University of Southern Queensland

The Application of Variable Speed Limits to Arterial Roads for Improved Traffic Flow

A dissertation submitted by

Hamdi Abdulkareem Mohammed Al-Nuaimi

B.Sc. Eng., M.Sc. Eng.

For the award of

Doctor of Philosophy

2014

ABSTRACT

Traffic congestion problems continue to increase in large cities due to rapidly increasing travel demand and a lack of transport infrastructure. Congestion causes mobility and efficiency loss, safety reduction, increased fuel consumption and excessive air pollution. A number of traffic management strategies have been proposed and some are applied in cities, such as diverting traffic from peak periods to off-peak periods using congestion pricing, reduced speed limits, coordinated traffic signals along major arterial roads, or adding additional lanes where network expansion is feasible. Among the many solutions to traffic congestion, operational treatments for existing road networks provide more cost efficient traffic operation due to their relatively low cost. This research looks to improve efficiency through the application of Variable Speed Limits (VSLs). While VSLs have been used to improve traffic conditions on congested motorways in terms of mobility, safety and travel time, they are largely untested on signalized urban arterial roads.

Griffith Arterial Road (GAR) U20 was selected as the case study for the research. GAR is part of the Brisbane Urban Corridor (BUC), and is approximately 11.5 km long and lies between the Gateway Motorway and the Ipswich Motorway. The average daily traffic volume (ADT) is between 18,000 vehicles to 24,000 vehicles. The number of lanes at approaches to signalised intersection varies from 1 to 4.

In the context of this research, the study used STREAMS data and real world data collected using six high definition (HD) video cameras to develop a VISSIM model and to discern the effectiveness of applying VSL control. VISSIM is a time step and a psycho-physical car following model developed to model urban traffic and public transit operations. The VISSIM model was extensively calibrated and validated with the empirical data collected regarding measure of effectiveness such as traffic volumes, volume distribution, and saturated headway along the west bound (WB) and eastbound (EB) directions. The simulated model allowed the testing of different control strategies for VSL and Integrated Traffic Control System (ITCS) under different scenarios and circumstances. It helped to contrast the traffic flow parameters of invariant (no controlled speed) and VSL (controlled speed) conditions. Multiple simulation runs were considered in the calibration and evaluation process.

The measures of effectiveness used to characterise the operational quality of signalized intersections were delay, queue length, and number of stops. In addition, flow, speed and density parameters were used to characterise the changes in traffic performance for the arterial road.

This thesis investigates the application of VSLs for control of upstream traffic as a proposed traffic control strategy on the GAR. The objective was to investigate how dynamic VSL and signal control systems could be used in an integrated approach to traffic management to improve the traffic efficiency, safety, and mobility of a congested urban arterial road. The research indicates that the application of VSL could improve the traffic performance and safety during the peak period. It helped to maintain a planned continuous flow through coordinated intersections to avoid congestion. Integrating VSL with other traffic congestion management (changing the signal timings for the congested traffic) appeared effective in improving traffic conditions and reducing total travel time on the GAR. The research highlighted some important elements that could be used for the design and implementation of VSL systems using intelligent transport systems.

Certification of Dissertation

I certify that the thoughts, experimental work, numerical outcomes and conclusions reported in this dissertation are entirely my own efforts, except where otherwise acknowledged. To the best of my knowledge, I also certify that the work presented in this thesis is original, except where due references are made.

……………………………………. / / 2014

Signature of Candidature Date
Hamdi Abdulkareem Mohammed

ENDORSEMENT

…………………………………… / / 2014

Signature of Supervisor Date
Prof. Ron Ayers

…………………………………... / / 2014

Signature of Supervisor Date
Prof. Frank Bullen

……………………………………. / / 2014

Signature of Supervisor Date
Dr. Kathirgamalingam Somasundaraswaran

Acknowledgments

I would like to express my sincere gratefulness to Prof. Ron Ayers, my principal supervisor, for his guidance, encouragement, persistence, patience and expert advice. I would also like to thank my associated supervisor Prof. Frank Bullen for his expert advice and valuable input to my research. It is my luck and great honour to be their student. Without their guidance, I would not be able to complete this research. Also, I am very thankful to my associated supervisor Dr. Kathirgamalingam, who provided me with invaluable help and support.

I owe my deepest gratitude to the spirit of my father, who always in my heart and dreams and to my mother, who without her pray, I wouldn't reach this stage.

My admiration and respect go to my wonderful wife, Enas, who without her support and encouragement I could not be able to finish my study. My thanks go to my smart son, Diyar, to my beautiful daughter, Dima, to my little gorgeous daughters Diane and Dania for their encouragement, patience and endless love during my PhD study.

Many thanks go to my brothers, sisters and relatives for their inspiration and pray. Great thanks and attitudes go to my brother, Husham Al-Nuaimi, for his infinite support and encouragement during all the period of my life.

I would like to extend my thanks to the University of Southern Queensland (USQ) and special thanks to Faculty of Civil Engineering and Surveying for their support throughout my academic studies. I would also like to acknowledge all the staff members of PTV vision in Brisbane and especially Dr. Julian Laufer and Dr. Mamun Rahman for their cooperation and support during the course of this research. Furthermore, great thanks go to the Department of Transport and Main Roads in Queensland for their help in providing and collecting traffic data for the Griffith Arterial Road.

I wish to express my sincerest gratitude to my sponsor Iraqi Government, and special thanks go to the Iraqi Cultural Attaché in Canberra for providing the funding required fulfilling this thesis during the period of the research. I wish to thank all Iraqi friends and colleagues I have met, who have helped to provide encouragement, guidance or made my life in Toowoomba an enjoyable experience.

Associated Publication

Al-Nuaimi, H, Ayers, R & Somasundaraswaran, K (2013), 'Modelling interrupted flow conditions on an arterial road using VISSIM software', proceeding paper presented to 14th Road Engineering Association of Asia and Australasia Conference (REAAA 2013): The Road Factor in Economic Transformation, 26-28 Mar 2013, Kuala Lumpur, Malaysia.

Al-Nuaimi, H, Ayers, R & Somasundaraswaran, K (2012), 'The effectiveness of using variable speed limit on the performance of an interrupted flow', proceeding paper presented to IASTED International Conference on Engineering and Applied Science (EAS 2012): Applications for the 21st Century, 27-29 Dec 2012, Colombo, Sri Lanka.

Table of Content

List of Figures

List of Tables

List of Abbreviations

ABS	Australian Bureau of Statistics
AD	Australian Dollar
AIMSUN	Advanced Interactive Microscopic Simulation for Urban and Non-urban Networks
ALINEA	Asservissement Lineaire d'Entree Autoroutiere
AMOC	Advanced Motorway Optimal Control
ARE	Absolute Relative Error
ATC	Area Traffic Control
BMA	Brisbane Metropolitan Area
BTRE	Bureau of Transport and Regional Economics
CORSIM	CORridor SIMulation
DTMR	Department of Transport and Main Roads
EMME	Equilibre Multimodal, Multimodal Equilibrium
EP	Evening Period
FMS	Freeway Management Systems
GAR	Griffith Arterial Road
GEH	Geoffrey E. Havers
HCM	Highway capacity manual
HDC	High Definition Cameras
HV	Heavy Vehicle
ITCS	Integrated traffic control system
LOS	Level of Service
METANET	Modèle d'Ecoulement du Trafic Autoroutier: NETwork
MFD	Macroscopic Fundamental Diagram
MOE	Measures of Effectiveness
MP	Morning Period
MPC	Model Predictive Control
MTFC	Mainstream Traffic Flow Control
PARAMICS	PARAllel MICroscopic Simulation
QDTMR	Queensland Department of Transport and Main Roads QDTMR
QLD	Queensland
RLC	Red-Light Camera
RLR	Red-Light Running
SATURN	Simulation and Assignment of Traffic to Urban Road Networks
SCL	Speed Control Limit
STREAMS	Synergised Transport Resources Ensuring an Advance Management System
TDM	Travel Demand Management
TFL	Transport for London
TH, RT, LT	Through, Right turn, Left turn
TRANSYT	TRAffic Network StudY Tool
TRB	Transportation Research Board
TTT	Total Travel Time
TU, WE, THU	Tuesday, Wednesday, Thursday
VISSIM	Verkehr In Städten-SIMulationsmodel
VMS	Variable Message Sign
VSL	Variable Speed Limit
VSMs	Variable Speed Message signs

CHAPTER ONE

Introduction

1.1 Background

Road traffic congestion is considered one of the most significant transportation problems throughout the world due to its impact on both the environment and economy. It typically occurs when traffic demand is close to or exceeds the available volume capacity of the road network. Solving traffic congestion is complex as traffic demand is not constant but varies according to season, day of the week, and the time of day. In addition, capacity is not constant, as it can reduce due to inclement weather, the presence of work zones, traffic incidents or other events. Increasing the demand or decreasing the capacity of a road system may lead to problems for both vehicle users and society. This congestion phenomenon commonly emerges in major cities on heavily trafficked roads. Congestion has a negative impact on the efficiency of the traffic system, safety, the environment (air contamination, fuel consumption), and the quality of life (health problems, noise, frustration), (Bellemans et al., 2006, Hongfeng et al., 2008).

Metropolitan vehicle travel in Australia is expected to continue to grow appreciably over the next decade and a half. Australian Bureau of Statistics (ABS) (2013) estimated there were 16.6 million vehicles registered in Australia in 2012 and that passenger vehicles made up 76.4% of all registered vehicles. Motor vehicles registered in Australia travelled an average of 14,000 kilometres per vehicle in 2012.

Bureau of Transport and Regional Economics (BTRE) (2007) estimated that in 2005 the total social costs of congestion in Australian capital cities was $9.4 billion. This total was comprised of approximately $3.5 billion in private time (losses from trip delay and travel time variability), $3.6 billion in business time costs (trip delay plus variability), $1.2 billion in extra vehicle operating, and $1.1 billion in extra air pollution damage costs. In the absence of improved congestion management, BTRE (2007) estimated that the social costs of congestion would be increased to $20.4 billion by 2020.

This thesis investigates the possibility of using variable speed limits (VSLs) for control of upstream traffic as a proposed traffic control strategy on the Griffith Arterial Road (GAR) in Brisbane (Figure 3.2). The research investigates the contribution of VSL to managing and operating existing transportation infrastructure for the purpose of increasing efficiency, safety, and mobility, and improving the environment by minimising delay, queue length, and number of stops, and increasing flow and speed.

1.2 Objectives of the study

The aim of the research was to investigate how dynamic variable speed limits (VSL) and signal control systems could be used in an integrated approach to traffic management to improve the traffic efficiency, safety, and mobility of a congested urban road network.

The study focused on the use of control impact of VSL on an urban arterial network using a micro simulation platform. The optimum traffic control management was determined by integrating the VSL with other traffic congestion management strategies. Selecting the optimum traffic management strategies was based on the consideration of several key parameters such as average delay, average stopped delay, average queue, and number of stops.

Using the GAR as a case study, the main components of the research are:

1. To evaluate the real traffic behaviour on an urban arterial road, and develop a VISSIM (Verkehr in Städten-Simulations) model based on data collected from real traffic,
2. To investigate the effect of using VSL controls on the level of service (LOS) at critical signalized intersections,
3. To explore the effect of using VSL controls on the performance of congested arterial roads in terms of speed, flow, and density,
4. To investigate the influence of VSL controls on the traffic safety during the congestion period,
5. To investigate the potential of integrating VSL control with other traffic congestion management (increase the signal timings of critical intersection), and
6. To optimise and identify the appropriate traffic control using VSL and Integrated traffic control system (ITCS).

1.3 Significance of the study

There are many definitions of an integrated traffic management system but in general, they refer to the use of distributed computer architecture where intersection control is utilised to optimise flow through an adaptive system.
This research moves to the next level through the application of VSL integrated with an existing signalised intersection control within a link. This is depicted schematically in Figure 1.1.

Most past research has been focused on using VSL applications on motorways for improving the traffic efficiency and safety. Exploring the effectiveness of VSL control on a signalised arterial road network has the potential to provide a new traffic management technique for busy arterial roads. It is considered that the main benefits of applying VSL to an urban arterial road will be to:

1. Enhance the performance of existing transportation infrastructure and reduce the need to construct new roads or add lanes,
2. Provide cost effective options to the mobility problems by low-cost solutions,
3. Improve the traffic efficiency, safety , and mobility for urban arterial roads, and
4. Provide an environmental friendly approach to reduce vehicle total travel time and consequently reduce overall fuel consumption and air pollution.

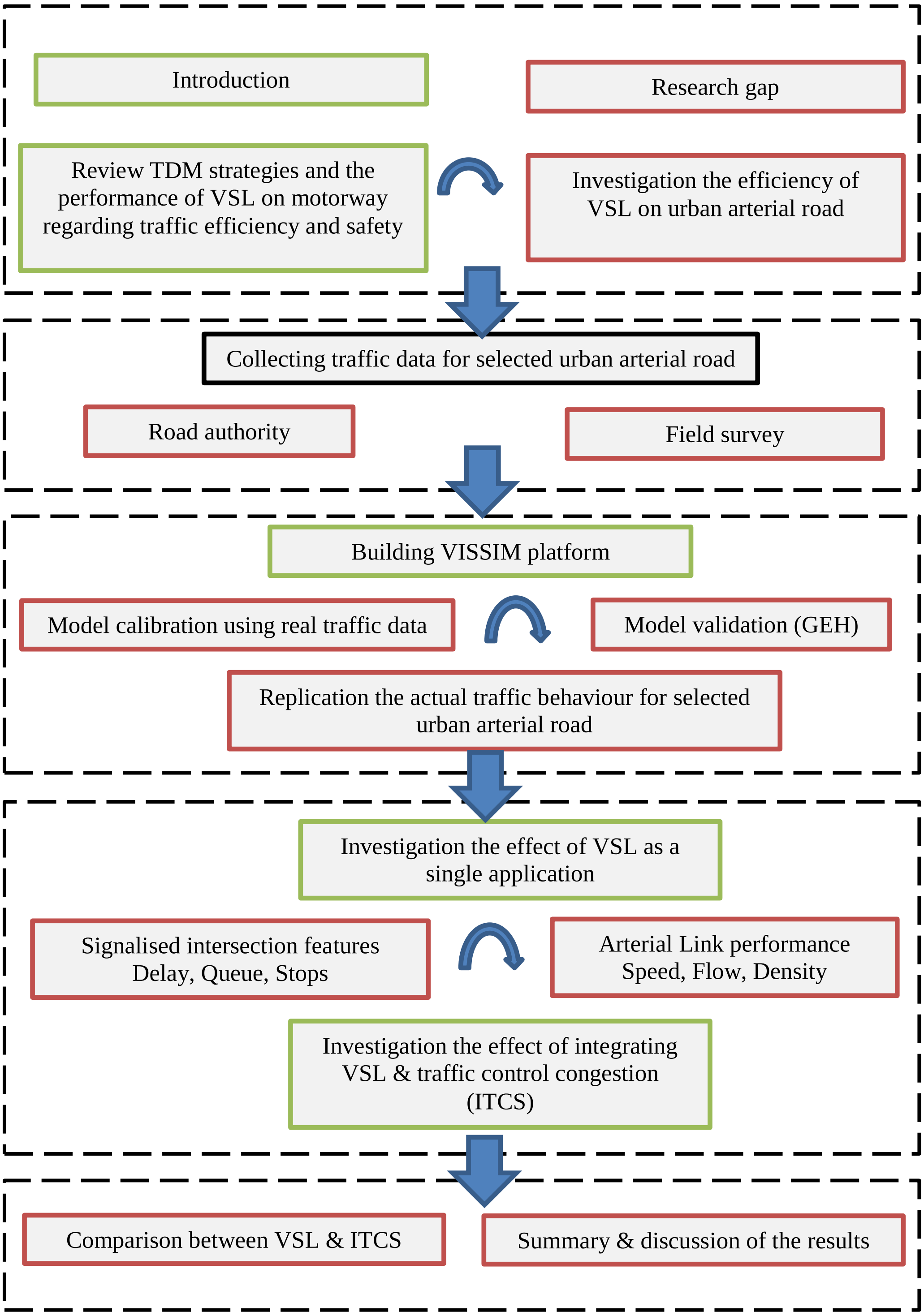


Figure 1.1 Schematic illustration showing the scope of the current research

1.4 Study methodology

The study methodology adopted may be depicted schematically as shown in Figure 1.2.

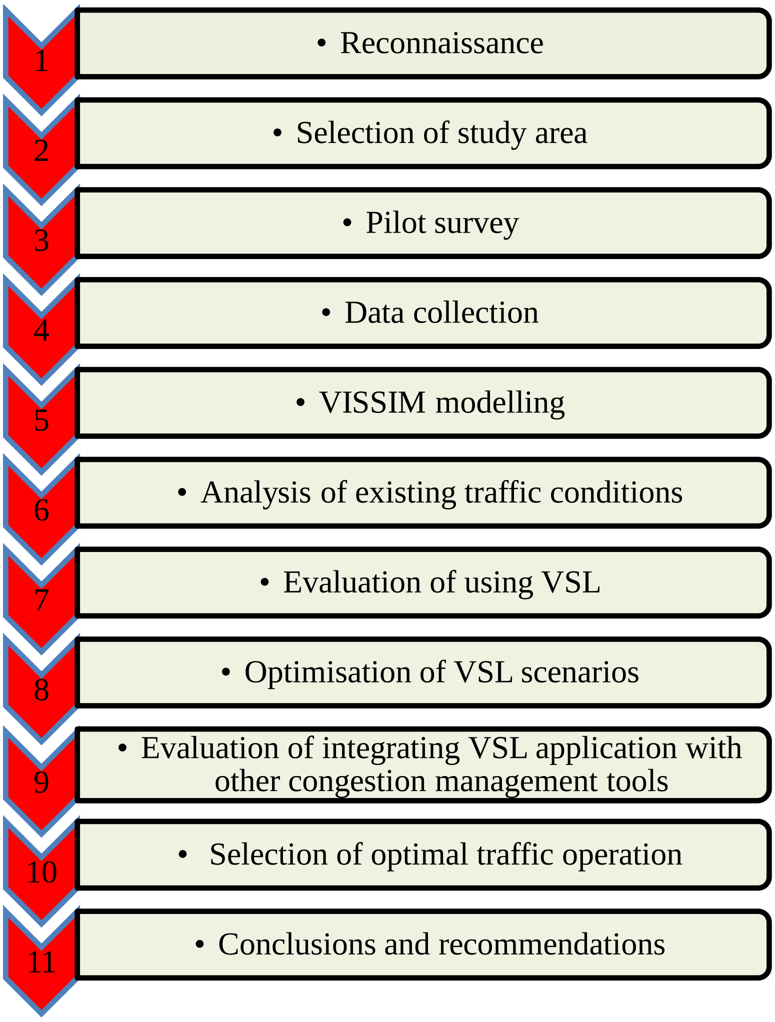


Figure 1.2 Study methodology

1.5 Thesis layout

The thesis is presented in a series of integrated chapters. Chapter two reviews traffic control management strategies used to alleviate or mitigate traffic congestion in urban areas and explore the effect of an Intelligent Transport System (ITS) on traffic performance of an interrupted flow. Chapter three provides details of the methods that were used to collect field data. It also provides the reasons for selecting the Griffith Arterial Road (GAR) in Brisbane as the case study area for the present research. Chapter four provides an overview of VISSIM software, its use in traffic modelling and describes the process of calibration and validation of the software. Chapter five describes the local traffic situation for GAR under base line conditions. Chapter six investigates the effect of using (only) VSL control on the performance of GAR. Chapter seven investigates the improved efficiency obtained by integrating VSL control with changes in the signal timings of congested intersections. Chapter eight presents the conclusions and discusses possible extensions of the research.

CHAPTER TWO

Literature Review

2.1 Introduction

Various traffic management strategies are frequently applied to deal with traffic in busy arterial roads in urban areas. Despite these efforts, traffic congestion continues to increase on arterial roads, particularly during peak periods. Traffic bottlenecks at intersections are the main cause of congestion. Traffic congestion is a source of immobility and efficiency loss, safety reduction, fuel consumption, excessive air pollution, health problems, noise and frustration.

A number of traffic management strategies have been proposed for alleviating cities' traffic congestions such as diverting traffic from peak-period to off-peak period, using congestion pricing, constraint flow, reducing speed limits, coordinating traffic lights along major arterials, actuated traffic signals, and adding additional lanes where expanding the road network is feasible.

As a result of the continuous increase of traffic in central urban areas, traffic congestion and delays are often experienced in the vicinity of intersections. This negatively affects the arterial road network (Garber and Hoel, 2009). Under non-saturated flow conditions (normal), traffic signals are designed to provide safe and efficient traffic operation by reducing vehicle delays and the number of stops. By contrast, in congested conditions, traffic queues and delays propagate from cycle to cycle either due to insufficient green signal time allocations or because of blockages. Excessive queues and spillbacks towards upstream flow may lead to gridlock in the network resulting in serious degradation of the traffic system.

The most common solution for improving congestion at intersections is to manage the traffic conflicts either by providing a fixed priority to some critical movements or by alternating priorities by means of the traffic signal. Constructing or expanding a road infrastructure is the obvious solution but often constrained due to the capital finance, land use, time of construction, and the well-known phenomenon of induced demand (Hansen and Huang, 1997, Mogridge, 1997, Parthasarathi et al., 2003). Accordingly, reducing congestion in urban areas and improving mobility needs to focus on better utilization of existing infrastructure (Geroliminis, 2007, Hadiuzzaman et al., 2012). Maximising the system throughput in a congested area has been a high priority for many researchers. Dai et al. (2013) reported that improving the arterial road traffic efficiency and easing traffic congestion has become the subject of urgent research. Appropriate traffic management requires a well-developed and efficient road control system and the application of intelligent transportation systems. Dynamic traffic control is one traffic solution that can be applied for recurrent and non-recurrent congestion. It is a control method that caters for variations of the traffic over time. Many dynamic traffic control strategies such as road pricing, dynamic pricing, road space rationing, metering flow and variable speed limit (VSL) have been proposed, aimed to improve the efficiency of existing road network. Dynamic traffic control strategies, such as VSL control has been widely used for motorway control for increasing traffic operation efficiency and safety. However, to date VSL control has not been used on urban arterial roads for managing adverse traffic conditions.

Variable speed limits are commonly used to regulate traffic flow on motorways by using variable speed message signs (VSMs). The original aim of VSMs was to improve traffic safety by increasing driver compliance. This led to decreasing rear-end collisions as reported by Zackor (1979) and Coleman et al. (1996). Smulders (1992) and Harbord (1995) investigated the use of speed limits and explained how speed limits had a homogenising effect on traffic flow during the peak periods. Chien et al. (1997) and Lenz et al. (2001) focused on how variable speed limits could reduce inbound traffic at bottlenecks, which avoided or alleviated traffic congestion. Kohler (1974) found that when the headways in a chain of vehicles were below a specific threshold, the chain became unstable.

Inhomogeneity in traffic flow can contribute to small perturbations in the flow which might then cause congestion to occur. Inhomogeneity in traffic can arise due to variations in speed between consecutive vehicles in one lane or different lanes, or flow differences among the lanes.

VSL strategies can create a more uniform distribution of traffic density over freeway links preventing the high traffic density that leads to breakdowns in traffic flow. Alessandri et al. (2002) reported that VSL controls were able to prevent congestion and improve flow using segment throughput as a measure of effectiveness, but that it had little impact on the reduction of total travel time (TTT) in the network. Hegyi et al. (2005) evaluated the impact of using VSL controls on the total travel time (TTT) as a measure of effectiveness using a hypothetical network. Their results showed a 21% saving in TTT could be achieved when using a VSL control strategy. Kejun et al. (2008) investigated the effect of VSL on a hypothetical 5 km work zone model. The study did not find any significant improvement in TTT. Lee et al. (2006) reported that VSL controls used in highly congested locations, reduced the potential for crashes and increased the safety by 25%, but that they also increased TTT. This finding was supported by Allaby et al. (2007). In contrast, Abdel-Aty et al. (2008) found that VSLs achieved a significant reduction in crash potential during non-congested periods, but had no significant influence during the congested periods.

VSL controls are widely used in European countries and the United States. The key difference in the use of VSLs in those areas is enforcement, where within Europe automated speed enforcement was used to achieve high driver compliance rates in European countries. For example, Transport For London (TFL) (2004) reported a 9% reduction in flow breakdown (escalating vehicles speed) and a 6% reduction in stop-go driving conditions (reducing number of vehicle stops). The VSL applications resulted in traffic headways becoming more uniformly distributed within the narrow range of 0.8-1.5 seconds.

Papageorgiou et al. (2008) conducted an empirical evaluation of using VSL strategies on Motorway 42 in the UK and found no clear evidence of improved traffic flow. Carlson et al. (2010) evaluated VSL performance using AMOC (Advanced Motorway Optimal Control) macroscopic software tool. The proposed VSL control in the Amsterdam ring road A10, which is about 32 km long, resulted in a 47% reduction in vehicle travel time. Jonkers et al. (2011) reported that traffic safety had been improved substantially and it was possible to resolve traffic shock waves by using lower speed limits. In addition, the researchers asserted that not all shock waves could be resolved by applying a lower speed limit. Hadiuzzaman et al. (2012) developed an analytical model to represent drivers' response to updated speed limits and macroscopic speed dynamical change with respect to changeable speed limits. The study revealed that VSL controls were most effective during periods of congestion. Specifically, they found improvements in total travel time, total travel distance, and total flow around 39%, 8%, and 5.5%, respectively.

In summary, inconsistent outcomes have been achieved by the application of VSL. It should be noted that, almost all VSL evaluation studies have been concentrated on a small number of links in the corridor. Moreover, those studies do not take into consideration the effect of speed limit on the non-congested links in the evaluation process. Most previous studies indicated that VSL applications were capable of improving the travel time, but that it had little impact on the flow. The studies have also showed consistent safety improvement by reducing speed variance or improving speed homogenization.

This chapter presents a review of the important measures used to characterise the operational quality of signalised intersections in terms of delay, queue length and number of vehicle stops. It also presents a brief description of some of the traffic control management strategies used to improve the performance of traffic flow in urban cities. Given the importance of VSL in the research, the theory underpinning the application of VSL in an interrupted flow condition is outlined.

2.2 The performance of signalised intersections

Traffic signals at intersections represent point (node) locations within urban arterial road networks. At these point locations, the measures of operational quality or effectiveness based on traffic speed and density are not relevant. Speed has no meaning at a point, and density requires a section of some length for measurement. The three measures of effectiveness commonly used to characterise the operational quality of signalized intersections are delay, queue length, and number of stops. The most common parameter used to describe traffic performance at a signalised intersection is delay, with queue length and/or number of stops often used as a secondary measure (Roess et al., 2004).

2.2.1 Delay

Delay at signalized intersections is associated with the time lost to a vehicle and/or driver due to the presence of traffic signals, geometric design of the road and traffic conditions (Darma et al., 2005). Delay is also defined as the difference in travel time between when a vehicle is affected and unaffected by a controlled intersection under ideal conditions. Ideal condition is when there is an absence of geometric delay, no incidents, and when there are no other vehicles on the road Transportation Research Board (TRB) (2000).

Several different parameters can be used to measure delay at an intersection. Each parameter has a different purpose for transportation engineers. Control delay is the portion of the total delay caused by a control device, either a traffic signal or a stop sign. It is comprised of initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay (TRB, 2000). In the earlier versions of the Highway Capacity Manual (Transportation Research Board (TRB), 1985), the control delay included only the stopped time delay. Olszewski (1993) reported that control delay time could be classified into three types, deceleration delay, stopped delay and acceleration delay. Typically, stopped delay can be determined easily being when the vehicle is fully immobilised (vehicle speed equals zero) however some traffic regulations define stopped delay to include when the vehicle moves less than the average pedestrian speed (1.2 m/s), for example in the Canadian model

(Teply et al., 1995). While stopped delay is easier to measure, overall delay (control delay) provides a better measure of the operational quality of a signalised intersection. The delay caused by a decelerating or accelerating vehicle is categorized as deceleration or acceleration delay. Several delay types used in the HCM at a signalized intersection for a single vehicle approaching a red signal are shown in Figure 2.1, (TRB, 2000).

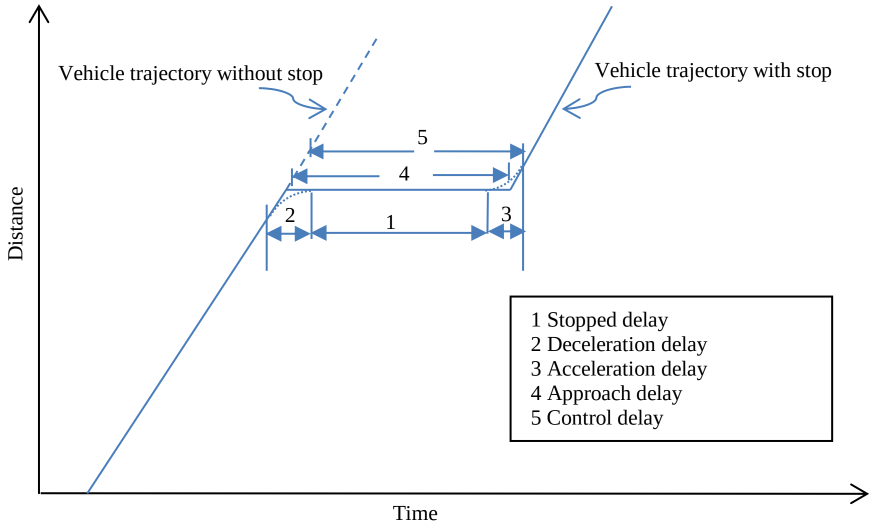


Figure 2.1 Delay components, Source: (TRB, 2000)

In the figure, stopped delay for the vehicle includes only the time spent stopped at the signal. It starts when the vehicle is fully immobilised and terminates when the vehicle starts to accelerate. Approach delay comprises an additional lost time due to the deceleration and acceleration. It is set up by extending the speed slope of the approaching vehicle as if no signal existed and the departure slope after full speed is achieved. Then the approach delay is the horizontal (time) difference between the hypothetical and departure speed slope. Time in queue delay cannot be effectively shown using one vehicle because it involves joining and departing a queue of several vehicles.
In TRB (2000), the average delay per vehicle for a lane group is computed as follow:

$$d = d_1 . f_{PF} + d_2 + d_3 . f_r \qquad 2.1$$

with

$$d_1 = 0.5C \frac{\left(1-\frac{g}{C}\right)^2}{\left[1-\min(X,1.0).\frac{C}{g}\right]} \qquad 2.2$$

$$d_2 = 900T\left[(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}\right] \qquad 2.3$$

$$f_{PF} = \frac{(1-P)f_P}{1-\frac{g}{C}} \qquad 2.4$$

where,

d = average overall delay per vehicle (seconds/vehicles),

d_1 = uniform delay (seconds/vehicles),
d_2 = incremental, or random, delay (seconds/vehicles),
d_3 = residual demand delay to account for over-saturation queues that may have existed before the analysis period (seconds/vehicles),
PF = adjustment factor for the effect of the quality of progression in coordinated systems,
C = traffic signal cycle time (seconds),
g = effective green time for lane group (seconds),
X = volume to capacity ratio of lane group,
c = capacity of lane group (vehicles/hour),
k = incremental delay factor dependent on signal controller setting (0.50 for pretimed signals; vary between 0.04 to 0.50 for actuated controllers),
I = upstream filtering/metering adjustment factor (1.0 for an isolated intersection),
T = evaluation time (hours),
P = proportion of vehicles arriving during the green interval,
fp = progression adjustment factor.

In this delay model the residual delay components d_3 make use of vehicles instead of passenger car units to quantify traffic flows. The period analysis T is reported in hours instead of minutes, but this change is reflected in the use of a different multiplication factor in each term involving the variable T. In Equation 2.3, parameters k and I are introduced in the last term of the equation, and this term reduces to 0.5 and 1.0 when the values associated with pre-timed traffic signal control and an isolated intersection respectively.
Based on the delay formula in TRB (2000), VISSIM software is used to compute the total delay every deci-second for every vehicle completing the travel time section by subtracting the theoretical (ideal) travel time from the real travel time. The theoretical travel time is the time that would be reached if there were no other vehicles and no signal controls or other stops in the network. The total delay is computed as the summation of all instantaneous delays along a link, for an entire trip, and for an entire network, (Van Aerde and Rakha, 2007).

$$D = \sum_{i=1}^{N} d_i = \sum_{i=1}^{N} \left[1 - \frac{u(t+i\Delta t)}{u_f}\right] . \Delta t \qquad 2.5$$

where,
D = the total delay incurred over entire trip,
d_i = the delay incurred during interval i,
Δt = the duration of interval,
$u(t+i\Delta t)$ = the vehicle instantaneous speed in interval i,
u_f = the expected free-flow speed of the facility on which the vehicle is traveling, and N = the number of time intervals in a speed profile.

Another delay type experienced by vehicles at signalized intersections is geometric delay. This delay is caused by the vehicle having to reduce its speed due to geometric features, such as when an arterial road makes a sharp turn, causing vehicles to slow, or the indirect route that a through vehicle must take through a roundabout (Transportation Research Board (TRB), 2010). Luttinen and Nevala (2002) defined the geometric delay as the time losses due to the intersection geometry. It may be large for large turning vehicles due to slow manoeuvre required to maintain stability. Secondly, incident delay may cause an additional time lose due to an incident condition. Thirdly, traffic delay may be caused by the interaction of vehicles, as a result of drivers having to reduce their speed below the free flow speed due to traffic conditions. Total delay of a vehicle is the sum of control, geometric, incident, and traffic delay. Driver perception and reaction time variations to the changes of traffic signal display at the start of green interval and during yellow interval to mechanical constraints and to individual driver behaviour also contribute to the traffic delay at signalized intersection. Most delay incurred at signalised intersections is due to the traffic signal operation itself while only a fraction of the entire delay is attributed to the time required by the driver to react to the traffic signal changes which depends on the driver behaviour.

Average control delay is used as the basis for determining the level of service (LOS) for a signalised intersection (TRB, 2000). Delay minimization is frequently used as a primary optimization criterion when determining the operating parameters of traffic signals at both isolated and coordinated signalized intersections. Intersection control delay is generally computed as a weighted average of the average control delay for all lane groups based on the volume within each lane group. Caution should be exercised when evaluating an intersection based on a single value of control delay because this is likely to over represent or under-represent operations for individual lane groups. Table 2.1 (TRB, 2000) provides the criteria of LOS levels for a signalised intersection. Six levels of service are assigned the letters A through F. Each LOS represents a range of operating conditions, with LOS A representing the best operating conditions and LOS F the worst. A higher delay experienced by vehicles at signalised intersection is represented by a low LOS.

Table 2.1 Motor vehicle LOS thresholds at signalized intersections

LOS	Control Delay per vehicle (seconds per vehicle)
A	≤ 10
B	> 10-20
C	> 20-35
D	> 35-55
E	> 55-80
F	> 80

Source: (TRB, 2010)

2.2.2 Vehicle queuing

Queue length is an important operational measure and a design consideration that should be evaluated as part of all analyses of signalized intersections. It reflects how far traffic backs up due to the presence of a traffic signal or a vehicle stopped in the travel lane while waiting to make a turn. Operational problems can be created if queues developed are longer than the designed storage length. Therefore, estimates of vehicle queues are required to determine the amount of storage needed for turn lanes and to determine whether spill over will occur at upstream facilities (such as driveways, signalized or un-signalized intersections).

Deterministic queuing analysis is one of the most commonly utilized approaches for estimating queue lengths at the macroscopic level because it is relatively simple. In this analysis, vehicles are assumed to queue vertically at the intersection stop line for under-saturated and oversaturated conditions. When traffic demand is under saturated, the estimates of maximum queue length at signalized intersections can be computed using the following equation (TRB, 2000).

$$Q_{max} = \frac{s}{s-q}.rq \qquad 2.6$$

where,
r = red interval (seconds),
s = saturation flow rate (vehicles/second),
q = arrival flow rate (vehicles/second).

Alternatively, when traffic demand is oversaturated, a residual queue remains at the end of the green interval. The difference between the arrival flow rate and the capacity of a signalized intersection gives the residual queue of vehicles at the end of a cycle. When the residual vehicles of a first cycle are known, the residual vehicles of the last cycle can be determined as the residual vehicles of the first cycle multiplied by the number of cycles (N). From the residual queue length at the end of each cycle an approximate estimate of the maximum queue length for each cycle at an oversaturated fixed-time signalized approach is calculated as indicated in the following equations (TRB, 2000).

$$Q_{res} = N[qr - (s-q)g] \qquad 2.7$$
$$Q_{max} = Nqr - [(N-1)(s-q)g] \qquad 2.8$$

where,
Q_{res} = residual queue length of end of each cycle (vehicles),
N = number of cycle,
q = arrival flow rate (vehicles/second),
r = red interval (seconds),
g = green interval (seconds),
s = saturation flow rate (vehicles/second), and
Q_{max} = approximate estimate of maximum queue length during a cycle (vehicles).

Shock wave theory can also be used to estimate queue lengths at signalized intersections. The main difference between shock wave and queuing analysis models is in the way vehicles are assumed to queue at the intersection stop line. Shock wave considers that vehicles are queued horizontally one behind each other. This means that each vehicle occupies a physical space and allows the analysis to capture more realistic queuing behaviour and to more realistically compute the extent of the queue. The maximum extent of queue and maximum number of queued vehicles caused by the fixed-time signalized intersection is shown as following respectively (TRB, 2000).

$$Q_{max} = \frac{qsr}{s(k_j - k_a) - q(k_d - k_j)} \qquad 2.9$$
$$N_Q = x_m . k_j \qquad 2.10$$

where,
q = arrival rate (vehicles/second),
s = saturation flow rate (vehicles/second),

r = red interval (seconds),
k_d = density of discharge flow (vehicles/kilometre),
k_j = jam density (vehicles/kilometre),
k_a = density of approach flow (vehicles/kilometre),
x_m = distance of maximum extent queue (kilometres), and
N_Q = number of vehicles in maximum queue (vehicles).

In the case of the VISSIM software, the user defines a queue according to the maximum vehicle speed for the beginning of the queue (default is 5 km/hr), the minimum vehicle speed at its end (default is 10 km/hr), and the maximum spacing between vehicles (default is 20 metres). It allows the user to identify a queue counter location. The distance to the farthest upstream point of any queue at this location is calculated. If the queue backs up onto multiple links, the longest distance is recorded. If the front of the queue begins to discharge, VISSIM keeps tracking to the back of the queue until no queued vehicles remain between the queue counter location and the current back of queue (Dowling, 2007). It is not the average of the back of queues when the signals turn from red to green that is calculated but the average queue of each time step, irrespective of signal head colour as indicated in the following equation.

$$Q_{avg} = \frac{\sum_i Q(i)}{I} \qquad 2.11$$

where,
Q_{avg} = average back of queue over analysis period (meters),
$Q(i)$ = observed back of queue length (meters) at end of time step (i) and
I = total number of time steps in analysis period.

Approaches to signals that experience extensive queues are likely to also experience more rear-end collisions (Rodegerdts et al., 2004). Wiles et al. (2005) stated that queue growth for a particular intersection is more likely to affect arrival drivers than departing drivers. Additionally, in many instances, multiple lanes were found to be impacted due to driver behaviour seeking access to network. Although urban drivers are generally aware of traffic conditions that are likely to be encountered in their daily travels, they still can be surprised by abrupt and extensive queues. Drivers unfamiliar with conditions might not be alert and be involved with or cause collisions. All drivers are particularly vulnerable when poor road geometric conditions coincide with queue build-up.

Yan et al. (2005) and Sugiyama and Nagatani (2012) indicated that rear-end crashes are the most common accident type at signalized intersections. These crashes result from a combination of lead-vehicle deceleration and inadequate deceleration by the following vehicle. Driving during the peak periods results in smaller headways between leading and following vehicles, which increases the possibility of rear-end crashes. Khattak (2001) reported that a majority of the crashes (54.9%) occurred during the peak hours of 7:00-9:00 a.m. and 3:00-6:00 p.m.

Wu et al. (2013) reported that speed limit reductions in conjunction with signal-warning flashers appeared to be an effective safety measure when the speed-limit reduction indicated to the driver is at least 16 km/h. Yang et al. (2013) discussed the positive effect of using automated enforcement of red-light cameras (RLC) as a measure of curbing red-light running (RLR) at signalized intersections with the aim of enhancing traffic safety. Stevanovic et al. (2013) focused on optimizing signal timings to increase surrogate measures of safety and thereby reduce risks of potential rear-end crashes. The estimated number of collisions at traffic signals was reduced by 9%. A strong relationship between increase in cycle length and reduction of vehicular conflicts was observed by the researchers.

2.2.3 Number of stops

The third important factor for evaluating the performance of traffic at signalised intersections is the number of vehicle stops in the road segment approaching the signalised intersection. This factor reflects how frequently drivers have to stop while traveling along a roadway because of traffic signals, turning vehicles, pedestrian area, and similar factors. Speed thresholds were often used to determine when a vehicle is stopped. The only non-arbitrary speed threshold for this purpose is zero. Practical considerations propose that simulation modelling that deals with stopping would be more stable if a near-zero speed were used instead. Some simulation modelling was applied (8 km/h) for determining number of stops when a vehicle has "stopped".

The accumulation of multiple stops poses more problems and usually depends on random thresholds that vary among various intersection management tools. The major problem with multiple stops is that after the first stop, later stops occur from a lower speed and therefore have a less adverse influence on driver comfort, operating costs, and safety. The estimated number of vehicle stops is important in safety considerations, traffic operations, vehicle fuel consumption and emissions.

For signalised approaches, some models use the probability of stopping where the maximum probability is 100% which means that the maximum number of stops is 1.0 on any approach. In other modelling algorithms subsequent stops are based on the release from the stopped state when the vehicle reaches an arbitrary threshold speed, often around 24 km/hr (TRB, 2010). Deterministic and simulation models are common tools for estimating the number of stops, and most allow user-specified values for the parameters that establish the start and end of a stop.

Many researchers (Webster, 1958); (Catling, 1977); (Cronje, 1983) have established definitions to deal with this measure of performance. An important contribution was by Webster (1958) who generated stop and delay relationships by simulating a one-lane approach at an isolated signalized intersection. In particular, simulation results have been a fundamental concept to developing traffic signal operating procedures since the concept was first generated. The number of vehicle stops using queuing analysis is computed as all vehicle arrivals when the traffic signal is red or when a queue exists at the approach stop line. The number of stops per vehicle can be calculated using the following equation.

$$N_s = \frac{s}{C(s-q)}.r \tag{2.12}$$

where,
Ns = number of stops per vehicle (stops/vehicle),
s = saturation flow rate (vehicles/second),
C = cycle length (seconds),
q = arrival flow rate (vehicles/seconds),
r = red interval (seconds).

Webster and Cobbe (1966) developed the Webster formula for estimating the number of stops under random vehicle arrival conditions. The number of vehicles that are stopped at least once by the signal operation during the evaluation time can be derived with the assumption of a random arrival pattern as indicated in the following equation.

$$N_s = \frac{k_f t_e q(C-g_e)}{[60C(1-y)]} \leq \frac{q t_e}{60} \qquad 2.13$$

where,
N_s = number of passenger car units stopped at least once during the evaluation time,
k_f = adjustment factor for the effect of the quality of progression from delay formula,
q = arrival flow (passenger car unit/hour),
g_e = effective green interval (seconds),
C = cycle length (seconds),
y = lane flow ratio; $y = q / S$ capped at $y \leq 0.99$, with
S = saturation flow (passenger car unit/hour),
t_e = evaluation time (minutes).

The formula was developed further for estimating the number of vehicle stops under saturated conditions by Catling (1977) and Cronje (1983).
Catling (1977) used classical queuing theory to imitate oversaturated traffic conditions and developed an estimation method for a full queue length that captured the time-dependent nature of queues.
Cronje (1983) developed equations for estimating the queue length and number of stops under a fixed-time signal operation. The models developed by Catling and Cronje were used for both under and over-saturated traffic conditions.

$$N = q\left[\frac{(q.r+Q_0)}{(s-q)} + r\right] + Q_0 \qquad 2.14$$

where,
N = number of stops per cycle (stops/cycle),
q = average arrival rate (vehicles per second),
s = saturation flow rate (vehicles per second),
r = effective red interval (second),

$$Q_0 = \frac{I.H(\mu)x}{2(1-x)} \qquad 2.15$$

I = ratio of variance to the average of the arrivals per cycle,
= 1 for Poisson distribution,

$$H(\mu) = e^{-(\mu+\mu^2/2)} \qquad 2.16$$

$$\mu = (1-x)(s.g)^{1/2} \qquad 2.17$$

g= effective red interval (second),
x = v/c ratio; degree of saturation,
v = traffic volume (veh/hr), and
c = road capacity (veh/hr).

The estimation of vehicle stops can be computed second by second as the ratio of the instantaneous speed reduction to the free flow speed. A reduction in speed from free flow speed to a speed of zero constitutes a full stop. A reduction in speed to a speed equal to 25% of the free-speed constitutes 0.25 of a full stop. The sum of all partial stops over the travel period constitutes the total number of stops as indicated in the following equation.

$$S = \sum_{i=\Delta t}^{T} \frac{u(t_{i)} - u(t_{i-1})}{u_f} \;\; \forall i \;\; \ni u(t_{i)} < u(t_{i-1}) \qquad 2.18$$

where,
S = estimated partial stops,
u_i , u_{i-1} = speed of vehicle at time i and time i-1 (kilometres/hour),
u_f = free speed on travelled link (kilometres/hour).

VISSIM computes a vehicle stop when a vehicle changes its speed from any speed greater than 0 to a speed of 0, that is when a vehicle comes to a standstill. The approach does not account for partial stops. VISSIM also reports the number of stops within the queue, which are defined as events when the vehicle enters the queue, that is when its speed falls below the speed for the beginning of the queue (PTV, 2011). Reducing the value of this measure of effectiveness reflects a significant increase in efficiency of traffic flow and safety along an urban arterial road (Zhou et al., 2013); (Factor et al., 2012); (He and Hou, 2012).

2.3 Traffic control management strategies

Many travel demand management (TDM) strategies to suppress traffic congestion in cities have been proposed in the literature. The following sections present a brief description of some of the strategies that have a large effect on traffic flow performance in cities.

2.3.1 Road pricing strategies

Road pricing strategies have been implemented in many countries to reduce traffic congestion. The objective of such a strategy is to alter the travellers' behaviour, to reduce congestion by charging them an amount of money for using a network during specific periods (for example during "rush" hours). There are three main model categories: Marginal-Cost Pricing Models, Bottleneck Models, and Cordon Models. The theoretical background of the Marginal-Cost Pricing Model relies on all links being tolled. Initial work was reported by Pigou (1912), Vickrey (1963), and later by Beckmann (1967). If the charge postulated on each link in the network equals the additional congestion cost on other car users, revenue is maximized. The Bottleneck Model, otherwise known as the second-best toll model, was studied by Marchand (1968) using a general equilibrium model for two routes where an untolled alternative road was available parallel to a toll road in the network for a fixed time period. The second-best pricing models are described in Arnott et al. (1990) and McDonald (1995), where bottleneck tolls are charged on selected roads.
The Cordon Model utilised an area based pricing instead of toll charging on individual segregated links. This method is applied in many countries to alleviate traffic congestion in and around cities such as Singapore, Trondheim, Oslo, Stockholm, and London. Transport For London (TFL) (2004) has implemented a congestion pricing scheme in central London to reduce traffic congestion both inbound and outbound in the control area and to improve other facilities such as bus services, travel time reliability, and to make the distributions of goods and services more convenient and sustainable. Inflow performance increased 30% in and around the control zone. Recently, the same method has been used by Geroliminis and Levinson (2009) to investigate the effect of cordon pricing and congestion pricing in a dynamic way during a morning commute. The results of this study indicated that these schemes can enhance mobility and significantly mitigate congestion in

downtown areas. Hongfeng et al. (2008) explained that Singapore has long experience in successfully implementing area tag licensing and roadway pricing schemes as one of its major strategies to deal with traffic congestion. The initial reduction in traffic flow entering the control area was 44% and declined further to 31% by 1988, despite an increase in employment of 30% and an increase in vehicle ownership of 77% during the same period. Zhang and Yang (2004) investigated cordon-based, second-best congestion pricing problems on road networks involving optimal selection of toll levels and toll locations. The study revealed that the elastic-demand traffic equilibrium constraint maximized social welfare. The same study explained the impact of implementing different pricing scenarios on trips for various lengths. Figure 2.2 shows the distribution of trip length for four cases: the marginal-cost or first-best pricing, the single-layered and double-layered cordon pricing and no toll pricing. The figure reflects the impacts of alternative pricing strategies on car users travelling between various origin-destinations pairs related to different trip lengths. Firstly, for trips of less than 10 min, there was an increase in demand after presenting cordon pricing compared with the no tolling case while the demand decreased respect to the marginal-cost pricing case. This was attributed by the researchers to a simple fact that drivers try to avoid toll charges when the distance of travel is short and do not cross over the toll cordon. Secondly, for trips between 10 and 35 min, the demand for no tolling charge increased rapidly in contrast with cordon pricing scheme. The demand for single cordon pricing is slightly higher than double- cordon pricing. This is because the cordon-based, second-best route primarily affects medium length trips and makes demand drop. Finally, for trips greater than 35 min, the demand after implementing the cordon-pricing plan decreased compared with no toll case whereas it is higher for marginal cost pricing. This reflects the fact that cordon pricing has limited effect on long distance travel demand.

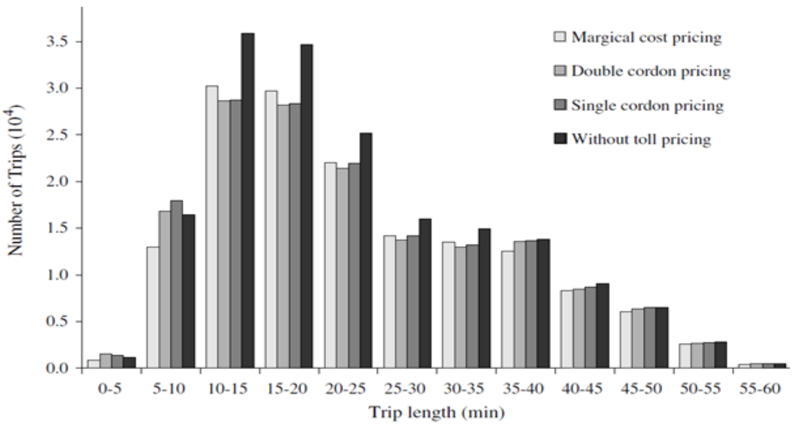


Figure 2.2 Traffic entering the restricted zone under electronic road pricing
Source: Zhang and Yang (2004)

Olszewski and Xie (2005) studied the effect of an Electronic Road Pricing (ERP) scheme in Singapore where charges are placed on vehicles entering the central business district (CBD) area using some motorways. Charges vary by time of day and are periodically updated in order to maintain traffic flows within appropriate levels. The study postulated a discrete choice based model of traveller’s response to pricing. This is imposed at a single toll point that reflects changeable pricing during the peak period. The authors confirmed that a variable pricing scheme is an effective method of suppressing congestion. The change in travel behaviour is reflected when

ERP is implemented on the road network. Such a variable pricing scheme creates a good opportunity to investigate the relationship between issues relating to congestion pricing. Luk and Hepburn (1993) studied the relationship between the elasticity of traffic volume and ERP rates for different locations and time periods using the arc formula. Figure 2.3 illustrates the effect of ERP on inbound flow over a time interval. The formula is as follow:

$$E_A = \frac{\Delta Q(C_1+C_2)/2}{\Delta C(Q_1+Q_2)/2} \qquad 2.19$$

where,
$Q_1, Q_2 =$ Average traffic demand before and after the change and
$C_1, C_2 =$ Price before and after the change.

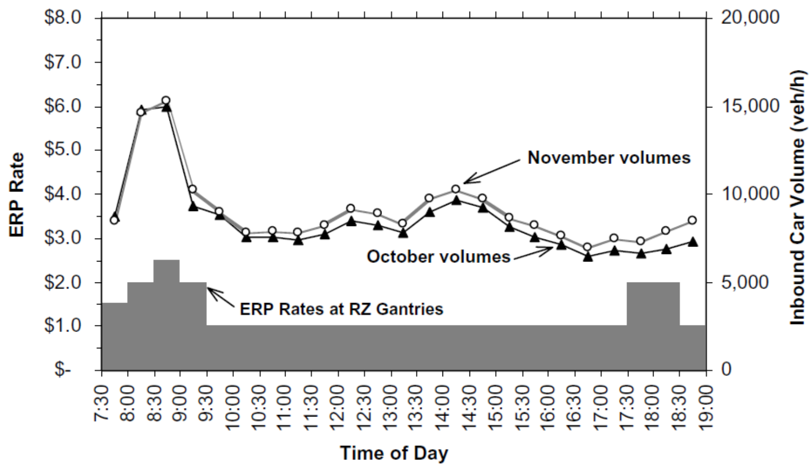


Figure 2.3 Traffic entering the restricted zone under ERP, (1998)
Source: Olszewski and Xie (2005)

There are many reports on using congestion pricing as a strategy to mitigating congestion of cities. However, there are some negative attitudes exhibited by drivers regarding this method. Verhoef et al. (1997) and Jones (1995) identified the concern of drivers about penalties that how to pay for using urban roads. This study found that drivers regarded themselves as victims of congestion and felt that they were not responsible for it, and as such they should not have to pay for it. More details about driver attitudes towards pricing congestion are found in Jones et al. (1991). Hongfeng et al. (2008) investigated the public attitude towards policy instruments for congestion mitigation in Shanghai. The study revealed that users had a negative attitude towards traffic policies such as congestion pricing, higher parking charges in congested areas and car restrictions.

2.3.2 Constraint traffic flow strategy

Control light devices have been used for decades to help solve traffic problems in urban areas. This approach to traffic solution has been used for over 40 years (Buchanan, 1963). Orski (1972) and Watson and Holland (1978) explained that the best application forms of traffic restraint were obtained by using principles from fundamental traffic theory. Traffic theory is based on the relationship between average speed, road density, and traffic flow. This method was initially used successfully in some older cities in Europe and then became popular for large metropolitan areas in developing countries. Wardrop (1953) studied the evolution of macroscopic models in arterial roads, and extended the concept to involve general road networks.

Smeed (1966) revealed that the maximum flow that could enter a city centre would be a function for that city. Vehicles per unit time per unit width of road are normally considered to represent the capacity of a road. Thomson (1967) pointed out that the relationship between average speed and flow appeared to be a linear-decreasing relationship (data collected refers to uncongested condition). This finding was based on data collected from streets in the metropolitan London over many years.

Zahavi (1972) postulated that traffic volume behaved inversely to speed of traffic. The study analysed the correlation between traffic volume and speed for different cities in Great Britain and the United States of America by augmenting data related to various regions of a city through the same time interval. This concept (Zahavi's theory) applies when traffic volume is low as the model is unable to cope with overloaded traffic conditions if the volume is high. Therefore, these models are not suitable to depict the peak period in a traffic jam.

Herman and Prigogine (1979) established a two-fluid model which explained the macroscopic relations of vehicular traffic in megacities. The model proposed that speed flow involves two categories. The first one deals with uninterrupted flow and the second one deals with interrupted flow that results in internal conditions such as congestion, stopping at traffic lights and slowing of traffic due to severe weather and accidents. The model is presented as a basic function that illustrates the interaction between average speed of moving vehicles (v_r) and the fraction of moving vehicles (f_r) as follows:

$$v_r = v_m . f_r^k \tag{2.20}$$

where v_m is average maximum running speed and k is a parameter that denoting the performance of traffic services in the network. The factor v_m refers to average speed for a vehicle moving arbitrarily in the network without any interference from other vehicles, only stopping due to the control devices. The model of the traffic network is complex and so the representation of the model involves the geometry of curves and surfaces. The fraction of stopped time for an individual vehicle running in the network for a sufficient period of time will be equal to the average fraction of stopped time for a group of vehicles during the same interval of time (f_s). Consequently, the following relationship can be expressed in terms of mean travel times, for example:

$$T_r = T_m^{\frac{1}{k+1}} . T^{\frac{k}{k+1}} \tag{2.21}$$

where T_r and T_m are the inverse of v_r and v_m respectively, and T is the mean total travel time (including stopped time T_s) per unit length.

Herman and Ardekani (1984) investigated and improved the concept of the two-fluid model for data collected in Austin, Houston and various other urban areas of the United States. The study was based on control vehicles following arbitrarily selected cars in designated networks. Herman et al. (1988) studied the impact of using a fluid model on travel behaviour and demonstrated that the two-fluid model is sensitive to the car user's behaviour. Mahmassani et al. (1987) studied the relationship of fundamental traffic flow by simulation of a network and considering the three

parameters speed, flow, and density. An additional relationship between average fractions of stopped time for vehicles for the two-fluid model and the total network accumulation of vehicles (density) were assumed to derive those relationships.
Rathi and Lieberman (1989) explained the effect of restrained traffic on the performance of mobility within cities. The study utilised simulated traffic to prove their hypothesis. They found that the use of metering control schemes on the external approaches to cities had a crucial influence on improving the overall performance of traffic in highly congested areas of New York CBD during peak periods.
Viegas (2001) stated that effective urban traffic management required a balance between the quantity of traffic in an area and a stabilised level of control necessary to maximize mobility. As an example, a novel traffic light operating system was developed to be an active policy to manage high vehicular flow in Zurich. The strategy is based on the concept that the flow towards overloaded areas should be restricted whereas the flow towards under-utilized areas should be promoted. Alleviating congestion in central areas is achieved by the metering of access to these areas to maintain the movement of cars at a stabilized level. Red phase time was increased on the perimeter of the city centre during rush hours for vehicles wanting to directly travel to the core of the city. Another more recent example given by Daganzo (2007) explained that the two-fluid model has one vague issue regarding its validity under various origin-destination demands. The author presented a simple and general model that could represent this state. The study proposed that travel productivity P (number of vehicle per length of road per unit time) under stable conditions could be expressed as a function of the entire density (accumulation, n) of the network without respect to disaggregate link data. This assumption was based on two characteristics: (i) the network is homogeneously loaded and (ii) congestion is uniformly distributed on the network. The following formula indicates this phenomenon.

$$\sum P_i\,(n_i) \cong P(\sum n_i) \equiv P(n) \qquad 2.22$$

where P_i is the travel production for a single link i and n_i is its accumulation.

The 1970 models presented a monotonic relationship between trip production and accumulation as shown in Figure 2.4. The 2007 proposal identified a macroscopic fundamental diagram (MFD) that identified three different stages of traffic flow in the network, resembling traffic behaviour of an individual link: (a) uncongested state when the number of vehicles used in the network are few (stage I), (b) congested when the value of density (n) is large (stage III) and (c) capacity level when the value of density (n) is in the range between the congested and uncongested states and permits zone accumulation (stage II).

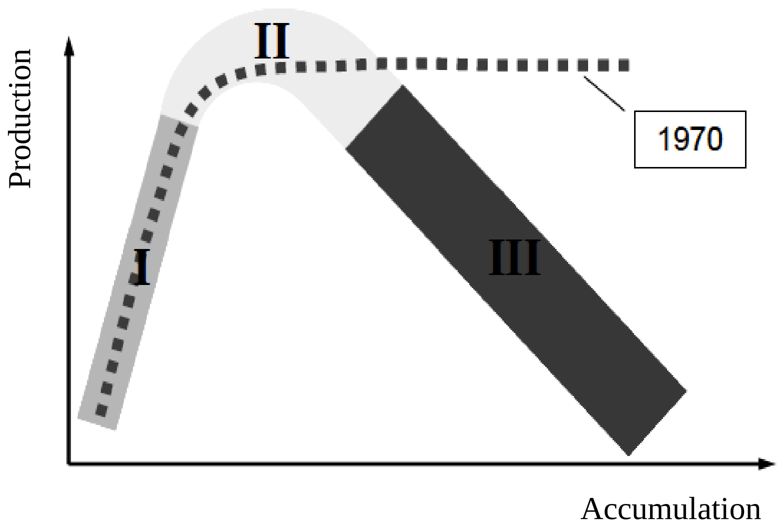


Figure 2.4 The shape of MFD
Source: Daganzo (2007)

The study revealed that car users would be exiting the network at rate $O(n) = P(n)/l$, if flow-density estimation was right, and drivers were to make trips for random lengths with average l. It was predicted that drivers naturally seek under-used parts of a network. Therefore, the entrance and exit points of a neighbourhood area should not disturb the movement of vehicles. This traffic condition could be achieved by macroscopic modelling of the area as an individual neighbourhood dynamic system (with queuing) and accumulation (n) as a single state variable. This depiction can be described dynamically according to the equation 2.23:

$$\frac{dn}{dt} = f(t) - O\big(n(t)\big) \text{ for } t \geq 0 \qquad 2.23$$

where $f(t)$ is the input flow to the system.

In addition, Daganzo (2007) demonstrated that dynamic systems (with queuing) may create a gridlock phenomenon in the network. To avoid this case, appropriate control policies irrespective of demand data can be implemented if trend of flow can be monitored. This policy could be readily understood by metering the input flow towards the centre of the congested area and directing vehicles onto other paths if applicable. The same model was successfully tested by Geroliminis (2007) using real data collecting for Yokohama in Japan. Use of the model improved urban mobility and relieved congestion in Yokohama.
However, although this strategy achieved satisfactory improvements in mobility inside the urban areas, it created dense traffic outside the control district that lead to worse levels of service in the controlled area. May (1986) previously reported that restrained traffic would be unbeneficial, ineffective and have a negative impact on business in the affected area. In addition, it would be unfair for certain groups in society and it would be difficult to enforce the strategy on people who would not benefit from the policy.

2.3.3 Metering flow strategies

The concept of freeway on/off ramp metering is based on the application of restrained traffic flow, on the periphery of an urban network. This technique can be used to relieve traffic congestion in metropolitan areas, as mentioned in Rathi and Lieberman (1989). Levinson et al. (2004) indicated that ramp meters were first used in 1963 on the Eisenhower Expressway (I-290) in Chicago. The first implementation was conducted by a police officer who was responsible for controlling the traffic between an entrance ramp and the mainstream of the freeway by stopping traffic at an entrance ramp and releasing vehicles at a predetermined rate in order to achieve safer and smoother merging onto the freeways without interrupting the mainstream flow. The concept of ramp metering has been deployed systematically in` developed countries such as the United States of America, Canada, Australia, and the United Kingdom.

Jin et al. (2009) developed a strategy to avoid congestion building up on a motorway due to an off-ramp becoming congested. This was achieved by metering the level of traffic services in the vicinity of motorway off-ramps on the normal street system. The strategy also was used to eliminate vehicle collisions where the off-ramp traffic joined the normal street system. The study focused on monitoring the traffic conditions at the off-ramp relating to the traffic intensity and the performance index for the surface street vehicles. The strategy optimized both traffic control signal split and cycle length within the constraint that the off-ramp queue must be kept shorter than the minimum permissible length. The average travel time was used to determine the effectiveness of this system. The three control scenarios considered in evaluating the travel time were no control, fixed-time signal timing, and an adaptive control (optimized control). Figure 2.5 explains the vehicle travel time for the three control modes. There were fewer differences in travel times for light traffic loads, whereas the reduction in travel times became more significant upon using variable control for heavy traffic loads. This was due to reduced off-ramp queue spill over onto the motorway. There was a 10% improvement in average travel time using variable control compared to using no control in the full simulation. The fixed signal timing did not enhance the performance for high traffic volumes. This may have been due to the fact that fixed timing lacks the ability to capture the temporary traffic variations.

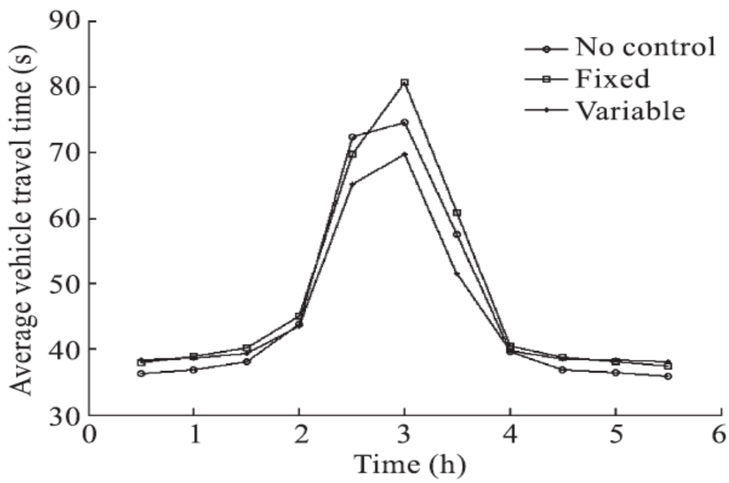


Figure 2.5 Average vehicle travel time for different control modes
Source: Jin et al. (2009)

Papageorgiou and Kotsialos (2000) explained why ramp metering could lead to a significant improvement of traffic conditions on motorways. They asserted that the traffic situation on a freeway resembles that on an urban street network prior to the introduction of traffic lights where blocked links, congested intersections and reduced safety occur. To understand the benefit of using ramp metering to alleviate traffic congestion on freeways, assume that Figure 2.6 represents any traffic network with demand emerging at several positions (usually at on ramp for a freeway network) and exit flows resulting at several destinations (normally at off-ramp on freeways).

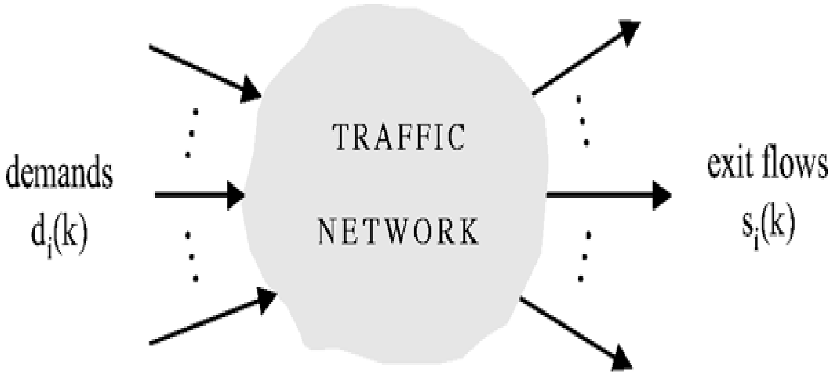


Figure 2.6 A general traffic network
Source: Papageorgiou and Kotsialos (2000)

Obviously, the accumulated demand can be understood to be equal to accumulated exit flows because in a simplistic model it is assumed no vehicles disappear and no vehicles are generated in the network. If it is assumed that traffic demand and its spatial and temporal distributions are independent of any control measures taken in the network, the accumulated time can be calculated for all drivers to reach their final destinations at the network exits. The total time spent by all drivers would be expected to be longer because of the lack of suitable control measures that make flows temporarily slower. As a result, any control strategy that is able to increase the earlier exit flow of vehicles will lead to a minimization of the entire time spent in the network. Simple mathematics were used to explore the significance of using this concept of control measures. The study used a discrete time representation of traffic variables with discrete time index $k = 0,1,2, ...$ and time interval T. A traffic volume or flow $q(k)$ in vehicles per hour could be defined as the number of vehicles passing a corresponding location during a specific time period $[kT, (k + 1)\ T]$, divided by T. Figure 2.7 shows that the network receives demands $d_i\,(k)$ in veh/h at its origin $i = 1, 2, ...$ and that total demand can be written as $(k) = d_1(k) + d_2(k) + \ldots$. The study assumed that $d(k), k = 0, \ldots, K - 1$, is independent of any control measures taken in the network. The exit flow is represented by $s_i(k)$ at the network destinations $i = 1, 2, ...$, so the total exit flows will be $s(k) = s_1(k) + s_2(k) + \ ...$.

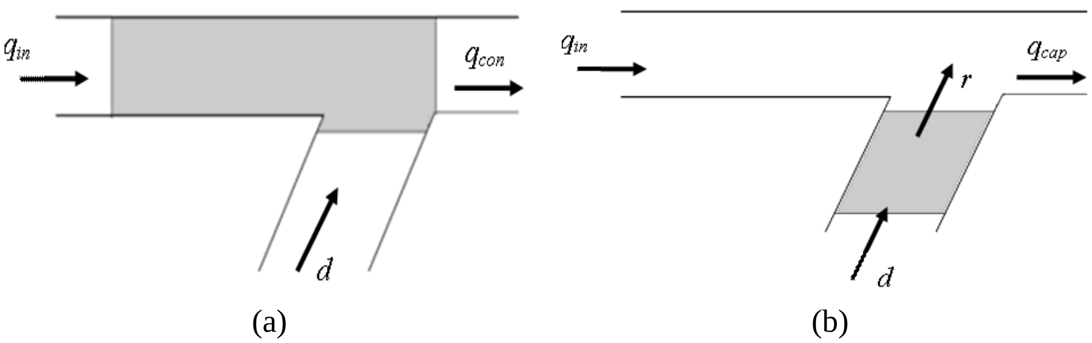


Figure 2.7 Two cases: (a) without ramp metering control and (b) with ramp metering control; grey areas indicate congestion zones.
Source: Papageorgiou and Kotsialos (2000)

The aim of the control measure is to minimize the total time T_s spent in the network during a time period K where:

$$T_s = T\sum_{k-1}^{K} N(k) \qquad 2.24$$

Where, $N(k)$ denoted the total number of vehicles in the network at time k. Due to conservation of vehicles, the value can be expressed as follows:

$N(k) = N(k-1) + T[d(k-1) - s(k-1)]$, and then the equation will be

$$\therefore N(k) = N(0) + T\sum_{k=0}^{k-1}\left[d(k) - s(k)\right] \qquad 2.25$$

Substituting Eq. 2.25 in Eq. 2.24, the general equation of total travel time will be as follows:

$$T_s = T\sum_{k=1}^{K}\left[N(0) + T\sum_{k=0}^{k-1} d(k) - T\sum_{k=0}^{k-1} s(k)\right] \qquad 2.26$$

In equation 2.26, the first two terms in the square brackets of the equation are independent of the control measures taken in the network; hence, the minimization of total travel time spent is equivalent to maximization of the following value:

$$S = T^2\sum_{k=1}^{K}\sum_{k=0}^{K-1} s(k) = T^2\sum_{k=0}^{K-1}(K-k)s(k) \qquad 2.27$$

Thus minimization of T_s is equivalent to maximization of the time weighted exit flows. Maximisation will be achieved by using appropriate control measures that ensure that the earlier the vehicles are able to exit the network the less time will have been spent in the network. Two cases are considered in Figure 2.7 for a freeway on-ramp (a) without metering control and (b) with metering control.

Let q_{in}, d, and q_{con} represent upstream freeway flow, ramp demand, and mainstream outflow in presence of traffic congestion respectively, and q_{cap} be the freeway capacity. It is well known that the capacity of freeway q_{cap} is usually higher by 5%-10% than the outflow q_{con} in cases of congestion, (Hegyi et al., 2005). As indicated in Figure 2.7 (b) ramp metering may be used to maintain flow capacity in the mainstream. The ramp metering will create a queue on the on-ramp but it leads to a decrease in the total travel time spent in the mainstream including the ramp waiting time. The enhancement ΔT_s in percentile of the entire time spent will be represented by the following equation.

$$\Delta T_s = \frac{q_{cap} - q_{con}}{q_{in} + d - q_{con}} 100 \qquad 2.28$$

As an example If $q_{in} + d = 1.2, q_{cap}$ (that means total demand is more than freeway capacity by 20%), and $q_{con} = 0.95 q_{cap}$ (that means capacity drops 5% because of the congestion condition), then according to equation 2.28, ΔT_s equals 20%. This demonstrates the importance of ramp metering. Levinson and Zhang (2004) studied and analyzed situations with and without ramp metering for several representative motorways during afternoon peak periods. The research revealed the effect on seven parameters that indicate the performance of traffic conditions. The parameters were mobility, equity, productivity, consumers' surplus, accessibility, travel time variation, and changes in travel demand. It was found that ramp metering is generally useful for long trips rather than short trips. In general, they concluded that metering is better than no ramp metering. Figure 2.8 illustrates the trends in the change of mobility and equity with and without ramp metering. The figure concentrated on the relationship between spatial equity and spatial mobility, which showed worsening equity and mobility when the peak was reached. A trade-off between mobility and spatial equity was evident. That means the metering-on case had better mobility but worse spatial equity compared to the metering-off case. Lee et al. (2006) tested the influence of ramp meters on safety. The research showed that the crash rate increased when ramp metering was not used.

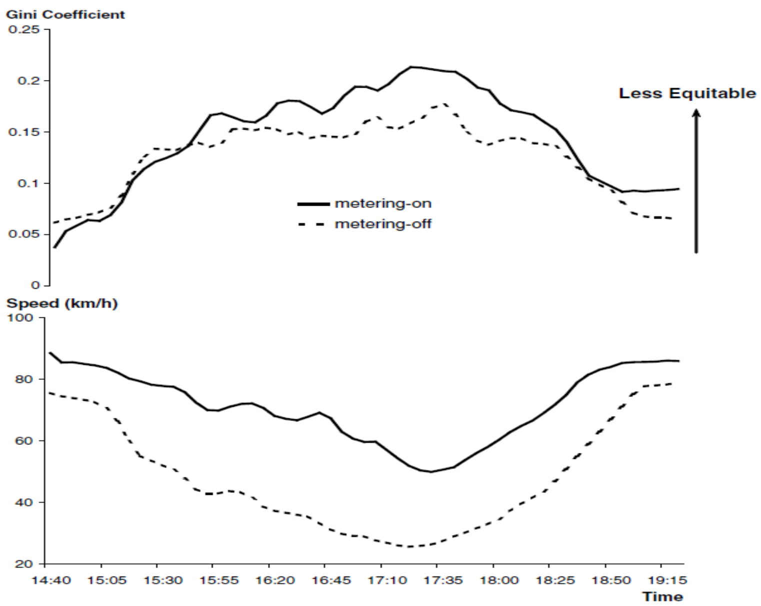


Figure 2.8 Spatial equity and mobility on Trunk Highway 169
Source: Levinson and Zhang (2004)

Generally, ramp metering has a significant impact when traffic volume is not too light. From the viewpoint of agencies such as a freeway authority, traffic ramp controls can be efficient if they take into account an appropriate control mechanism that ensures the traffic entering the freeway does not produce an overflow or an underflow condition. Overflow means that traffic flow is over the available utilized road and where an accident or traffic congestion may occur. Underflow means that the freeway is underutilized, which is not cost beneficial in any analysis for the construction of the freeway.

Hou et al. (2008) applied a new model to address the traffic accumulation dilemma that occurs with ramp metering control in a macroscopic level freeway environment. The model introduces iterative learning control that can effectively deal with this sort of problem. The model is intended to develop a new locally actuated control dependent on recurrent traffic conditions. Papageorgiou and Kotsialos (2000) reported that this local control method is far easier to design and implement, and proved to be as effective as more sophisticated coordinated approaches under recurrent traffic conditions. Bellemans et al. (2006) studied the influence of ramp metering on an actual motorway in Belgium during the morning rush hour. Two types of controller were used to show the effect of using this method: a traditional ALINEA (Asservissement Lineaire d'Entree Autoroutiere-French) controller and a model predictive control (MPC) based ramp-metering controller. A comparison was made between the two types of controllers to identify the effect of application of this procedure on the service level of the freeway. The most significant finding to emerge from this study was the reduction in total time spent on the metered ramps for the MPC.

Ramp metering is usually affected by the capacity of the storage lane to accommodate the number of vehicles. When the length of the ramp storage lane is long enough to accommodate all waiting vehicles, the efficiency of ramp metering will be high. Effective ramp metering needs to be supported by other traffic management strategies to ensure the capacity of the storage lane is not exceeded (Chen et al., 2001).

Many strategies such as ramp metering coordination (Papamichail et al., 2010), route guidance (Wang et al., 2006) and dynamic speed limit (Papageorgiou et al., 2008), have been implemented to support and enhance the performance of traffic on the freeway, by suppressing the traffic in the vicinity of on/off ramps, with or without ramp metering. Each one of these strategies provides benefits in improving the traffic condition on the freeway in conjunction with the use of ramp metering.

This literature review will focus on the strategy of Variable Speed Limit (VSL), which has been shown to be promising area for further research.

2.3.4 Variable speed limit Strategy (VSL)

Variable speed limits have been widely used to regulate traffic flow speeds by using variable message signs (VMS). The original aim was to improve traffic safety by increasing driver compliance that subsequently leads to decreasing the potential for collisions, as reported by Coleman et al. (1996). Carlson et al. (2010) proposed a new traffic management tool to optimize mainstream traffic flow over large-scale motorway networks. Their approach to optimization of traffic flow was based on the concept of variable speed limits to match the mainstream traffic flow control (MTFC) either as a stand-alone measure or in combination with ramp metering.

A previous study by Papageorgiou (1983), Lenz et al. (2001), Hegyi et al. (2003) Carlson et al. (2010) which included MTFC for optimal integrated motorway network traffic control demonstrated that traffic flow could potentially improve via MTFC with or without ramp metering actions.

Hegyi et al. (2003) investigated the influence of shock waves that may appear on a freeway due to sudden changes in traffic conditions. They studied the use of dynamic speed limits to eliminate, or at least decrease, the impact of creating shock waves.

The variable speed limits were connected with a model predictive control (MPC) to optimise the coordination of variable speed limits in order to dissipate the occurrence of shock waves in freeway traffic. Several methods have been used to model suitable control laws for dynamic speed limit controls such as multi-layer control (Papageorgiou, 1983), sliding model control (Lenz et al., 2001) and optimal control (Alessandri et al., 2002, Di Febbraro et al., 2001).

Bertini et al. (2006) investigated the effect of variable speed limits and a car user information system (overhead dynamic message signs) on traveller behaviour and bottleneck creation and location. The speed limit and text messages were periodically compared with actual variations in traffic on the segment of a German autobahn. The analysis revealed a strong correlation between prior warning of traffic congestion and effective management of traffic load. In other words when drivers were previously warned of approaching a congested area, speed limits were decreased before crossing into the congested area as a means of managing the traffic load. Reduction of the speed limit to control dense traffic can maintain the traffic flow at 30-40 km/hr in the congested area.

Zackor (1991) proposed a quantitative model to present the possible increase of the critical occupancy (critical density) under the influence of VSLs. The model was focused on avoiding or mitigating traffic flow breakdown by reducing the inbound flow at bottlenecks using speed limit controls. The resulting change in capacity is shown in Figure 2.9. In this figure, b represents the ratio of the applied VSLs to the free flow speed, and $b = 1$ denotes the no VSLs case.

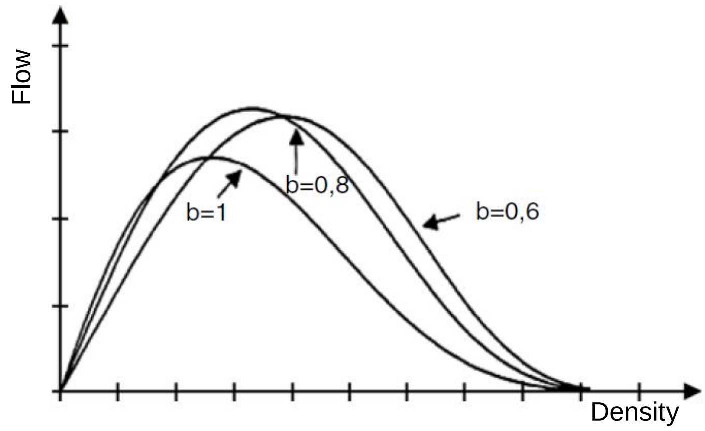


Figure 2.9 Flow-density diagram versus VSLs. Where b=1, 0.8 and 0.6 correspond to no speed limit, VSL=0.8vf and 0.6vf respective
Source: (Zackor, 1991)

Papageorgiou et al. (2008) found that variable speed limits decrease the slope of flow-density diagram under critical conditions, shift the critical density to higher values and enable higher flows at the same density value in over critical conditions. The study was based on the criteria of (Zackor, 1991). The variation in capacity due to using VSL controls was exploited to enhance the traffic flow efficiency on freeways. Thorough understanding of VSL controls may achieve acceptable levels of traffic safety and environmental improvements. Figure 2.10 depicts the correlation of flow-density with and without VSLs.

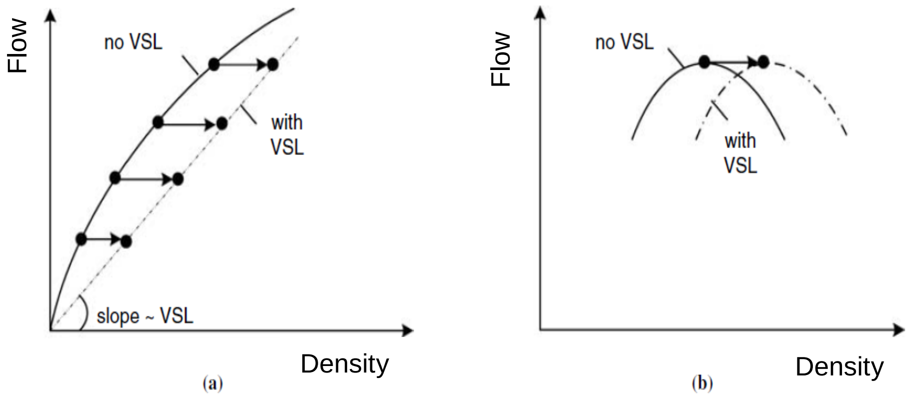


Figure 2.10 (a) The relationship between flow and density under the impact of VSL and (b) cross- point of diagrams with and without VSL
Source: (Zackor, 1991)

Hegyi et al. (2005) discussed the optimal coordination of variable speed limits and ramp metering on a freeway traffic network to minimize the total time spent in the network. The study proposed a Model Predictive Control (MPC) system which included the extension of METANET (Modèle d'Ecoulement du Trafic Autoroutier: NETwork), (Kotsialos et al., 2002). In this VSL control model, the desired speed in the relaxation term of the METANET represents the minimum value between the fundamental diagram (speed-density relationship) and the displayed speed limit. Carlson et al. (2010) extended the METANET model to incorporate the impact of displayed VSL values on the traffic dynamics in a different way with the use of the affine functions. The use of dynamic speed limits potentially decreased the congestion period and resulted in traffic spending less time in the road network.

Lin et al. (2004) studied the effectiveness of variable speed limit controls on highway work zone operations using two online algorithms. The aim of using the two online algorithms was firstly to reduce the traffic queue preceding the work zone area and secondly to maximize the entire throughput through the work zone. The study showed that VSL models could produce a potential increase in both work zone throughput and a reduction in vehicle delays. The study presented that VSL models can produce a potential increase in both work zone throughput and reduction in overall vehicle delays. Kwon et al. (2007) developed a practical method to reduce traffic conflicts and to improve the efficiency of work zone operation. The method proposed two stages of speed reduction accompanied with sending advisory speed limit messages. The method operated so as to reduce the speed of the upstream flow approaching the work zone area to the same speed as the downstream flow. It was implemented for three weeks in Twin Cities, Minnesota in 2006. The method had the efficiency of reducing the longitudinal speed variances along the work zone area. The results showed that a 25-35% reduction in speed resulted in an approximately 7% increase of the entire downstream work zone throughput during a 6:00-7:00 am period.

Retting and Cheung (2008) assessed the effect of increasing vehicle speeds on the productivity of trips on motorways. Generally, the downstream area was distinguished by low levels of service during peak hours (high traffic demand). Escalating speeds towards the downstream area increased the number of vehicles and reduced the performance for the area. This increase in speed created unbalanced conditions in the flow rate, making the inbound flow rate much higher than the outbound flow rate. Consequently, traffic breakdown occurred due to degradation of road capacity caused by lack of appropriate traffic control.

Hadiuzzaman et al. (2012) developed an analytical model involving the drivers' reaction on freeway to updated dynamic speed signs and macroscopic speed dynamical change with respect to changeable speed limits. The findings proved that the application of a VSL strategy could create a uniform traffic distribution over freeway links. In addition it was found that VSL controls may prevent high traffic density that leads to traffic flow breakdown. This study demonstrated that VSL control has the capability to delay the onset of congestion and to recover the desired speed earlier than without it.

2.4 The implementation of VSL on arterials

One of the basic traffic solutions in mitigating congestion and maintaining the flow in cities is using coordination system along signalised urban arterial roads. However, this type of solution will not be able to make the traffic flow operates correctly under such circumstances that is severe traffic congestion associated with short signalised roads. The following sections outline the concepts of using coordination system under the two conditions with and without VSL applications.

2.4.1 Signal coordination for arterial roads when no queue formation

Coordinated signalized intersections are used in large cities to maintain flow and minimize the delay. The basic element required to coordinate the intersections is the offset factor. The offset is used to organize the successive green phases in a way to enable upstream traffic flow without a requirement stop at a signal. The offset can be defined as the difference in time when both upstream and downstream intersections turn green. Many parameters are considered in designating the offset including length block, posted speed and the number of queued vehicles at the intersection. The determination of the empirical offset is not an easy task in complicated traffic conditions. Figure 2.11 represents the time space plot for a vehicle traversing between two signalised intersections with no vehicles queued at the upstream traffic.

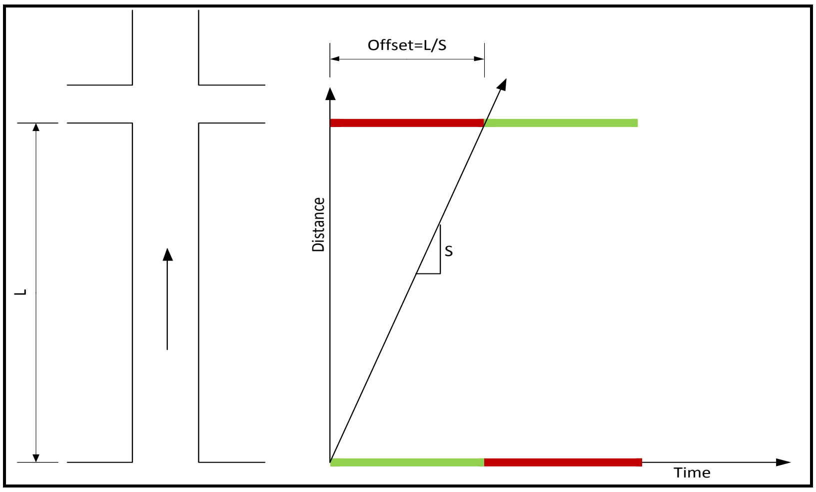


Figure 2.11 Ideal offset
Source: TRB, 2000

Figure 2.11 shows that there was no vehicle queue at downstream signal. The signal will turn green so to let the leader vehicle of a platoon traverse without stop. In this case, the ideal offset calculation will depend solely on the length of the block and the posted speed. Therefore, the ideal offset can be expressed as follow:

$$Ideal\ offset\ (I.O) = L/s \qquad 2.29$$

where,
L = block length and s= the posted speed.

If L is considered constant, the variable factor in the above equation will be the speed factor. The offset signal is based on the arterial posted speed between upstream and downstream signal. This method of calculation may not be correct because it depends on the posted speed for the arterial rather than the actual speed of a leader vehicle. When the leader vehicle drives with speed less than the posted speed, the vehicle will lose some of green time due to delay in arriving. This situation contributes to decrease the intersection throughput, increase the number of stopping at the intersection, and increase the travel time along the networks.
When the leader vehicle drives at a speed higher than the posted speed, the vehicle will reach the downstream intersection earlier than the green phase initiation. In this case, vehicle delay and stop will increase queue forming of vehicles.
The determination of actual speed along the arterial road is potentially significant to identify the accurate offset that can improve on the performance of traffic flow. Figure 2.12 shows different travel speed behaviour for one-way lane direction.

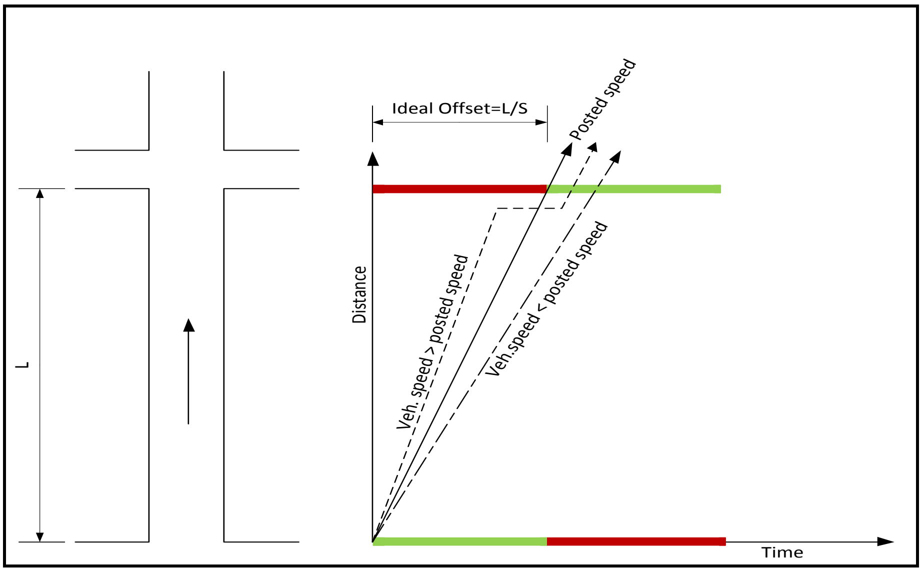


Figure 2.12 The effect of speed on the traffic operation

2.4.2 Signal coordination for arterial roads when queue formation

The presence of queued vehicles at a downstream intersection requires adjustments to be made for calculating the ideal offset. This adjustment assumes that the queued vehicles at a downstream signal, during upstream flow, are progressing towards the intersection. In this case, the offset will trigger the initiation of downstream green before the upstream traffic reach the stop line of the downstream intersection. This helps clear the downstream intersection of vehicles before the upstream traffic approach the downstream intersections. The initiation of green time will depend on how the number of queued vehicles and the service rate at the downstream intersection. In order to discharge the queued vehicles, the time will be taken from the green portion devoted for the upstream flow. Figure 2.13 demonstrates the relationship between the offset estimation and the number of queued vehicles.

