	4	318	870	15	885	330	1	331	156	1	50	1	628	19	172	3	126	0	28	0
19:00-19:15	257	2	259	827	13	840	259	0	259	187	3	75	3	664	7	14	3	153	1	19	1
19:15-19:30	260	3	263	711	3	714	259	1	260	152	2	82	2	678	5	50	3	126	0	66	0
19:30-19:45	284	3	287	708	12	720	236	0	236	160	2	88	2	557	3	107	1	122	0	52	0
19:45-20:00	226	5	231	591	9	600	207	4	211	171	3	75	3	595	12	71	0	142	0	72	0
20:00-20:15	180	1	181	573	5	578	219	0	219	123	4	43	4	522	9	34	5	133	4	73	4
20:15-20:30	160	13	173	545	8	553	188	0	188	123	1	65	1	486	5	0	1	131	0	61	0
20:30-20:45	156	3	159	459	5	464	156	1	157	104	2	38	2	488	8	4	4	112	0	60	0
20:45-21:00	160	1	161	406	4	410	111	1	112	86	3	38	3	380	5	66	1	94	0	60	0
21:00-21:15	163	3	166	389	8	397	129	0	129	92	1	52	1	338	4	56	0	58	0	16	0
21:15-21:30	133	2	135	341	1	342	127	2	129	91	2	19	2	302	8	94	4	59	0	9	0
21:30-21:45	89	1	90	442	2	444	114	1	115	75	0	23	0	276	1	102	1	65	2	7	2
21:45-22:00	18,104	97	18,201	51,703	329	52,032	17,573	57	17,630	40	0	18	0	386	2	170	2	65	0	11	0
Minimum Volume	85	0		335	0		111	0	112	40	0	12	0	233	0	0	0	47	0	1	0
Maximum Volume	18,104	97		51,703	329		17,573	57	17,630	228	4	152	4	1,021	19	280	6	168	4	79	4

Narahenpita Junction	E-W Direction												N-S Vs E-W Direction											
	LT				TH				RT				LT				TH				RT			
Wednesday	Sum		Difference		Sum		Difference		Sum		Difference		Sum		Difference		Sum		Difference		Sum		Difference	
24/06/2009	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV
6:00- 6:15	22	0	22	0	102	0	64	0	98	0	62	0	85	0	41	0	335	2	131	2	145	0	51	0
6:15- 6:30	40	0	18	0	120	0	86	0	135	1	107	1	96	0	16	0	388	3	148	3	188	1	82	1
6:30- 6:45	68	0	2	0	167	0	103	0	169	0	143	0	152	0	16	0	569	5	235	5	226	0	112	0
6:45- 7:00	75	0	11	0	199	0	125	0	211	0	171	0	181	0	31	0	739	0	341	0	289	0	133	0
7:00- 7:15	82	3	32	3	212	0	90	0	226	0	196	0	221	3	57	3	899	3	475	3	347	0	105	0
7:15- 7:30	128	1	32	1	287	0	167	0	250	0	216	0	269	1	13	1	977	2	403	2	374	0	126	0
7:30- 7:45	119	0	9	0	275	0	81	0	226	0	184	0	280	0	42	0	949	1	399	1	340	0	112	0
7:45- 8:00	136	1	44	1	334	0	154	0	250	0	204	0	295	1	23	1	1,001	1	333	1	358	0	142	0
8:00- 8:15	146	0	14	0	223	0	115	0	243	0	175	0	364	0	72	0	959	0	513	0	406	0	80	0
8:15- 8:30	153	0	23	0	223	0	127	0	305	1	239	1	321	0	15	0	1,007	1	561	1	419	1	191	1
8:30- 8:45	124	0	44	0	205	1	85	1	216	0	154	0	333	0	85	0	1,099	4	689	2	369	0	63	0
8:45- 9:00	119	0	13	0	196	0	130	0	240	1	128	1	299	0	61	0	914	6	522	6	360	1	120	1
9:00- 9:15	125	0	25	0	185	0	65	0	192	1	104	1	315	1	65	1	1,166	5	796	5	335	1	49	1
9:15- 9:30	114	0	30	0	162	0	54	0	187	1	115	1	301	0	73	0	746	8	422	8	323	1	51	1
9:30- 9:45	120	0	12	0	304	0	138	0	177	2	121	2	287	0	47	0	929	5	321	5	300	2	54	2
9:45-10:00	155	0	7	0	244	0	110	0	199	2	117	2	343	0	33	0	936	10	448	10	312	2	86	2
10:00-10:15	173	0	37	0	204	0	66	0	193	3	133	3	359	1	13	1	760	8	352	8	331	3	55	3
10:15-10:30	159	0	33	0	143	2	31	2	192	4	68	2	325	1	7	1	755	9	469	5	347	4	37	4
10:30-10:45	138	0	18	0	167	0	1	0	274	1	78	1	342	0	66	0	671	2	337	2	407	1	141	1
10:45-11:00	131	0	37	0	245	3	9	3	167	1	57	1	334	0	72	0	704	10	214	4	309	1	25	1
11:00-11:15	111	0	51	0	175	1	5	1	169	1	67	1	298	0	76	0	674	5	324	3	337	3	1	1
11:15-11:30	149	0	27	0	203	0	1	0	179	1	91	1	338	0	40	0	822	8	416	8	302	2	56	0
11:30-11:45	123	0	53	0	166	0	18	0	148	1	98	1	290	0	44	0	839	4	507	4	284	1	12	1
11:45-12:00	178	0	38	0	180	1	58	1	187	0	119	0	372	0	16	0	782	7	422	5	298	0	76	0
12:00-12:15	130	0	6	0	209	1	1	1	229	3	77	3	308	2	48	2	836	4	418	2	367	3	91	3
12:15-12:30	101	0	27	0	183	0	9	0	176	0	114	0	329	1	127	1	939	8	573	8	298	0	54	0
12:30-12:45	164	0	74	0	208	0	40	0	158	0	96	0	349	0	21	0	947	4	531	4	275	0	41	0
12:45-13:00	121	0	51	0	189	1	17	1	167	1	89	1	333	0	91	0	952	8	574	6	304	1	30	1
13:00-13:15	104	0	44	0	199	0	5	0	190	0	124	0	292	0	84	0	935	5	537	5	317	0	63	0
13:15-13:30	150	0	54	0	199	0	11	0	188	1	118	1	343	1	43	1	951	6	553	6	289	1	87	1
13:30-13:45	143	0	59	0	217	0	23	0	174	0	106	0	319	0	33	0	981	2	547	2	281	0	67	0
13:45-14:00	95	2	23	2	196	0	14	0	145	0	63	0	312	2	122	2	894	4	502	4	267	0	23	0
14:00-14:15	142	0	20	0	155	1	7	1	111	1	39	1	346	3	62	3	866	2	556	0	250	1	28	1
14:15-14:30	121	1	29	1	146	0	18	0	155	2	59	2	324	3	82	1	845	1	553	1	278	2	32	2
14:30-14:45	115	1	47	1	139	0	35	0	130	1	76	1	327	2	97	0	960	3	682	3	255	1	5	1
14:45-15:00	107	0	21	0	176	0	16	0	136	0	66	0	300	3	86	3	767	5	415	5	259	0	13	0
15:00-15:15	113	0	51	0	158	0	12	0	121	0	51	0	284	0	58	0	754	3	438	3	234	1	8	1
15:15-15:30	109	2	45	2	192	0	2	0	155	0	93	0	272	3	54	1	777	3	393	3	262	2	48	2
15:30-15:45	105	2	55	2	159	0	5	0	101	1	45	1	275	2	65	2	810	5	492	5	179	2	23	0
15:45-16:00	114	0	52	0	185	0	25	0	172	3	122	1	282	1	54	1	780	3	410	3	298	3	46	3
16:00-16:15	128	0	46	0	202	0	6	0	175	0	71	0	312	0	56	0	897	4	493	4	268	0	82	0
16:15-16:30	111	2	47	2	170	0	18	0	172	1	100	1	319	2	97	2	912	3	572	3	266	1	78	1
16:30-16:45	109	1	41	1	160	0	12	0	150	1	78	1	315	1	97	1	768	4	448	4	252	1	48	1
16:45-17:00	103	0	81	0	131	0	5	0	120	0	78	0	324	0	118	0	1,098	3	836	3	255	0	15	0
17:00-17:15	126	1	40	1	149	0	57	0	134	0	66	0	303	1	51	1	1,030	5	732	5	238	0	30	0
17:15-17:30	126	1	70	1	172	0	48	0	141	2	51	2	326	2	74	0	1,193	4	849	4	281	2	1	2
17:30-17:45	147	2	21	2	175	0	107	0	129	0	41	0	375	2	81	2	1,070	7	720	7	252	0	6	0
17:45-18:00	174	2	26	2	177	0	107	0	151	0	69	0	389	2	41	2	886	4	532	4	266	0	36	0
18:00-18:15	115	4	39	4	156	0	78	0	162	1	84	1	323	4	93	4	874	4	562	4	260	1	64	1
18:15-18:30	117	2	37	2	138	0	66	0	123	0	47	0	336	2	102	2	873	4	597	4	240	0	6	0
18:30-18:45	120	2	52	2	190	0	30	0	174	0	76	0	296	3	56	1	850	8	470	8	269	0	79	0
18:45-19:00	153	5	71	5	148	0	34	0	146	0	52	0	309	6	3	4	776	19	480	19	272	0	20	0
19:00-19:15	127	1	45	1	206	8	68	8	177	0	7	0	314	4	60	2	870	15	458	1	330	1	24	1
19:15-19:30	105	0	35	0	149	8	21	8	133	0	17	0	257	2	47	2	827	13	529	3	259	0	7	0
19:30-19:45	100	1	56	1	154	0	10	0	137	1	3	1	260	3	60	1	711	3	403	3	259	1	15	1
19:45-20:00	113	0	63	0	113	0	17	0	94	0	18	0	284	3	58	3	708	12	482	12	236	0	48	0
20:00-20:15	103	1	75	1	69	0	29	0	74	0	30	0	226	5	20	3	591	9	453	9	207	4	59	4
20:15-20:30	57	0	33	0	87	0	47	0	88	0	14	0	180	1	66	1	573	5	399	5	219	0	43	0
20:30-20:45	56	11	28	9	57	0	19	0	76	0	8	0	160	13	48	9	545	8	431	8	188	0	36	0
20:45-21:00	70	0	58	0	79	0	21	0	62	1	30	1	156	3	16	3	459	5	301	5	156	1	32	1
21:00-21:15	68	0	28	0	68	0	32	0	53	1	9	1	160	1	24	1	406	4	270	4	111	1	5	1
21:15-21:30	72	1	38	1	87	0	37	0	70	0	2	0	163	3	19	1	389	8	215	8	129	0	11	0
21:30-21:45	58	2	42	2	65	0	19	0	62	0	10	0	133	2	17	2	341	1	211	1	127	2	3	2
21:45-22:00	49	1	33	1	56	0	24	0	49	1	3	1	89	1	9	1	442	2	330	2	114	1	16	1
Minimum Volume	22	0	2	0	56	0	1	0	49	0	2	0	85	0	3	0	335	0	131	0	111	0	1	0
Maximum Volume	178	11	81	9	334	8	167	8	305	4	239	3	389	13	127	9	1,193	19	849	19	419	4	191	4