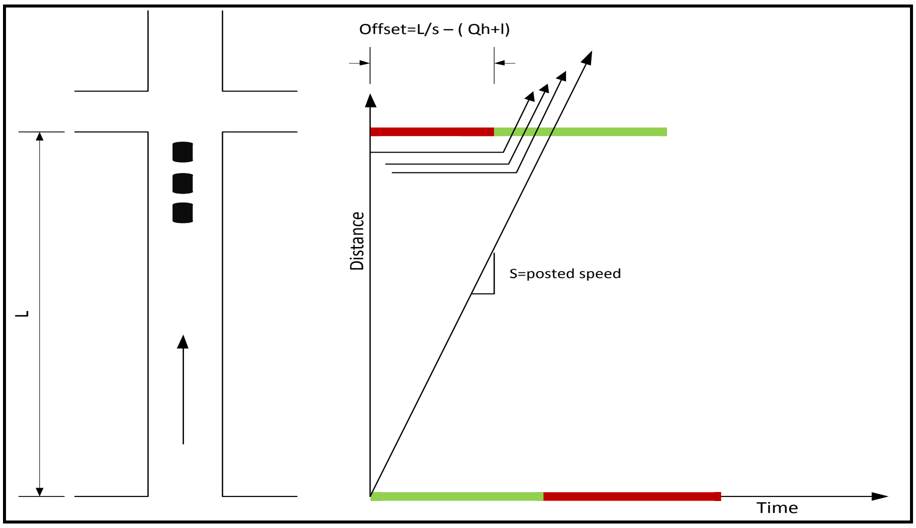


Figure 2.13 The effect of queued vehicles on the offset calculation

Figure 2.13 shows that upstream traffic arriving loses green time due to the initiation of the green signal earlier because of the presence of queued vehicles. This results in reducing the number of vehicles able to pass through the intersection, increasing the number of stops and increasing the total stopped delay. A new green time interval exists due to this operation and can be estimated as follow:

$$G_n = G_o - (Q*h + l) \qquad 2.30$$

where,

G_n = new effective green time,

G_o = original effective green time,

Q = number of queued vehicles,

h = discharged headway and

l = start up lost time normally range between (2-4secs).

The capacity of downstream intersection will be impacted negatively as well due to decreasing green time for upstream flow. Consequently, a reduced capacity can be calculated as follows:

$$c_d = (G_n / C) * S \qquad 2.31$$

where,

c_d = reduced capacity of downstream intersection due to a queue existing,

G_n = new effective green time,

C= cycle length and

S= saturation flow.

Since the capacity of intersection has decreased, the delay and number of stops per vehicle will increase as well. The capacity and delay at downstream intersection can be estimated as follows:

New Capacity = Original capacity – Reduced capacity

$$c_n = (G_o / C) * S - (G_n / C) / S \quad 2.32$$
$$c_n = ((G_o - G_n) / C) * S \quad 2.33$$
$$D = c_n * h / 3600 \quad 2.34$$

where,

c_n = new capacity due to the queued vehicles,

D = delay in term of number of vehicles, and

h = discharged headway.

The number of queued vehicles at the intersection can identify the performance of platoon progression and the intersection throughput efficiency. The offset calculations due to queued vehicles can be determined as follow:

Adjustment offset = Ideal offset - time required to clear the queued vehicles

$$Offset_{adj} = L/s - (Qh+l) \quad 2.35$$

All the symbols in above equation have been defined earlier.

2.4.3 Signal coordination for arterial roads under oversaturated condition

The previous section focussed on the queued vehicles at the downstream intersection under the condition that the queue doesn't exceed the block length. When the queue length propagates back and fills the entire block length of the arterial, in the case of a short link, the network will block and the coordination of traffic signal loses its purpose. Figure 2.14 shows that the entire green time at the downstream intersection has been consumed by the queued vehicles. All new vehicles which approach the intersection do so during the red phase. The side cross street is also impacted by this operation because blockage of the intersection area.

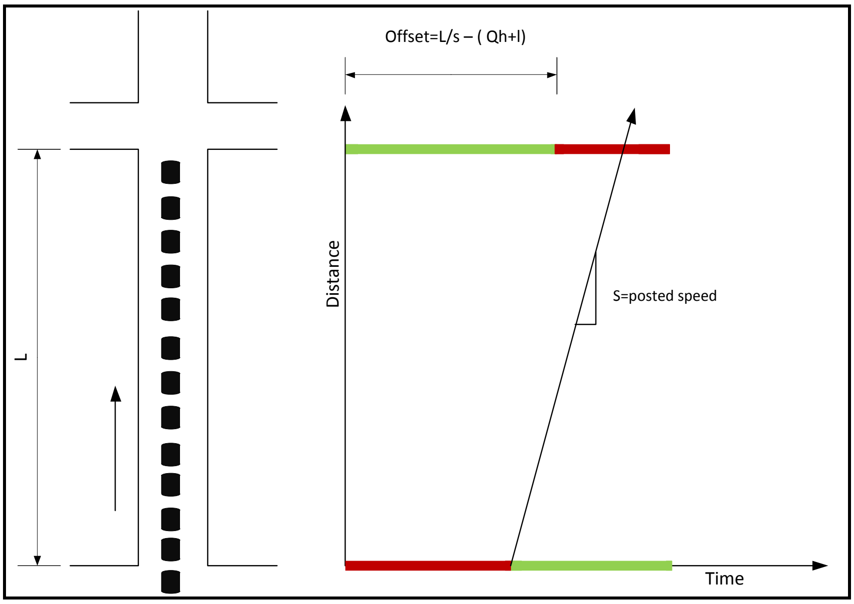


Figure 2.14 The operation of traffic flow under using normal offset

Usually signal offsets switch the upstream intersection to green relative to the downstream intersection. If the length block is filled with vehicles and the upstream intersection is turned to green, the vehicles would be stacked at the upstream link and block the network. Because of the upstream blockage, the cross street will also probably be poorly utilised. In order to avoid this traffic condition, the signal at the upstream intersection of the main street will turn red instead of green and grant the cross street a fraction of green time. This type of control is called *Equity Offset*. Figure 2.15 shows the traffic light system under those circumstances and the concept of *Equity Offset* can be expressed in the following equations.

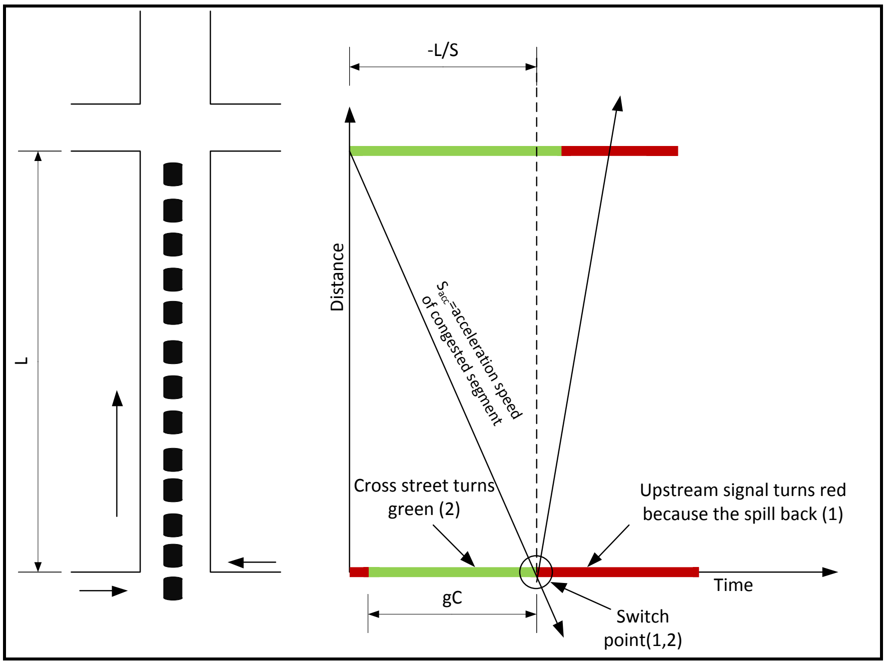


Figure 2.15 Equity offset process

$$Equity\ offset\ (t_{equity}) = g*C - L / s_{acc} \qquad 2.36$$

where,
g = fraction green of main street,
C = upstream cycle length and
s_{acc}= accelerated speed at congested link.

In Figure 2.15, the upstream signal switched red once vehicles at the upstream intersection start moving. This is intended to flush out all the vehicles accommodated at the congested section with an accelerated speed to reach about 16 ft/s which tries to ensure that no new vehicles enter the section. At the same time, a green portion for upstream traffic in the main road is devoted to the cross street in order to enhance the departure service and to avoid the probability of forming long queue due to the intersection blockage by clearing the intersection.

2.4.4 VSL and the performance of signalised intersection

VSL has the potential to improve performance by using the speed of the upstream platoon on the arterial road as a reference to match the initiation of green at the downstream signal.
Success depends on using the actual variable speed instead of the fixed posted speed. This procedure requires installation of variable speed signs at each intersection to guide the drivers to follow a specific speed. The speed sign is linked to upstream detectors and is synchronised with downstream detectors at each intersection. The downstream detectors provide traffic information on how many queued vehicles exist as information to the upstream detectors responsible for computing the desired speed and subsequent display. Figure 2.16 shows the conceptual operation of VSL.

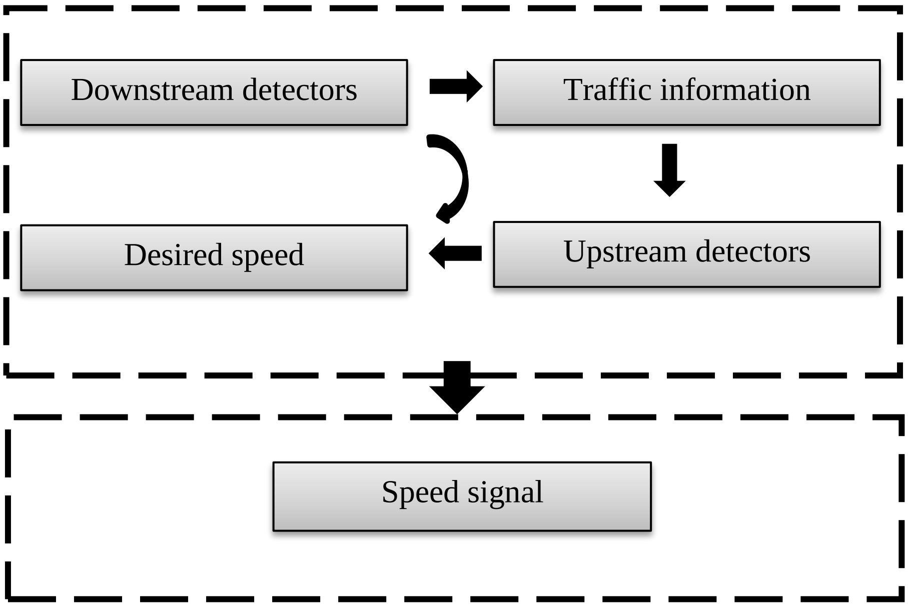


Figure 2.16 VSL plan

The new technique concentrates on the total time required to clear the downstream intersection of queued vehicles and the time spent in traversed link. The outcome is that a new speed will be calculated and displayed to control the upstream flow.

The ratio of length block to the total time spent is used to determine the desired speed as shown in equation 2.38. The number of queued vehicles represents the main factor in calculating the desired speed. If the number of queued vehicles is zero the desired speed will equal the posted speed and the calculation will be as equation 2.29. When vehicles are queued, the speed of upstream traffic can be reduced to allow clearing the queued vehicle without recalculating the new offset for the upcoming traffic. The desired speed can be determined through the following equations:

Total Time Spent = ideal time (ideal offset) + time needed to clear the queued vehicles

$$TTS = (L/s) + (Q*h+l) \qquad 2.37$$
$$\text{Desired Speed (DS)} = L / TT \qquad 2.38$$

2.4.5 VSL use when the queued vehicles consume the entire green interval

Customarily, the spilt cycle for a long length block would be less than the spilt cycle for short consecutive links. This procedure raises the efficiency of flow at short lengths and reduces the effect of queue spill back towards the upstream intersection. VSL focusses on keeping the offset calculation constant and controlling the speed of vehicle through the arterial road. Figure 2.17 shows vehicles queued at the stop line waiting the green signal which resulted in consuming the full green interval.

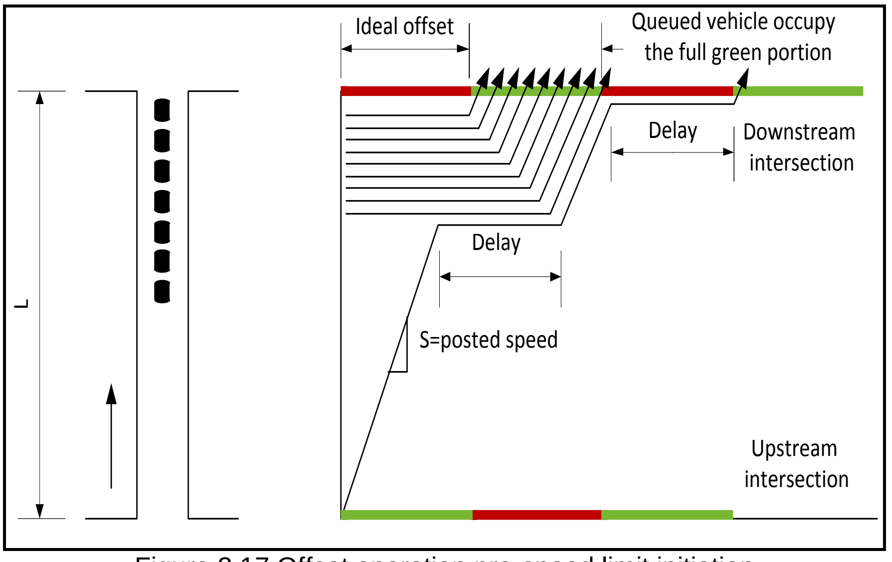


Figure 2.17 Offset operation pre-speed limit initiation

In this case, the arrival traffic will not able to pass the downstream intersection during the cycle and vehicles will be forced to stop at the back of the queue or at the start of the red signal. This traffic pattern can negatively influence the performance of arterial roads by increasing the number of stops that then reduce the safety and increase the queue length that may lead to breakdown the traffic flow.

In order to overcome such traffic situations, the speed of arrival traffic is managed by the application of a speed less than the posted speed.

Figure 2.18 shows the trajectory of arrival flow towards the downstream intersection under pre and post speed control.

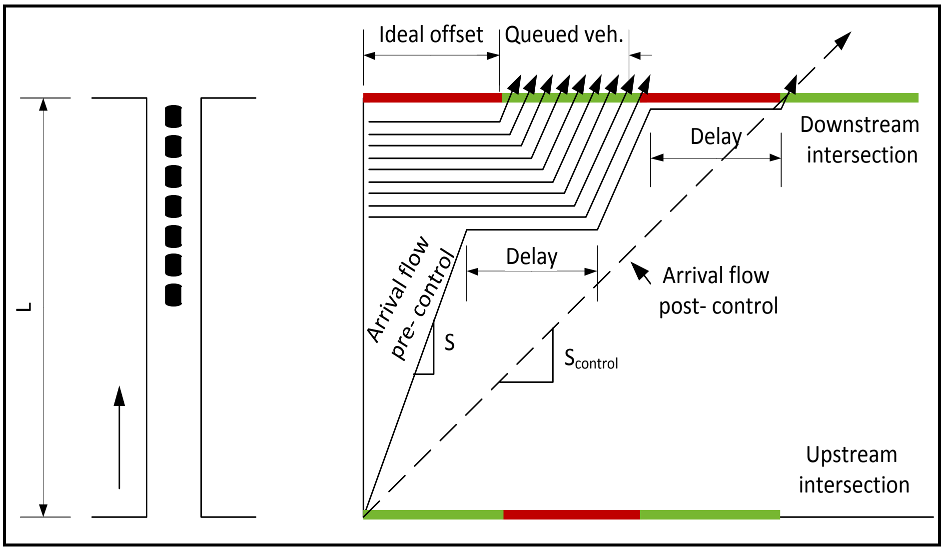


Figure 2.18 Arrival flow under post- speed limit control

The speed of arriving vehicles can be determined by the following equations:

$t_{ideal} = L/s$ 2.39

$t_{queued\ veh} = Qh+l$ 2.40

$S_{control} = L/\ (tideal + tqueued\ veh + r)$ 2.41

where,

$S_{control}$ = controlled speed that needed to manage the arrival flow,

r = fraction of upstream red signal and

L= length block.

2.4.6 Applying VSL under spill back conditions

The function of equity offset was discussed earlier, proposing a red signal for the upstream flow and granting the green time for the side street when the spillback spread towards the upstream intersection. This maintains arterial functionality when the spillback occurs from the main street or cross street.

VSL can be used to achieve similar functionality but differs in the operation and outcomes. The new concept proposes that vehicles within the through lanes at the upstream intersection progressing towards the congested intersection should be restricted using speed signal signs while the left and right turn lanes continue working. The method may assist to ensure good traffic flow for both the main street and side cross street. It should also, reduce or mitigate the effect of spillback on the downstream intersection by controlling the upstream traffic.

This method may not affect the flow of side cross street and may eliminate the likelihood of a queue propagating due to the main street backup. Figure 2.19 illustrates the mechanism of employing speed signal sign at upstream intersection.

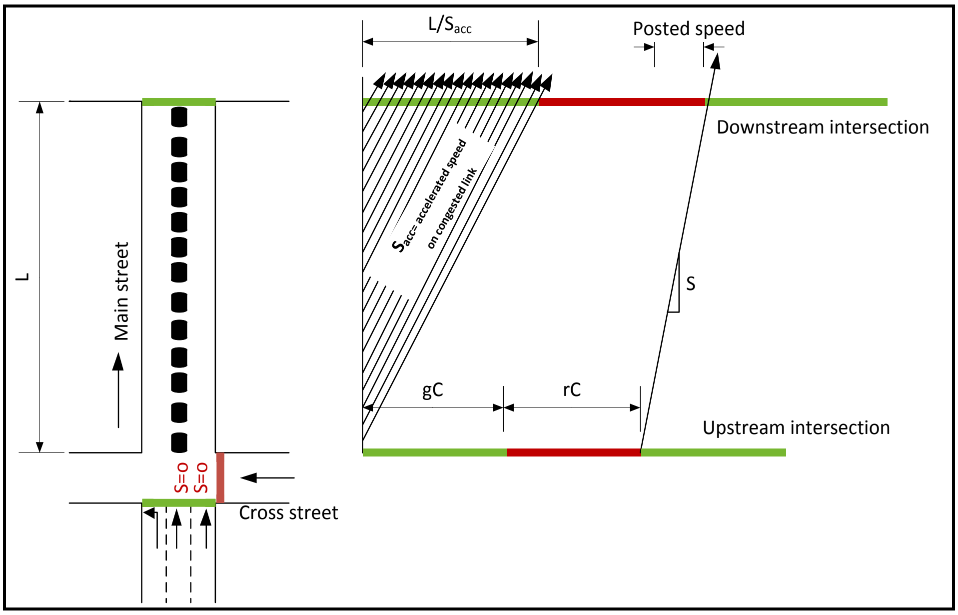


Figure 2.19 Schematic of using new spill back management

The following points can be taken from the figure above:

1. The offset at downstream intersection will not switch the green light at upstream intersection to red signal as in normal offset (i.e. equity-offset function). The green interval will retain a switch-on under condition of limit the speeds of through movement towards the upstream traffic. The value of the speed limit will be based on the length of queue at downstream intersection. In some traffic conditions, the speed limit is set out to zero in order to prevent only the through movement and allow the left turn movement.
2. The application of this method will retain the signal timing of cross street constant and prevent right turn vehicles from the cross street entering the main road at this congested period. Unlike the normal management that gives opportunity to the cross street vehicles to turn into empty spaces for main road during the spillback period.

The offset calculation for upstream intersection under spill back condition can be determined as follow:

$$t_{equity} = g*C - L / S_{acc} \qquad 2.42$$

$$t_{speed\ technique} = L / S + (r * C - (g * C - L / S_{acc}) \qquad 2.43$$

$$t_{speed\ technique} = L / S + r * C - t_{equity} \qquad 2.44$$

where,
L= length block,
S= posted speed,
r = red portion of upstream intersection,
C = cycle length of upstream intersection and
S_{acc}= accelerated speed on the congested link.

2.4.7 Using VSLs in urban areas

To understand the benefit of using the new approach, the congestions in conventional traffic moving toward the urban area is briefly explained. Figure 2.20 represents a schematic drawing for a roadway operation-containing bottleneck towards urban areas. The bottleneck is located where the flow capacity upstream (q_{cap}^{up})is higher than the flow capacity downstream (q_{cap}^{down}) of the bottleneck location. It can be seen from the figure that q_{cap}^{down} is the maximum flow that could be maintained at the bottleneck situation if the arrival flow (q_{in}) equal or less than the q_{cap}^{down} . On the other hand, traffic congestion is formed when the arrival flow (q_{in}) is higher than the q_{cap}^{down}. Thus, the congestion head is located at the bottleneck, whereas the tail of the congestion is moving towards traffic upstream causing a net gridlock. This traffic flow phenomena appears clearly during the peak periods, that is when the traffic volume on the road is high.

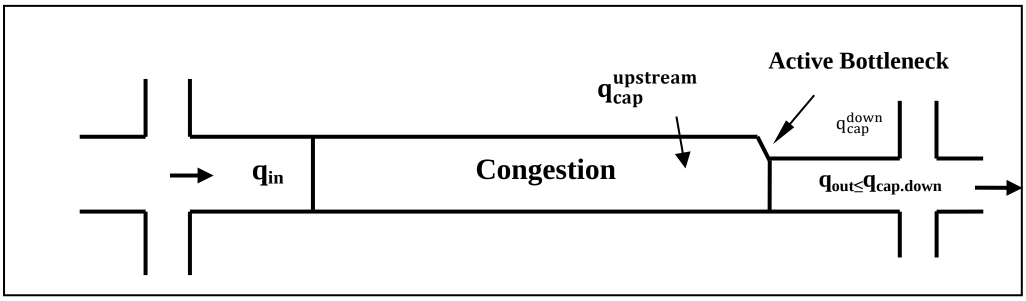


Figure 2.20 Bottleneck activation view of roadway without using VSLs

Basically, the new approach is based on controlling the arrival traffic (q_{in}) of bottleneck using variable speed limits technique in order to form an optimal traffic flow performance for travel demand. A local aspect for this approach is illustrated in Figure 2.21.

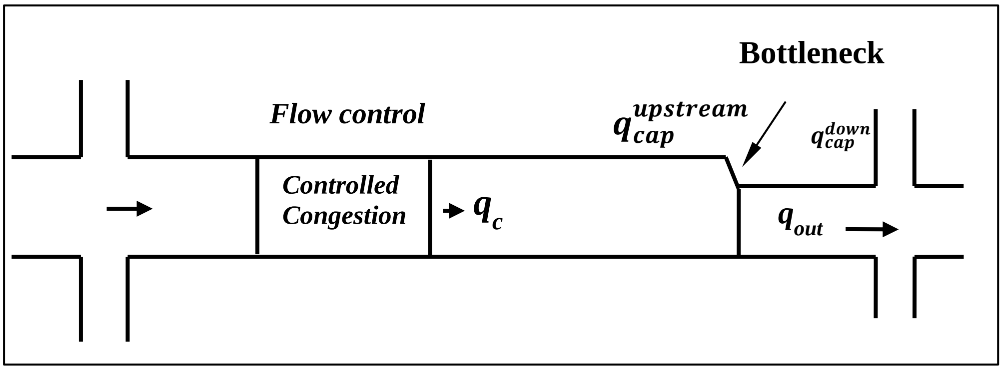


Figure 2.21 Schematic view of roadway traffic flow control using VSLs

The bottleneck indicated in Figure 2.21 does not cause up stream congestion (and no control strategy is needed) as long as $q_{in} \leq q_{cap}^{down}$. Once q_{in} grows larger than the bottleneck capacityq_{cap}^{down}, the bottleneck will be activated without any control strategy as seen previously in Figure 2.20. The primary aim of the new approach is to control the flow q_c to be equalized to the bottleneck capacity. This can be achieved by implementing the variable speed limits (VSLs) to control the amount of trips on the main roads, i.e. decreasing the traffic volume by reducing vehicles' speed. A roadway congestion will be formed upstream of the roadway traffic flow control location where vehicles approaching this area with relatively low speed related to controlled upstream traffic q_c. Traffic flow can be enhanced in the urban areas by VSL and the reduction in the speed might result in decreasing the total delay of the traffic. In other words, there will be an increase in the performance of the traffic. Hegyi et al. (2005) found that the congestion after a breakdown has a reduction in the outflow by about 5-10%. Consequently, any control strategy improving the breakdown may attain 5-10% improvements in the outflow rate.

2.4.8 Mathematical approach of VSLs in urban areas

The macroscopic description of traffic flow implies the definition of adequate variable expression the aggregate behaviour of traffic at certain times and locations. The discrete time step is denoted by T in second where m is divided into N_m segments of equal length L_m in meter as indicted in Figure 2.22. The traffic in each segment i of link m at discrete time $t = kT, k = 0,1 \ldots,$ is macroscopically characterized via the following variables:

1. Traffic density $\rho_{m,i}(k)$ (veh/km/lane), is the number of vehicles in segment i of link m at time $t = kT$ divided by L_m and by the number of lanes,
2. Mean speed $v_{m,i}(k)$ (km/h), is the mean speed of the vehicles included in segment i of link m at time $t = kT$, and
3. Traffic volume or flow $q_{m,i}(k)$ (veh/h) is the number of vehicles leaving segment i of link m during the time period $[kT, (k + 1)T]$ divided by T.

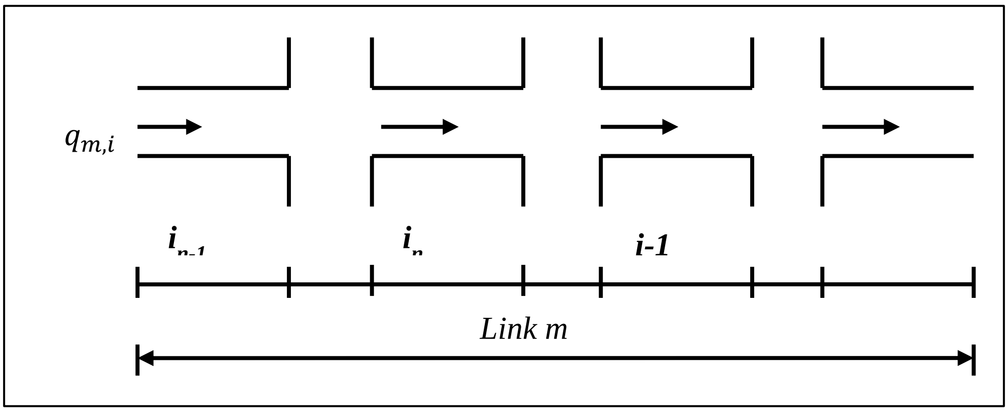


Figure 2.22 Schematic view of traffic flow details

Basically, the new model is based on the evaluation of total delay during any critical periods with reference to the utilization VSLs. The simulation of this concept can be conducted using VISSIM software. Usually, when the travel demand (q_{in}) becomes higher than the q_{cap}^{down} in the main roads, VSL can be applied to avoid forming the bottleneck by forcing lower speeds (critical speed, V_{cr}) in the critical section. The critical speed value acts as a benchmark to organize the entire preceding traffic towards the critical area via sending a variable signal message (VSM) at each intersection. Generally, the speed will be decreased gradually over the whole road until the critical speed that caused the bottleneck is reached. Therefore, the entire delay created by this strategy which includes three stages, will be adopted to compute the total delay of vehicles utilizing the roadway as below:

1. Compute the critical speed that leads to the formation of a bottleneck in a segment i (critical segment).

$v_{cr} = \frac{q_{out,i} - q_{in,i}}{k_{out,i} - k_{in,i}}$ 2.45

where,

v_{cr} = critical speed that causes bottleneck at segment i,

$q_{out,i}$ = discharge rate for segment i ,

$q_{in,i}$ = traffic demand for segment i and

$k_{out,i}$, $k_{in,i}$ = traffic density for out and in respectively, for segment i

2. After determining the critical speed that causes bottleneck for segment i, this speed will be used to organize the inbound traffic on each of the other segments preceding the segment i.

$v_{i-1} = Min[v_{cr} + 5km/hr, v_{free,i}]$ 2.46

$v_{i-n} = Min[v_n + 5km/hr, v_{free,i}]$ 2.47

where,

v_{i-1} = controlled speed at segment i-1that close to critical segment,

v_{i-n} = controlled speed at final segment on link m, and

$v_{free,i}$ = free flow speed for segment i

3. After computing a new controlled speed value for each segment of link m, delay can be estimated in three stages, as described below:

Stage1: Delays caused by reducing speed limit for links approaching critical segment.

$D_{app,i} = \sum L_i (\frac{1}{v_i} - \frac{1}{v_{free,i}})$ 2.48

where,

$D_{app,i}$ = entire approached delay due to reducing speed in each segments in sec, and

L_i = length of each segment in meter.

Stage2: Delays due to acceleration of vehicles after the critical segment.

$D_a = \frac{(v_{free,i} - v_{cr})^2}{2a v_{free,i}}$ 2.49

Where,

D_a = delay from acceleration after passing critical segment,

a = vehicle acceleration value (constant value),

v_{cr} = critical sped that causes bottleneck.

Stage3: Delays caused by stochastic queuing (delays due to waiting time in queue).

$D_{st,i} = \frac{q_{in,i}}{q_{out,i}(q_{out,i} - q_{in,i})}$ 2.50

where,

$D_{st,i}$ = stochastic delay (hr).

Eventually, the total travel time estimation using VSL application includes three components, delay from reducing speed limits in segments approaching the critical segment ($D_{app,i}$), delay from acceleration of vehicles after passing the critical segment (D_a) , and delay yielded from stochastic queuing ($D_{st,i}$). This result assumes that delay presented from traffic lights is constant, in order to facilitate understanding the VSL module. This value will be compared with the base value of delay without any control measures to reveal the improvement of using VSL on decreasing the delay during the peak period. Figure 2.23 shows a flow chart for the VSL process.

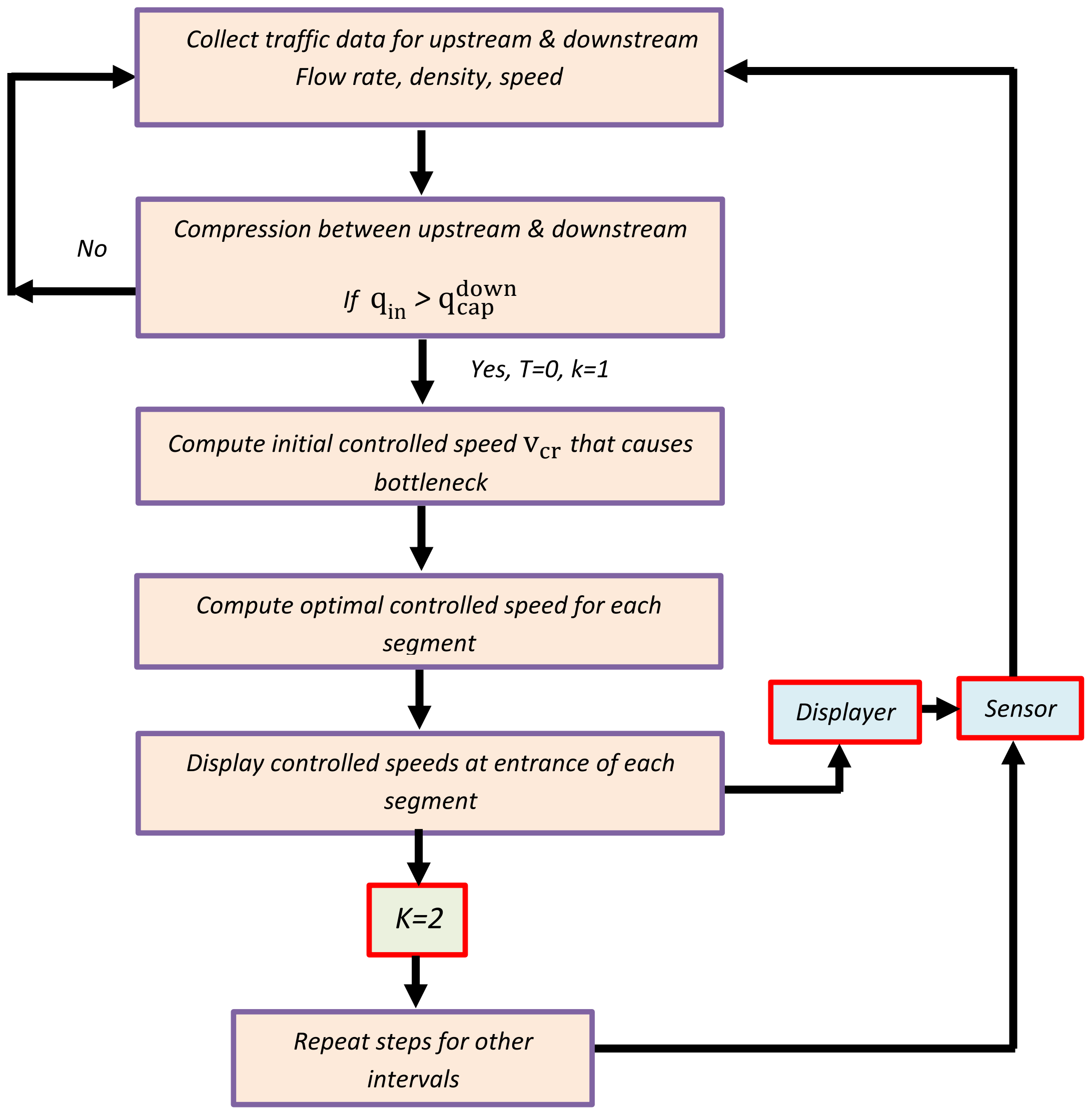


Figure 2.23 VSLs control algorithm

2.5 Summary

From the extensive review of the literature, it has been concluded that the issue of traffic congestion in urban areas remains a continuing problem and that much research is ongoing. Previous studies of congestion in urban areas have involved pricing, and transfer of congestion. Research has aimed of reducing the inlet traffic towards the congested links during the peak period that result in maximising traffic throughput, minimising delay and enhancing traffic safety. The available research on Variable Speed Limit controls has shown their potential for reducing congestion on motorways. However, VSLs has not been used on urban arterial road networks to alleviate congestion, as an urban road network is a more complex control challenge than a motorway. This research helps address that challenge.

CHAPTER THREE

Survey & Data Collection

3.1 Introduction

This chapter provides the rationale for selecting the Griffith Arterial Road (GAR) in Brisbane as a case study area for the present research. It also explains in detail the methods used to collect the relative traffic data.

Three sources of data were integrated to evaluate the traffic performance of the selected area. First, Synergised Transport Resources Ensuring an Advance Management System (STREAMS) data was obtained from the Queensland Department of Transport and Main Roads (QDTMR). STREAMS is based on a database of loop detector records. Second, field observations were conducted using video cameras and manual counting to determine vehicle types (truck, bus, and car), existing traffic volumes, traffic lane configuration and average headway at the signalised intersections on the GAR. Virtual Dub Software (VDS) was used to facilitate counting traffic volume and to precisely identify the traffic flow configuration. Third, a floating car survey (FCS) along the study area was implemented to estimate travel times on the arterial road. Based on estimated travel time data and VISSIM (Verkehr in Städten-Simulations) software features, optimum offsets between the signalised intersections were identified. Figure 3.1 illustrates the design of the survey that was conducted in the study area. A full description of the data collection, methods and traffic survey procedures are discussed in the following sections.

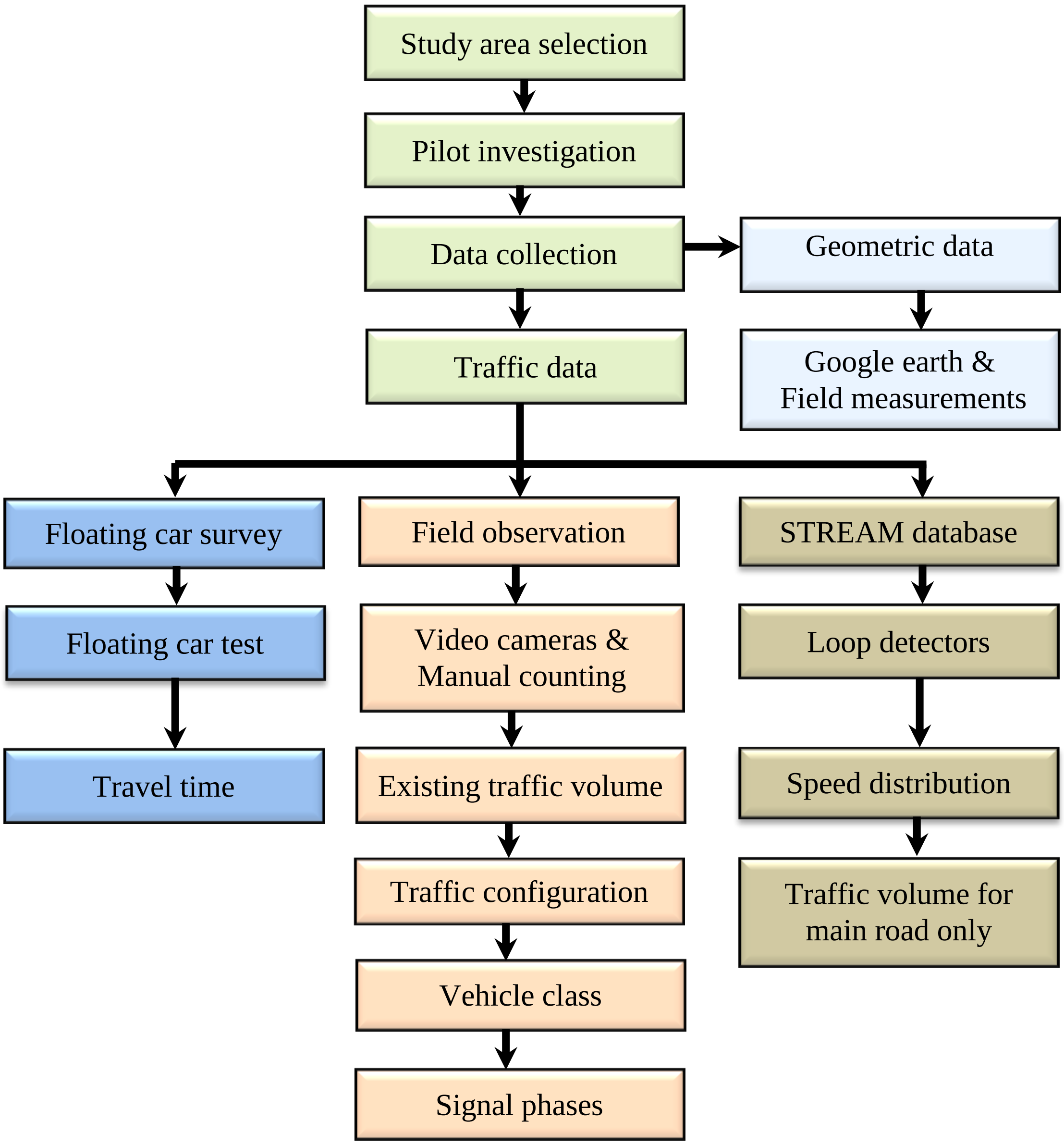


Figure 3.1 Survey design

3.2 Selection of study area

Griffith Arterial Road U20 is part of the Brisbane Urban Corridor (BUC), and is approximately 11.5 km long and lies between the Gateway Motorway and the Ipswich Motorway. It is part of the Australian National Land Transport Network. The arterial road is under the jurisdiction of the Queensland Department of Transport and Main Roads (QDTMR). The corridor consists of three main sections Granard Rd, Riawena Rd, and Kessels Rd and involves several signalised intersections. These are the intersections of Granard Rd - Beatty Rd, Granard Rd - Beaudesert Rd - Riawena Rd, Riawena Rd - Perrin Place, Riawena Rd - Orange Grove Rd, Kessels Rd - Troughton Rd, and Kessels Rd - Mains Rd. Figure 3.2 (a),(b) show the location of U20 captured from Google earth website. Figure 3.3 shows the layout of the study area and Table 3.1 provides the names of intersections along Griffith Arterial Road U20. Table 3.2 shows the number of lanes at each approach of the intersection.

(a)

(b)

Figure 3.2 U20 Griffith Arterial Road, Brisbane, Queensland (QLD), Australia

According to the field observations and QDTMR data, the eastbound (EB) and westbound (WB) directions of this corridor experience heavy congestion during the morning and evening peak traffic periods especially at the intersection of Granard Rd - Beaudesert Rd - Riawena Rd and Kessels Rd - Mains Rd.

The average daily traffic volume count (ADT) for this corridor is approximately 18,000-24,000 vehicles. The distances between intersections varies and ranges from 0.4 km to 1.5 km. The two ends of the corridor have closely spaced signalised intersection approximately 400m apart whereas in the middle section of the corridor the intersections are approximately 1 km to 1.5 km apart. The speed limit varies along the length of the road. The shorter links have speed limits of 60 km/hr and the longer links have speed limits of 70-80 km/hr.

The number of lanes at approaches to the signalised intersections varies from 1 to 4. In addition, the arterial links between the intersections have from 1 to 3 lanes in each direction. Multiphase traffic signals control all intersections along U20, with a cycle length 120-160 seconds during the daytime. The study area had high volumes of heavy trucks (3% to 18%) during the survey period especially on the EB and WB lanes. This may be due to the adjoining land uses. The area is characterised by multiple land uses including residential, heavy and light industry, retail and commercial activities, and freight companies.

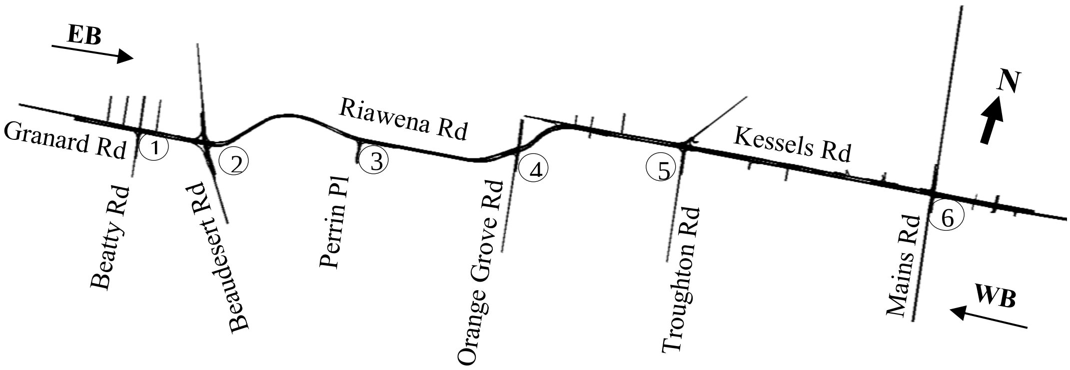


Figure 3.3 Study area layout

Table 3.1 Intersection names

Intersection Number	Intersecting Roads	Distance to next intersection (m)
1	Granard Rd- Beatty Rd	400
2	Granard Rd - Beaudesert Rd - Riawena Rd	1015
3	Riawena Rd - Perrin Place	978
4	Riawena Rd - Orange Grove Rd	1062
5	Kessels Rd - Troughton Rd	1541
6	Kessels Rd - Mains Rd	639

Table 3.2 Number of lanes

Intersection Number	Number of lanes at each approach							
	NB		SB		EB		WB	
	TH	RT	TH	RT	TH	RT	TH	RT
1	1*	1	1	1	3	1	2	1
2	3	1	3	1	2	2	3	1
3	0	2	0	0	2	1	2	0
4	2	0	2	1	3	1	3	0
5	1*	1	1	1*	3	1	3	1
6	3*	1	3*	1	3	1	3	1

* Refers to a shared TH/RT lane

Key characteristics that lead to the selection of Griffith Arterial Road U20 as a case study are:

1. The availability of traffic data, provided from QDTMR assisted in understanding the traffic situation along the road.
2. The strong support received from QDTMR to carry out the field traffic survey.
3. It is a major urban arterial road with periods of high congestion.
4. It has high daily traffic volumes.
5. The diversity of adjoining land use activities may have a negative effect on the performance of proposed traffic scenarios for the road. This provides an opportunity to examine the efficiency of any proposed traffic management actions.
6. Short links with recurrent congestion (morning/evening peak) and long links with high speeds occurring at the same times satisfied the traffic pattern requirements for testing the proposed management plan.

7. Traffic management strategies were already implemented at the site such as adaptive traffic control and coordinated traffic systems however, some places still experienced traffic degradation due to high traffic demand. New traffic management strategies had the potential to alleviate the risk of traffic degradation.

3.3 Data Collection

Traffic data was collected from the study area on Wednesday, July 11, 2012 for the peak period between 3:30 pm and 5:30 pm with the cooperation of QDTMR. Traffic flow, traffic distribution, vehicle type and traffic headway were collected by using high-resolution video cameras at each intersection. Public holidays and school holiday were avoided. Data were collected from three sources as explained in the following sections.

3.3.1 QDTMR database

Initially traffic data regarding the study area was collected from the Queensland Department of Transport and Main Roads (QDTMR) database. QDTMR uses an adaptive traffic control (ATC) system STREAMS, which is based on detector records. For the GAR these detectors were placed on the main eastbound (EB) and westbound (WB) traffic streams at a distance of 35-40 meters from the intersection stop line. STREAMS data were collected for five days (Tue 7, Wed 8, Thu 9, Tue 14 & Wed 15 Sep 2010) from 07/09/2010, 12:00:00 am to 15/09/2010, 12:00:00 am. Data was collected every 2 seconds and included traffic volume, expected cycle length, posted speed, expected travel time and occupancy.
This data was used to select the appropriate day for the survey, identify the heaviest peak traffic periods for morning and evening intervals and determine the speed distribution along the road. These issues are discussed briefly in the following sections.

3.3.1.1 Selection of survey day

In order to identify which day had the highest traffic volume, STREAMS data was analysed to find the highest daily traffic over two weeks of data. This step is important because the application of VSLs is recommended for use during congestion periods and when the volume to capacity ratio (v/c) approaches 1, otherwise the application will not be effective.
Table 3.3 shows the distribution of average daily traffic (ADT) along the EB arterial road during five normal workdays. The EB direction includes five arterial links (Beatty-Beaudesert link, Beaudesert-Perrin link, Perrin-Orange link, Orange-Troughton link and Troughton-Mains link). In this table, Thursday and Wednesday have the highest traffic counts followed by Tuesday. Thursday has one period of high traffic volume while Wednesday has two periods of high traffic volume. Since Wednesday has two high traffic volume periods and is close to Thursday traffic volume it was selected as the traffic survey day.
Figure 3.4 shows the average daily traffic for five days. The link experienced only a small variation in ADT over the five days indicating that it was operating with nearly constant traffic demand during the survey period. Thursday's data had the highest ADT for all links followed by Wednesday and lastly Tuesday. Thursday and Wednesday were chosen as the days on which to collect data.

Table 3.3 ADT for two weeks survey

Link	EB Direction	STREAMS data surveying				
		1	2	3	4	5
		Tuesday 7th Sep 2010	Wednesday 8th Sep 2010	Thursday 9th Sep 2010	Tuesday 14th Sep 2010	Wednesday 15th Sep 2010
1	Beatty-Beaudesert	9927	11174	11807	10553	11600
2	Beaudesert-Perrin	16727	17453	18146	17146	17757
3	Perrin-Orange	13255	14035	14429	13236	14461
4	Orange-Troughton	17305	17791	17920	16685	17677
5	Troughton-Mains	15061	16375	16846	15653	16580
ADT		14455	15366	15830	14655	15615

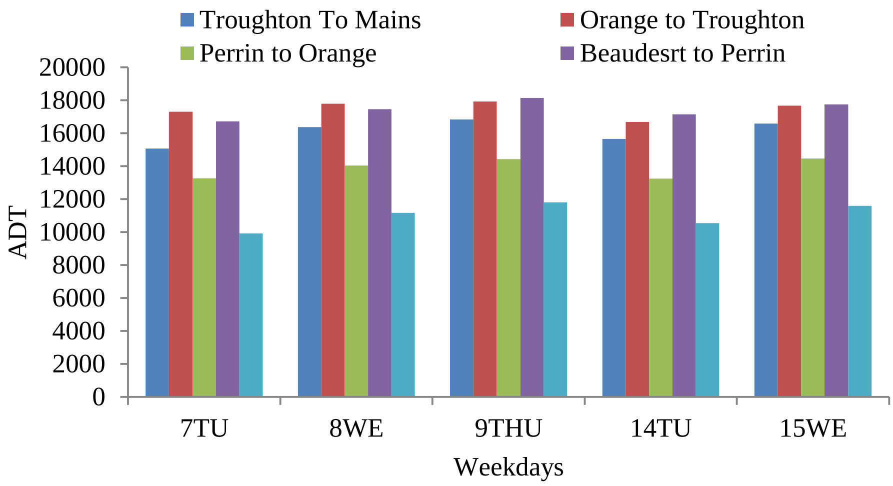


Figure 3.4 ADT for EB arterial links

3.3.1.2 Determination of peak periods

Because VSL was planned to be used to optimise traffic performance, it was important to select the peak periods of traffic flow. In order to capture accurate variations in traffic flow, count intervals of 15 minutes were used. This interval is recognised as being the shortest interval for which a stable flow exists.

The data collected from the STREAMS database, calculated for each 2 seconds interval, was aggregated for each 15-minute period. Normally two peak periods are considered during the day, morning and evening. Figure 3.5 and Figure 3.6 show hourly traffic volumes for the EB morning period (MP) and WB evening period (EP) respectively. The EB traffic was investigated from 6:00 am to 12:00 pm and the WB traffic was investigated from 12:00 pm to 7:00 pm.

Figure 3.5 shows that the morning peak starts at 7:30 am and continues to about 10:15 am. It is dominated by a large number of commuters going to work during this period. In addition, the trend of volume variation is quite noticeable for the EB morning period and the WB evening period. The links that contribute a high volume to the EB direction showed less contribution to WB direction and vice versa. One of the characteristics of the U20 route is that it is more of a circumferential route, rather than a radial route. Therefore, peak hour traffic is not dominant in any one direction.

Figure 3.6 shows no clear WB evening peak. This occurs because the variation in traffic volume is small for long periods. This may cause small variations in hourly volume and less shift in travel behaviour where drivers try to travel during the off peak period to avoid encountering the peak period. However, the evening peak period is more concentrated and this concentration is mostly from 3:30 pm to 5:30 pm. It is dominated by commuters returning to their residences after their day's work. For study purposes, the evening period was selected as the primary data collection and modelling period.

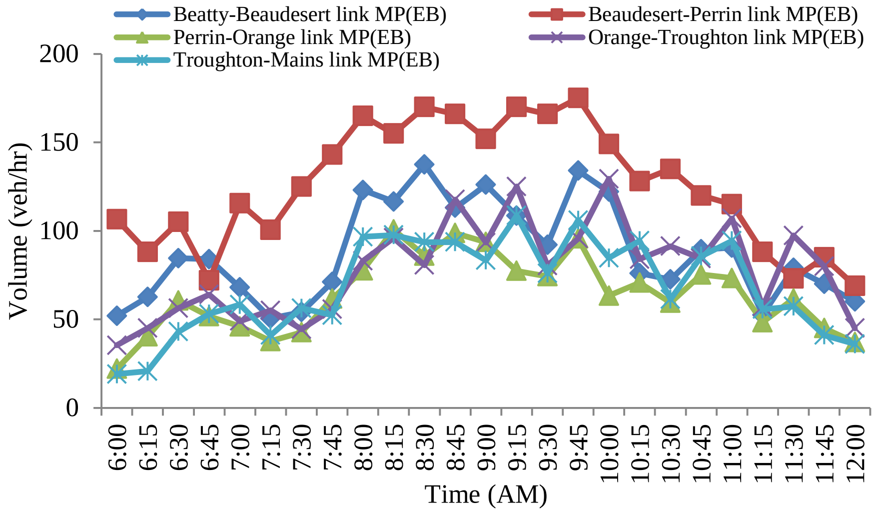


Figure 3.5 Variation in traffic volumes at the EB direction

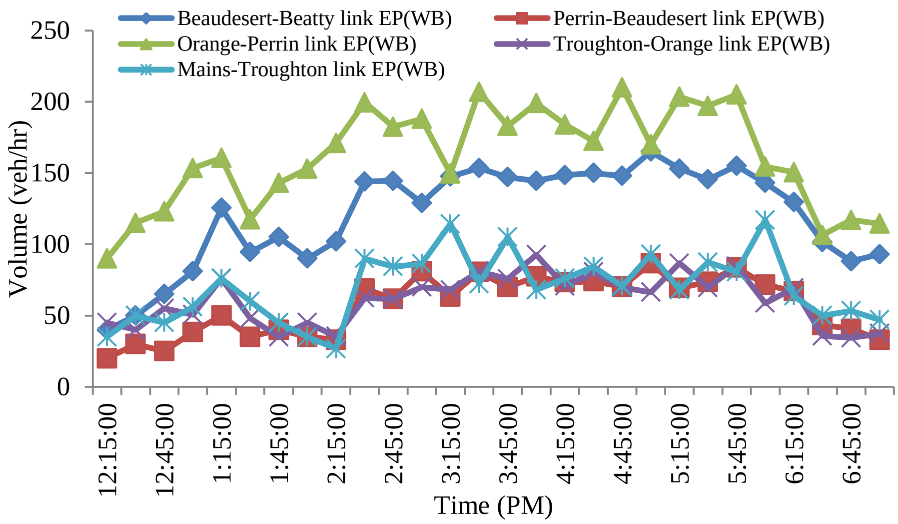


Figure 3.6 Variation in traffic volumes at the WB direction

3.3.1.3 Performance indicators in STREAMS system

In this section, some of the performance indicators for the road network operations are investigated. Usually, traffic performance data are collected a few times a year using the same methods for data collection and analysis. In recent years researchers have concluded that the limited amounts of survey data available are inadequate to present a true picture of prevailing traffic situations especially for mega cities with long peak traffic periods. Some jurisdictions have attempted measuring live traffic flow characteristics for freeways and arterials. Such measurements are based on using vehicle detectors in freeway management systems (FMS) and signal area traffic control (ATC) systems to identify traffic performance. Five indicators have been proposed to evaluate the performance of traffic conditions for both freeway and arterial control systems. These indicators are however considered controversial by

Troutbeck et al. (2007). A brief description and example calculations of these indicators is provided in the following sections:

1. Traveller Efficiency (Travel Speed): This parameter indicates the traffic congestion in terms of speed. On a freeway, spot speed of vehicles can be determined using sensors such as loop or microwave detectors. On arterial roads, it can be calculated using two methods, sensors and traffic signal systems. The spot speed information provided from sensors does not indicate the actual speed along the arterial road. This is because speeds vary along the arterial link and are significantly affected by the signal timing and the quality of platoon progression.

Normally speed calculations will be determined from signal area traffic control (ATC) by counting the inverse of travel time for vehicles that traverse the link. Estimation of travel times from signal setting or ATC requires converting the travel time to spatial speed in order to estimate the effect of signal settings on speeds. The calculation of travel speed can be explained as follow, (Troutbeck et al, 2007):

$$TS(i,t,d) = \frac{L_{i,t,d}}{ATT_{i,t,d}} \qquad 3.1$$

where,

$TS(i,t,d)$ is travel speed on link i at time t and survey day d,
L is the length of link i and
$ATT_{i,t,d}$ is the average travel time on link i at time t and day d.

In order to calculate the effect of flow on the speed of vehicles during the survey period, the flow for each link along the network was considered in estimating the average travel speed.

The average travel speed (*ATS*) for a network can be calculated as:

$$ATS = \frac{\sum_{i,t,d}(L_i V_{i,t,d})}{\sum_{i,t,d}(L_{i,t,d} V_{i,t,d} / TS(i,t,d))} \qquad 3.2$$

where, $V_{i,t,d}$ is the flow on link i monitored in time t and day d.

2. Traveller Efficiency (variation from posted speeds): This parameter calculates the deviation from the posted speed limits for the road network. The deviation in travel speed can be computed as :

$$SpeedVariation(\%) = (PostedSpeed - AverageSpeed)*100 / PostedSpeed \qquad 3.3$$

The average speed can be determined from on the spot speeds on the freeway or as the inverse of travel time obtained from the ACT on arterial roads.

3. Reliability (travel speed): This indicator measures the variation in speeds based on the coefficient of variation (the standard deviation over the mean). It can be displayed as the proportions of the road network experiencing different levels of variability per unit time. Customarily, the travel times for arterial roads can be obtained from the signal settings of the ATC system and their reciprocals are utilised to estimate the speed reliability. The traffic flow has various route destinations, so the route variability can be determined as follow:

$$\text{Route variability } RV_k = 1.44 \times \left(SD_k \ / \ T_k \times 100 \right) \qquad 3.4$$

Where T_k is the mean travel time on route k measured in units of min/km and the SD_k is the standard deviation of travel times on route k in (min/km).

$$T_k = \frac{\sum_{n=1}^{N} t_n}{N} \qquad 3.5$$

$$SD_k = \sqrt{\frac{\sum_{n=1}^{N} (t_n - T_k)^2}{N-1}} \qquad 3.6$$

where,
N= is the total number of travel time samples t_n on route k.
1.44 = represents the calculation factor for 85% confidence interval.
The sample size for the calculation is usually 15 min for all surveys.

4. Traffic productivity: This indicator represents the product of speed and flow. High traffic productivity can be achieved when the speed indicates free flow speed and the flow reaches the traffic capacity. Low productivity occurs due to low traffic flow and/or low speed. In addition, low productivity can be attributed to low traffic demand and does not necessarily reflect poor performance for the road network. It is shown as the proportion of a network at various levels of productivity over time. The productivity can be illustrated as:

$$productivity = \frac{speed * flow}{speed_{nom} * flow_{nom}} * 100 \qquad if\ speed < speed_{nom} \qquad 3.7$$

$$= 100 \qquad if\ speed \geq speed_{nom} \qquad 3.8$$

Where, $speed_{nom}$ and $flow_{nom}$ represent the normalisation speed and flow for the road network before the productivity starts to decrease (traffic congestion).
Table 3.4 indicates the recommended normalisation values to identify optimal conditions. The productivity of speed-flow equal to or greater than 100 indicates that the network is operating at good level of service during the specified period.

Table 3.4 Speed and flow normalisation values

Parameters	Arterial road	Freeway
Normalisation speed	35 km/h	80 km/h
Normalisation flow	900 veh/h/lane	2000 veh/h/lane

Source: (Troutbeck et al. 2007)

To sum up, STREAMS data can be extracted from data detectors and is useful to identify peak time-periods and the appropriate days for conducting field surveys of

selected area. The present study used VISSIM software to analyse and investigate the effectiveness of VSL application. This software requires a lot of traffic information about the road network to execute the simulations correctly.
Data collected from detectors does not provide all the traffic data needed to test the proposed traffic scenarios. Field data collection is necessary because:

1- STREAMS data were collected for only the EB and WB directions and no data was collected for the intersecting SB and NB roads.
2- The traffic volumes counted by detectors represent the total volume travelling in that link. There is no count of the number of vehicles turning at each intersection.
3- Vehicle type (truck, car, bus) is not recorded within the detector database and vehicle composition needs to be known to determine the percentage of heavy vehicles (% HV).
4- The loop detectors database had some unrealistic traffic volumes which suggested the readings were unreliable for traffic analysis.

For these reasons, additional field data were collected to overcome the unreliable counter data and to complete the missing parts of the collected data.
The following sections discuss in details how the field data survey was conducted.

3.3.2 Field data

In order to gain accurate data about the traffic flow in the selected study area, manual counting and video counting were used to supplement STREAMS data. The procedures for collecting field data were as follows.

3.3.2.1 Pilot survey

Before undertaken the detailed survey in depth, a pilot survey of the selected study area was performed to identify the best locations, the number of video cameras needed and the number of observers required doing the manual counting. This step is crucial to minimise errors that could be made during the production survey, and to increase survey accuracy. The preliminary survey was conducted to determine:

1- The number of video cameras needed to conduct the survey and the number of video cameras required to be available as standby in case of malfunction in one of the cameras.
2- The time needed to install the video equipment on the site. A check was also made on the battery capacity of cameras, to ensure the video recording could be performed with no stoppages.
3- The number of persons needed to manage and control both the video monitoring and perform the manual counting at each intersection.
4- Safe locations for positioning each video camera.
5- The appropriate safe working requirements needed to minimise the risk to the survey staff.

3.3.2.2 Video cameras set up and manual counting distribution

Despite the recent advances in technology regarding traffic data collection, some traffic data still needs to be collected manually. Situations requiring manual surveys include short duration surveys or surveys focused on specific locations such as signalised intersections. The data collection for this study required two types of

surveying: manual counting conducted by hand counters, and video recording using High Definition Cameras (HDC).
Six video cameras were deployed along Griffith Arterial Road (GAR) U20. One camera was allocated for each intersection at a position which enabled coverage of all the traffic movements at the intersection. Some of the intersections are very wide and one camera was not capable of capturing all traffic movements at the intersection. The data missing in the video recording were always found to be the turning of vehicles. Therefore, manual counting during the video recording was implemented for some left turning movements. Both methods of data survey were used at intersections 2 and 5. Table 3.5 and Figure 3.7 outline the deployment of video cameras and manual counting for the study area.
A group of 18 people conducted simultaneous counts along the route. The large group were divided into two smaller groups: group number 1 was responsible for the video processing where two persons were allocated to each camera; and group number 2 was responsible for manually counting the missing left turn vehicles. Two people were allocated to record each turning location.

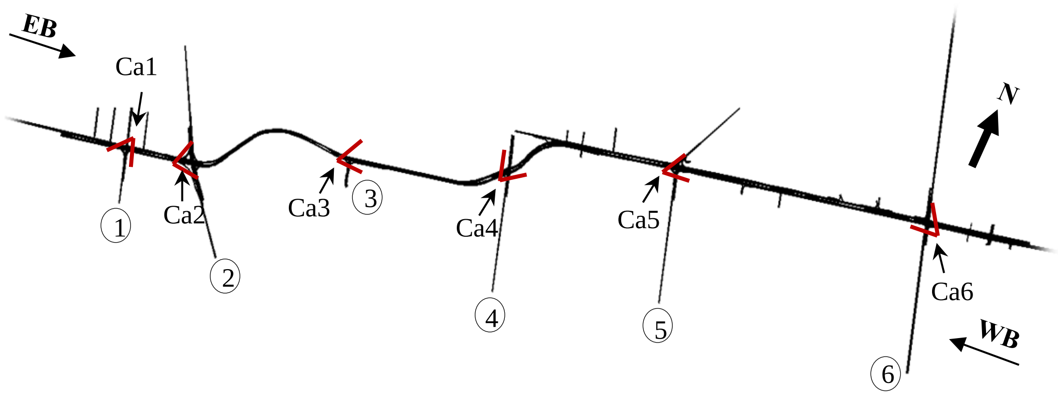


Figure 3.7 Video camera locations

Table 3.5 Video camera locations and manual traffic recording

Intersection No.	Intersection Name	Video Camera No.	Manual Counting
1	Granard Rd- Beatty Rd	1	None
2	Granard Rd - Beaudesert Rd - Riawena Rd	2	Left turn for EB and NB direction
3	Riawena Rd - Perrin Place	3	None
4	Riawena Rd - Orange Grove Rd	4	None
5	Kessels Rd - Troughton Rd	5	Left turn for EB
6	Kessels Rd - Mains Rd	6	None

In order to ensure accurate data collection, instructions were given to each survey group to start counting and videoing exactly at 3.30 pm and terminate processing at 5.30 pm. Video cameras and observers were positioned on site half an hour before the survey period to ensure good data collection.
All camera types used in the area were Sony models with high definition (1980× 1080 pixels) and high capacity capable of sustaining videoing for more than two hours. Each camera was mounted on a tripod at a height of 1.5 metres from the

ground. The video tapes of these intersections were used to get accurate volume counts and vehicle classification along the road network.

Tally sheets were used to record the left turning movements for intersections 2 and 5 at 15 min intervals. Since two observers were available to do this task, each recorder was responsible for counting either small vehicles or heavy vehicles. This process reduced the counters' errors by allowing them to concentrate on one vehicle type instead of two. The road geometry for these turning movements comprised only one travel lane. This made the situation for counting easy for the counters. Figure 3.8 shows a photo taken from the video recording for intersection 1. Video counts are considered more accurate than manual or loop detector counts because:

1- Manual counts may become inaccurate due to human errors associated with the counting process. Human counters may find it difficult to track more than one vehicle at a time when several different vehicle movements are involved. Also, some observers may not be able to count the movement of left on red because the counters were focused on counting vehicles turning movements for other directions.
2- Loop detectors are not necessarily accurate for counting high traffic volumes under congested conditions. Loop detectors are governed by a threshold value that establishes the presence of vehicles. Therefore, the accuracy of counting traffic volume will be affected by this threshold.
3- The ability to replay the videotape (more than once) provides the opportunity for increased accuracy from video counts because the human counter can focus on a single movement and once finished can replay the video and count other movements.

Figure 3.8 Snapshot photo for intersection 1

3.3.3 Field data processing

After videoing the traffic conditions along the study area, data processing is needed to determine the traffic volumes, vehicle types, traffic distribution and discharge headways at each individual intersection. The data processing was performed by using video software as outlined in the following sections.

3.3.3.1 Virtual dub video software

It is difficult to extract the traffic information precisely and directly from videotapes because the speed of display is relatively high, making traffic counting unmanageable. Therefore, Virtual Dub Version 1.6.15.0 was used to facilitate the data processing (Brust et al., 2009). Virtual Dub software (VDS) is a video capture and processing utility developed by Microsoft written in C++ language and compatible with 32-bit and 64-bit Windows systems. It is designed to deal with formats such as MP4, MOV, AVI, WMV, BMP and MPEG but not with Advanced Video Codec High Definition (AVCHD) format. Cameras with high resolution store the video file in AVCHD format therefore the video files obtained in this study needed to be converted to a format to match with the VDS. The benefit of using Virtual Dub is that it provides much more information than can be collected by pneumatic road tube detectors as up to 50 frames per second can be taken. It also provides more flexibility in processing data because the videotape can be stopped, rewound and replayed if some details are missed. The drawback in using this software is the large amount of time required and tediousness of the job. However, the results are of greater accuracy. Figure 3.9 shows an image for intersection 2 captured by Virtual Dub software.

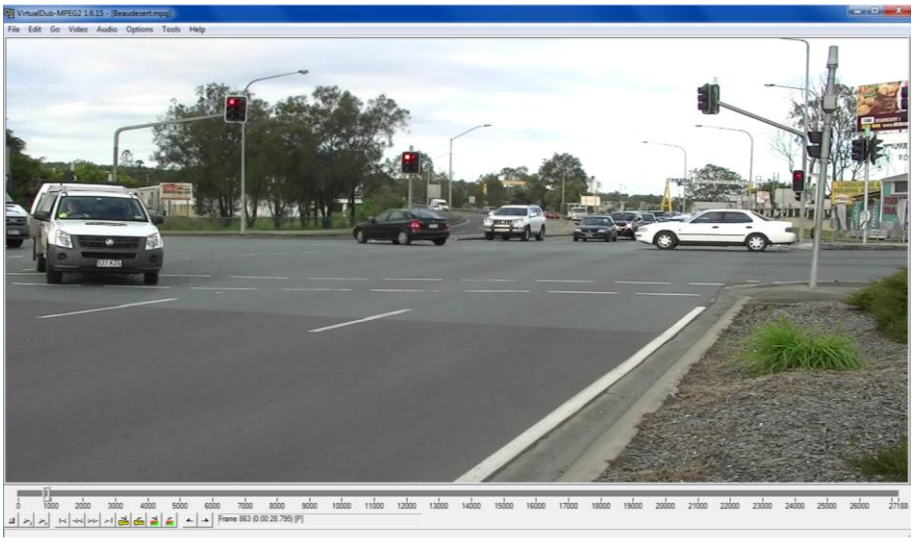


Figure 3.9 Virtual Dub snapshot for intersection 2

3.3.3.2 Field traffic volumes analysing

Traffic volumes were counted for 15-minute intervals for all the travel movements in the selected study area of U20. Data was aggregated to vehicles per hour for the two-hour survey. Vehicle class (Truck or car) were also recorded in order to determine the percentage of heavy vehicles (HV) in the network. The percentage of heavy vehicles fluctuated from link to link and from one intersection to another. This variation is due to the demography of the network. The demography of the study area is distinguished by commercial and industrial services. The percentage of heavy vehicles varied from 3% to 18%. The highest level of HV was in the NB direction for intersection 3 due to the large number of freight companies in the area around this intersection.

Figure 3.10 shows the traffic volume distribution for the two-hour survey during the evening peak for intersection 1. This volume was collected from the video record of camera 1. No manual counting was conducted at this site. The figure shows the EB and WB directions have the highest volume during the evening period whereas the NB and the SB directions have the lowest volume. This highlights that the EB and the WB directions are the main roads while the others directions are considered minor roads. The variation in traffic volume between EB and WB can be noticed during the first hour of the survey and this variation disappeared during the second hour when the two directions reached the same level of traffic volume. The variation in volume from one interval to another is between 5% and 10% during the evening peak. This shows that the intersection carries reasonable constant vehicular traffic during this time. The signals at the intersection are set to operate on a pre-timed setting during this period.

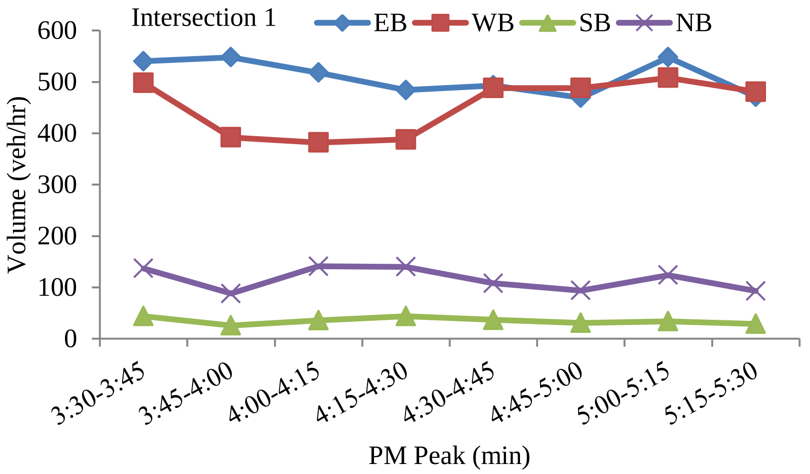


Figure 3.10 Traffic distribution at intersection 1

The traffic composition on the approaches to this intersection varied from approach to approach and over time. Figure 3.11 indicates the variation of traffic composition during the survey period for intersection 1. The rate of HV traffic changes from 4% to 12% over the four approaches. The high percentages are for the WB and EB directions during the first hour of survey (3.30-4.30 pm). During this period, HV traffic had more affect on the performance of traffic flow than during the second hour of the survey. The percentages of HV were 10% for the EB direction and 12% for the WB direction during 3.30-4.30 pm and 6% for both directions during 4:30-5:30 pm.

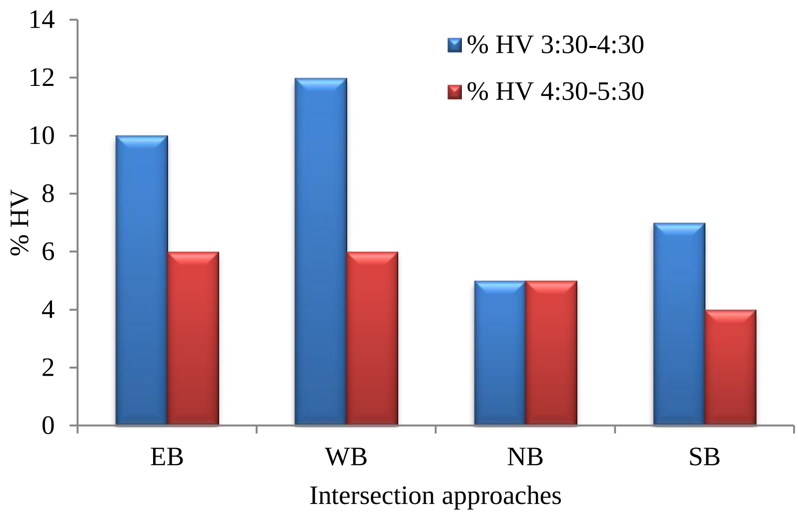


Figure 3.11 Variation of HV for intersection 1

Figure 3.12 illustrates the variation in traffic volume at intersection 2. The traffic flow for the EB and WB directions followed the same trend over the afternoon period. This means the traffic volume was equal along these approaches during this period. The EB and WB directions carried the highest volume in contrast to the SB and NB directions at intersection 2. The NB and SB directions also carry a heavy traffic volume during the time of the survey. The volume fluctuated between 300 and 500 vehicles per each 15 minutes. Traffic volumes for intersection 2 were more than for intersection 1 in all four directions. This confirmed the classification of intersection 2 as a major intersection by QDTMR.

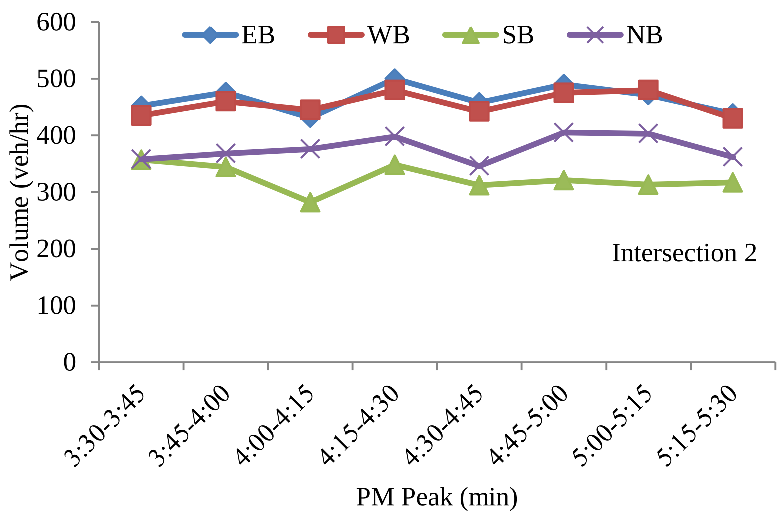


Figure 3.12 Traffic variations at intersection 2

Figure 3.13 illustrates the distribution of HV at intersection 2. The profile of HV for intersection 2 is similar to the profile for intersection 1 where the EB and WB lanes have the highest level of HV traffic. The rate of HV traffic for 3.30-4.30 pm ranges from 6% to 9% while the second hour experienced a decrease in the percentage of HV for all the lanes from 4% to 7%.

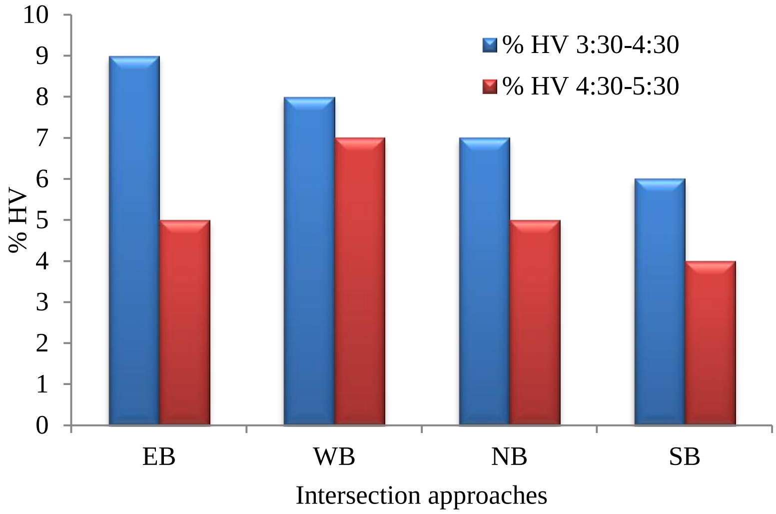


Figure 3.13 Variation of HV at intersection 2

Figure 3.14 shows the variation of traffic volume along three directions, EB, WB and NB, for Perrin Place intersection (Intersection 3). Since the intersection includes three directions of travel movement, it is considered a T- intersection design. The EB and WB lanes had the highest traffic volumes during the evening period. The NB direction had a low traffic volume. The reason for this is that the NB link does not have other nodes feeding into it. The only traffic came from the buildings, freight companies and car parks that surround this road.

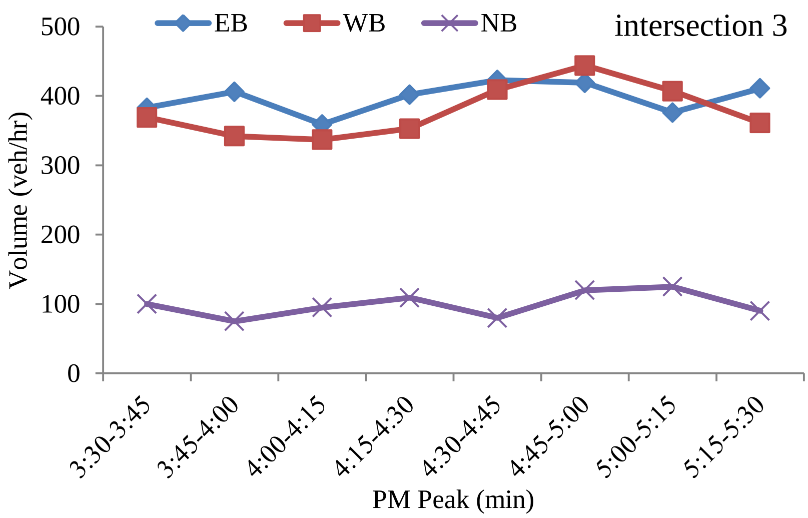


Figure 3.14 Traffic variations at intersection 3

Many freight companies are located in the area around this intersection. This results in an increased percentage of heavy vehicles especially on the NB approach of this intersection. Figure 3.15 shows the statistical profile of HV for intersection 3. The percentage of HV increased up to 18% during 3:30 to 4:30 pm and decreased to 12% during 4:30 to 5:30 pm for the NB direction. This percentage was the highest in the study area.

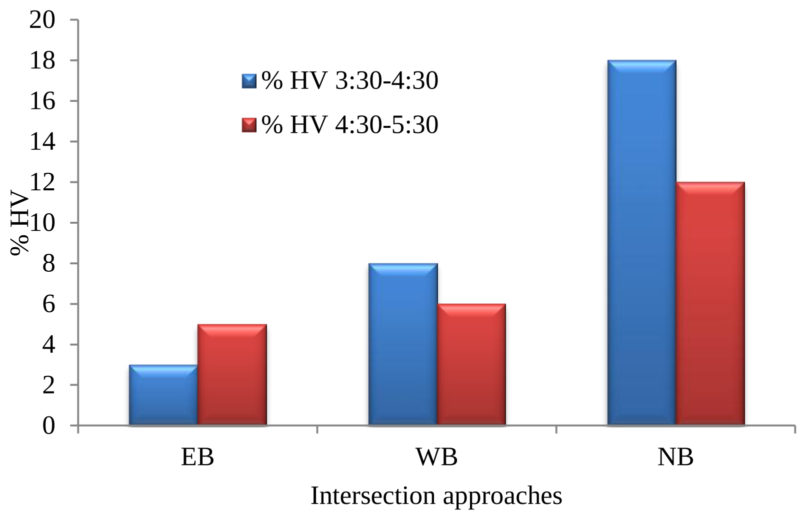


Figure 3.15 Variation of HV at intersection 3

Figure 3.16 illustrates the distribution of traffic volume for intersection 4 for the two-hour survey. The high traffic volumes can be seen to have spread out along the EB and the WB lanes whereas the NB and the SB lanes still have low traffic volumes. The differences in traffic volume between the EB and the WB lanes were 30% for the first hour and 10% for the second hour. The EB lanes had a higher volume than the WB lanes during the first hour. They had similar high volumes during the second hour. The variation in volume between time intervals (15 min) for all approaches was very small. This reflects the normal distribution of the traffic flow in an urban area. Consequently, the period of congestion may be spread out for longer on urban arterial roads.

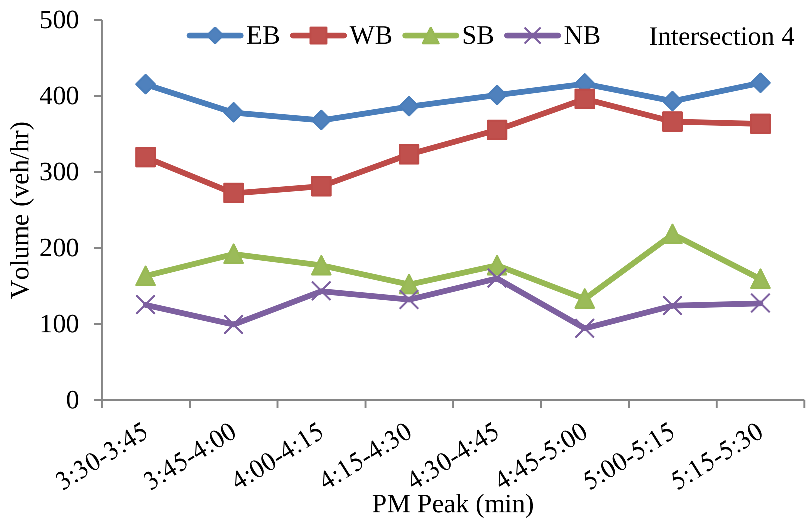


Figure 3.16 Traffic variations at intersection 4

Figure 3.17 illustrates the variation in percentage of HV at intersection 4. HV volume declined from 7% to 3% in the EB lanes during the second hour of the survey. The SB lanes retained the same 4% during the survey period. Overall, the percentage of heavy vehicles decreased for all approaches during the second hour of the survey except in the SB direction.

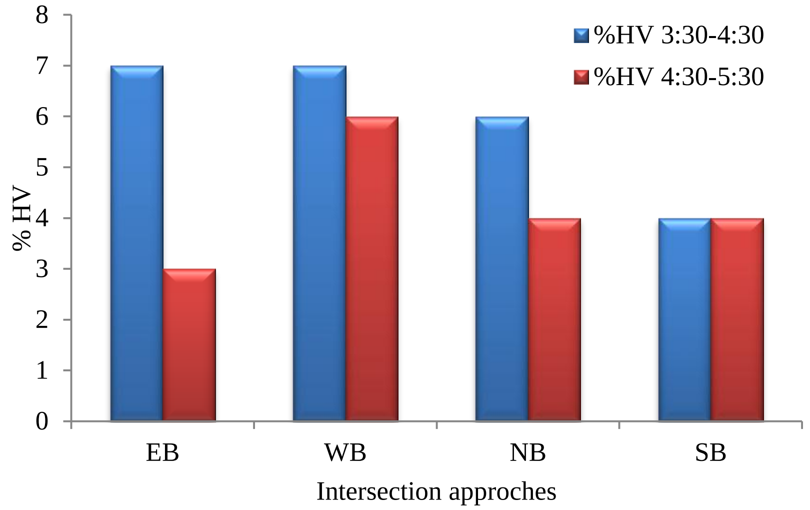


Figure 3.17 Variation of HV at intersection 4

The traffic distribution depicted in Figure 3.18 shows that the major traffic flow at intersection 5 was on the EB and the WB directions and the minor traffic flow was in for the SB and NB directions. The EB approach had a greater number of vehicles during the first hour and then decreased during the second hour. The NB and SB lanes had many fewer traffic movements than the EB and WB lanes.

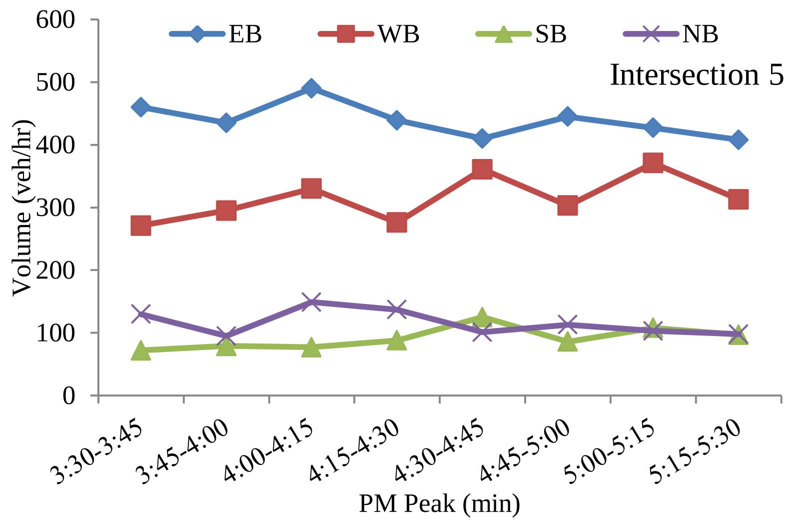


Figure 3.18 Traffic variations at intersection 5

Figure 3.19 illustrates the variation of HV at intersection 5. HV travelling in the EB direction decreased from 8% during the first hour to 3% in the second hour. Small variation occurred in the WB lanes where HV traffic dropped just 1% from the first hour to the second hour. This indicates that HV will continue to affect the performance of WB lanes for longer than the EB lanes. The movement of HV in the NB lanes showed little increase in the second hour where the percentage of HV increased by only 1%. There were no changes in the percentage of HV in the SB lanes during the second hour of the survey (4%).

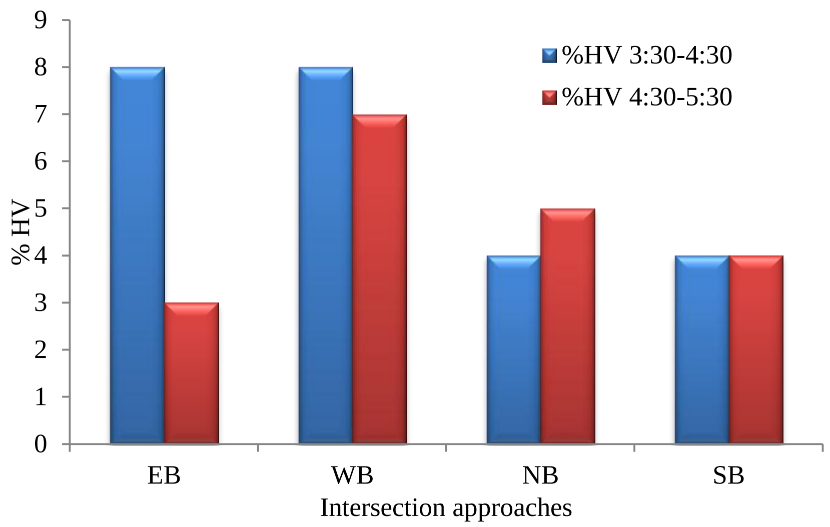


Figure 3.19 Variation of HV at intersection 5

Figure 3.20 shows that the traffic volumes were high on the EB, WB and SB lanes at intersection 6. The major traffic flow was EB and WB but at this intersection the traffic volume for the SB lanes also recorded a high volume and is considered as a major source of traffic flow. This is attributed to the SB lanes being connected to the Brisbane motorway. The intersection experienced heavy traffic volumes on all approaches during the survey period and this situation made it the most congested intersection after intersection 2. Traffic volumes for the EB lanes were higher than the WB lanes during the first hour of the survey.

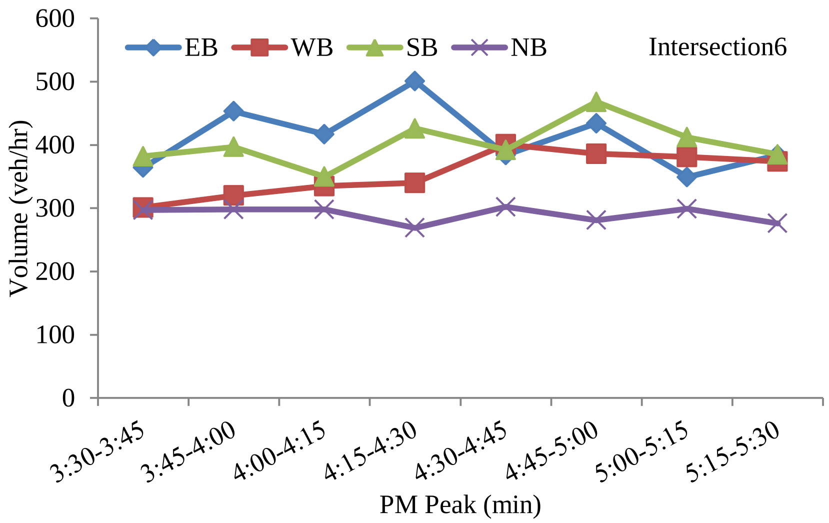


Figure 3.20 Traffic variations at intersection 6

Figure 3.21 illustrates the distribution of HV at intersection 6 during the PM survey. HV constituted the highest percent of the traffic volume in each direction during the first hour (3-7%) and lower in the second hour (2-3%). The WB lanes had the heaviest percentage of HV (5-7%) during the two hours of the survey and this may affect the traffic efficiency for these lanes.

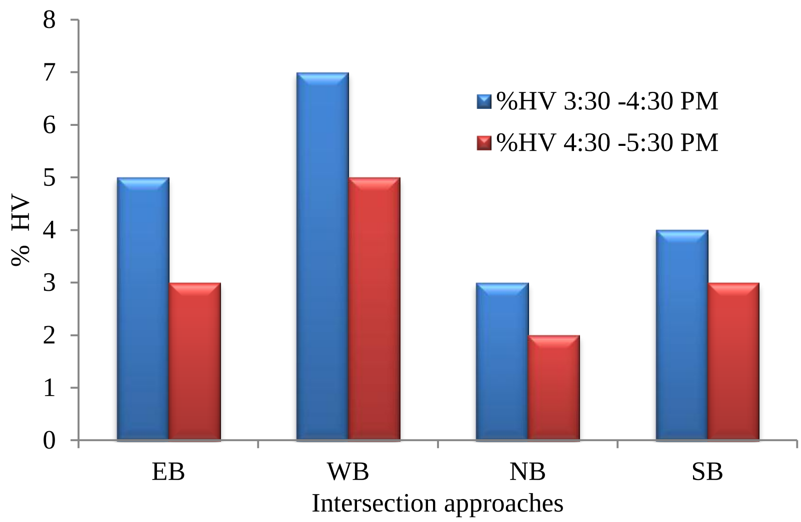


Figure 3.21 Variation of HV at intersection 6

The flow in the WB direction is the heaviest volume and the EB direction is the second heaviest during the two-hour survey. These two directions had the major traffic flow during the study and the NB and SB direction had the minor traffic flow. Intersections 2 and 6 had heaviest traffic volume for all approaches during the two hours of the survey.

3.3.3.3 Saturation headway identification

The advantage of video recording for traffic survey work is its ability to identify the saturation rate at signalised intersections. The headway between vehicles during the green interval is recorded using Virtual Dub software. The computation of headway is based on the time that vehicles need to cross a reference line such as the stop line, and clear the intersection. The headway for the first vehicle in the queue was recorded as the time when the green interval started and ended when the rear bumper of the vehicle crossed the reference line. For the remainder of vehicles, headway was recorded once the rear bumper of each vehicle crossed the reference line. Vehicle headways are measured in units of time, usually in seconds. In this study, average headways were considered instead of maximum headways, as this is a more realistic consideration during congested flow. The process was measured for different cycle lengths during the same rush hour. The headway is computed for the through movement and the right turn movement to detect differences in headway values for both EB and WB directions.