Table 10-2: Comparison of traffic turning movements of selected intersections in Colombo District

SIDRA NUMBERING	7 NL-LT		8 NL-TH		9 NL-RT		6 EL-RT		4 EL-LT		5 EL-TH		2 SL-TH		3 SL-RT		1 SL-LT		10 WL-LT		11 WL-TH		12 WL-RT	
	Northern Leg						Eastern Leg						Southern Leg						Western Leg					
Traffic Turning Movement	LT		TH		RT		RT		LT		TH		TH		RT		LT		LT		TH		RT	
Numbering Traffic Count	12		13		14		21		23		24		31		32		34		41		42		43	
	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV
Thimbirigasyaya Junction Tuesday 14/07/2009																								
Minimum Volume	6	0	152	0	1	0	7	0	1	0	1	0	155	0	0	0	2	0	11	0	1	0	4	0
Maximum Volume	69	1	603	10	49	0	99	2	18	0	76	0	750	9	60	0	126	2	82	0	101	1	112	7
Punchikawatta Junction Friday 31/07/2009																								
Minimum Volume or Ratio	9	0	89	0	11	0	0	0	9	0	29	0	76	0	4	0	0	0	37	0	22	0	1	0
Maximum Volume or Ratio	124	3	349	4	1	3	45	1	133	1	303	2	487	4	55	3	33	2	120	5	127	2	41	1
Narahenpita Junction Wednesday 24/06/2009																								
Minimum Volume	29	0	100	0	27	0	23	0	6	0	16	0	87	0	9	0	11	0	0	0	17	0	13	0
Maximum Volume	184	4	589	8	121	4	272	3	74	5	244	1	623	11	67	2	69	0	119	10	142	8	98	1
Kirulapona Junction Wednesday 15/07/2009																								
Minimum Volume or Ratio	32	0	0	0	32	0	33	0	0	0	91	0	26	0	0	0	4	0	4	0	31	0	0	0
Maximum Volume or Ratio	220	6	285	1	104	4	199	4	18	1	479	3	244	1	52	0	46	1	43	4	564	1	31	2
Grandpass Junction Tuesday 04/08/2009																								
Minimum Volume or Ratio	0	0	0	0	3	0	0	0	1	0	41	0	3	0	1	0	0	0	19	0	5	0	0	0
Maximum Volume or Ratio	41	3	39	1	50	1	18	2	48	2	255	2	61	3	67	1	228	6	176	3	48	1	153	4
Orugodawatta Junction Tuesday 09/06/2009																								
Minimum Volume or Ratio	10	0	0		19	0	7	0	36	0	21	0	241	0	20	0	1	0	0	0	9	0	0	0
Maximum Volume or Ratio	55	19	8		111	6	60	15	167	9	110	8	1,090	12	105	3	138	4	26	1	125	12	100	1
Minimum Volume or Ratio	0	0	0	0	1	0	0	0	0	0	1	0	3	0	0	0	0	0	0	0	1	0	0	0
Maximum Volume or Ratio	220	19	603	10	121	6	272	15	167	9	479	8	1,090	12	105	3	228	6	176	10	564	12	153	7

Tuesday 09/06/2009	Northern Leg						Southern Leg						N-S Direction											
Traffic Turning Movement	LT		TH		RT		TH		RT		LT		LT				TH				RT			
Numbering Traffic Count	12		13		14		31		32		34		Sum		Difference		Sum		Difference		Sum		Difference	
	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV
Minimum Volume or Ratio	0	0	0	0	1	0	3	0	0	0	0	0	0	0	0	0	6	0	0	0	4	0	0	0
Maximum Volume or Ratio	220	19	603	10	121	6	1,090	12	105	3	228	6	240	19	217	19	1,846	19	713	9	181	7	101	6

Table 10-3: Detailed comparison of traffic turning movements of selected intersections in Colombo District