Determination of saturated headway is very important in identifying the real capacity of a signalised intersection during the peak traffic hours. As long as the headway calculations are accurate, the identification of field-signalised capacity will also be accurate. Figure 3.22 and Figure 3.23 show the variation of saturated headway for different signal cycles in the EB lanes of intersection 2. These figures show that the headway between the first four vehicles decreases and beyond the 4^{th} vehicle, the headway fluctuates up and down. The first four vehicles have the highest headway at the initiation of the green interval compared to others vehicles queued at the intersection. This is due to the time taken to respond to the signal change and to the mechanical characteristics of the vehicles. However, in the field many vehicles showed higher headway values than the first four vehicles due to factors such as the vehicle type, vehicle properties, and driver behaviour. Consequently, the average headway computed for different cycles is 2.1 sec for the EB lane through movements and 1.96 sec for the EB lane right turn movements. Figure 3.24 and Figure 3.25 show the average headway for the EB right turn movements. Figure 3.26 and Figure 3.27 show the average headway of WB through movements which were 1.98 sec. Overall it was found that, the average headway for all approaches is close to 2 sec. This means that the study area is operating under high traffic demand.

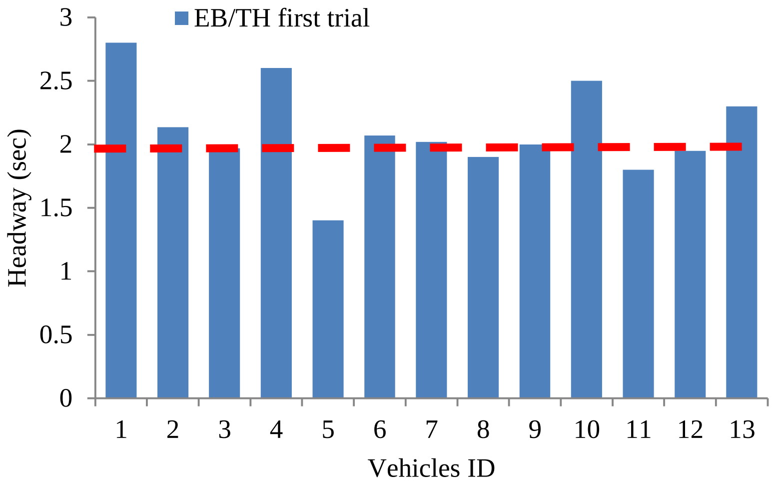


Figure 3.22 Saturated headway at EB of intersection 2, first trial

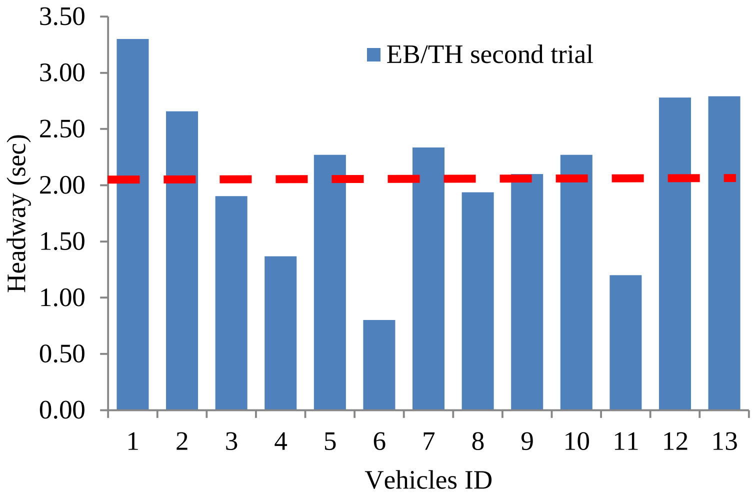


Figure 3.23 Saturated headway at EB of intersection 2, second trial

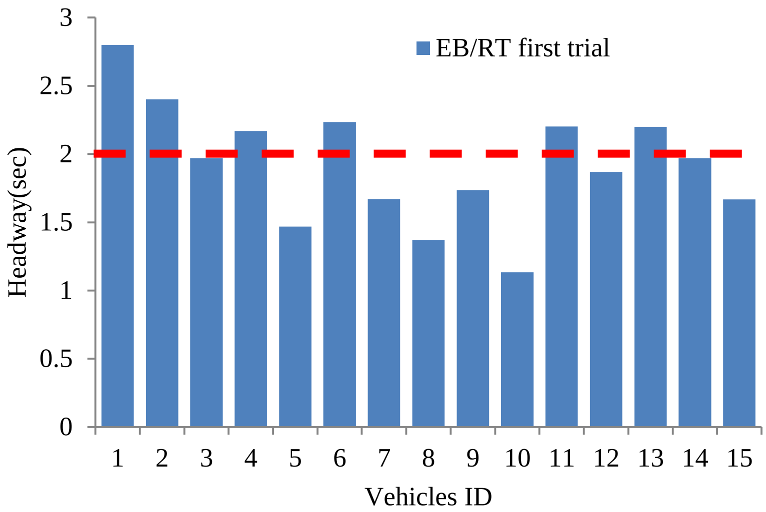


Figure 3.24 Saturated headway at EB of intersection 2, first trial

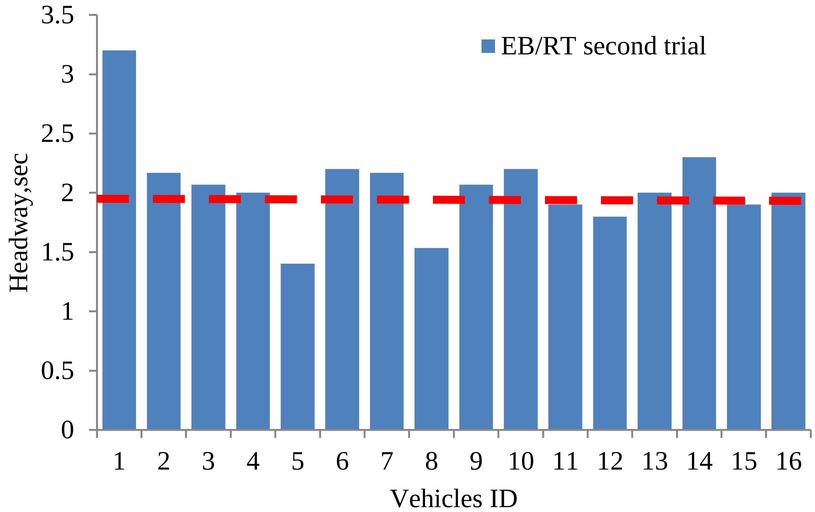


Figure 3.25 Saturated headway at EB of intersection 2, second trial

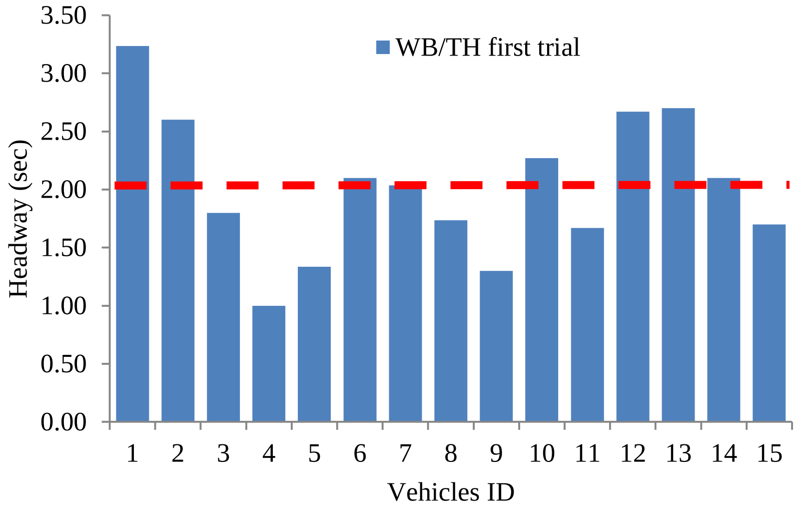


Figure 3.26 Saturated headway at WB of intersection 2, first trial

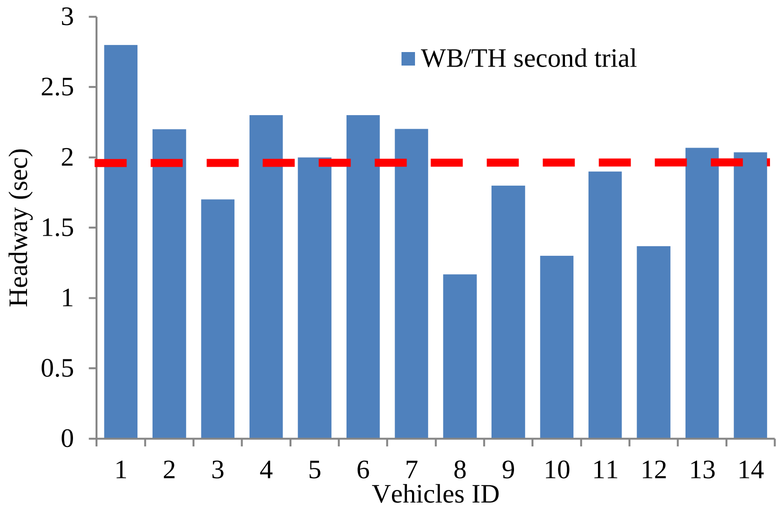


Figure 3.27 Saturated headway at WB of intersection 2, second trial

3.3.3.4 Signal time setting and traffic phases

Data about the timing plan and signal phasing for the study area were required to complete the traffic analysis. Videorecords and field observations were made to obtain this data. Virtual Dub Software (VDS) was used to find the signal time setting due to its accuracy in extracting the changes in traffic signal conditions for times less than one second. The cycle length (C) for red intervals (R), all red periods (AR), green intervals (G) and amber times (A) was recorded for six signalised intersections as shown in Table 3.6. In addition, the phase signal plan for each intersection was recorded by tracking vehicle traffic movements in each lane. This was performed by viewing the videotape, recording the behaviour of each signal and then matching the details with the field observations. Table 3.6 shows the signal-timing plan for all intersections involved in the study. The timing plan for the through and right turn movements were recorded for all intersections. The timing plan for the left turn movement is not shown in this table either because the intersection doesn't have a left turn movement or the left turn is incorporated with the through movement timing. For example when the left turn lane is shared with the through movement phase and when left turn is permitted on the red signal.

All the investigated intersections have four approaches EB, WB, NB and SB except intersection 3 which has three approaches. The cycle length over the survey period varies at the intersections ranging from 120 sec to 160 sec. The biggest C for intersection 3 was created in order to ensure the continuity of flow for vehicles travelling along the EB and WB with fewer stops since the NB had low traffic demand if compared to other 4-leg intersection design. Also, the cycle length at 3-leg intersection design produces less delay to the traversed vehicles than the 4-leg intersection design. The all red intervals detected for all intersections were 2 sec.

The blank spaces indicate that no movement was available for that direction. Some right turn movements were also permitted such as in the WB lanes of intersection 1 and some others have protected-permitted turns such as in the EB lanes of intersection 1 and 3. Signal timing and traffic phases are essential steps in modelling a precise case study using VISSIM software. Therefore the data extracted from the videotape was continuously checked with field observations to ensure its accuracy. Figure 3.28 shows a simple schematic for the traffic operation system along the studied area. Several traffic signal phases were created in the selected area to control and manage traffic movement. Intersections 1, 4 and 6 involved four traffic phases whereas intersections 2 and 5 included five traffic phases and intersection 3 had only three traffic phases. The solid arrows represent protected traffic movements while the dashed arrows refer to permitted traffic movements. Some phases have dual traffic operating controls for vehicle movements i.e. the right turn movement in the EB lanes of intersection 1 has a protected right turn for phase number 1 and a permitted right turn movement for the phase number 2. The right turn in the WB lane for intersection 1 has a permitted control during the traffic signal operation. This type of protected-permitted system can be implemented when high traffic volume exists in the right turn lane. Permitted traffic operations can be applied when a low traffic volume exists and is recommended for volumes less than 200 vehicles per hour. Since vehicles in Australia are on the left hand side of the road, most left turn movements on red are of the permitted type in order to relieve and mitigate the traffic flow on the main arterial roads and the intersections.

Table 3.6 Signal timing plan of the selected area

Int. No.	WB		EB		NB		SB	
Int.6	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**
G	47	13	47	13	27	27	47	47
A	4	4	4	4	4	4	4	4
R+AR	91	125	91	125	111	111	91	91
C	142	142	142	142	142	142	142	142
Int.5	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**
G	57	16	57	23	26	26	26	26
A	5	5	5	5	5	5	5	5
R+AR	78	119	78	112	109	109	109	109
C	140	140	140	140	140	140	140	140
Int.4	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**
G	51		71	13	33		55	19
A	5		5	5	5		5	5
R+AR	64		44	102	82		60	96
C	120		120	120	120		120	120
Int.3	**TH**	**RT**	**TH**	**RT**	**TH**	**T-Intersection**		
G	120		139	16	10			
A	5		5	5	5			
R+AR	35		16	*	135			
C	160		160	160	160			
Int.2	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**
G	34	12	58	37	40	21	40	21
A	4	4	4	4	4	4	4	4
R+AR	112	134	88	109	106	125	106	125
C	150	150	150	150	150	150	150	150
Int.1	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**	**TH**	**RT**
G	74	*	95	37	16	16	12	12
A	4	*	4	4	4	4	4	4
R+AR	57	*	36	*	115	*	119	119
C	135	135	135	135	135	135	135	135

The sign "*" indicates the traffic movement was permitted when the intersection signal was red.

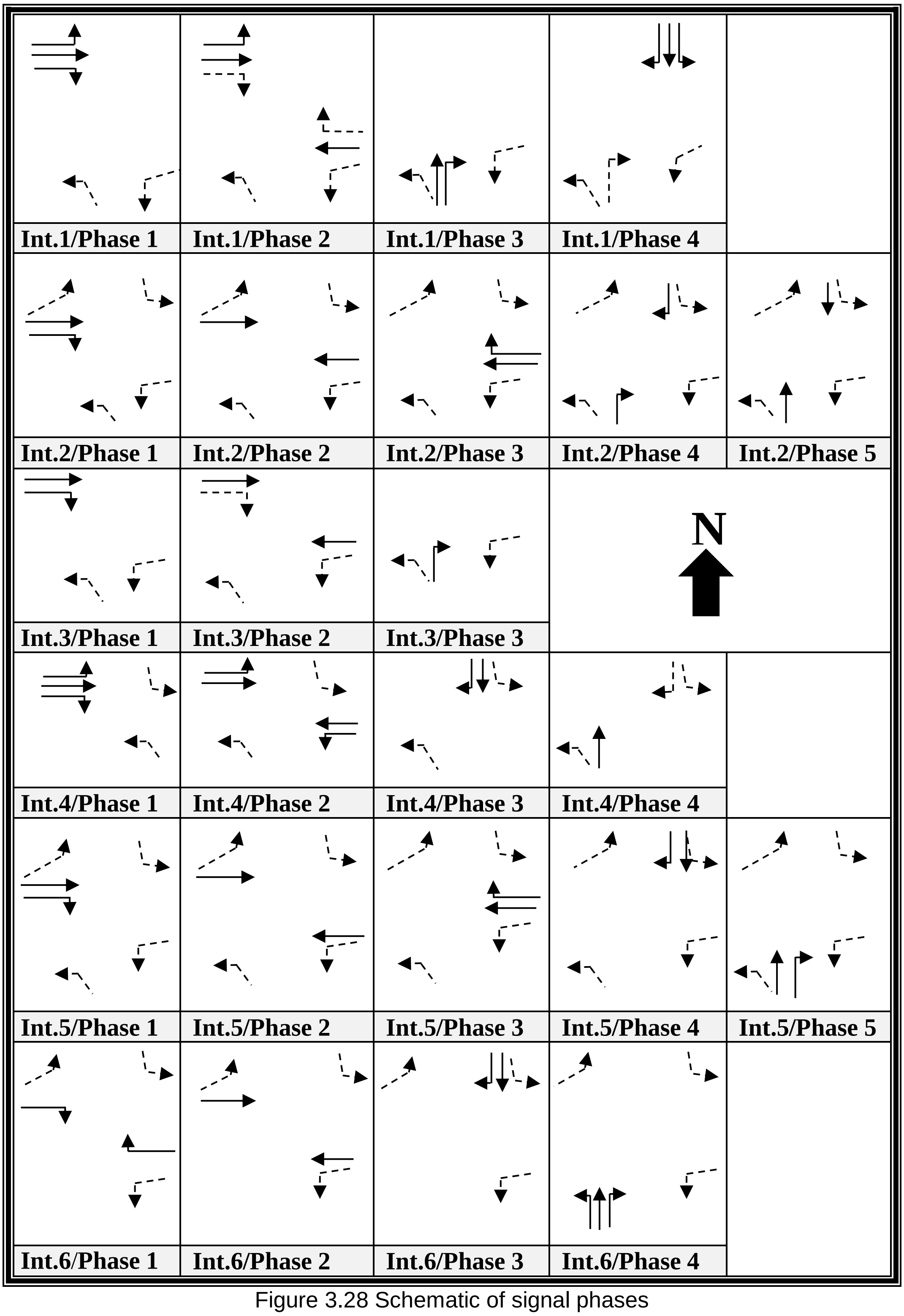


Figure 3.28 Schematic of signal phases

3.4 Floating car test

Travel time is an important parameter by which to identify the traffic flow performance for main roads. In the present study, the EB and the WB directions travel times were used to predict signal offsets between intersections. The link is the distance between two consecutive intersections and this distance varies from one link to another. In order to collect practical travel time data, a floating car technique was

used. The length of the route investigated was 5 km which included a major multilane arterial road and five signalised intersections. Checkpoints were located at intersections at which travel times were counted. Checkpoint 1 for the EB direction was at intersection 2 and checkpoint 5 was at intersection 6.
Travel time was recorded at each checkpoint while the vehicle travelled through each link. It was measured with a stopwatch and delay times were recorded. Table 3.7 and Table 3.8 show the data for 10 trials to determine the travel times along each link. The total time stopped and the numbers of stops were recorded. The travel time along each link can be calculated as the difference between cumulative times at two successive checkpoints.

Table 3.7 Travel time field data for EB

Link (EB)	Check point	Cumulative length (km)	Cumulative travel time (min:sec)	Stopped delay (sec)	No.of stops	Travel time (min:sec)	Notes
Beatty-Beaudesert	1	0.4	0:23	0	0	0:23	
Beaudesert-Perrin	2	1.415	1:50	88	1	1:27	stop due to signal
Perrin-Orange	3	2.393	2:40	0	0	0:50	
Orange-Troughton	4	3.455	4:33	44	1	1:53	stop due to signal
Troughton-Mains	5	4.996	6:06	0	0	1:33	
Total Section		4.996		132	2	6:06	

Table 3.8 Travel time data for WB

Link (WB)	Check point	Cumulative length (km)	Cumulative travel time (min:sec)	Stopped delay (sec)	No.of stops	Travel time (min:sec)	Notes
Mains-Troughton	1	1.541	1:28	0	0	1:28	
Troughton-Orange	2	2.603	3:10	112	1	1:42	stop due to signal
Orange-Perrin	3	3.581	4:18	64	1	1:08	stop due to signal
Perrin-Beaudesert	4	4.596	5:21	0	0	1:03	
Beaudesert-Beatty	5	4.996	6:04	0	0	0:43	
Total Section		4.996		176	2	6:04	

The data in Table 3.7 and Table 3.8 show the variation between the EB and WB checkpoints. The highest travel time was evaluated according to the link length and the delay in the link. For example, checkpoint 5 in the WB direction displayed a high travel time in contrast with checkpoint 2, regardless of the delay caused by signal operation. The length to checkpoint 5 is just 0.4 km and the travel time was 43 sec without considering the delay, while the length for checkpoint 2 is 1 km and the travel time was 102 sec including the delay. That means the time needed to approach the link in checkpoint 2 is more than the time required to cross-checkpoint 5. The time spent in crossing checkpoint 1 of the EB direction was approximately half the travel time for checkpoint 5 of the WB direction under the same circumstances. The delay of the WB direction was greater than the delay of the EB direction. The differences in results are due to traffic signal timing, signal offsets between intersections and the traffic volume.
The offset was empirically determined for the site. Field results were examined using VISSIM to ensure correct offsets. The selected criteria were based on a compromise between the empirical offset times and the intersections throughput. The optimum offset was chosen as the point at which VISSIM output produced the best

convergence with field intersection throughput. Table 3.9 shows the empirical offsets determined for the selected study area.

Table 3.9 Optimum signal offsets

Intersection #	1	2	3	4	5	6
Optimum Offset	24 sec	35 sec	45 sec	50 sec	67 sec	58 sec

3.5 Summary

This chapter provided an overview of data collection procedures employed to obtain the essential traffic data for the study area. Two approaches were followed to collect the data needed for the study. The first approach used the traffic data provided from QDTMR which is based on STREAMS. STREAMS data were used to determine the most significant elements that influenced traffic performance. These elements are the day of the field survey and identification of appropriate peak survey hours. The elements are essential for implementing an accurate field survey. Moreover, traffic volume variations over time for each individual link were also extracted from STREAMS data.

However, since the collected data from STREAMS database merely represented the EB and WB directions, Data for NB and SB directions were needed to fully analyse the traffic situation for signalised intersections with VISSIM software.

Field data was collected from the study area using video cameras and manual counting. Six high-resolution video cameras and a group of 18 people were used to collect the survey data.

Virtual Dub Software (VDS) was used to count traffic volumes and to identify the traffic flow configuration precisely. The drawback of using such software was the time it consumed but the results were very accurate.

A floating car test was implemented to determine the travel time spent in each link and to identify empirical optimum offsets along the main roads. A private vehicle was used to measure the travel time along the study area and the results were obtained from an average of 10 trials. VISSIM software was employed to verify the optimum offsets between the signalised intersections.

For the sake of consistency, the current research has concentrated on data collected from the field so as to provide an integrated view of the traffic operations during peak survey hours. STREAMS data was used for verification and calibration purposes.

The chapter reports extensive field data obtained from the physical survey which was undertaken to augment the historical QDTMR. This exercise was essential as the QDTMR lacked the comprehensive intersection flow details required for subsequent model inputs. The QDTMR data were useful for planning purposes for the implementation of the field survey.

The outcome from the data collection revealed reliable and accurate traffic data studies for temporal distributions and intersection characteristics operation as input into VISSIM.

CHAPTER FOUR
Modelling the Griffith Arterial Road using VISSIM software

4.1 Introduction

This chapter provides an overview of VISSIM software and its use in traffic modelling and its specific application in this research. The study uses VISSIM microscopic software as a tool to carry out the traffic analysis. Accordingly, the simulation model requires calibration and validation to accurately model the actual traffic conditions. The calibration process is based on comparing results between VISSIM output and real field data. The comparison process requires collection of data for field parameters known as Measures of Effectiveness (MOE). The current study investigated MOE such as traffic volumes, volume distribution, and saturated headway along the WB and EB directions of the study area. Validation criteria were used to determine whether VISSIM output was within acceptable levels or not. The Geoffrey E. Havers (GEH) statistical procedure was used to evaluate the calibration results (Gomes et al., 2004, Oketch and Carrick, 2005). Since VISSIM is stochastic software (its output changes spontaneously with each simulation run), it is necessary to determine the minimum number of simulation runs needed to achieve a set level of accuracy.

4.2 VISSIM an overview

In order to control real world traffic flows, many models have been created to help relieve or mitigate the effects of traffic congestion in urban areas, particularly for interrupted flow conditions. The application of these models to real world conditions is difficult due to the complexity of interrupted traffic flow. New technology has made it possible to simulate and/or visualize traffic flow under different conditions on road networks. Abdy and Hellinga (2008) reported that traffic simulation programs such as TRANSYT-7F, EMME, SATURN, AIMSUN, PARAMICS, CORSIM, STREAM and VISSIM were the most popular software packages used in simulating traffic systems.
VISSIM is a time step, stochastic behaviour model, developed initially in the 1970s at the University of Karlsruhe, Germany, to model the urban public transportation system. It was further refined in the 1990s to model freeway traffic behaviour and is sold commercially by PTV AG, Karlsruhe, Germany throughout the world. The software was calibrated with field research from the Technical University of Karlsruhe, Germany. Planning Transport Verkekr (PTV) AG (2011) illustrated that the psychophysical driver behaviour model used and developed by Wiedemann74 and 99 effectively represented traffic behaviour for urban arterial roads and motorways respectively. Bloomberg and Dale (2000) explained that VISSIM simulation consists of two main elements: a traffic simulator and a signal state generator. The traffic simulator is responsible for modelling the movement of vehicles, while the signal state generator is responsible for modelling the signal decision status developed from detector information. It transfers the signal status back to the traffic simulator. The primary concept of this model is based on speed

discrepancy between two successive vehicles. The driver of the slower moving vehicle accelerates until he or she reaches their individual perception threshold relative to the faster moving vehicle and vice versa. Drivers cannot precisely identify the speed of the other vehicle and their speed will fall below the other vehicle's speed until the driver starts to accelerate again after realising they are going slower than the vehicle ahead of them. This result is an iterative process of acceleration and deceleration. Laufer (2007) explained that the basic concept of the Wiedemann model was based on a vehicle driver being in one of four driving modes:

1. **Free driving mode**: there is no noticeable effect of the preceding vehicles. The driver accelerates to reach and maintain a certain desired speed. In reality, speeds at free driving mode cannot be kept constant but oscillate around the desired speed due to driver behaviour.
2. **Approaching mode**: the speed of the driver slows down to the lower speed of a preceding vehicle. While approaching, the driver gradually decelerates the vehicle until it reaches the desired safety distance. At this moment, the difference in speed between the two consecutive vehicles is zero, that is two consecutive vehicles are travelling at the same speed.
3. **Following mode**: the driver tracks the preceding vehicle without any awareness of acceleration or deceleration. The driver tries to maintain the safety distance between his vehicle and the preceding vehicle more or less constant. Estimating this distance is based on the drivers' behaviour so that the speed difference oscillates round zero.
4. **Braking mode**: the driver applies medium to high deceleration if the distance to the preceding vehicle falls below the desired safety distance. This driving mode can take place if the preceding vehicle changes speed abruptly or in cases where a third vehicle changes lanes in front of the driver.

Figure 4.1 shows the concept of car following model by Wiedemann.

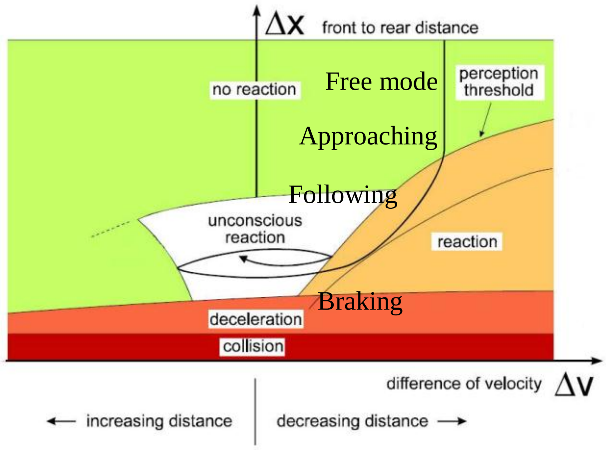


Figure 4.1 Car following model by Wiedemann: Source PTV 2011

The "car following" model is one of three main models used during simulation. The "discretionary lane change" model and "necessary lane change" model are the other two types of models that are used. The lane change model is based on either a driver's reaction to their sensitivity to the adjacent vehicle or it is used in response to asses future opportunities and requirements for each vehicle. Gap acceptance criteria between adjacent vehicles are used to determine whether the drivers are able to change lanes or not. Gap acceptance is the minimum gap required to commence and

finish changing lanes safely. Vehicles presented with a gap greater than the gap acceptance are able to change lanes.
A number of conditions are required to change lanes including the desire to change lanes, favourable driving conditions in the adjacent lanes, and a suitable gap.
Lane change behaviour is classified into two types based on these conditions. First is discretionary lane change behaviour which occurs when drivers intend to change from slow lanes to fast lanes. Second, is necessary lane change behaviour which is used to achieve the origin-destination path. More details about the concept of lane change can be found in (Wiedemann and Reiter, 1992).
Parameters are imposed separately on each link in the network that leads to the decision to change lanes. These are:

1. **Look back distance**: It defines the distance at which vehicles begin to attempt to change lanes. At this distance, the driver is able to change lanes successfully. The default value for lane change distance is 200.0 m. in VISSIM.
2. **Emergency stop distance**: It defines the last possible position for a vehicle to change lanes. At this position, the vehicle will stop to wait for an opportunity to change lanes. The default value is 5 m in VISSIM.

The aggressiveness of a driver's behaviour, which is characterised by accepting or rejecting gaps, has a significant effect on lane change decision making. Parameters such as the minimum and maximum deceleration values of trailing vehicles, and reduction of the safety distance factor motivate acceptance or rejection of the decision to change lanes. Reduction of the safety factor decreases the safe distance between trailing and leading vehicles in the desired lanes as well as the safe distance to the leading vehicle in the current lane. This factor is specified in VISSIM by a default value of 0.6. That means that the safety distance has reduced by 40% during the lane change. Reducing this factor indicates a more aggressive behaviour in accepting shorter gaps. Lownes and Machemehl (2006) explained that this parameter in the model played a significant role in affecting a wide variety of traffic measures.
Six different vehicle types can be modelled in VISSIM software: Car, HV, Bus, Tram, Pedestrian and Bike. Each type has its own driving characteristics that are responsible for simulating vehicles in the network.
In the current study, VISSIM was used to simulate the traffic system along Griffith Arterial Road (GAR) for many reasons. VISSIM can simulate not only automobile transport, but also interaction with pedestrians, cyclists and light rail. The priority rules feature of VISSIM seems to allow complex modelling of join behaviour, including friendly merging, diverging and weaving as it arises in the real world. The availability of VAP (Vehicle Actuated Program) module provides more flexibility to allow the organizer to skip phases, lengthen, and shorten phases as required. VISSIM provides a lot of flexibility in network coding i.e. dynamic traffic assignment, code complex signal systems, mimic some ITS features and implement dynamic speed limit along the network. A 3-D visualisation model allowed the actual impacts caused by the development to be physically observed. Evaluations can be obtained for any time interval within the time frame of the simulation. Outputs can be easily obtained via the evaluation toolbox in VISSIM, and then documented in the designated evaluation files using Microsoft assess. From above, it is not apparent that other simulation models are able to represent such features.

4.3 Development of VISSIM modelling

The aim of VISSIM modelling was to investigate and identify the traffic flow problems of the GAR and to test proposed traffic management strategies to relieve and mitigate traffic flow bottlenecks at congested intersections. In order to build an effective microscopic simulation model for GAR, three main types of input are required. These are the physical design of the road network, vehicle characteristics, and driver behaviour. In the present study, the physical design and vehicle characteristics are kept constant during the modelling. The third stage is varied where it is necessary to replicate the actual traffic conditions along the network. More details about these three stages are provided in Appendix A1.

4.3.1 Physical road network & traffic signal timing

The first step in constructing VISSIM is drawing the geometric design and layout of the study area. This was done using the VISSIM interface screen. An accurate and easy way to obtain the layout of the network was found to be by utilising Google-Earth. The model is generated by tracing the image background using VISSIM links and connectors. The lane width of the roadways was obtained from the aerial image using the toolbox in Google-Earth or from field measurements.

4.3.2 Coding traffic Data

Traffic volume, vehicle composition, desired speed and vehicle distribution parameters are needed to run the model simulation. Traffic flow can be inserted as a time flow series. Hourly observed volume for each entrance point in the network is used as input data in VISSIM. Also, a warm-up period of 600 sec was used when starting simulations to fill the network with vehicles. In the present study, four different vehicle types were modelled to reflect the actual traffic composition. Cars, trucks, buses and motorcycles represent the mix of vehicle types that are observed in field studies and thus executed in VISSIM.

4.3.3 Driving behaviour

The concept of “car following” and “lane change” models in VISSIM are set for two types of driving behaviour based on Wiedeman (1974 and 1999). The first model defined the traffic flow behaviour in urban areas whereas the second model used freeway behaviour. The study utilised the Wiedeman (1974) driving behaviour because it closely matched the driving behaviour of the selected area.

4.4 Calibration and validation processes

4.4.1 VISSIM calibration

An overview of VISSIM’s default parameters (provided in previous sections) is included, as some of those parameters needed to be adjusted to match the model to the site-specific traffic flow conditions. The adequacy of calibration is based on validation statistics which measure the percentage of errors between simulated results and field measurements. The modelling animation must also present realistic scenarios. These factors should be achieved together in order to produce a well-

calibrated model. If initial VISSIM outputs are not qualitatively and quantitatively compatible with the selected MOE, default parameters need to be changed. The range of acceptance is affected by the validation criteria used to accept or to refuse VISSIM results. Figure 4.2 shows the calibration process.
The following sections outline the calibration and validation processes. Further information is provided in Appendix A 2.

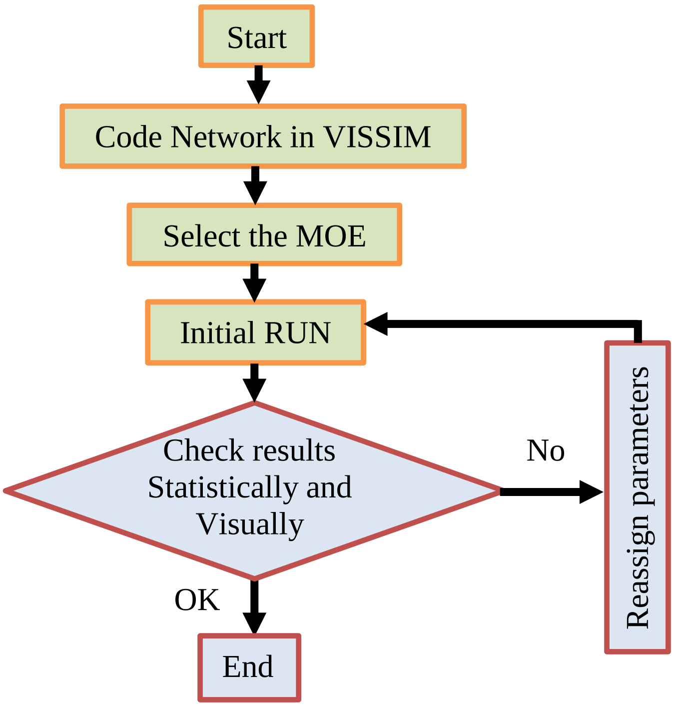


Figure 4.2 Calibration process

4.4.1.1 Identification the measure of effectiveness (MOE)

Vehicle distribution at the signalised intersection, the total traffic volume along the main road network and the headway departure at the main signalised intersections are defined as MOEs. These parameters are compared to field observations. Since VISSIM has the ability to depict the traffic operation visually, recurring congestion in VISSIM can be corrected.

4.4.1.2 Initial iteration run

An initial iteration run is recommended to identify coding errors that could have occurred during building of the VISSIM model and to reveal how much deviation has occurred compared with the selected MOE. It is done by visually checking whether the locations of the recurrent bottlenecks in the VISSIM simulation match the congested links in the GAR.

4.4.1.3 Determination the minimum number of simulated run

In order to know the minimum number of runs needed to determine statistical significance, performance parameters have to be defined, that is the standard deviation, the standard level of confidence, and the confidence interval. In the current study, a high level of confidence is 95 % has been applied. In order to determine the minimum number of repetitions, the following equation was used:

$$CI_{(1-\alpha)\%} = 2 * t_{(1-\alpha),N-1} * \frac{S}{\sqrt{N}} \quad 4.1$$

where,

$CI_{(1-\alpha)\%}$ = (1-α) % confidence interval for the true mean, where α equals the probability of the true mean not lying within the confidence interval,

$t_{(1-\alpha),N-1}$ = Student's t-statistic for the probability of a two-sided error summing to α with $N-1$ degrees of freedom, where N equals the number of repetitions and

S = standard deviation for the results model.

In order to obtain the initial standard deviation for traffic volume, five runs were performed. The number of additional runs is determined based on the desired mean value. An initial five runs with different random seeds were used to determine the total simulated traffic volume among the signalised intersections for the EB and WB directions. This preliminary estimation needs to be reconsidered later to check whether additional runs are needed or not. The number of additional runs is determined based on the desired mean value. Table 4.1 and Table 4.2 show the total simulated volume for the mainstream flow for the EB and WB directions respectively. Note the low standard deviation value for Intersection 1 in Table 4.1 is due to the use of VISSIM default driving behaviour parameters for the whole network as a first step.

Table 4.1 Total simulated traffic volume (veh/h) for the EB direction

#Run	#Seed	Int. 1	Int. 2	Int.3	Int.4	Int.5	Int.6
1	100	2089	1881	1510	1500	1743	1651
2	110	2090	1852	1522	1527	1716	1611
3	120	2090	1791	1452	1462	1722	1623
4	130	2090	1875	1537	1538	1764	1654
5	140	2090	1805	1515	1507	1755	1596
Standard deviation		0.45	40.84	32.49	29.30	20.68	25.19
Simulation average		2089.8	1840.8	1507.2	1506.8	1740	1627

Table 4.2 Total simulated traffic volume (veh/h) for the WB direction

#Run	#Seed	Int. 1	Int. 2	Int.3	Int.4	Int.5	Int.6
1	100	1463	1221	1212	1109	988	1166
2	110	1583	1255	1235	1129	996	1180
3	120	1519	1254	1244	1122	1007	1222
4	130	1533	1307	1281	1189	1023	1300
5	140	1490	1212	1192	1089	1003	1229
Standard deviation		46.74	36.61	33.09	37.55	14.09	52.44
Simulation average		1517.6	1249.8	1232.8	1127.6	1003.4	1219.4

Table 4.3 shows that 5% of average simulations for the EB direction of the Beaudesert intersection is 92 veh/h while the *CI* values for the five runs at a level of 95% confidence are above the threshold value (*CI* > 5% AVG). That means the simulated results need more than five iterations to satisfy threshold accuracy criteria. Equation 4.1 will be recalculated again to estimate the required number of runs for this part of the network.

Table 4.3 Estimation of minimum required number of simulation runs for EB

Beatty (Int.1)					Beaudesert (Int.2)				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	8	104	2	2	12.71	734	92	> 5
3	4.3	2			3	4.3	202		
4	3.18	1			4	3.18	129		
5	2.78	1			5	2.78	101		
Perrin (Int.3)					Orange Grove (Int.4)				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	584	75	> 5	2	12.71	526	75	5
3	4.3	161			3	4.3	145		
4	3.18	103			4	3.18	93		
5	2.78	80			5	2.78	72		
Troughton (Int.5)					Mains (Int.6)				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	371	87	4	2	12.71	452	81	4
3	4.3	102			3	4.3	125		
4	3.18	65			4	3	80		
5	2.78	51			5	2	62		

Table 4.4 shows the initial required number of simulation runs for the WB direction. Five simulation runs are insufficient to achieve calibration results within the 95% confidence interval. The process of finding the minimum number of repetitions aids estimating how many ultimate calibration parameters are needed. For the purpose of calibrating the whole network, 50 simulation runs were needed to meet the required statistical confidence level.

Table 4.4 Estimation of minimum required number of simulation runs for WB

Mains WB					Troughton WB				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	943	61	> 5 Runs	2	12.71	253	50	4 Runs
3	4.3	260			3	4.3	70		
4	3.18	167			4	3.18	45		
5	2.78	130			5	2.78	35		
Orange Grove WB					Perrin WB				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	675	56	> 5 Runs	2	12.71	595	62	> 5 Runs
3	4.3	186			3	4.3	164		
4	3.18	119			4	3.18	105		
5	2.78	93			5	2.78	82		
Beaudesert WB					Beatty WB				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	658	62	> 5 Runs	2	12.71	840	76	> 5 Runs
3	4.3	182			3	4.3	232		
4	3.18	116			4	3	149		
5	2.78	91			5	2	116		

4.4.2 Validation criteria

Since the calibration procedure is an iterative approach to compare the simulated results with the field measurements, statistical validation is needed to accept or reject the calibration processing under specific limitations. GEH statistics is widely used in traffic departments for engineering, modelling and forecasting. The GEH formula is employed to show the variation between the two sets of data, field data and simulated result. This formula was determined in 1970s by Geoffrey E. Havers (Chung and Choi, 2010). The mathematical procedure resembles a chi-squared test but it is not used as a general statistic test because it is an empirical equation relevant to traffic analysis. The formula and the validation criteria for this type are:

$$GEH = \sqrt{(S - F)^2/(0.5 \times (S + F))} \quad 4.2$$

Where S is the simulated result and F is the field data.
The goodness of fit indicators for GEH between two data sets can be classified into three criteria as follows:
GEH < 5, Results can be considered to be a good fit.
5 < GEH < 10, Results may need more investigation.
GEH > 10, Results cannot be utilised because there is a problem in the selected data.

4.5 Calibration and validation implications

4.5.1 Calibration results of mainstream flow

After establishing the calibration process using VISSIM default parameters, testing is necessary to evaluate if they replicate real traffic conditions. If the default parameters can predict the field measurements, further calibration is not required. There is no single accepted procedure for managing the calibration and validation for complex networks. The modeller can choose different ways of calibrating and validating the model to provide acceptable and statistically significant results. Inlet traffic flow is selected as one of the MOE parameters to compare field data to the simulated results. During the calibration process, it was found that not all the default-driving parameters represent situations in the study area. Some parameters required adjustment to give acceptable results such that the absolute relative error (*ARE*) was less than 5%. *ARE* can be defined as the difference between the simulated and the field data.

$$ARE = \left| ((F-S)/F) \times 100 \right| \quad 4.3$$

where all the variables were defined previously.

The percentage of *ARE* can be decreased by using more simulation model runs. The most significant parameters that affect the volume of traffic flow are the saturated parameters such as average standstill distance (*ax*), the additive part of the desired safety distance (*bx_add*) and the multiplicative part of the desired safety distance (*bx_mult*). These were used to adjust the simulated traffic flow. The calibrated results were validated using GEH. The average value for both field and simulated output data are considered in the validation process.
As shown in Table 4.5, the simulated flow as compared to the observed field flow for the EB lane of the Beatty intersection were acceptable because the calculated GEH value (0.48) was less than the acceptable value (GEH < 5). The results also show that the *ARE* between the simulated flow and the observed flow is less than the 5% threshold. The outcome indicated that no further adjustments to driver behaviour parameters were required for calibration.

Table 4.5 Calibrating simulated flow versus field observed flow for Beatty EB

Beatty EB	Default	Adjustments
Average standstill distance (*ax*)	2	No Adjustments
Additive part of desired safety distance (*bx_add*)	2	
Multiplicative part of desired safety distance (*bx_mult*)	3	
Average Simulated Flow	2068	
Average Field Observed Flow	2090	
Absolute Relative Error (ARE) %	1.0	
GEH	0.48	

Table 4.6 shows the calibration process for the NB lane at the Beatty intersection. An *ARE* of 7.1 (> 5) shows that the default driver behaviour parameters were not suitable for achieving an acceptable level of modelling accuracy for this approach. This means that the default parameters were successful in calibrating the EB lanes but failed in calibrating the NB lanes at the Beatty intersection. The GEH value for the default parameters is 1.62 which lies in the acceptable range (GEH < 5). However, the default parameters need further calibration because the *ARE* does not meet the recommended allowance for modelling error (*ARE* < 5%).
The calibration should pass the validation criteria in order to accept the calibrating process otherwise further alteration to the default parameters is required. Therefore, three new adjusted driver parameters were selected to calibrate the simulated flow to observed field flow. As can be seen from Table 4.6, altering only the parameter (*bx_mult*) from 3 m in trial 1 (default) to 1 m in trial 2 produced an increase in correlation between the simulated and the observed flow. The GEH for trial 1 was 1.62 and *ARE* was 7.1%. These values changed significantly for trial 2 where they were 0.7 and 3.1% respectively. Trial 3 records produced still better results when the value of (*bx_add*) was changed from 2 m to 1 m. It produced a GEH of 0.27 and an *ARE* of 1.2%. This was the optimum calibration among the adjusted driver behaviour parameter sets.

Table 4.6 Calibrating the simulated flow versus field observed flow for Beatty NB

Beatty NB	Default	Adjusted		
Trial	1	2	3*	4
Average standstill distance (*ax*)	2	2	2	2
Additive part of desired safety distance (*bx_add*)	2	2	1	0.5
Multiplicative part of desired safety distance (*bx_mult*)	3	1	1	1
Average Simulated Flow	470	490	512.2	520
Average Field Observed Flow	506	506	506	506
Absolute Relative Error %	7.1	3.1	1.2	2.7
GEH	1.62	0.71	0.27	0.61

* Refers to the selected parameters

In Table 4.7 the GEH values are satisfactory for all the proposed parameter sets but a discrepancy occurs with the *ARE* results. Trial 4 achieved the required confidence criteria by reducing the absolute relative error to 4.67%. The selected parameters in Table 4.6 are not appropriate for conducting adequate calibration for the EB lanes at the Mains intersection. Each intersection has its own traffic characteristics that distinguish it from other intersections. The user is responsible for proposing suitable driver behaviour parameter to produce an acceptable level of confidence in the model results.

Table 4.7 Calibrating the simulated flow versus field observed flow for Mains EB

Mains EB	Default	Adjusted		
Trial	1	2	3	4*
Average standstill distance (*ax*)	2	2	2	2
Additive part of desired safety distance (*bx_add*)	2	1	1	0.5
Multiplicative part of desired safety distance (*bx_mult*)	3	1	0.5	0.5
Average Simulated Flow	1569	1611	1617	1654
Average Field Observed Flow	1735	1735	1735	1735
Absolute Relative Error %	9.5	7.1	6.8	4.67
GEH	4.08	3.03	2.88	2.68

* Refers to the selected parameters

In summary, these steps were followed to calibrate all the intersections along the study area that involved NB, SB, EB, and WB traffic lanes. The results showed that there is no correlation between the parameter sets and the degree of verification. That means each set of parameters has its own characteristics and these could not be accommodated with other links under the same prevailing conditions.

4.5.2 The variability of simulated flow at peak hour

The coefficient of variation (COV) (standard deviation divided by the mean) measures the variability of flow during the peak hour (flow reaches capacity). Fifty replications were run using VISSIM micro simulation during the peak survey time to obtain the final result of estimated flow. The simulation used different random number seeds and a constant traffic flow setting. Figure 4.3 shows the COV for the simulated flow by VISSIM. The selected intersections at which the EB and WB flow was analysed did not vary very much during the simulation. This means that as long as the traffic flow is high (v/c ratio near 1), the mean space headway between vehicles is small and the randomness in traffic flow is low. This results in a traffic pattern with a close to uniform flow distribution. Moreover, the temporal headway between vehicles is low when the traffic volume is high, and that in turn reduces the standard deviation. Since the mean flow value under high traffic volume is large, dividing the simulated standard deviation (a relatively low number) by the mean flow value (high number) produces a low measure of variation (COV). Figure 4.3 shows that the COV for both EB and WB directions for all the intersections is low. This is due to the high traffic volume. Figure 4.4 shows the percentage of COV for simulated EB flow compare with EB field flow during the rush hour. Both lines indicate a low COV %. The average variability in field data collection was 4.4% and 1.33% for the simulated data. The variability for both types of flow is low and less than 5%.

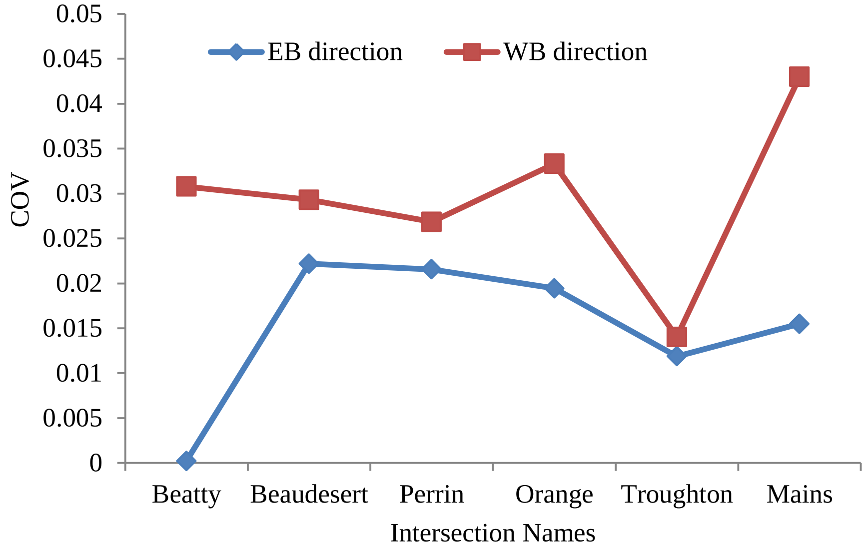


Figure 4.3 Coefficient of variation of simulated flow under high traffic volumes

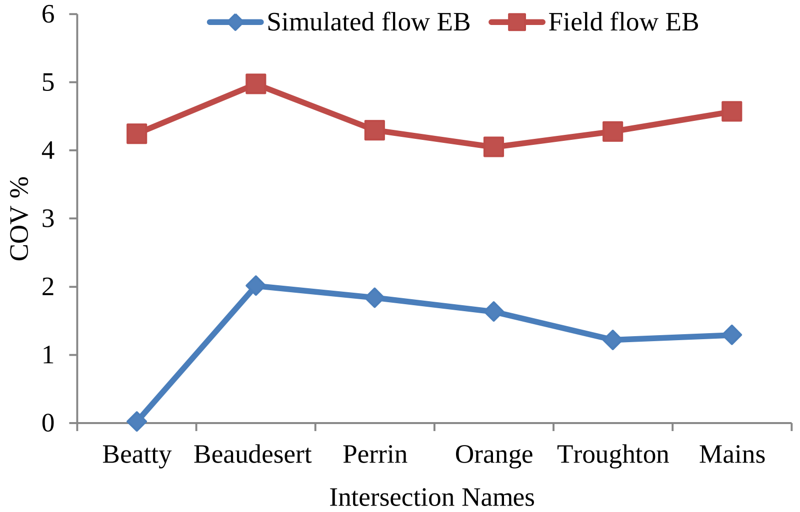


Figure 4.4 Coefficient of variation of simulated flow versus field flow

These results are consistent with findings such as those by Abdy and Hellinga (2008), Sadek and Smith (2006), Dion et al. (2004). Traffic flow behaviour under high traffic demand yields low COV values when using random number seeds and constant traffic volumes.

4.5.3 Calibration results of flow distribution

The driver behaviour parameter sets that involve the variables: *ax*, *bx_add*, and *bx_mult* are considered the major parameters used to set up the traffic flow quantities. Other parameters such as the lane change parameters will effect the operation of vehicle movement via VISSIM modelling such as vehicles overlapping, unrealistic lane change manoeuvres, and vehicles disappearing during simulation.
The second MOE used to compare the simulated results with the field observed data is the flow distribution at each intersection. This was carried out for all directions along the GAR. In order to assess the calibration method for flow distribution, the same procedure used in section 4.4.1 was used to calibrate the flow distribution

factor. The results showed that no more adjustments were needed for the driver behaviour parameter sets to calibrate the flow distribution except the statistical verification. This was due to reasons such as the adequate calibration of the traffic flow inlet, selection of appropriate lane change parameters (Section 4.2.4.2) and suitable estimation of the number of runs. Statistical GEH requirements are based on hourly volumes to conduct the analysis. Therefore, the field observed flow that was collected for 15-minute intervals was converted to hourly volumes for the GEH analysis. Flow distribution at the signalised intersections can be divided into through movement (TH), left turn (LT) and right turn (RT) movements.
In the current study, detectors were located at downstream intersections to count the total number of vehicles for each individual traffic movement. The number of vehicles was aggregated when the traffic pattern had more than one lane. The VISSIM model simulates for 3600s plus 600s for the warm up period. The simulated results are considered for the period from 600s to 4200s, excluding the warm up period.

Table 4.8 shows that the GEH results for GAR are below 5. This implies the VISSIM model is well fitted for the GAR. In addition, the Absolute Relative Error (*ARE*) values for the area are less than 5 %. This indicates the parameters in the model are well matched with reality. Moreover, there is strong correlation between the calibration of the entire flow and the calibration of the flow distribution at signalised intersections. This means good calibration for the entire flow will result in good calibration for the flow distribution.

Table 4.8 Statistic validation for VISSIM modelling

Name	Dir.	Move.	ARE %	GEH	Name	Dir.	Move.	ARE %	GEH
Beatty	EB	TH	1.37	0.56	Orang Gr.	EB	TH	4.57	2.41
		LT	4.62	0.28			LT	2.94	0.30
		RT	1.25	0.25			RT	3.19	1.14
	WB	TH	4.70	3.89		WB	TH	3.20	3.16
		LT	2.32	1.56			LT	2.96	0.75
		RT	2.00	0.84		NB	TH	3.58	1.35
	NB	TH	0.50	0.06			LT	2.26	0.78
		LT	2.88	1.92		SB	TH	4.33	1.04
		RT	4.11	2.44			LT	2.0	0.63
	SB	TH	1.10	0.12			RT	0.54	0.06
		LT	1.67	0.25	Troughton	EB	TH	4.63	2.99
		RT	1.85	0.20			LT	2.04	0.85
Beaudesert	EB	TH	3.81	1.33			RT	3.54	1.18
		LT	1.85	0.73		WB	TH	2.25	2.17
		RT	1.09	1.44			LT	2.73	1.22
	WB	TH	4.44	3.49			RT	3.33	0.86
		LT	1.88	0.56		NB	TH	5.00	0.47
		RT	4.32	1.45			LT	1.46	0.21
	NB	TH	1.17	0.34			RT	1.22	0.18
		LT	3.16	1.12		SB	TH	1.63	0.20
		RT	3.80	0.53			LT	2.44	0.16
	SB	TH	4.56	1.83			RT	2.09	0.70
		LT	3.13	0.38	Mains	EB	TH	3.11	2.64
		RT	4.80	0.69			LT	2.84	1.40
Perrin	EB	TH	2.74	1.07			RT	2.77	0.28
		RT	1.15	0.83		WB	TH	4.60	1.25
	WB	TH	4.00	2.92			LT	1.40	0.24
		LT	5.00	0.35			RT	4.20	0.63
	NB	LT	4.14	0.45		NB	TH	4.44	1.22
		RT	1.22	0.09			LT	2.22	0.23
							RT	4.41	1.16
						SB	TH	3.52	1.23
							LT	3.50	0.64
							RT	3.00	0.45

4.5.4 Calibration results of saturated headway

The saturated headway was investigated for EB and WB lanes for the Beaudesert intersection based on the previous calibration. The field-saturated headways (Chapter 3 Section 3.3.3.3) were 1.98s, 2.03s and 2.1s for WB through movements and EB right turn and through movements respectively. The discharge headway for all vehicles departing during a single cycle length in VISSIM was counted. The average simulated discharge headways in VISSIM were 1.85s, 1.96s and 2s for the same traffic movements as mentioned above. Figure 4.5, Figure 4.6 and Figure 4.7 show the variability in discharge headways for simulated and field conditions. It appears that the discharge headway for the field conditions is greater than the simulated headway but generally follows the same pattern as the simulated headway. The differences may be attributed to the headway calculation process that was used to count the headway values for both conditions. The difference could also be related to differences in vehicle characteristics (weight and power) between the real traffic and simulated vehicles. This difference causes variation in acceleration and deceleration.

After calculating the average saturated headway for field and simulated conditions, the traffic capacity was compared for both conditions.

Table 4.9 explains the variation in traffic capacities between the field data and simulated results for the Beaudesert intersection. VISSIM results appear to be consistent with field conditions because the GEH is less than 5 and the differences are less than 10%. Accordingly, no further calibration was deemed necessary for the proposed network.

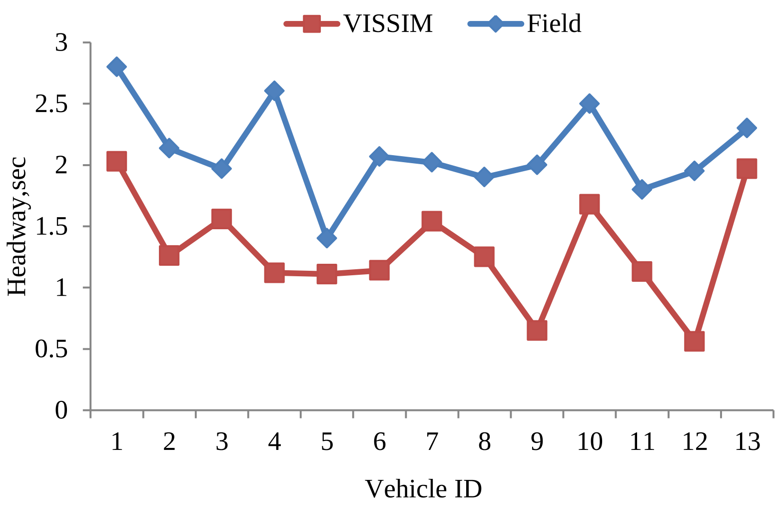


Figure 4.5 Comparison of saturated headway for EB/TH

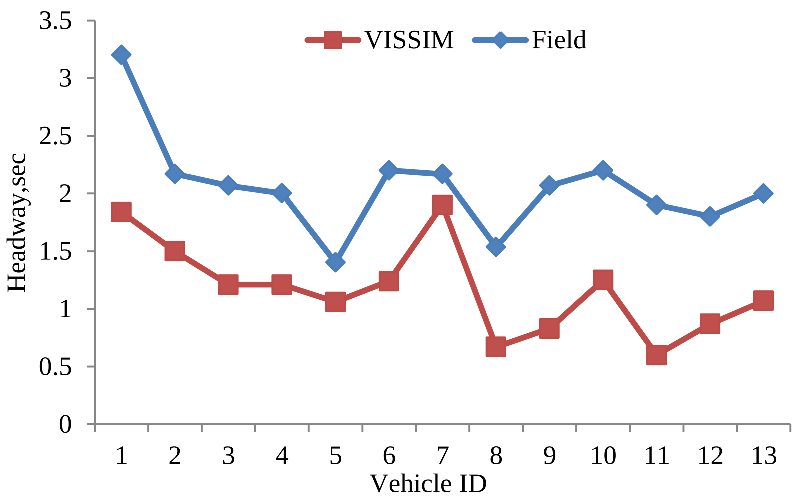


Figure 4.6 Comparison of saturated headway for EB/RT

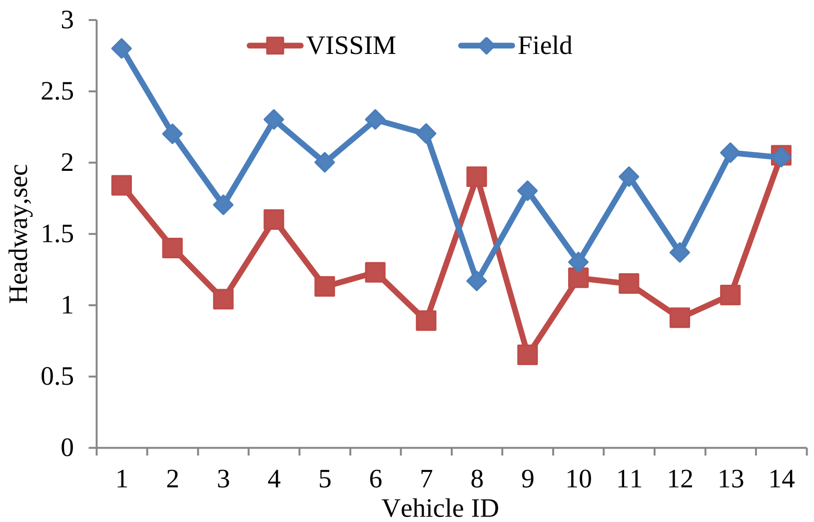


Figure 4.7 Comparison of saturated headway for WB/TH

Table 4.9 Comparison of simulated capacity and field capacity

		Headway(sec)	Cap/lane (veh/hr)	Diff %	GEH
WB/TH	Field	1.98	461	6.56	1.45
WB/TH	VISSIM	1.85	493		
EB/TH	Field	2.1	663	4.76	1.25
EB/TH	VISSIM	2	696		
EB/RT	Field	2.03	437	3.44	0.73
EB/RT	VISSIM	1.96	453		

4.6 Summary

This chapter has outlined a comprehensive methodology for building, calibrating and validating a simulation model for an urban arterial road in Brisbane, Australia. Constructing the simulated model was carried out using the links and connectors in VISSIM. The study area for GAR was drawn by tracing the Google map after scaling and saving it as VISSIM background. Field data was then used as input data using the VISSIM toolbox. After completing the simulated modelling, calibration and validation processes were needed to replicate the field traffic observations. Many of the traffic parameters required calibrating due to their effect on the evaluation of traffic performance.

The most significant parameters that were chosen to represent the measure of effectiveness (MOE) in the current study were the traffic inlet, traffic distribution and average saturated headway at signalised intersections. The observed recurrent congestion at signalised intersections was used to calibrate the VISSIM model. The accessibility of the VISSIM 3D mode contributed greatly to imitating the field observations and replicating the unusual traffic behaviour of the drivers. The initial run of the model had errors attributed either to the model construction process or to traffic operations. The minimum required number of simulation runs required to achieve consistency was based on an acceptance confidence interval of less than 5% and acceptance errors of less than 5%. The capacity measurements allowed up to 10% error. Some VISSIM default parameters required modification to replicate traffic reality. From the calibration process, it was found that the driver behaviour parameters had a significant influence on the traffic flow capacities and the traffic flow distribution.

There were no unique driver parameters used to calibrate the whole network and each link had its own parameters. Thus, in addition to the link default parameters, five extra types of link parameters were created to control and replicate the real traffic flows.

GEH statistics were used to validate the model and to decide whether the calibration process achieved the required level of accuracy. Statistical criteria were based on comparison of processes between observed data and the simulated results. The study confirmed that VISSIM micro simulation software was appropriate for urban arterial road studies involving complex interactions. The simulation model created was capable of replicating the field situation with an accuracy of better than 90%.

The model now provides the foundation the investigation of the effectiveness of VSL in mitigating traffic congestion on the GAR.

CHAPTER FIVE
Description of Local Traffic Situation at Griffith Arterial Road

5.1 Introduction

Having considered the challenges of creating the software model for flow on GAR in the previous chapter, the actual flow conditions are now considered.

Traffic flow can be defined to occur in two categories, uninterrupted flow and interrupted flow. Uninterrupted flow has no external interruptions to the traffic stream. This type of flow exists mainly on freeways, where there are no intersections on the same grade, traffic signals or other interruptions to the flow. Interrupted flow has permanent external interruptions to the traffic stream due to the design and operation of the roadway. Traffic signals are the most frequent and significant external interruption to traffic flow on urban arterial roads. Other fixed interruption points include stop signs, yield signs, curb parking manoeuvre signs and other land access operations. Temporary interruptions to normal flow occur by traffic incidents, work zones, and inclement weather.

The traffic control system for the interrupted flow is considered more complex than the uninterrupted flow. Traffic streams can be analysed by either a macroscopic approach or a microscopic approach. In the macroscopic approach, the traffic stream is described by the three elements of speed, flow and density. In the microscopic approach, it is described by two variables, the spacing between vehicles and the individual speed of vehicles.

This chapter outlines the macroscopic approach adopted for the interrupted flow of the Griffith Arterial Road (GAR) in Brisbane. Understanding the physics of congestion is a crucial step in proposing strategies to modify traffic congestion in large cities.

5.2 The study site

The traffic situation on the WB and EB lanes of the GAR was investigated to determine the physics of the congestion and to identify the congested intersections. Speed-time variation in both directions was also recorded and plotted to provide a better understanding of traffic performance. There was a large variation between the posted speed and the average travel speed that indicated poor traffic performance and contributed to long delays at downstream intersections.

The study concentrated on describing the traffic situation for the WB links at intersection 1 (Beatty intersection) and the EB links at intersection 6 (Mains intersection). The site includes six signalised intersections. There are different numbers of links of varying length between the intersections (for more details, see Chapter 3 Section 3.2). Figure 5.1 is a sketch of the GAR study site.

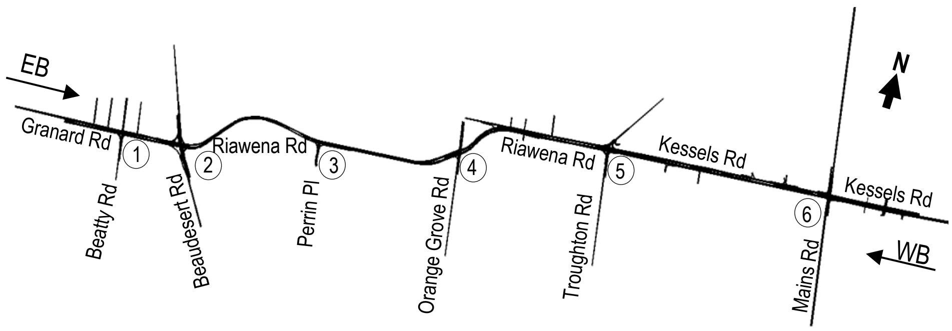


Figure 5.1 Study site

5.3 Traffic flow description

5.3.1 Description of traffic flow in WB direction

This section provides details of speed-volume relationship and speed-time variation. Details of volume-time were provided earlier in chapter 3.

5.3.1.1 Traffic flow in the link joining intersections 2 & 1

This section describes the pattern of traffic flow for the WB links at intersection 1. Basic data was obtained from the STREAMS database. Speed-flow relationships were plotted to describe the interrupted flow characteristics. Figure 5.2 (a), (b), (c) shows scatter plots of speed and flow for different survey days. The results are aggregated data for two or more lanes for each link because the detectors count the traffic volume for all lanes rather than for each individual lane. Two traffic conditions were observed; low traffic volume during off-peak conditions (100-550 veh/hr) and high traffic volume in peak periods conditions (> 550 veh/hr). Travel speeds dropped gradually as the flow rate increased. For example, the travel speed decreased from 60 km/hr (posted speed) to 10 km/hr as the flow rate increased from 100 veh/hr to 1700 veh/hr. Most travel in peak periods was less than 20 km/hr associated with high traffic volumes as shown on the lower right part of the curves. This indicated that the link suffered delays due to the low travel speed and high traffic volumes. It was found that the optimum best fit curve for the speed-flow relationship with a heavy traffic volume on a multi-lane road was an exponential expression, as shown in Figure 5.2.

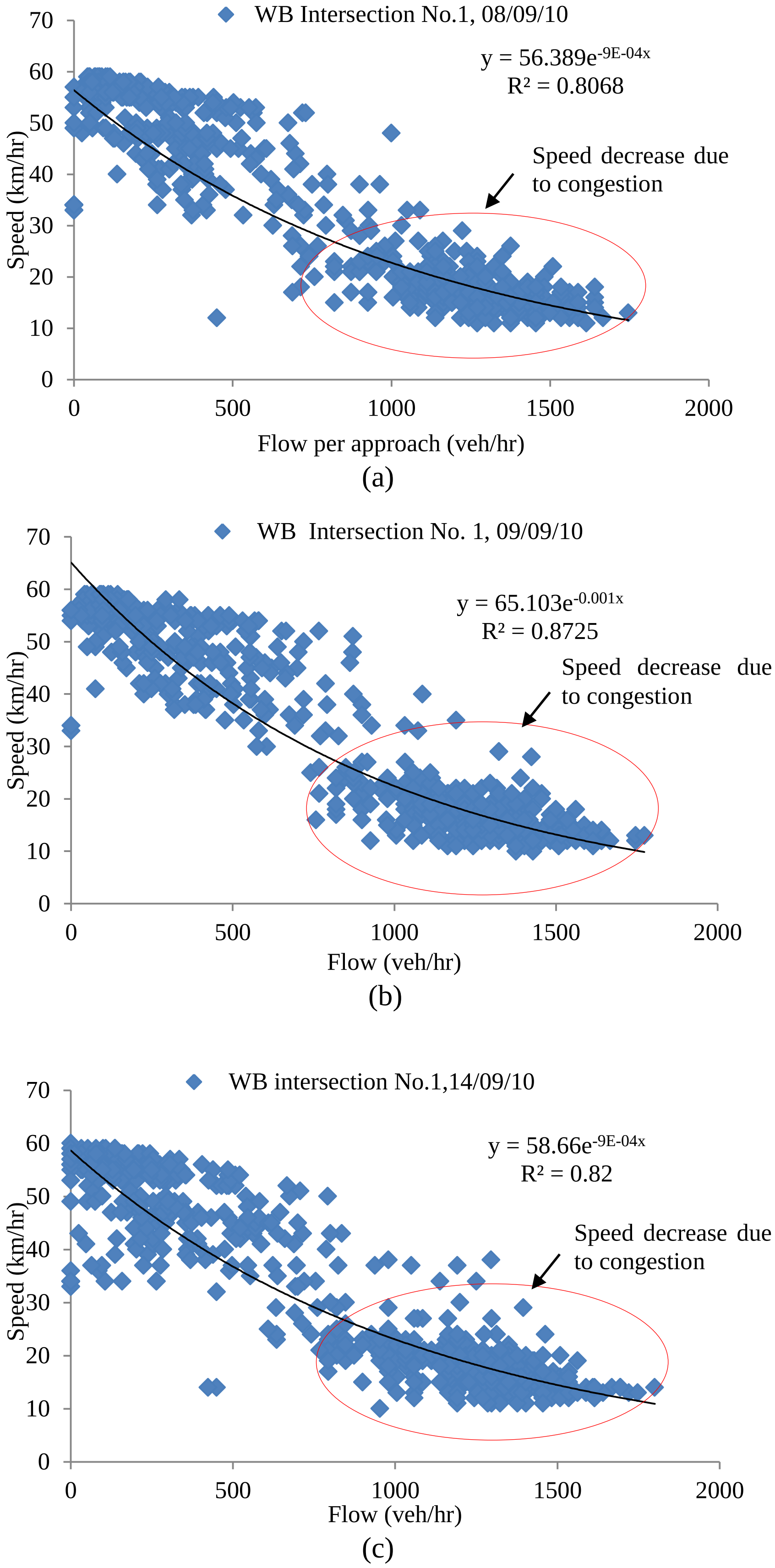
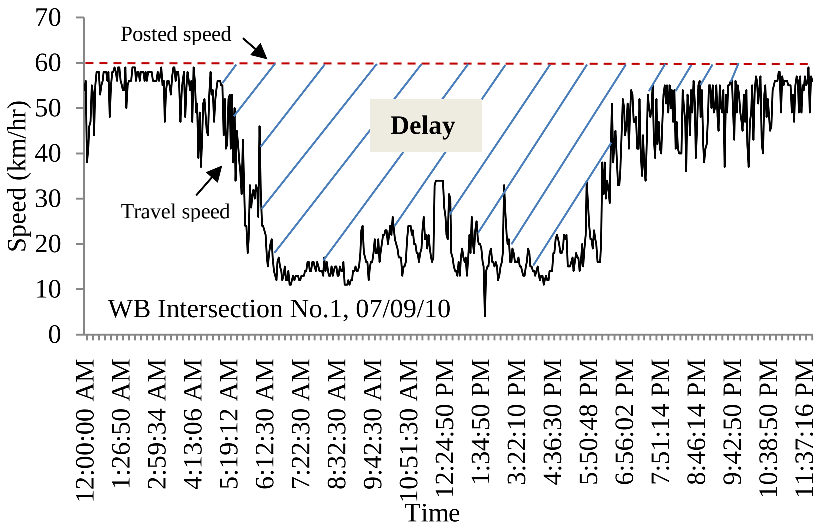


Figure 5.2 Speed-flow relationship for WB link of intersection 1

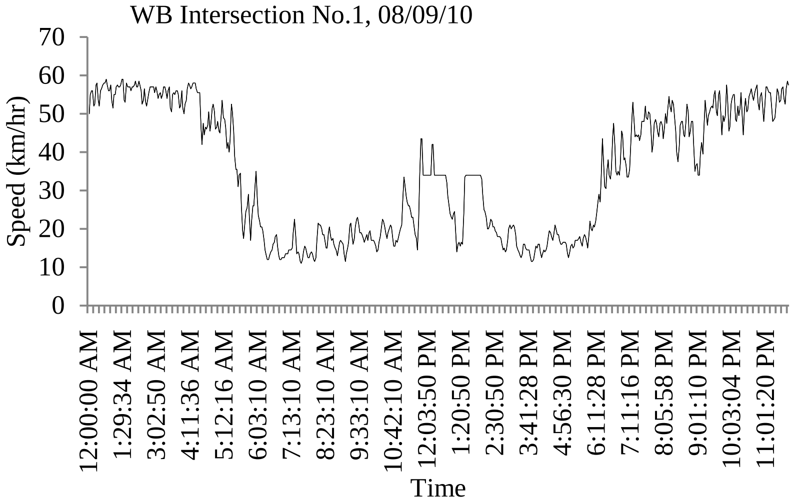
Figure 5.3 (a), (b) and (c) show the average travel speeds for the WB link of intersection 1 from 12:00 am to 11:00 pm for three survey days (Tuesday, Wednesday, and Thursday). The link encountered considerable slow periods during the day. Slower speeds started in the early morning (6.30 am) and continued until late afternoon (6.30 pm). The variation between the posted speed and the average travel speed is a measure of the amount of delay for the vehicles travelling along each link. The delay can be estimated for each individual vehicle by calculating, the variation between the vehicle's travel time and the travel time that the vehicle would have experienced if it had travelled at the posted speed, as shown in Figure 5.3 a. This delay can be a proxy for estimating the amount of disruption for upstream flow by downstream congestion during any period of time.

The problem was clearly identified in this link and most of the delay was caused by the low average speed (or the highest travel times). The reduction in speed may be due to the following reasons:

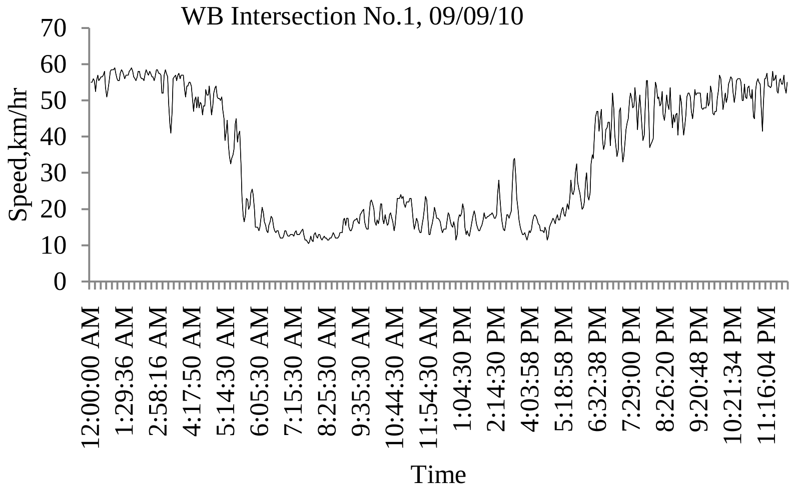
1. The traffic demand along the link exceeded the road capacity,
2. The link joining intersection 2 and 1 is short (400 m), and it carried a high traffic volume. This caused high interaction between moving vehicles through this short link that resulted in increasing the traffic density and reducing the speed.
3. The traffic signal is not allocating sufficient time to discharge all the vehicles entering the link during the red phase of the cycle.
4. The links further upstream from the congested intersection have relatively high travel speeds of approximately 70-80 km/hr. This leads to increase flow prior to the congested link, resulting in further congestion at the link and on the subsequent link.



(a)



(b)



(c)

Figure 5.3 Speed variation for various survey days of STREAMS data

5.3.1.2 Traffic flow in the link joining intersections 3 & 2

Figure 5.4 (a), (b), (c) illustrates the speed-flow relationship for the WB link at intersection 2 for different survey days. The link is more than 1 km long. The figure shows that the average travel speed dropped from 60 km/hr to 30 km/hr despite the posted speed limit being 60 km/hr. This means that the link experienced delays due to the drop in level of performance at intersection 2. This resulted in decreasing the travel speed for a period of time. The average speed concentrated mostly on the lower right part of the figure at approximately 35 km/hr and this was associated with high traffic flow. The maximum flow reached 1600 veh/hr at an average speed of 35 km/hr.

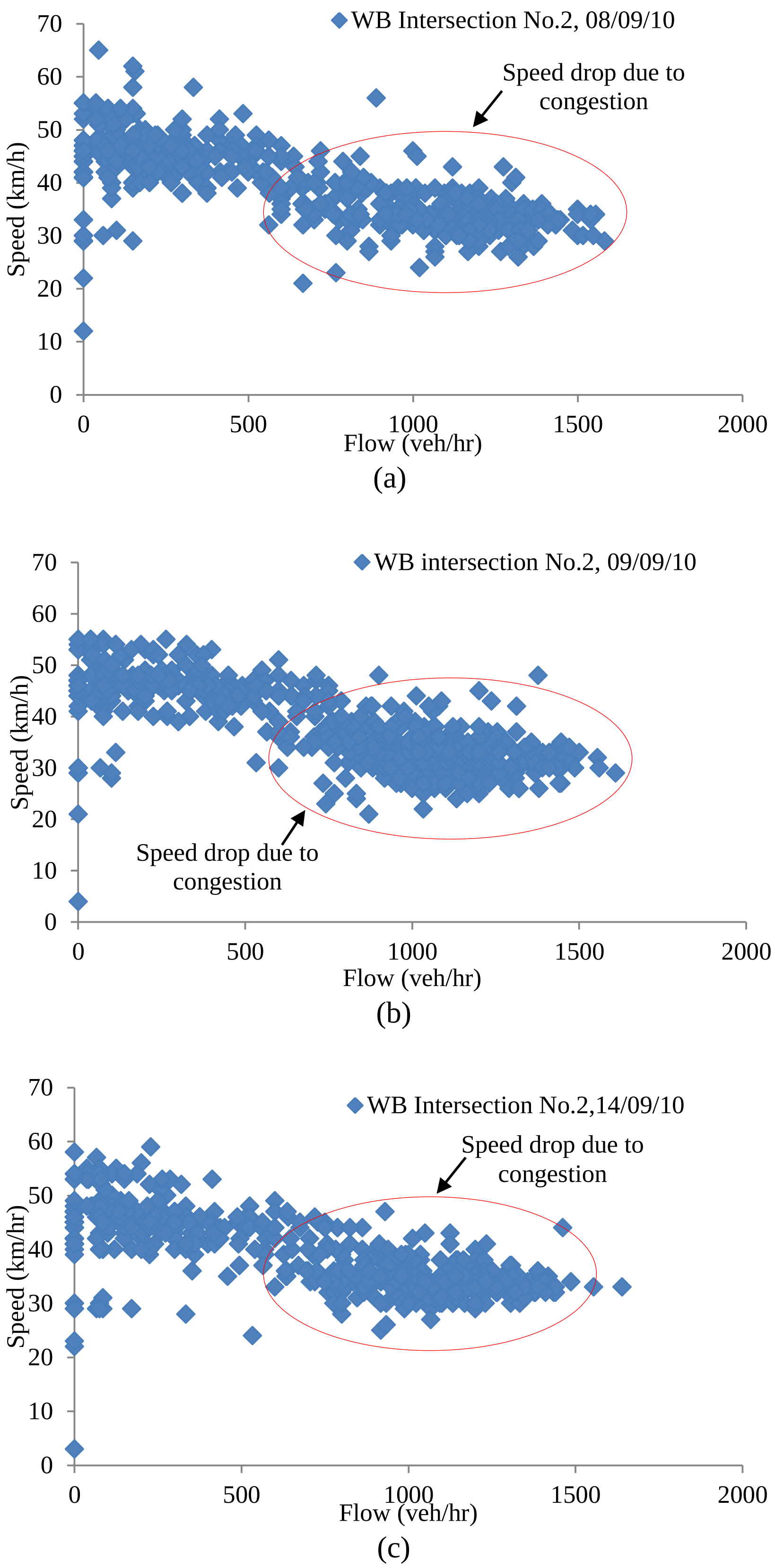
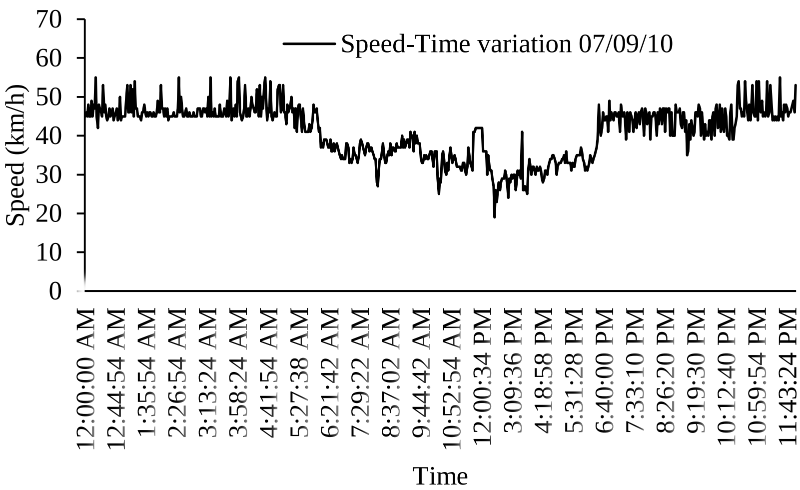


Figure 5.4 Speed-flow relationship for WB link at intersection 2

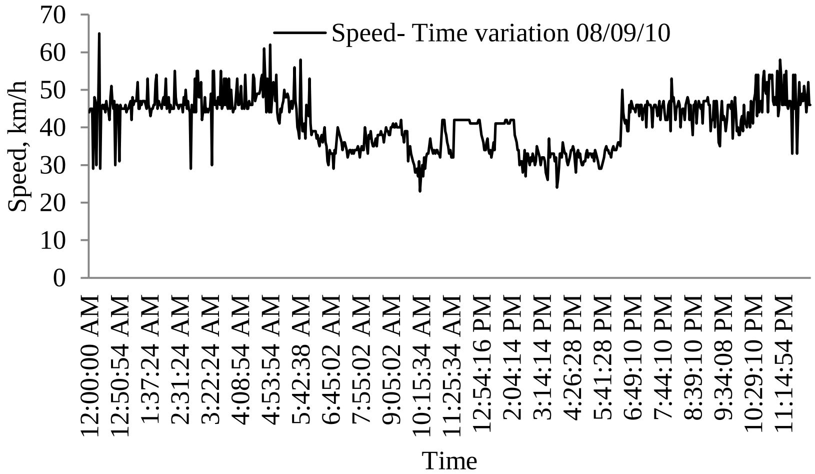
The speed distribution for this link is shown in Figure 5.5 (a), (b), (c). It shows that average speed dropped to 30 km/hr at about 6:00 am and continued fluctuating around this speed until nearly 6:00 pm. By comparing Figure 5.3 and Figure 5.5, the following points can be seen.

1. Traffic speed dropped to less than 20 km/hr for the WB route at intersection 1 for a distance of 400m and the speed decreased to approximately 35 km/hr for the WB route at intersection 2 for a distance of more than 1 km.
2. The congestion period is nearly the same for both links. For example, 6:00 am to 6:00 pm for each link.
3. The speeds of the WB link at intersection 2 recovered to the posted speed of 60 km/hr once the speed on the WB link at intersection 1 increased.

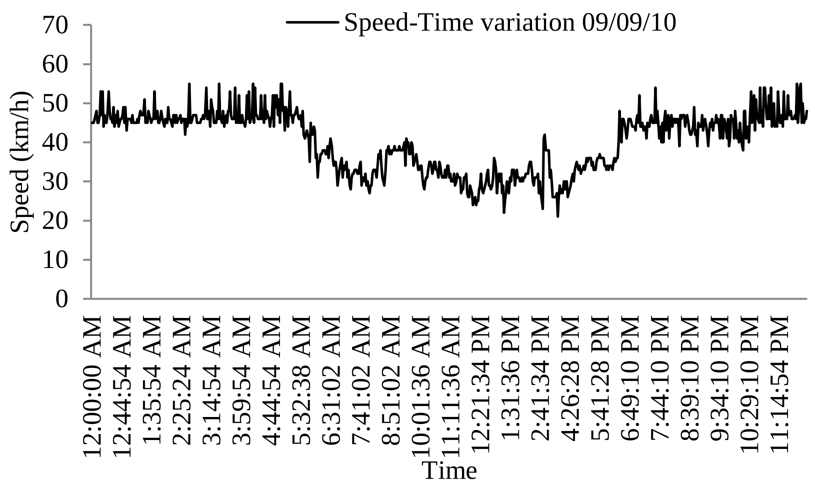
These observations indicate that the congested WB lanes of intersection 1 affected the flow discharged from intersection 2 for a long period of time. This condition caused a long queue and made intersection 2 vulnerable to traffic bottlenecks at any time. These factors decreased the traffic performance and reduced safety. Consequently, the traffic performance of the preceding links and intersections were affected causing increased delay and disruption. This phenomenon made intersection 2 vulnerable to traffic bottlenecks at any time. To increase efficiency of this intersection and its WB link it should be prevented from becoming congested due to the traffic congestion situation at intersection 1.



(a)



(b)

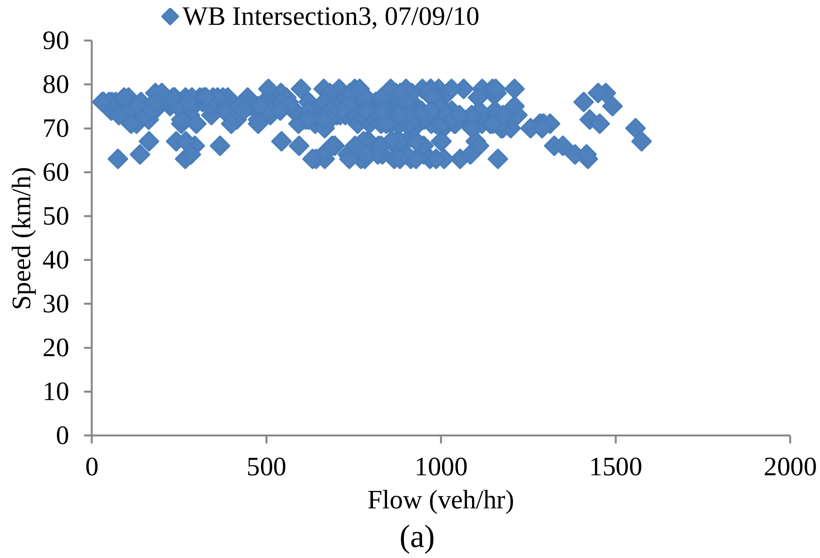


(c)

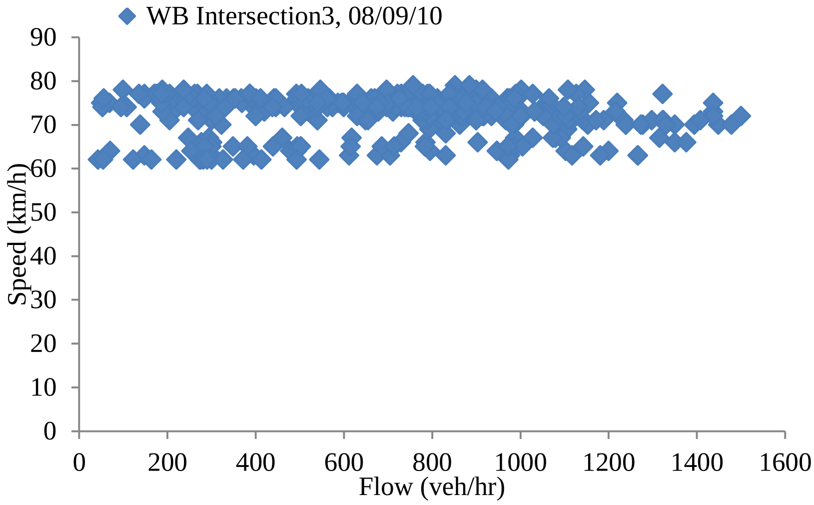
Figure 5.5 Speed variations for WB link at Intersection 2

5.3.1.3 Traffic flow in the link joining intersections 4 & 3

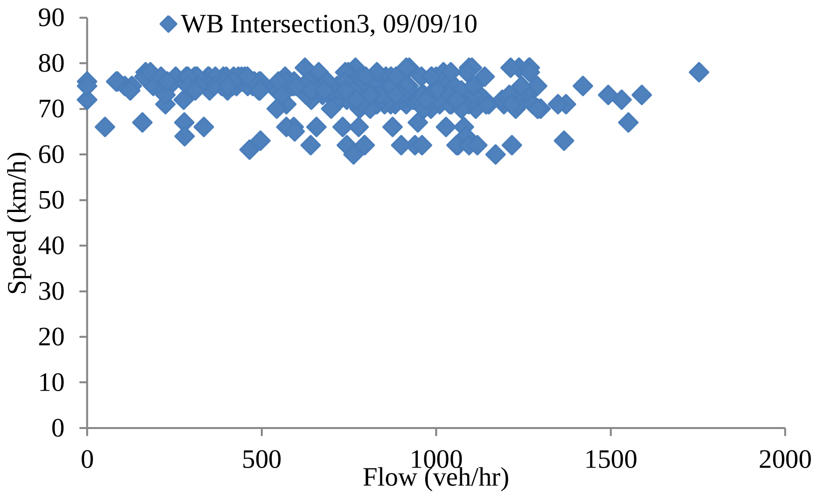
Figure 5.6 (a), (b), (c) shows the speed-flow relationship for the WB link at intersection 3 for different survey days. The flow on this link was smooth and demonstrated little variation in average travel speed even though the flow increased and approached the posted speed of 80 km/h. This means that the traffic on this link is not affected by the traffic at intersections 1 and 2. The speed-flow relationship indicated that no traffic congestion occurred on this link under normal circumstances.