	Sum		Total	For all Legs LT Sum		Total	For all Legs TH Sum		Total	For all Legs RT Sum		Total	N-S Direction LT Sum		LT Difference		TH Sum		TH Difference		RT Sum		RT Difference		E-W Direction LT Sum		LT Difference		TH Sum		TH Difference		RT Sum		RT Difference		N-S Vs E-W Direction LT Sum		LT Difference		TH Sum		TH Difference		RT Sum		RT Difference		N-S Direction Ratio LT w.r.t TH North	LT w.r.t TH South	TH w.r.t TH North	TH w.r.t TH South	RT w.r.t TH North	RT w.r.t TH South	E-W Direction Ratio LT w.r.t TH East	LT w.r.t TH West	TH w.r.t TH East	TH w.r.t TH West	RT w.r.t TH East	RT w.r.t TH West
Traffic Turning Movement Numbering Traffic Count	LV	HV		LV	HV		LV	HV		LV	HV		LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	LV	LV	LV	LV	LV	LV	LV	LV	LV	LV	LV
Thimbirigasyaya Junction Tuesday 14/07/2009																																																												
Minimum Volume	361	0	370	31	0		312	0		18	0	18	11	0	0	0	307	0	1	0	4	0	2	0	12	0	4	0	2	0	0	0	12	0	0	0	31	0	1	0	312	0	302	0	18	0	0	0	0.02	0.01	0.39	0.63	0.00	0.00	0.07	0.25	0.24	0.17	0.65	0.65
Maximum Volume	1591	18	1594	194	2		1283	16		203	7	203	153	2	100	2	1235	16	447	7	79	0	46	0	100	0	64	0	130	1	73	1	168	7	78	7	194	2	120	2	1283	16	1187	16	203	7	133	7	0.16	0.23	1.58	2.55	0.16	0.16	1.08	7.50	5.86	4.25	4.33	4.38
Punchikawatta Junction Friday 31/07/2009																																																												
Minimum Volume or Ratio	410	0	411	58	0		279	0		18	0	18	10	0	8	0	174	0	4	0	10	0	0	0	46	0	0	0	48	0	4	0	1	0	1	0	58	0	5	0	279	0	16	0	18	0	0	0	0.01	0.00	0.28	0.52	0.00	0.02	0.11	0.51	0.62	0.22	0.00	0.03
Maximum Volume or Ratio	1265	12	1266	302	7		953	7		139	3	140	132	4	119	2	789	7	306	4	115	3	77	3	215	5	76	5	391	3	215	2	82	1	41	1	302	7	170	5	953	7	633	7	139	3	104	3	0.90	0.19	1.92	3.64	0.50	0.25	1.41	4.64	4.45	1.62	0.45	1.14
Narahenpita Junction Wednesday 24/06/2009																																																												
Minimum Volume	565	0	0	85	0		335	0		111	0	112	40	0	12	0	233	0	0	0	47	0	1	0	22	0	2	0	56	0	1	0	49	0	2	0	85	0	3	0	335	0	131	0	111	0	1	0	0.10	0.07	0.47	0.39	0.10	0.04	0.16	0.00	0.24	0.17	0.67	0.22
Maximum Volume	1,816	25	1,823	389	13		1,193	19		419	4	420	228	4	152	4	1,021	19	280	6	168	4	79	4	178	11	81	9	334	8	167	8	305	4	239	3	389	13	127	9	1,193	19	849	19	419	4	191	4	0.75	0.35	2.57	2.11	0.58	0.35	2.11	2.00	6.06	4.15	3.15	1.70
Kirulapona Junction Wednesday 15/07/2009																																																												
Minimum Volume or Ratio	304	0	310	50	0		169	0		67	0	73	41	0	23	0	47	0	1	0	32	0	6	0	6	0	1	0	122	0	1	0	35	0	31	0	50	0	32	0	169	0	75	0	67	0	0	0	0.57	0.08	0.08	0.19	0.18	0.00	0.00	0.01	0.26	0.24	0.31	0.01
Maximum Volume or Ratio	1577	10	1579	261	8		1128	3		303	6	305	240	6	201	6	352	1	244	1	110	4	101	4	48	4	43	4	801	3	385	3	216	4	199	4	261	8	229	4	1128	3	544	3	303	6	129	3	7.08	0.55	5.17	13.31	4.46	0.88	0.09	0.19	4.10	3.81	0.77	0.29
Grandpass Junction Tuesday 04/08/2009																																																												
Minimum Volume or Ratio	94	0	94	25	0		64	0		5	0	5	0	0	0	0	6	0	0	0	4	0	0	0	25	0	4	0	56	0	20	0	1	0	1	0	25	0	2	0	64	0	16	0	5	0	1	0	0.00	0.00	0.09	0.18	0.47	0.09	0.01	0.66	1.56	0.04	0.00	0.00
Maximum Volume or Ratio	787	10	787	427	9		362	3		218	4	218	239	9	217	5	99	3	39	3	78	1	56	1	195	3	157	3	303	2	234	2	162	4	144	4	427	9	181	9	362	3	244	3	218	4	115	4	10.25	21.83	5.50	11.67	11.25	4.00	0.28	16.20	27.83	0.64	0.13	25.60
Orugodawatta Junction Tuesday 09/06/2009																																																												
Minimum Volume or Ratio	**1,063**	**1**	**1,079**	101	0		789	1		107	0	115	24	0	1	0	678	1	1	1	53	0	1	0	40	0	32	0	47	0	1	0	14	0	1	0	101	0	0	0	789	1	549	0	107	0	0	0	0.01	0.00	0.34	0.37	0.04	0.03	0.89	0.00	0.26	0.24	0.10	0.00
Maximum Volume or Ratio	**2,381**	**56**	**2,424**	340	26		1,986	24		287	21	287	172	19	104	19	1,846	18	713	9	181	7	57	6	168	9	166	9	161	15	93	9	147	15	76	15	340	26	102	16	1,986	24	1,706	14	287	21	155	14	0.11	0.27	2.67	2.96	0.21	0.20	2.97	1.00	4.22	3.91	1.44	4.71
Minimum Volume or Ratio	94	0	0	25	0	0	64	0	0	5	0	5	0	0	0	0	6	0	0	0	4	0	0	0	6	0	0	0	2	0	0	0	1	0	0	0	25	0	0	0	64	0	16	0	5	0	0	0	0.00	0.00	0.08	0.18	0.00	0.00	0.00	0.00	0.24	0.04	0.00	0.00
Maximum Volume or Ratio	2,381	56	2,424	427	26	0	1,986	24	0	419	21	420	240	19	217	19	1,846	19	713	9	181	7	101	6	215	11	166	9	801	15	385	9	305	15	239	15	427	26	229	16	1,986	24	1,706	19	419	21	191	14	10.25	21.83	5.50	13.31	11.25	4.00	2.97	16.20	27.83	4.25	4.33	25.60