(a)



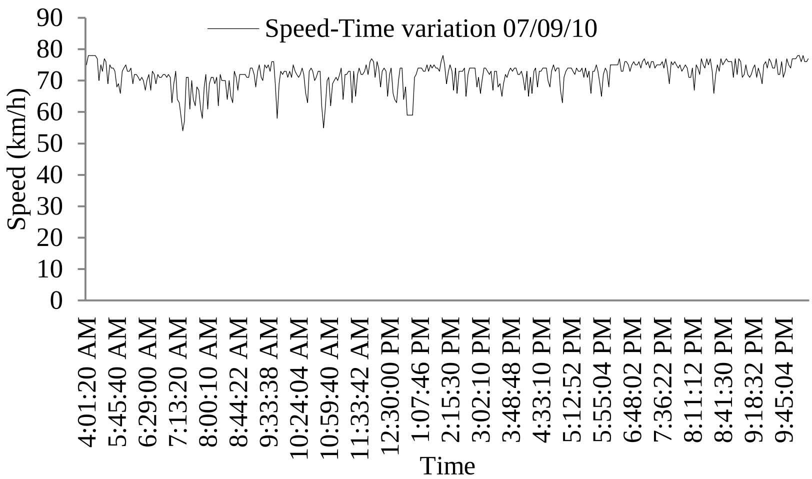
(b)



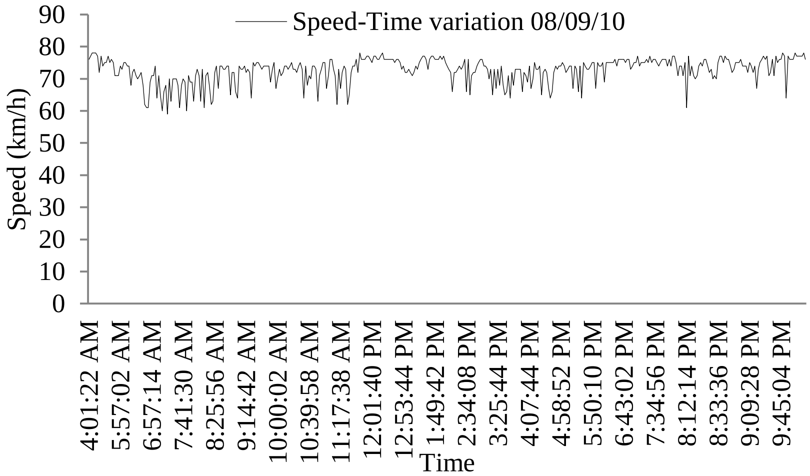
(c)

Figure 5.6 Speed-flow relationship for WB link at intersection 3

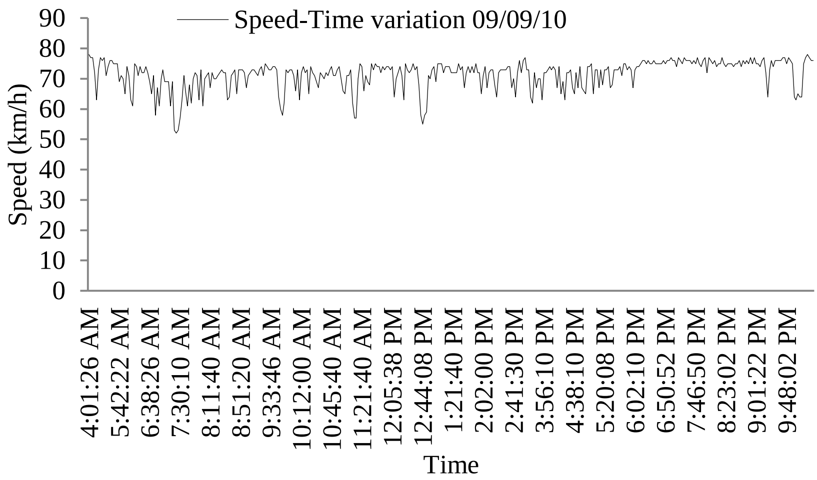
The speed profile for this link is shown in Figure 5.7 (a), (b), (c). It shows that the link had relatively high speeds which were close to the posted speed of 80 km/hr. This indicates the traffic performance of this link was not affected by the traffic at intersection 1. There was very little variation between the posted speed and average travel speed meaning the drivers would not experience much delay. Intersection 3 had a high level of performance. The flow coming from this link travelled at a high speed and contributed to the flow reaching intersection 2 before the existing traffic had cleared intersection 2, (due to the congested situation at intersection 1). As a result, the queue at intersection 2 increased causing further traffic congestion and decreased safety. Intersection 2 will be at risk of traffic breakdown unless appropriate traffic management is implemented that controls the upstream traffic.



(a)



(b)



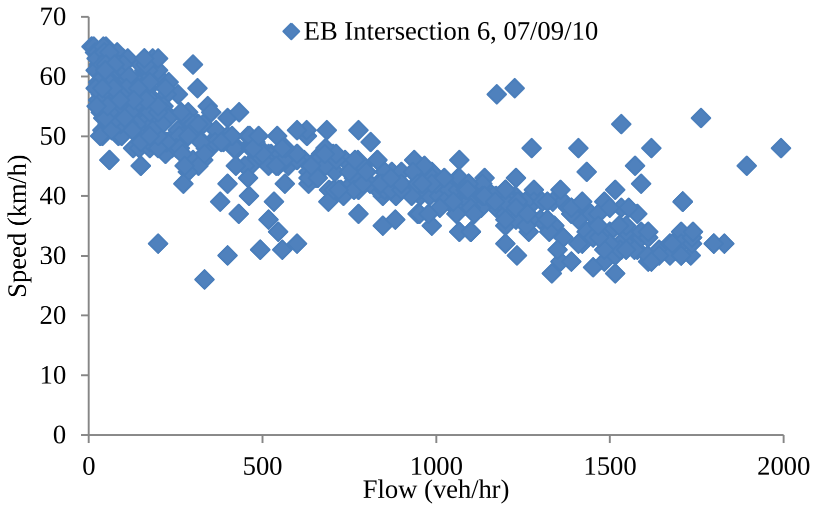
(c)

Figure 5.7 Speed variations for the WB link at Intersection 3

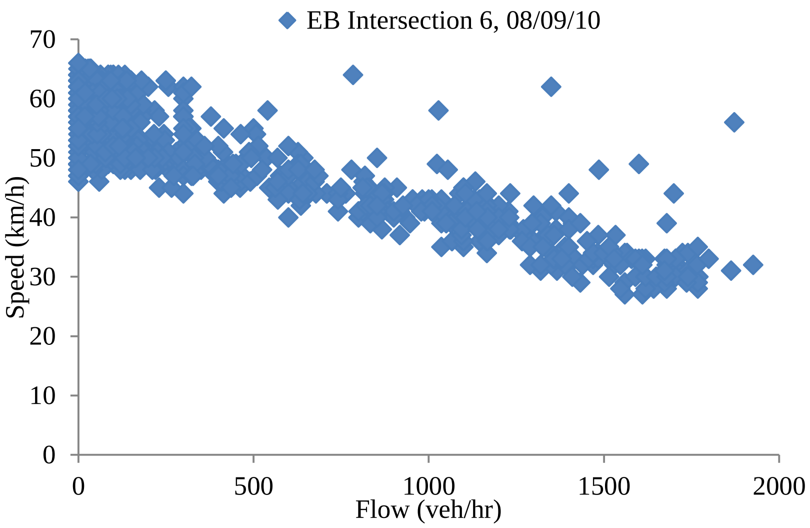
5.3.2 Description of traffic flow in EB direction

5.3.2.1 Traffic flow in the link joining intersections 5 & 6

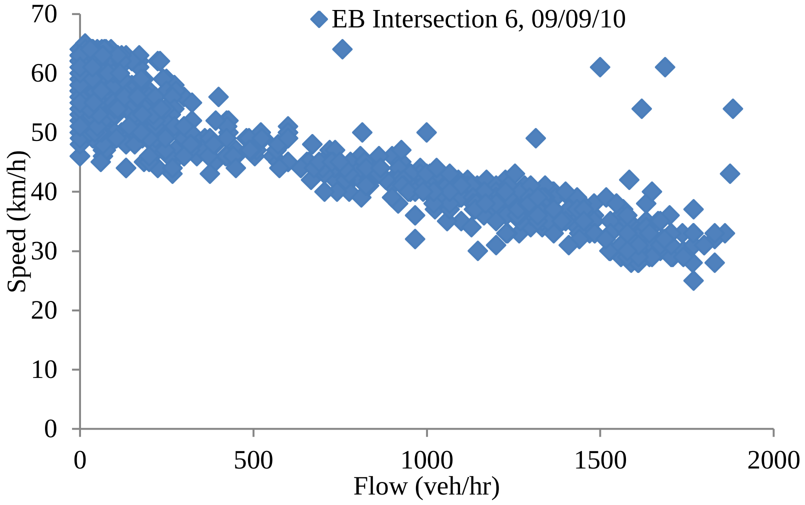
Speed-flow relationships for this section are shown in Figure 5.8 (a), (b), (c) for the survey days. Speed was 60 km/hr during the off-peak period and about 30 km/hr during the peak flow in the heavily travelled eastbound direction. The speed reduced as the flow increased in the link. This scatter plot shows the link started to experience traffic congestion at an average speed of about 45 km/hr (600veh/hr) during the peak hour period resulting in increased delay and safety risk.



(a)



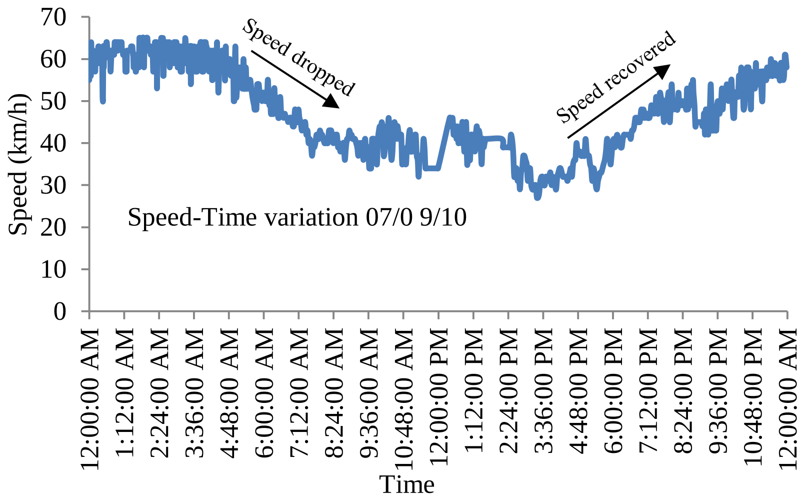
(b)



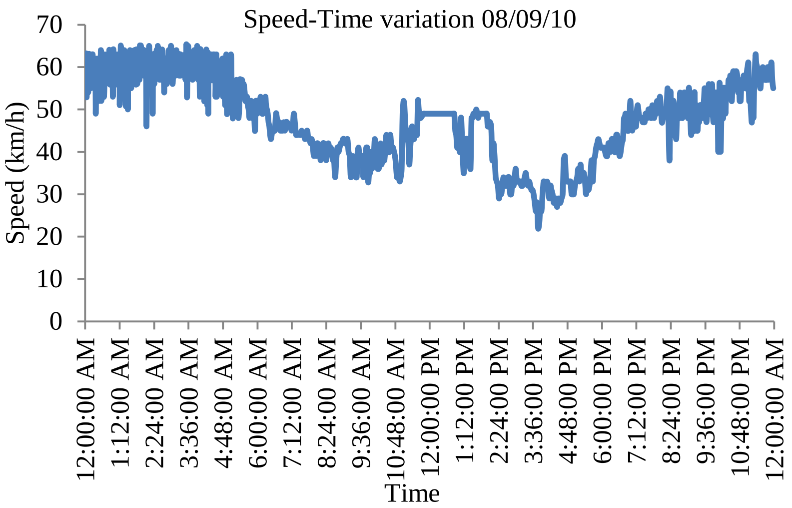
(c)

Figure 5.8 Speed-flow relationships for the EB link at intersection 6

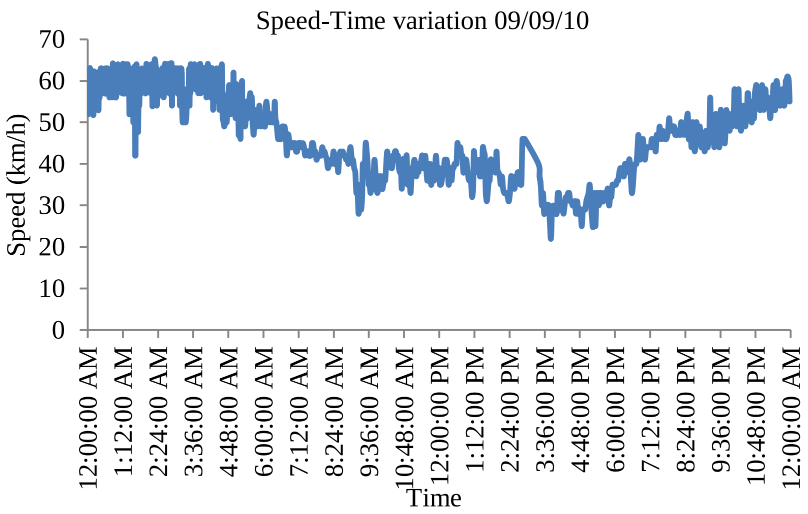
Figure 5.9 (a), (b), (c) plots the speed variation from 12:00 am to 12:00 pm for different survey days. The speed started decreasing at 5:15 am and continued at low speed until around 5:17 pm. After this time, the speed started to recover to the posted speed of 60 km/hr.



(a)



(b)

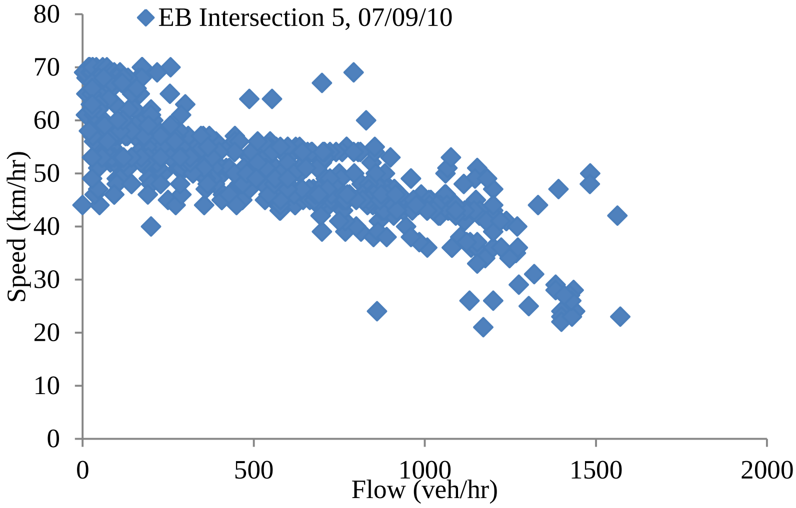


(c)

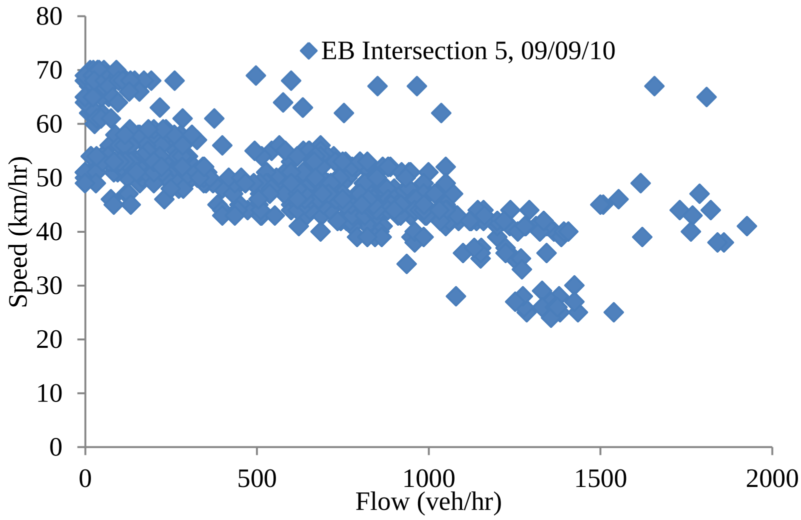
Figure 5.9 Speed variations for the EB link at Intersection 6

5.3.2.2 Traffic flow in the link joining intersections 4 & 5

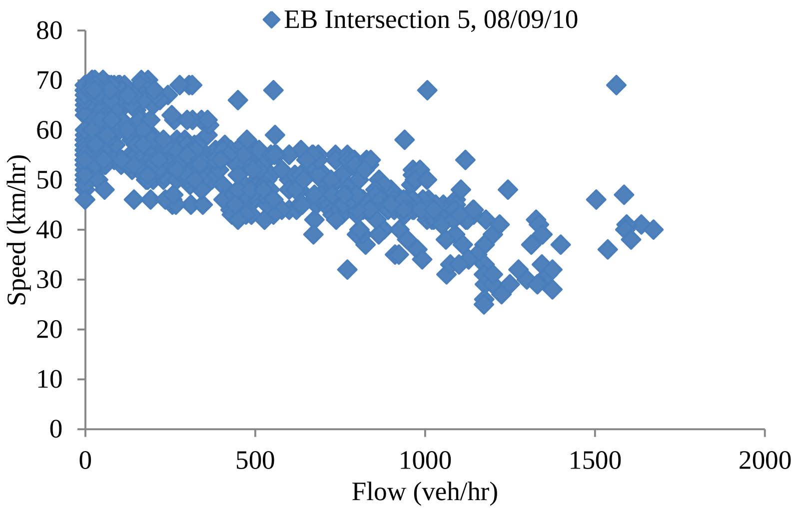
The speed-flow relationship indicated in Figure 5.10 (a), (b), (c) shows that the speed decreased from a maximum of about 70 km/hr at off-peak periods to a minimum of about 25 km/hr at peak periods.



(a)



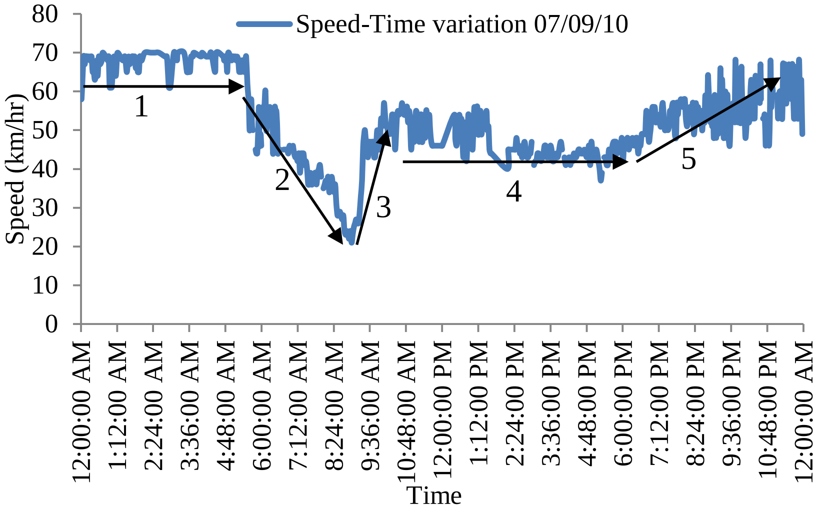
(b)



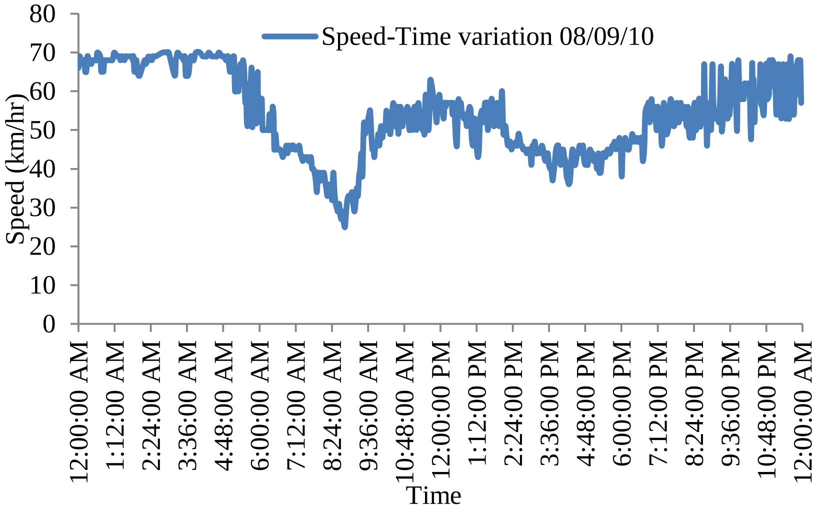
(c)

Figure 5.10 Speed-flow relationships for the EB link at intersection 5

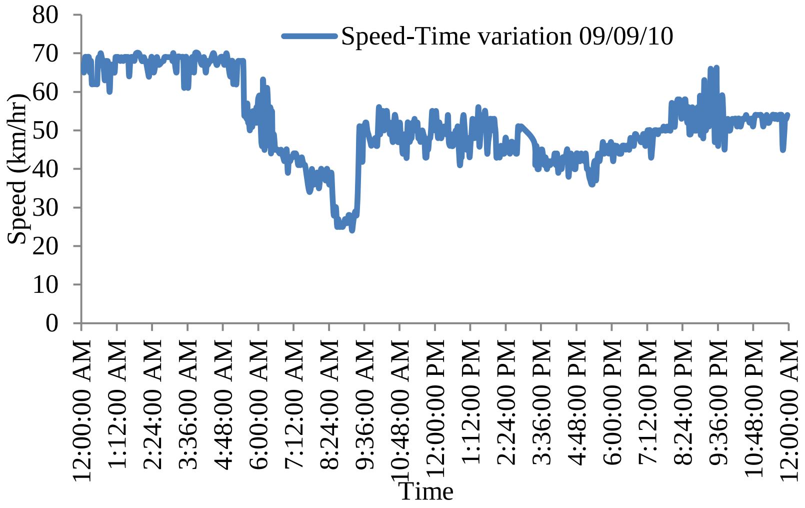
Figure 5.11 (a), (b), (c) shows the speed variation for the different survey days. In the early morning, the speed was almost constant at about 70 km/hr due to low traffic volume as indicated at point 1 (Figure 5.11a). After about 5:00 am, the traffic volume increased causing the speed to drop to about 25 km/hr by 9:00 am as shown at point 2. After 9:00 am, the speed quickly increased to about 55 km/hr (Point 3), and continued at around 45 km/hr until about 7:00 pm, as shown at point 4. The speed then started to recover to the posted speed of 70 km/hr as a result of reduction in traffic (Point 5).



(a)



(b)

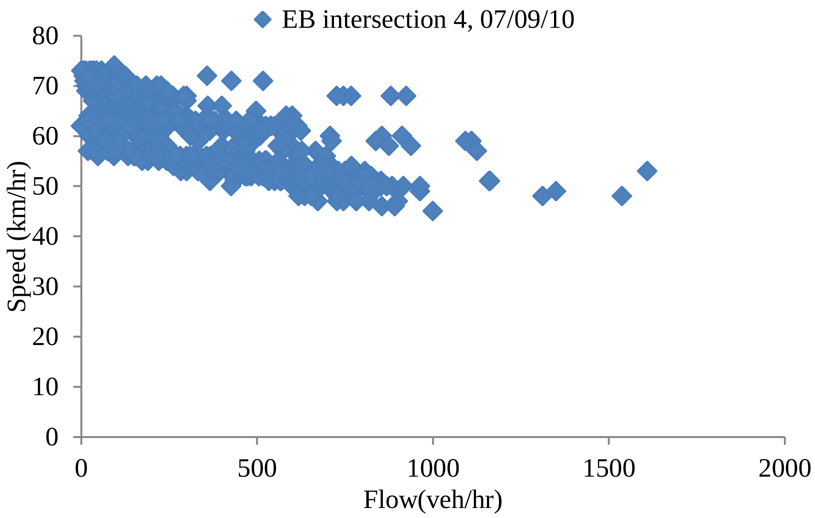


(c)

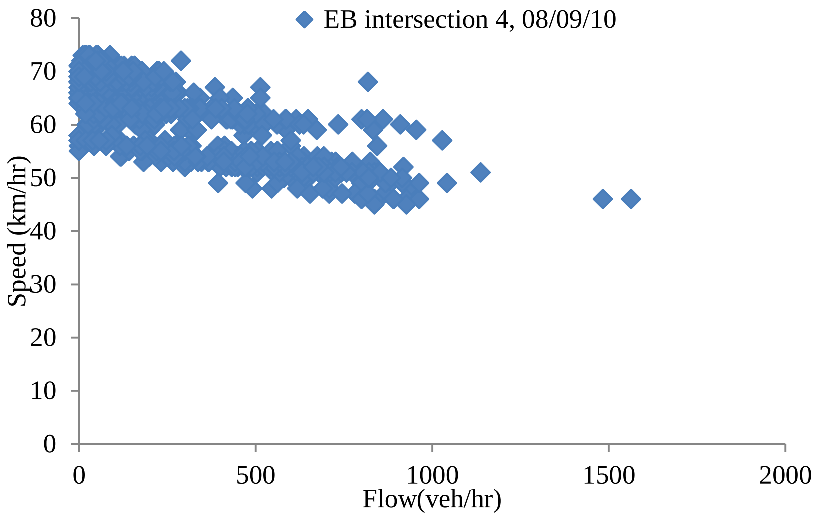
Figure 5.11 Speed variations for the EB link at Intersection 5

5.3.2.3 Traffic flow in the link joining intersections 3 & 4

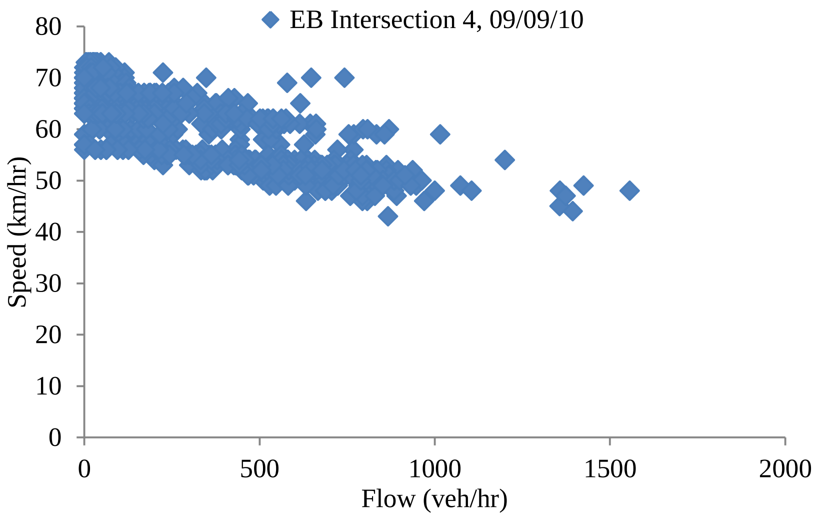
The speed-flow relationship for this link showed relatively little reduction in travel speed during the day. The minimum speed on this section was about 45 km/hr whereas the speed in the previous section was 25 km/hr during the peak period. The traffic performance was better than the traffic performance of the previous link. Figure 5.12 (a), (b), (c) shows the speed-flow relationship for the link.



(a)



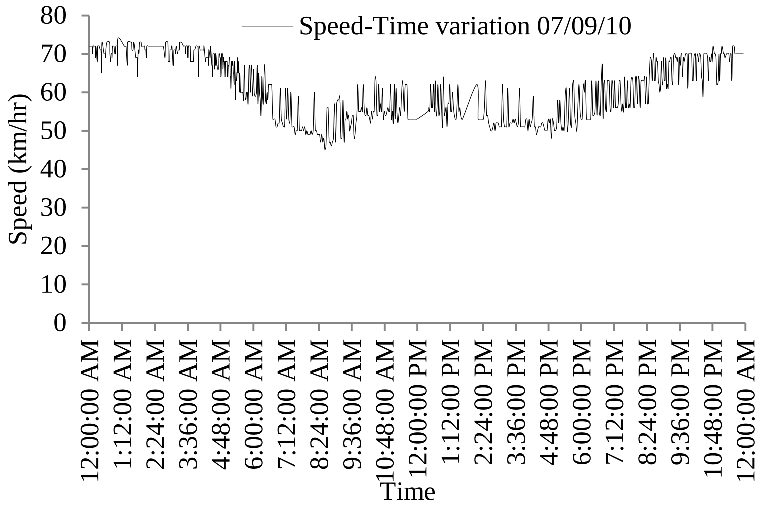
(b)



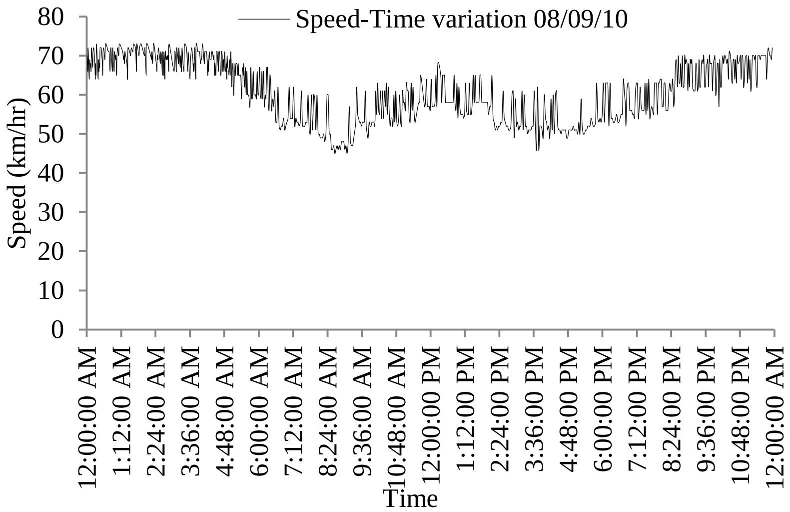
(c)

Figure 5.12 Speed-flow relationships for the EB link at intersection 4

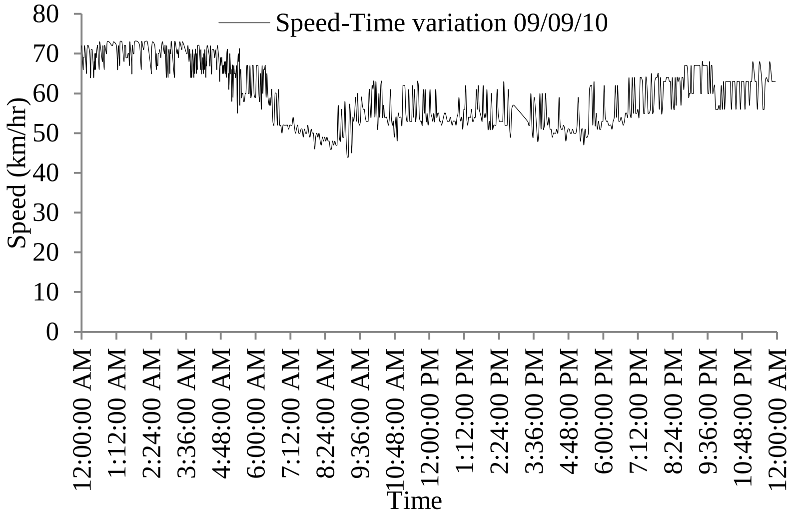
Figure 5.13 (a), (b), (c) show the profile of speed during the day which gradually decreased after 5:00 am to an average of 50 km/hr and this lasted until 5:00 pm. The speed shows relatively low variation between the posted speed of 70 km/hr and the average speed during the peak period. The disruption of the flow from intersection 4 was minor and the signal operation was acceptable. This link experienced smooth flow and a low delay when contrasted with other links.



(a)



(b)

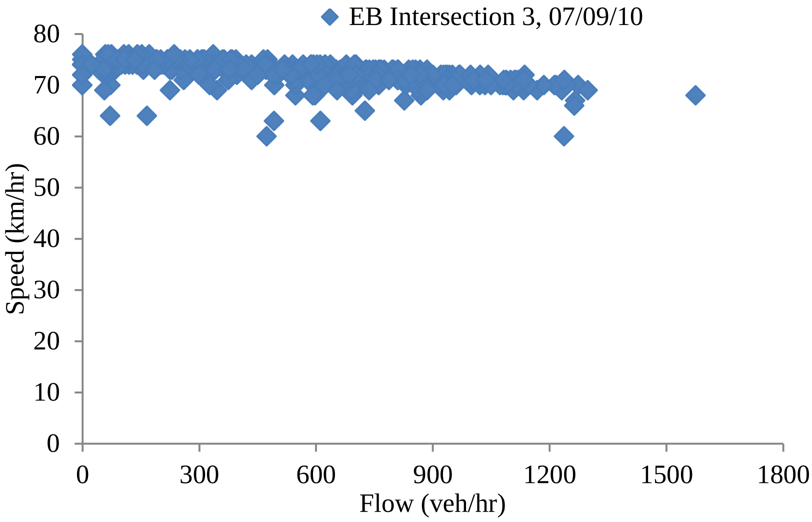


(c)

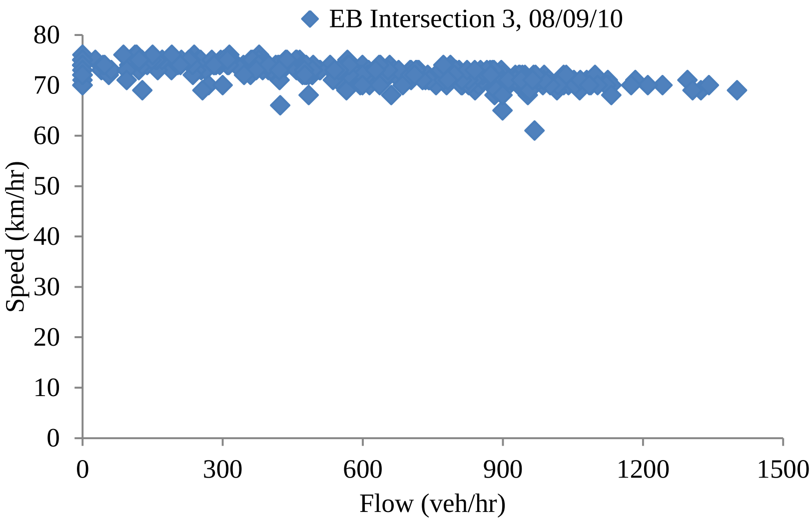
Figure 5.13 Speed variations for the EB link at Intersection 4

5.3.2.4 Traffic flow in the link joining intersections 2 &3

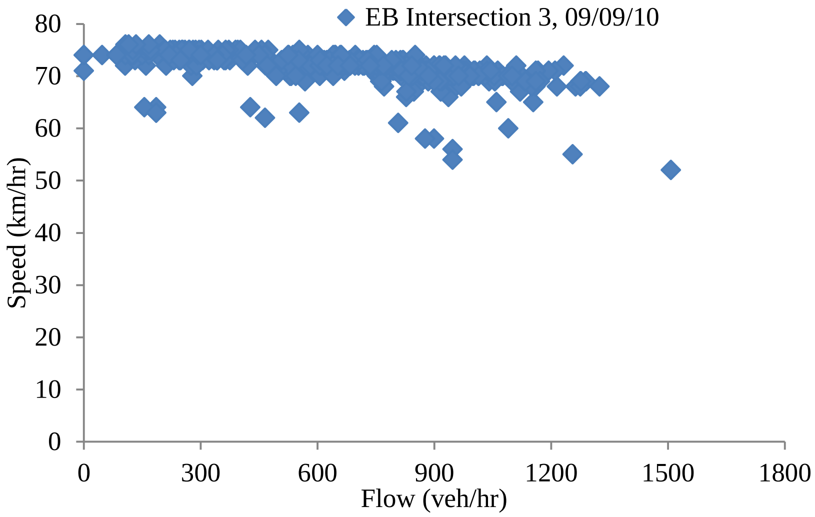
The speed-flow relationship for this link shows good consistent traffic performance with the travel speed unaffected by increasing traffic volume during the day. The speed remained nearly steady and close to the posted speed of 70 km/hr. The traffic on this link travelled well and no congestion occurred. The link exhibited high traffic performance when compared to other EB links. Figure 5.14 (a), (b), (c) show the speed-flow relationship for different survey days.



(a)



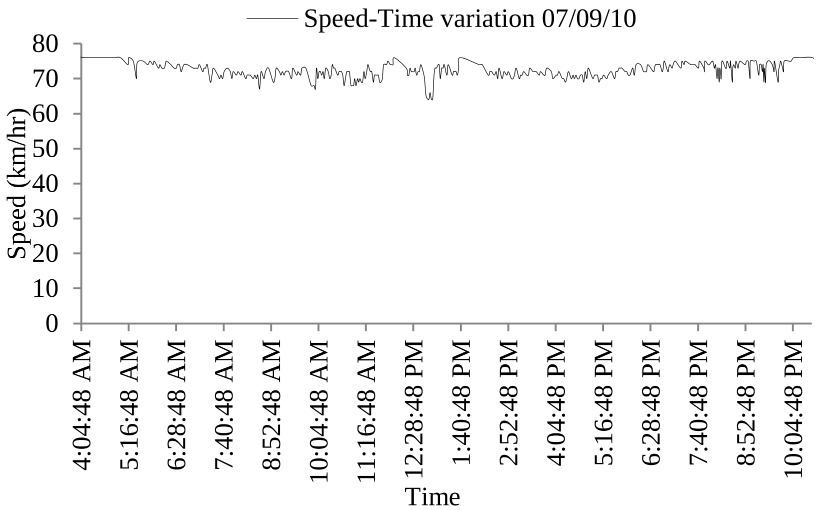
(b)



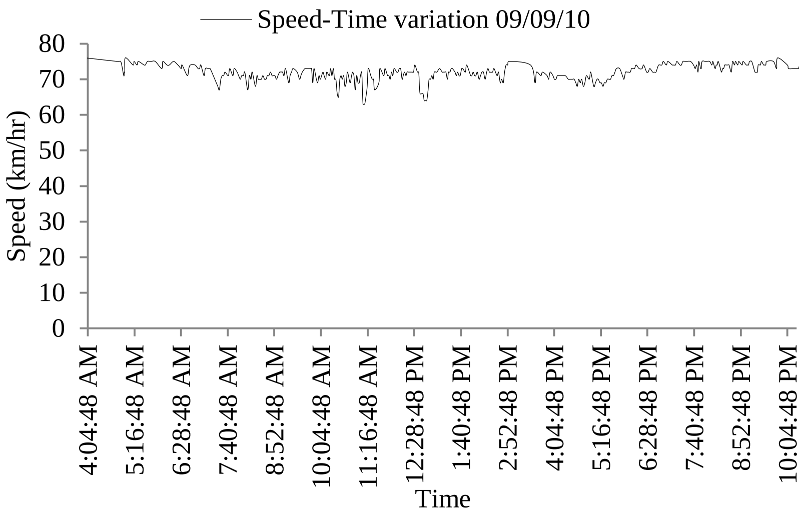
(c)

Figure 5.14 Speed-flow relationships for the EB link at intersection 3

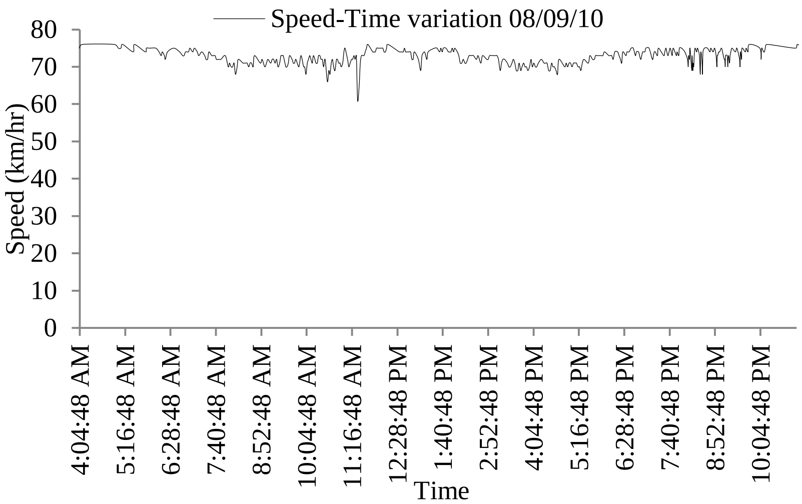
Figure 5.15 (a), (b), (c) shows the speed for the survey days. The figures show that increasing traffic volumes did not disrupt travel speeds even during the peak hour period. The speed was nearly constant at around 70 km/hr and the traffic along this link did not experience much delay due to the high speed (short travel time) and low disruption from intersection 3.



(a)



(b)



(c)

Figure 5.15 Speed variations for the EB link at Intersection 3

5.4 Summary

The link joining interactions 5 and 6 experienced a high level of delay due to a low speed of 30 km/hr when compared to preceding links in the same direction of flow. The speeds for the links more distant from intersection 5 to 6 link increased. Vehicles experienced long delays in the link between interaction 5 to 6 and short delays in the links away from intersection 6. The speed was 40 km/hr for the link joining intersections 4 and 5, 50 km/hr for the link joining intersections 3 and 4 and 70 km/hr for the link joining intersections 2 and 3.

Travel speed distribution along the WB and EB directions can be seen in Figure 5.16. Four speed ranges appeared in the WB and EB directions during the peak hour. Firstly, a speed of 30-40 km/hr occurred WB of intersections 1 and 2 and EB of intersection 6. Secondly, a speed of 40-50 km/hr occurred WB of intersections 4, 5 and 6 and EB of intersections 1, 2 and 5. Thirdly, a speed of 50-60 km/hr occurred EB of intersection 4. Fourthly, a speed of more than 60 km/hr occurred WB and EB of intersection 3.

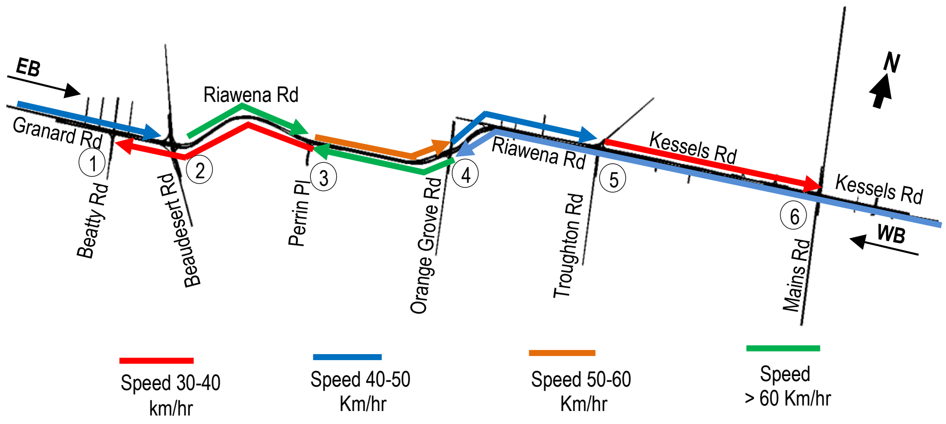


Figure 5.16 Speed distributions in the WB and EB direction

Decreasing the speed of vehicles from the desired speed to a slow speed along the link will contribute to increase travel time (i.e., delay). This delay will attribute to the difference between speeds during the time interval (i.e., increase the difference between speeds will result in more delay and vice versa). In addition, it will contribute to increase traffic road density that may cause more interaction between vehicles thereby reducing the traffic safety. Speed-flow relationships showed evidence of different traffic performance situations along the WB and EB links and speed-time profiles illustrated the location and duration of traffic congestion along the study area. The speed-time profiles present a detailed description about the traffic situation for all links. These profiles allow us to understand the amount of congestion in the different links. They also assist the researcher in understanding the physics of the congestion over these links. This is necessary to develop a traffic solution to relieve or mitigate the traffic congestion.

Reduction in speed was caused mainly by interruption to flow by the intersections. This interruption to the flow affected performance at the downstream intersections. Measurement of the variation between the posted speed and average speed along the links provides an estimate of the amount of delay and the level of performance.

The most critical areas were the WB link at intersection 2 and the EB link at intersection 6. Their lower speeds need to be managed and controlled. The speeds for the links away from intersections 2 and 6 increased gradually.

Uneven distribution of speed increased the flow towards the congested intersection area. Consequently, the performance of the traffic flow, safety and mobility decreased and the likelihood of traffic gridlock increased. Controlling traffic flow during the busiest period is considered a major method for maintaining mobility, and enhancing the level of service and the traffic safety. It is necessary to restrict flow towards links which are already congested or under threat of suffering congestion by applying preventive policies.

CHAPTER SIX
Evaluation of VSL Application to the Griffith Arterial Road

6.1 Introduction

This chapter describes the application of VSL to the study area. The impacts of signalised intersection indicators and traffic flow indicators on a congested segment were investigated. The indicators were examined for both non-control (base condition) and control conditions so as to evaluate the effectiveness of the application of VSLs. The parameters used to evaluate intersection operation were the average queue length, average delay, stopped delay and the average number of stops per vehicle. Macroscopic traffic parameters such as average density, average travel speed and average flow rate were used to explore the effectiveness of VSL. Analysis of the base condition and the VSL scenarios were carried out using VISSIM micro-software.

6.2 Speed Limits management

Two types of speed management models were implemented for the selected study area, internal speed control (close to the congested segment) and external speed control (far from the congested link). As detailed in Chapter 3, the WB lane of intersection 2 and the EB lane of intersection 6 were considered critical intersections (intersection 6 is discussed in detail in Appendix B). VSL was used to model alleviation and enhancement of the traffic characteristics of those intersections in terms of mobility and safety. The average speed, flow and density for the link preceding the WB lanes of intersection 2 were also investigated by this application during rush hour. Three speed ranges within the 30-60 km/hr speed ranges; 30-35, 40-45 and 48-58 km/hr, were applied to the WB lanes at the Beaudesert intersection to control the upstream flow travelling towards the congested intersection. These speed control limits were selected to be lower than the field speeds for the controlled upstream links because the controlled speed limit would not be effective if they exceeded the actual speed. In practice, traffic police and camera speed enforcement can be utilised to raise the driver compliance. Figure 6.1 shows the location of controlled speed (CS) zones along the WB direction. The aim of these control speeds was to control the flow towards intersection 2. The locations of these control speeds are identified in Table 6.1.

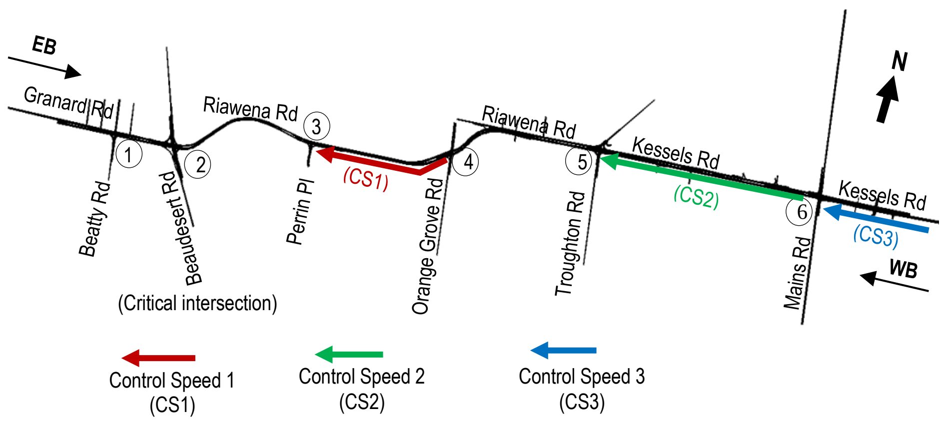


Figure 6.1 Locations of controlled speed in the WB direction

Table 6.1 Identification of controlled speed for WB

Control speed (CS) #	Location	Distance (m)	Type
1	Between intersection 4 & 3	710	Internal
2	Between intersection 6 & 5	1230	External
3	Between the WB traffic inlet & intersection 6	1075	External

The controlled speed, proposed to manage the upstream flow towards the congested area, was activated for four different periods during the rush hour as shown in Table 6.2.

Table 6.2 Control speed activation system

Control Speed (CS) #	Speed limits (km/hr)	Activation time (sec)
1	30-35,40-45,48-58	600-4200(1)
	30-35,40-45,48-58	1200-4200(2)
	30-35,40-45,48-58	1800-4200(3)
	30-35,40-45,48-58	2400-4200(4)
2	30-35,40-45,48-58	600-4200
	30-35,40-45,48-58	1200-4200
	30-35,40-45,48-58	1800-4200
	30-35,40-45,48-58	2400-4200
3	30-35,40-45,48-58	600-4200
	30-35,40-45,48-58	1200-4200
	30-35,40-45,48-58	1800-4200
	30-35,40-45,48-58	2400-4200

(1) Refers to VSL was activated for one hour at the beginning of traffic congestion excluded the warm up period (600 sec).

(2)Refers to VSL was activated for 50 minutes after 600 sec of the beginning of traffic congestion excluded the warm up period.

(3)Refers to VSL was activated for 40 minutes after 1200 sec of the beginning of traffic congestion excluded the warm up period.

(4) Refers to VSL was activated for 30 minutes after 1800 sec of the beginning of traffic congestion excluded the warm up period.

Various VSL management strategies were applied to maintain and manage the traffic flow in the WB lane of intersection 2 (Beaudesert intersection). Table 6.3 shows the VSL scenarios proposed for application along the WB lane of Griffith arterial road U20. As can be seen from Table 6.3 four scenarios were employed to evaluate the performance of the WB lane of intersection 2 during the congested period. In Scenario 1, control speed 1 (CS1) was ON while all other controls towards the critical intersection were OFF. The scenario was tested for activation periods of 600s-4200s, 1200s-4200s, 1800s-4200s, and 2400s-4200s. The first period 600s-4200s means control speed was immediately activated at the initiation of congestion. The second period 1200s-4200s means the control speed was activated 1200s after congestion started to form and so on for the other periods. Scenario 2 presumed that CS1 and CS3 were OFF while CS2 was ON. This scenario was also tested for different activation periods. Scenario 3 presumed that CS1 and CS2 were OFF and CS3 was ON for different activation periods. Scenario 4 presumed that CS1 was OFF and CS2 and CS3 were ON. In this scenario, CS2 was synchronised with CS3 during the whole of the activation period.

Table 6.3 VSLs Scenarios

Scenario #	CS1	CS2	CS3
1	ON	OFF	OFF
2	OFF	ON	OFF
3	OFF	OFF	ON
4	OFF	ON	ON

6.3 Evaluation of Scenario 1 on the performance of intersection 2

As outlined in the previous section four different periods were tested to find the impact of Scenario 1 on the WB lane at intersection 2. Average queue length, average delay, average stopped delay and average number of stops were investigated under control speeds and non-control speed. The intersection parameters were analysed for the first scenario as follows:

6.3.1 Evaluation of average queue length

Table 6.2 shows that evaluation commenced after a minimum 600s because the simulated results before this time were not considered in the evaluation (simulation warm-up period). The model was run without implementing any traffic management to represent the field traffic condition. At this stage, signalised intersection characteristics and the macroscopic parameters were measured to represent the field flow condition. The simulated results were measured over a one-hour calculation. The average queue length was around 44 m under the free flow scenario (Figure 6.2). After applying the first scenario (different control speeds Table 6.3), shortened average queue length values were obtained. Speed activation periods of 600s-4200s, and VSL ranges of 30-35, 40-45 and 48-58 km/hr reduced the queue length to 42 m, 39 m and 40 m respectively (Figure 6.2). For an activation period of 2400s-4200s, the entire VSL range produced the same queue length of 39.5 m. The best reduction in average queue length of 37m was achieved at a VSL range of 40-45 km/hr with a speed activation time of 1800s-4200s.

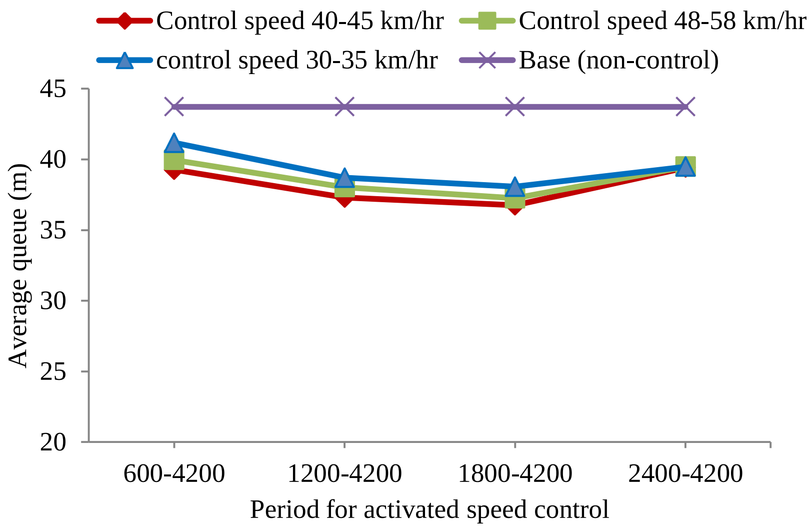


Figure 6.2 Average queue length for Scenario 1

The efficiency of VSLs on this parameter was calculated for each speed limit and speed activation periods. The calculation is based on the variation between the base condition and VSLs control. Figure 6.3 shows the variation in VSL efficiency for Scenario 1. The highest efficiency of VSL (16%) occurred when the speed was controlled between 40-45 km/hr with an activation control time of 1800s-4200s. Control speeds of 48-58 km/hr recorded 15% efficiency and control speeds of 30-35 km/hr recorded 13% efficiency. Speed controls of 40-45 km/hr and 48-58 km/hr yielded similar results due to synchronisation between the vehicle speeds upstream and the signal operation system downstream.

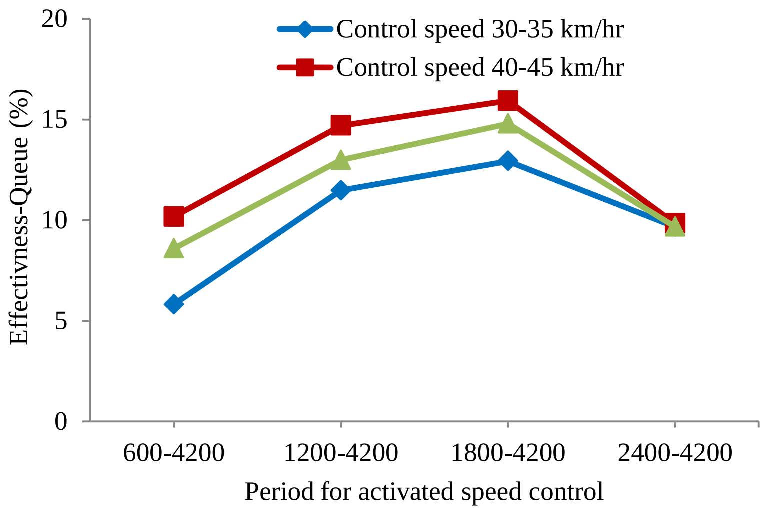


Figure 6.3 The efficiency of Scenario 1 on the average queue length

6.3.2 Evaluation of average delay parameter

Average delay was measured for both base and control conditions. The base average delay was 55 seconds per vehicle (no control). This means the LOS (Level of Service) at the intersection 2 was operating at LOS E (Table 2.1).
Delay results obtained by applying Scenario 1 are shown in Figure 6.4. It shows that VSL ranges of 40-45 and 48-58 km/hr have nearly the same effect on improving the average delay for all speed activation times. A VSL range of 30-35 km/hr reduced

the delay times less than the other two speed limits. The maximum reduction was achieved after applying speed control limits 40-45 km/hr for activation period of 1800s-4200s. The delay reduced from 55 seconds to 46 seconds resulted in improving the LOS E to LOS D. All speed control limits improved the performance at intersection 2 to LOS D but with different degrees of effectiveness. It can be seen in Figure 6.4 that delay improvements for all types of VSL control became almost constant when the control speed was activated for 2400s-4200s. This means delaying the operation of VSLs after the initiation of congestion by more than a quarter of the length of the congestion time yields a constant amount of improvements. This limitation in improvement can be attributed to the traffic situation becoming so dense as to reduce the variation in vehicle speeds to a constant speed threshold. Figure 6.4 also shows that Scenario 1 achieved the best outcome for a speed activation time of 1800s-4200s.

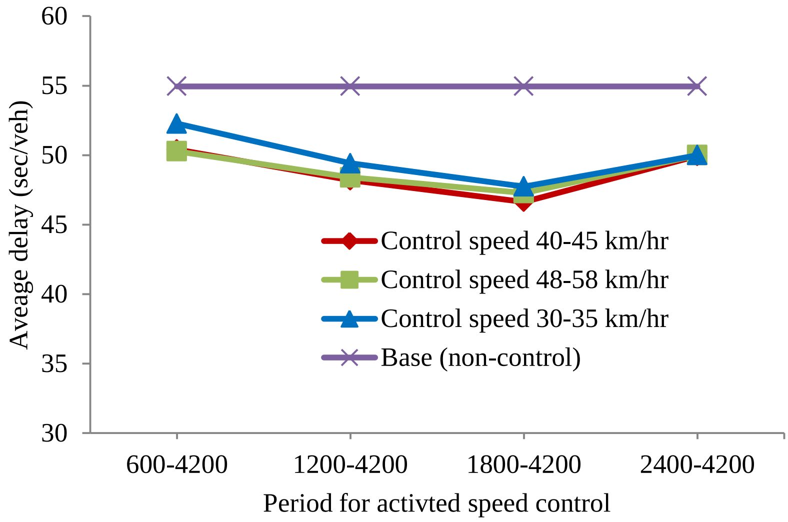


Figure 6.4 The trend of delay under applying scenario 1

The efficiency of Scenario 1 in terms of average delay was determined by finding the change in delay results for both non-control and control situations, as shown in Figure 6.5. This figure shows that the best efficiency was achieved at a speed activation time of 1800s-4200s where 15%, 14% and 13% improvements were attained from applying VSL ranges of 40-45, 48-48 and 30-35 km/hr respectively. Delaying the application of VSL after the initiation of congestion was more effective. This is because VSLs have more potential for reducing delay at higher congestion levels. Similar findings have also been reported by Hadiuzzaman et al. (2012), (Papageorgiou et al., 2008), and Jonkers et al. (2011).

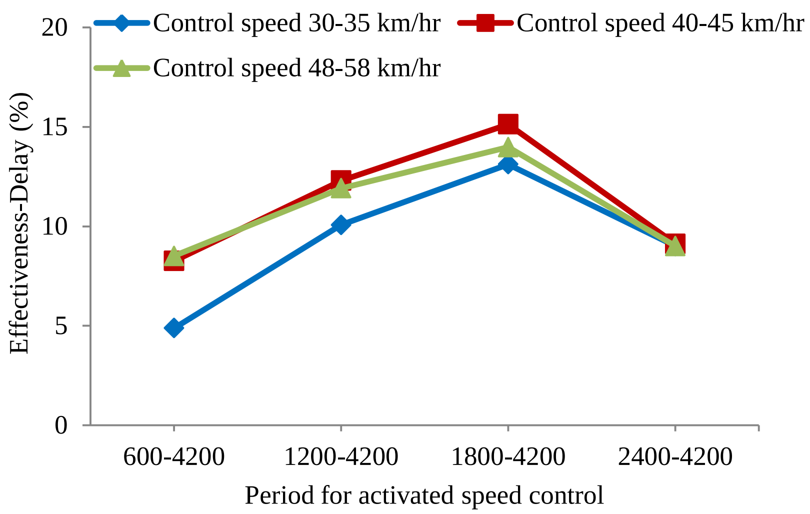


Figure 6.5 Efficiency of scenario 1 on delay indicators

The savings due to reduced delays were calculated using the difference between the average delays per vehicle multiplied by the total intersection throughput for the base conditions and the average delays per vehicle multiplied by the improved intersection throughput for the control conditions. The process used the time savings for the only evening peak hour. The result is then multiplied by the number of weekdays per a year (240 days) and the delay time cost values to determine the total annual cost savings.

The uncontrolled annual delay constituted 4,884 hours for average intersection throughput of 1,332 veh/hr computed based on one evening peak hour. Imposing a speed limit 40-45 km/hr reduced the annual delay time to 4,391 hours due to the intersection throughput increasing from 1,332 to 1,432 veh/hr (Figure 6.12). The reduced delay reduction saved 493 vehicle hours per year for travel on the WB of intersection 2. Applying a vehicle delay time value of $10.6 per hour (Booz, 2003) the savings would be roughly $5,226. Speed control limits of 30-35 and 48-58 km/hr produced annual time savings of 427 and 460 hours corresponding to VSL efficiencies of 13% and 14% respectively. Figure 6.6 shows the savings in hour/year when applying Scenario 1. Table 6.4 summarises the effect of Scenario 1 on the annual savings occurred by reducing the delay.

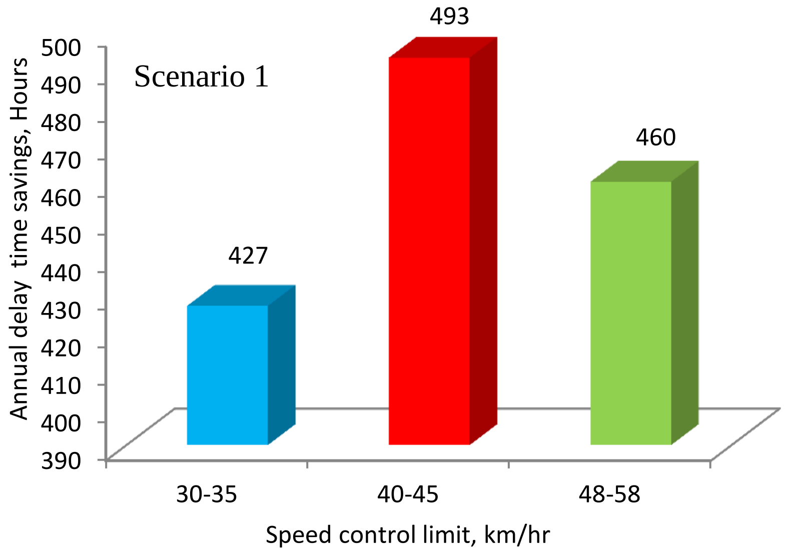


Figure 6.6 Annual time savings due to Scenario 1, VSL

Table 6.4 Annual time savings from Scenario 1

Annual delay time without control (Hour)	Scenario1 speed control limit (km/hr)	Efficiency (%)	Annual delay time savings	
			(Hour)	($AUD)
4,884	30-35	13	427	4,526
	40-45	15	493	5,226
	48-58	14	460	4,876

6.3.2.1 Statistical significant of delay results

To evaluate the influence of of VSL on delay time parameter for traffic signal, IBM SPSS (International Business Machines, Software package and Service Solutions) software was used. The method is based on the comparison of delay time models using data before and after the implementation of the VSL. One-Way ANOVA (Analysis of Variance) at 95% level of confidence were considered to estimate the significant of employing VSL. Table 6.5 describes the variance in results for the base condition and VSL. Table 6.6 indicates that VSL applications has statistically significant impact on vehicle delay time results because the significant value for ANOVA was less than the confidence interval (5%), that is 0.001(< 0.05).

Table 6.5 Delay statistical descriptive

Status	Number of samples	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base Condition	30	54.94	9.04	1.65	51.56	58.32	35.86	72.14
VSL	30	46.63	9.10	1.66	43.23	50.03	29.30	60.00
Total	60	50.79	9.92	1.28	48.22	53.35	29.30	72.14

Table 6.6 Delay, One-Way ANOVA

Status	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	1036.70	1	1036.70	12.58	.001
Within Groups	4777.56	58	82.372		
Total	5814.26	59			

6.3.3 Evaluation of average stopped delay

Figure 6.7 shows that activation of VSLs in the range of 30-35 km/hr at the initiation of congestion is not recommended because it creates a higher delay when compared with higher control speeds at the same activation time. The differences in delay-controlled results were reduced for all VSL applications when traffic Scenario 1 was launched 15 minutes after congestion started. This may be due to optimum synchronisation between vehicle speeds upstream and the signal operation system downstream. The similarity in delay output for all controlled speeds for this period caused by the actual speed of the controlled link at CS1 being lower than the speed limits of 40-45 km/hr and 48-58 km/hr and similar to the speed control range 30-35 km/hr. The average stopped delay dropped significantly from 45 second for the base condition to 38 second for controlled speeds of 40-45 km/hr and 48-58 km/hr at an activation time of 1800s-4200s.

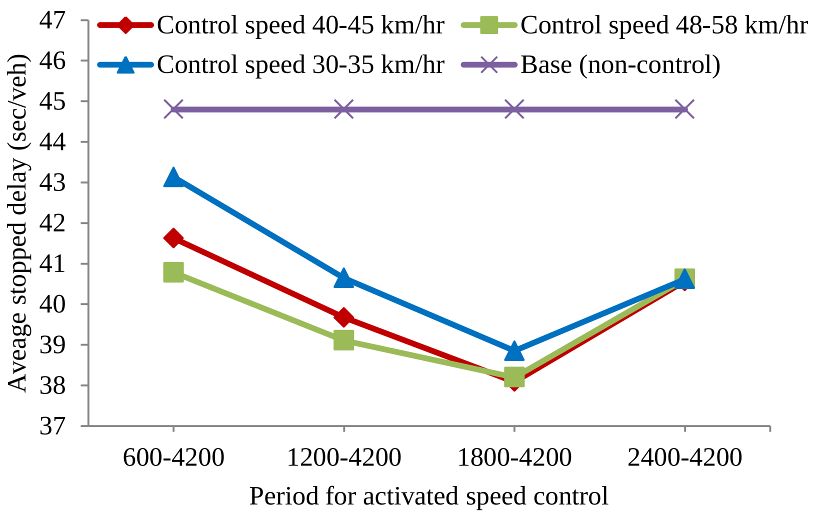


Figure 6.7 The impact of scenario 1 on the average stopped delay

The efficiency of VSLs is indicated in Figure 6.8. The highest percentage of efficiency (15%) was achieved at a speed activation time of 1800s-4200s for speed control limits of 40-45 and 48-58 km/hr.

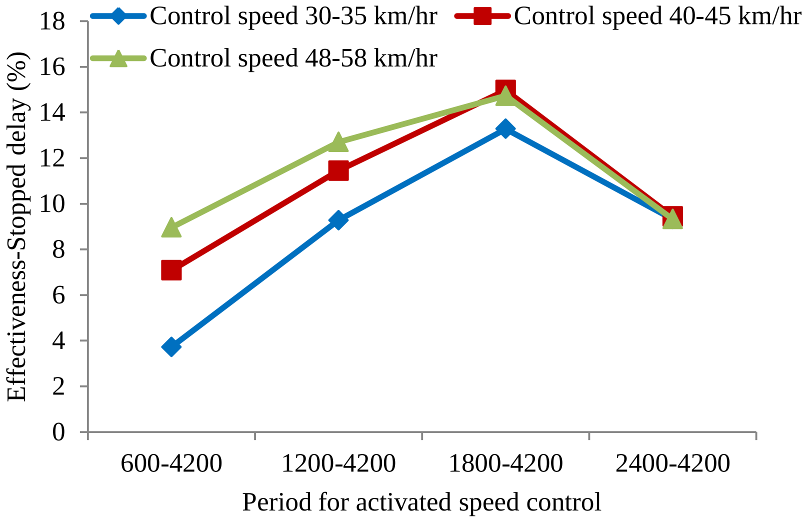


Figure 6.8 The efficiency of scenario 1 on stoped delay indicators

6.3.4 Evaluation of average number of stops

The average number of stops in the WB lane of intersection 2 was investigated. The change in the number of stops is a good indicator of traffic flow characteristics and traffic safety issues. Reducing the number of stops reduces the likelihood of rear-end collisions and may minimise delay at signalised intersections.

The total cost of all crashes per year in Australia is estimated at $20.64 billion (Cairney, 2013). Turner et al. (2008) reported that 14% of all crashes occurred at traffic signals in Queensland. Ogden and Newstead (1994) estimated that 21.8% of the crashes at traffic signals occurred due to rear-end collisions. Rodegerdts et al. (2004) stated that the most frequently occurring collision is a rear-end crash, which represents 42% of all reported signalised intersection crashes.

The number of stops decreases as the average queue length decreases when VSLs are applied. Figure 6.9 shows the results achieved for Scenario 1 at intersection 2, WB lane. The most significant reduction was achieved with speed controls of 40-45 and 30-35 km/hr at two speed activation periods of 1200s and 1800s. A speed control of 48-58 km/hr produced the lowest reduction compared to the other two speed controls. This was because vehicles at this threshold speed range formed a longer queue than in the other two speed controls, thereby increasing the number of stops.

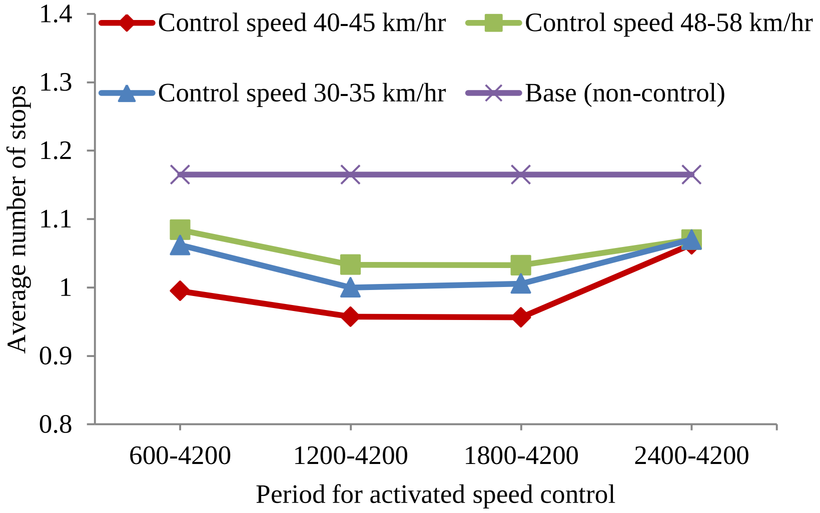


Figure 6.9 The application of scenario 1versus average number of stops

This traffic indicator was potentially affected by all VSL ranges as illustrated in Figure 6.10. Scenario 1 immediately produced more than a 14% improvement at a control speed of 40-45 km/hr if this was initiated at the formation of congestion. This percentage increased to 18% at speed activation times of 1200s and remained constant till 1800s. Control speeds of 30-35 km/hr were the second most effective in reducing number of stop which ranged from 9% to 14%. Control speeds of 48-58 km/hr were the third most effective and achieved a 7% to 11% improvement.

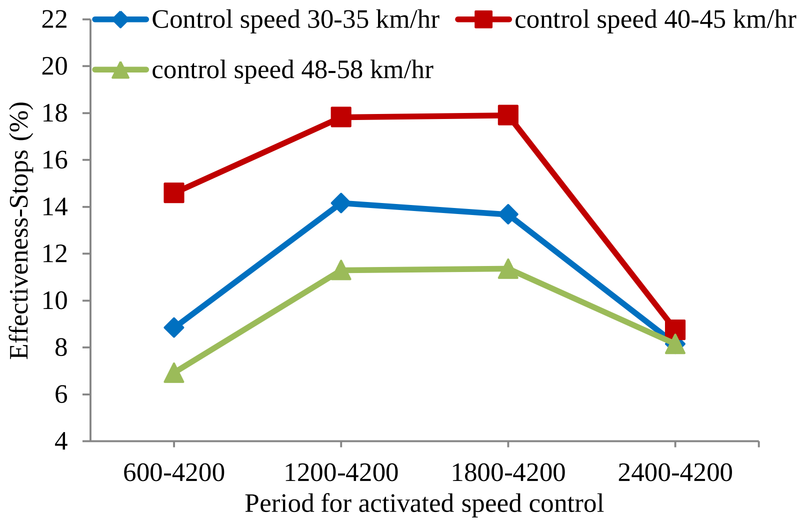


Figure 6.10 The efficiency of scenario 1 on the number of stops

The annual savings in rear-end collisions was calculated by taking the percentage of crashes at traffic signals in QLD multiplied by the total annual cost of crashes in Australia. If the average rear-end collisions cost represents 35% of the cost of collision at traffic signals in QLD. The result is then multiplied by the percentage of improvement in number of stops (safety efficiency) to determine the total annual cost savings due to rear-end crashes in QLD.
Generally, all VSL ranges enhanced traffic safety by reducing the average number of stops up to a maximum of 18% during the evening peak hour. The estimated annual cost of rear-end collisions is $1.011 billion for the base conditions (without control).

Applying the speed control overall QLD may reduce the rear-end collisions cost. The 18% would amount to $182 million per year in savings in Queensland due to the use VSL, of 40-45 km/hr. Speed control limits of 30-35 km/hr and 48-58 km/hr are projected to return a savings of $141 million and $111 million based on a reduction of 14% and 11% respectively in rear-end collisions. These values are shown in Figure 6.11. It is recognised that the estimated savings are based at a state level and involving extrapolation. The data is provided only as an example of the potential savings possible through the application of VSL.

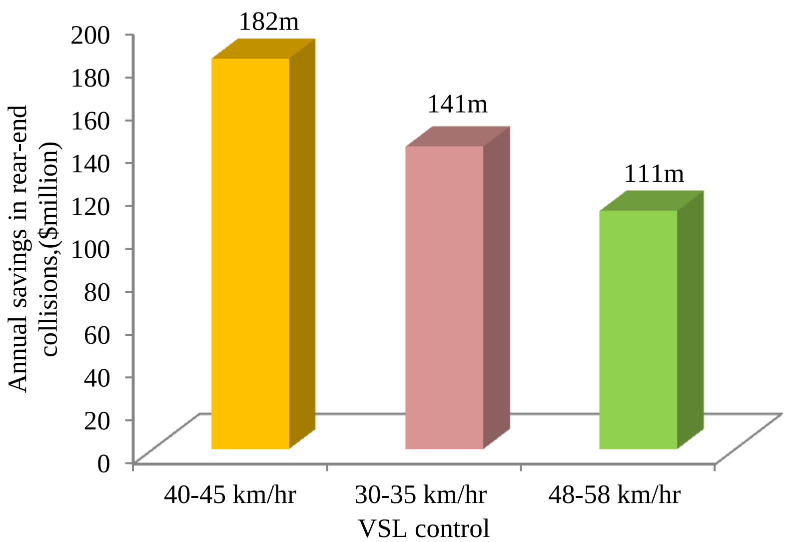


Figure 6.11 Annual savings due to reduced rear-end collisions by Scenario 1, VSL

6.4 Evaluation of Scenario 1 on the performance of the congested link

In this section, the traffic characteristics such as average density, average speed and average traffic flow for the congested link (WB link of intersection 2) were compared with and without the control strategy of Scenario 1. The parameters investigated are outlined in this section.

6.4.1 Evaluation of average traffic flow

Figure 6.12 shows that the flow rate for the base situation was increased by about 7% when VSLs were implemented. The throughput of the link was maximised from 1,332 veh/hr to 1,432 veh/hr for all proposed VSL ranges. There was little variation in the link throughput for all conditions of Scenario 1. This can be attributed to the limited capacity of the intersection that defined the discharge rate of the link.

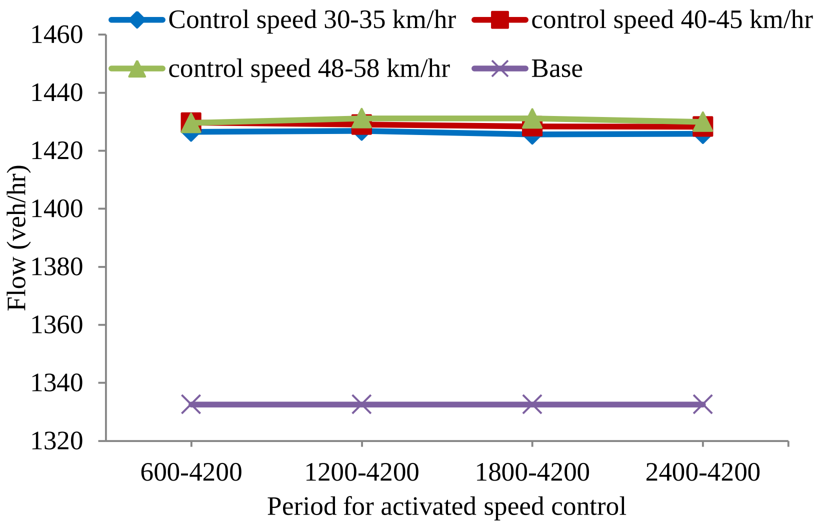


Figure 6.12 Link throughput before and after VSL application

The efficiency of Scenario 1 was 7% for VSL range 30-35 km/hr and 7.4% for both VSL ranges 40-45 and 48-58 km/hr (Figure 6.13). Slowing the upstream traffic using VSLs succeeded in improving flow in the congested link. Since the speed control was applied during the peak period and the road capacity was almost full, the variation in efficiency due to VSL was negligible. Activating speed control (CS1) for different time periods had a small effect on VSL efficiency. Improving the flow by 7% increased the vehicle accessibility by 24,000 vehicles per a year. Figure 6.13 shows the variation in flow efficiency for Scenario 1.

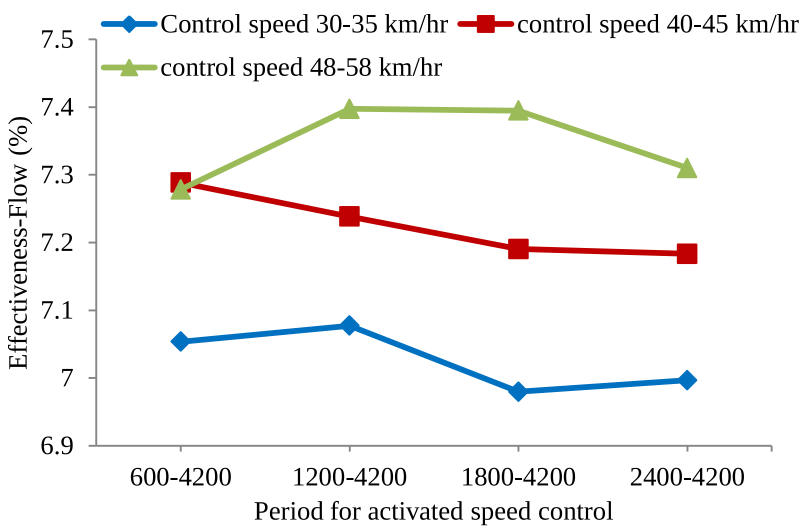


Figure 6.13 The efficiency of Scenario 1 on the throughput of the congested link

6.4.2 Evaluation of average speed

The variations in average travel speed for the WB direction of intersection 2 under the base traffic condition (no control) and VSLs application (control) are given in Figure 6.14. The speeds of the downstream link increased when VSLs were employed for upstream traffic control. Figure 6.14 shows that the travel speed rose from 39 km/hr to around 42 km/hr when VSLs were applied. The results can be attributed to the VSL creating relatively uniform flow over the whole of the activation period, resulting in a nearly constant vehicle density and speed distribution along the congested link.

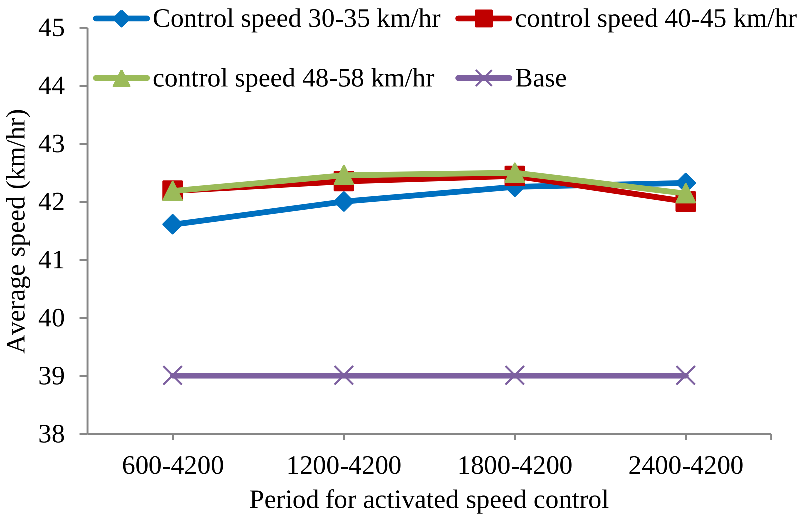


Figure 6.14 Average speed before and after VSL application

The effect of VSL on the average speed efficiency is indicated in Figure 6.15. The effect on efficiency for controlled speed ranges of 40-45 km/hr and 48-58 km/hr were between 7.5% and 9%. The efficiency for control speed 30-35 km/hr was between 6.5% and 8.5%. The low variation in speeds for the controlled link justified the low throughput for the previous section where lower speeds produced lower throughput compared to higher speeds.

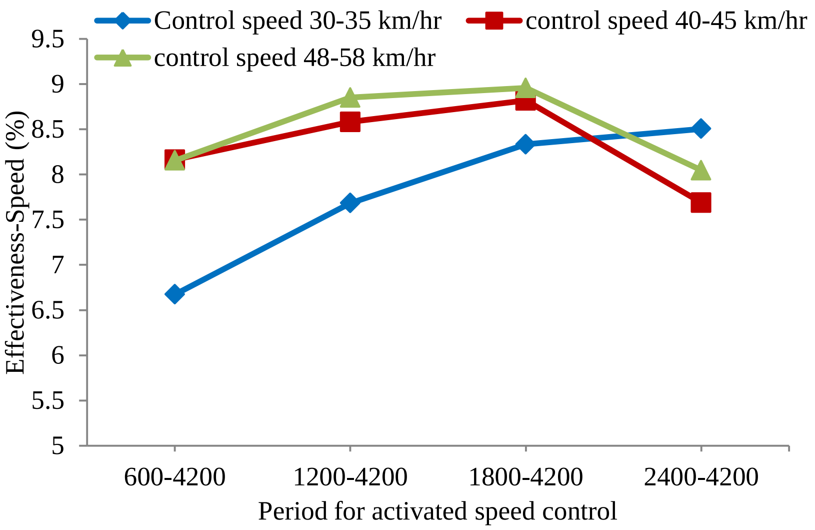


Figure 6.15 The efficiency of Scenario 1 on average speeds

Improving the speed along the congested link contributed to increased speed homogeneity and reduced speed deviation from the posted speed. The speed deviation is the difference between the posted speed and average space speed in km/hr. Reducing speed fluctuation along the congested link potenially reduce fatal collisions. O'Day and Flora (1982) and (Joksch, 1993) found that the risk of a vehicle driver being killed in a collision increased with increase in speed variation. Table 6.7 shows that the probability of fatality at GAR reduced from 3.6% (without control) to 2.25% after employing VSL. This was because VSL reduced speed devations along the congested link from 31 to 27.5 km/hr. Improving the speed along the congested link by 3.5 km/hr reduced the probability of fatality by 1.35%.

Table 6.7 Probability of fatality at GAR before and after utilising VSL

Status	Average space speed (km/hr)	Posted speed (km/hr)	Speed deviation (km/hr)	Probability of Fatality* (%)	Fatal improvement (%)
Base condition	39	70	31	3.6	1.35
VSL	42.5	70	27.5	2.25	

Probability of fatality= (Speed deviation/71)^4. Source O'Day and Flora (1982)

6.4.2.1 Statistical significant of speed results

In order to evaluate the influence of of VSL on space speed parameter for the WB link at intersection 2, the link was divided into two parts. Part 1 had low speed, short distance (200m) and adjacent to the intersection area. Part 2 had relatively high speed, and was a long distance (710m) away from the intersection area.

The method is based on the comparison of space speed models using data before and after the implementation of the VSL for both parts. The number of samples was 30 and considered for every 600 seconds for before and after VSL. One-Way ANOVA at 95% level of confidence were considered to estimate the significant of employing VSL. Table 6.8 and Table 6.10 show the variance in speed results for part 1 and part 2 under the base condition and VSL. Table 6.9 and Table 6.11 show that VSL has a statistically very significant impact on vehicle speeds. The significant value of ANOVA was much more less than the confidence interval (5%) for both parts, that is (.000 < 0.05).

Table 6.8 Speed statistical descriptive for part 1

Status	Number of reading	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base condition	30	12.04	1.83	.33	11.36	12.73	8.90	16.35
VSL	30	14.02	2.19	.40	13.20	14.84	10.32	17.58
Total	60	13.03	2.23	.28	12.45	13.61	8.90	17.58

Table 6.9 Speed, One-Way ANOVA for part 1

Status	Sum of Squares	df	Mean Square	F-Statistic	Sig.
Between Groups	58.541	1	58.54	14.31	.000
Within Groups	237.21	58	4.09		
Total	295.75	59			

Table 6.10 Speed statistical descriptive for part 2

Status	Number of reading	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base condition	30	65.96	.89	.16	65.63	66.30	64.32	68.22
VSL	30	70.87	.84	.15	70.55	71.18	69.15	73.53
Total	60	68.41	2.61	.33	67.74	69.09	64.32	73.53

Table 6.11 Speed, One-Way ANOVA for part 2

Status	Sum of Squares	df	Mean Square	F-Statistic	Sig.
Between Groups	360.92	1	360.92	479.66	.000
Within Groups	43.64	58	.75		
Total	404.56	59			

6.4.2.2 Evaluation of savings in crash costs on the GAR

The available crash data for the GAR was obtained from the Department of Transport and Main Roads (DTMR) for period 2006-2010. Severity of crash data involved fatal, hospitalisation, medical treatment, minor injury and property damage only. The crash data were solely considered for the peak hours (6:00-9:00a.m) and (3:00-6:00p.m) in order to evaluate the impact of VSL during these periods. The average cost of crashes of different severities were estimated from Cairney (2013). The impact of 1.35% improvement on the estimated annual cost in severity of fatal crashes has been assumed to apply to all crash types for the GAR during the peak periods. Table 6.12 shows that if an improvement of 1.35% was achieved on the GAR due to the use of speed control, the total annual saving in crash costs would be around $82,000.

Table 6.12 Savings in severity of crash costs at GAR using VSL

Year	Fatal	Hospitalisation	Medical Treatment & Minor injury	Property damage
2006	0	11	31	16
2007	1	14	19	9
2008	1	13	25	19
2009	1	14	27	17
2010	0	7	30	13
Average	0.6	11.8	26.4	14.8
Improve 1.35%	0.0081	0.1593	0.3564	0.1998
Cost/crash ($AUD)	3,083,000	307,500	17,000	11,500
Savings/year ($AUD)	24,972	48,985	6,059	2,298
Total annual savings ($AUD)		82,314		

6.4.3 Evaluation of average density

As explained in previous sections, the enhancements in travel speed and average link throughput, obtained by applying the VSLs to the congested link, contributed to maintaining the traffic density, with possibly a slight improvement. The application of VSL maximises the speed and flow of the congested link without changing the traffic density. In other words, VSLs maximise the road capacity by maintaining traffic density nearly constant while the link flow rate is increased. Figure 6.16 shows there were no significant differences between the traffic density in the base condition and the traffic density under VSL application. The average traffic density was 34.2 veh/km before applying VSL and only changed slightly when the VSL system was activated, ranging between 33.6 and 34.3 veh/km.
The effect of VSL on traffic density is illustrated in Figure 6.17. The figure shows there were only slight improvements in reducing the base traffic density when using VSL. The maximum enhancement in the traffic density was 1.5% for the activated speed control period of 1800-4200s.

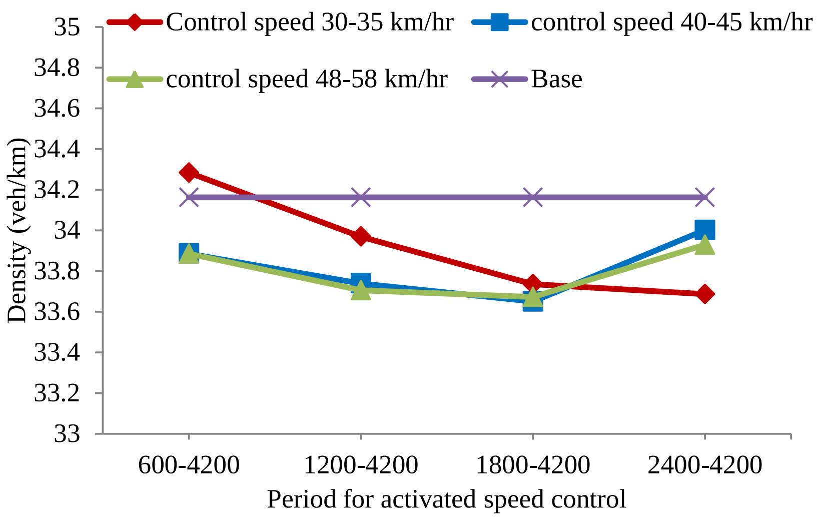


Figure 6.16 Traffic density before and after VSLs application

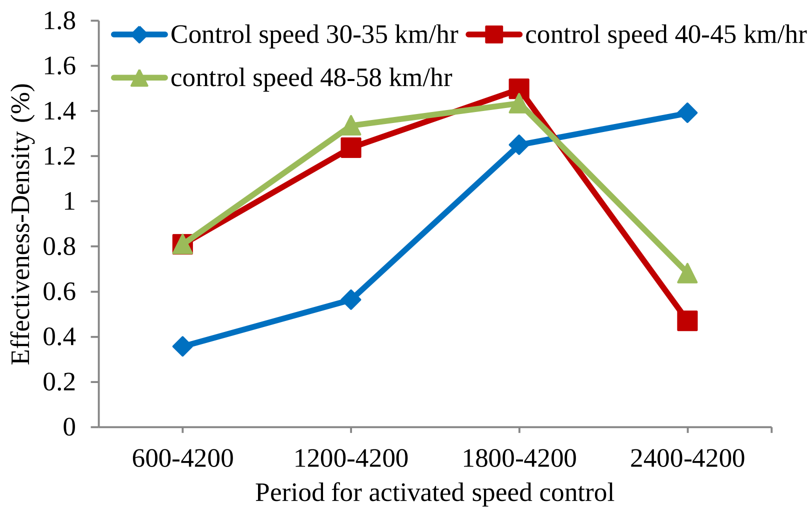


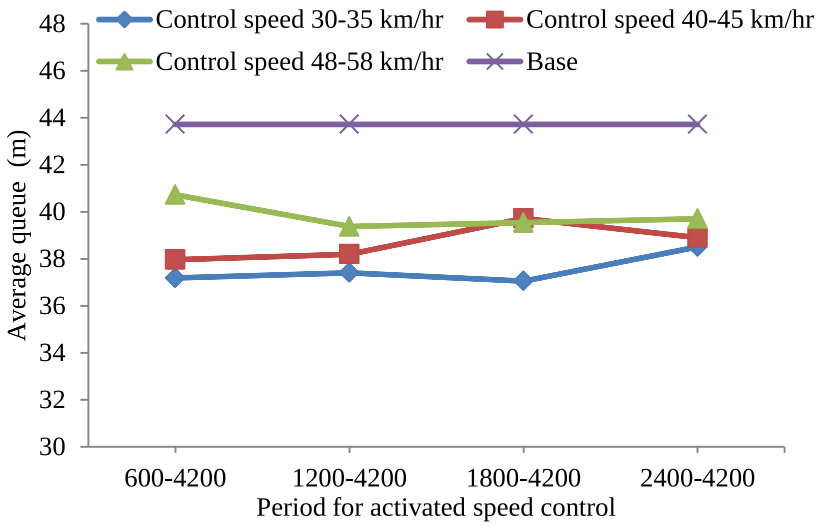
Figure 6.17 The efficiency of Scenario 1 on the traffic density

6.5 Evaluation of Scenario 2 on the performance of intersection 2

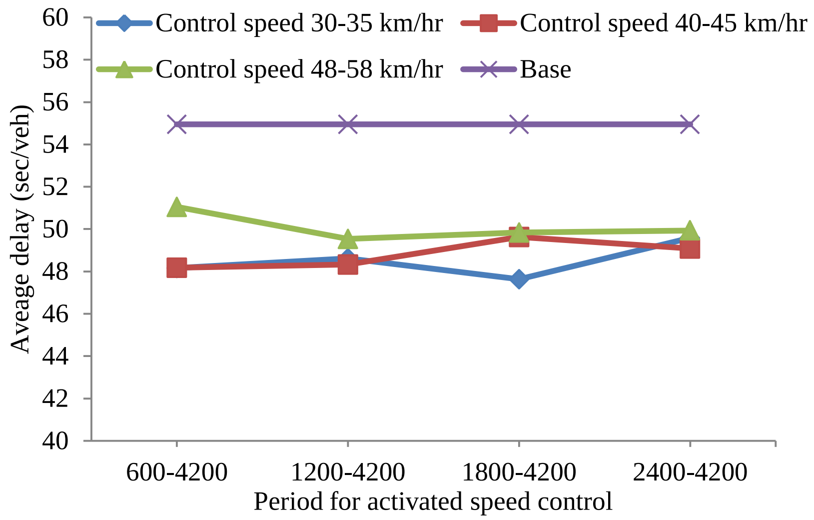
Scenario 2 involved activation of speed control (CS2) at different control speed ranges and different activation periods during peak traffic. CS2 was placed on the WB lane of the main road in approximately the middle of the network (4.2km). The aim of this scenario was to investigate the impact of the position of the speed control on the traffic characteristics of a congested intersection. Signalised intersection indicators and traffic flow indicators are discussed to explain the effect of VSL application on the performance of interrupted flow.

6.5.1 The influence of Scenario 2 on the intersection parameters

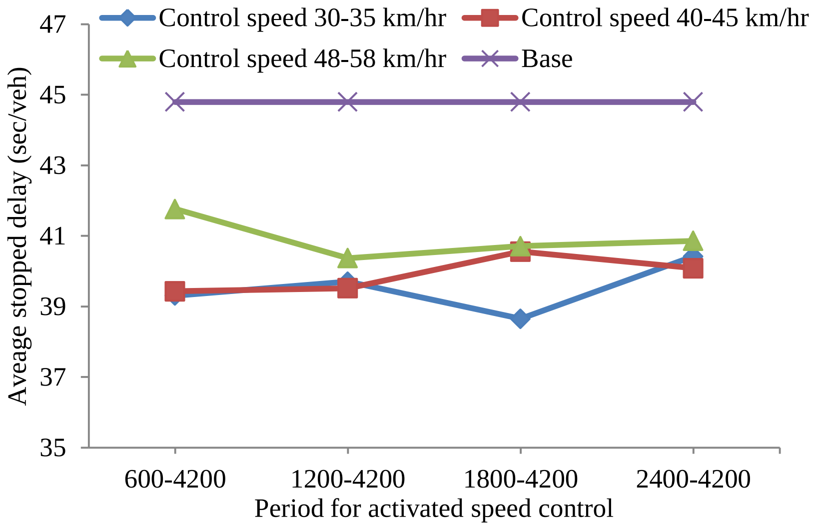
The effect of CS2 on the characteristics of intersection 2 were measured by the average queue length, average delay, average stopped delay and average number of stops as shown in Figure 6.18 (a), (b), (c), and (d). This figure shows the differences between the performance of intersection 2 without speed control (base) and with speed control, at various operating speeds and activation times. Activating VSL during congested periods resulted in a significant improvement in the characteristics of intersection 2 as seen in Figure 6.18. The base condition had the lowest traffic performance (LOS E) when compared with VSL applications (LOS D) under all proposed scenarios. Control speed ranges of 30-35 and 40-45 km/hr resulted in the largest overall improvements compared to the base condition. CS2 had the most effect when activated for 1800s-4200s at the control speed of 30-35 km/hr.



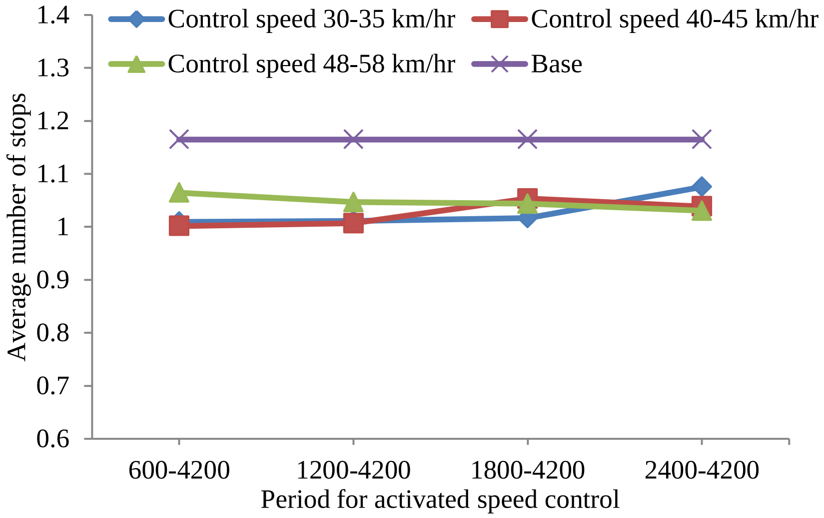
(a)



(b)



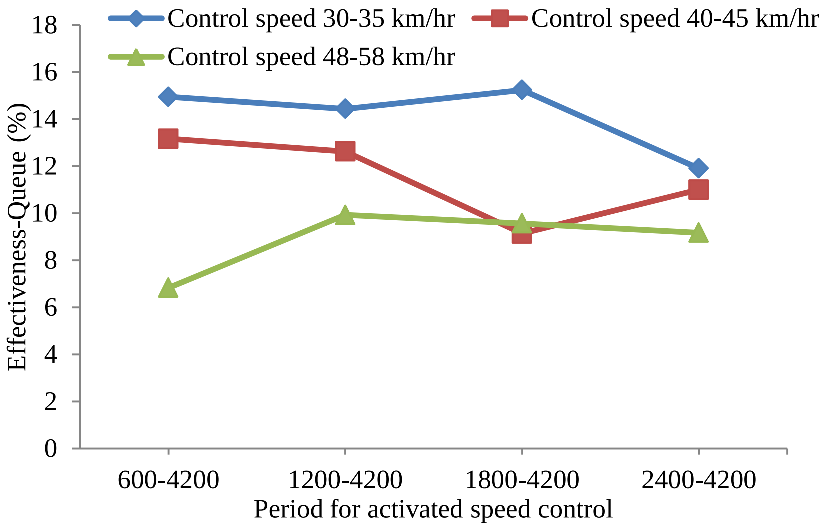
(c)



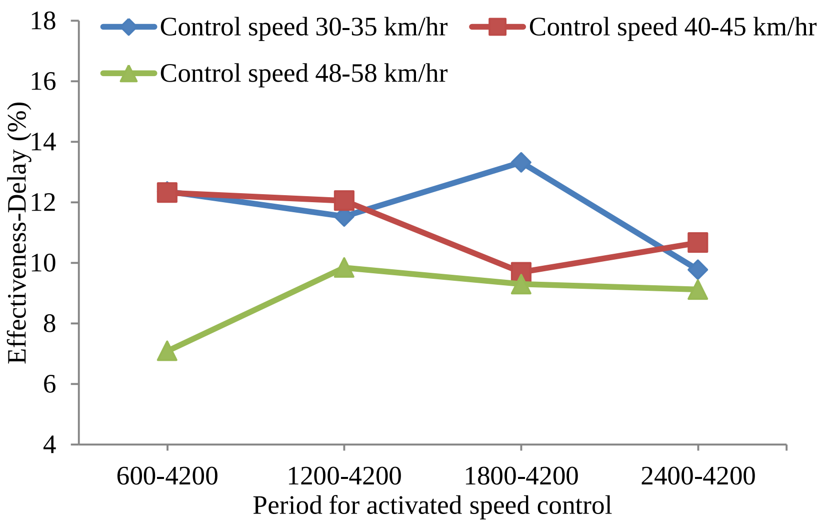
(d)

Figure 6.18 The simulation of Scenario 2 along the WB direction of the main road

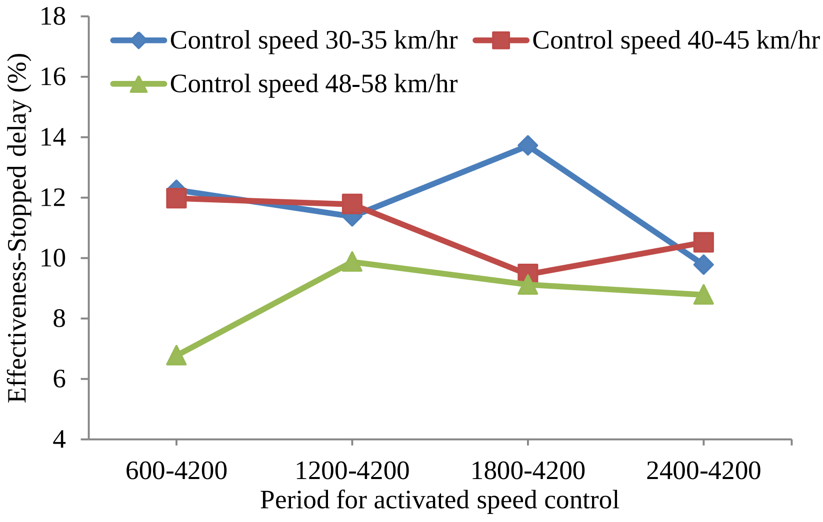
The efficiency of applying scenario 2 on the intersection indicators is shown in Figure 6.19 (a), (b), (c), (d). The figure shows a 7% to 15% improvement in average queue length, 7% to 13% improvement in average delay, 7% to 14% improvement in average stopped delay and 8% to 14% improvement in the average number of stops.



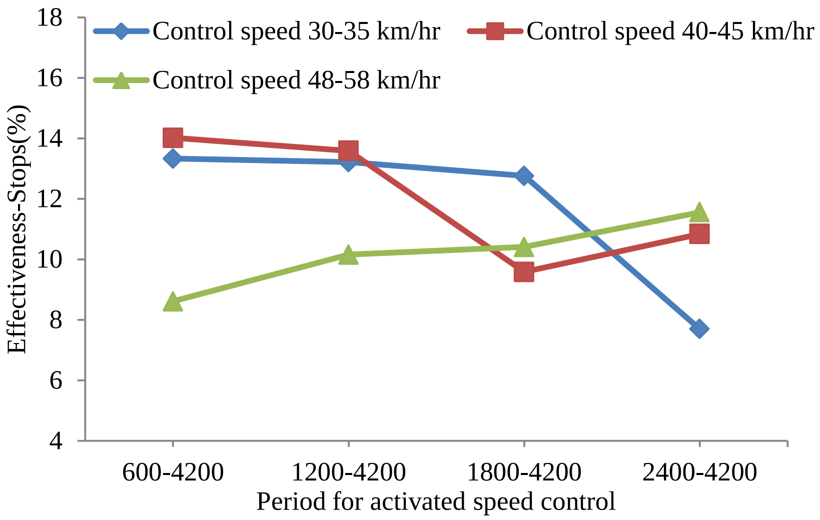
(a)



(b)



(c)

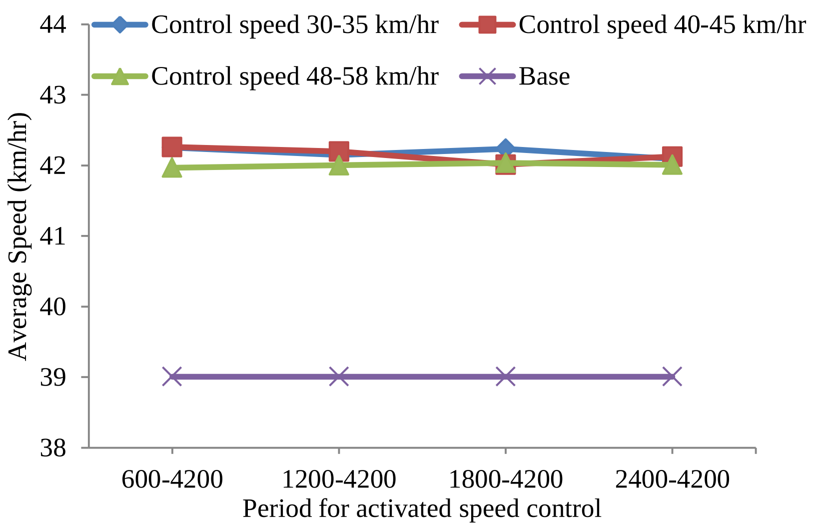


(d)

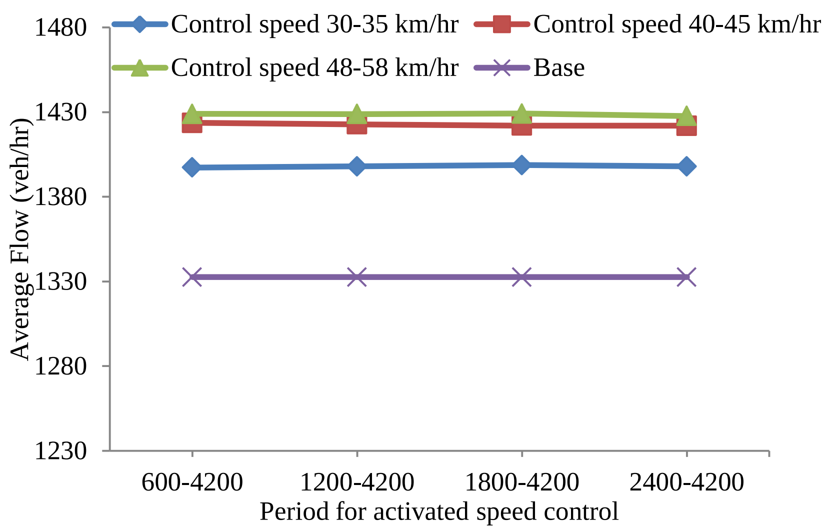
Figure 6.19 The efficiency of scenario 2 on the signalised intersection characteristics

6.5.2 Evaluation of Scenario 2 on the performance of the congested link

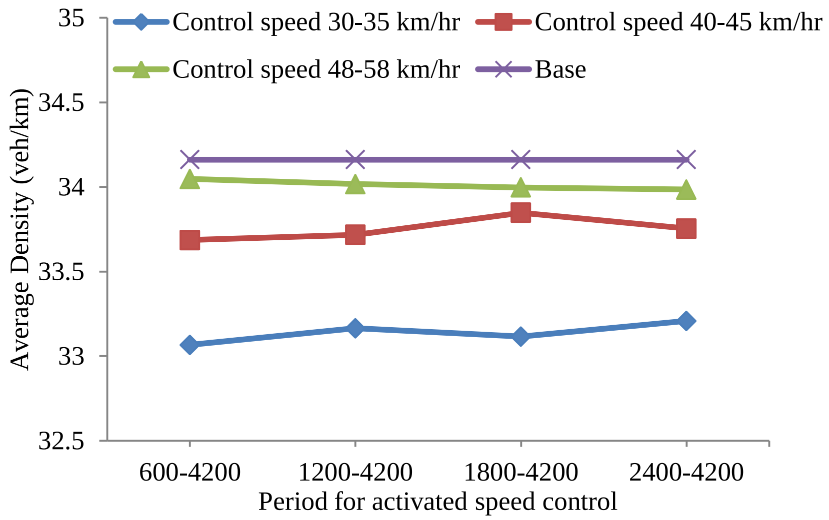
The second part in evaluating Scenario 2 was to examine the macroscopic characteristics of traffic flow in terms of average travel speed, average flow and average traffic density. Results were collected before and after activating VSLs to evaluate any impact and the outcomes are shown in Figure 6.20 (a), (b), (c). Scenario 2 resulted in large improvements in both the average speed and average flow compared with base line conditions. Traffic density was sustained in spite of the average speed and flow rate increasing after applying Scenario 2. The traffic density was not reduced significantly compared with base line condition.



(a)



(b)



(c)

Figure 6.20 The effect of Scenario 2 on the characteristics of congested link

Improvements in efficiency due to Scenario 2 are shown in Figure 6.21 (a), (b), (c). There was a 7% to 8% increase in the average speed, a 5% to 7% increase in flow productivity and a less than 3% enhancement in the traffic density. The results confirm that VSL maintains and increases the average speed and average flow rate of the congested link and also maintains and enhances the traffic density during the period of congestion.

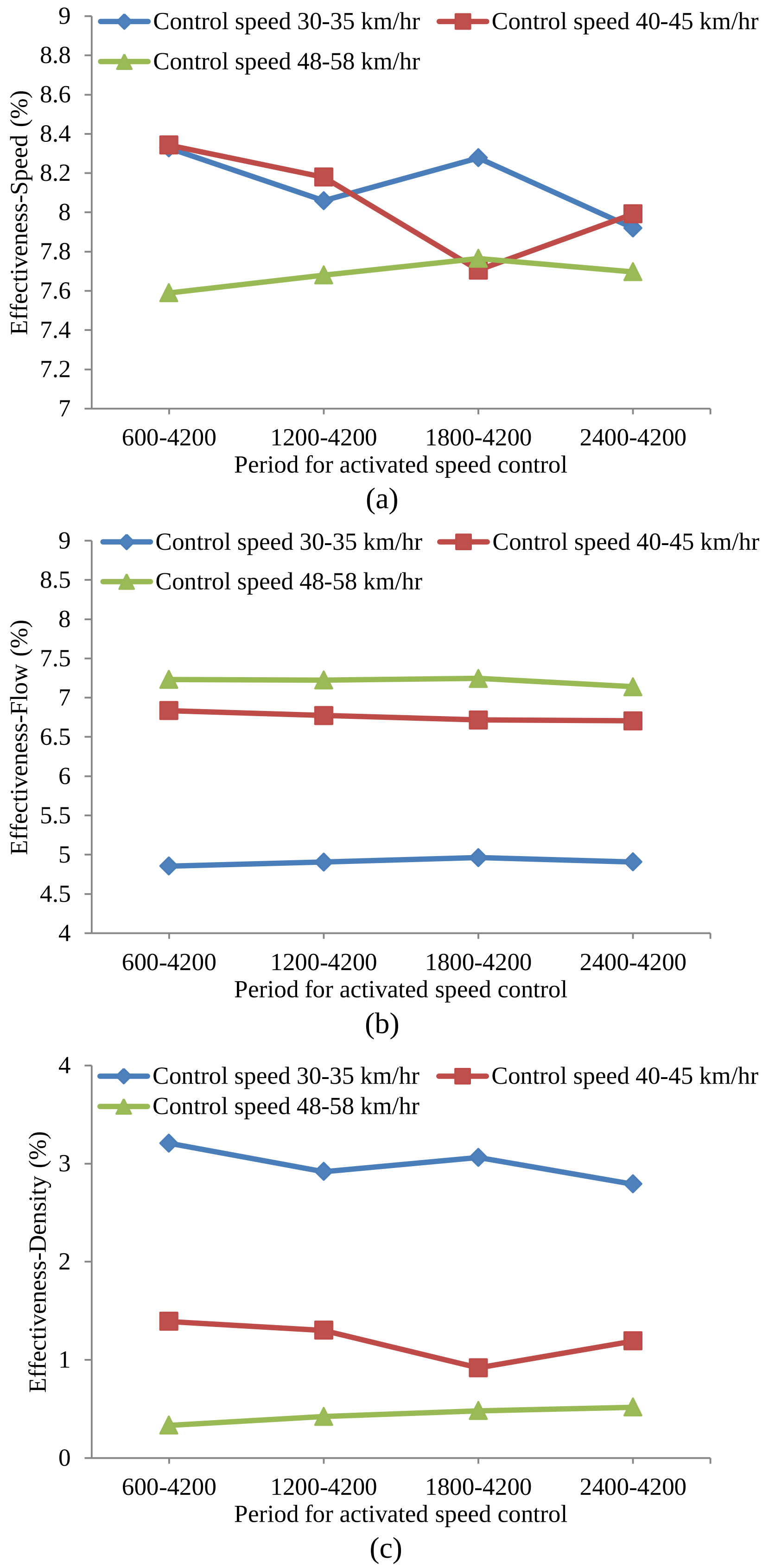


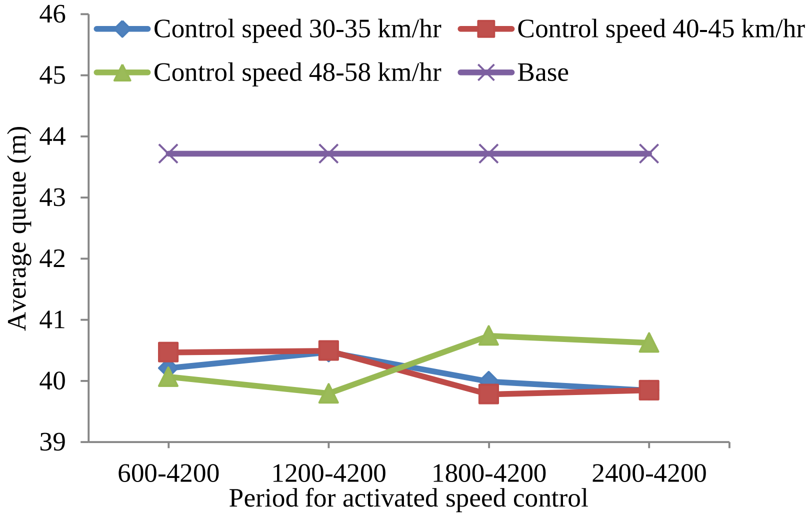
Figure 6.21 The efficiency of Scenario 2 in terms of traffic macroscopic aspect

6.6 Evaluation of Scenario 3 on the performance of intersection 2

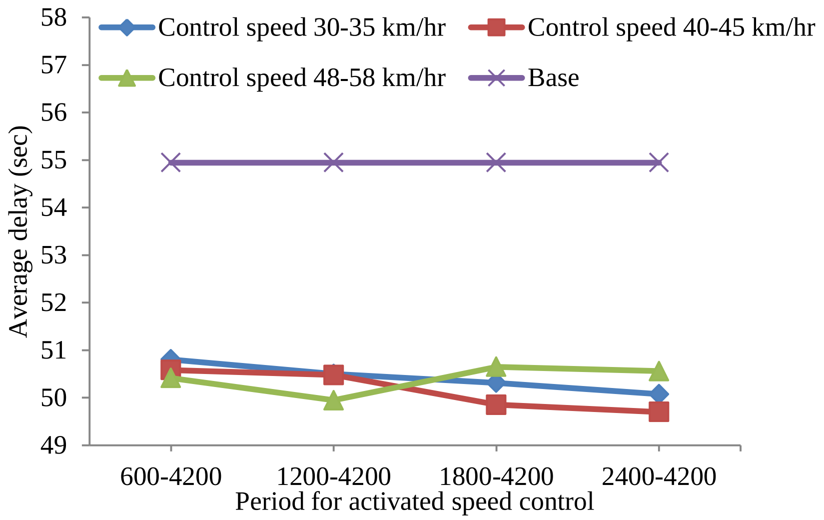
In the previous section, the influence of VSL position was investigated when the speed control is positioned relatively far from the congested area. In Scenario 3, the position of CS3 (at the WB entrance of the network (5.7km)) is placed further away from intersection 2 than the position of CS2. The reason for doing this was to study the effect of VSL when it was introduced far from the congested link. The traffic characteristics were analysed before and after the application of Scenario 3. The same previous traffic conditions were implemented.

6.6.1 The influence of Scenario 3 on the intersection parameters

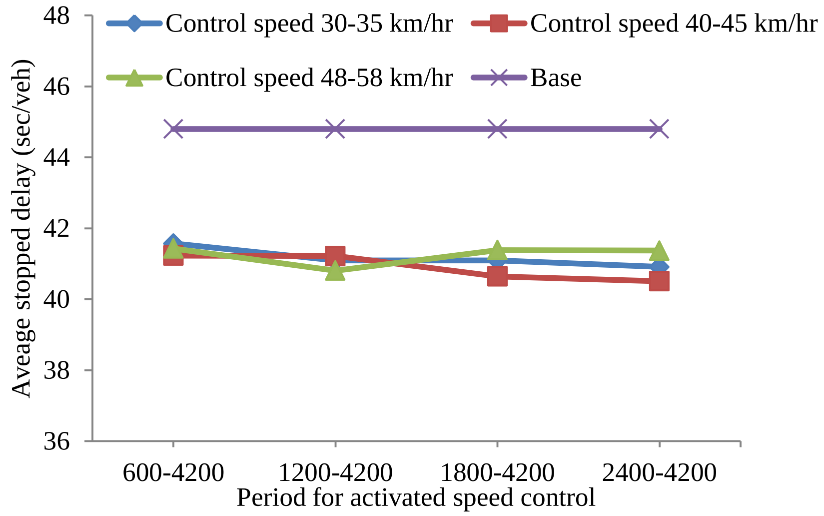
Figure 6.22 (a), (b), (c), (d) shows the influence of Scenario 3 on the signalised intersection parameters. These results show that applying VSL has a significant impact on traffic at intersection 2 even though it was implemented further from the congested intersection. However, the effect was less than for other scenarios. Average queue length, average delay, average stopped delay and average number of stops improved. For all cases, the base condition had the worst values. The variation in improvement between the three VSL control speeds was very small. This can be attributed to the proposed speed controls of 40-45 and 48-58 km/hr not being matched to the actual speed of the links.



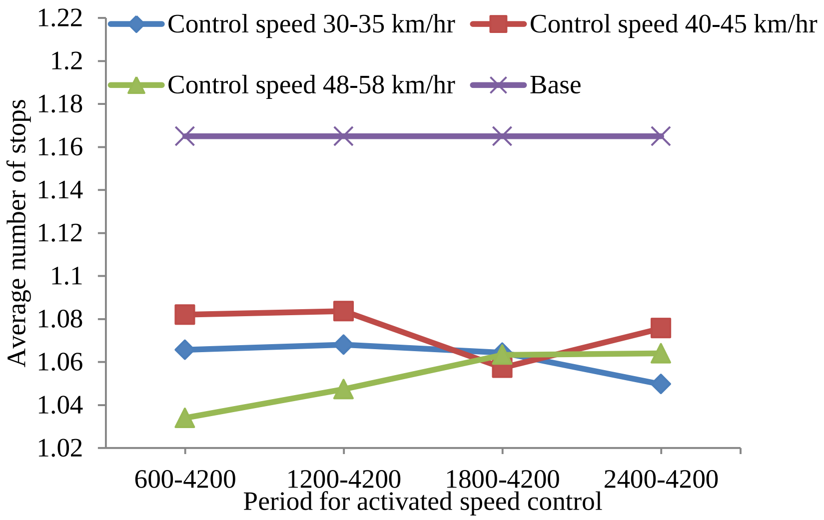
(a)



(b)



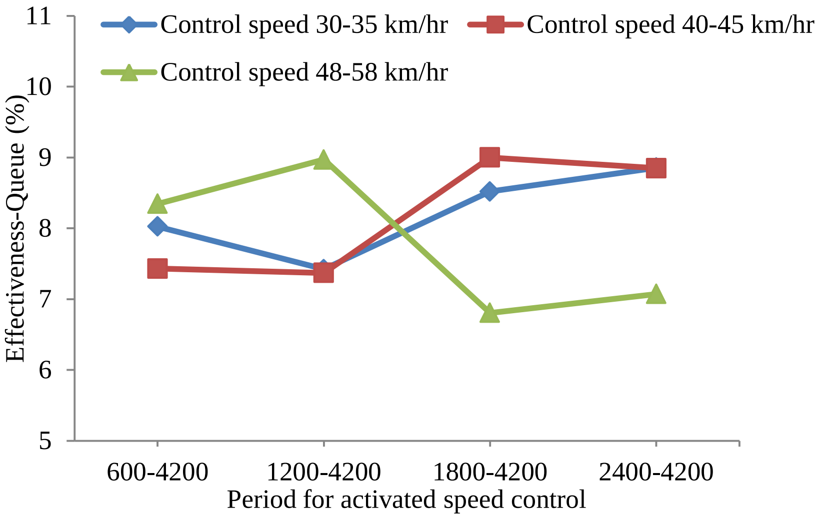
(c)



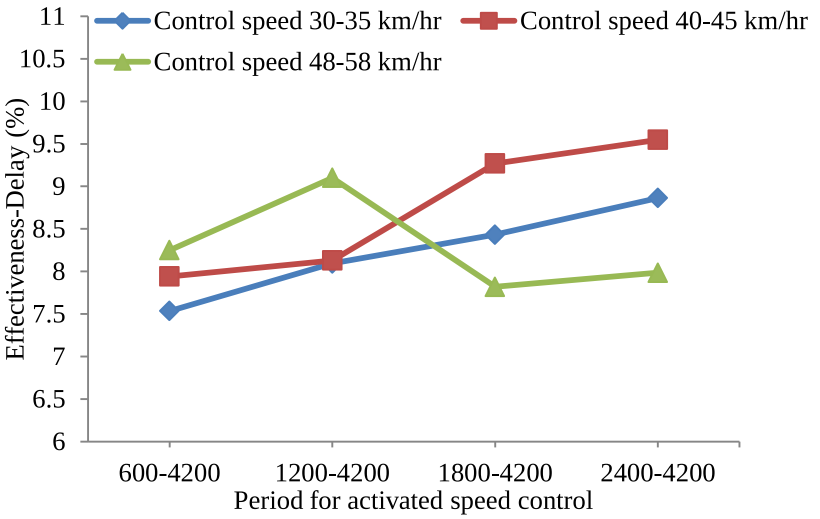
(d)

Figure 6.22 The parameters of signalised intersection 2 before and after Scenario 3

The efficiency of Scenario 3 was determined from the above figures and plotted in Figure 6.23 (a), (b), (c), (d). This shows that the efficiency of VSLs was less than in previous scenarios. This is evidence that the efficiency of VSL applications reduce the further they are located away from the congestion source. It produced a maximum efficiency (9%) for all parameters when it was activated for 1800s-4200s and at a speed range of 40-45 km/hr.



(a)



(b)

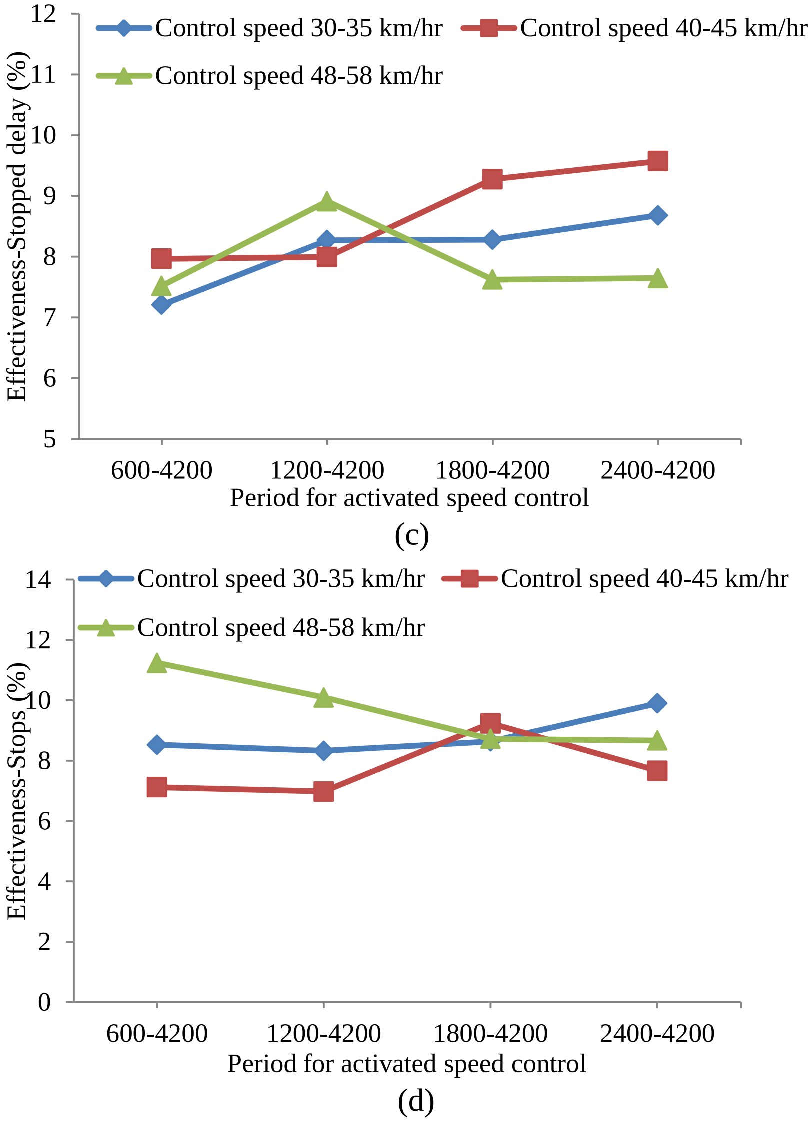


Figure 6.23 The efficiency of Scenario 3 on the signalised parameters

6.6.2 Evaluation of Scenario 3 on the performance of the congested link

Speed, flow and density for WB lane of intersection 2 under Scenario 3 were investigated and the results are shown in Figure 6.24 (a), (b), (c). They show that Scenario 3 produced significant improvements for speed and flow at all activation periods. There were no large differences in traffic density before and after application of Scenario 3. The results were nearly the same for all speed controls for the reasons mentioned previously.

The efficiency of Scenario 3 was measured and plotted in Figure 6.25 (a), (b), (c). Highest improvements were achieved in speed and flow which were around 6% to 8% and the lowest improvement was in traffic density which was less than 1%. The efficiency of VSL on the traffic density was considered a positive result because it maintained the traffic capacity and prevented the traffic congestion from becoming worse with the increasing flow rate.

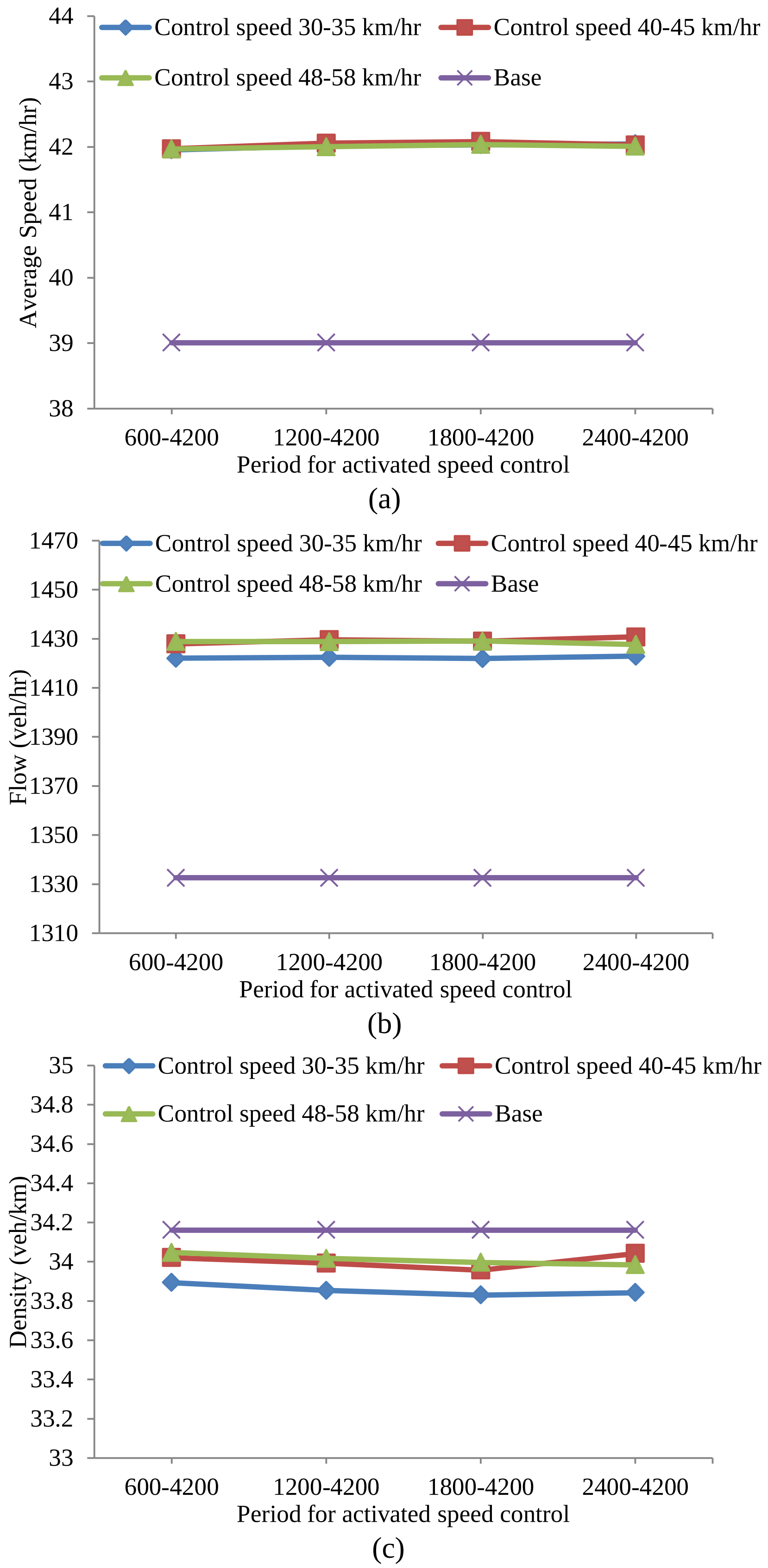
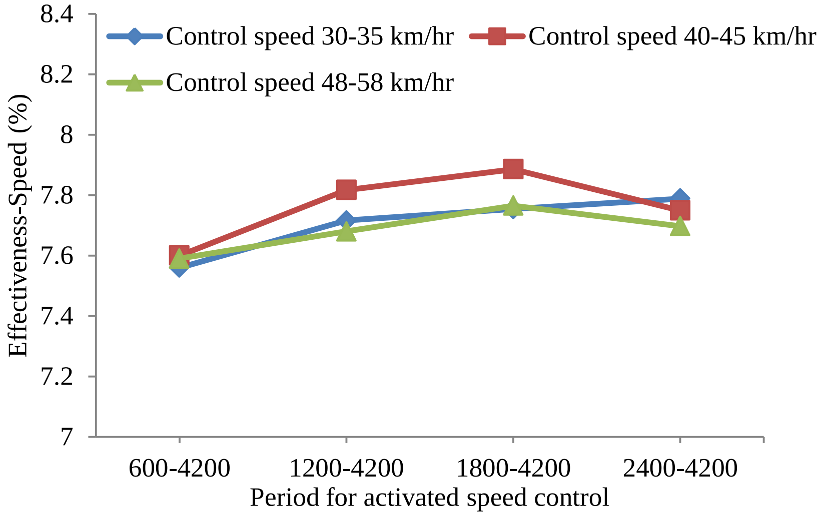
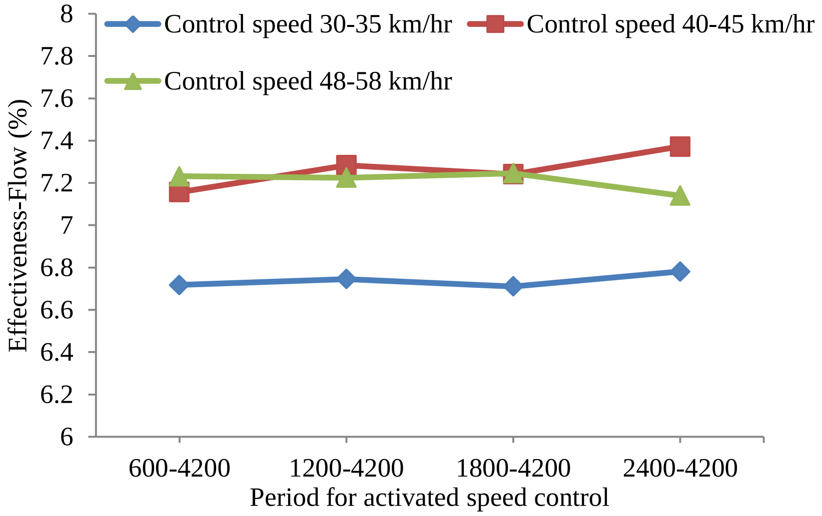


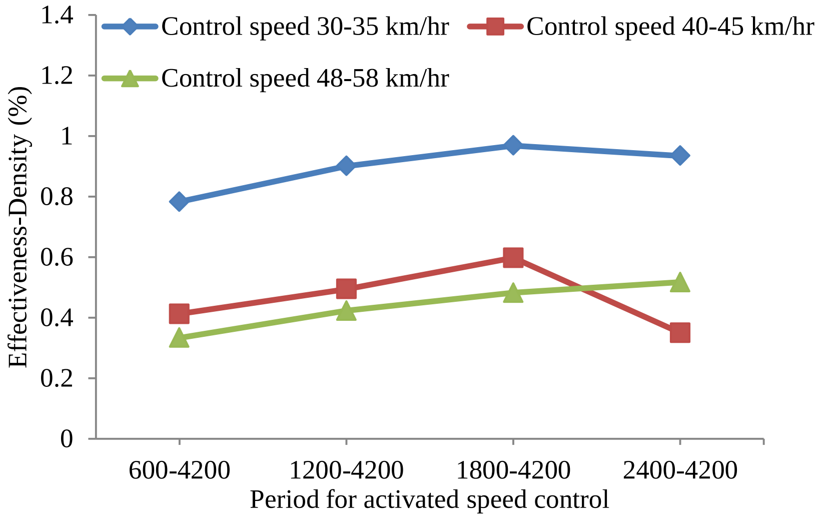
Figure 6.24 The macroscopic characteristics before/after the application of Scenario 3



(a)



(b)



(c)

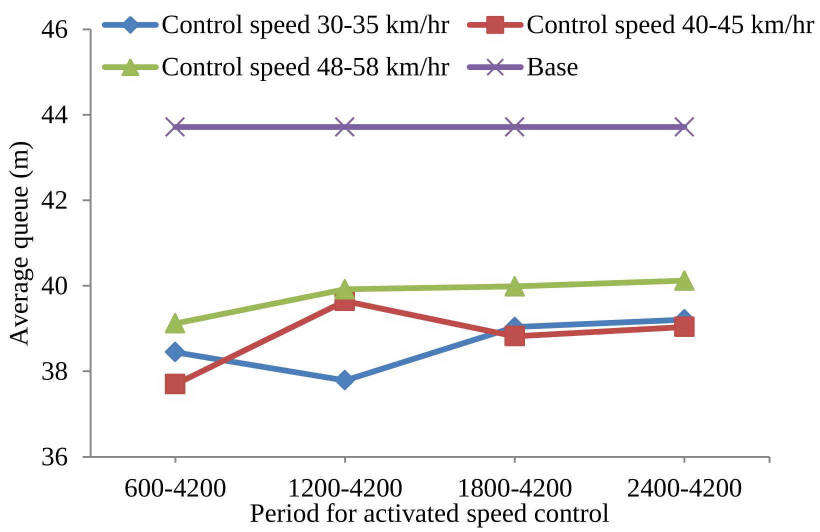
Figure 6.25 The efficiency of Scenario 3 on the characteristics of congested link

6.7 Evaluation of Scenario 4 on the performance of intersection 2

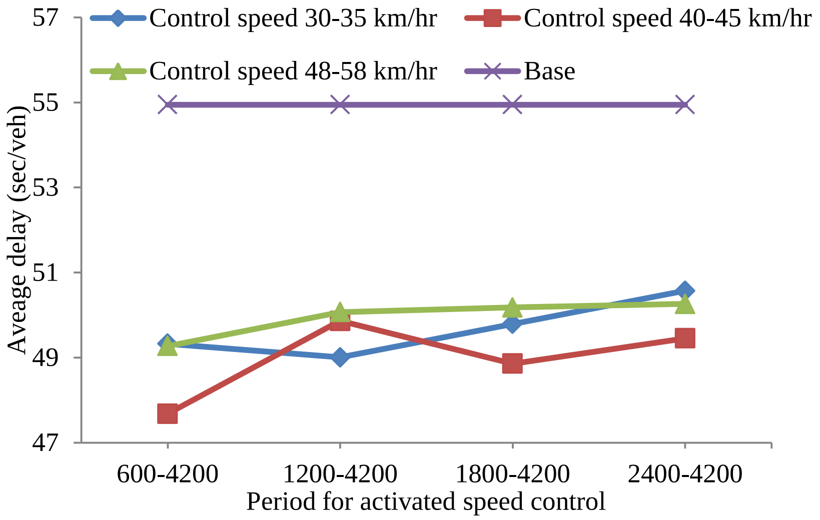
As explained in section 6.3, Scenario 4 involved activating SC2 and SC3 simultaneously. The aim of this scenario was to increase the speed control distance and to study its influence on traffic characteristics. The operation of the control system followed the same methods used in previous scenarios. This scenario influences both the signalised intersection parameters and the arterial road characteristics.

6.7.1 The influence of Scenario 4 on the intersection parameters

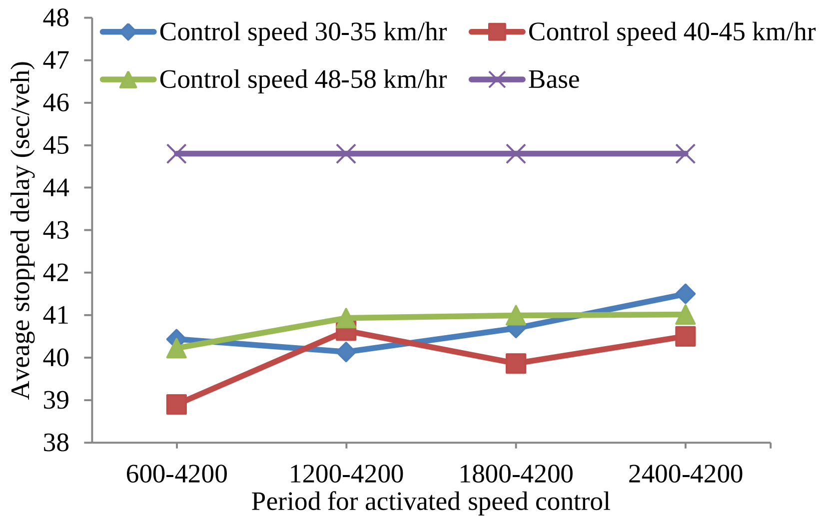
The effect of increasing the speed control distance is shown in Figure 6.26 (a). (b), (c),(d). The outcome was a positive effect on the performance of intersection 2 because all the proposed speed controls at different levels of activation enhanced the operation of intersection when compared with base line conditions. The figures also show that a speed control range of 40-45 km/hr at an activation time starting from 600 seconds produced the best results. This can be attributed to synchronisation between the proposed speed control and the operating system of traffic signals along the main road. Increasing or decreasing the speed control ranges from 40-45 km/hr generally produced lower traffic improvements. Raising the speed limits increased the number of vehicles flowing towards intersection 2, resulting in increased interaction between vehicles and increasing the queue length, number of stops, and delay. Decreasing the speed increased vehicle platooning and thus increased interaction between vehicles and reduced the synchronisation efficiency. This also reduced the enhancements in terms of speed, flow and density.



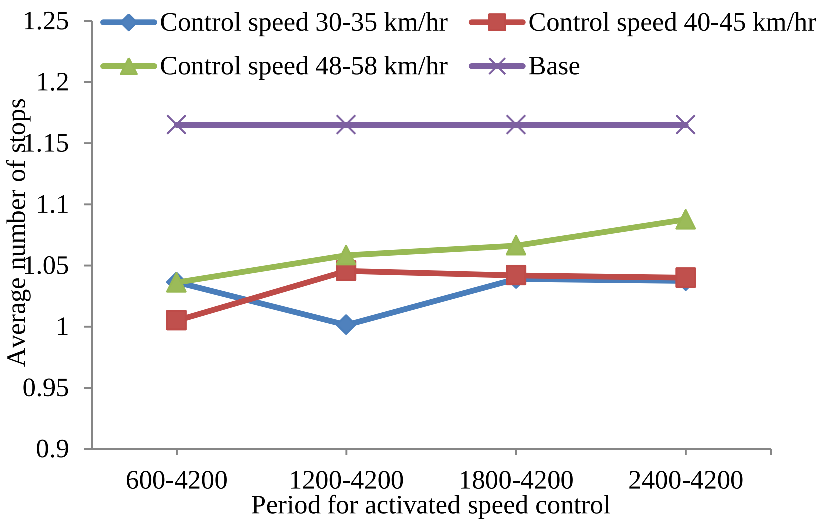
(a)



(b)



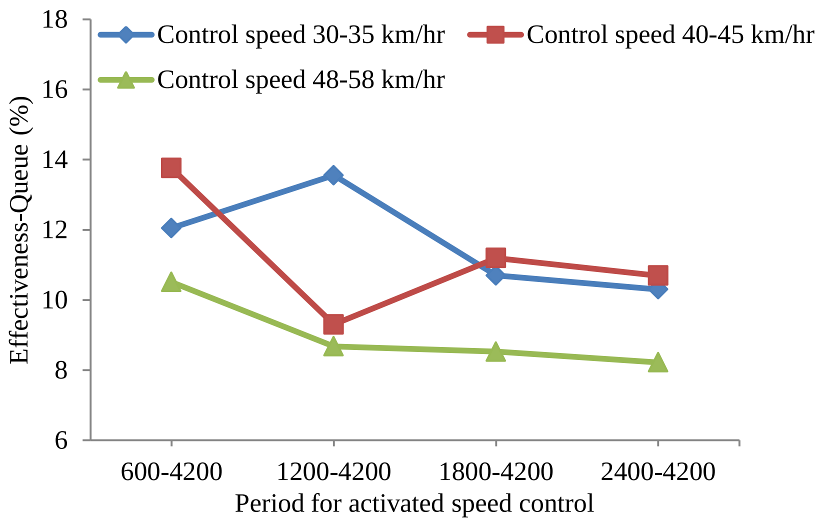
(c)



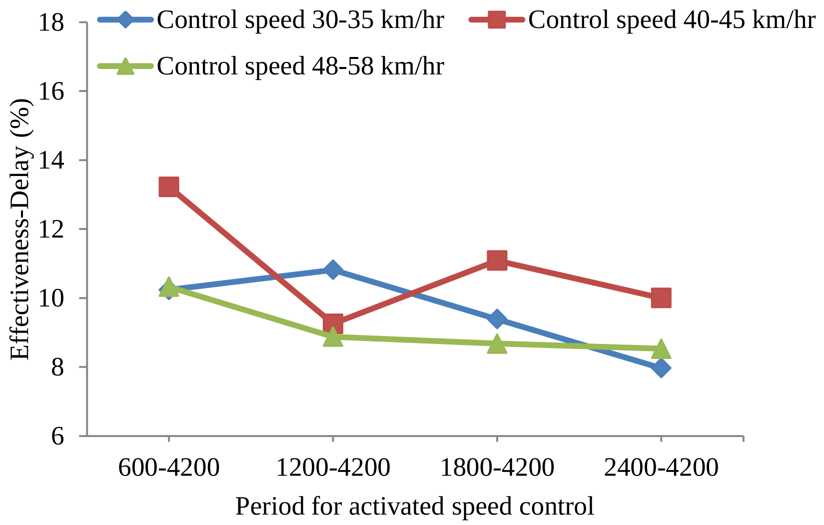
(d)

Figure 6.26 Signalised intersection parameters under application of Scenario 4

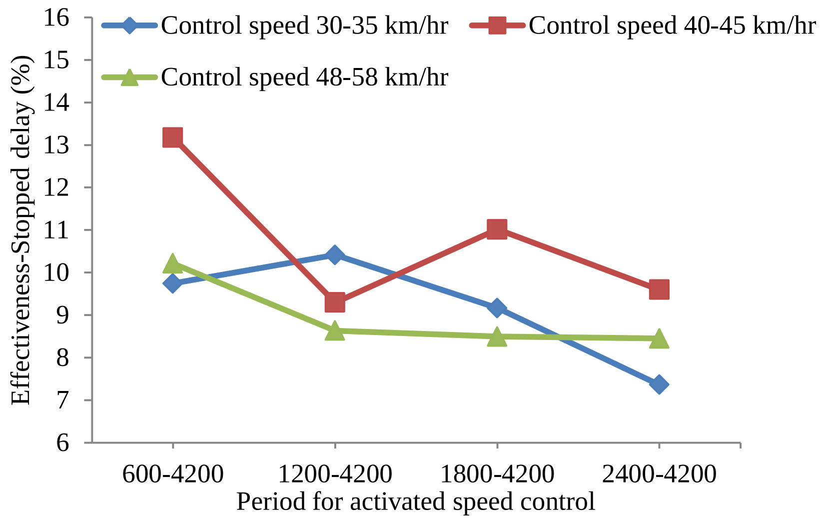
The efficiency of Scenario 4 is shown in Figure 6.27 (a), (b), (c), (d). Activating VSL at period 600s-4200s and at a speed limit of 40-45 km/hr produced the best overall improvement of all intersection parameters. The maximum efficiency of all intersection parameters due to VSL was approximately 14% and the lowest improvement was 8%. The results indicate that using VSL during the peak hour increases the performance of the intersection by reducing the average delay and increases the traffic safety by reducing the queue length and the number of stops.



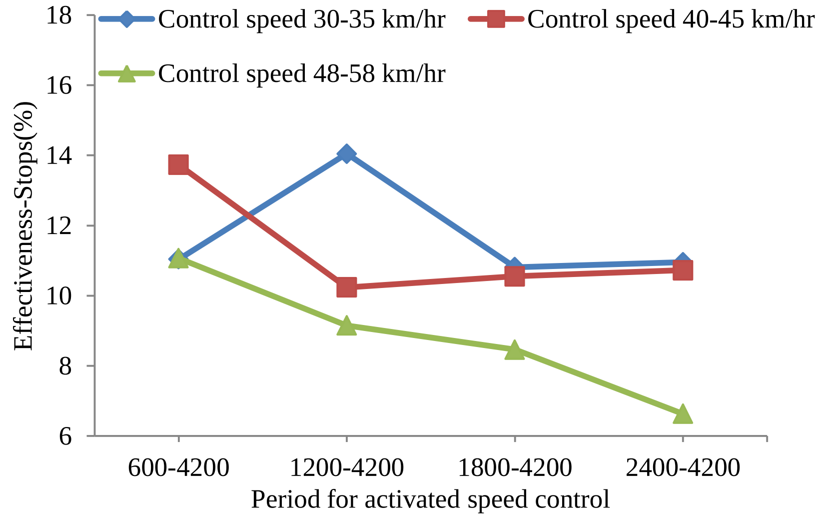
(a)



(b)



(c)

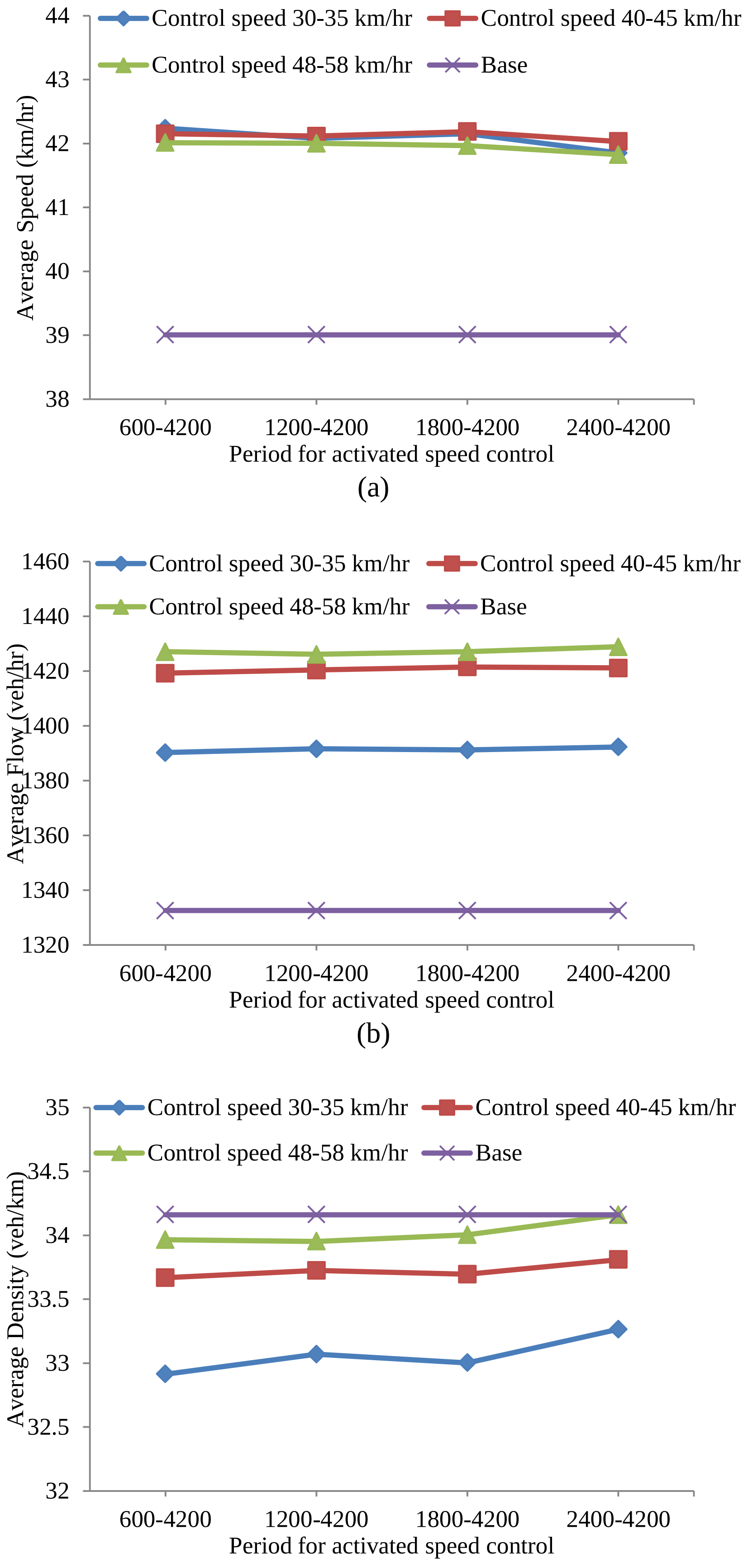


(d)

Figure 6.27 the efficiency of Scenario 4 on the performance of intersection 2

6.7.2 Evaluation of Scenario 4 on the performance of the congested link

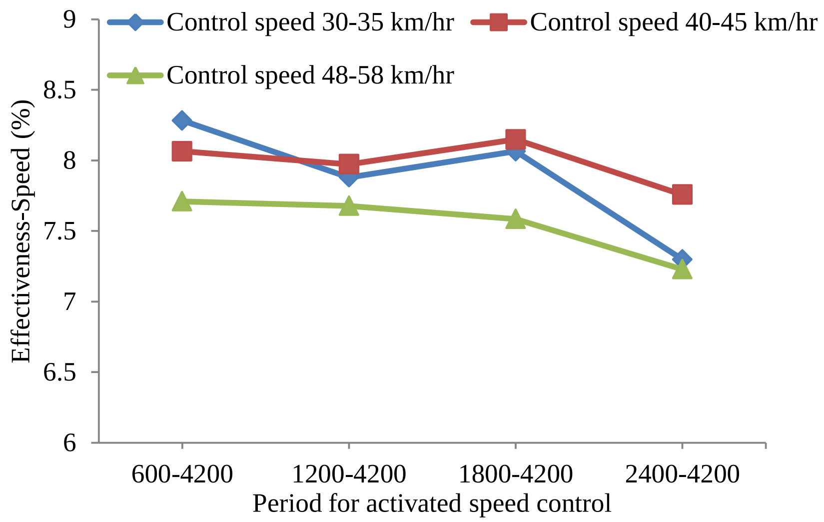
Scenario 4 had a positive effect by enhancing traffic performance of the congested link as shown in Figure 6.28 (a), (b), (c). This figure shows that improvement in speed was approximately constant throughout the application of the scenario. This was due to the link having an increase in flow productivity and uniform traffic density for all simulated cases. It can be noted that the high-speed control ranges yielded a somewhat higher flow rate than the lower speed control ranges. This behaviour may assist increasing traffic density for the high-speed control ranges rather than the lower speed control ranges.



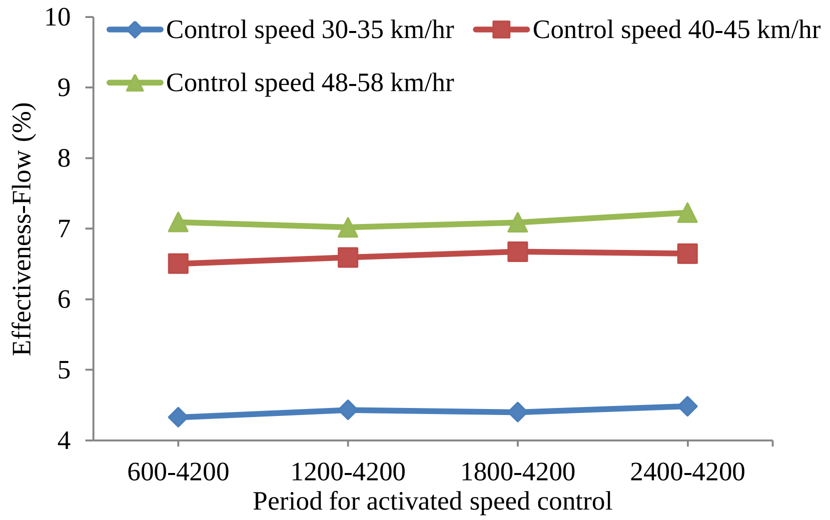
(c)

Figure 6.28 The application of Scenario 4 on the link traffic characteristics

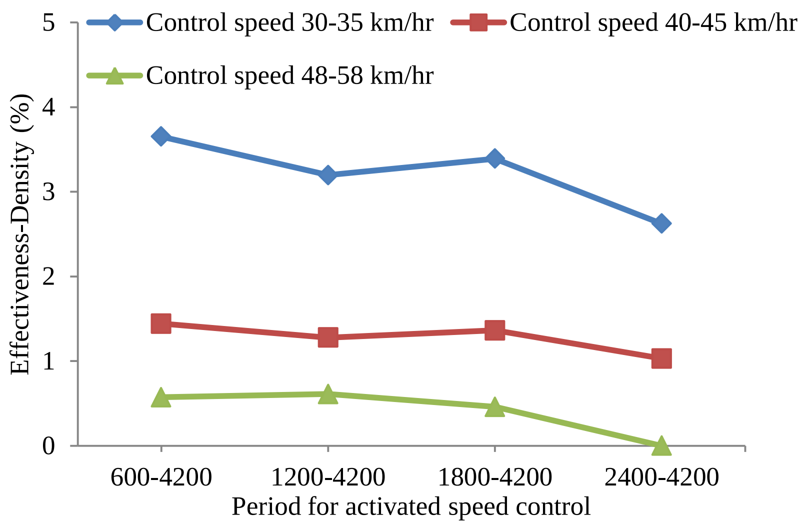
The efficiency of Scenario 4 on the link parameters is shown in Figure 6.29 (a), (b), (c). A 7-8% improvement in speed was achieved across the control ranges. An approximate 7% improvement in flow occurred for both speed controls ranges of 40-45 and 48-58 km/hr and a 4% improvement for the speed control range of 30-35 km/hr. Low levels of improvement also occurred in terms of traffic density where 3%, 1.5% and 0.5% improvement were achieved for speed control ranges of 30-35, 40-45 and 48-58 km/hr respectively.



(a)



(b)



(c)

Figure 6.29 The efficiency of Scenario 4 on the link traffic parameters

6.8 Comparison and optimisation of VSL scenarios

6.8.1 Evaluation of effectiveness of VSL at busy Intersection

The four scenarios produced different results in managing the traffic situation at intersection 2 during periods of congestion. Different outcomes were produced by the variation of the speed control ranges, the position and the distance of speed controls from intersection 2 and the activation time periods. Table 6.13 shows the maximum selective efficiency of each scenario under their subcategories. The shaded rows show the most suitable values for each scenario. For example in Scenario 1, the speed control range of 40-45 km/hr was the best traffic management strategy because it achieved the highest percentage of efficiency in terms of average queue, average delay , average stopped delay and average number of stops. Selection of the best traffic management for each scenario was based on the same criteria that were used in Scenario 1. After identifying the maximum efficiency for each scenario, optimising the selected scenarios was necessary to achieve an optimum traffic operating system.

Table 6.13 The max efficiency of all proposed scenarios under each subcategory

Scenario #	Speed Status	Speed ranges (km/hr)	Activation period (sec)	% Eff. Ave Queue	% Eff. Avg Delay	% Eff. Avg Stops #	% Eff. Avg Stopped Delay
1	Close to congested link	30-35	1800	12.92	13.11	13.67	13.27
		40-45	1800	15.92	15.13	17.9	14.97
		48-58	1800	14.78	13.97	11.35	14.72
2	Relatively far from congested link	30-35	1800	15.23	13.32	12.75	13.71
		40-45	600	13.16	12.32	14.01	11.97
		48-58	1200	9.93	9.84	10.15	9.87
3	Far from congested link	30-35	2400	8.85	8.86	9.89	8.67
		40-45	1800	9.00	9.26	9.23	9.27
		48-58	1200	8.97	9.09	10.09	8.91
4	Combine 2 & 3	30-35	1200	13.55	10.81	14.04	10.41
		40-45	600	13.75	13.22	13.73	13.17
		48-58	600	10.51	10.32	11.06	10.21

From the table above, a bar chart representing the maximum efficiency for each proposed scenarios was created as in Figure 6.30. It shows that when Scenario 1 was activated at 1800s-4200s, after the congestion started forming and a speed of 40-45 km/hr was imposed, it would produce optimum traffic characteristics for intersection 2. Scenarios 2 and 3 produced the second and third best performance for this intersection. These results indicated that the position at which the speed control was implemented had a large influence on traffic behaviour at intersection 2. When the position of the speed control was close to the congested area (as in Scenario 1), the VSL was highly efficient in developing a favourable traffic situation. This efficiency decreased if the speed control was installed further away from the source of congestion (Scenario 2 and 3). Scenario 4 involved synchronisation of Scenario 2 and 3, in other words increasing the distance of the speed control away from the congested area. This scenario improved traffic management by increasing the traffic efficiency of intersection 2 to a value close to the optimum efficiency (Scenario 2). This finding led to a conclusion that increasing the distance of speed control further from the congested area minimise the improvement for the traffic performance of the congested intersection. The application of VSLs improved the efficiency of traffic parameters at intersection 2 during the congestion period for all proposed scenarios. The traffic performance (average delay and average stopped delay) at intersection 2 improved by about 9-15% by the implementing VSL system.

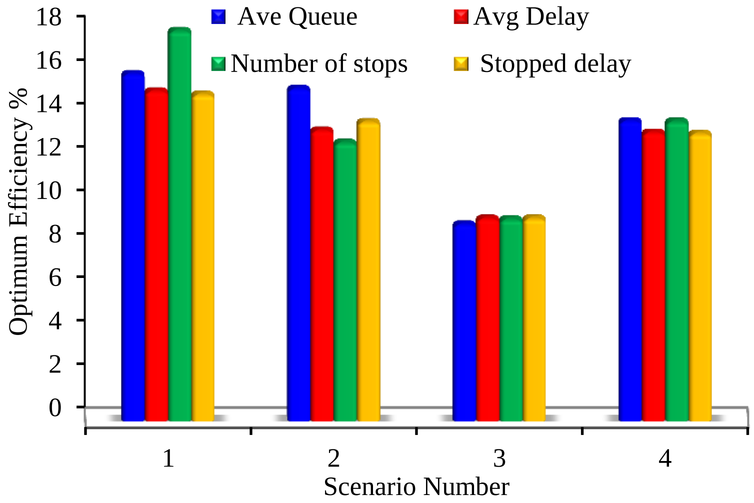


Figure 6.30 Maximum efficiency of traffic scenarios regarding the intersection characteristics

Figure 6.31 shows the annual cost of delay time for all vehicles traveling at intersection 2 under base conditions (without control) and with VSLs (control). The annual cost ($51,770) based on one peak hour period/day (evening peak), five working days per week (240 days/year) and link throughput (1,332 veh/hr).
Scenario 1 achieved a maximum reduction in delay of 15%. This reduced the annual cost of vehicle delay by $46,720 and reflected an increase of 7% (1,432 veh/hr) in vehicles travelling through this intersection. This 15% reduction in delay amounts to 493 hours per year. Scenarios 2 and 4 achieved similar savings of $46,827 with a VSL efficiency of 13%. This amounted to a combined savings of 483 hours.
Scenrio 3 reduced the annual cost of delay time to $46,933 reflecting a saving of 474 hours at a VSL efficiency of 9%. Evaluation of different VSL scenarios showed that Scenario1 was better than the other scenarios. These results are summarised in Table 6.14.

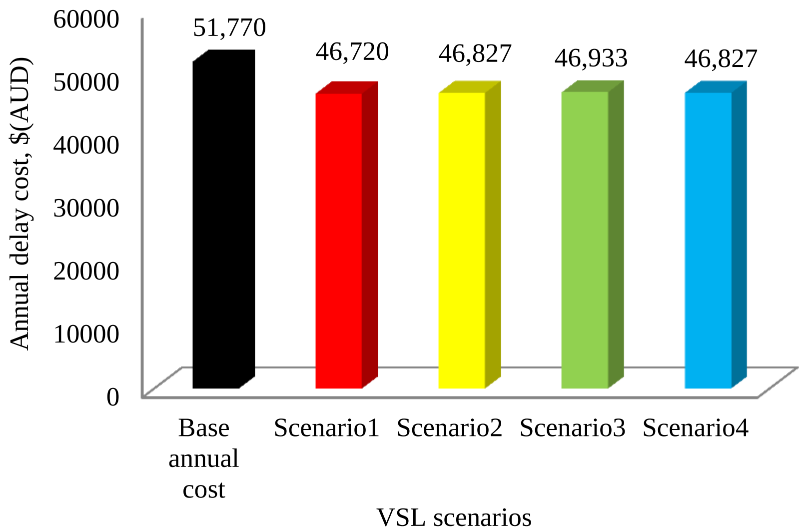


Figure 6.31 Annual delay time cost before and after applying VSL scenarios

Table 6.14 Annual delay time savings cost

Annual cost of delay time without control		VSL Scenarios	Max. delay reduction factor (%)	Annual savings in delay time	
($AUD)	(Hours)			(Hours)	($AUD)
51,770	4,884	Scenario 1	15	493	5,226
		Scenario 2	13	483	5,121
		Scenario 3	9	474	5,021
		Scenario 4	13	483	5,121

6.8.2 Evaluation of effectiveness of VSL on busy link

Optimisation of macroscopic traffic parameters for the congested link was based on the optimum scenarios mentioned above. Figure 6.32 shows a steady improvement in speed and flow parameters for the congested link for all VSL scenarios. The optimum efficiency for speed and flow were 8-9% and 5-7% respectively.

Little fluctuation can be seen in the traffic density. This may because the link is running near capacity which creates a constant spacing between vehicles, making the flow more homogeneous. Another reason could be the accompanying increase in speed and flow due to the VSL reducing the variation in vehicle traffic density (traffic density is the flow to speed ratio) when compared with base line conditions. A reduction in upstream traffic speed yielded a lower flow towards the downstream traffic during the peak hour. Reducing the rate of flow may effect the traffic density for the congested link. As long as the speed control limits increased towards the congested link, the efficiency of traffic density at the affected link improved. The optimum efficiency for the traffic density was 0.5-3%.

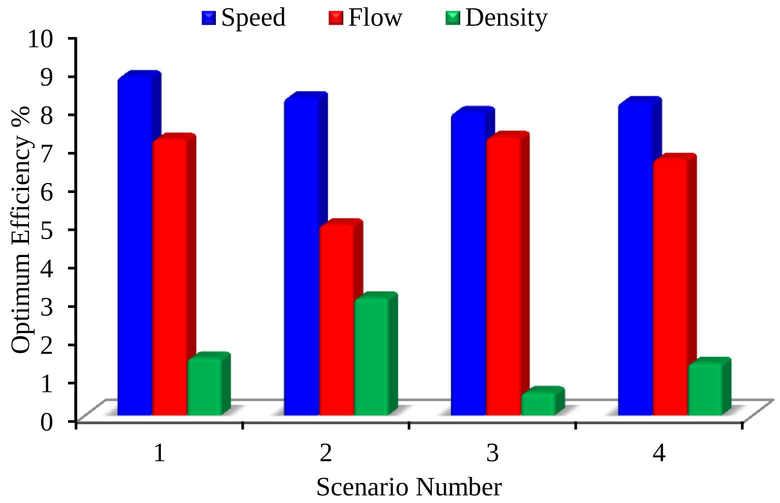


Figure 6.32 Optimum efficiency of traffic scenarios regarding the link characteristics

As discussed in the previous section, reducing the speed variation along the congested link contributed to an increase in speed homogeneity and reduced the probability of fatality. The probability of fatality decreased from 3.6% to 2.25% after applying Scenario1 and achieved an improvement of 1.35%. This 1.35% potentially saved around $82,000 in total annual crash costs at the GAR during peak hours. Table 6.15 shows the probability of fatality and its impact on the annual savings in crash costs for the GAR for the VSL scenario.

Table 6.15 Probability of fatality and total annual saving in crash costs for GAR

VSL		Base condition	VSLs application			
			Scenario 1	Scenario 2	Scenario 3	Scenario 4
*Probability of fatality (%)		3.6	2.25	2.44	2.56	2.48
Improve (%)			1.35	1.16	1.04	1.12
Savings/yr ($AUD)	Fatal		24,972	21,458	19,238	20,718
	Hospitalisation		48,985	42,091	37,736	40,639
	Medical treatment & minor injury		6,059	5,206	4,668	5,027
	Property damage		2,298	1,974	1,770	1,906
Total annual savings ($AUD)			82,314	70,729	63,412	68,290

*=(Speed deviation/71)^4

Improvement in traffic flow contributed to an increase in the annual accessibility for vehicles travelling along the congested link. Several enhancements for vehicle accessibility were achieved by using VSL applications. The maximum annual vehicle accessibility was 24,000 vehicles under VSL, Scenarios 1 and 3 while Scenarios 2 and 4 recorded 17,143 and 23,000 vehicles respectively. Figure 6.33 shows the variation in annual vehicle accessibility under different VSL scenarios.

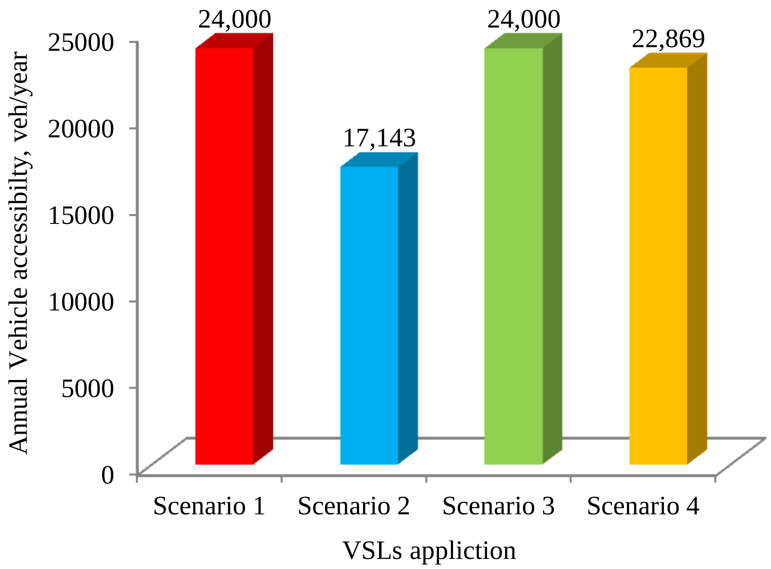


Figure 6.33 Annual vehicle accessibility after applying VSL scenarios

6.9 The influence of VSLs optimisation on the total travel time

The total travel time (TTT) along the WB lane with and without control was studied in order to explore the influence of VSL application on TTT. TTT was considered for all vehicles starting at the WB direction of the entrance link until exiting at intersection 1. This is a distance of about (6.7 km). The VSL application created different TTT results, depending on the scenario used as illustrated in Figure 6.34. This figure shows that the TTT for the base condition was 598 seconds whereas the use of various VSL scenarios resulted in TTTs of 580, 616, 576 and 597 seconds. Variation in TTT was due to the speed control limit and the distance at which it was applied in each scenario. Scenarios 1 and 3 reduced the TTT by 3.7%.

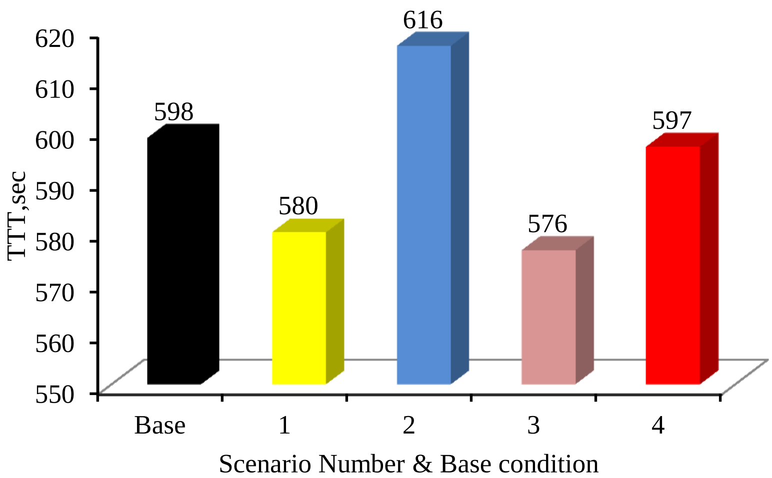


Figure 6.34 TTT comparison under control/non-control condition

For example under base condition during one peak hour there was a total of 1,542 veh/hr. At a mean travel time of 598 seconds per vehicle, this amounts to 61,474 hours per year of vehicle travel time. A maximum saving of 22 seconds per vehicle travelling this link, a reduction of 3% may at first appear trivial, but amounts to a large benefit when considering fuel of travel time saved. Based on the results from one hour of VSL simulation during peak travel time it was estimated that 59, 212 hours per year of vehicle travel time was generated.

Travel time savings have a monetary value. This 3.7% reduction in travel time translates to 2,261 hours of time saved per year for all vehicles traveling on WB beginning at the inlet link of Mains intersection until exit the link of Beatty intersection in the evening peak hour. The travel time cost value for vehicles in Brisbane city during peak hour is $10.6 (Booz, 2003). Applying this value produced a time savings estimated of $24,000.

Travel time savings can be calculated by taking the difference in average travel time per vehicle for the base and control conditions. This difference was multiplied by the total number of vehicles travelling along the link and by the total number of weekdays per a year (240 days). The result was then multiplied by the travel time value to determine the total annual travel time cost savings.

No more improvements, or negative results, were obtained by applying Scenarios 2 and 4. The reduction in the travel time for Scenarios 1 and 3 was approximately equally due to the use of speed control ranges of 40-45 km/hr, controlled distances of 750-1075 metres and an activation time of 1800s-4200s. Scenario 2 caused an increase in TTT because it constrained the flow by applying a speed control range of 30-35 km/hr, which was more restrictive than the other scenarios. In the case of Scenario 4, the effect on TTT was minimal even though the speed control range was 40-45 km/hr. This was attributed to it being applied at nearly double the controlled distance of the other scenarios. The 40-45 km/hr speed control was activated 600 seconds earlier than in the other cases. Table 6.16 shows the estimated annual time and cost scenarios for applying VSLs.

Overall, VSL application has little impact on TTT due to the constrained speed limits for non-congested links. These have a major effect on the final TTT calculation. The best traffic system for the congestion period should take into consideration trade-offs between improvement in traffic characteristics for the congested link and the total travel time.

Table 6.16 TTT cost after applying VSL

TTT without control (sec)	VSLs application	TTT (sec)	Efficiency (%)	Vehicle time saving (sec)	Annual time saving	
					(hours/year)	($AUD)
598	Scenario 1	579	3.1	19	1,953	20,704
	*Scenario 2	616	-3.0	-18	-1,850	-19,614
	Scenario 3	576	3.7	22	2,262	23,972
	Scenario 4	596	0.3	2	206	2,179

*(-) refer to increasing TTT and the annual cost

6.10 Summary

This chapter illustrated the potential impact of using VSL strategies on the performance of GAR during the peak period under various conditions. Two sets of traffic characteristics were investigated for an arterial road. These were the signalised intersection parameters and the macroscopic traffic stream parameters. The signalised intersection parameters were defined in terms of average queue, average delay, average stopped delay, and average number of stops. The macroscopic traffic parameters were defined in terms of speed, flow and density.

Four VSL scenarios were used to examine the potential of applying VSL strategies under various situations. All proposed scenarios improved traffic performance at intersection 2 and enhanced the traffic safety along the congested urban arterial road. The LOS improved from LOS E (base line condition) to LOS D (control) with different savings in delay time for all VSL scenarios.

The position at which speed controls were implemented had a large effect on the traffic performance at intersection 2. As long as the position of the speed control was close to the congested area (as in Scenario 1), VSL application was highly efficient in creating favourable traffic performance. The efficiency decreased the further the speed control was installed away from intersection 2 (e.g. Scenario 2 and 3).

Scenario 1 achieved an optimum efficiency in traffic characteristics for intersection 2 if it was activated 1800s-4200s after the start of congestion and the speed control was 40-45 km/hr. Traffic performance for intersection 2 improved by approximately 15%. For example during the peak hour considered total vehicles of 1,332 veh/hr return 4,884 delay hours per year for one evening peak hour under the base conditions. While the delay produced by VSL was 4,391 hours per year despite of the throughput at intersection 2 optimised by 7% (1432 veh/hr). This delay reduction (15%) saved a combined 493 hours per year for one evening peak hour simulation. The travel time cost values for vehicle in Brisbane city during peak hour is $10.6 (Booz, 2003), which returned time savings roughly $AUD 5,226.

Generally, all VSL scenarios enhanced traffic safety on the GAR by reducing the average number of stops up to a maximum of 18% during the peak hour. Applying the speed control overall QLD may reduce the rear-end collisions cost from $1.011billion to $829 million.

VSL scenarios achieved an approximately constant improvement in speed and flow. The maximum improvement for speed and flow were 9% and 7% respectively. Improving the speeds along the congested link contributes to increased speed homogeneity and reduce vehicle speed variation from the posted speed. Reducing speed variation reducted fatal traffic collisions.

The probability of fatality reduced from 3.6% (without control) to 2.25% after employing VSL. This was because VSL reduced the vehicles speed variation from 31 to 27.5 km/hr. Reducing the speed variation along the congested link by 3.5 km/hr reduced the probability of fatality by 1.35%. This reduction saved $82,314 in the total annual crash costs for the GAR after activating Scenario 1 of VSL.

Improved flow also contributed to an increase in annual accessibility for vehicles travelling along the congested link. A 7% improvement in traffic flow increased vehicle accessibility by 24,000 vehicles per a year. Little improvement was noticed in traffic density. This was due to the link running at near capacity with limited headway space between vehicles. This made the flow homogeneous. The maximum improvement in traffic density was 3.5%.

In addition, the total travel time (TTT) for all vehicles traveling in WB, beginning at the inlet link for the Mains intersection until exiting at the Beatty intersection was studied. Scenarios 1 and 3 achieved an acceptable result in reducing the TTT by 3.7%. There was no improvement or a worsened situation was obtained by applying Scenarios 2 and 4. This 3.7% travel time savings translated to 2,261 hours per year

of travel time which return a savings in time of roughly $24,000. VSL application had little impact on the TTT due to constrained speed limits for non-congested links.
IBM SPSS investigation found that VSL has a great significant impact on vehicle delay time at traffic signal and vehicle speeds for a link close to or away from the signalised intersection area during peak hour.

CHAPTER SEVEN
Evaluation of Signalised / VSL Integrated Traffic Control System

7.1 Introduction

This chapter provides details on the study of integrating the VSL strategy with other traffic congestion management strategies (changing the signal timings for the congested traffic) into a single unified traffic control strategy. The integrated traffic strategy was examined to determine its effectiveness in reducing the downstream intersection congestion parameters of queue length, delay, stopped delay, and number of stops during the evening period. Its effectiveness in enhancing traffic flow characteristics in terms of speed, flow and density was also studied.

The total travel time before and after applying the control strategy along the westbound direction was considered in the evaluation of the efficiency of this new control system. Comparisons between traffic base line conditions (no-control) and the new integrated strategy were made to evaluate its effectiveness.

7.2 Traffic scenario descriptions

The combination of VSL applications and changes in signal timing for congested traffic is typically called an Integrated Traffic Control System (ITCS).

In the research, two traffic control scenarios were applied to manage the traffic situation on the GAR. These are discussed in detail in the following sections.

7.2.1 Traffic Scenario 1 (ITCS 1)

ITCS 1 includes two types of traffic management, intersection management and approach link management. Intersection management controls the operation of traffic signals at congested intersections during the evening peak period, while approach link management manages the upstream flow towards the congested intersections during the same peak period. Since the network operated at peak periods, and the cycle length for intersections 1 and 2 was relatively high, a reasonable increasing of 10 seconds and 20 seconds were added to the WB lanes in Scenarios 1 and 2 respectively to avoid forming an extra delay for other approaches.

This scenario (ITCS 1) increased the cycle length of intersections 1 and 2 by 10 seconds, and managed upstream traffic with two control speed limits (a and b). Concurrently the upstream traffic flow was controlled by using speed control ranges of 30-35 km/hr and 40-45 km/hr. The two speed control ranges were activated at different locations on the road network and for various time periods. The locations of the control speeds (CS) were illustrated in detail in Chapter 6, section (6.2). Three control speeds CS1, CS2, and CS3 were placed along the WB lane at different distances from intersection 2 so as to be close to, in the middle of and far from intersection 2. The CS was activated for the four time periods of 600s-4200s, 1200s-4200s, 1800s-4200s, and 2400s-4200s. VISSIM software was used to test the results of applying the ITCS scenarios. Figure 7.1 shows the flow chart for ITCS 1.

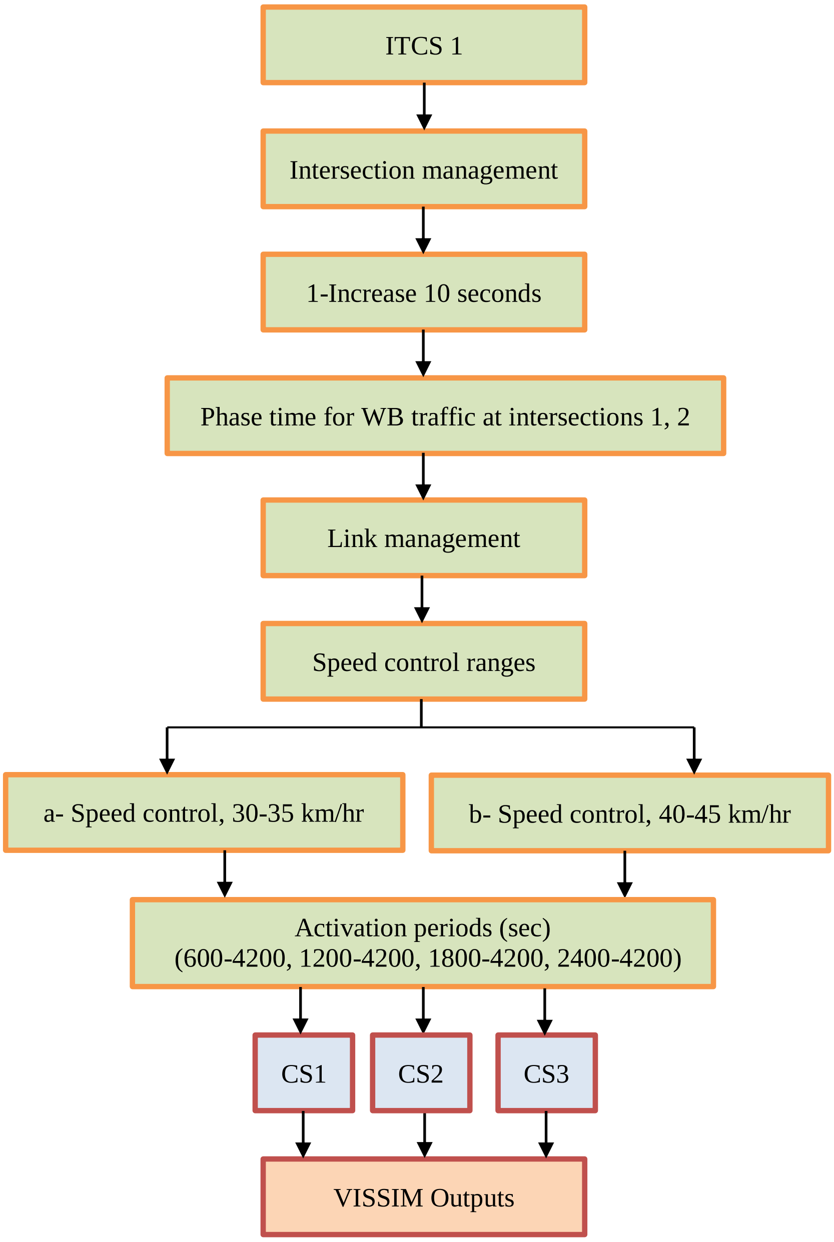


Figure 7.1 Flow chart for traffic Scenario 1 (ITCS1)

7.2.2 Traffic Scenario 2 (ITCS 2)

ITCS 2 closely resembles ITCS 1 in regard to its sequence but it is different in that it increases the signal timing for the WB traffic lanes of intersection 1 and 2 by 20s instead of 10s. The management of the approach link remained the same as ITCS 1. Figure 7.2 depicts the flow chart for constructing ITCS 2.