Tuesday 09/06/2009

	Eastern Leg RT 21		Eastern Leg LT 23		Eastern Leg TH 24		Western Leg LT 41		Western Leg TH 42		Western Leg RT 43		E-W Direction LT Sum		LT Difference		TH Sum		TH Difference		RT Sum		RT Difference		Sum		Total	For all Legs LT Sum		Total	TH Sum		Total	RT Sum		Total	N-S Vs E-W Direction LT Sum		LT Difference		TH Sum		TH Difference		RT Sum		RT Difference		N-S Direction Ratio LT w.r.t TH North	LT w.r.t TH South	TH w.r.t TH North	TH w.r.t TH South	RT w.r.t TH North	RT w.r.t TH South	E-W Direction Ratio LT w.r.t TH East	LT w.r.t TH West	TH w.r.t TH East	TH w.r.t TH West	RT w.r.t TH East	RT w.r.t TH West
Traffic Turning Movement Numbering Traffic Count	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV		LV	HV		LV	HV		LV	HV		LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	HV	LV	LV	LV	LV	LV	LV	LV	LV	LV	LV	LV	LV
Minimum Volume or Ratio	0	0	0	0	1	0	0	0	1	0	0	0	6	0	0	0	2	0	0	0	1	0	0	0	94	0	0	25	0	0	64	0	0	5	0	5	25	0	0	0	64	0	16	0	5	0	0	0	0.00	0.00	0.08	0.18	0.00	0.00	0.00	0.00	0.24	0.04	0.00	0.00
Maximum Volume or Ratio	272	15	167	9	479	8	176	10	564	12	153	7	215	11	166	9	801	15	385	9	305	15	239	15	2,381	56	2,424	427	26	0	1,986	24	0	419	21	420	427	26	229	16	1,986	24	1,706	19	419	21	191	14	10.25	21.83	5.50	13.31	11.25	4.00	2.97	16.20	27.83	4.25	4.33	25.60

10.2. Contents of attached CD

- DVD RW Drive (F:) MEng_Thesis2011
  - Data Collection from Planning-I
    - TM Soft copy 3
      - Kelanitissa
      - Kirimandala
      - Narahenpita
      - Peliyagoda
    - TM-softcopy
      - Dematagoda-TM369
      - lesly-TM371
      - Liberty-TM378
      - Library-370
      - Pittala-TM379
      - Prison-TM372
      - Samantha hall- TM368
      - Urugodawatta- TM365
    - TM-Softcopy2
      - Borella
      - Diulgas
      - Hettiyawatta
      - Kirulapona
      - Panchikawatta
      - Senanayaka jn
      - Thimbirigasyaya
  - Licensed SIDRA Ver-4_Planning-I Run -Outputs
    - 4-Legs Three-Lanes_RT Bay_Actuated Phasing-A
    - 4-Legs Three-Lanes_RT Bay_Actuated Phasing-B
    - 4-Legs Three-Lanes_RT Bay_Fixed Phasing-A
    - 4-Legs Three-Lanes_RT Bay_Fixed Phasing-B
      - Sample Output_Site-00001
        - Data Summary
        - Output
          - Flow Displays_Veh-h_ped-h
          - Movement Displays
            - Cost, Fuel and Emissions
            - Delay, LOS and Capacity
            - Queue and Stops
            - Speed and Travel Time
      - Sample Output_Site-00002
      - Sample Output_Site-00014
      - Sample Output_Site-00050
  - SIDRA User Manuals_Screenshots
  - SIDRA Ver5_Software Trial Version 30days
  - Thesis References_Selected
    - Downloads
    - Highway Capacity Manual 2000
    - Road Sensors

11. GLOSSARY OF ROAD TRAFFIC ANALYSIS TERMS

Also see the Highway Capacity Manual Glossary available on www.sidrasolutions.com (Resources section).

11.1. Actuated Signal Control

A signal control method in which traffic signal phases and their timings are determined by traffic demands identified by detector actuations subject to various signal controller settings. See Fixed-Time (Pretimed) Signal Control.

11.2. Back of Queue

Maximum extent of the queue relative to the stop line or give-way (yield) line during a signal cycle or gap-acceptance cycle. The last queued vehicle that joins the back of queue is the last vehicle that departs at the end of the saturated part of green interval or the available gap interval.

11.3. Capacity

The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions; usually expressed as vehicles per hour, passenger cars per hour, or persons per hour.

11.4. Circulating Flow

The vehicle flow rate in all lanes of the circulating road in front of a roundabout entry lane, determined using Stopline Flow Rates.

11.5. Control Delay

Sum of Stop-Line Delay and Geometric Delay.

11.6. Cost (Operating Cost)

A measure that includes the direct vehicle operating cost (the resource cost of fuel and additional running costs including tyre, oil, repair and maintenance as a factor of the cost of fuel) as well as the time cost of vehicle occupants.

11.7. Critical Gap (Headway)

The minimum time between successive vehicles in the opposing (major) traffic stream that is acceptable for entry by opposed (minor) stream vehicles. See Headway.

11.8. Critical Intersection

The intersection in a coordinated signal system that operates with the highest overall degree of saturation during a given period.

11.9. Critical Lane

The lane in a lane group or approach that has the highest degree of saturation and places the highest demand on green time.

11.10. Critical Movements

The set of movements that determine the capacity and timing requirements of a signalised intersection.

11.11. Cycle

A complete sequence of signal phases.

11.12. Cycle-Average Queue

The average queue length that incorporates all queue states including zero queues as counted at regular intervals (e.g. every 5 seconds).

11.13. Cycle Length (Cycle Time)

Time required for one complete sequence of signal displays (sum of phase green and intergreen times). For a given movement, cycle time is the sum of the durations of red, yellow and green signal displays, or sum of Effective Green and Red Times. In gap-acceptance analysis, this is the equivalent average cycle time corresponding to the block and unblock periods in the opposing traffic stream.

11.14. Degree of Saturation

The ratio of arrival (demand) flow rate to capacity during a given flow period. Also known as the volume to capacity ratio.

11.15. Delay

The additional travel time experienced by a vehicle or pedestrian with reference to a base travel time (e.g. the free-flow travel time).

11.16. Demand Flow (Demand Volume)

The number of vehicles or pedestrians arriving during a given period as measured at the back of queue (as distinct from departure flows measured in front of the queue). See Stopline Flow Rate.

11.17. Density

The number of vehicles per unit distance along a road segment as measured at an instant in time.

11.18. Design Life

The number of years into the future while the intersection operates satisfactorily considering increases in traffic demand volumes.

11.19. Detector

A device by which vehicle or pedestrian traffic registers its presence. The most common detectors are the inductive loop detectors for vehicles and the push-button detectors for pedestrians.

11.20. Downstream

In the direction of the movement of traffic.

11.21. Effective Green and Red Times

The movement green and red times for capacity and performance analysis purposes, which are determined by adjusting the displayed green and red times for Start Loss and End Gain effects.

11.22. Effective Intersection Capacity

An aggregate measure of intersection capacity determined as the ratio of total intersection demand flow to the intersection degree of saturation, where the intersection degree of saturation is the largest lane degree of saturation considering all lanes of the intersection.

11.23. End Gain

Duration of the interval between the end of the displayed green period and the end of the effective green period for a movement. This is used in signal timing and performance analysis to allow for additional departures after the end of green period.

11.24. Equivalent Stop Value

Value of a deceleration-acceleration cycle in terms of a major stop-start cycle. See Major Stop.

11.25. Exclusive Pedestrian Phase

The phase at an intersection during which all pedestrian displays are green and all vehicle displays are red, allowing all pedestrian movements to operate simultaneously while all vehicle movements are stopped. Also see Scramble-Crossing Phase.

11.26. Exclusive Lane

A lane (or length of lane) allocated for use only by a particular movement or a type of vehicle, e.g. left-turn lane, through lane, right-turn lane, bus lane, as opposed to a Shared Lane.

11.27. Filter Turn

A turning movement that must give way to and find safe gaps in conflicting (opposing) vehicle or pedestrian traffic before proceeding, e.g. filter right-turn, slip-lane left turn, left turn on red. Also see Opposed Movement.

11.28. Fixed-Time (Pretimed) Signal Control

A signal control method that allows for only a fixed sequence and fixed duration of displays. See Actuated Signal Control.

11.29. Flow Rate

Number of vehicles or pedestrians per unit time passing (arriving or departing) a given reference point.

11.30. Flow Ratio

The ratio of arrival (demand) flow rate to saturation flow rate.

11.31. Follow-up Headway

The average headway between successive opposed (minor) stream vehicles entering a gap available in the opposing (major) traffic stream. The Follow-up Headway (seconds) is a saturation (queue discharge) headway, and the corresponding saturation flow rate (vehicles per hour) in gap-acceptance analysis is 3600 / Follow-up Headway. See Saturation Flow Rate.

11.32. Free-Flow Speed

The uninterrupted traffic speed when density is approximately zero, i.e. when only few vehicles are present in the traffic stream.

11.33. Full Control

Control of a turning movement using three-aspect (red, yellow, green) turn arrows on a six-aspect signal face, where the green arrow indicates that the vehicle can turn unopposed (with no opposing vehicle or pedestrian traffic) and the red arrow indicates that the vehicle is not permitted to turn (filter turns not permitted).

11.34. Gap Acceptance

The process by which an opposed (minor) stream vehicle accepts an available gap in the opposing (major) stream for entering (departing from queue or merging). See Critical Gap (Headway).

11.35. Gap Setting

A controller setting equivalent to a predetermined space time measured between successive vehicles at the given (approach) speed, detection zone length and vehicle length values that will cause the signal controller to terminate the green display. See Space Time.

11.36. Geometric Delay

Delay due to physical and basic traffic control factors as experienced by a vehicle negotiating the intersection in the absence of any other vehicles (due to a deceleration from the approach cruise speed down to an approach negotiation speed, travel at that speed, acceleration to an exit negotiation speed, and then acceleration to the exit cruise speed).

11.37. Geometric Stop

The effective value of a slow-down and speed-up manoeuvre associated with Geometric Delay, which is measured in terms of equivalent Major Stops.

11.38. Green Time

Duration of the green display for a phase or a movement.

11.39. Headway

The time between passage of the front ends of two successive vehicles. See Spacing.

11.40. Intergreen Time

Duration of the clearance part of the phase corresponding to the period between the phase change point (the end of running intervals) and the beginning of the green display for the next phase (end of phase). Normally, it comprises Yellow Time and All-Red Time.