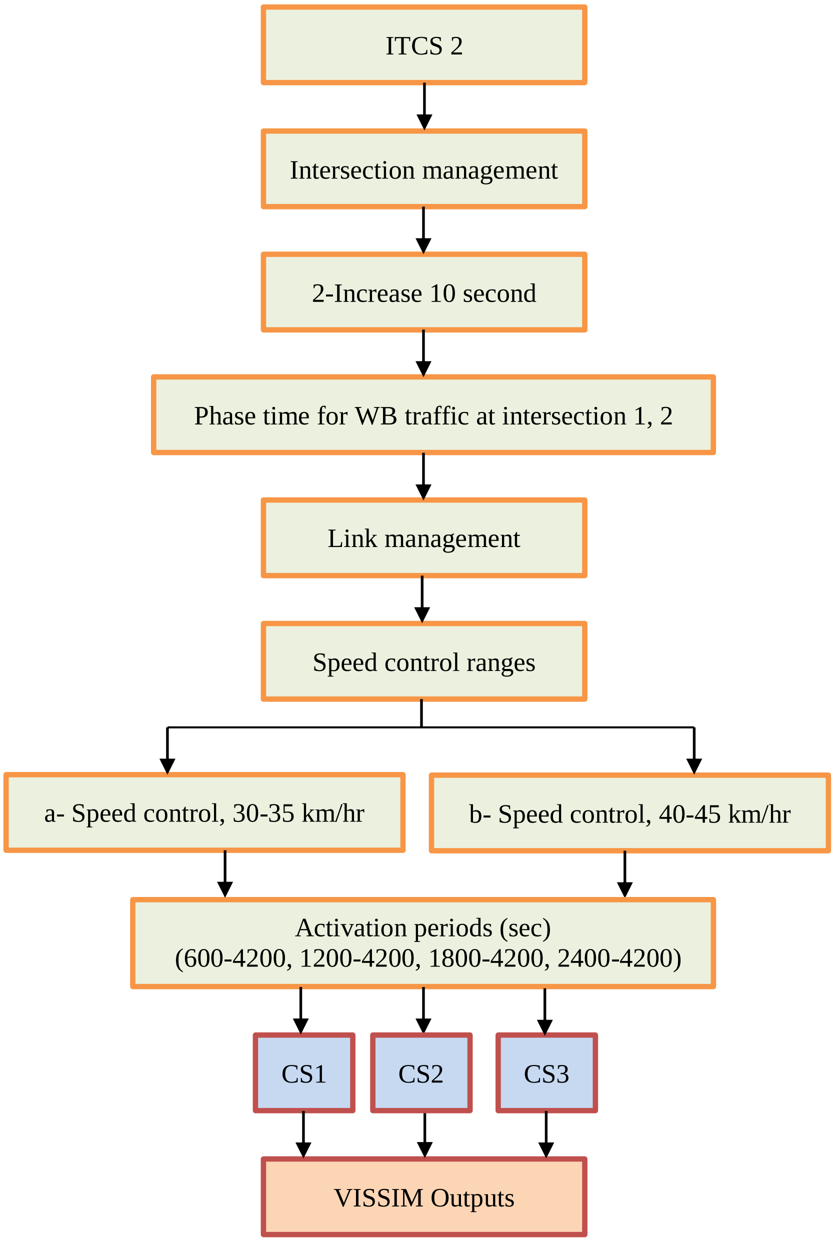


Figure 7.2 Flow chart for traffic Scenario 2 (ITCS2)

7.3 The aim of ITCS

The aim of using ITCS was to improve intersection performance in terms of queue length, delay and stopped delay, and the number of stops, by changing the timing signal plan for the congested intersections, and improving the macroscopic traffic characteristics of the approach link to intersection 2 in terms of speed, flow, and density. This was done by controlling the speed of upstream flow towards the downstream intersections.

7.4 Evaluation procedure of ITCS

The evaluation is based on VISSIM outputs for the variation in traffic conditions before and after implementing ITCS. It only considers the peak period because VSLs have best potential for use during this period. Adverse traffic performance may result

if VSL was used in off-peak period as reported by Heygi et al. (2005), Daganzo (2007) and Carlison et al. (2009).
Two sets of parameters were used to measure the efficiency of using ITCS. These were signalised intersection indicators and macroscopic traffic flow indicators. The differences in each individual indicator were examined for both non-control and control conditions. Total travel time (TTT) for vehicles that entered and exited the WB direction of GAR was investigated for both no-control and control conditions.

7.5 Results and discussions

7.5.1 Effect of ITCS 1 on the performance of Intersection 2

Evaluation was based on the variation between two sets of traffic conditions before and after applying ITCS on the WB lane at intersection 2. ITCS 1 has two-operational parts. The first case SCL (a) with proposed a speed control limit (SCL) of 30-35 km/hr and the second case SCL (b) with proposed a SCL of 40-45 km/hr each for different activation periods. The following sections discuss in detail how the traffic characteristics of intersection 2 were affected by ITCS 1.

7.5.1.1 Evaluation of average queue length

Figure 7.3 shows the impact of a SCL of 30-35 km/hr in ITCS 1 on the average queue length at intersection 2. The base condition resulted in a longer queue length than the other proposed sub- scenarios. The average queue length was affected by the location of the control speed zone (CS) and the length for which the CS was activated, i.e. CS1, CS2 and CS3 are all different. The queue length for each activated control speed differs from other activated control speed periods. CS2 activated for (1800s-4200s) produced a shorter queue length than other control speed locations. This means the location of CS2 at the mid distance from intersection 2 was the optimum position to manage the queue parameter. All sub-categories of SCL 30-35 km/hr produced acceptable reductions in queue length.

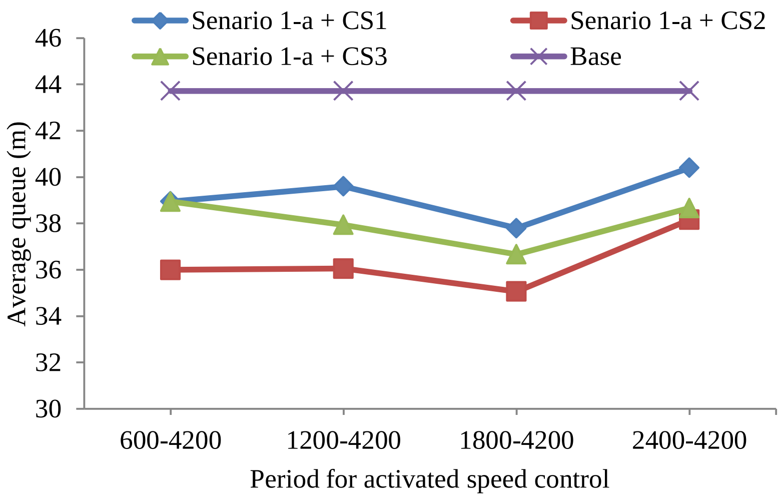


Figure 7.3 Average queue length before and after applying ITCS1, SCL (a)

Figure 7.4 shows the impact of Scenario 1 and a SCL of 40-45 km/hr on the performance of queue length in the WB lane of intersection 2. The location of control speed CS3 and CS2 had nearly the same effect on the reduction of queue length for all activation periods. All sub-categories of SCL 40-45 km/hr produced approximately the same improvement in queue length when activated for 1800s-4200s. This indicates the speed limit range of 40-45 km/hr produced a good synchronisation between upstream traffic and the timing signal plan for downstream intersections for all sub-scenarios.

Applying SCL (b) yielded less improvement than SCL (a) because the increased speed of vehicles increased the number of vehicles moving towards the downstream intersection. This resulted in longer queue lengths. The base condition produced a 44m queue length. SCL (a) and (b) of Scenario1 produced 35m and 37m queue lengths, respectively.

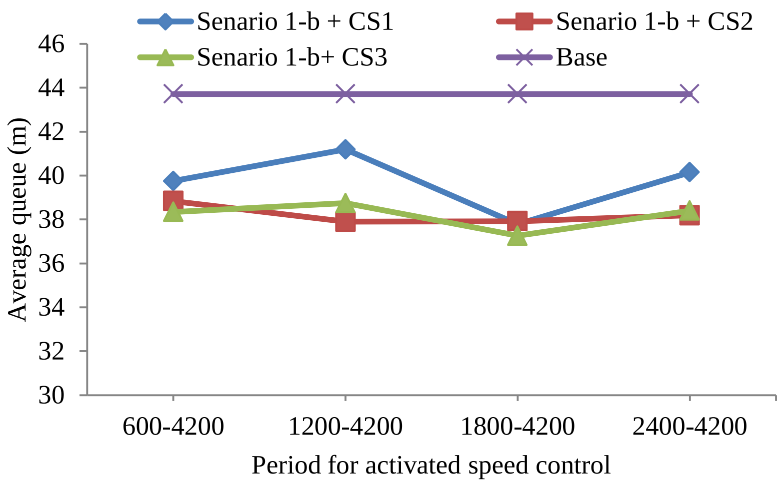


Figure 7.4 Average queue length before and after applying ITCS1, SCL (b)

Figure 7.5 and Figure 7.6 show the efficiency of SCL (a) and SCL (b) of Scenario 1 on queue length. SCL (a) achieved the highest efficiency (20%) when CS2 was activated for 1800-2400s. CS3 and CS1 produced efficiencies of 16% and 14% respectively. In contrast to VSLs, SCL (a) reduced the queue length by 3% more than by using VSL (17%). This difference is due to cycle length increments. The best location for VSL was CS1 which is close to the downstream intersection but in case of ITCS1 it was CS2 which is mid distance to the downstream intersection. The change in the control speed location was necessary to provide extra time for vehicles at the downstream intersection to clear before vehicles from the upstream flow arrived.

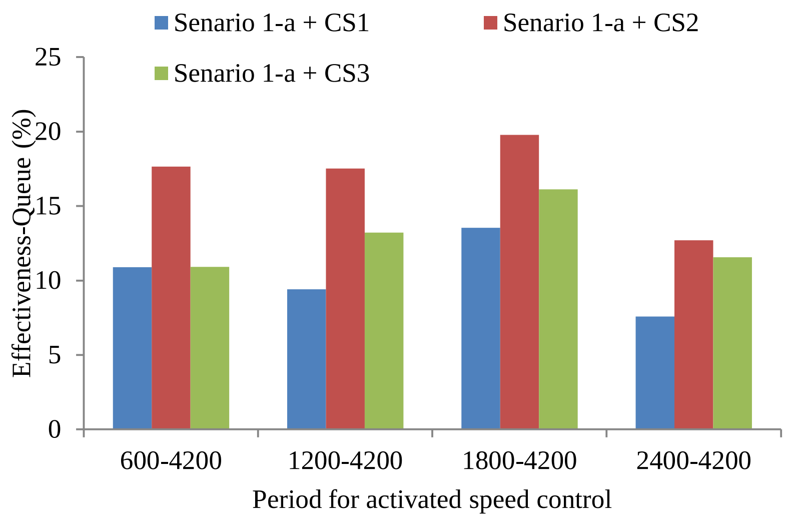


Figure 7.5 Efficiency of ITCS 1, SCL (a) on average queue length

SCL (b) achieved 14% efficiency at CS3 and 13% at CS1 and CS2. CS3 recorded a higher efficiency than other locations because increasing the speed of vehicles required a greater control distance. This location provided the upstream vehicles additional time to arrive downstream intersection and this minimised the interaction with the downstream vehicles resulting in a shorter queue length. Utilising the speed limit range of 40-45 km/hr at the link close to the downstream intersection reduced the improvement for queue length.

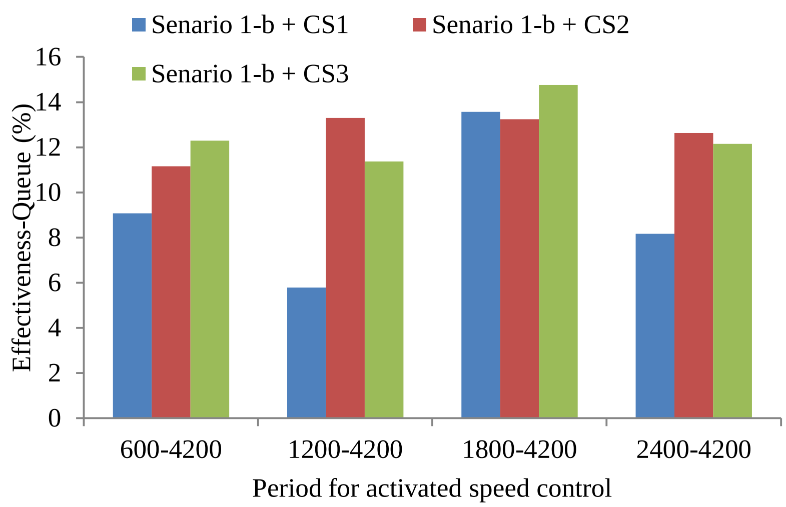


Figure 7.6 Efficiency of ITCS 1, SCL (b) on average queue length

7.5.1.2 Evaluation of average delay parameter

Figure 7.7 and Figure 7.8 show the impact of ITCS 1 on average vehicle delay at intersection 2. The delay was 55 seconds for the baseline situation (no-control). This means the intersection was operated at level of service (LOS) E, (TRB, 2000). The lowest delay, 44.5 seconds, incurred after activating SCL (a) at CS2 for 1800s-4200s. All sub-categories of SCL (a) had a reduced delay parameter. This means the LOS improved form E to D. The variation in reduction between scenarios was small because the control speed locations were close to each other. The reduction in vehicle queue length achieved in previous section improved the performance of the intersection through reducing the delay. This state support that a strong relationship between the length of the queue and the delay of vehicles.

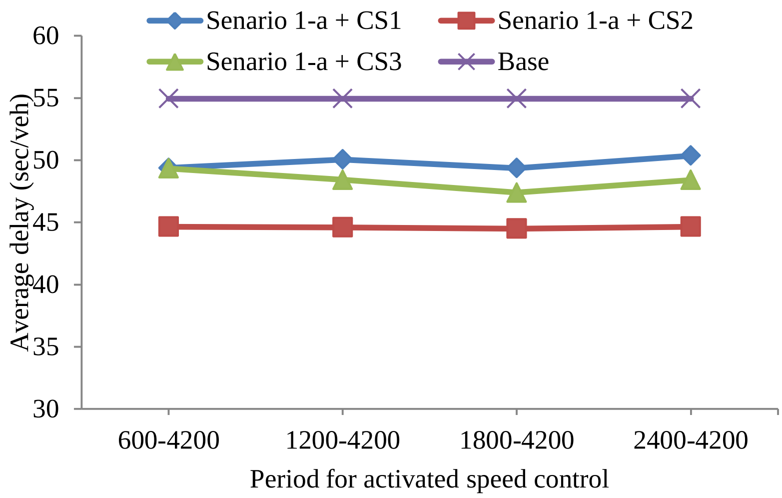


Figure 7.7 Average delay before and after applying ITCS 1, SCL (a)

Figure 7.8 shows the effect of SCL (b) on average delay. The delay was reduced despite increasing the speed limit to 40-45 km/hr. The lowest delay (45.5s) was incurred by CS2. This means the LOS at downstream intersections also improved to LOS D. This control speed positions produced good synchronisation between the upstream flow and operation of the downstream intersection.

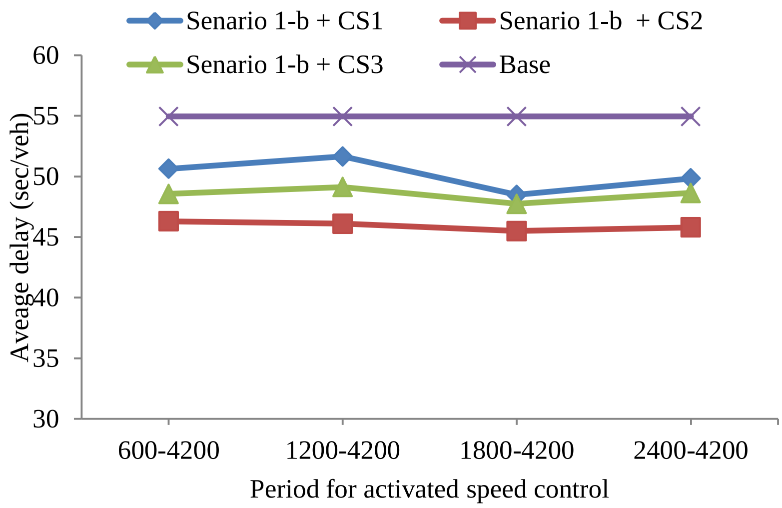


Figure 7.8 Average delay before and after applying ITCS 1, SCL (b)

Figure 7.9 and Figure 7.10 show the efficiency of ITCS 1 with respect to average delay factor. The maximum efficiency was 19% for SCL (a) and 17% for SCL (b). This difference was related first to the reduction in queue length, second to the type of speed limit, and third to the location of control speed along the WB route. By contrast with VSL efficiency gain of 15%, SCL (a) produced a 4% better efficiency and SCL (b) produced a 2% better efficiency than VSL application alone. This was due to the speed activation period for VSL being longer (600s-4200s) than the activation period for ITCS (1800s-4200s). ITCS1 was effective at improving the delay parameter if it was activated 1800s after the congestion began forming.

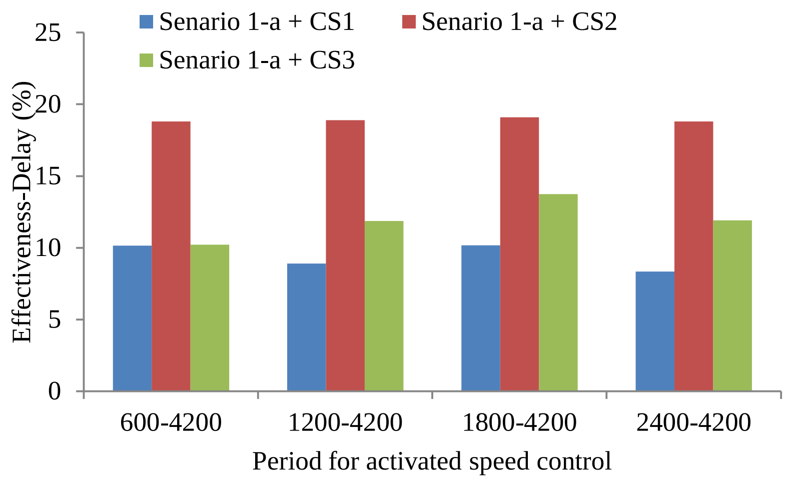


Figure 7.9 Efficiency of ITCS 1, SCL (a) on average delay

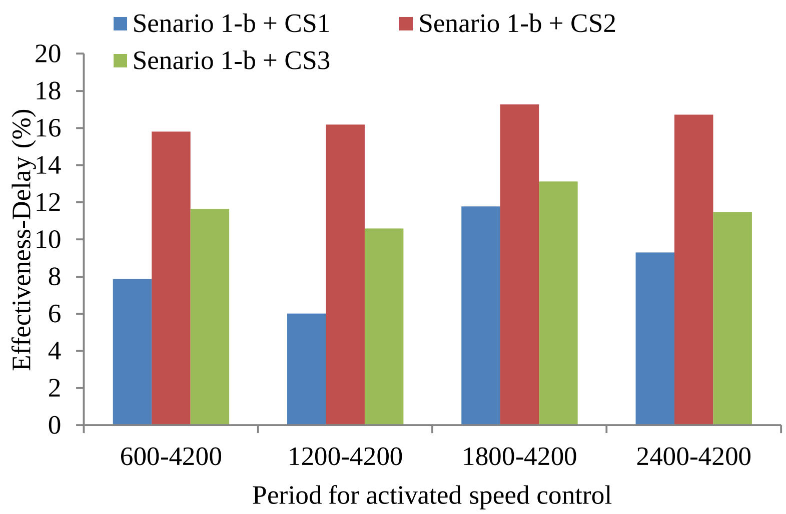


Figure 7.10 Efficiency of ITCS 1, SCL (b) on average delay

The efficiencies obtained from using SCL (a) and (b), were a reduction in the annual delay time for the current traffic situation from 4,884 hours to 4,248 hours and 4,344 hours per year respectively. These time savings of 636 and 540 hours per year and returned savings of $6,742 and $5,724. Table 7.1 shows the savings produced by ITCS1 at intersection 2 in the evening peak hour.

Table 7.1 Annual delay time savings after using ITCS1

	Base condition	ITCS1	
		SCL (a)	SCL (b)
Annual delay hours (hours)	4,884	4,248	4,344
Annual cost of delay (AUD)	51,770	45,029	46,046
Annual savings (hours)		636	540
Annual savings ($AUD)		6,742	5,724

7.5.1.2.1 Statistical significant of delay results

In order to evaluate statistically the significant of ITCS1 on vehicle delay time at traffic signal, One-Way ANOVA at 95% level of confidence was used using IBM

SPSS software. The method is based on the comparison of delay time models using data before and after the implementation of the ITCS1.
Table 7.2 describes the variance in results for the base condition and ITCS1. Table 7.3 indicates that VSL has very significant impact on vehicle delay time because the significant value for ANOVA was much more less than confidence interval (0.000 < 0.05).

Table 7.2 Delay statistical descriptive

Status	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base condition	30	54.94	9.04	1.65	51.56	58.32	35.86	72.14
ITCS1	30	44.49	7.44	1.35	41.71	47.27	28.53	62.03
Total	60	49.72	9.75	1.25	47.20	52.24	28.53	72.14

Table 7.3 Delay, One Way-ANOVA

Status	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	1638.07	1	1638.07	23.88	.000
Within Groups	3977.10	58	68.57		
Total	5615.18	59			

7.5.1.3 Evaluation of average stopped delay

Figure 7.11 and Figure 7.12 show the impact of ITCS 1 on average stopped delay at intersection 2. The average stopped delay was 45 seconds before applying ITCS1 and reduced considerably after its application. Figure 7.11 shows the impact of SCL (a) on the average stopped delay factor. The maximum reduction in delay was 37.1 seconds when SCL (a) was activated at CS2 for 1800s-4200s. Despite CS2 reducing the delay when it was activated at 1800 seconds, all sub-categories of SCL (a) produced results that were very close to this minimum value.

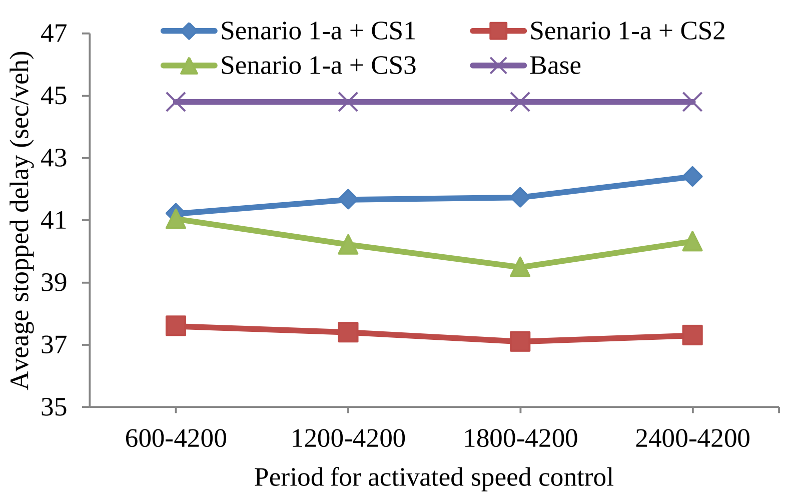


Figure 7.11 Average stopped delay before and after applying Scenario1, SCL (a)

Figure 7.12 shows the impact of SCL (b) on the average stopped delay. This sub-scenario had less effect on improving the average stopped delay compared to SCL (a) because the speed control limit was changed from 30-35 km/hr to 40-45 km/hr. The lower stopped delay was 38.5 seconds when CS2 was activated for 1800s-4200s. Changing the SCLs increased the queue length at downstream intersections and increased the interaction between vehicles. This reduced the effectiveness of this sub-scenario.

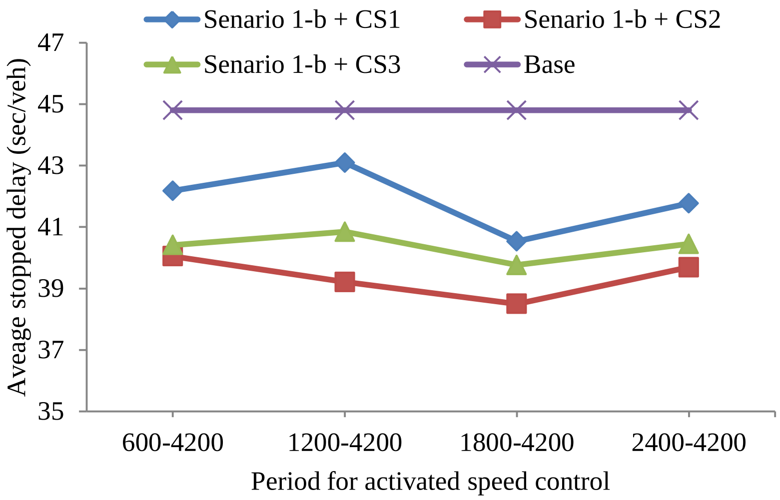


Figure 7.12 Average stopped delay before and after applying ITCS 1, SCL (b)

The effect of this sub-scenario on the average stopped delay is shown in Figure 7.13 and Figure 7.14. The maximum efficiency reached 17% by SCL (a) and 14% for SCL (b). In contrast with VSLs, SCL (a) was 2% better and SCL (b) was 1% worse. The differences in the percentage of ITCS1 efficiency were because of the differences in the speed control limit ranges. The speed limit of SCL (a) was lower than the limit for SCL (b). This caused more restriction on the upstream flow and reduced interaction between vehicles at the downstream intersection.

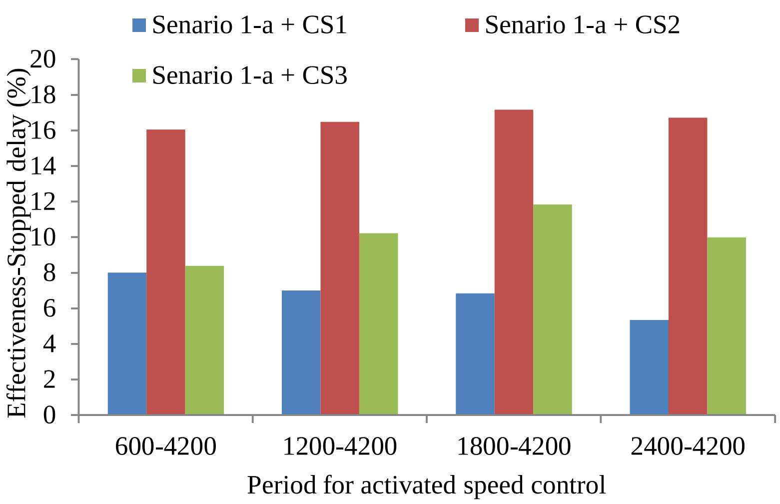


Figure 7.13 Efficiency of ITCS 1, SCL (a) on average stopped delay

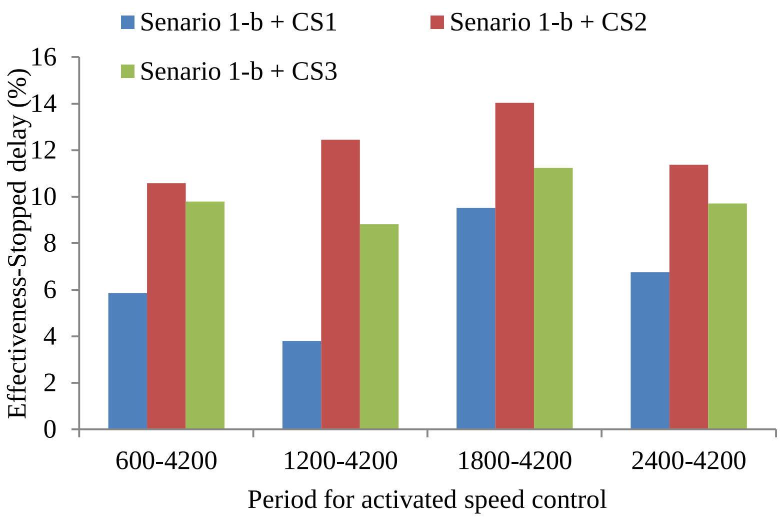


Figure 7.14 Efficiency of ITCS 1, SCL (b) on average stopped delay

7.5.1.4 Evaluation of average number of stops

Figure 7.15 and Figure 7.16 show the impact of ITCS 1 on the average number of stops. The base condition had the highest number of stops of approximately 1.16 during the evening peak period. ITCS1 minimised the number of stops to 0.91 and 0.93 using SCL (a) and (b) respectively. Reducing this number increased the traffic safety at downstream intersections by reducing the braking of vehicles and the probability of rear-end collisions as they approached the intersection. There was an insignificant difference in results for SCL (a) and SCL (b) among all sub-scenarios.

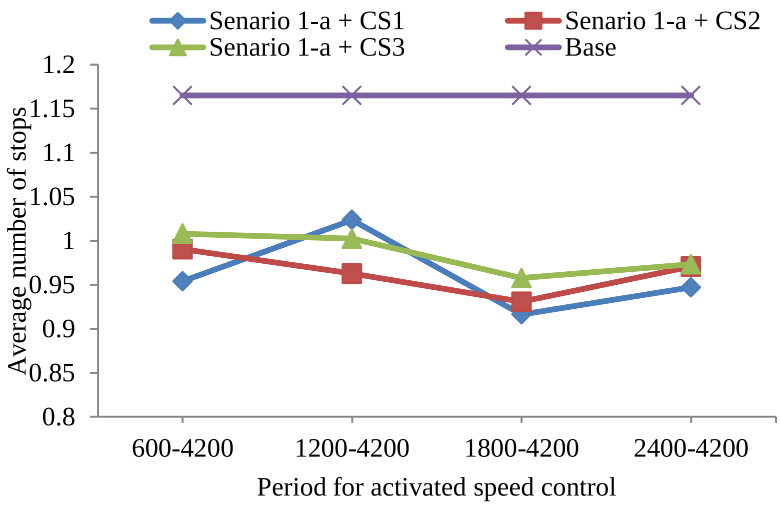


Figure 7.15 Average number of stops before and after applying Scenario1, SCL (a)

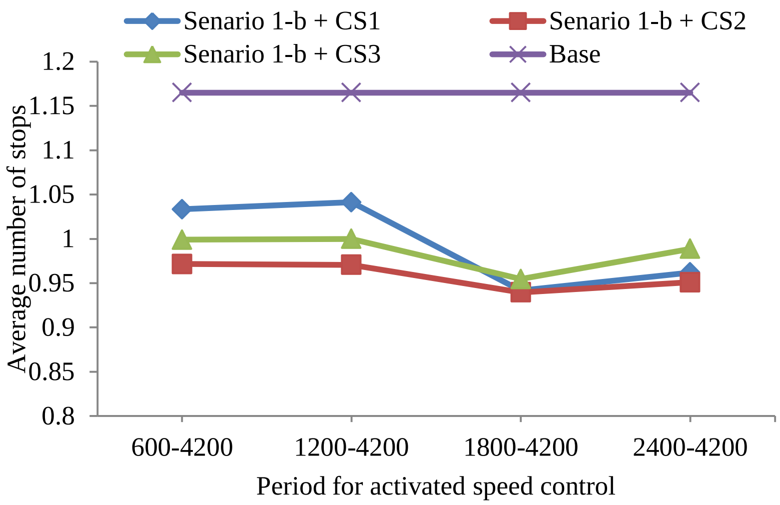


Figure 7.16 Average number of stops before and after applying Scenario1, SCL (b)

Figure 7.17 and Figure 7.18 show the effect of ITCS 1 on the average number of stops. SCL (a) decreased the number of stops by 21% and SCL (b) decreased them by 19%. These results explain why increasing the speed limits produced less improvement than reducing the speed limit. Increasing the speed limits increased the flow towards the downstream intersection, increasing the deceleration and acceleration between vehicles which resulted in more partial stops. In contrast with VSLs, ITCS1 reduced the number of stops by 2-8%. This increased the traffic safety by reducing the probability of vehicle collisions through the need to brake.
Table 7.4 indicates ITCS1 (among the four-traffic parameters) reduced the average number of stops by 21% and the average delay by 19%. These findings support using ITCS1 to raise the traffic safety and improve the level of services during peak periods in the vicinity of intersections.

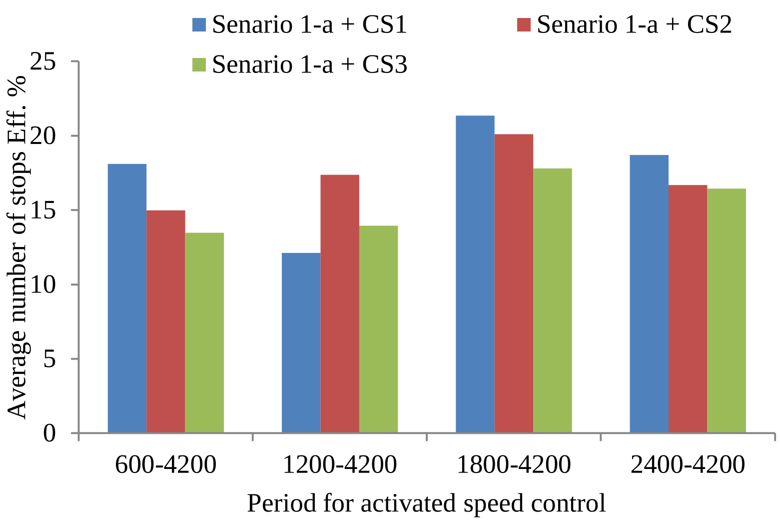


Figure 7.17 Efficiency of ITCS 1, SCL (a) on average number of stops

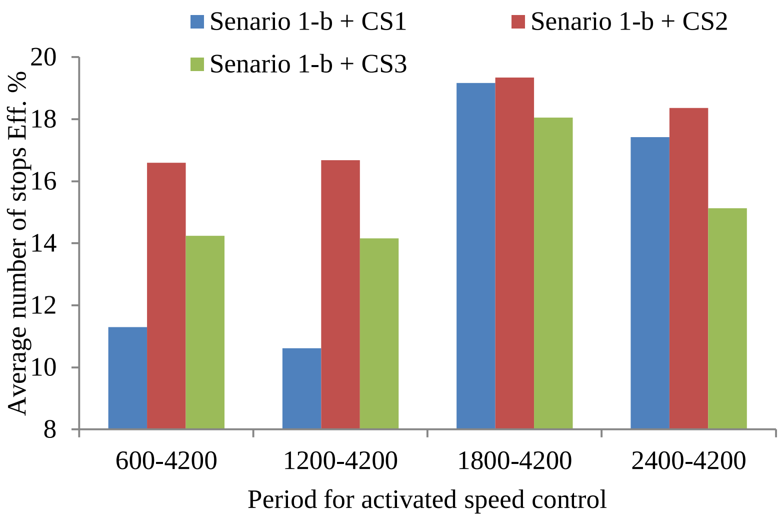


Figure 7.18 Efficiency of ITCS 1, SCL (b) on average number of stops

Table 7.4 Maximum efficiency of ITCS 1 regarding intersection parameters

Efficiency ITCS 1	Average queue %	Average delay %	Average stopped delay %	Average number of stops %
SCL (a)	20	19	17	21
SCL (b)	15	17	14	19

7.5.2 Evaluation of ITCS1 on the performance of macroscopic traffic flow parameters

The following section discusses the traffic flow characteristics of the WB link at intersection 2 under base line conditions (no-control) and with ITCS1. The macroscopic traffic flow parameters were speed, flow and density.

7.5.2.1 Evaluation of changes in average space speed

Figure 7.19 and Figure 7.20 show the influence of ITCS 1 on the average speed parameter during the evening period for SCL (a) and (b) respectively. The average speed was 39 km/hr for base line traffic conditions (no-control). After applying ITCS1, the average speed increased from 39 km/hr to around 42 km/hr for all sub-scenarios. These results provide further support for the use of ITCS1. It was able to create a homogeneous traffic platoon with constant space headways which resulted in more uniform speed distribution.

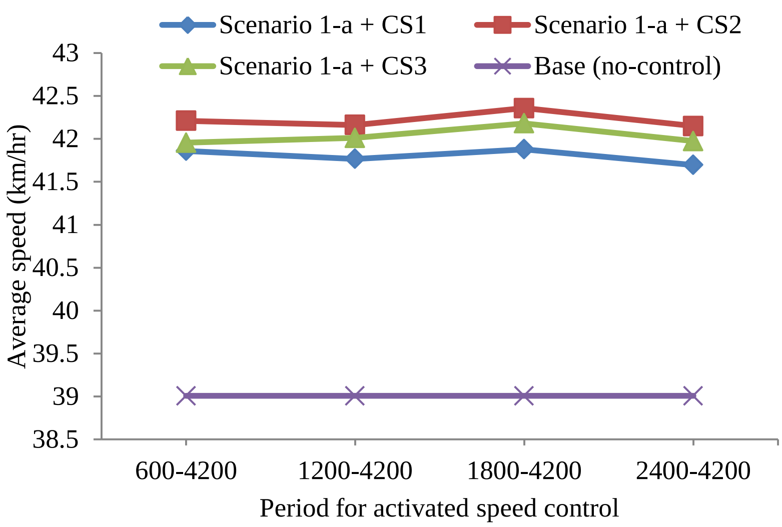


Figure 7.19 Average speed before and after applying Scenario1, SCL (a)

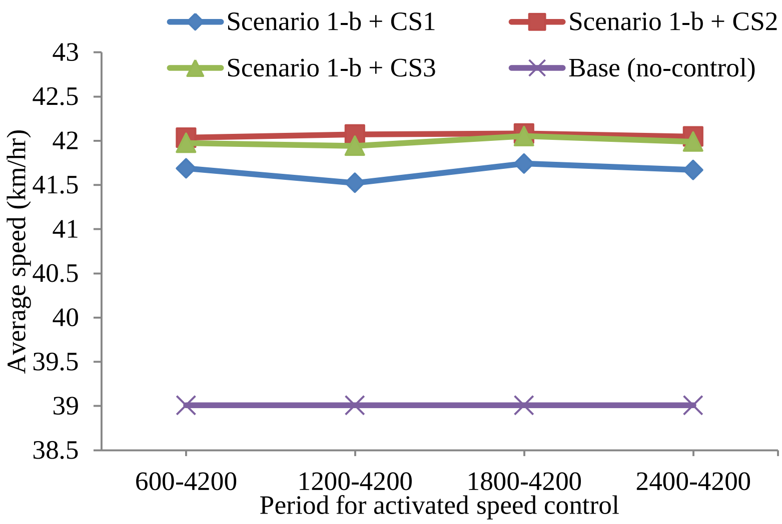


Figure 7.20 Average speed before and after applying Scenario1, SCL (b)

Figure 7.21 and Figure 7.22 show the effect of ITCS 1 on average speed. The effect of ITCS1 on congestion at intersection 2 was nearly constant in the range of 7-8% for all sub-scenarios. This limited improvement occurred for peak hour traffic by creating nearly uniform space headway. The location at which the control speed was implemented did not effect the speed. It did have an effect on the signalised intersection parameters. The location of the control speed made no difference because the link was already experiencing a high traffic volume. The efficiency of ITCS1 was almost the same as VSL. The similarity in results for both scenarios was due to the limited capacity of the road and because the analysis was carried out during the period of maximum congestion.

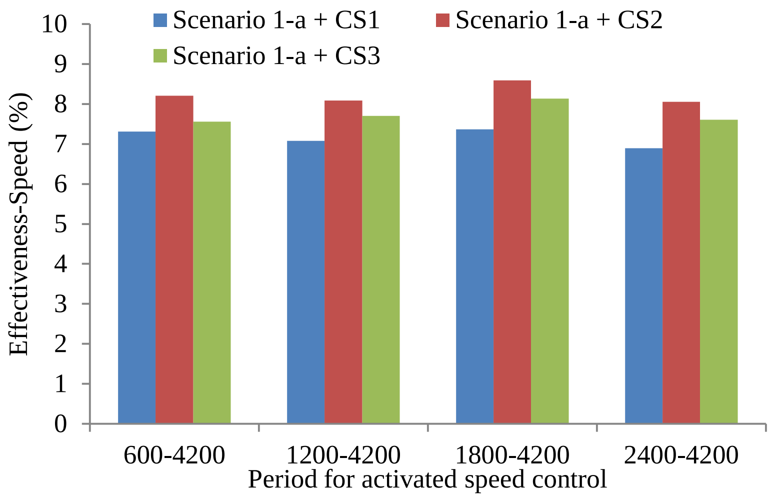


Figure 7.21 Efficiency of ITCS 1, SCL (a) on average speed

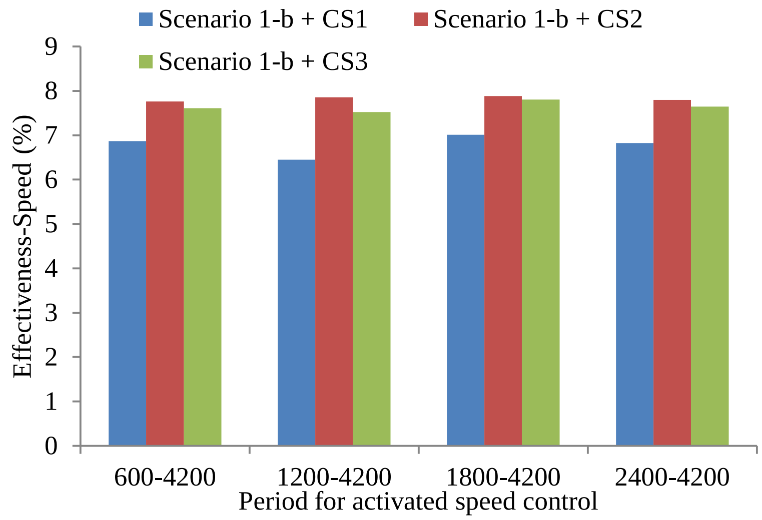


Figure 7.22 Efficiency of ITCS 1, SCL (b) on average speed

7.5.2.1.1 Evaluation of savings in crash costs on the GAR

ITCS1 reduced the variation in speeds along the congested link and potentially reduced the probability of fatalities during the congestion period. The reduction in the rate of fatal collisions is attributed to better speed homogeneity and reduce variation in speed. Table 7.5 shows the probability of fatality reduced from 3.6% (without control) to 2.41% after employing ITCS1. The reason being that ITCS1 reduced the speed variation for the vehicles travelling along the congested link from 31 km/hr to 27.65 km/hr. The decrease in speed variation 3.35 km/hr reduced the probability of fatality at GAR by 1.3%. Table 7.6 shows different severity of crash data based on 2006-2010 for the GAR during peak hour. Applying ITCS1 reduced the estimated costs to all crash types and returned savings approximatlly $79,000 per year.

Table 7.5 Probability of fatality before and after using ITCS1

Status	Average speed (km/hr)	Posted speed (km/hr)	Speed deviation (km/hr)	Probability of Fatality* (%)	Improve (%)
Base condition	39	70	31	3.6	1.3
ITCS1	42.35	70	27.65	2.3	

Probability of fatality= (Speed devaition/71)^4

Table 7.6 Total annual savings in crash costs at GAR after applying ITCS1

Year	Fatal	Hospitalisation	Medical treatment & minor injury	Property damage
2006	0	11	31	16
2007	1	14	19	9
2008	1	13	25	19
2009	1	14	27	17
2010	0	7	30	13
Average	0.6	11.8	26.4	14.8
Improve 1.3%	0.0078	0.1534	0.3432	0.1924
Cost/crash ($AUD)	3,083,000	307,500	17,000	11,500
Savings/year ($AUD)	24047.4	47170.5	5834.4	2212.6
Total annual savings ($AUD)		79,265		

7.5.2.1.2 Statistical significant of speed results

In order to evaluate the significance of ITCS1 on speed parameter for the WB link at intersection 2, using data before and after the implementation of the ITCS1 for both parts was compared. One-Way ANOVA at 95% level of confidence considered to estimate the significant of employing ITCS1. Table 7.7 and Table 7.9 show the variance in speed results for part 1 and part 2 under the base condition and ITCS1. Table 7.8 and Table 7.10 present that VSL statistically had significant impact on vehicle speeds.

Table 7.7 Speed statistical descriptive for part 1

Status	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base condition	30	12.04	1.83	.33	11.36	12.73	8.90	16.35
ITSC1	30	13.66	1.37	.25	13.15	14.17	10.19	16.66
Total	60	12.85	1.80	.23	12.39	13.32	8.90	16.66

Table 7.8 Speed, One-Way ANOVA for part 1

Status	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	39.295	1	39.295	14.946	.000
Within Groups	152.492	58	2.629		
Total	191.787	59			

Table 7.9 Speed statistical descriptive for part 2

Status	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base Condition	30	65.96	.89334	.16310	65.63	66.30	64.32	68.22
ITCS1	30	71.04	1.148	.20	70.62	71.47	68.70	73.86
Total	60	68.50	2.75	.35610	67.79	69.22	64.32	73.86

Table 7.10 Speed, One-Way ANOVA for part 2

Status	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	387.48	1	387.48	365.97	.000
Within Groups	61.40	58	1.05		
Total	448.89	59			

7.5.2.2 Evaluation of the changes in flow

Figure 7.23 and Figure 7.24 show the variation in flow before and after applying SCL (a) and SCL (b) of ITCS 1 respectively. The flow along the congested link before applying ITCS 1 was 1,332 veh/hr and improved after using ITCS1. After applying SCL (a) at CS2, the flow was 1,398 veh/hr and 1420 veh/hr when SCL (a) was implemented at CS1 and CS3. After applying SCL (b), the flow was 1,430 veh/hr at all control speed (CS) locations. This indicates that ITCS1 maximised vehicle throughput during the evening peak period. Only small differences were found in the flow profiles after using ITCS1. This is due to the limited road capacity that constrained the improvement of flow rate during the peak period.

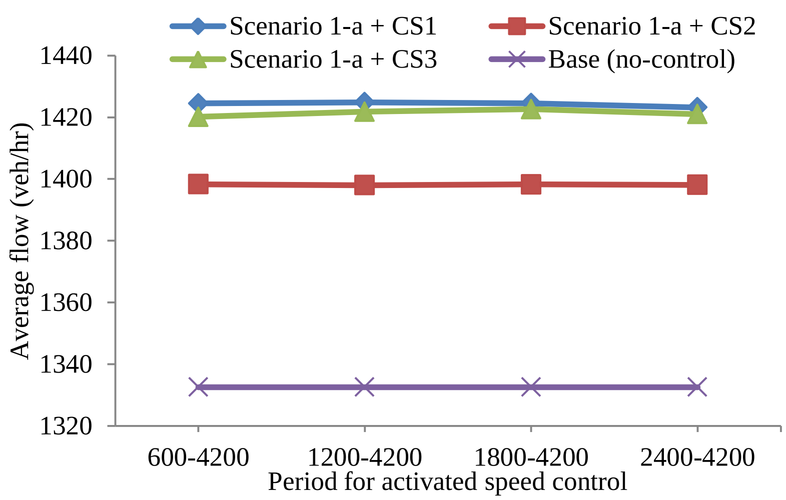


Figure 7.23 Average flow before and after applying Scenario1, SCL (a)

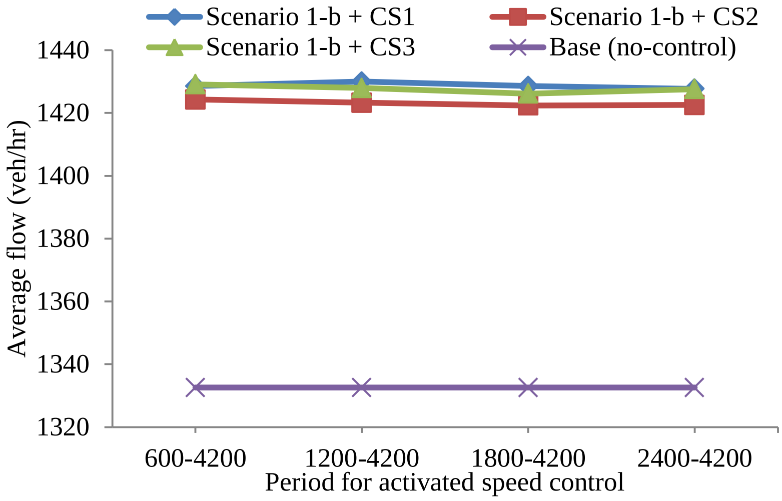


Figure 7.24 Average flow before and after applying Scenario1, SCL (b)

Figure 7.25 and Figure 7.26 show the effect of ITCS 1 on average flow. The maximum improvement from ITCS1 was 5-7% for all sub-scenarios. ITCS1 had a similar effect to VSLs in improving the flow rate of the congested link (5-7%). One of the issues that emerged from these findings was that increasing the cycle length by 10 seconds did not significantly influence the vehicle throughput during the peak period. The large number of interactions between vehicles due to the high traffic volume negated the small improvements in cycle length during the peak period.

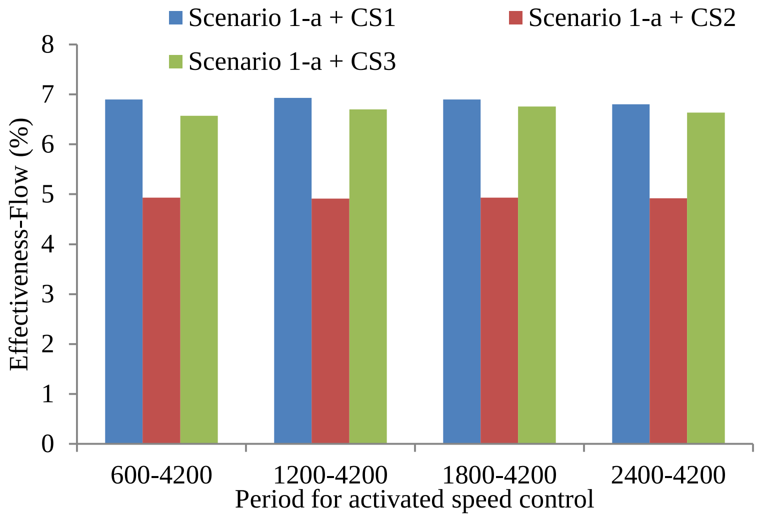


Figure 7.25 Efficiency of ITCS 1, SCL (a) on average flow

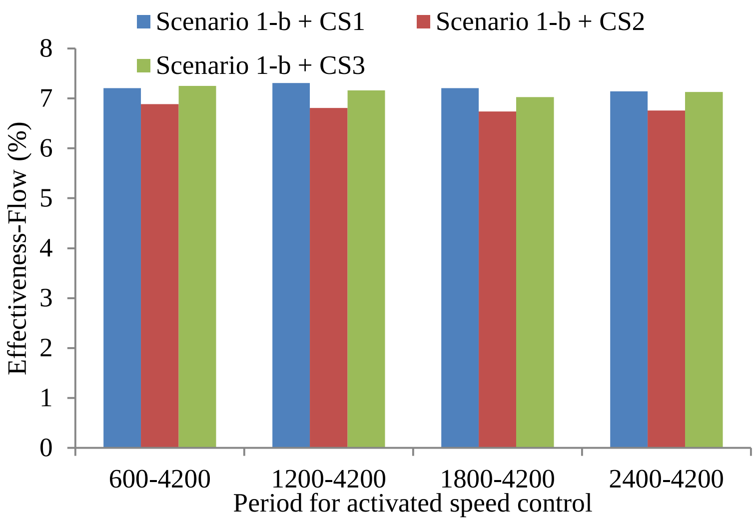


Figure 7.26 Efficiency of ITCS 1, SCL (b) on average flow

Figure 7.27 shows the effect of ITCS1, SCL (b) at control speed location 1 (CS1) on the annual vehicle accessibility during the congestion period. The annual vehicle accessibility increased by around 24,000 vehicles per year for all activation period.

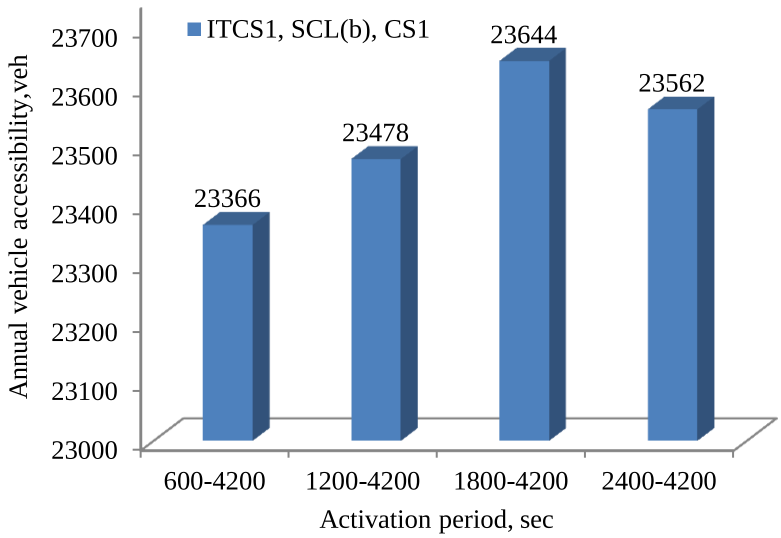


Figure 7.27 Annual vehicle accessibility after activating CS1 at SCL (b), ITCS1

7.5.2.3 Evaluation of the changes in average density

Figure 7.28 and Figure 7.29 show the effect of ITCS 1 on average density for SCL (a) and (b) respectively. The data show that the traffic density was maintained after using ITCS1. All sub-scenarios of SCL (a) produced a lower density than the base line condition as shown in Figure 7.28. There were no large differences between the traffic density in the base line condition and the traffic density under ITCS1. The average base line density was 34 veh/km, which was either reduced or similar after using ITCS1 due to increased speed and flow. ITCS1 was able either to maintain the traffic density with the same original value or to improve it, despite increasing both speed and flow by 7-8%. There was a slight increase in traffic density over the original value for CS1 at SCL (b) due to unbalanced increments in the flow and speed when it was activated.

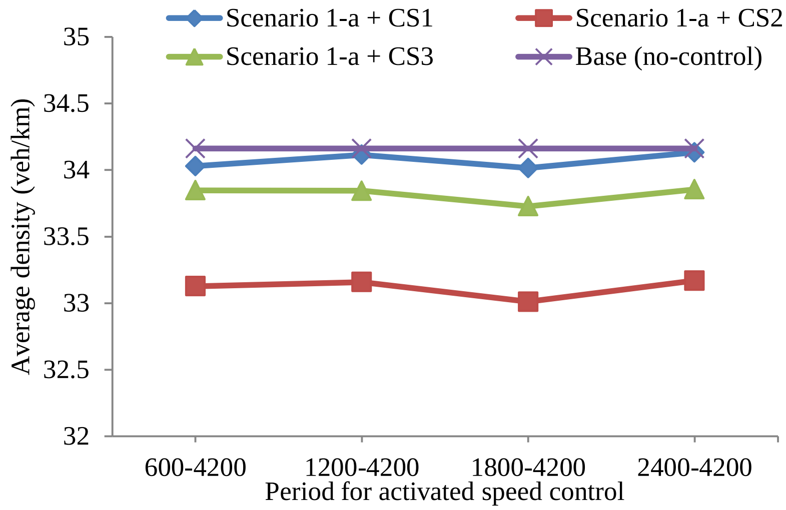


Figure 7.28 Average density before and after applying Scenario1, SCL (a)

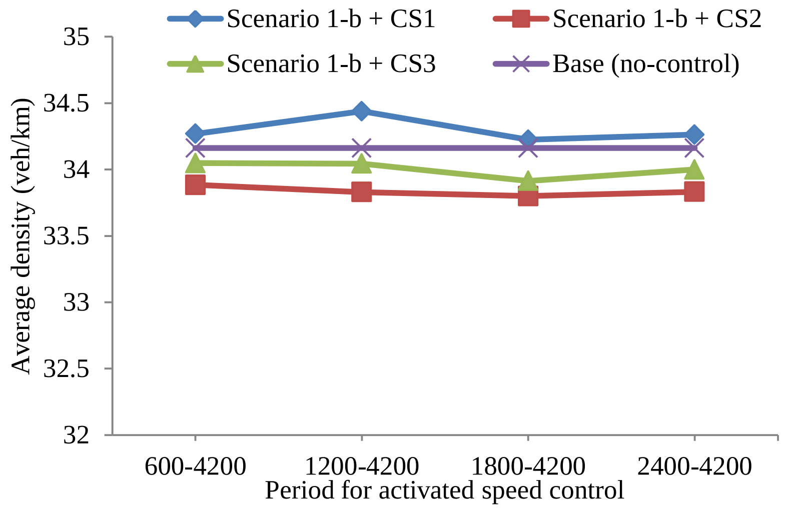


Figure 7.29 Average density before and after applying Scenario1, SCL (b)

Figure 7.30 and Figure 7.31 show the effect of ITCS 1 on average density for SCL (a) and SCL (b) respectively. The maximum effect achieved by SCL (a) for ITCS1 was 3%. These findings were consistent with VSL findings and led to the conclusion that traffic density was not enhanced by using either VSL applications or ITCS1. This was considered to be due to the high traffic volume during the peak period. Table 7.11 summarises the results of ITCS 1 for speed, flow, and density. The maximum improvement was 9% in speed for SCL (a) with a modest improvement of 1% in density for SCL (b).

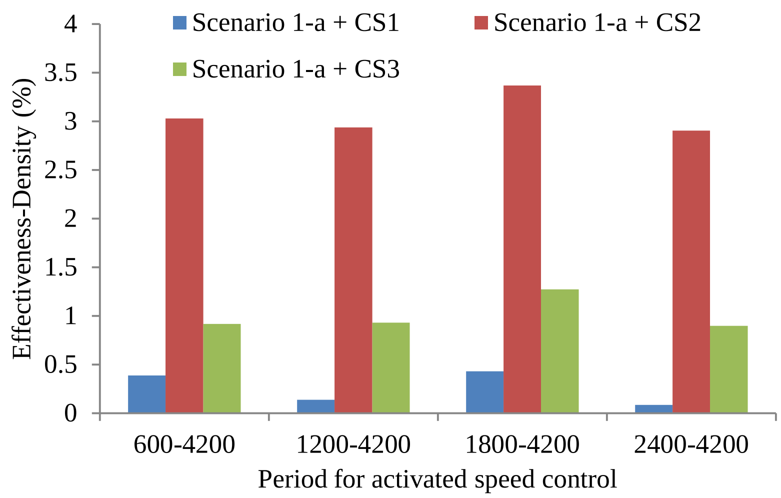


Figure 7.30 Efficiency of ITCS 1, SCL (a) on average density

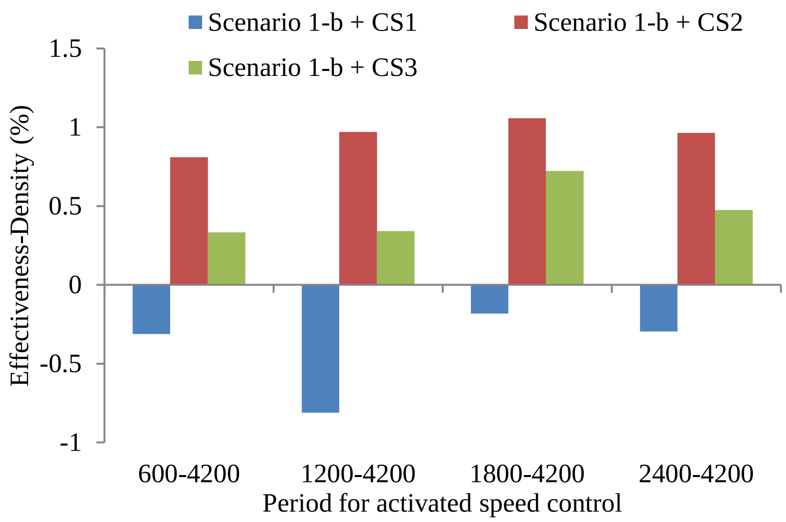


Figure 7.31 Efficiency of ITCS 1, SCL (b) on average density

Table 7.11 Maximum efficiency of ITCS 1 regarding link parameters

ITCS 1	Efficiency		
	Average speed %	Average flow %	Average density %
SCL (a)	9	7	3
SCL (b)	8	7	1

7.5.3 Evaluation of ITCS 2 on the performance of intersection 2

The following sections, evaluate the traffic characteristics of intersection 2 with respect to ITCS2. The evaluation was based mainly on the variation in results between the base condition and ITCS 2.

7.5.3.1 Evaluation of the variation in average queue length

Figure 7.32 and Figure 7.33 show the influence of ITCS 2 on the average queue length before and after applying SCL (a) and (b) respectively. The initial queue length was 44m. It decreased considerably after applying SCL (a) to 31-34m at control speed locations CS1 and CS2 for an activation time of 600 seconds and 1200 seconds respectively. The reduction in queue length was primarily due to the new cycle length setting for the WB lane signal at intersection 2. Increasing the signal timing by 20 seconds resulted in a considerable reduction in the queue length. The queue profiles show that CS1 provided better traffic movement than other control speed locations because it yielded a lower queue length. Activating the control speed at 600s-4200s gave a longer time to harmonise the upstream traffic, hence inducing smoother flow to correspond with the new timing threshold at the downstream intersection. An inverse relationship was found between the location of the control speed zone and the queue length as shown in Figure 7.34. This means the queue length became long when the position of speed control was close to the source of congestion. The queue length decreased after applying SCL (b) because the speed control limits changed from (30-35) km/hr to (40-45) km/hr. Reducing the speed limit caused overflow queues due to insufficient cycle time or capacity when the increased demand arrived.

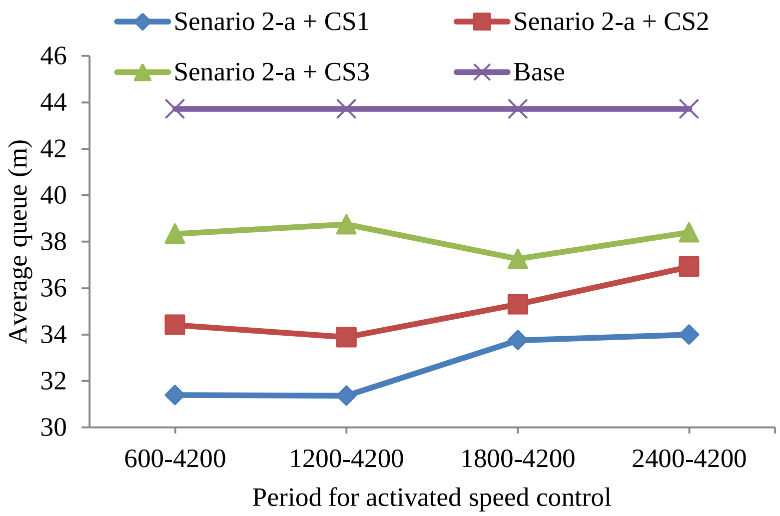


Figure 7.32 Average queue length before and after applying ITCS 2, SCL (a)

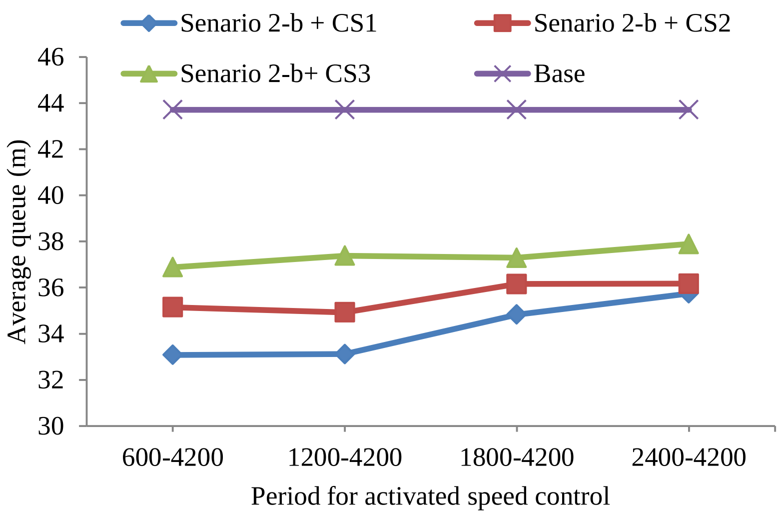


Figure 7.33 Average queue length before and after applying ITCS 2, SCL (b)

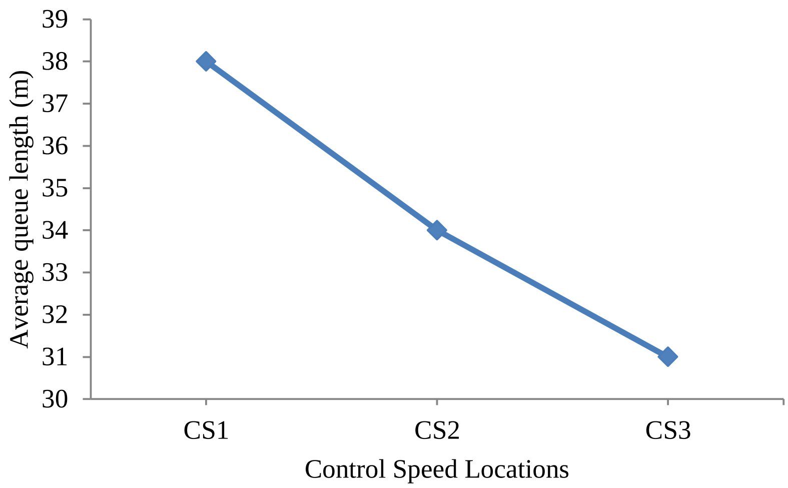


Figure 7.34 The relationship between control speed locations and average queue

The efficiency of ITCS 2 is shown in Figure 7.35 and Figure 7.36. The maximum efficiency was 28% for SCL (a) and 24% for SCL (b) while the minimum efficiency was 13% for both SCLs setting. ITCS 2 managed the traffic flow better than ITCS 1 by 8%.

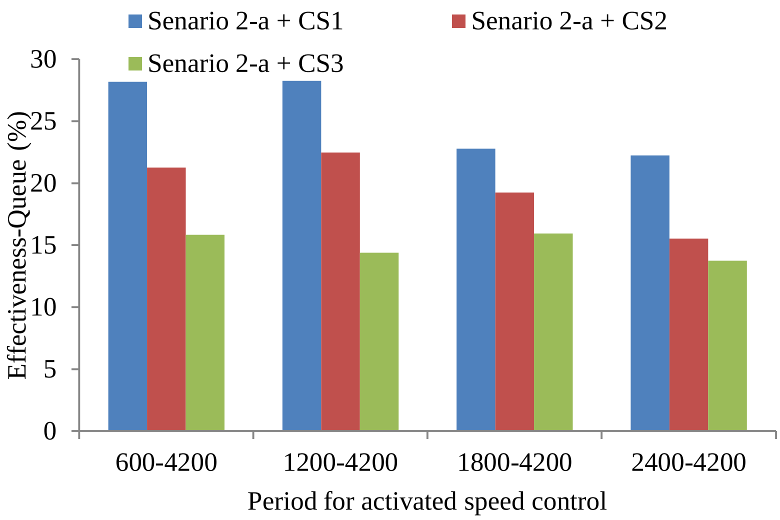


Figure 7.35 Efficiency of ITCS 2, SCL (a) on average queue

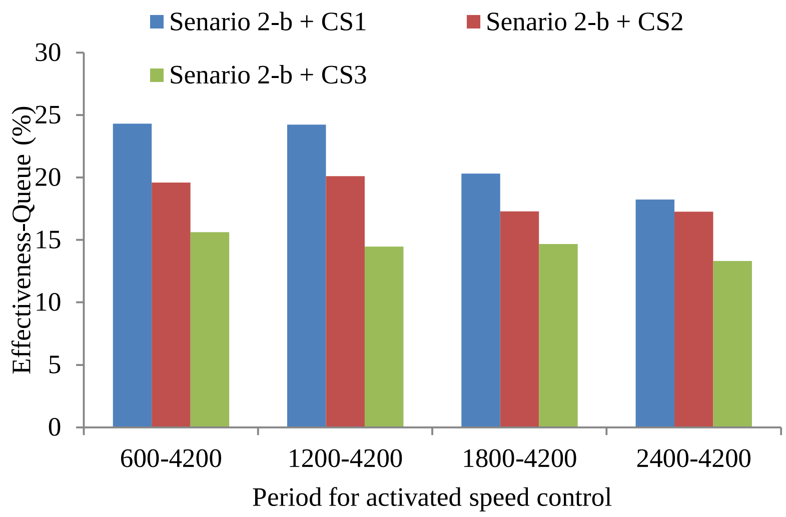


Figure 7.36 Efficiency of ITCS 2, SCL (b) on average queue

7.5.3.2 Evaluation of the variation in average delay

Figure 7.37 and Figure 7.38 show the trend of delay before and after using ITCS 2 SCL (a) and SCL (b), respectively. The initial delay was 55 seconds and declined significantly after applying ITCS2. SCL (a) at CS1 produced the highest level of improvement for this parameter when SCL (a) was activated for 600s-4200s and 1200s-4200s. Up to 16 seconds was saved by activating SCL (a) at CS1. The LOS improved at this intersection from LOS E to LOS D.

The time saving was due to a reduction in queue length during the same period and there was a better match between CS1 and the new signal timing at the downstream intersection. SCL (b) caused up to 3 seconds more delay than the delay caused by SCL (a). The reduced delay due to VSL was 15% and reduced further after using ITCS2 to 29%. The mean delay reduced from 46 seconds per vehicle using VSL to 39 seconds by ITCS2.

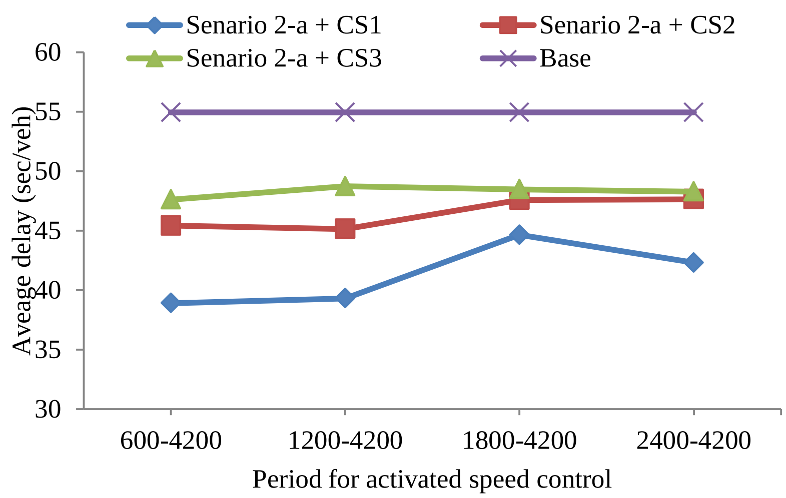


Figure 7.37 Average delay before and after applying ITCS 2, SCL (a)

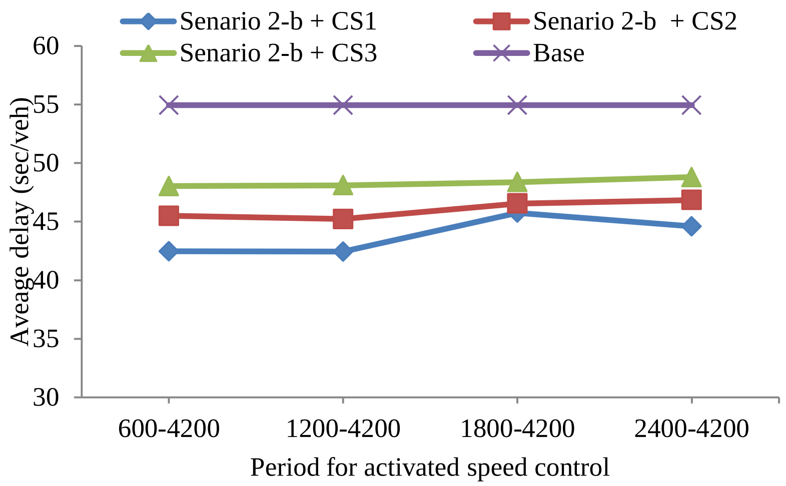


Figure 7.38 Average delay before and after applying ITCS 2, SCL (b)

Figure 7.39 and Figure 7.40 show the effect of ITCS 2 on the average delay at intersection 2. SCL (a) reduced the delay 29% and SCL (b) reduced delay by 23% which is 6% less than for SCL (a). The minimum improvement by ITCS 2 was 12%.

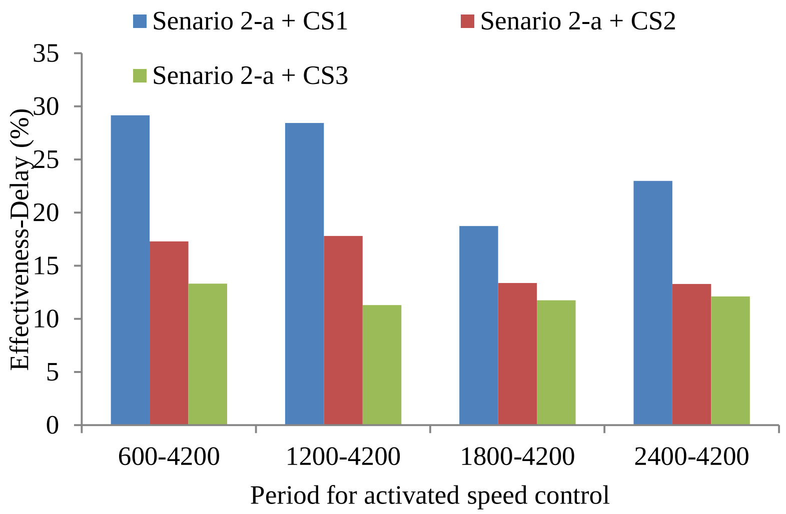


Figure 7.39 Efficiency of ITCS 2, SCL (a) on average delay

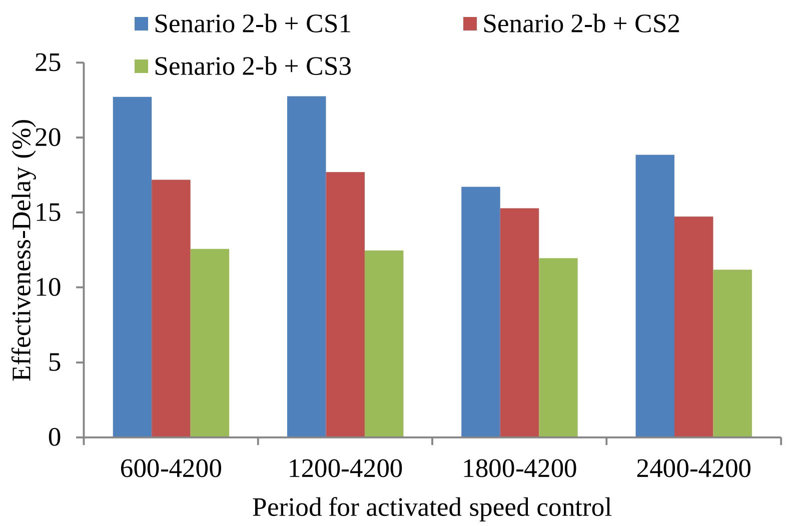


Figure 7.40 Efficiency of ITCS 2, SCL (b) on average delay

This maximum improvement (29%) reduced annual delay time from 4,884 to 3,723 hours per year despite increasing the intersection throughput by 7% more than the base condition in the evening peak hour. The reduction in delay saved a combined 1,161 hours per year, which had a value of approximately $12,307. Table 7.12 summarises the total annual savings of reduced delay after activation CS1, CS2 and CS3 of ITCS2, SCL (a).

Table 7.12 Annual delay time savings cost after applying ITCS2, SCL (a)

Base condition delay (sec/veh)	Base condition Annual delay (hour/year)	ITCS2, SCL(a)	Controlled delay (sec/veh)	Annual controlled delay (hour/year)	Annual delay time saving (hour/year)	Annual delay time saving cost $(AUD)
55	4,884	CS1	39	3,723	1,161	12,307
		CS2	45	4,296	588	6,233
		CS3	48	4,582	302	3,201

7.5.3.2.1 Statistical significant of delay results

The variance in standard deviation for ITCS2 was lower than base condition as indicated in Table 7.13. ITCS2 showed a significant impact on vehicle delay at traffic signal during peak hour because the significant value for ANOVA was less than the confidence interval (5%), (0.000 < 0.05) as shown in Table 7.14.

Table 7.13 Delay statistical descriptive

Status	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base condition	30	54.94	9.04	1.65	51.56	58.32	35.86	72.14
ITCS2	30	38.91	7.00	1.27	36.29	41.53	24.30	47.80
Total	60	46.93	11.38	1.47	43.98	49.87	24.30	72.14

Table 7.14 Delay, One-Way ANOVA

Status	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	3856.08	1	3856.08	58.91	.000
Within Groups	3796.08	58	65.45		
Total	7652.16	59			

7.5.3.3 Evaluation of the variation in average stopped delay

Figure 7.41 and Figure 7.42 show the effect of ITCS 2 on average stopped delay at intersection 2. The initial average stopped delay was 45 seconds. Similar to the average delay indicator, stopped delay changed considerably by using SCL (a) of ITCS 2. The average stopped delay declined from 45 seconds (non-control) to 31 seconds when SCL (a) was implemented at CS1 and launched 600s-4200s after at the start of congestion and declined to 34 seconds when SCL (b) was implemented at CS1 and launched at the same period.

The maximum reduction in stopped delay was achieved using SCL (a) because it produced a reduced stopped delay. These results led to the conclusion that reducing the speed limit increased arrival flow thereby the stopped delay was increased accordingly. Positions CS2 and CS3 achieved a lower reduction than CS1 for the average stopped parameter because they were located further away from the downstream intersection. The main effect of ITCS 2 was due to the location of the control speed zone and secondly due to the activation time period. Increasing the speed control limits contributed to increasing the stopped delay because it increased the number of vehicles and the queue length at the downstream intersection.

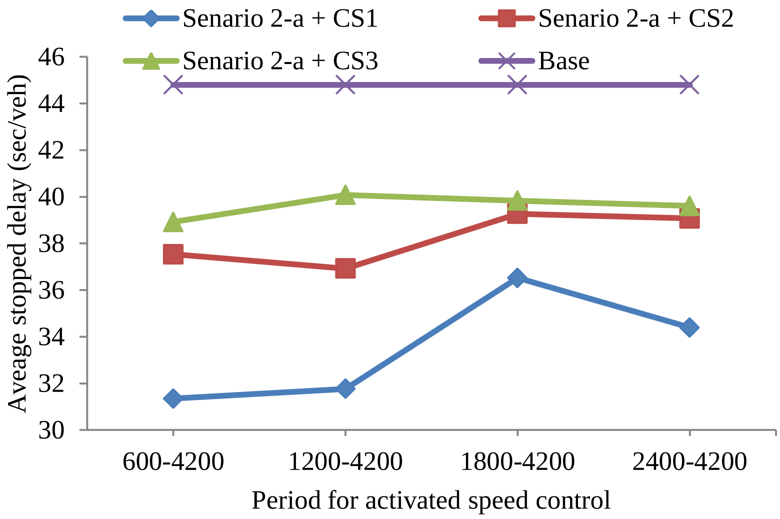


Figure 7.41 Average stopped delay before and after applying ITCS 2, SCL (a)

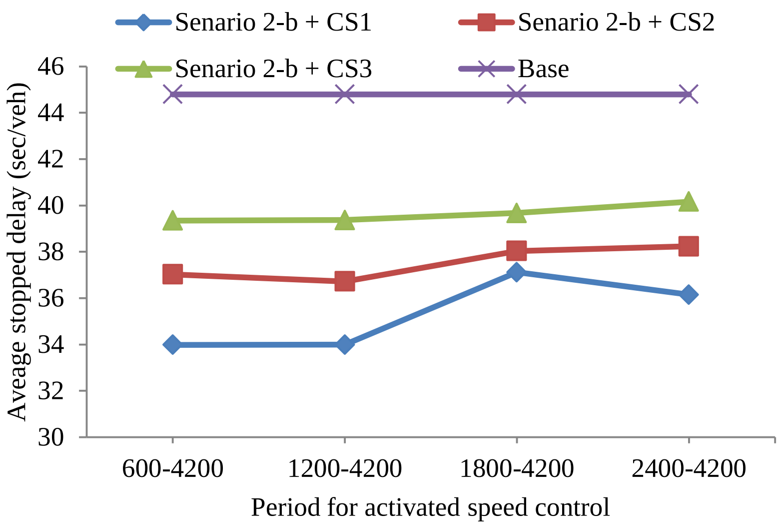


Figure 7.42 Average stopped delay before and after applying ITCS 2, SCL (b)

Figure 7.43 and Figure 7.44 show the effect of ITCS 2 on average stopped delay at intersection 2. A maximum improvement of 30% was achieved by SCL (a). SCL (b) achieved 24% improvement due to increasing the speed limit ranges from 30-35 to 40-45 km/hr.

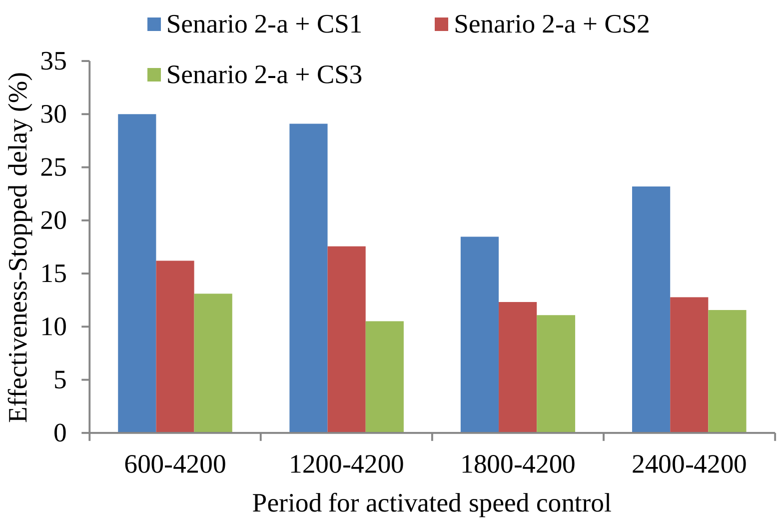


Figure 7.43 Efficiency of ITCS 2, SCL (a) on average stopped delay

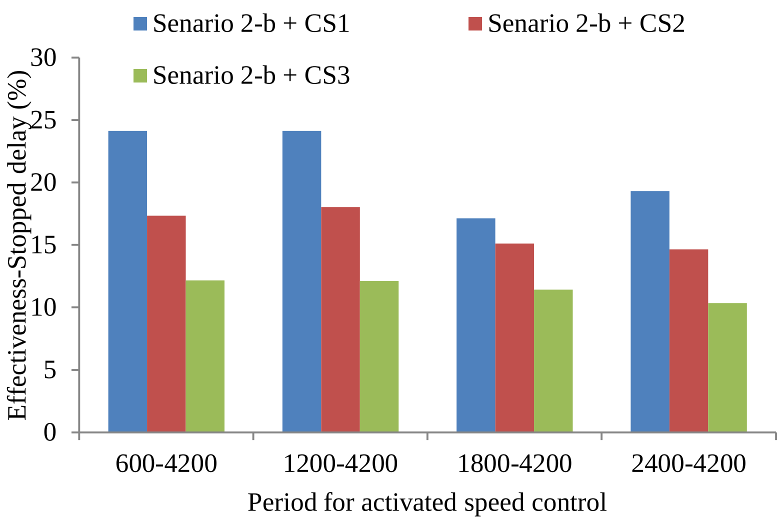


Figure 7.44 Efficiency of ITCS 2, SCL (b) on average stopped delay

7.5.3.4 Evaluation of the variation in average number of stops

Figure 7.45 and Figure 7.46 show the effect of ITCS 2 on the average number of stops at intersection 2. The initial number of stops was 1.16 and this decreased dramatically after applying SCL (a) and (b). Activating SCL (a) at CS1 decreased the number of stops to 0.86 while activating SCL (b) at the same control speed location reduced the number of stops to 0.95. The difference was caused by increasing the speed control limits. Minimising the number and frequency of stops reduced the acceleration and deceleration of vehicles which turn in reduced braking. These factors assisted in improving the traffic safety in the vicinity of signalised intersections during the congestion period. ITCS 2 improved this parameter more than ITCS 1 due to the reduction in queue length at intersection 2. All sub-scenarios of ITCS 2 showed a positive improvement in reducing the number of stops which resulted in enhancing the traffic safety.

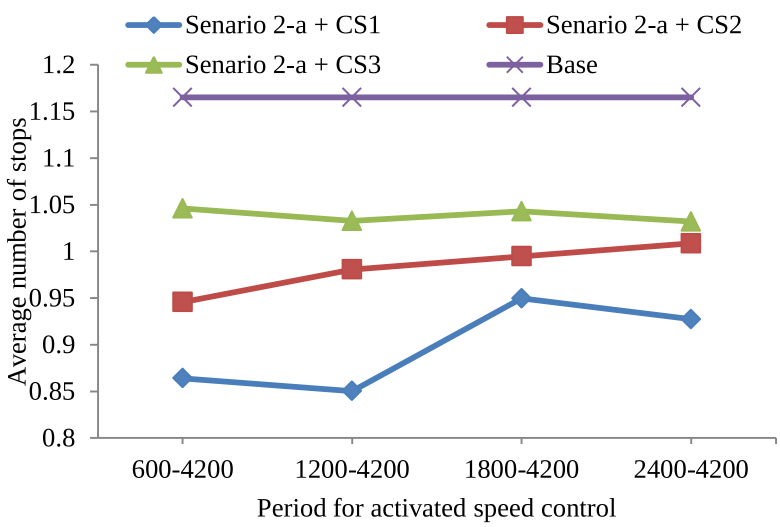


Figure 7.45 Average number of stops before and after applying ITCS 2, SCL (a)

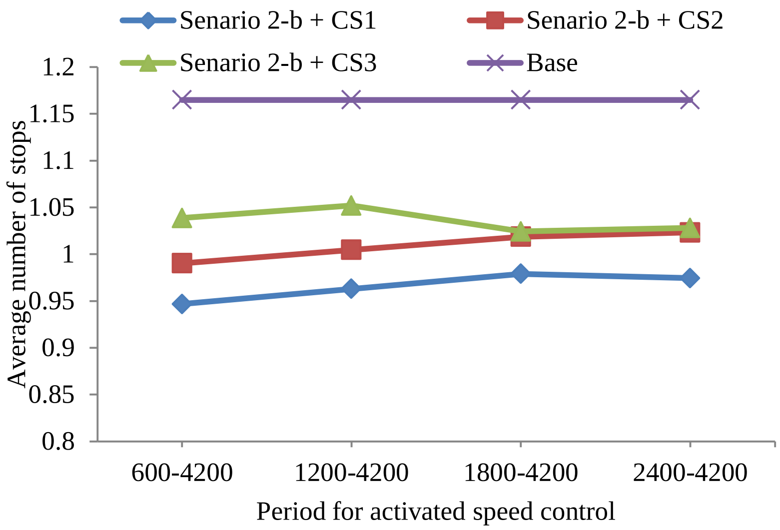


Figure 7.46 Average number of stops before and after applying ITCS 2, SCL (b)

Figure 7.47 and Figure 7.48 show the efficiency of ITCS 2 on the average number of stops parameter at intersection 2. The maximum improvement was 27% due to SCL (a) and approximately 19% due to apply SCL (b). Table 7.15 shows the effectiveness of ITCS 2 on the performance of intersection 2 in terms of queue length, delay and stopped delay, and number of stops. SCL (a) achieved the highest traffic management efficiency at the intersection 2.

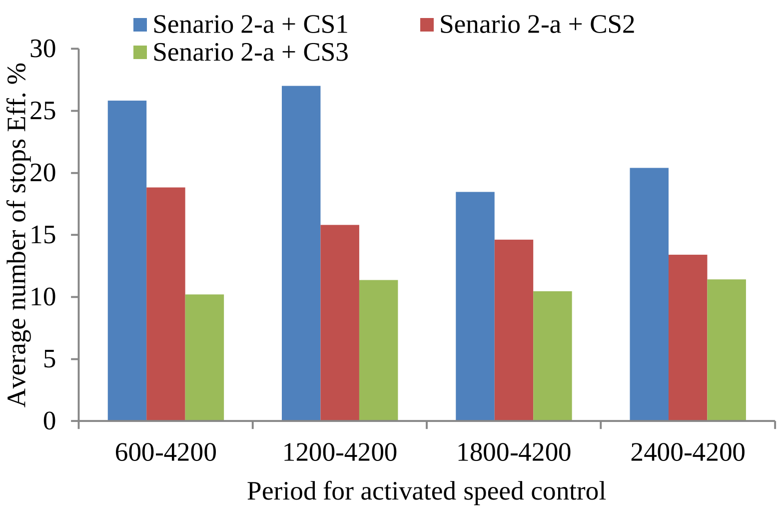


Figure 7.47 Efficiency of ITCS 2, SCL (a) on average number of stops

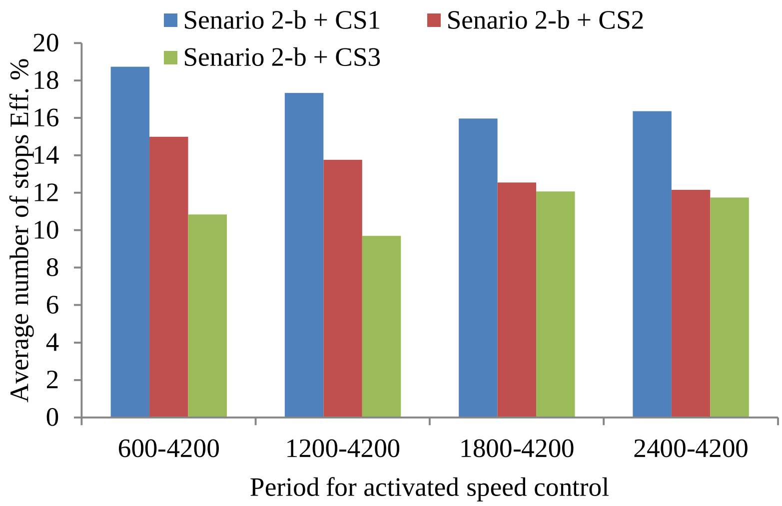


Figure 7.48 Efficiency of ITCS 2, SCL (b) on average number of stops

Table 7.15 Maximum efficiency of ITCS 2 for intersection parameters

Efficiency of ITCS 2	Average queue %	Average delay %	Average stopped delay %	Average number of stops %
SCL (a)	28	29	30	27
SCL (b)	24	23	24	19

7.5.4 Evaluation of ITCS 2 on the performance of traffic flow parameters

In the following sections, macroscopic traffic flow characteristics of speed, flow and density for the WB link at intersection 2 were estimated to investigate the effect of ITCS 2 before and after applying ITCS2.

7.5.4.1 Evaluation the changes in average speed

Figure 7.49 and Figure 7.50 show the effect of ITCS 2 on average speed for SCL (a) and (b) respectively. The initial speed was 39 km/hr and rose to 43 km/hr after activating CS1 for SCL (a) for 600s-4200s. This produced only a small variation in the speed profiles of all sub-scenarios. The flow was nearly constant and of a uniform distribution because the analysis was carried out during the peak period. Increasing the timing of the signal by 20s improved vehicles speed to 43 km/hr once SCL (a) was implemented at CS1 and launched for 600s-4200s. The variation in results for SCL (a) and SCL (b) was not significant (less than 1%) because both SCLs only improved the speed of vehicles up to 43 km/hr.