11.41. Intra-Bunch Headway

Average headway between vehicles in a moving queue (minimum headway in a stream). This is used in order to define moving queues (bunches) of vehicles for the purpose of modelling headway distribution of vehicles.

11.42. Lane Group

A set of lanes with one or two shared lanes (e.g. Lane 1: Left-Turn and Through, Lane 2: Through) or a set of exclusive turn lanes (e.g. a single Right-Turn lane).

11.43. Lane Utilisation

The distribution of vehicles among lanes when two or more lanes are available for a movement.

11.44. Lane Utilisation Ratio

Ratio of the lane degree of saturation to the highest (critical) lane degree of saturation in a Lane Group.

11.45. Level of Service

An index of the operational performance of traffic on a given traffic lane, carriageway, road or intersection, based on service measures such as delay, degree of saturation, density and speed during a given flow period.

11.46. Major Stop

A drive cycle element that involves a deceleration from the approach cruise speed to zero speed and an acceleration from zero speed to the exit cruise speed.

11.47. Occupancy Time

The time that starts when the front of a vehicle enters the detection zone and finishes when the back of the vehicle exits the detection zone, i.e. the duration of the period when the detection zone is occupied by a vehicle.

11.48. Off-Peak Period

The periods that have low demand volumes of traffic during the day (24-hour period).

11.49. Offset

The difference between the start or end times of green periods at adjacent (upstream and downstream) signals.

11.50. Opposed Movement

A movement (Filter Turn, Permitted Turn, Minor Movement) that gives way to one or more opposing traffic streams at a signalised or unsignalised intersection.

11.51. Opposing Movement

A movement that conflicts with, and has priority over, another (opposed) movement.

11.52. Overflow

An interrupted traffic condition when a number of queued vehicles are not able to depart due to insufficient capacity during a traffic signal or gap-acceptance cycle (also known as cycle failure).

11.53. Overflow Queue

Average number of vehicles per cycle left over at the end of green periods at signals or at the end of acceptable gap (unblock) periods during gap-acceptance process.

11.54. Overlap Movement

A movement that runs in consecutive phases without stopping during the associated intergreen period(s).

11.55. Parallel Pedestrian Movement

A signalised pedestrian movement that runs at the same time as the parallel vehicle movement (s) that are controlled by circular green displays.

11.56. Peak Flow Factor (PFF)

Ratio of the average demand flow rate in the Total Flow Period (e.g. one hour) to the demand flow rate in the Peak Flow Period (e.g. 15 minutes). This is equivalent to the more traditional term Peak Hour Factor (PHF) when the Total Flow Period is one hour.

11.57. Peak Period

The period that has the highest demand volume of traffic during the day (peak hour, peak half hour, etc).

11.58. Pedestrian Clearance Period

The Flashing Don't Walk period that immediately follows the termination of pedestrian Walk display to enable pedestrians, who have just stepped off the kerb at the commencement of this period, to complete their crossing to the nearest kerb or refuge.

11.59. Pedestrian Crossing

A transverse strip of carriageway marked for the use of pedestrians crossing the road (mid-block or at intersections) at a place with a pedestrian crossing sign, and with or without alternating flashing twin yellow lights. Also called Zebra Crossing where indicated by parallel white stripes on the road surface.

11.60. Pedestrian Minimum Green Time

Minimum time required for both Walk and Flashing Don't Walk displays, but excluding any overlaps with terminating intergreen displays.

11.61. Performance Index

A measure that combines several performance statistics such as delay, number of stops and queue length.

11.62. Phase

That part of a signal cycle during which one or more movements receive right of way subject to resolution of any vehicle or pedestrian conflicts by priority rules. A phase is identified by at least one movement gaining right of way at the start of it and at least one movement losing right of way at the end of it.

11.63. Phase Sequence

The order of phases in a signal cycle.

11.64. Phase Split

Duration of each phase (Green Time and Intergreen Time) within a signal cycle. It is normally expressed as a percentage of cycle length.

11.65. Platoon

A group of vehicles or pedestrians travelling together because of signal control, geometric conditions or other factors.

11.66. Platoon Ratio

Ratio of the average arrival flow rate during the green period to the average arrival flow rate during the signal cycle.

11.67. Practical Degree of Saturation

A target, or maximum, degree of saturation that corresponds to an acceptable level of traffic performance.

11.68. Practical Spare Capacity

The amount of increase possible in the demand flow rate to obtain a degree of saturation equal to the practical (target) degree of saturation.

11.69. Progression

Progression is a time-relationship, between adjacent traffic signals, which allows vehicle platoons to be given a green signal as they pass through the sequence of intersections.

11.70. Progression Factor Method

A simple technique to determine signal coordination effect on delay, queue length, stop rate, etc. where detailed platoon patterns generated at upstream signals are not available.

11.71. Proportion Queued

Proportion of traffic that is queued due to the effects of traffic control and the existence of other vehicles. This is related to the Major Stops or Slow Downs from the approach cruise speed.

11.72. Queue

A line of vehicles or pedestrians waiting to proceed through an intersection. Slowly moving vehicles or pedestrians joining the back of the queue are usually considered part of the queue. The internal queue dynamics can involve starts and stops. A faster-moving line of vehicles is often referred to as a moving queue or a platoon. See Back of Queue and Cycle-Average Queue.

11.73. Queuing Delay

Part of the Stop-Line Delay that includes the Stopped Delay (while vehicle is idling at near-zero speed) and the Queue Move-up delay (while a queued vehicle accelerates towards the stop-line but stops again, e.g. because the signal display changes to red).

11.74. Queue Storage Ratio

The ratio of the queue length to the available queue storage distance.

11.75. Red Time

Duration of the red signal display for a phase or a movement.

11.76. Saturation Flow Rate

The maximum departure (queue discharge) flow rate achieved by vehicles departing from the queue during the green period at traffic signals. Saturation Headway (seconds) is 3600 / Saturation Flow Rate (vehicles per hour). The Follow-up Headway parameter used in gap-acceptance analysis is a saturation (queue discharge) headway. See SCATS Maximum Flow (MF) and Follow-up Headway.

11.77. SCATS Maximum Flow (MF)

A maximum departure flow rate during a fully saturated green period averaged over the green and intergreen times as a special measure of saturation flow rate. See Saturation Flow Rate.

11.78. Scramble-Crossing Phase

An Exclusive Pedestrian Phase at an intersection where pedestrians are allowed to cross in any direction including diagonally within the limits of the crosswalk lines.

11.79. Shared Lane

A lane allocated for use by two or more movements, e.g. shared through and right-turn lane, as opposed to an Exclusive Lane.

11.80. Short Lane

A lane of limited length, e.g. a turn bay or part of a lane available downstream of parked vehicles.

11.81. Signalised Crossing

An area of the road used by pedestrians when crossing the road with the guidance of pedestrian signals at a mid-block or intersection location, and can be used by cyclists if bicycle signals are provided.

11.82. Signal Phasing

Sequential arrangement of separately controlled groups of vehicle and pedestrian movements within a signal cycle to allow all vehicle and pedestrian movements to proceed.

11.83. Slip Lane

A turning movement lane separated from an adjacent lane by a triangular island.

11.84. Slow Down

A drive cycle element that involves a deceleration from the approach cruise speed to a non-zero intermediate speed and an acceleration from the intermediate speed to the exit cruise speed.

11.85. Space Length (Gap Distance)

The following distance between two successive vehicles as measured between the rear end of one vehicle and the front end of the next vehicle in the same traffic lane (spacing less vehicle length).

11.86. Space Time

The time between the detection of two consecutive vehicles when the presence detection zone is not occupied.

11.87. Spacing

The distance between the front ends of two successive vehicles in the same traffic lane.

11.88. Speed

Distance travelled per unit time. In a time-distance diagram, the slope of the time-distance trace of a vehicle is its speed. Approach Speed is the uninterrupted (mid-block) cruise speed of vehicles before being affected by traffic signals. This can be represented by the speed limit. Negotiation Speed is the safe speed of a vehicle moving through the controlled area of the intersection. For turning vehicles, this can be determined as a function of the negotiation radius. Running Speed is the average speed including the effect of delays due to interrupted conditions but not including any stopped (idling) times. Travel Speed is the average speed including the effect of all delays. See Free-Flow Speed.

11.89. Staged Signalised Crossing

A system by which a long signalised crossing is divided or "staged" into several time-separated sections, each being a separate group controlled by individual signals.

11.90. Start Loss

Duration of the interval between the start of the displayed green period and the start of the effective green period for a movement. This is used in signal timing and performance analysis to allow for queue discharge time losses at the start of green period due to vehicles accelerating to saturation speed, or due to giving way to opposing vehicle or pedestrian movements.

11.91. Stop-Line Delay

Delay determined by projecting vehicle time-distance trajectories from the approach and exit negotiation speeds to the stop line (or give-way / yield line), which includes the Queuing Delay and the deceleration and acceleration delay associated with the negotiation speeds.

11.92. Stopline Flow Rate

Departure flow rate measured at the stop line (or give-way / yield line), which is the same as the demand (arrival) flow rate for undersaturated cases, and is limited to the capacity rate for oversaturated cases. See Demand Flow.

11.93. Stop Rate

Average number of all acceleration-deceleration manoeuvres including the queue move-ups, partial stops and geometric stops, expressed in terms of equivalent Major Stops.

11.94. T-Intersection

An intersection where two roads meet (whether or not at right angles) and one of the roads ends.

11.95. Total Delay

Sum of delay experienced by all vehicles or pedestrians (vehicle-hours per hour or pedestrian-hours per hour). Obtained as the product of average delay per vehicle or pedestrian and the flow rate.

11.96. Total Travel Distance

Sum of distance travelled by all vehicles (vehicle-kilometres per hour or vehicle-miles per hour). Obtained as the product of travel distance per vehicle and the flow rate.

11.97. Traffic-Actuated Control

A control method that allows a variable sequence and variable duration of signal displays depending on vehicle and pedestrian traffic demands.

11.98. Traffic Delay

Delay that results when the interactions between vehicles cause drivers to reduce speed below the free-flow (desired) speed. See Free-Flow Speed.

11.99. Traffic Volume

The number of vehicles or pedestrians passing a given point on a lane or carriageway during a specified period of time.

11.100. Uninterrupted Flow

A condition in which vehicles travelling in a traffic stream do not have to stop or slow down for reasons other than those caused by the presence of other vehicles in that stream.

11.101. Unopposed Turn

A left-turn or right-turn movement at a signalised intersection that is made with no opposing or conflicting vehicular or pedestrian flow allowed.

11.102. Upstream

In the direction opposite to the movement of traffic.

11.103. Walk Time

Duration of the Walk display (steady green person) for pedestrians.

11.104. Yellow Time

Duration of the yellow display for a phase or a movement.