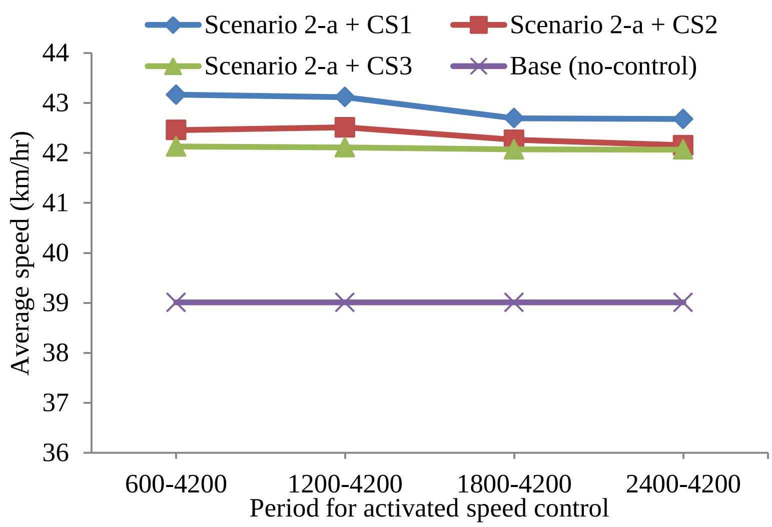


Figure 7.49 Average speed before and after applying ITCS 2, SCL (a)

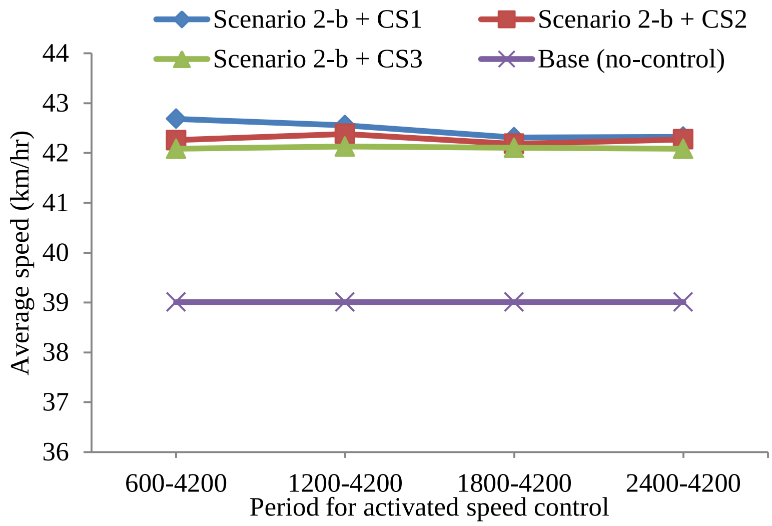


Figure 7.50 Average speed before and after applying ITCS 2, SCL (b)

Figure 7.51 and Figure 7.52 show the improvement due to ITCS 2 on the average speed for SCL (a) and (b) respectively. SCL (a) was approximately 8% to 11% better while SCL (b) was approximately 8% to 10% better. This variation in improvement was due to the difference in speed control range. In general, the improvement increased from 8% to 9% for almost all control speed locations and for all speed control periods. These results showed that using ITCS2 during the peak period created a limited improvement in speed for all sub-scenarios.

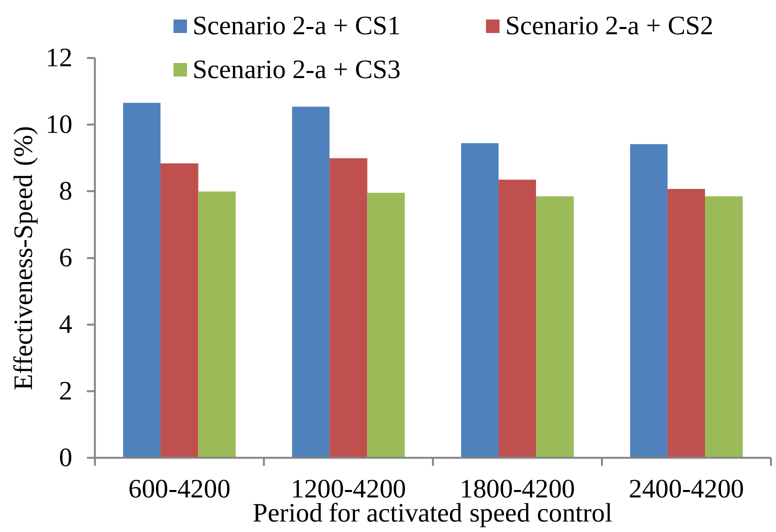


Figure 7.51 Efficiency of ITCS 2, SCL (a) on average speed

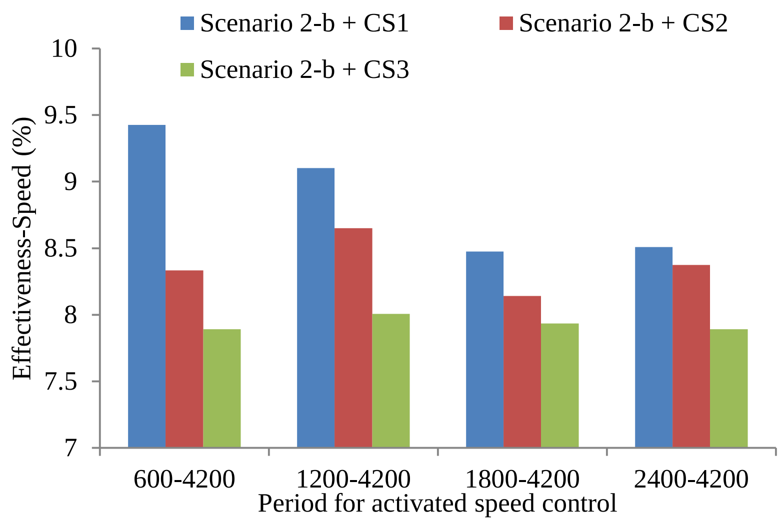


Figure 7.52 Efficiency of ITCS 2, SCL (b) on average speed

7.5.4.1.1 Evaluation of savings in crash costs on the GAR using ITCS2

ITCS2 reduced variation in the speed of vehicles travelling along the congested link. This reduced the probability of fatality during the congestion period. Reducing speed variation by 4.16 km/hr during congestion periods increased speed homogenity and decreased the probability of fatality by 1.56%. Table 7.16 shows that the probability of fatality for the current traffic situation decreased from 3.6% to 2.04% after employing ITCS2 and improved the crash costs by 1.56%. Table 7.17 shows that if ITCS2 was applied on the GAR, the total annual saving in fatality relate crash costs would be $95,118.

Table 7.16 Probability of fatality at GAR before and after using ITCS2

Status	Average speed (km/hr)	Posted speed (km/hr)	Speed deviation (km/hr)	Probability of Fatality* (%)	Improve (%)
Base condition	39	70	31	3.6	1.56
ITCS2	43.16	70	26.84	2.04	

Probability of fatality= $(\text{Speed devaition}/71)^4$

Table 7.17 Total annual savings in crash costs a GAR after applying ITCS2

Year	Fatal	Hospitalisation	Medical treatment & minor injury	Property damage
2006	0	11	31	16
2007	1	14	19	9
2008	1	13	25	19
2009	1	14	27	17
2010	0	7	30	13
Average	0.6	11.8	26.4	14.8
Improve 1.56%	0.00936	0.18408	0.41184	0.23088
Cost/crash ($AUD)	3,083,000	307,500	17,000	11,500
Savings/year ($AUD)	28,857	56,605	7,001	2,655
Total annual savings ($AUD)		95,118		

7.5.4.1.2 Statistical significant of speed results

The statistical variance of speed results under the base condition and ITCS2 were non-significant for parts 1 and 2 as illustrated in Table 7.18 and Table 7.20. The ITCS2 showed significant impact on vehicle speed results regardless of whether the part was close to or away from the signalised intersection area. Table 7.19 and Table 7.21 show that ITCS2 had a great significant impact on vehicle speed because the significant value for ANOVA was less than the confidence interval (5%), (.000 <.05) for both parts.

Table 7.18 Speed statistical descriptive for part 1

Status	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base condition	30	12.04	1.83	.33	11.36	12.73	8.90	16.35
ITCS2	30	15.88	3.17	.57	14.69	17.06	11.53	22.53
Total	60	13.96	3.21	.41	13.13	14.79	8.90	22.53

Table 7.19 Speed, One-Way ANOVA for part 1

Status	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	220.55	1	220.55	32.80	.000
Within Groups	389.95	58	6.72		
Total	610.51	59			

Table 7.20 Speed statistical descriptive for part 2

Status	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean		Minimum	Maximum
					Lower Bound	Upper Bound		
Base condition	30	65.96	.89	.16	65.63	66.30	64.32	68.22
ITCS2	30	70.44	.72	.13	70.17	70.71	69.51	71.97
Total	60	68.20	2.39	.30	67.58	68.82	64.32	71.97

Table 7.21 Speed, One-Way ANOVA for part 2

Status	Sum of Squares	df	Mean Square	F	Sig.
Between Groups	301.26	1	301.26	456.16	.000
Within Groups	38.30	58	.66		
Total	339.57	59			

7.5.4.2 Evaluation of the changes in flow

Figure 7.53 and Figure 7.54 show the effect of ITCS 2 on average flow for SCL (a) and (b) respectively. The initial flow was 1332 veh/hr and this flow increased after applying ITCS2. The variation in flow between all sub-scenarios was nearly constant except for the flow produced by activating CS2 for SCL (a). The maximum flow reached 1432 veh/hr for SCL (a) and (b). Changing the control speed locations did not make a big difference in throughput at intersection 2 despite increasing the signal timing for the WB lanes by 20 seconds. The reason was due to limited improvement in the traffic density during the peak simulation period.

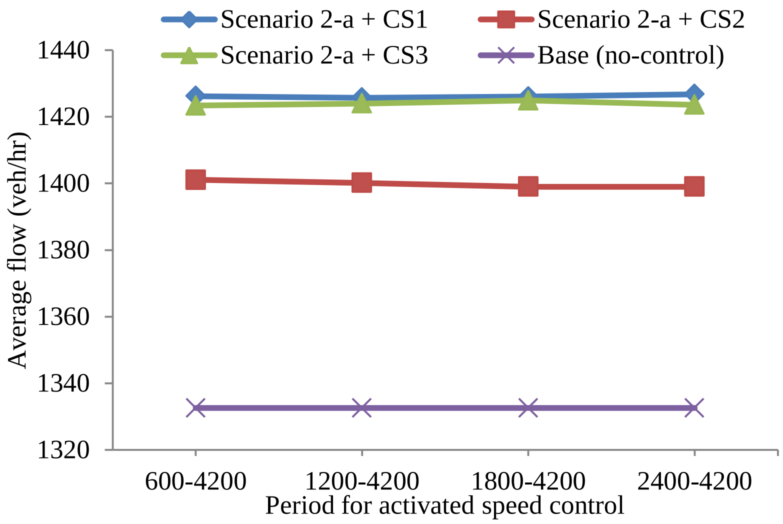


Figure 7.53 Average flow before and after applying ITCS 2, SCL (a)

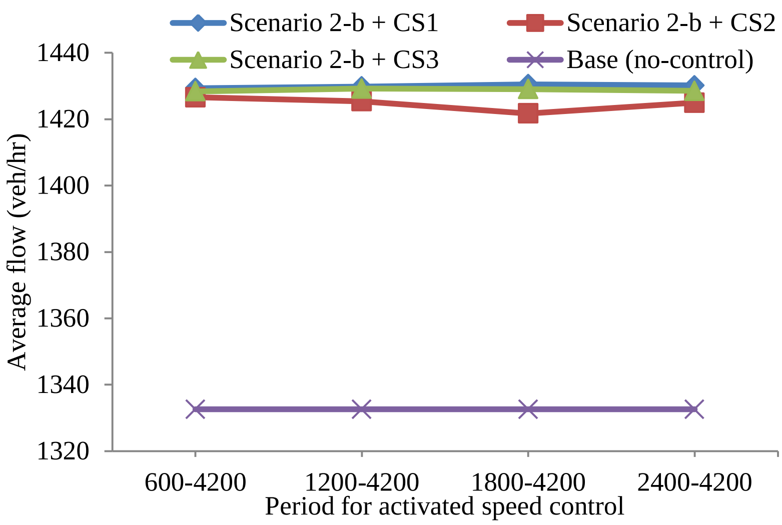


Figure 7.54 Average flow before and after applying ITCS 2, SCL (b)

Figure 7.55 and Figure 7.56 show the efficiency of ITCS 2 on average flow for SCL (a) and (b) respectively. The efficiency of ITCS 2 was an almost constant 7% over all sub-scenarios. By comparison with ITCS 2 and VSL applications, there were no differences in the maximum efficiency of the flow between all traffic management strategies. Scenario 2 (ITCS2) improved vehicle accessibility by about 24,000 vehicles per year. The improvement in vehicle accessibility was nearly constant for VSL, ITCS1 and ITCS2 during the peak period.

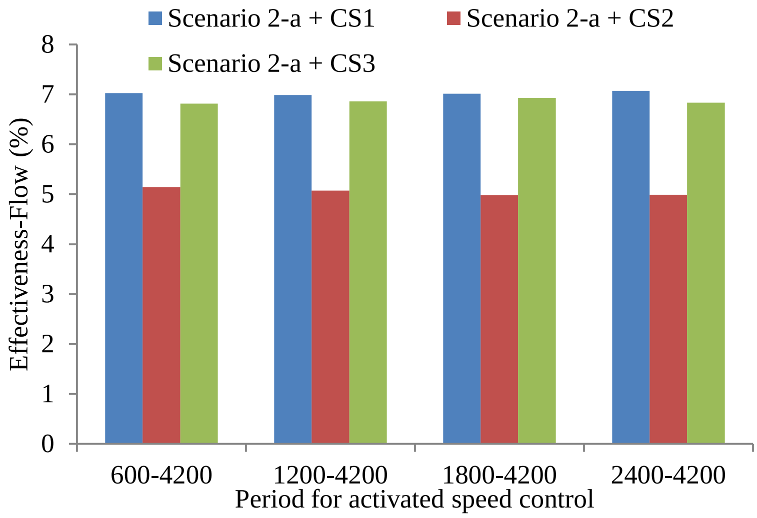


Figure 7.55 Efficiency of ITCS 2, SCL (a) on average flow

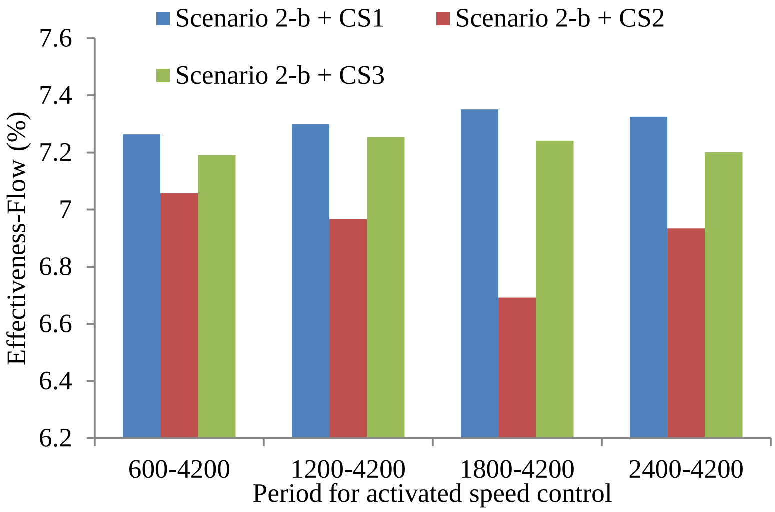


Figure 7.56 Efficiency of ITCS 2, SCL (b) on average flow

7.5.4.3 Evaluation of the changes in density

Figure 7.57 and Figure 7.58 show the effect of ITCS 2 on average density for SCL (a) and (b) respectively. There was hardly small variation in traffic density before and after using ITCS 2 due to a constant improvement in speed and flow. The initial density was 34 veh/km which only dropped to 33 veh/km. This supported the finding that ITCS was able to maintain the traffic density while maximising flow and speed along the approach link at intersection 2.

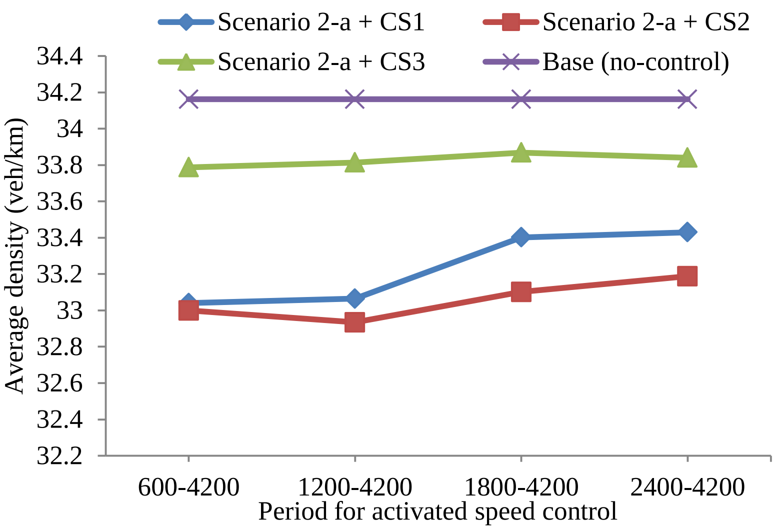


Figure 7.57 Average density before and after applying ITCS 2, SCL (a)

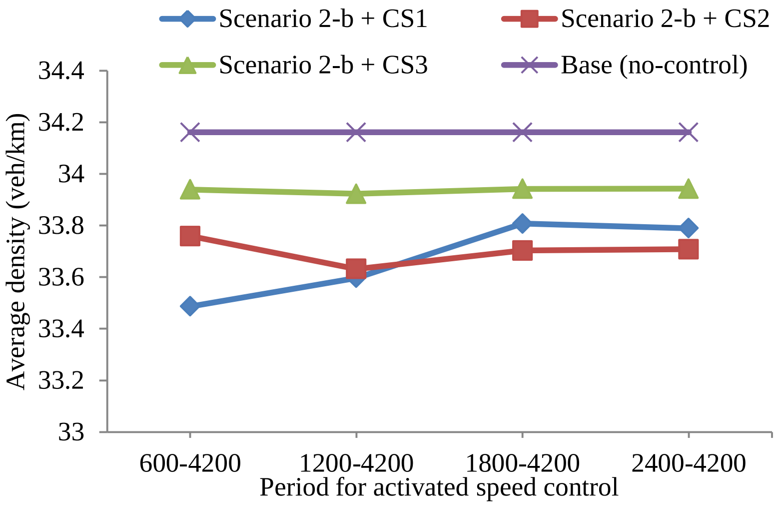


Figure 7.58 Average density before and after applying ITCS 2, SCL (b)

Figure 7.59 and Figure 7.60 show the efficiency of ITCS 2 on average density for SCL (a) and (b) respectively. The maximum efficiency was 3.5% for SCL (a) and 1% for SCL (b). The efficiency of SCL (b) was less than the efficiency of SCL (a) because the speed control limits were changed from 30-35 km/hr to 40-45 km/hr. This change increased the flow of vehicle arrivals and reduced the improvement due to increased traffic density.

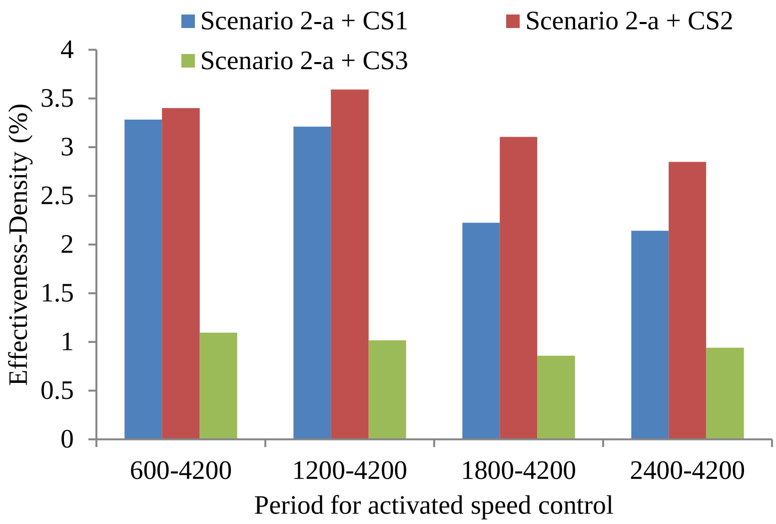


Figure 7.59 Efficiency of ITCS 2, SCL (a) on average density

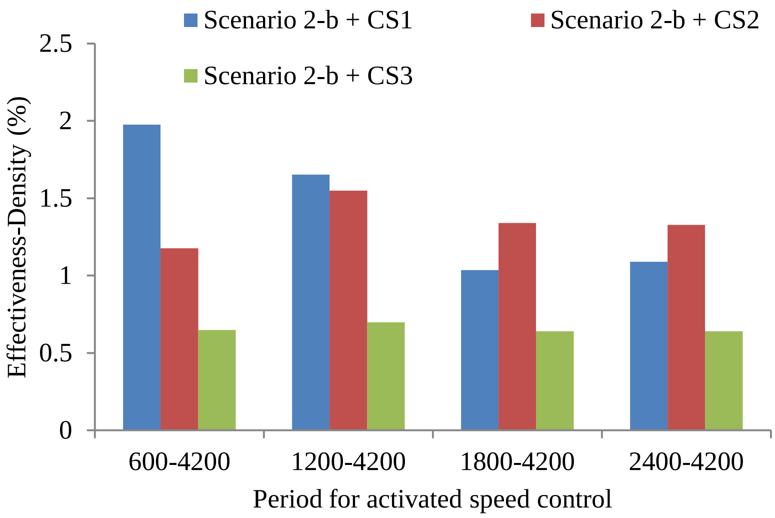


Figure 7.60 Efficiency of ITCS 2, SCL (b) on average density

Table 7.22 summarises the maximum efficiency of ITCS 2 on macroscopic traffic flow parameters. The maximum improvement in speed was 10%, 7% for flow and 3.5 % for density. There is little difference between the data in Table 7.22 and Table 7.11.

Table 7.22 Maximum efficiency of ITCS 2 regarding link parameters

Scenario 2 (ITCS2)	Maximum Efficiency %		
	Average speed	Average flow	Average density
SCL (a)	11	7	3.5
SCL (b)	9	7	2

7.6 Comparison of VSL applications and ITCS scenarios

This section discusses current (July 2013) traffic management on the GAR during the evening peak period. A comparison was made between VSL alone and ITCS on the flow situation to investigate which traffic management strategy was most effective.

Four traffic parameters representing the main properties of the signalised intersections were examined using both strategies. Both strategies produced significant improvements in the performance of intersection 2 when contrasted with baseline conditions during the congestion period. VSL achieved a 16% improvement in average queue length while ITCS1 and ITCS2 achieved 20% and 28% improvements respectively. ITCS1 produced a 4% increase over VSL and ITCS2 resulting in nearly double the improvement of VSL.

Both VSL and ITCS reduced the average delay at intersection 2. The average delay was 55 seconds for the base line conditions and reduced to 46, 44.5 and 39 seconds after applying VSL, ITCS1 and ITCS2 respectively. The LOS E for the baseline condition improved to LOS D after implementing VSL and ITCS. The LOS D for VSL control was similar to the LOS of ITCS despite there being 11 seconds difference between both strategies. This was due to the standard specification for motor vehicle LOS thresholds by TRB (2010). The maximum reduction in delay time was 15% after using VSL control. After integrating VSL application, the reduction in delay time increased to 19% and 29% after employing ITCS1 and ITCS2 respectively.

All the proposed traffic management strategies were able to improve the level of service at intersection 2 from LOS E to LOS D. Improving traffic conditions began with the application of VSLs, continued with ITCS1, and reached a maximum improvement with ITCS2. Figure 7.61 shows the variation in the percentage of efficiency for three traffic scenarios, VSLs, ITCS1, and ITCS2 with four traffic parameters at intersection 2. ITCS2 produced the highest improvement in traffic management, ITCS1 produced the second most improvement, and VSLs produced the third most improvement. This shows that, VSLs can produce high levels of service at intersection 2 if they are combined with other traffic scenarios (ITCS 1 and ITCS 2). Table 7.23 summarises the maximum improvements achieved by VSL and ITCS strategies.

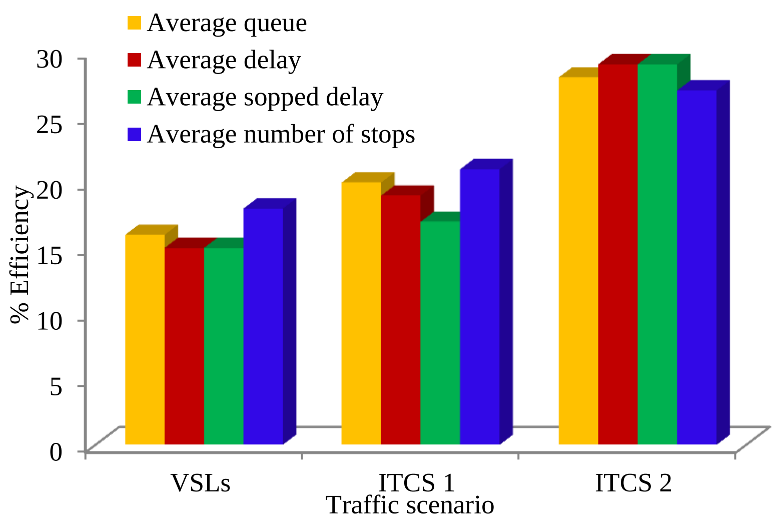


Figure 7.61 Optimum efficiency of various traffic scenarios

Table 7.23 Maximum improvement due to VSL application and ITCS

Intersection parameters	% Efficiency		
	VSLs	ITCS1	ITCS2
Average queue	16	20	28
Average delay	15	19	29
Average stopped delay	15	17	29
Average number of stops	18	21	27

The annual delay time for base conditions amounted to 4,884 hours computed in evening peak hour. VSL, ITCS1 and ITCS2 reduced the annual delay time to 4,391, 4248 and 3,723 hours respectively despite the intersection throughput increasing from 1,332 to 1,432 veh/hr. The reduction in delay by using VSL, ITCS1 and ITCS2 could save 493, 636 and 1,161 hours per year respectively for vehicles travelling on the WB of intersection 2. Since the value of vehicle delay time in Brisbane during peak hour is \$10.6 (Booz, 2003) this amounts to a savings of approximately \$5,226, \$6,742 and \$12,307. Table 7.24 shows the annual savings by reducing delay time after employing VSL and ITCS for all vehicles travelling on WB link of intersection 2.

Table 7.24 The annual savings in delay time for VSL and ITCS

Base condition		Control strategy	Max. delay reduction (%)	Annual delay time (hour)	Annual delay time cost ($AUD)	Annual delay time saving (hour)	Annual delay time savings ($AUD)
Annual delay time (hour)	Annual delay time cost ($AUD)						
4,884	51,770	VSL	15	4,391	46,545	493	5,226
		ITCS1	19	4,248	45,029	636	6,742
		ITCS2	29	3,723	39,464	1,161	12,307

In addition, all VSL ranges and ITCS scenarios enhanced traffic safety by reducing the average number of stops and this reduced the likelihood of rear-end collisions. The effect of these traffic management strategies varies from one scenario to another. VSL reduced the average number of stops by 18% and this increased to 21% and 27% after applying ITCS1 and ITCS2 respectively.

The influence of applying VSL and ITCS on macroscopic traffic parameters (speed, flow and density) for the congest link are shown in Table 7.25. All proposed scenarios improved speed, flow and density by 3-11%. ITCS1 and ITCS2 did not improve the performance parameters more than VSL except for the speed parameter (11%) in the case of ITCS2. This was due to a small improvement in traffic density (3.5%). This occurred because the link reached optimum capacity by using the VSL strategy. It was also due to the traffic analysis being carried out during the peak hour period. During this period, the space headways between vehicles were small, which resulted in increasing the interaction between vehicles (high traffic density) thereby constraining the speeds for vehicles.

Table 7.25 Maximum improvement regarding link parameters

Macroscopic parameters	% Efficiency		
	VSLs	ITCS1	ITCS2
Average speed	9	9	11
Average flow	7	7	7
Average density	3	3	3.5

ITCS was able to increase the efficiency of the signalised intersection but it was unable to optimise the efficiency of macroscopic parameters beyond what was achieved by VSL. Both VSL and ITCS increased flow by 7% and this indicated they were able to increase mobility along the GAR during the peak hour and increase the vehicle accessibility by about 24,000 vehicles per a year. Figure 7.62 shows a constant improvement in efficiency for all proposed traffic scenarios at GAR for the macroscopic parameters of speed, flow, and density.

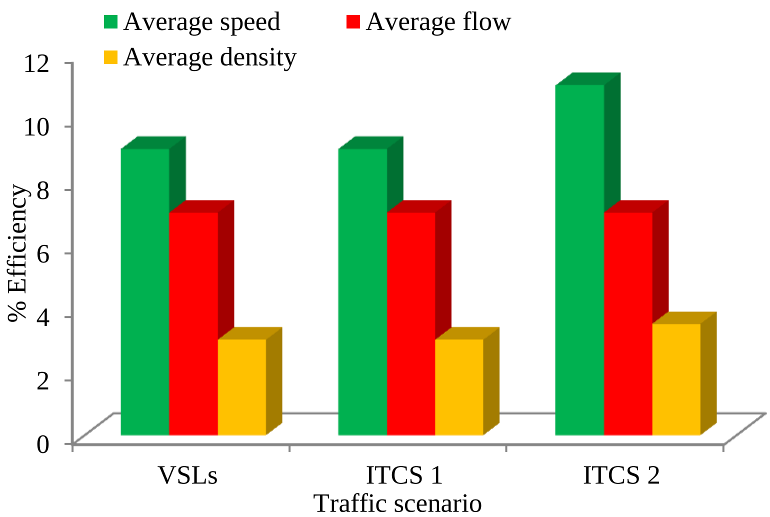


Figure 7.62 Maximum improvement at the approach link of intersection 2

Reducing the speed variations along the congested link during the peak hour reduced the probability of fatality at the GAR. VSL and ITCS improve the risk of crashes at the GAR by 1.35% and 1.56% respectively. These improvements decreased the total annual costs to all crash types on the GAR and potentially saved around $82,000 and $95,000 as shown in Table 7.26.

Table 7.26 Maximum expected annual saving from VSL and ITCS application

Status	Improve (%)	Annual saving in crash costs ($AUD)				Total annual savings ($AUD)
		Fatal	Hospitalisation	Medical treatment & minor injury	Property damage	
VSL	1.35	24,972	48,985	6,059	2,298	82,314
ITCS1	1.30	24,047	47,171	5,834	2,213	79,265
ITCS2	1.56	28,857	56,605	7,001	2,655	95,118

The research investigated the effect of VSL and ITCS during peak hours on a congested link in Brisbane city which is about 6.7 km long. The result from this study was used to estimate the total saving from application of VSL and ITCS during peak hours for Brisbane Metropolitan Area (BMA). The total length of arterial road network in Brisbane is 469 km. Out of this, about 191 km (41%) of arterial roads are congested (Department of Transport and Main Roads (TMR), 2010).
Table 7.27 shows that the total annual saving that can be expected from reducing crashes in the congested arterials in Brisbane will be around $2,347,000, $2,260,000 and $ 2,712,000 due to the use of VSL, ITCS1 and ITCS2 respectively.

Table 7.27 Total annual savings in crashes for BMA

Severity of crash	Annual saving in crash costs ($AUD)		
	VSL	ITCS1	ITCS2
Fatal	711,888	685,519	822,640
Hospitalisation	1,396,438	1,344,726	1,613,665
Medical treatment & minor injury	172,727	166,313	199,581
Property damage	65,510	63,087	75,687
Total annual savings ($AUD	2,346,563	2,259,644	2,711,573

7.7 Influence of ITCS on total travel time

The effect of ITCS on total travel time (TTT) was investigated along the WB lanes of GAR (6.7km). Speed control limits were used in evaluating total travel time. A comparison between base line conditions (no-control) and ITCS scenarios was made to find the effect of ITCS on total travel time.

Figure 7.63 shows that the base line conditions had higher TTT than all ITCS scenarios. The base line total travel time was 598 seconds which reduced to 593, 572, 590, and 566 seconds by using ITCS 1 of SCL (a) and SCL (b), and ITCS 2 of SCL (a) and SCL (b) respectively. SCL (a) of each scenario recorded a TTT higher than SCL (b) because SCL (a) used a speed control limit of 30-35 km/hr and SCL (b) used a speed control limit of 40-45 km/hr. SCL (a) achieved a high reduction in delay and stopped delay at the signalised intersection but it achieved a smaller reduction in TTT.

For example during the evening peak hour, the actual mean travel time for the WB was 598 seconds with a total flow of 1,542 veh/hr. This TTT translated to 61,474 hours per a year which cost $651,629. ITCS2 generated 58,184 hours per year of TTT for one peak hour simulation which cost $616,750. A maximum savings of 32 seconds, which is about 5.3% reduction in actual TTT, may firstly appear trivial, the quantifiable benefits for a large volume of traffic can be substantial.

This 5.3% reduction in TTT amounted to 3,290 hours of travel time saved per year for all vehicles traveling on WB direction in the evening peak hour. This produced a savings of approximately $34,874. Table 7.28 indicates the variation in travel time savings for VSL and ITCS.

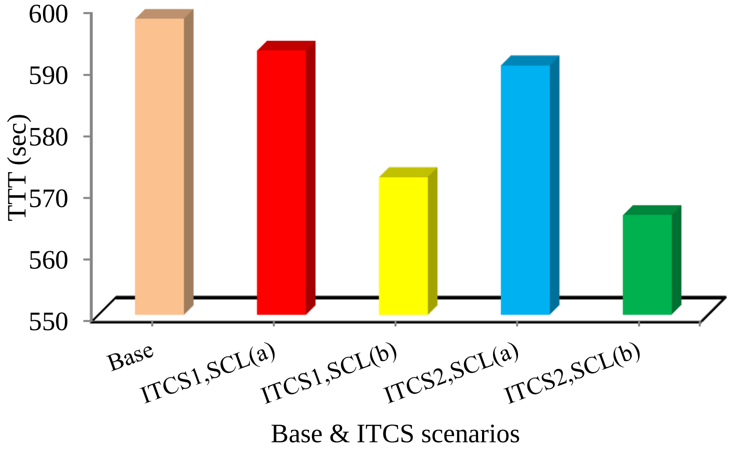


Figure 7.63 Comparison between traffic base and ITCS scenarios regarding TTT

Table 7.28 Maximum expected annual saving in TTT along GAR

Base condition Annual delay time (hour/year)	Base condition Annual TTT $(AUD)	Control strategy	% Max TTT reduction	Annual TTT saving (hours/year)	Annual TTT saving cost $(AD)
61,474	651,629	VSL	3.7	2,261	24,000
		ITCS1	4.3	2,669	28,291
		ITCS2	5.3	3,290	34,874

The outcomes of VSL and ITCS throughput this research can be extended to conclude all the congested arterials (191kms) in BMA. Table 7.29 shows that the total annual saving that can be expected from reducing TTT in the congested arterials in BMA will be around $696,000, $820,000 and $1,011,000 due to the use of VSL, ITCS1 and ITCS2 respectively.

Table 7.29 Total annual savings in TTT for BMA

Traffic control strategy	Annual TTT saving cost $(AD)
VSL	696,000
ITCS1	820,439
ITCS2	1,011,346

There is a trade-off in selecting the best ITCS scenario that can provide both high performance and minimum TTT. Reducing speed limits can improve traffic performance at signalised intersections but it also increases the TTT. Figure 7.64 shows that the maximum efficiency for ITCS scenarios for TTT was 5.3% and 4.3% after using SCL (b) of ITCS2 and ITCS1 respectively. In contrast to VSL applications, all ITCS scenarios achieved a positive reduction in TTT while some VSLs scenarios increased TTT. ITCS minimised TTT by about 1.5% more than VSLs.

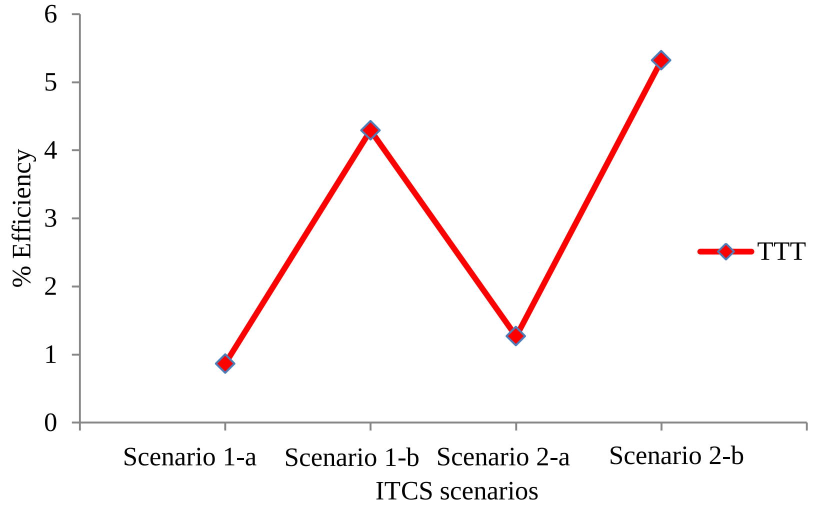


Figure 7.64 Maximum efficiency of ITCS regarding TTT

7.8 Summary

This chapter investigated the efficiency of integrating two types of traffic control systems, VSL applications and incremental changes to the cycle length of congested traffic signals. The applications were tested for traffic management under two traffic control scenarios on the GAR. Scenario 1 (ITCS1) was based on increasing the signal timing for the WB lanes at intersections 1 and 2 by 10s, and managing the upstream traffic with two speed control limits (SCLs), 30-35 km/hr and 40-45 km/hr under different circumstances. Scenario 2 (ITCS2) was similar to Scenario 1 except that the incremental increase in the signal timing was 20s.
Four traffic parameters covering the main performance properties of signalised intersections were evaluated for the Integrated Traffic Control System (ITCS)

scenarios. ITCS1 and ITCS2 improved the level of service at intersection 2 from LOS E to LOS D despite there being a 5.5-second difference between both strategies. This improvement in LOS enhanced the performance of the traffic signals during the congestion period. The difference in results between ITCS1 and ITCS2 was due to the change in the signal timing. ITCS2 produced more improvement in all signalised parameters than ITCS1. This means that VSL applications can produce higher performance at intersection 2 when combined with other traffic scenarios (ITCS1 and ITCS2).

A 20% improvement in average queue length, a 21% improvement in average number of stops, a 19% improvement in average delay and 17% improvement in average stopped delay were achieved by using ITCS1. A 28% improvement in average queue length, a 27% improvement in average number of stops, a 29% improvement in average delay and average stopped delay were achieved by using ITCS2.

TTT decreased by 5.3% and 4.3% after employing ITCS1 and ITCS2. These results reduced the cost of congestion by minimising the amount of delay and reducing the travel time. At the same time, they improved the safety at traffic signals through reducing rear-end collisions.

A maximum reduction in delay of 29% reduced the annual delay time from 4,884 hours to 3,723 hours despite increasing the intersection throughput by 7% in the evening peak hour. The reduction in delay saved 1,161 hours per year, which valued at approximately $12,307. This was more than twice the savings from using VSL.

A 27% reduction in the number of stops reduced the risk of rear-end collisions at traffic signals. These results showed that using ITCS could enhance traffic safety in the vicinity of intersection 2.

A constant increase in improvement was achieved for all proposed traffic scenarios at GAR. All proposed scenarios improved speed, flow and density by 3-11%. ITCS1 and ITCS2 did not optimise the efficiency of link parameters more than the use of VSL except for speed (11%) in the case of ITCS2. This was due to a small improvement in traffic density (3.5%).

ITCS was able to increase the efficiency of the signalised intersection but it was unable to optimise the efficiency of macroscopic parameters beyond what was achieved by VSL. Both VSL and ITCS increased flow by 7% and this indicated they were able to increase mobility along the GAR during the peak hour and increased the annual vehicle accessibility by approximately 24,000 vehicles.

Due to the improvement in speed parameter from using ITCS, the probability of fatality during the peak period was improved. A 9% decrease in speed variations reduced the probability of fatality by approximately 1.35% while 11% enhancement reduced the probability of fatality by around 1.56%. These improvements saved annually $82,314-$95,118 in total estimated crash costs for the GAR during peak hour.

The influence of the ITCS scenarios on TTT was investigated along the WB direction of GAR. ITCS was able to reduce TTT by 5.3% at the most. For example, during the evening peak hour, the actual mean travel time was 598 seconds for the WB direction with total vehicle throughput of 1,542 veh/hr. This TTT translated to 61,474 hours per a year under uncontrolled conditions. A maximum saving of 32 seconds, which is about 5.3% reduction in actual vehicle travel time, may firstly appear trivial at the high traffic volume but it can yield substantial benefits overall.

A savings of 58,184 hours per year was calculated from using ITCS for one peak hour period. This 5.3% travel time savings translated to 3,290 hours of time saved per year for all vehicles traveling on the WB direction in the evening peak hour which returned a TTT savings approximately $34,874.

The outcomes of VSL and ITCS can be extended to conclude the national level for BMA in terms of crashes and TTT. The total estimated annual savings in crashes was $2,347,000and $2,712,000 due to the use of VSL, and ITCS respectively. While the total annual estimated savings in TTT was around $696,000 and $1,011,000.

All ITCS scenarios reduced TTT while some VSL scenarios increased TTT. ITCS gave more than 1.5% improvement over VSL in TTT savings.

IBM SPSS proved that ITCS1 and ITCS2 had significant impact on vehicle delay time at traffic signal and on vehicle speeds in a congested link regardless of whether the link was close to or away from the intersection area.

CHAPTER EIGHT
Summary and Conclusion

8.1 Main conclusions from this study

8.1.1 The impact of VSL and ITCS on intersection parameters

The research investigated the application of Variable Speed limits (VSL) and its incorporation into a new Integrated Traffic Control System (ITCS). Several traffic control scenarios were used to estimate the effect of applying VSL control and ITCS. The aim of the VSL control was to improve the level of service (LOS) and traffic safety at downstream-signalised intersection. The key research outcomes were:

1. All VSL scenarios were useful for improving the traffic performance of congested intersections from LOS E to LOS D.
2. VSL application improved the level of service during the peak hour by reducing the overall delay and average stopped delay by approximately 15%.
3. VSL application improved the traffic safety during the peak hour by reducing the average queue length and average number of stops by approximately 18%.
4. The position at which the speed control was implemented had a large impact on the traffic performance of the downstream-congested intersections. VSLs achieved greater efficiency the closer they were applied to the source of congestion.
5. The efficiency of ITCS was based on the pre-emption time. The pre-emption time for ITCS1 was small (10s) in contrast with ITCS2 where it was (20s). The pre-emption times of ITCS1 and ITCS2 improved the level of service by 19% and 29% respectively. ITCS 2 was able to maximise the initial LOS to the lower limit of LOS D and close to the upper limit of LOS C (35seconds).
6. ITCS1 and ITCS2 estimated the improvement of traffic safety by 21% and 27% respectively. VSLs applications can be more efficient when combined with other traffic control strategies as shown for ITCS 1 and ITCS 2.

8.1.2 The impact of VSL and ITCS on macroscopic traffic characteristics

The effect of VSL and ITCS on macroscopic traffic characteristics (speed, flow, and density) of a congested link were investigated for the peak traffic period. The same signalised intersection scenarios were used in investigating the efficiency of both VSL and ITCS. The aims of VSL control were to maximise the mobility (flow) and travel speed, and to enhance or maintain the traffic density for the congested link during the peak period. The aim of ITCS was to support the VSL operation during the same peak period. The research outcomes were:

1. All scenarios of VSL and ITCS improve speed and flow and reduce density by 3-9%.
2. ITCS1 and ITCS2 did not increase the efficiency of link parameters more than the use of VSL except for speed (11%) in the case of ITCS2. This was

due to only a small reduction in traffic density (3.5%) and the link reaching its optimum capacity through the use of VSL (i.e. any other traffic scenario could not improve this capacity).

3. ITCS increased the efficiency of the signalised intersection but it was not able to maximise the efficiency of the link parameters more than the efficiency achieved by VSL applications.
4. Both VSL and ITCS increased flow by 7% during the evening peak hour and this indicated they were able to increase vehicle accessibility along the GAR by 24,000 vehicles per year.
5. Statistical analysis showed that VSL and ITCS had a significant impact on vehicle delay time at traffic signal and on vehicle speeds for a link close to or away from the signalised intersection area during the peak hour.

8.1.3 The impact of VSL and ITCS on TTT

The third part of this work investigated the changes in total travel time (TTT) due to VSL and ITCS. The TTT was considered for all west bound (WB) vehicles travelling from intersection 6 to intersection 1. This is a distance of about 6.7 km. The investigations lead to the following conclusions:

1. VSL and ITCS showed an acceptable reduction in TTT of 3.7% and 5.5% respectively. Variation in improvements was due to different pre-emption times for ITCS.
2. In comparison to VSL applications, all ITCS scenarios reduced TTT. Some VSL scenarios increased TTT.
3. Overall, VSL and ITCS showed little impact on TTT due to the constrained speed limits for non-congested links.
4. VSLs investigations on the east bound (EB) direction (Appendix B) showed nearly the same positive results of WB direction for improving the performance of intersection 6 and reducing TTT.

8.2 The impact of VSL and ITCS on annual savings

The research was able to estimate potential savings by using VSL and ITCS with respect to congestion time savings and collision costs. The revenue was calculated for one peak hour of control, with a five working day week. The vehicle delay time value used for Brisbane during peak hour was $10.60 per hour (Booz, 2003).

1. The reduction in average delay time at WB of intersection 2 due to the use of VSL, ITCS1 and ITCS2 was 493, 636 and 1,161 hours per year respectively. This reduction in delay would save approximately $5,000, $7,000 and $12,000 respectively.
2. The reduction in TTT would be 2,261, 2,669 and 3,290 hours per year for all vehicles traveling on WB direction of GAR due to the use of VSL, ITCS1 and ITCS2 respectively. This would produce savings of approximately $24,000, $28,000 and $35,000 respectively.
3. VSL application lowered the probability of fatalities along the congested link by 1.35% while ITCS lowered the probability of fatality by 1.56%. These improvements contributed significantly to reduce total estimated crash costs on the GAR during peak hour of about $85,000 per year.

4. Extrapolation shown that the total annual saving that can be expected from reducing crashes in the congested arterials in Brisbane Metropolitan Area (BMA) will be around $2,347,000, $2,260,000 and $2,712,000 due to the use of VSL, ITCS1 and ITCS2 respectively. In addition to that, the total annual savings estimated from decreasing TTT along the congested arterial roads in BMA are about $696,000, $820,000 and $1,011,000 due to the use of VSL, ITCS1 and ITCS2 respectively.

8.3 Summary

Urban traffic congestion is worsening globally with the ever-increasing urbanisation and vehicle ownership. Long queues at intersections and blockage of traffic on urban arterial roads have become routine during peak traffic hours and especially in the vicinity of intersections. Traffic flow through signalised intersections and along arterial roads is often interrupted and negatively affected by traffic congestion. Congestion negatively influences people's quality of life by increasing their travel time, decreasing the performance of signalised intersection, decreasing mobility and safety, increasing fuel consumption and contaminating the atmosphere, and creating frustration.

Over the last three decades, the development of Intelligent Transport Systems (ITS) has led to innovative new ways of managing and operating existing transportation infrastructure. As part of ITS management strategies, Variable Speed Limits (VSL) has been applied to improve the operation of congested motorways. Its application has been intended to decrease the breakdown in traffic flow, reduce pollution and increase the efficiency, reliability and safety of the road without needing major physical changes to the road infrastructure. However, VSL does not appear to be utilised to improve the operation of interrupted flow facilities such as occurs on arterials, in terms of the level of service, safety and mobility.

Most of the control strategies that have been used to test the effectiveness of VSL have been either theoretical studies or limited empirical evaluations. The number of strategies required to validate efficiencies for a transportation system may be large and field-testing would be prohibitively expensive. For this reason, microscopic traffic simulation is a tool that is often used.

In this study VISSIM software was used to model a road network link and the Griffith Arterial Road in Brisbane was selected as a case study area. The model used data collected by the Queensland Department of Transport and Main Roads (QDTMR) and data collected during the research by surveillance video cameras. The model was developed to incorporate the road network and lane configurations, signal timing and coordination, and vehicle and driver behaviour. One of the most difficult tasks in the process was the calibration of saturation flow rate at each intersection to represent actual conditions.

The GEH formula was employed to show the variation between the two sets of data, field data and simulated result. One-Way ANOVA at 95% level of confidence was applied to estimate the significant of employing VSL using IBM SPSS software.

The original hypothesis of the thesis was that VSL had potential to help alleviate congestion and improve safety on urban arterial roads through its integration with signalised control systems. The research has demonstrated through modelling using the parameters obtained from QDTMR and extensive field work that VSL does have great potential for traffic congestion relief on arterials and for associated savings related to reduced travel time and crashes.

8.4 Recommendations for future work

There is scope to extend the research finding to the arterial network level. Some of the potential work possible to extend the research is outlined below:

1. VSL application can be employed in other Australian urbanised area and other countries in order to mitigate traffic congestion.
2. This initial research on the application of VSL was undertaken for the peak period. The impact of VSL on interrupted flow traffic performance during off-peak periods could be investigated.
3. This investigation focused on WB and EB directional flows for GAR. The research could be expanded to consider the effect of applying VSL on all four directions of flow for intersections in the congested area.
4. VSL was combined with a single traffic control strategy (ITCS) in this study. Further research could study the impact of VSL with other traffic control strategies such as dynamic congestion pricing and adaptive control system.
5. The investigation did not consider the impact of VSL on public transport. Further research could be undertaken to understand the impact of the interaction between VSL and the efficiency of public transport.
6. The VSL application is implemented as a means of improving the traffic conditions on urban arterial roads. The results indicate that VSL had a significant impact on traffic conditions when drivers comply with variable message signs. Further investigations may be needed to reveal the impact of driver compliance on the effectiveness of VSL application.

References

Abdel-Aty, M, Cunningham, RJ, Gayah, VV & Hsia, L 2008, Dynamic variable speed limit strategies for real-time crash risk reduction on freeways, *Transportation Research Record,* vol. 2078**,** pp. 108-116.

Abdy, ZR & Hellinga, BR 2008, Use of Microsimulation to Model Day-to-Day Variability of Intersection Performance, *Transportation Research Record,* vol. 2088**,** pp. 18-25.

Alessandri, A, di Febbraro, A, Ferrara, A & Punta, E 2002, Nonlinear optimization for freeway control using variable-speed signaling, *Vehicular Technology,* vol. 48**,** pp. 2042-2052.

Allaby, P, Hellinga, B & Bullock, M 2007, Variable speed limits: Safety and operational impacts of a candidate control strategy for freeway applications, *Intelligent Transportation Systems,* vol. 8**,** pp. 671-680.

Arnott, R, de Palma, A & Lindsey, R 1990, Departure time and route choice for the morning commute, *Transportation Research Part B,* vol. 24**,** pp. 209-228.

Australian Bureau of Statistics (ABS) 2013, Survey of motor vehicle use, Canberra, Australia.

Beckmann, M 1967, On Optimal Tolls for Highways, Tunnels, and Bridges Proceedings of the Third International Symposium on the Theory of Traffic Flow, New York, USA, pp. 331-341.

Bellemans, T, De Schutter, B & De Moor, B 2006, Model predictive control for ramp metering of motorway traffic, *Control Engineering Practice,* vol. 14**,** pp. 757-767.

Bertini, RL, Boice, S & Bogenberger, K 2006, Dynamics of variable speed limit system surrounding bottleneck on german autobahn, *Transportation Research Record,* vol. 1978**,** pp. 149-159.

Bloomberg, L & Dale, J 2000, Comparison of VISSIM and CORSIM traffic simulation models on a congested network, *Transportation Research Record,* vol. 1727**,** pp. 52-60.

Booz, AH 2003, Transport Demand Elasticities Study ACT, Canberra Department of Urban Services.

Brust, H, Smolic, A, Mueller, K, Tech, G & Wiegand, T 2009, Mixed resolution coding of stereoscopic video for mobile devices, The True Vision-Capture, Transmission and Display of 3D Video, pp. 1-4.

Buchanan, C 1963, *Traffic in towns,* London UK, Penguin Books.

Bureau of Transport and Regional Economics (BTRE) 2007, Estimating urban traffic and congestion cost trends for Australian cities, *Working Paper No.71*. Canberra, Australia.

Cairney, P 2013, *Guide to Road Safety Part 1: Road Safety Overview,* Australia, Austroads, Sydney.

Carlson, RC, Papamichail, I, Papageorgiou, M & Messmer, A 2010, Optimal motorway traffic flow control involving variable speed limits and ramp metering, *Transportation Science*, vol. 44**,** pp. 238-253.

Catling, I 1977, A time-dependent approach to junction delays, *Traffic Engineering and Control*, vol. 18**,** pp. 520-526.

Chen, C, Jia, Z & Varaiya, P 2001, Causes and cures of highway congestion, *Control Systems*, vol. 21**,** pp. 26-32.

Chien, C, Zhang, Y & Ioannou, PA 1997, Traffic Density Control for Automated Highway Systems, *Automatica*, vol. 33**,** pp. 1273-1285.

Chung, Y & Choi, K 2010, Dynamic Toll Concept to Assess Feasibility of High-Occupancy Vehicle Lane on Kyungbu Freeway, South Korea, *Transportation Research Record*, vol. 2178**,** pp. 162-169.

Coleman, J, Paniati, J, Cotton, R, Parker , M, Covey, R, Pena, H, Graham, D, Robinson, M, MaClauley, J & Taylor, W 1996, FHWA study tour for speed management and enforcement technology, *US Department of Transportation, Washington, DC.*

Cronje, W 1983, Optimization model for isolated signalized traffic intersections, *Transportation Research Record*, vol. 905**,** pp. 80-83.

Daganzo, CF 2007, Urban gridlock: macroscopic modeling and mitigation approaches, *Transportation Research Part B*, vol. 41**,** pp. 49-62.

Dai, LL, Li, Y, Wang, YX & Liu, DB 2013, Application of Collaborative Optimization Strategies for Traffic Control of Urban Arterial Intersection, American Society of Civil Engineers, pp. 1031-1039.

Darma, Y, Karim, MR, Mohamad, J & Abdullah, S 2005, Control delay variability at signalized intersection based on HCM method, *Proceedings of the eastern Asia society for Transportation Studies*, vol. 5**,** pp. 945-958.

Department of Transport and Main Roads (TMR) 2010, South-east Queensland travel time survey, TMR Modelling Data and Analysis Centre, Brisbane QLD: *http://www.tmr.qld.gov.au/Travel-and-transport/Road-and-traffic-info/South-east-Queensland-travel-time-survey.aspx*.

Di Febbraro, A, Parisini, T, Sacone, S & Zoppoli, R 2001, Neural approximations for feedback optimal control of freeway systems, *Vehicular Technology,* vol. 50**,** pp. 302-313.

Dion, F, Rakha, H & Kang, YS 2004, Comparison of delay estimates at under-saturated and over-saturated pre-timed signalized intersections, *Transportation Research Part B,* vol. 38**,** pp. 99-122.

Dowling, R 2007, T*raffic Analysis Toolbox Volume VI: Definition, Interpretation, and Calculation of Traffic Analysis Tools Measures of Effectiveness*, Washington, Department of Transportation Federal Highway Administration.

Factor, R, Prashker, JN & Mahalel, D 2012, The flashing green light paradox, *Transportation Research Part F, Traffic Psychology and Behaviour,* vol. 15**,** pp. 279-288.

Garber, NJ & Hoel, LA 2009, *Traffic and Highway Engineering*, USA, International Thomson Publishing.

Geroliminis, N. 2007, *Increasing mobility in cities by controlling overcrowding,* PhD thesis, University of California.

Geroliminis, N & Levinson, D 2009, Cordon pricing consistent with the physics of overcrowding. *In:* H.K., W, Lam, SC & Wong, HKL (eds.), *Transportation and Traffic Theory 2009, Golden Jubilee,* Netherland, Springer.

Gomes, G, May, A & Horowitz, R 2004, Congested freeway microsimulation model using VISSIM, *Transportation Research Record,* vol. 1876**,** pp. 71-81.

Hadiuzzaman, M, Qiu, TZ & Lu, XY 2012, Variable Speed Limit Control Design for Relieving Congestion Caused by Active Bottlenecks, *Journal of Transportation Engineering,* vol. 139**,** pp. 358-370.

Hansen, M & Huang, Y 1997, Road supply and traffic in urban areas: a panel study, *Transportation Research A,* vol. 31**,** pp. 205–218.

Harbord, B 1995, The application of SISTM to dynamic control on the M25,Colloquium on Dynamic Control of Strategic Inter-Urban Road Networks, *London, UK The Institution of Engineering and Technology,* pp. 1-5.

He, J & Hou, Z 2012, Ant colony algorithm for traffic signal timing optimization, *Advances in Engineering Software,* vol. 43**,** pp. 14-18.

Hegyi, A, De Schutter, B & Hellendoorn, H 2005, Model predictive control for optimal coordination of ramp metering and variable speed limits, *Transportation Research Part C,* vol. 13**,** pp. 185-209.

Hegyi, A, Schutter, BD & Hellendoorn, J 2003, Optimal coordination of variable speed limits to suppress shock waves, *Transportation Research Record,* vol. 1852**,** pp. 167-174.

Herman, R & Ardekani, S 1984, Characterizing traffic conditions in urban areas, *Transportation Science,* vol. 18**,** pp. 101-140.

Herman, R, Malakhoff, LA & Ardekani, SA 1988, Trip time-stop time studies of extreme driver behaviors, *Transportation Research Part A,* vol. 22**,** pp. 427-433.

Herman, R & Prigogine, I 1979, A two-fluid approach to town traffic, *Science,* vol. 204**,** pp. 148-151.

Hongfeng, C, Pu, X, Xiangzhao, F & Fen, L 2008, Public Attitudes Towards Policy Instruments for Congestion Mitigation in Shanghai, *Chinese Journal of Population,* vol. 6**,** pp. 40-47.

Hou, Z, Xu, J-X & Yan, J 2008, An iterative learning approach for density control of freeway traffic flow via ramp metering, *Transportation Research Part C,* vol. 16**,** pp. 71-97.

Jin, S, Wang, D & Wang, L 2009, Traffic control strategy for a surface street on an expressway-arterial corridor, *Tsinghua Science & Technology,* vol. 14**,** pp. 776-781.

Joksch, HC 1993, Velocity change and fatality risk in a crash: a rule of thumb, *Accident Analysis & Prevention,* vol. 25**,** pp. 103-104.

Jones, B, Janssen, L & Mannering, F 1991, Analysis of the frequency and duration of freeway accidents in Seattle, *Accident Analysis & Prevention,* vol. 23**,** pp. 239-255.

Jones, P 1995, Road pricing: The public viewpoint. *In:* Johansson, B & Mattson, L (eds.), *Road pricing: theory, empirical assessment and policy,* Boston, Kluwer.

Jonkers, E, Wilmink, I, Stoelhorst, H, Schreuder, M & Polderdijk, S 2011, Results of field trials with dynamic speed limits in the Netherlands: improving throughput and safety on the A12 freeway, *Intelligent Transportation Systems*, Washington DC, USA, pp. 2168-2173.

Kejun, L, Meiping, Y, Jianlong, Z & Xiaoguang, Y 2008, Model predictive control for variable speed limit in freeway work zone, *Chinese Control Conference*, pp. 488-493.

Khattak, AJ 2001, Injury severity in multivehicle rear-end crashes, *Transportation Research Record,* vol. 1746**,** pp. 59-68.

Kohler, U 1974, Stability of vehicle platoons,Transportation and Traffic Theory, University of New South Wales, Sydney, Australia Elsevier Publishing Company, pp. 39-55.

Kotsialos, A, Papageorgiou, M, Diakaki, C, Pavlis, Y & Middelham, F 2002, Traffic flow modeling of large-scale motorway networks using the macroscopic modeling tool METANET, *Intelligent Transportation Systems,* vol. 3**,** pp. 282-292.

Kwon, E, Brannan, D, Shouman, K, Isackson, C & Arseneau, B 2007, Development and Field Evaluation of Variable Advisory Speed Limit System for Work Zones, *Transportation Research Record,* vol. 2015**,** pp. 12-18.

Laufer, J 2007, Freeway capacity, saturation flow and the car following behavioural algorithm of the VISSIM microsimulation software,*30th Australasian Transport Research Forum,* Melbourne, Australia, pp. 1-15.

Lee, C, Hellinga, B & Ozbay, K 2006, Quantifying effects of ramp metering on freeway safety, *Accident Analysis & Prevention,* vol. 38**,** pp. 279-288.

Lenz, H, Sollacher, R & Lang, M 2001, Standing waves and the influence of speed limits, *space [km],* vol. 518**,** pp. 1-5.

Levinson, D, Zhang, L, Das, S & Sheikh, A 2004, Measuring the Equity and Efficiency of Ramp Meters, Minnesota: *University of Minnesota,* Department of Civil Engineering.

Lin, P, Kang, K & Chang, G 2004, Exploring the effectiveness of variable speed limit controls on highway work-zone operations, *Journal of Intelligent Transportation Systems,* vol. 8**,** pp. 155-168.

Lownes, NE & Machemehl, RB 2006, VISSIM: a multi-parameter sensitivity analysis, *Simulation Conference,* pp. 1406-1413.

Luk, J & Hepburn, S 1993, *New review of Australian travel demand elasticities,* Victoria, Australia, Australian Road Research Board.

Luttinen, RT & Nevala, R 2002, *Capacity and level of service of Finnish signalised intersections,* Helsinki, Finnish Road Administration.

Mahmassani, HS, Williams, J & Herman, R 1987, Performance of urban traffic networks, *The Tenth International on Transportation and Traffic Theory Symposium,* Cambridge, Massachusetts, Elsevier, pp. 78-88.

Marchand, M 1968, A note on optimal tolls in an imperfect environment, *Econometrica: Journal of the Econometric Society,* vol. 36**,** pp. 575-581.

May, AD 1986, Traffic restraint: a review of the alternatives, *Transportation Research Part A,* vol. 20**,** pp. 109-121.

McDonald, JF 1995, Urban highway congestion, *Transportation*, vol. 22, pp. 353-369.

Mogridge, MJH 1997, The self-defeating nature of urban road capacity policy : A review of theories, disputes and available evidence, *Transport Policy*, vol. 4, pp. 5-23.

O'Day, J & Flora, J 1982, Alternative measures of restraint system effectiveness: interaction with crash severity factors, *Society of Automotive Engineers*, Warrendale, pp. 1-6.

Ogden, KW & Newstead, SV 1994, *Analysis of crash patterns at Victorian signalised intersections:* Monash University, Accident Research Centre.

Oketch, T & Carrick, M 2005, Calibration and validation of a micro-simulation model in network analysis, Proceedings of the 84th TRB Annual Meeting, Washington DC, pp. 1-17.

Olszewski, P 1993, Overall delay, stopped delay, and stops at signalized intersections, *Journal of transportation engineering*, vol. 119, pp. 835-852.

Olszewski, P & Xie, L 2005, Modelling the effects of road pricing on traffic in Singapore, *Transportation Research Part A*, vol. 39, pp. 755-772.

Orski, C 1972, Car-Free Zones and Traffic Restraints: Tools of Environmental Management, *Highway Research Record*, vol. 406, pp. 37-46.

Papageorgiou, M 1983, *Applications of automatic control concepts to traffic flow modeling and control*, New York, Springer-Verlag.

Papageorgiou, M, Kosmatopoulos, E & Papamichail, I 2008, Effects of variable speed limits on motorway traffic flow, *Transportation Research Record*, vol. 2047, pp. 37-48.

Papageorgiou, M & Kotsialos, A 2000, Freeway ramp metering: An overview, *Intelligent Transportation Systems*, Dearborn, pp. 228-239.

Papamichail, I, Kotsialos, A, Margonis, I & Papageorgiou, M 2010, Coordinated ramp metering for freeway networks–A model-predictive hierarchical control approach, *Transportation Research Part C*, vol. 18, pp. 311-331.

Parthasarathi, P, Levinson, D & Karamalaputi, R 2003, Induced demand: a microscopic perspective, *Urban Studies*, vol. 40, pp. 1335.

Pigou, A 1912, *Wealth and Welfare*, London: Macmillan.

Planning Transport Verkekr (PTV) AG 2011, *User's Manual, 5.3*, Karlsruhe, Germany.

Rathi, A & Lieberman, E 1989, Effectiveness of traffic restraint for a congested urban network: A simulation study, *Urban Traffic Systems and Operations*, vol. 1232**,** pp. 95-102.

Retting, R & Cheung, I 2008, Traffic speeds associated with implementation of 80 mph speed limits on West Texas rural interstates, *Journal of Safety Research*, vol. 39**,** pp. 529-534.

Rodegerdts, LA, Nevers, B, Robinson, B, Ringert, J, Koonce, P, Bansen, J, Nguyen, T, McGill, J, Stewart, D & Suggett, J 2004, *Signalized intersections: informational guide*, USA: Federal Highway Administration.

Roess, RP, Prassas, ES & McShane, WR 2004, *Traffic engineering*, Pearson/Prentice Hall.

Sadek, AW & Smith, MC 2006, Challenges Calibrating Large-Scale Microscopic Simulation Model of Diverse Urban, Suburban, and Rural Network: Practical Guide, *Transportation Research Board 85th Annual Meeting*, Washington DC, pp. 1-19.

Smeed, R 1966, Road capacity of city centers, *Traffic Engineering and Control*, vol. 8**,** pp. 455-458.

Smulders, S 1992, Control by variable speed signs: The Dutch Experiment, International Conference on Road Traffic Monitoring and Control London, *Institution of Electrical Engineers*, pp. 99-103.

Stevanovic, A, Stevanovic, J & Kergaye, C 2013, Optimization of traffic signal timings based on surrogate measures of safety, *Transportation Research Part C*, vol. 32**,** pp. 159-178.

Sugiyama, N & Nagatani, T 2012, Multiple-vehicle collision induced by a sudden stop in traffic flow, *Physics Letters A*, vol. 376**,** pp. 1803-1806.

Teply, S, Allingham, D, Richardson, D & Stephenson, B 1995, *Canadian capacity guide for signalized intersections*, Canada, Institute of Transportation Engineers.

Thomson, J 1967, Speeds and Flows of Traffic in Central London, *Traffic Engineering and Control*, vol. 8**,** pp. 721–725.

Transport For London (TFL) 2004, Congestion Charging Central London-Impacts Monitoring, *Second Annual Report.* London: www.tfl.gov.uk/tfl/cclondon/ccmonitoring-2nd-report.shtml.

Transportation Research Board (TRB) 1985, *Highway Capacity Manual 1985*, National Research Council, Washington DC.

Transportation Research Board (TRB) 2000, *Highway Capacity Manual 2000*, National Research Council, Washington DC.

Transportation Research Board (TRB) 2010, *Highway Capacity Manual 2010*, National Research Council, Washington DC.

Troutbeck, R, Su, M & Luk, J 2007, National Performance Indicators for Network Operations, Sydney, Australia.

Turner, S, Turner, B & Wood, G 2008, Accident prediction models for traffic signals,*ARRB Conference*, Adelaide, South Australia, Australia, pp. 1-15.

Van Aerde, M & Rakha, H 2007, INTEGRATION © Release 2.30 for Windows: User's Guide Volume I: Fundamental Model Features, *Van Aerde & Assoc., Ltd., Blacksburg.*

Verhoef, E, Nijkamp, P & Rietveld, P 1997, The social feasibility of road pricing: a case study for the Randstad area, *Journal of Transport Economics and Policy,* vol. 31**,** pp. 255-276.

Vickrey, W 1963, Pricing and Resource Allocation in Transportation and Public Utilities: Pricing in Urban and Suburban Transport, *American Economic Review,* vol. 53**,** pp. 452-465.

Viegas, JM 2001, Making urban road pricing acceptable and effective: searching for quality and equity in urban mobility, *Transport Policy,* vol. 8**,** pp. 289-294.

Wang, Y, Papageorgiou, M, Sarros, G & Knibbe, WJ 2006, Feedback route guidance applied to a large-scale express ring road, *Transportation Research Record,* vol. 1965**,** pp. 79-88.

Wardrop, J 1953, Some theoretical aspects of road traffic research, vol. 4**,** pp. 72-73.

Watson, P & Holland, E 1978, Relieving traffic congestion: the Singapore area license scheme, *Staff Working Paper No. 281,* Washington DC, World Bank.

Webster, FV 1958, Traffic signal settings, *Road Research Laboratory,* London: Road Res, Tech.

Webster, FV & Cobbe, BM 1966, *Traffic signals*, London, Her Majestyás Stationery Office.

Wiedemann, R & Reiter, U 1992, Microscopic traffic simulation: the simulation system MISSION, background and actual state, *Project ICARUS (V1052) Final Report, Brussels,* vol. 2**,** pp. 1-15.

Wiles, PB, Cooner, SA, Rathod, Y & Wallace, DG 2005, Advance Warning of Stopped Traffic on Freeways: *Field Studies of Congestion Warning Signs*, Austin, Texas Transportation Institute.

Wu, Z, Sharma, A, Mannering, FL & Wang, S 2013, Safety impacts of signal-warning flashers and speed control at high-speed signalized intersections, *Accident Analysis & Prevention,* vol. 54**,** pp. 90-98.

Yan, X, Radwan, E & Abdel-Aty, M 2005, Characteristics of rear-end accidents at signalized intersections using multiple logistic regression model, *Accident Analysis & Prevention,* vol. 37**,** pp. 983-995.

Yang, Q, Han, LD & Cherry, CR 2013, Some measures for sustaining red-light camera programs and their negative impacts, *Transport Policy,* vol. 29**,** pp. 192-198.

Zackor, H 1979, Self-sufficient control of speed on freeways, *International Symposium on Traffic Control Systems,* vol. 2**,** pp. 226-249.

Zackor, H 1991, Speed limitation on freeways: *Traffic-responsive strategies*: Pergamon Press, Oxford, United Kingdom.

Zahavi, Y 1972, Traffic performance evaluation of road networks by the α-relationship, *Traffic Engineering and Control,* vol. 14**,** pp. 228-231.

Zhang, X & Yang, H 2004, The optimal cordon-based network congestion pricing problem, *Transportation Research Part B,* vol. 38**,** pp. 517-537.

Zhou, S, Sun, J, Li, K & Yang, X 2013, Development of a Root Cause Degree Procedure for measuring intersection safety factors, *Safety Science,* vol. 51**,** pp. 257-266.

APPENDIX A

A.1 Development of VISSIM modelling

The aim of VISSIM modelling is to investigate the traffic problems of the GAR and test proposed traffic management strategies that could relieve and mitigate flow at congested intersections. In order to build a microscopic simulation model for GAR, three types of data are required; physical design of the road network, definition of vehicle characteristics, and knowledge of driver behaviour. In the present study, the first two data sets are kept constant during the model building. The third category of data is varied where it is necessary to replicate the actual traffic conditions along the network.

A.1.1 Physical road network & traffic signal timing

The first step in constructing a VISSIM model is drawing the geometric design of the study area. This was done using the VISSIM interface screen. An accurate and easy way to obtain the layout of the network is by utilising Google-Earth, where an unscaled aerial image can be taken of the selected study area and saved in the same directory as the VISSIM files. Secondly, the image is uploaded as a background screen for the VISSIM window. Thirdly, the image is scaled by following VISSIM guidelines to match the actual measurements. Eventually, the scaled map is saved in the same directory as the VISSIM files to avoid repeating the steps (more details can be found in the VISSIM (2011). Figure A.1 depicts the aerial image layout of the study area. The geometric design of the whole network, including lane configurations and intersection locations, are setup by tracing the image background using VISSIM links and connectors. The lane width of roadways can be obtained from the aerial image using the Google-Earth toolbox or from field measurements.

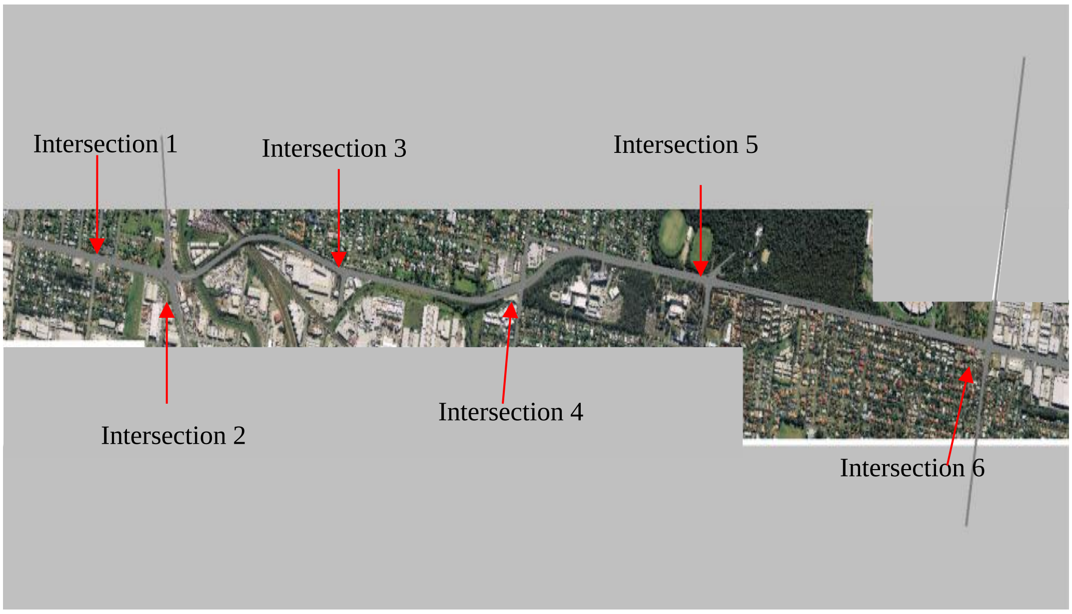


Figure A.1 VISSIM background image of the study area

Links area segments of any selected length of uniform roadway (same number of lanes of the same width. Connectors are used to join segments. The following points should be considered when creating VISSIM links and connectors:

1. Link type: By default, links have four driving behaviour classifications (urban, freeway, footpath, or cycle path) and one of these behaviours should be assigned to each link. Since the study area is focused on an urban area, the links were allocated as urban (Wiedemann 1974). In addition, the number of lanes and their width should be defined during the link design stage.
2. The lane change distance and the emergency stop distance needs to be defined for each connector. In order to replicate actual traffic flow, appropriate distances are used to control the traffic flow and the capacity of intersections.
3. Priority is assigned for merging lanes and for permitted signal operations.
4. Areas of reduced speed are modelled for the left and right turn movements at each intersection as well as for curves along the road network.
5. Options for conflict areas can also be assigned at intersections to avoid blocking by vehicles when an oversaturated condition occurs.

The next stage, after modelling the road network, is modelling the traffic signals. Six isolated signalised intersections were created using the traffic control signal and signal head tool. Since the research was focused only on the congested period, the traffic signal behaviour was fairly close to a fixed timing plan. The signal timing information and the traffic phase configuration were collected from the surveillance videos taken along the studied area. In VISSIM, signalized intersections can be created in different ways such as by built-in fixed-time control including the VISSIG add-on or by using various external signal control logic add-on software such as Econolite ASC/3, LISA+OMTC, SCATS, COOT, Vehicle-Actuated Signal Control (VAP) , and Ring Barrier Controller (RBC). Based on the selected control logic, VISSIM can simulate up to 125 signal groups per signal controller. It can also distinguish between signal groups and signal heads.
In VISSIM, a signal group is associated with each signal head that belong to an external control signal. All signal heads of the same group exhibit the same signal status at all times. For the present study, a separate signal group was formed for each individual signal head, in order to control the flow at each intersection. Signal heads were created for each travel lane at the location of the signal stop line. The signal group identifies the flow direction at each intersection (e.g., through movement, left and right turn). Sometimes one signal group shared more than one signal head if they have the same traffic characteristics in the same signal control. This occurs when two lanes had the same flow directions (two through lanes). In order to create the signal-timing plan, details about the cycle length and offset for each signal control need to be entered in the signal program. Information about how long each signal group is green, red, amber, and all red should be allocated via the signal program window. The cycle length and the offset time is also entered in his window. Figure A.2 illustrates the signal control window for intersection 1.
Some of the traffic movements have special traffic phases such as permitted or protected-permitted right turn. Since a pre-timed signal system doesn't have this operation, signal priority with signal head is used to imply the extra options in a fixed timing plan.

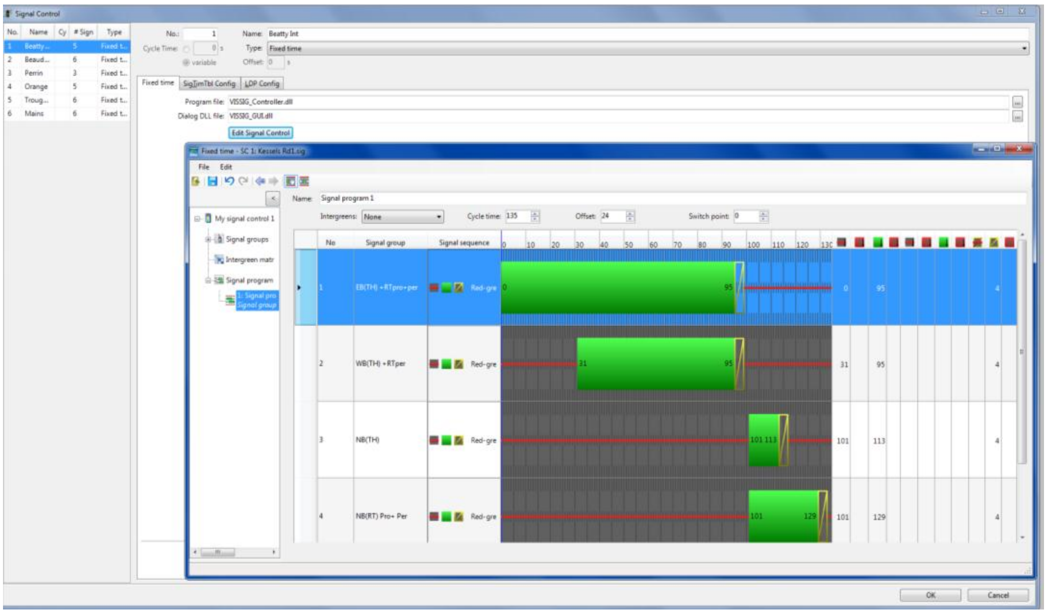


Figure A.2 Signal control window for intersection 1

A.1.2 Coding traffic data

Traffic volume, vehicle composition, desired speed and vehicle distribution parameters are required for model simulation. Traffic flow can be inserted as a time flow series. Observed hourly volume was used in VISSIM for each entrance point in the network. Also, a warm-up period of 600 sec was considered for simulation execution to fill the network with vehicles. the results for this period were excluded from the analysis. VISSIM has the ability to model a variety of vehicle types that reflect the diversity of vehicles in practice. Vehicle composition comprises a list of one or more vehicle types. A flow percentage and a speed distribution are assigned to each of these vehicle types. Four different vehicle types were modelled to reflect the actual traffic compositions in the present study. Cars, trucks, buses and motorcycles represent the mix of vehicles observed in practice and executed in VISSIM. The first two types comprise the major input flow in creating the model whereas buses and motorcycles represent minor input flows because their numbers were very small during the survey period. The default characteristics contained in VISSIM for each vehicle type are:

1. **Static parameter:** this section identifies the length, width and occupancy of each vehicle type where the user is able to adjust the relative values except the vehicle's length which is considered constant. The present study used VISSIM default static parameter values. Figure A.3 illustrates the vehicle type classifications for the selected area.

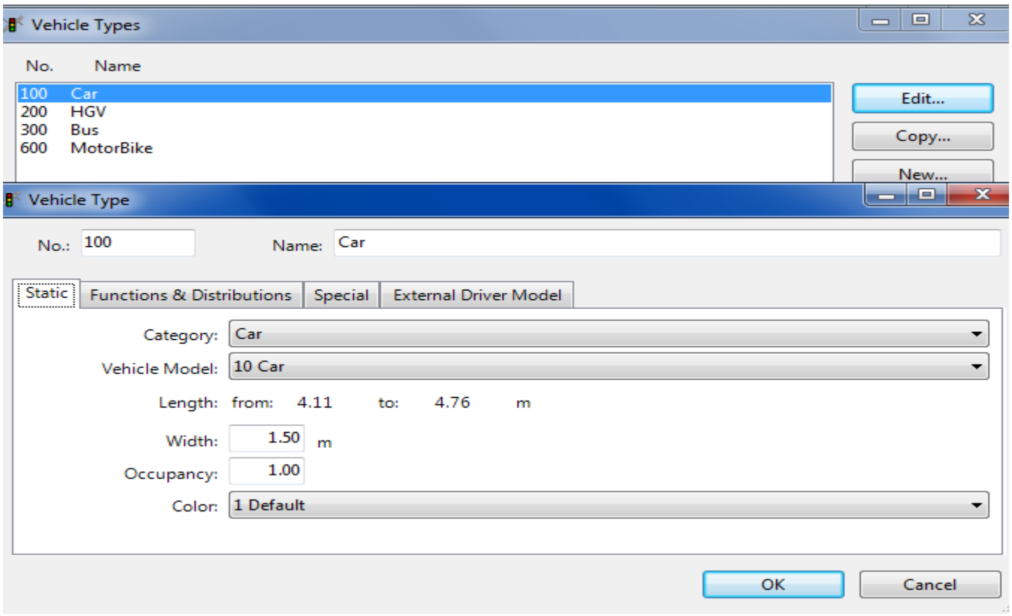
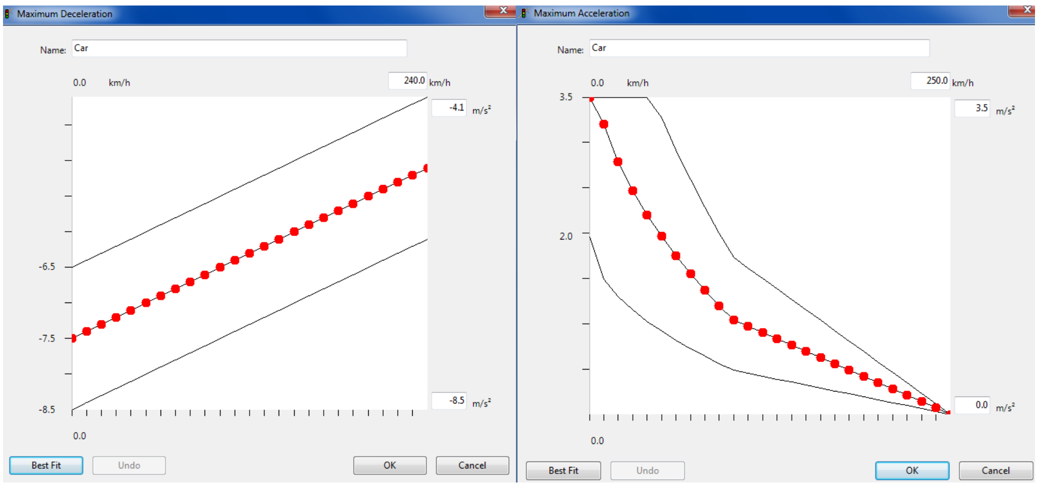


Figure A.3 VISSIM snapshot for such vehicle type

2. Function characteristics: In this section, minimum and maximum acceleration and deceleration for each type of vehicle can be assigned and adjusted to reflect the actual behaviour of drivers. VISSIM has default values for each function and this option was used in the current study. Figure A.4 (a) and (b) shows the deceleration and acceleration properties for car vehicle type.



(a) (b)

Figure A.4 Acceleration and deceleration functions

3. Vehicle distribution: This parameter defines the speed distribution, weight and power of each type of vehicle. The user also can adjust these values and insert new values. In the current study, the default values for the weight and power distribution remained invariant. Since there are different link speeds along the road network, various speed distributions were created based on each link speed value. The actual desired speeds are varied from 60 km/h to 80 km/hr. Since the most urbanised links are equipped with speed cameras, drivers may be more compliant with the speed limit in these links. Because drivers are used to driving 5 to 10 km/h less than the actual speed in urban area, the desired speed distribution was based on these thresholds.

For example, if the speed limit was 80 km/h for such a link, the desired speed distribution was set from 70 to 80 km/h.

The vehicle inputs from the previous step were used to spread traffic out along the road network. In VISSIM, a route represents the path of vehicles through a group of links and connectors. Route decisions can be assigned via two options, a static route and a dynamic assignment route. Static assignment routes are required for at least two points, the start point and the finish point at which the flow percentage or exact volume can be identified for each defined destination. Sometimes, one point route decision has more than one route destination, resembling a tree with several branches.

Dynamic assignment routes include multiple origin-destination matrices based on an iterative process. In this case, the drivers select their routes according to the travel cost among a group of route destinations for several simulations, not only for one simulation. The present study used the static assignment route to distribute the traffic flow along the study area.

When it starts, the static route generates a defined route decision point which is placed at the start of the origin link for the route. Then each route decision must be followed by at least one defined destination during the simulation process. Simulated time can be allocated and vehicle classes can be assigned for each signal route decision. Figure A.5 shows an example of the static route created at the NB lane for Beatty intersection.

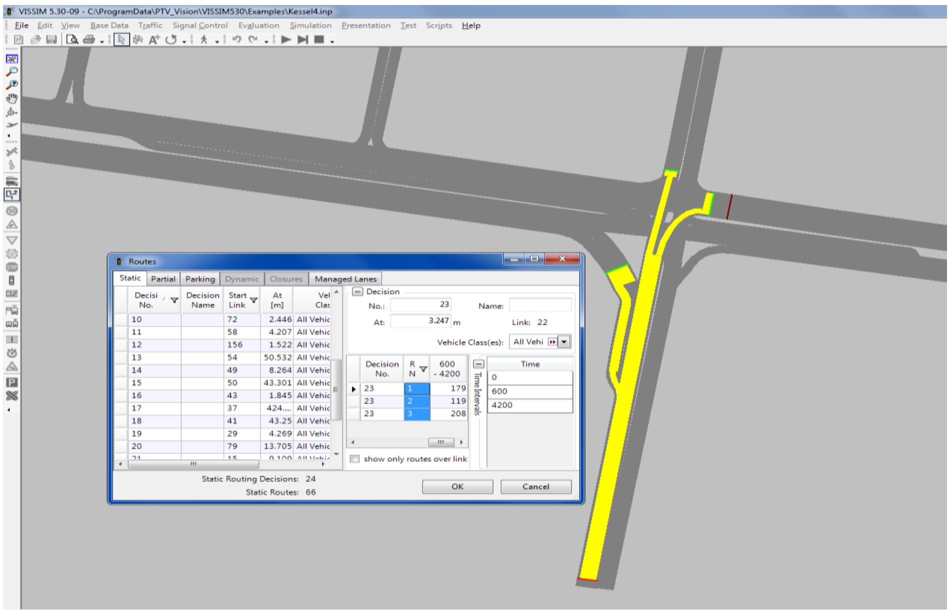


Figure A.5 Static routes distributions

A.1.3 Driving behaviour

The concept of “car following” and the “lane change” models in VISSIM are set for two types of driving behaviour based on Wiedeman 1974 and 1999. The first model defines traffic flow behaviour in urban area whereas the second model defines traffic flow on freeways. The study utilised the patterns defined by Wiedeman 1974 because they matched the driving behaviour observed in the study area. In VISSIM a wide range of parameters can be used to set up both the “car following” and “lane change” models.

Some of these may be modified by experienced users to change the basic rules of driver behaviour. Modified parameters drastically affect vehicle interactions, which in turn changes the simulation results. Driving behaviour can be selected through selection of the type of link behaviour. By default, five link types are pre-defined. There are four behaviour sub categories influencing the psychophysical “car-following” model. These are; following behaviour, lane change behaviour, lateral behaviour, and signal control behaviour. Each sub-category is associated with a set of parameters that controls the driving of the simulated vehicles along the road network. There is no correlation expected between vehicle type and driver behaviour.

In some cases, the initial execution of the model may exhibit correct traffic movement along the road network but not necessarily reflect the correct real measurements such as the number of vehicles for each lane at signalised intersections and the intersection throughput. The first execution of the model aims to determine the preliminary performance level that enables the user to understand whether the model matches the observed data or needs further calibration of the current driving behaviour parameter sets.

For study purposes, the car following behaviour, lane change parameters and signal control behaviour were considered so as to model an exclusive interrupted flow.

A.1.3.1 The car following behaviour

The default parameters did not match the field volume distribution along the network and did not reflect the discharged traffic movements (through, left and right movements) at signalised intersections. This section will be governed by a set of parameters that organise the flow of vehicles across the road network. These are:

1. **Look ahead distance:** This defines the distance that a driver can see forward in order to react to other vehicles, either in front or to his side (within the same link). The minimum default value is set to zero whereas the maximum value can be 250 m. The study used the default setting.
2. **Observed vehicle**: This factor affects how well drivers in the network can predict other vehicle’s movements and react accordingly. By default, four vehicles are set to influence this element. The study used two observed vehicles in order to increase the possibility of lane change and to speed up the simulation run. Selecting higher values for this factor may slow down the simulation run and reduce the possibility of changing lane particularly when the traffic volume is high.
3. **Look back distance:** This defines the distance that a driver can see backwards in order to react to other drivers behind him (within the same link). The minimum default value is set to zero whereas the maximum value is set to 150 m. The study used the default settings.
4. **Average standstill distance (a_x):** This represents the average desired distance between stopped cars with a variation of ± 1 m and normal distribution around 0.0 m with a standard deviation of 0.3 m.
5. **Safety distance:** It is the distance between two vehicles traversing through the link .The additive part of the desired safety distance ($b_{x\_}add$) is set to 2 m, the multiplicative part of desired safety distance (bx_mult) is set to 3 m, and both affect the computation of the safety distance as shown in the following formula.

$$d = ax + bx \quad \text{A.1}$$

where (d) is the standstill distance and

$$b_x = (b_{x_{add}} + b_{x_{mult}} * z) * \sqrt{v} \quad \text{A.2}$$

where

v: is the vehicle speed [m/s]

z:is a value of range [0,1] which is normal distributed around 0.5 with a standard deviation of 0.15.

These parameters have a major influence on the safety distance and thus affect the saturation flow rate. By changing the default parameters, various saturation flows can be produced. In the present study, the default parameters were decreased to match the actual field capacity. Defining the saturation flow in VISSIM is not governed by a single value but rather it requires testing a set of driving behaviour parameters that yield the actual saturation flow. This step is done by trial and error. Since each link has its own geometric design and its own traffic characteristics, several types of links were created to replicate real behaviour over the study area as shown in Figure A.6.

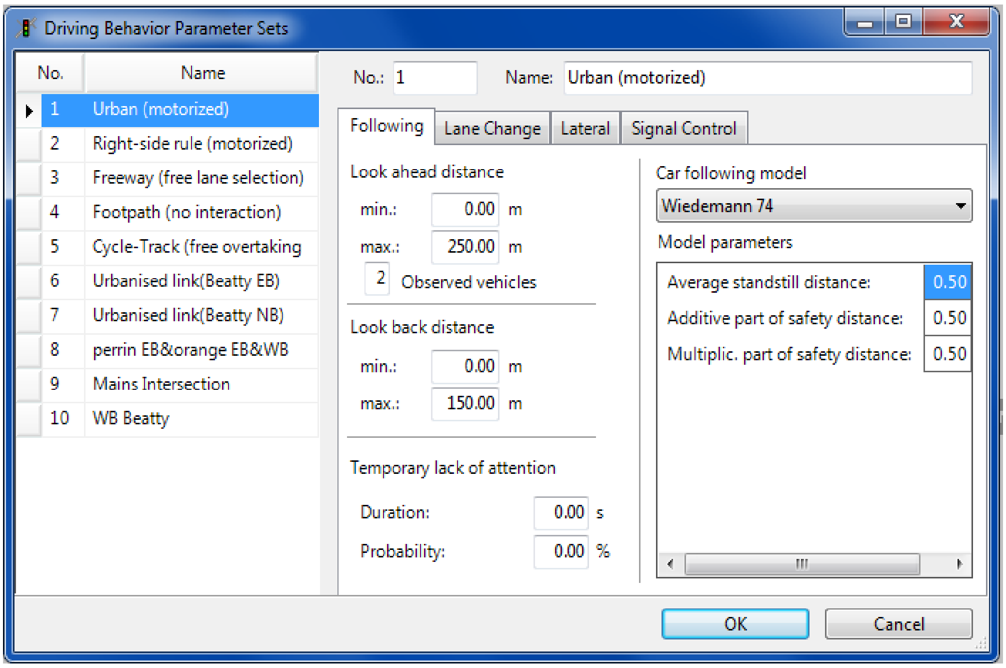


Figure A.6 Links behaviour

A.1.3.2 The lane change parameters

In VISSIM 2011, there are two types of lane change setting and each type is governed by a set of parameters:

1. **Necessary lane change** (in order to reach the next connector of a route): This parameter is explained in the previous section and is mainly based on the maximum acceptable deceleration for the leading vehicle and the trailing vehicle on the new lane. It depends on the distance to the emergency stop position of the next connector of the route. The present study used the VISSIM default values for acceptable deceleration to change lanes. The other two parameters (look back distance and emergency stop distance) that define the connector properties were changed to match the habits of the drivers in

the study area. The default, “look back distance” is 200 m but the study used a higher distance, primarily to capture the actual driver behaviour. Since the study was conducted during a congested period, it meant the road network was under high traffic demand, and the drivers were not able to change lanes within 200 m. Therefore, they were stopped at the emergency stop distance waiting for an opportunity to change lanes. The reasons could be that the spaces between vehicles was insufficient due to the high traffic volume, the distance between two successive signals being short, and the proportion of vehicles wanting to turn left/right being large. The initial simulation using the default values yielded a lot of vehicles trying to change lanes within the signalised area. This impedes the upstream flow and reduces the overall traffic flow performance which in turn leads to creating unrealistic traffic behaviour.

In order to overcome this situation the emergency stopping distance was increased to prevent vehicles interacting downstream of signalised intersections. The solution was successful where there was low traffic demand, a long distance between signals and a low percentage of turning vehicles. The second most popular solution which is used, is to increase the look back distance. Large distances lead to unrealistic interactions between vehicles far from the signal area. The study used different values in order to adjust the look back distances individually for each link allowing vehicles sufficient space to change lanes. Table A.1 shows a list of adjusted look-back distances for the mainstream links.

Table A.1 Modified look back distances for EB direction

Intersection No.	Direction	TH look-back	RT look-back	LT look-back
1	EB	350	400	600
2	EB	250	1000	250
3	EB	600	1500	-
4	EB	250	250	500
5	EB	250	1600	1200
6	EB	800	1500	1500

2. **Free lane change:** This parameter allowed vehicles to overtake in any travel lane. VISSIM periodically checks the desired safety distance for the vehicles intending to change lanes based on their speed and the speed of the trailing vehicle in the new lane. In addition to the parameters above, there are some other parameters that influence on the decision to change lanes which are:

   a. **Waiting time before diffusion:** This defines the maximum amount of time a vehicle can wait at the emergency stop position waiting for a gap to change lanes in order to stay in its route. Vehicles that not able to change lanes within this period will disappear from the network (diffusion) and an error file will be written explaining the time and location of the removal. The default value is 60s. The study used values of 20-40s instead of the default value for some mainstream links. This setting was adjusted to minimise the waiting time for vehicles stopped at emergency stop locations so as to reduce obstacles for the upstream traffic. This in turn minimises detrimental effects on intersection throughput and total travel time. Normally not many vehicles are deleted in contrast to the total flow. This solution is only recommended as a last stage resort after adjusting the look-back distances for all affected vehicles.

b. Safety distance reduction factor: The factor of 0.6 is specified in VISSIM by default. It means the safety distance is reduced by 40 % during lane changes. The current research reduced the value of this factor by 0.1 to 0.3 in order to increase the driver's opportunity for lane change further from the emergency stopping distance. This condition assumes a more aggressive driver behaviour by accepting shorter gaps in which to change lanes.

c. Maximum deceleration for cooperative braking: This factor has been set by default (-3 m/s^2) to harmonise vehicle deceleration during braking. Since the network layout consists of short links with high traffic demand, the vehicles will not have sufficient space to change lanes. For that reason, and in order to facilitate vehicles changing lanes smoothly, the current study increased the deceleration braking for the trailing vehicle to between -6 to -9 m/s^2 to allow a leading vehicle to change from an adjacent lane into the trailing lane. A higher value indicates more braking and thus an increased probability of changing lanes.

A.1.3.3 Signal control behaviour

This parameter is responsible for simulating vehicle behaviour in front of a signal control showing red or amber. There are two options set in VISSIM 2011, a continuous check model and a one-decision model. The continuous check model is the default option. The user can select Go (same as green) or Stop (same as red) to make the driving behaviour match the habits or rules of a particular country or region. In the case of the continuous check option, if the drivers are not able to stop safely at the stop line during the amber signal, vehicles will continue to progress through the intersection. In the case of the one-decision option, the drivers make their decision whether they stop or continue through the intersection based on a calculated probability. In other words, vehicles may continue through the amber light even though vehicles could stop within a safe distance, or vehicles may stop even though drivers could continue safely. The current study used the first option (a continuous check model) because it matched with the traffic rules in Australia.

A.2 Calibration process

The calibration process was divided into several steps. Firstly, GAR was coded in VISSIM and measures of effectiveness were selected to conduct the comparison. Secondly, an initial evaluation was conducted using VISSIM's default parameter values to identify unrealistic recurrent congestion. Thirdly, if VISSIM outputs were not compatible with real conditions for the selected measure of effectiveness, an examination of the key parameters was undertaken and calibration parameters were changed. Fourthly, determination of the minimum number of runs required to satisfy the statistical significance criteria.

A.2.1 Identification of the measure of effectiveness (MOE)

Vehicle distribution at signalised intersections, the total traffic demand along the main road network and the headway departure of the main signalised intersections are calibrated by comparison with field observation. Since VISSIM software has the ability to depict traffic operation visually, recurring congestion in VISSIM can be calibrated using this feature. Recurring congestion was observed during the survey at:

1. The link joining Beaudesert intersection and Beatty intersection in the WB lane.
2. The link joining Troughton intersection and Mains intersection in the EB lane.
3. The NB lane of the Mains intersection.

A.2.2 Initial iteration run

After coding the road network in VISSIM and selecting the measure of effectiveness (MOE) for the study area, the simulation should be tested with the default driver behaviour parameters using multiple runs associated with different seed numbers. This is recommended to identify potential errors that can occur during building the model. It also reveals how much deviation occurs in contrast to the selected MOE. The visual model output is checked to see whether the position of the recurrent bottlenecks in VISSIM simulation match or consistent with the defined congested links in reality. Five multiple runs with different random seeds were implemented initially. In addition, the initial run is crucial to identifying the minimum number of repetitions required to reach the prescribed level of accuracy.

A.2.3 Error messages

Customarily, error messages are written during the initial simulation execution to notify the user. Most of these errors are related to VISSIM network coding or to the VISSIM link capacity. Examples are a routing decision that is placed too close to the start of a connector or capacity problems resulting in a queue outside the network at the end of the defined time interval. Some of these errors can be solved either by changing the location of physical facilities or by changing the driving behaviour parameters if there is a link capacity problem. The user needs to solve the warning messages and repeat the simulation execution until all error messages disappear. If the VISSIM outputs are not compatible qualitatively with the MOE, default parameters need to be reassigned. The range of acceptance is limited by the validation criteria used to accept or to refuse the VISSIM results. Figure A.7 illustrates the main calibration steps.

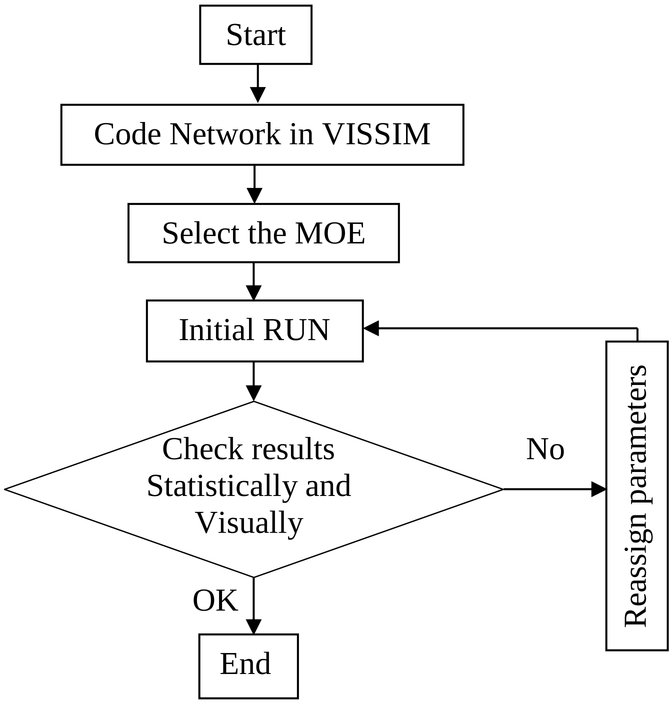


Figure A.7 Calibration Process

A.2.4 Visual evaluation

The purpose of using the 3D mode is to represent the field conditions as closely as possible. The model cannot be considered calibrated if the animations are unrealistic. For example, a parameter set may be statistically acceptable but the animations may not reflect reality. The user should reject such a model.

Animation in the model showed recurrent congestion at locations in which there was no congestion in reality such as in the eastbound lanes at intersection 3 and the northbound lanes at intersection 5. There were two causes for this; firstly, the signal offset was set incorrectly during the designation of the signal control. Most vehicles arrive at the intersection at the beginning of the red interval and this contributes to propagating a long queue of vehicles over the time interval. This was overcome by adjusting the signal offset for that direction. Secondly, blockage of a left/right turn lane by the through traffic in the case of a share laned configuration may hinder the progress of the exiting vehicles. Several adjustments were applied to the routing decision detectors to correct this condition.

Some unrealistic driver behaviour appeared near the approaches to intersections and at locations where two links or connectors overlapped, i.e. drivers may not always respond to the traffic signal and continue driving even when a signal turns red. This occurs because the signal ahead is setup in the connector boundary that connects two links rather than in the link boundary. Changing the position of the signal ahead to inside the link boundary solved the problem.

In the case of saturated conditions, vehicles on the main road may block the intersection due to a long propagating queue. This obstructs the flow of the minor road. Nevertheless, vehicles wanting to travel on the minor road continue through the intersection even though it may be filled with queued vehicles. To overcome this, a conflict area mode is utilised to manage the traffic movement at the intersection. Priory rules apply to all situations where vehicles on different links and connectors can recognize each other.

A.2.5 Determination of the minimum number of runs

After eliminating most of the physical errors that appear in the initial runs, the user needs to determine the minimum number of runs required to meet the prescribed level of statistical significance. Conceptually, the micro simulation tools employ different random seeds to attempt to replicate and anticipate local traffic conditions. The number assigned by the random seeds affects realization of the stochastic quantities in VISSIM, such as inlet flows and vehicle capabilities. If the model is run with a constant random seed number, the same sequence of random numbers, and all model inputs remain unchanged, it will yield the same results. Changing the seed number changes the stochastic behaviour of simulated vehicles and produces different final simulation results.
Traffic engineers and simulation users usually perform several model runs for each set of traffic conditions (each with a different random number seed) in order to replicate the randomness experienced in reality. The results are averaged with the expectation that the mean imitates the average conditions found in the field.
In order to know the minimum number of runs to prove statistical significance, the statistical parameters have to be defined, i.e. the standard deviation, the standard level of confidence, and the desired length of the confidence interval. However, the standard deviation and the mean value cannot be obtained directly until after executing the simulation. For that reason, initial runs are required to estimate these parameters. The estimated results utilise an initial statistical parameter to identify the required number of simulation runs. The VISSIM output that represents the selected MOE parameters (traffic volumes, volume distributions, saturated headway) is be calibrated against actual traffic conditions. The following steps represent the hierarchical determination of the number of simulation runs required.

A.2.5.1 Initial estimation of standard deviation

Since the study area was primarily coded by VISSIM for research purposes, initial runs were implemented to estimate the standard deviation for traffic volume. The following equation was used to calculate the preliminary standard deviation.

$$S = \sqrt{\frac{\sum (x - x^{\circ})^2}{N - 1}} \qquad \text{A.3}$$

where,
S = standard deviation for a selected MOE,
x= each value in the simulated data,
x°= mean value of the simulated MOE output and
N= number of runs

In order to obtain the initial standard deviation for traffic volume, five runs were performed. This preliminary estimation needs to be redone later to check whether additional runs are needed or not. The number of additional runs is determined based on the desired mean value.

A.2.5.2 Identification of desired confidence level

The confidence level is used to indicate the reliability of estimated data. This reliability is based on the probability that the actual mean lies within a specified range of values. The common level of confidence used in micro simulation calibration is 90 % or 95 %. If the user intends to use higher levels of confidence, more runs are required. The present study used a 95% level of confidence.

A.2.5.3 Determination of confidence interval

A confidence interval (*CI*) is an interval estimate of simulated results within which the actual mean value should lie in order to be accepted or refused**.** Factors that affect the confidence interval size are the size of the sample, the confidence level, and the variability of the results. Normally a larger sample size will produce a better-simulated estimate of results. Confidence intervals are determined from the variance of the estimated output. If the results have a significant variance, a bigger range may be recommended. In the current research, the range of the confidence interval is approximately 95 % and an accepted error is less than 5 % from the actual mean value.

A.2.5.4 Estimation of minimum runs required

As explained earlier, the user cannot determine the number of runs in advance unless they know a mean or other statistical value. Therefore, an initial set of runs was executed to find the mean and the standard deviation as a first step. Five simulation runs with different seed number was executed as a first step in the present research. In the second step, the user needs to identify the critical value of the t-distribution at a significance level (α) with the number of degrees of freedom. The third step identifies the accepted error that is specified as a percentage of the actual mean value. In order to determine the minimum number of repetitions, the following equation is used:

$$CI_{(1-\alpha)\%} = 2 * t_{(1-\alpha),N-1} * \frac{S}{\sqrt{N}} \qquad \text{A.4}$$

where

$CI_{(1-\alpha)\%}$ = (1-α) % confidence interval for the true mean, where α equals the probability of the true mean not lying within the confidence interval,

$t_{(1-\alpha),N-1}$ = Student’s t-statistic for the probability of a two-sided error summing to α with *N-1* degrees of freedom, where *N* equals the number of repetitions and

S = standard deviation for the model results.

An initial five runs with different random seeds were used to determine the total simulated traffic volume among the signalised intersections for EB and WB lanes. Table A.2 and Table A.3 show the total simulated volume for the mainstream flow for EB and WB lanes respectively.

Table A.2 Total simulated traffic volume for the EB lanes (veh/h)

#Run	#Seed	Int. 1	Int. 2	Int.3	Int.4	Int.5	Int.6
1	100	2089	1881	1510	1500	1743	1651
2	110	2090	1852	1522	1527	1716	1611
3	120	2090	1791	1452	1462	1722	1623
4	130	2090	1875	1537	1538	1764	1654
5	140	2090	1805	1515	1507	1755	1596
Standard deviation		0.45	40.84	32.49	29.30	20.68	25.19
Simulation average		2089.8	1840.8	1507.2	1506.8	1740	1627

Table A.3 Total simulated traffic volume for the WB lanes (veh/h)

#Run	#Seed	Int. 1	Int. 2	Int.3	Int.4	Int.5	Int.6
1	100	1463	1221	1212	1109	988	1166
2	110	1583	1255	1235	1129	996	1180
3	120	1519	1254	1244	1122	1007	1222
4	130	1533	1307	1281	1189	1023	1300
5	140	1490	1212	1192	1089	1003	1229
Standard deviation		46.74	36.61	33.09	37.55	14.09	52.44
Simulation average		1517.6	1249.8	1232.8	1127.6	1003.4	1219.4

According to the initial simulation runs, the mean and standard deviation of the total travel demand for the EB and the WB lanes was counted. The calculation of standard deviation was based on equation A.1. After calculating this, the required number of simulation runs were calculated. A sample of calculation can be explained as following for the EB lanes of the Beatty and Beaudesert intersections:

Sample calculation one: EB lanes of the Beatty intersection.
Initial number of runs = 5, Level of confidence = 95%, α = 1- 0.95 = 0.05.
$t\ (1-\alpha), N-1 = t(1-0.05/2),5-1$ = 2.78
x°=2089.8

$$S = \sqrt{\frac{\sum (x - x^\circ)^2}{N-1}} = 0.45$$

In order to obtain the minimum required number of simulation runs, the confidence interval should be computed for each simulation run using equation A.2. Then simple comparison between the desired confident interval, which is specified by 5% of the average simulation, and the confidence interval (*CI*), is needed to identify the required number of runs. As indicated in Table A.4 the iterative process of simulation yields different numbers for *CI* because it depends on the value of N and the t-statistic value, both of which are changeable for each run. If the value of *CI* for a particular number of runs is less than or equal to the value of 5% of the average simulation, this will be the minimum number of runs required for the simulation. The calculation of *CI* starts with number 2 because there is no t-test value for run number 1 (the number of degrees of freedom will be zero). For example, run number 2 for the EB lane of the Beatty intersection yielded a *CI* value of 8 meanwhile the average simulation for this iterative process is 104 veh/h. That means the minimum required number of simulation runs for this part of the network is 2.

Sample calculation two: EB lanes of Beaudesert intersection.
Initial number of runs = 5, Level of confidence = 95%, α = 1-0.95 = 0.05.
$t\ (1-\alpha), N-1 = t(1-0.05/2),5-1$ = 2.78
x°=1840.8

$$S = \sqrt{\frac{\sum (x - x^{\circ})^2}{N-1}} = 40.84$$

Table A.4 shows that 5% of the average simulation for the EB lane of Beaudesert intersection is 92 veh/h while all the *CI* values for the five runs, at a level of confidence of 95%, are above the threshold value (*CI* > 5 % AVG). This means the simulated results need more than five iterations to satisfy the threshold accuracy level. Thus, equation A.2 was used again to estimate the required number of runs for this part of network. Most the EB lanes in Table 4.3 required less than five runs except the EB lanes for the Beaudesert and Perrin intersections which required more than five runs.

Table A.4 Estimation of minimum required number of simulation runs for EB

Beatty					Beaudesert				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	8	104	2	2	12.71	734	92	> 5
3	4.3	2			3	4.3	202		
4	3.18	1			4	3.18	129		
5	2.78	1			5	2.78	101		
Perrin					Orange Grove				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	584	75	> 5	2	12.71	526	75	5
3	4.3	161			3	4.3	145		
4	3.18	103			4	3.18	93		
5	2.78	80			5	2.78	72		
Troughton					Mains				
N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #	N	$t_{(0.05,N-1)}$	CI	5 %AVG	Run #
2	12.71	371	87	4	2	12.71	452	81	4
3	4.3	102			3	4.3	125		
4	3.18	65			4	3	80		
5	2.78	51			5	2	62		

Table A.5 shows the number of simulation runs initial required for the WB lanes. As can be seen from this table, five simulation runs are insufficient to achieve calibration results within a 95% confidence interval and a 5% significance level. The process of finding the minimum number of repetitions is merely an aid until the ultimate calibration parameters are planned. Therefore, equation A.2 was used again to determine the final set of parameter values which produce the best calibration. The study area has six signalised intersections with four approaches (NB, SB, EB, and WB) except intersection 3 which has three approaches. Each approach was investigated independently in order to identify the relevant number of runs required. Table 4.3 and Table 4.4 showed that there is variation in determining the minimum number of simulation runs along the EB and WB lanes. Some links achieve the threshold criteria using 2, 4 or 5 runs and others need more simulation runs to achieve the threshold criteria. This was caused by the stochastic behaviour of traffic flow and the flow distribution that causes a high standard deviation. This drastically affects the required number of runs. In addition, the study concentrated only on the peak hours (volume reached the capacity). This caused an increase in variation thereby requiring more runs. The current study used the maximum number of estimated simulation runs along the road network because the calibration of the network required different numbers of simulation runs due to variation in the traffic flow characteristics and the physical layout. For purpose of calibrating the whole network, 50 simulation runs were executed to meet the statistical requirements.

Table A.5 Estimation of minimum required number of simulation runs for WB

<table>
<tr><th colspan="5">Mains WB</th><th colspan="5">Troughton WB</th></tr>
<tr><td>N</td><td>$t_{(0.05,N-1)}$</td><td>CI</td><td>5 %AVG</td><td>Run #</td><td>N</td><td>$t_{(0.05,N-1)}$</td><td>CI</td><td>5 %AVG</td><td>Run #</td></tr>
<tr><td>2</td><td>12.71</td><td>943</td><td rowspan="4">61</td><td rowspan="4">> 5 Runs</td><td>2</td><td>12.71</td><td>253</td><td rowspan="4">50</td><td rowspan="4">4 Runs</td></tr>
<tr><td>3</td><td>4.3</td><td>260</td><td>3</td><td>4.3</td><td>70</td></tr>
<tr><td>4</td><td>3.18</td><td>167</td><td>4</td><td>3.18</td><td>45</td></tr>
<tr><td>5</td><td>2.78</td><td>130</td><td>5</td><td>2.78</td><td>35</td></tr>
<tr><th colspan="5">Orange Grove WB</th><th colspan="5">Perrin WB</th></tr>
<tr><td>N</td><td>$t_{(0.05,N-1)}$</td><td>CI</td><td>5 %AVG</td><td>Run #</td><td>N</td><td>$t_{(0.05,N-1)}$</td><td>CI</td><td>5 %AVG</td><td>Run #</td></tr>
<tr><td>2</td><td>12.71</td><td>675</td><td rowspan="4">56</td><td rowspan="4">> 5 Runs</td><td>2</td><td>12.71</td><td>595</td><td rowspan="4">62</td><td rowspan="4">> 5 Runs</td></tr>
<tr><td>3</td><td>4.3</td><td>186</td><td>3</td><td>4.3</td><td>164</td></tr>
<tr><td>4</td><td>3.18</td><td>119</td><td>4</td><td>3.18</td><td>105</td></tr>
<tr><td>5</td><td>2.78</td><td>93</td><td>5</td><td>2.78</td><td>82</td></tr>
<tr><th colspan="5">Beaudesert WB</th><th colspan="5">Beatty WB</th></tr>
<tr><td>N</td><td>$t_{(0.05,N-1)}$</td><td>CI</td><td>5 %AVG</td><td>Run #</td><td>N</td><td>$t_{(0.05,N-1)}$</td><td>CI</td><td>5 %AVG</td><td>Run #</td></tr>
<tr><td>2</td><td>12.71</td><td>658</td><td rowspan="4">62</td><td rowspan="4">> 5 Runs</td><td>2</td><td>12.71</td><td>840</td><td rowspan="4">76</td><td rowspan="4">> 5 Runs</td></tr>
<tr><td>3</td><td>4.3</td><td>182</td><td>3</td><td>4.3</td><td>232</td></tr>
<tr><td>4</td><td>3.18</td><td>116</td><td>4</td><td>3</td><td>149</td></tr>
<tr><td>5</td><td>2.78</td><td>91</td><td>5</td><td>2</td><td>116</td></tr>
</table>

APPENDIX B
Evaluation of VSL Applications on Intersection 6

B.1 Preface

This appendix illustrates the effect of using VSL control on the performance of intersection 6 during the peak hour. Signalised intersection indicators and traffic flow indicators for the EB direction were studied during this period. Average queue length, average delay, stopped delay and the average numbers of stops were tested before and after applying VSL. Additionally average travel speed, average flow and average density were evaluated for the congested segment.

B.2 VSL control management

Three speed limit ranges 30-35, 40-45 and 48-58 km/hr were applied along the EB lanes at intersection 6. The aim of these control speeds was to control the flow towards intersection 6 during the peak hour. The locations of these control (CS) speeds are identified in Table B.1 and shown in Figure B.1.

Table B.1 Identification of controlled speed for the EB lane

Control Speed (CS) #	Location	Controlled Distance (m)	Distance from intersection 6 (m)
5	Between intersection 4 & 5	1050	2600
6	Between intersection 2 & 3	986	4592
7	Between the EB traffic inlet & intersection 1	998	5993
8	Between intersection 3 & 4	707	3586

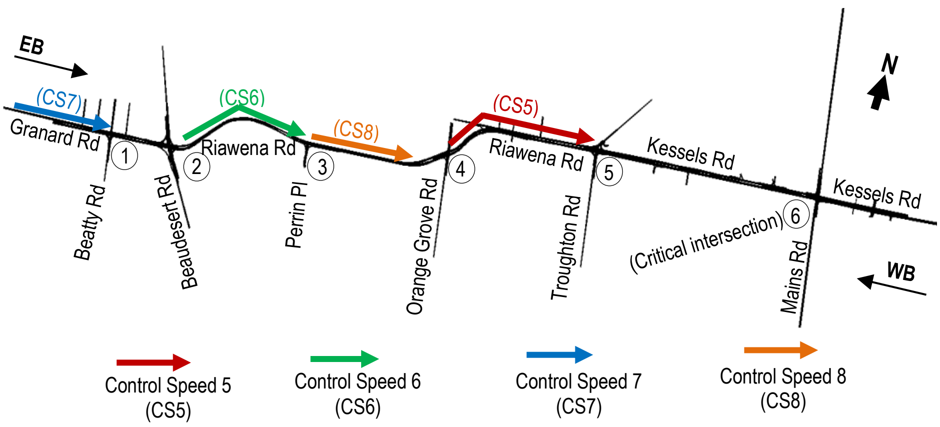


Figure B.1 Locations of controlled speed towards the EB direction

The control speeds were activated for four different time periods during the rush hour to find out how they affected VSL efficiency. Table B.2 indicates the activation time for all controlled speed.

Table B.2 Speed control activation system

Control Speed (CS) #	Speed limits (km/hr)	Activation time (sec)
5	(30-35),(40-45),(48-58)	(600-4200)
	(30-35),(40-45),(48-58)	(1200-4200)
	(30-35),(40-45),(48-58)	(1800-4200)
	(30-35),(40-45),(48-58)	(2400-4200)
6	(30-35),(40-45),(48-58)	(600-4200)
	(30-35),(40-45),(48-58)	(1200-4200)
	(30-35),(40-45),(48-58)	(1800-4200)
	(30-35),(40-45),(48-58)	(2400-4200)
7	(30-35),(40-45),(48-58)	(600-4200)
	(30-35),(40-45),(48-58)	(1200-4200)
	(30-35),(40-45),(48-58)	(1800-4200)
	(30-35),(40-45),(48-58)	(2400-4200)
8	(30-35),(40-45),(48-58)	(600-4200)
	(30-35),(40-45),(48-58)	(1200-4200)
	(30-35),(40-45),(48-58)	(1800-4200)
	(30-35),(40-45),(48-58)	(2400-4200)

Various VSL management strategies were applied over the EB direction of Griffith arterial road U20 to find out which scenario was the most efficient at improving the mobility and safety at intersection 6 (Mains intersection). Table B.3 shows the VSL scenarios that were proposed along the EB direction of Griffith arterial road U20. In Scenario number 1, control speed 5 (CS5) was ON while all other controls towards the critical intersection were OFF. The scenario was tested for different activation periods such as 600s-4200s, 1200s-4200s, 1800s-4200s, and 2400s-4200s. The first period 600s-4200s means speed control was immediately activated at the initiation of congestion. The second period 1200s-4200s means the control speed was activated 1200s after congestion started to form and so on for the other periods. Scenario number 2 had control speed numbers 5, 7 and 8 OFF while speed control 6 was ON. This scenario also was tested for different activation periods. Scenario number 3 had control speeds 5, 6 and 8 OFF and control speed 7 ON for different activation periods. Scenario number 4 had control speeds 5 and 7 OFF and control speeds 6 and 8 ON. In this scenario, speed control 6 was synchronised with control speed 8 during the whole activation period.

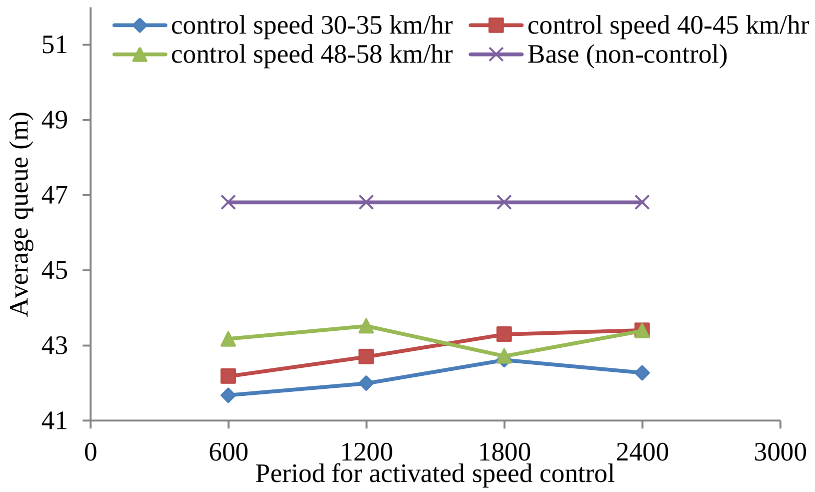
Table B.3 VSLs Scenarios

Scenario #	Controlled Speed (CS) #			
	CS5	CS6	CS7	CS8
1	ON	OFF	OFF	OFF
2	OFF	ON	OFF	OFF
3	OFF	OFF	ON	OFF
4	OFF	ON	OFF	ON

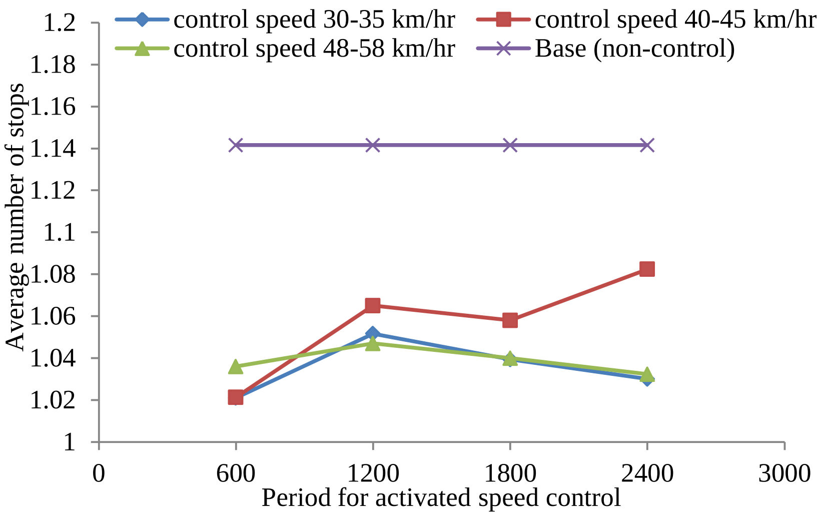
B.3 Evaluation of scenario 1

B.3.1 Evaluation of intersection parameters

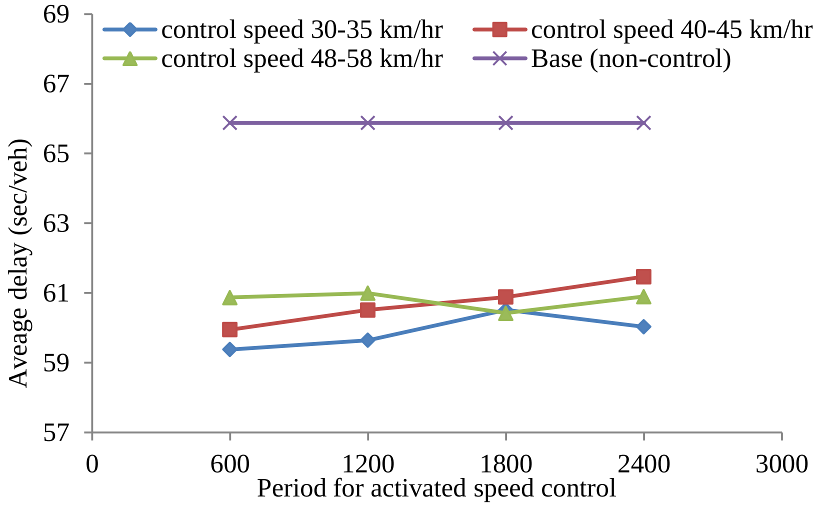
The effect of SC5 on the characteristics of intersection 6 were measured by the average queue length, average delay, average stopped delay and average number of stops as shown in Figure B.2 (a), (b), (c), and (d). This figure shows the differences between the performance of intersection 6 without speed control (base line condition) and with various speed controls and activation times. Activating VSL during congested periods resulted in a significant improvement in the traffic performance of intersection 6 because the base line condition had the lowest traffic performance when compared with VSL applications under all proposed scenarios. Control speed ranges of 30-35 and 40-45 km/hr resulted in the largest improvement compared to the base line condition. Activation of CS5 at different time periods during the peak hour make little variation in the level of improvement for all intersection parameters. This may be due to the proposed speed limit ranges of 40-45 and 48-58 km/hr being less than the field speed of the link joining intersections 4 and 5 and close to the speed limit range of 30-35 km/hr. CS5 had the most effect when activated for 600-4200s.



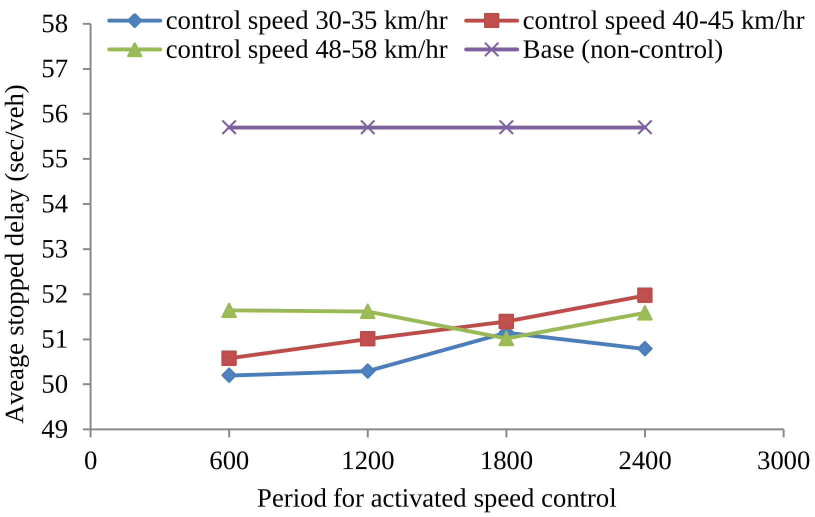
(a)



(b)



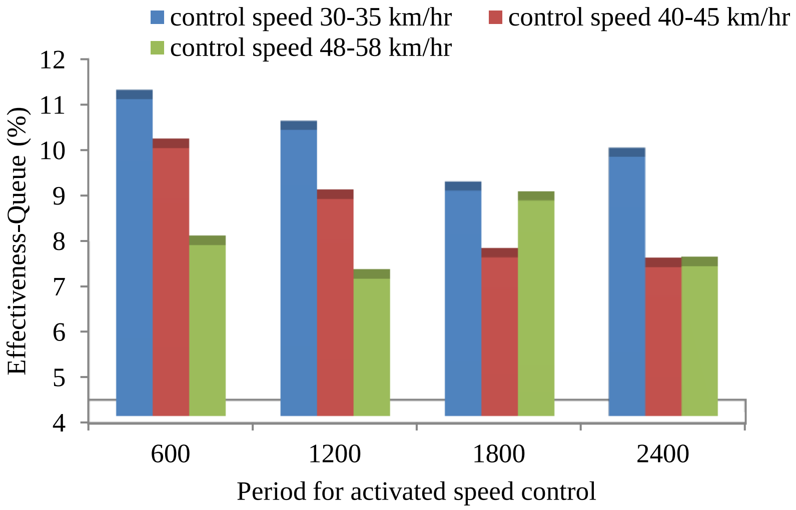
(c)



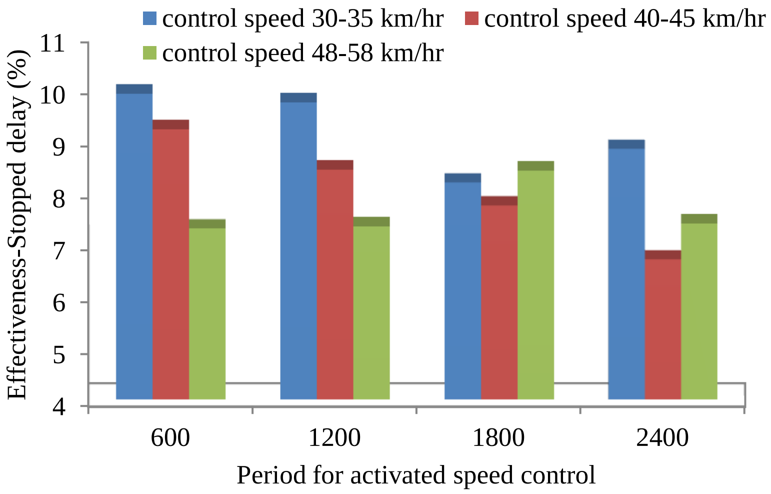
(d)

Figure B.2 The simulation of scenario 1 before and after VSL application

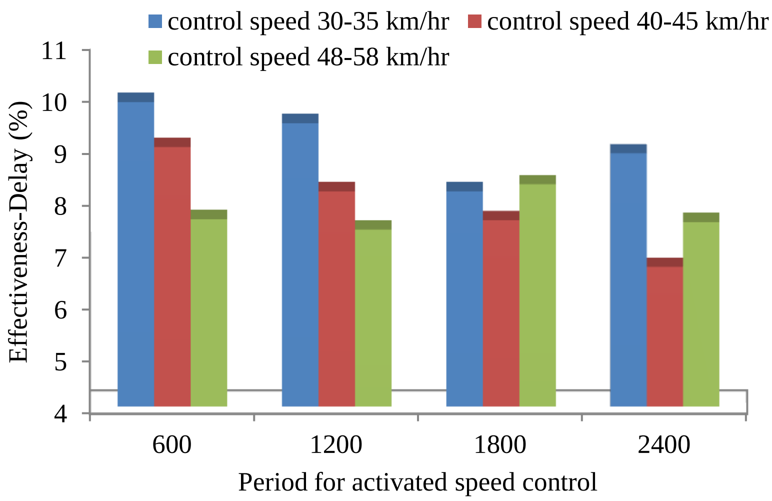
The effect of applying scenario 1 on intersection performance indicators is shown in Figure B.3 (a), (b), (c), (d). The figure shows a 9-11 % improvement in average queue length and 8-10 % improvement in average number of stops, average delay, and average stopped delay. Scenario 1 achieved the highest level of traffic operation when SC5 was activated at the start of congestion (600s-4200s) with a speed limit range of 30-35 km/hr. These results may be due to CS5 being close to the critical intersection. This required more activation time and more constraint of the upstream speed when compared with other sub-scenarios.



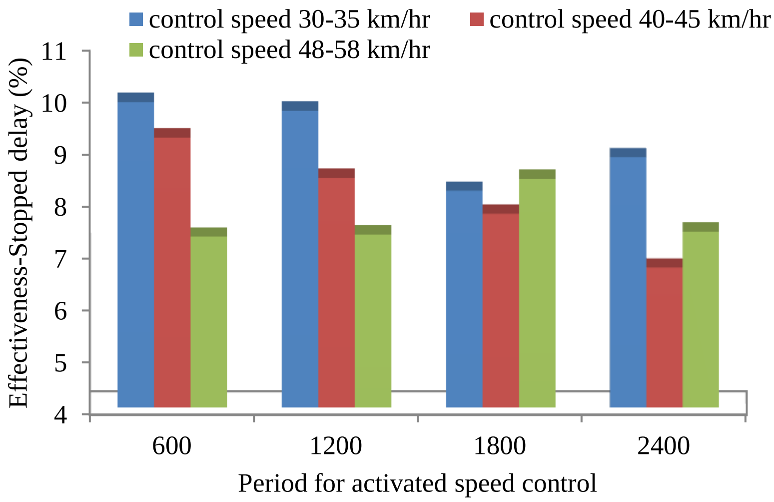
(a)



(b)



(c)

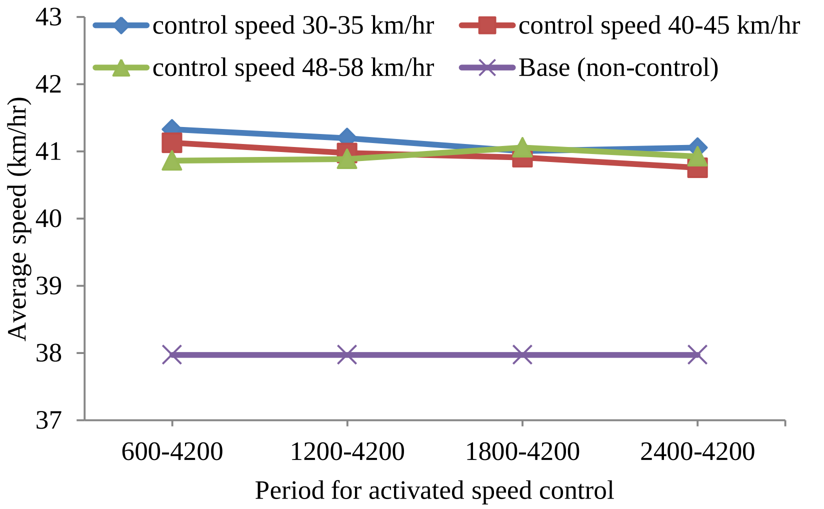


(d)

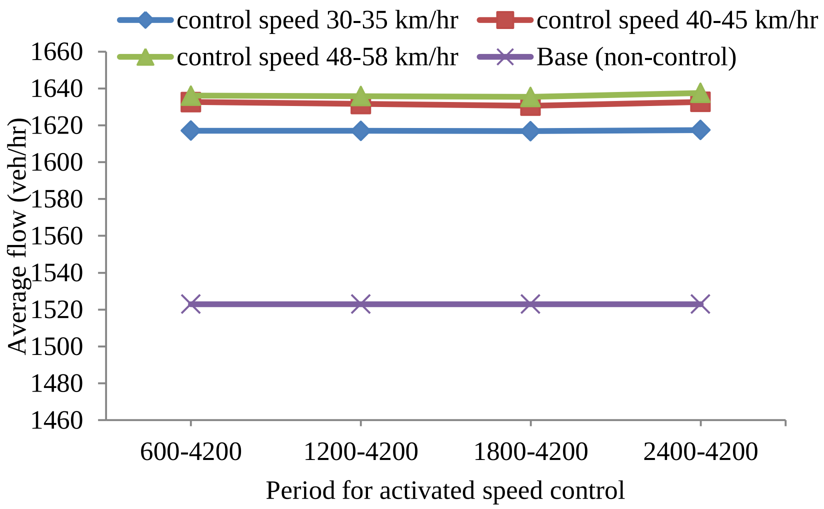
Figure B.3 The efficiency of scenario 1 on the signalised intersection characteristics

B.3.2 Evaluation of macroscopic traffic characteristics

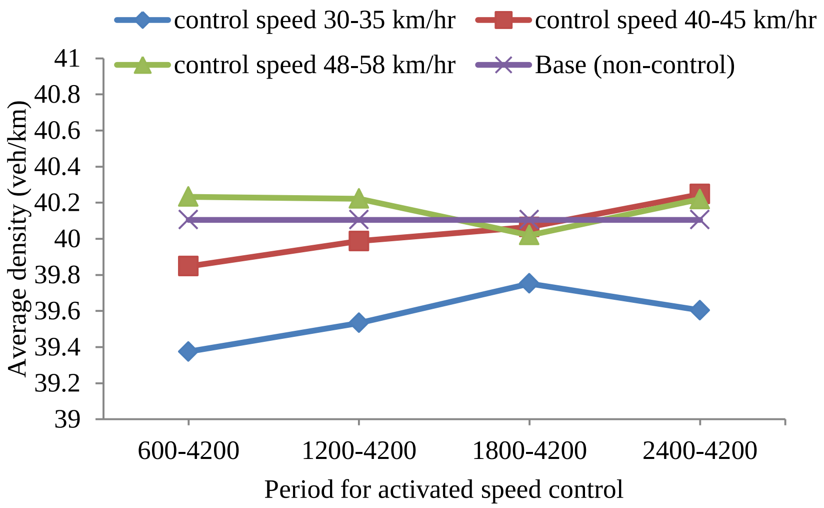
The second part in evaluating Scenario 1 was to examine the macroscopic characteristics of traffic flow in terms of average travel speed, average flow and average traffic density. Figure B.4 (a), (b) and (c) show the impact of scenario 1 for several situations. The figures show that scenario 1 resulted in large improvements in the average speed and average flow compared with base line conditions. Traffic density was sustained in spite of average speed and flow rate increasing after applying Scenario 1. Various VSL activation times did not produce much difference in results. This was because they were applied during the peak hour when all variables were nearly constant such as the flow. The control speed limits used were close to the field speed over the peak hour.



(a)



(b)

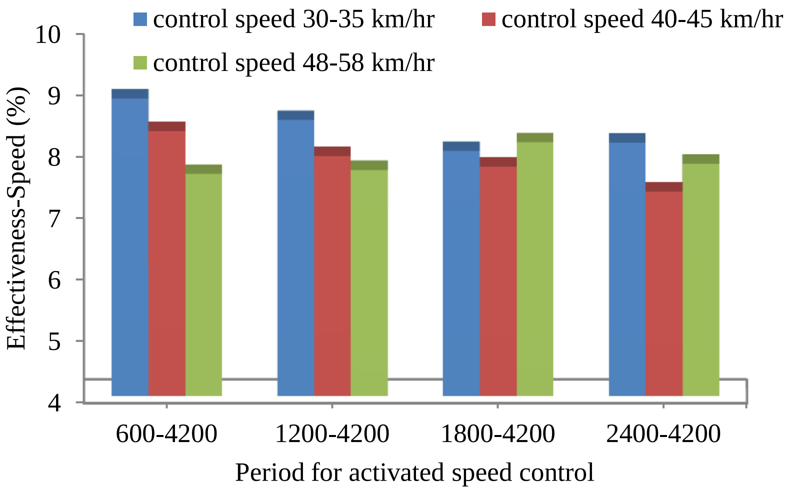


(c)

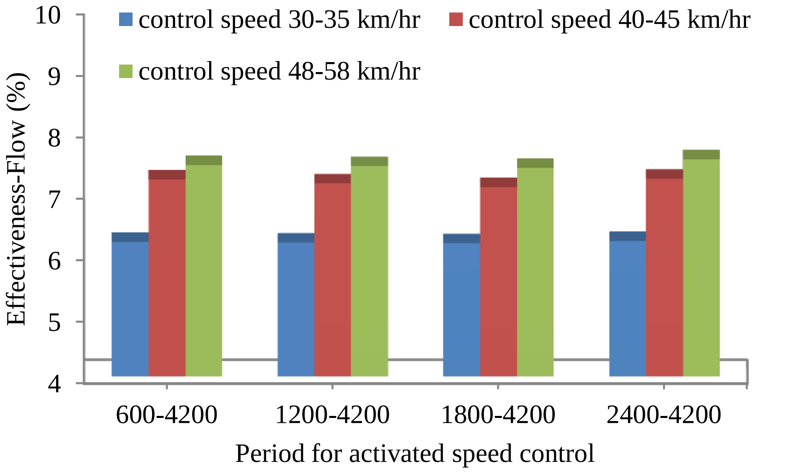
Figure B.4 The effect of Scenario 1 on the macroscopic traffic characteristics

Improvements in efficiency due to scenario 1 are shown in Figure B.5 (a), (b), (c). There was a 7-9 % increase in the average speed, 6-8 % increase in flow productivity and 0-2 % enhancement in traffic density. The negative results refer to increase in traffic density due to the speed limit ranges of 40-45 and 48-58 km/hr resulting in increased flow thereby increasing traffic density.

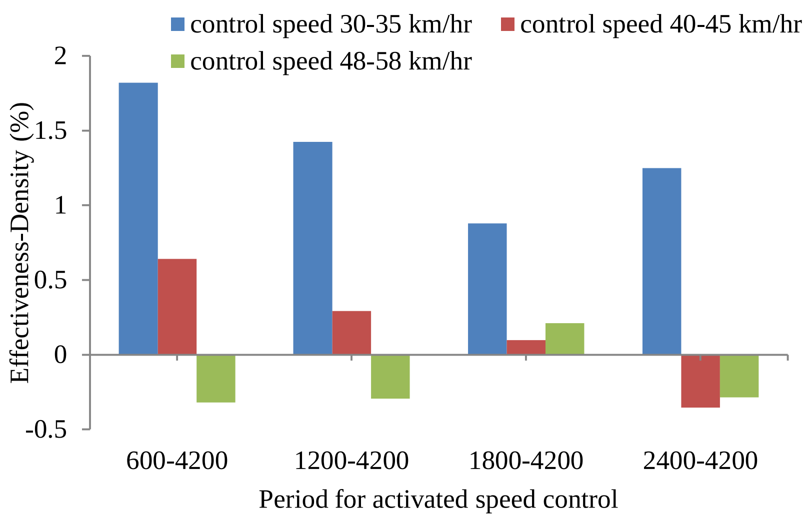
The results show that VSL increases the average speed and average flow rate of the congested link and also maintains and enhances the traffic density during the congestion period. All proposed VSL scenarios produced an enhancement (less than 9 %) in speed and flow and (less than 2 %) in traffic density.



(a)



(b)



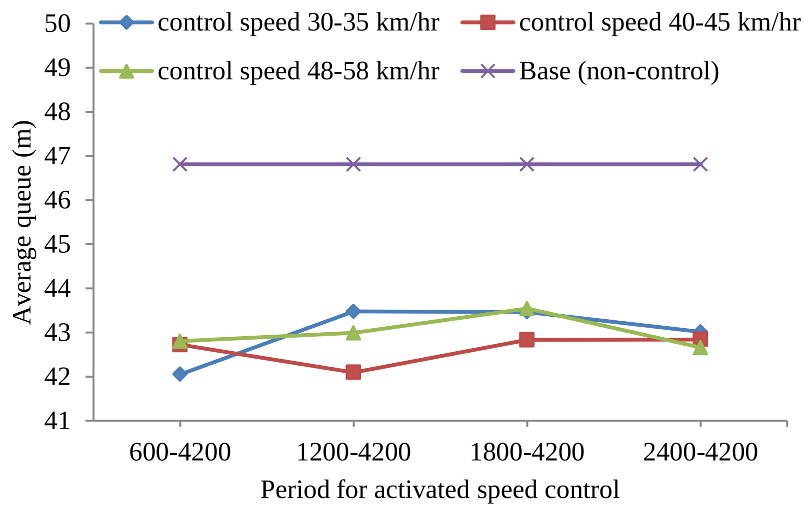
(c)

Figure B.5 The efficiency of Scenario 1 in terms of macroscopic traffic aspect

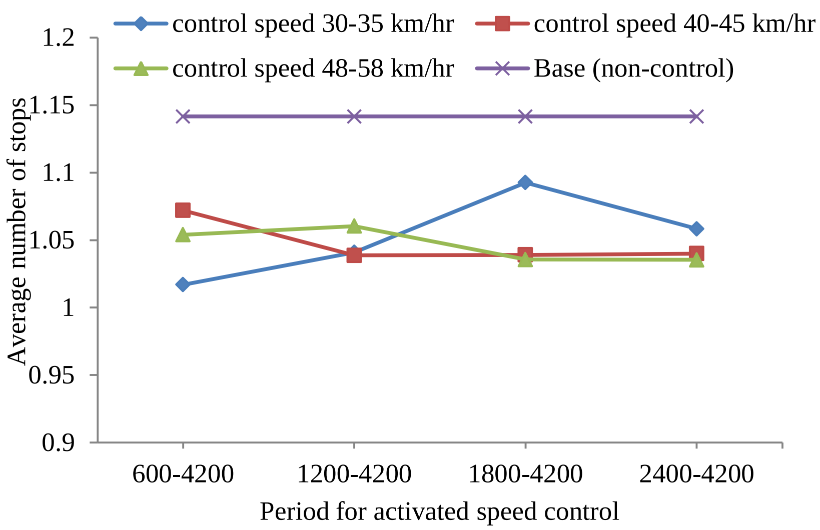
B.4 Evaluation of Scenario 2

B.4.1 Evaluation of intersection parameters

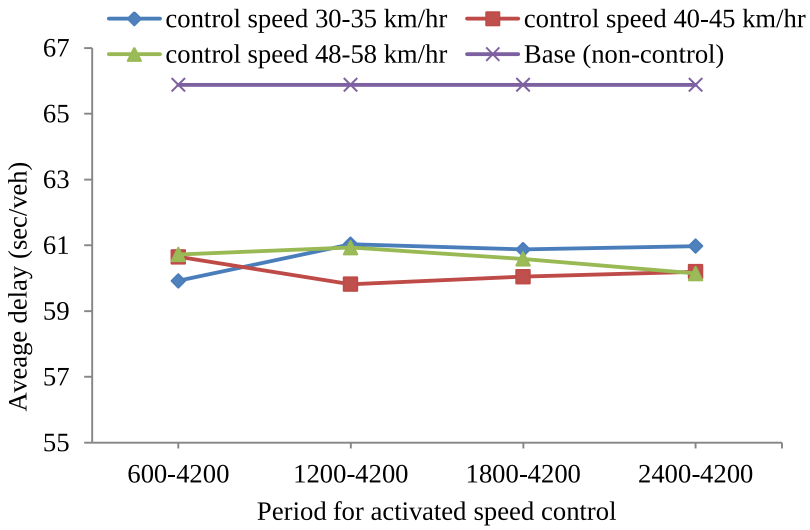
In Scenario 2, the position of CS6 is located distance from SC5. This was done to find out the effect of VSL when introduced far from the congested link. Figure B.6 (a), (b), (c), and (d) show the changes in the properties of intersection 6 before and after applying VSL. All intersection parameters (queue length, number of stops, delay, and stopped delay) were improved after applying VSL. The base line condition was the worst scenario.



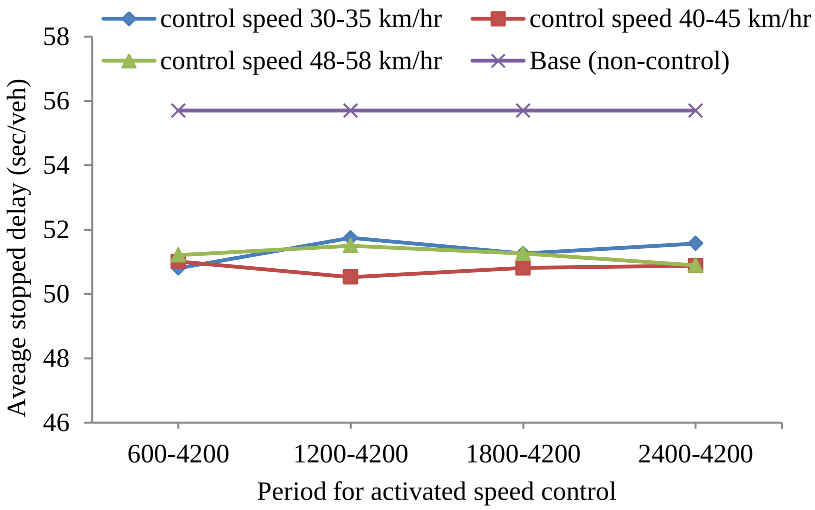
(a)



(b)



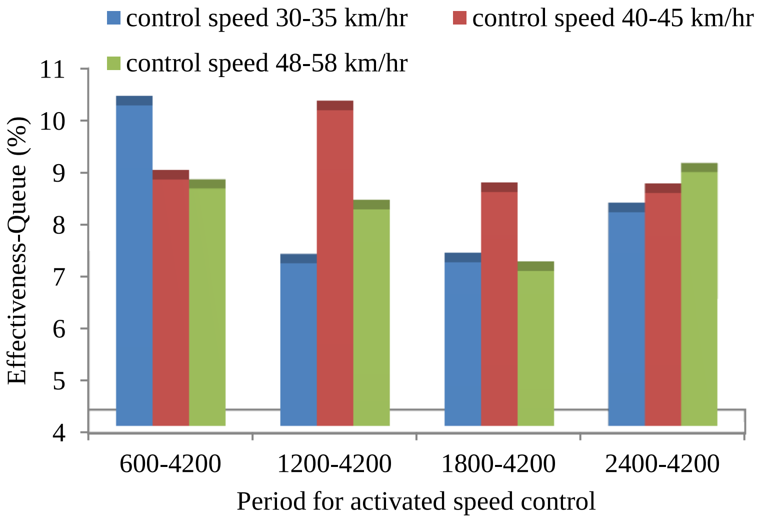
(c)



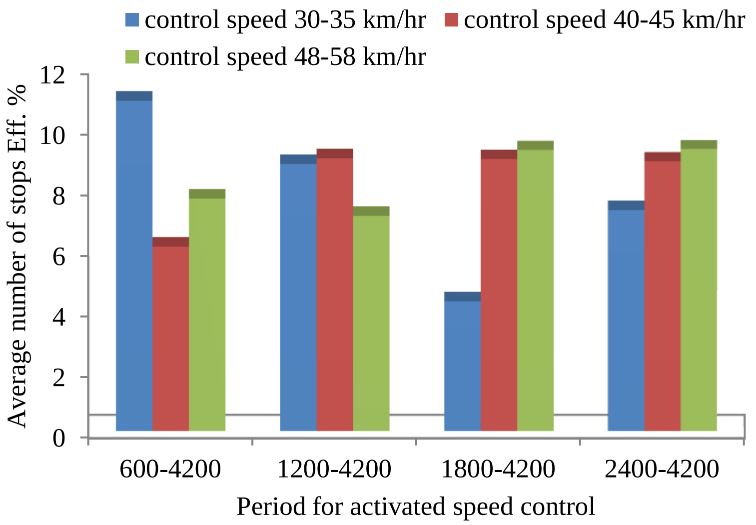
(d)

Figure B.6 The simulation of scenario 2 before and after VSL application

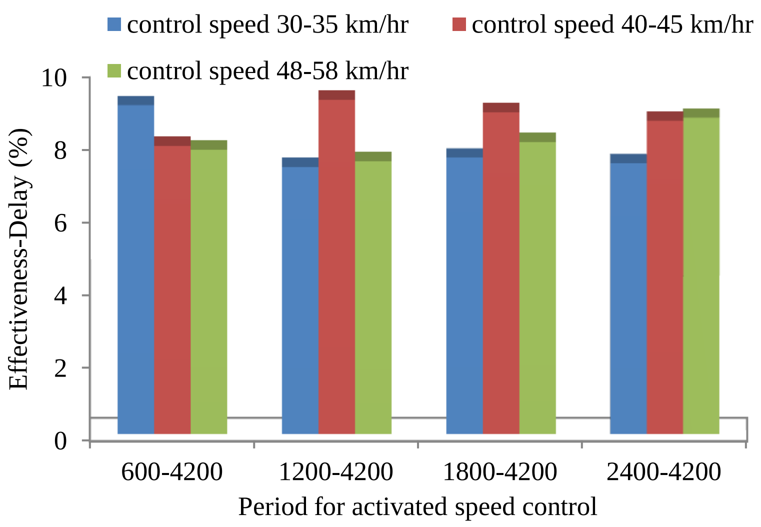
The improvements in efficiency due to scenario 2 are shown in Figure B.7 (a), (b), (c), (d). These include a 7-11% improvement in average queue length, 4-11% improvement in average number of stops, 7-9% improvement in average delay, and average stopped delay. The reduction in queue length and number of stops increased traffic safety by 11%. It also contributed to raising the level of service at the intersection by 9% due to the reduction in delay and stopped delay.



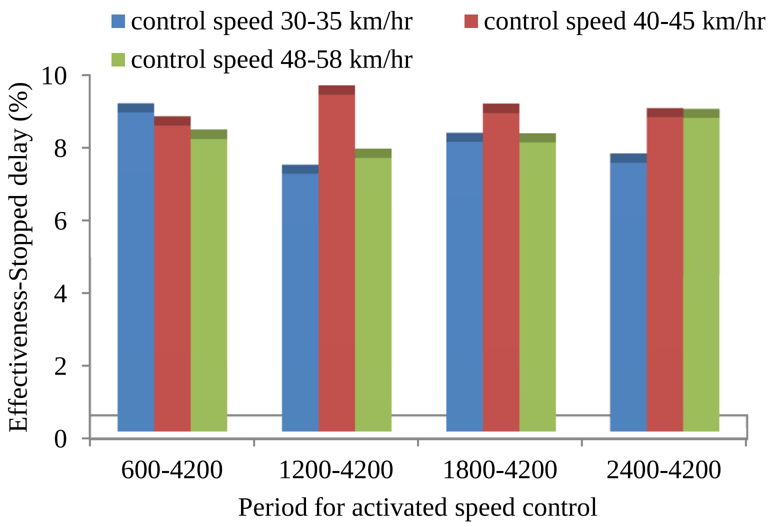
(a)



(b)



(c)



(d)

Figure B.7 The efficiency of scenario 2 on the signalised intersection properties

B.4.2 Evaluation of macroscopic traffic characteristics

The macroscopic traffic characteristics for the EB direction of intersection 6 under Scenario 2 were investigated and the results are shown in Figure B.8 (a), (b), (c). Speed and flow parameters improved after applying VSL. Travel speed increased from 38 to 41 km/hr and flow increased from 1522 to 1636 veh/hr by using VSL. Traffic density showed little change after applying VSL. In some cases, traffic density showed some increase due to increasing flow and speed for all proposed speed limits and activation times.

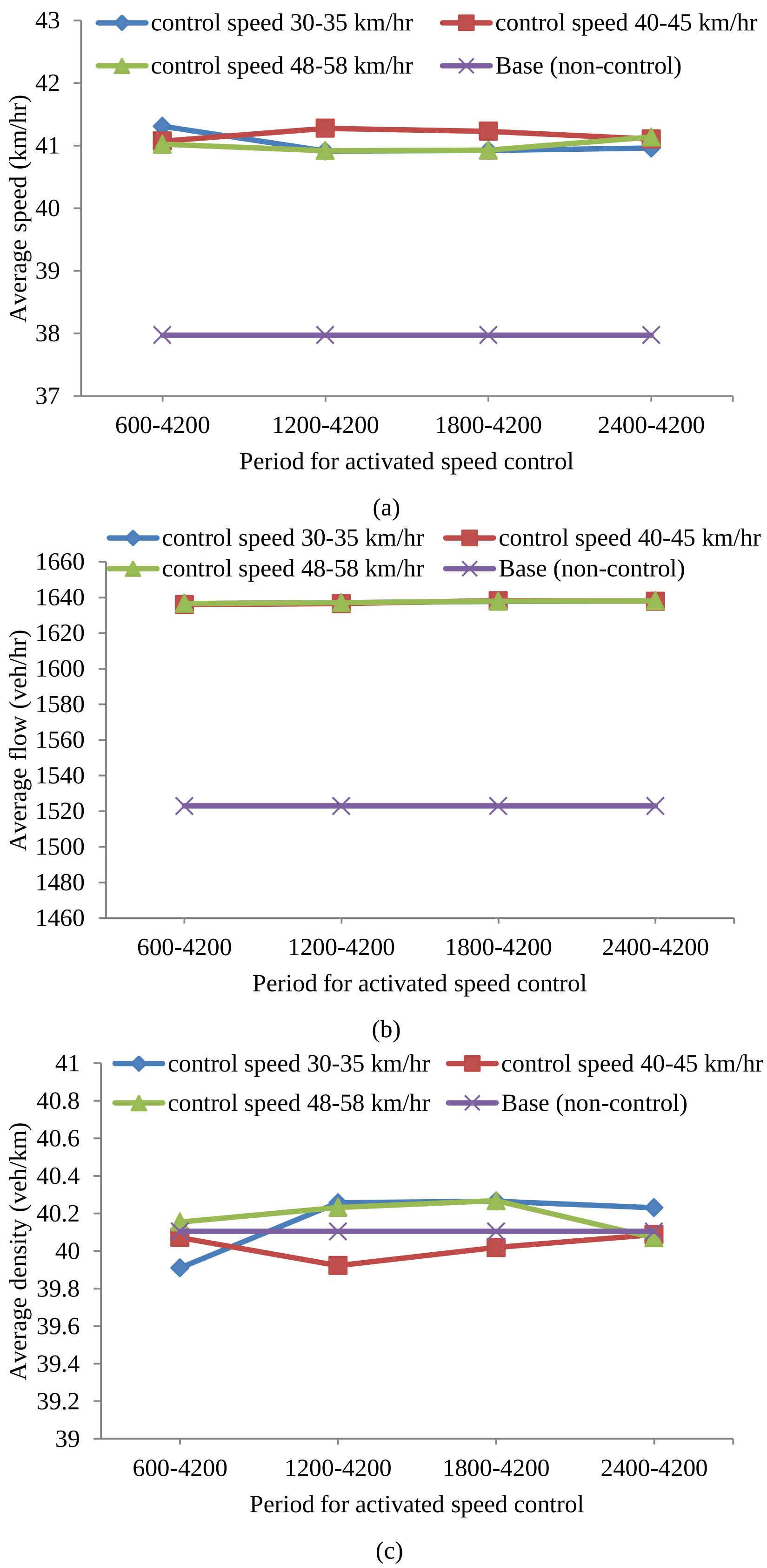
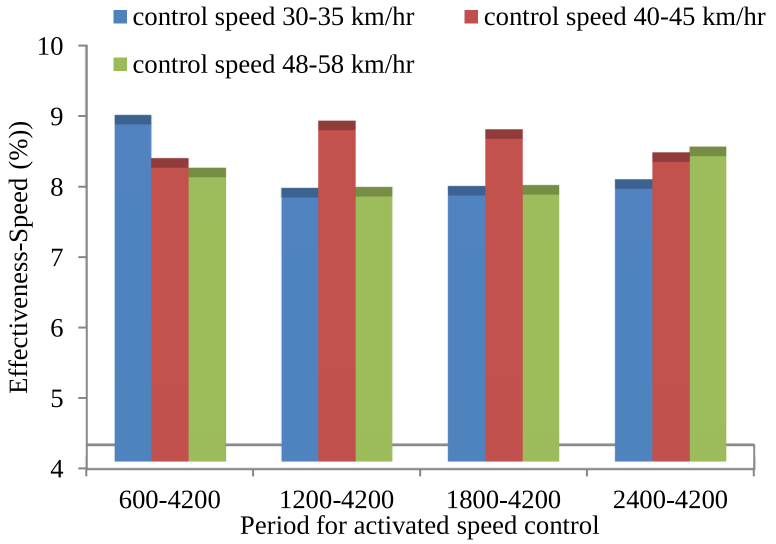
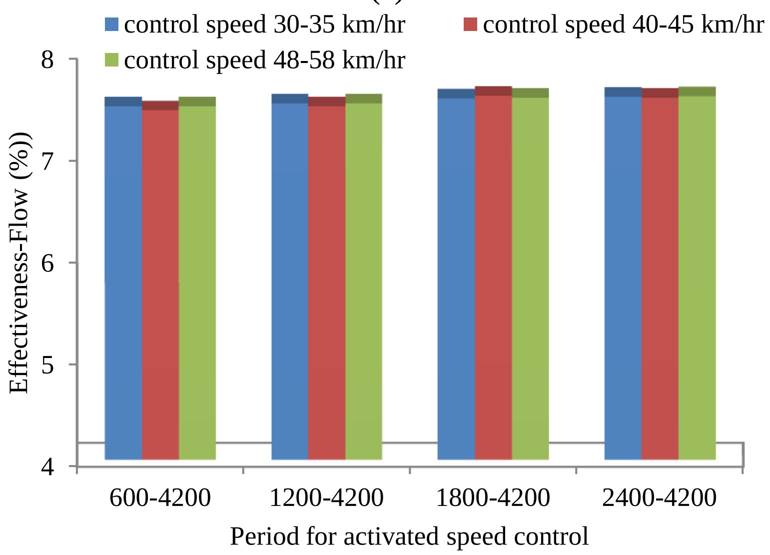


Figure B.8 The macroscopic traffic parameters before and after VSL

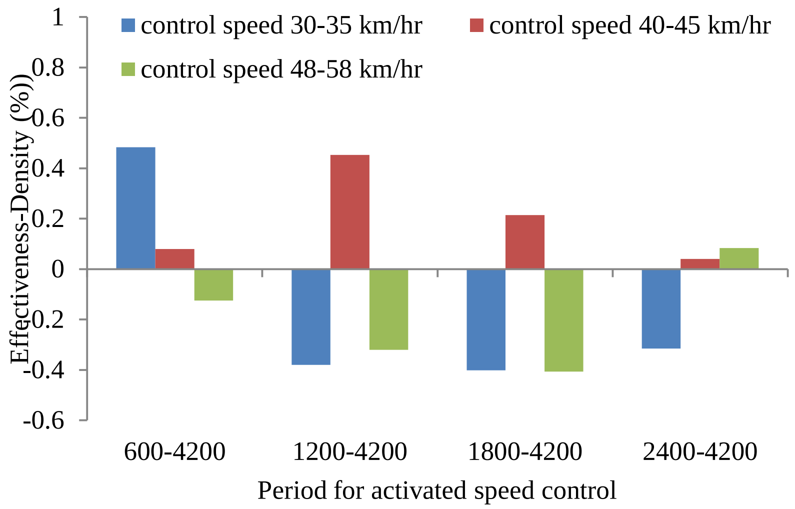
The efficiency of Scenario 2 was measured and plotted in Figure B.9 (a), (b), (c). Highest improvements were achieved in speed which was around 7-9%. Around 7% improvement was achieved in flow and the lowest improvement was in traffic density which was less than 1%. The negative efficiency in the traffic density graph refers to increase in traffic density and the positive efficiency refers to a reduction in traffic density. The efficiency of VSL on the traffic density was considered a positive result because it maintained the road capacity and prevented the traffic congestion from getting worse due to the increasing flow rate.



(a)



(b)



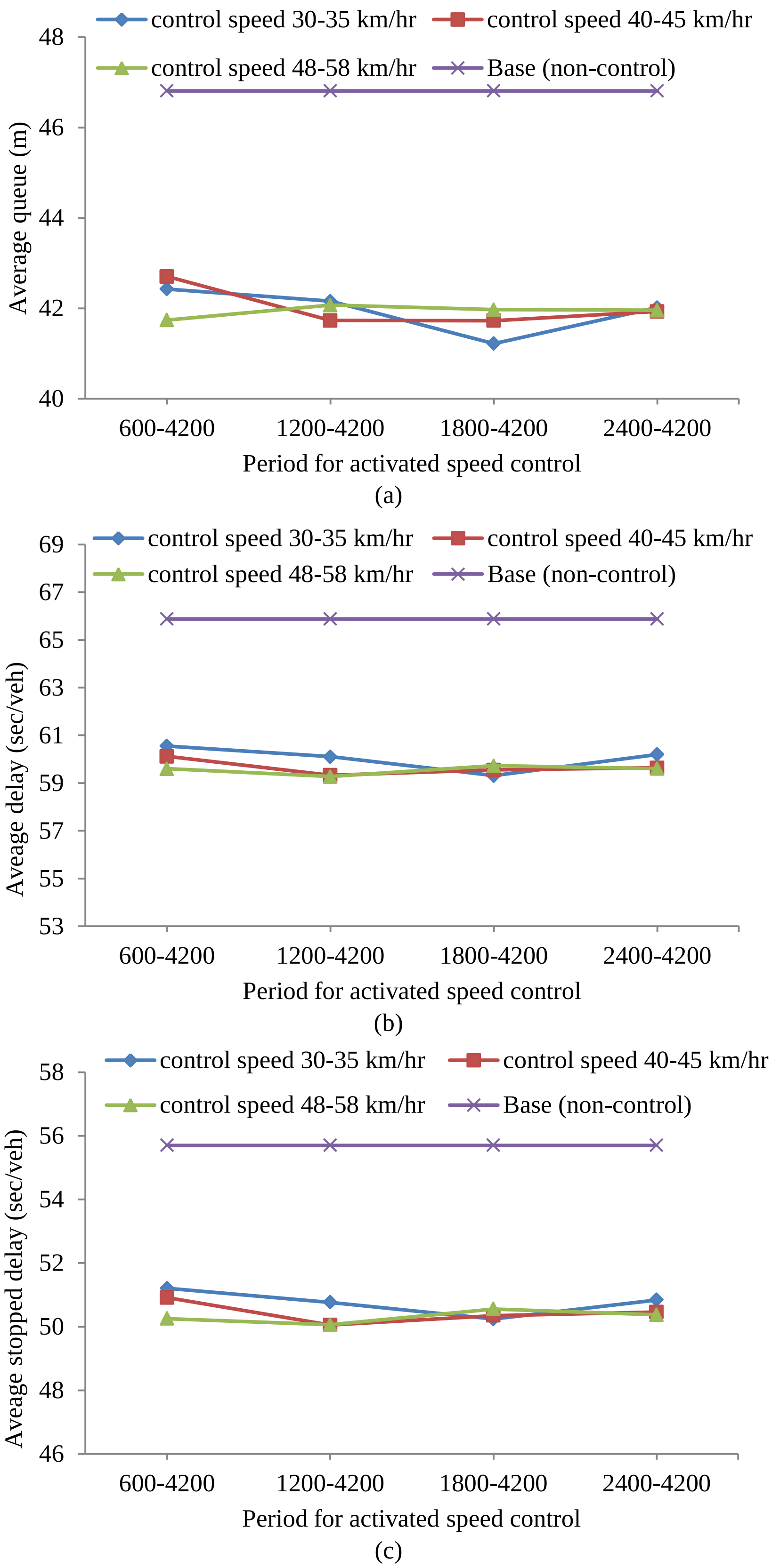
(c)

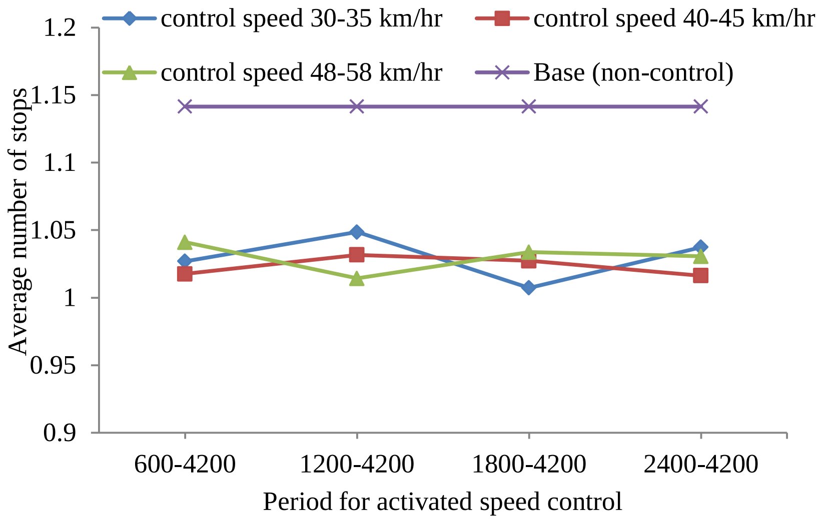
Figure B.9 The efficiency of scenario 2 in terms of speed, flow, and density

B.5 Evaluation of Scenario 3

B.5.1 Evaluation of intersection parameters

Scenario 3 investigated the impact of VSL when CS7 was located at the inlet to the EB direction 5993 m further away from intersection 6. The effect of this scenario on the intersection properties is shown in Figure B.10 (a), (b), (c), (d). The figures show that VSL applications still had a positive influence on the intersection properties. All the sub-categories of Scenario 3 produced a high level of control when compared to the base line condition. VSL improved all signalised intersection properties by reducing the waiting time and the queue length at the intersection. These results led to the conclusion that VSL are able to enhance the level of service by minimising delay and stopped delay and raising the traffic safety by minimising the length of the queue and the number of stops during the peak hour. The variation in improvement between the three VSL control speeds was very small. The reasons for this were mentioned previously.

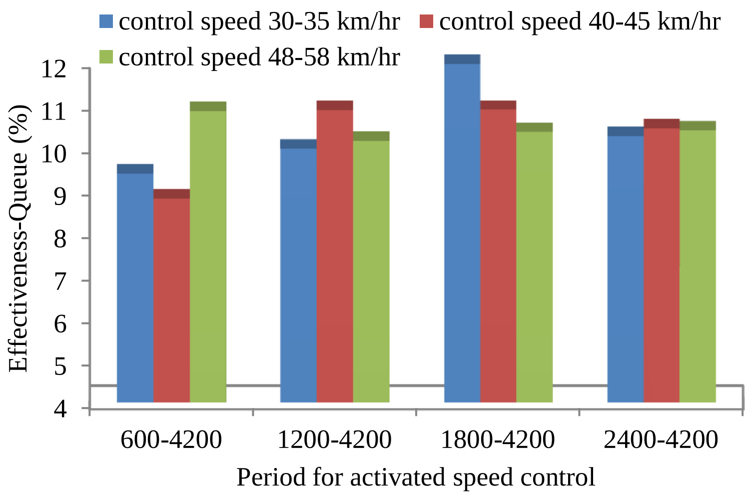

control speed 30-35 km/hr
control speed 40-45 km/hr
control speed 48-58 km/hr
Base (non-control)
Average queue (m)
48
46
44
42
40
600-4200
1200-4200
1800-4200
2400-4200
Period for activated speed control
(a)
control speed 30-35 km/hr
control speed 40-45 km/hr
control speed 48-58 km/hr
Base (non-control)
Aveage delay (sec/veh)
69
67
65
63
61
59
57
55
53
600-4200
1200-4200
1800-4200
2400-4200
Period for activated speed control
(b)
control speed 30-35 km/hr
control speed 40-45 km/hr
control speed 48-58 km/hr
Base (non-control)
Aveage stopped delay (sec/veh)
58
56
54
52
50
48
46
600-4200
1200-4200
1800-4200
2400-4200
Period for activated speed control
(c)



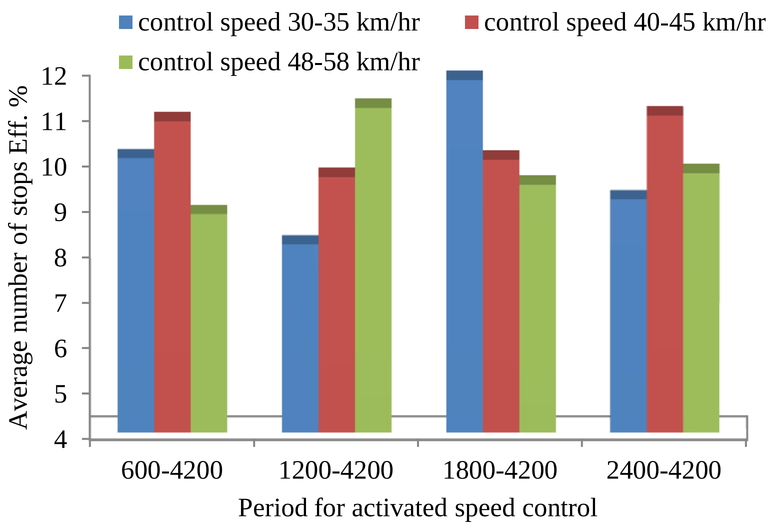
(d)

Figure B.10 The simulation of Scenario 3 before and after VSL applications

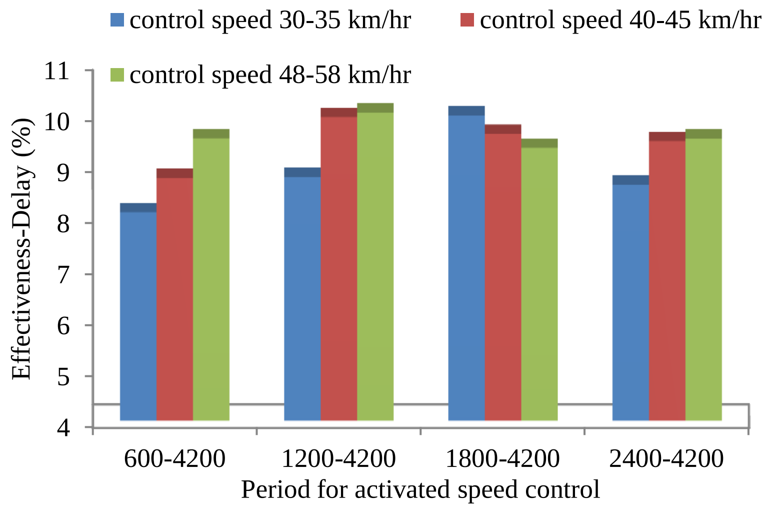
Figure B.11 (a), (b), (c), (d) show the effect of Scenario 3 on the intersection parameters. It shows that all proposed scenarios produced acceptance level of improvement in all intersection properties. At an activation period of 600-4200s the control speed limit range of 48-58 km/hr produced significant improvements when compared to others speed limit ranges in the same period. This is because the control speed (CS7) was installed far from the controlled intersection, VSLs were activated for a long period which produced low speed limits, and these resulted in suitable synchronising of vehicle speeds upstream of the signal operating system. At an activation periods of 1200-4200s and 1800-4200s, the best control speed limits were 40-45 km/hr and 30-35 km/hr respectively. This led to the conclusion that if the position of VSL control was further away from the controlled intersection and it was implemented for a long activation time, lower constraints to the control speed are needed to obtain better traffic management. A 9-12% improvement in average queue length, 8-12% improvement in average number of stops, and an 8-10% improvement in average delay and stopped delay were achieved using Scenario 3.



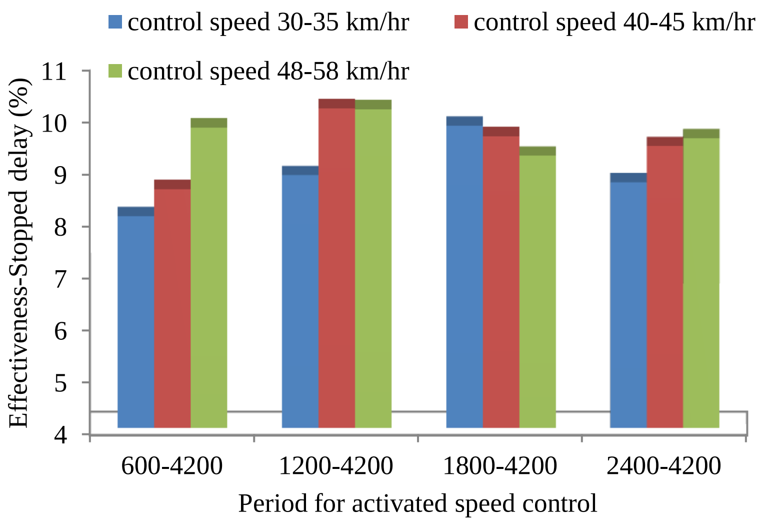
(a)



(b)



(c)

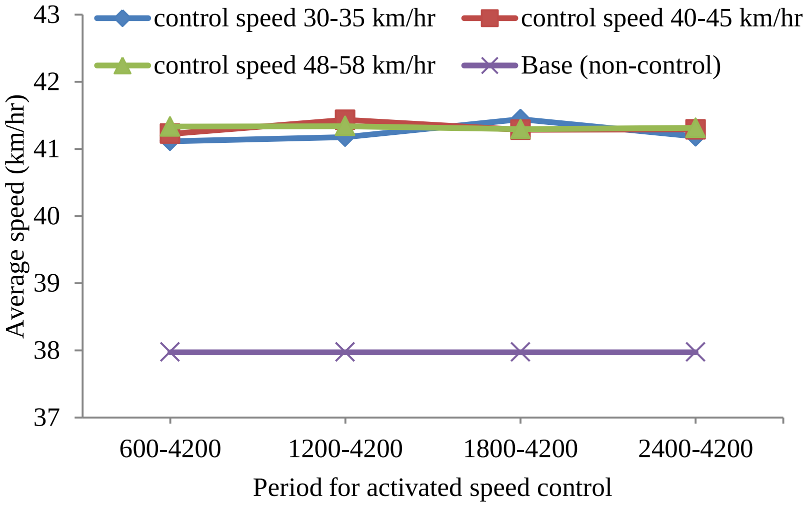


(d)

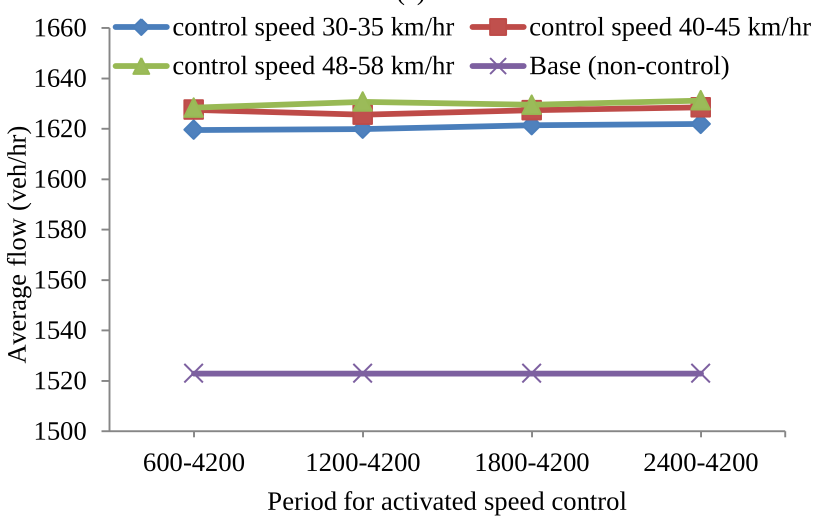
Figure B.11 The efficiency of Scenario 3 on the intersection parameters

B.5.2 Evaluation of macroscopic traffic characteristics

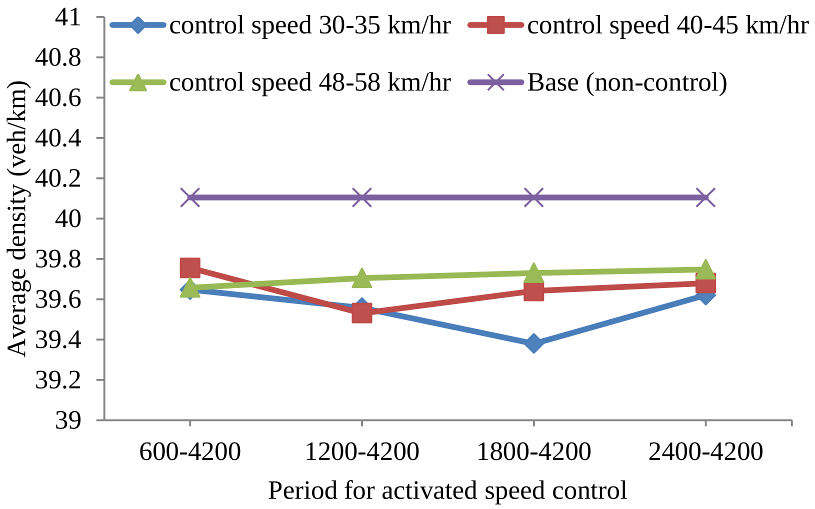
Figure B.12 (a), (b), (c) shows the influence of scenario 3 on speed, flow and density for the congested link. The figures show positive result because all the traffic parameters improved after using VSL control. The base line condition resulted in poorer traffic management when compared with the application of VSL. Speed increased from 38 km/hr to 41 km/hr, the throughput of the link escalated from 1,523 veh/hr to 1,629 km/hr, and the traffic density was maintained during the congestion period in spite of increase in flow.



(a)



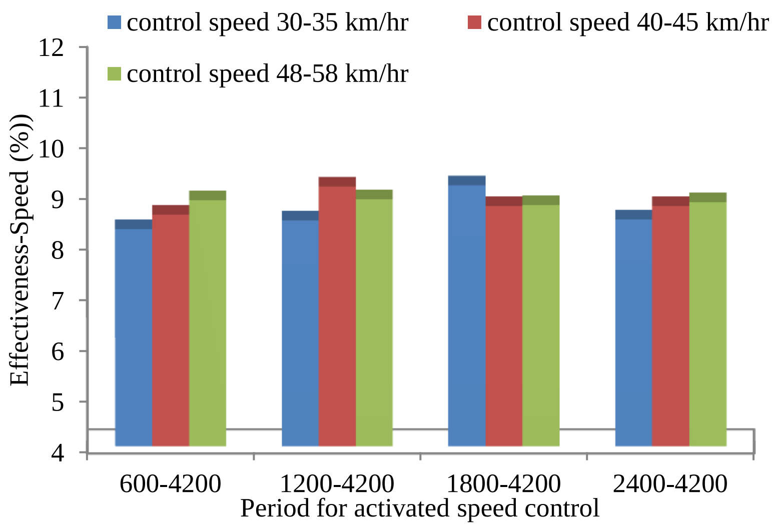
(b)



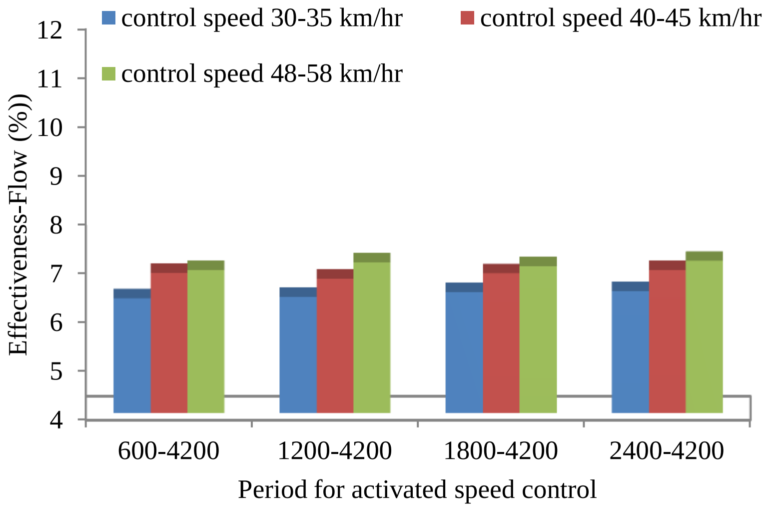
(c)

Figure B.12 The macroscopic traffic parameters before and after VSL

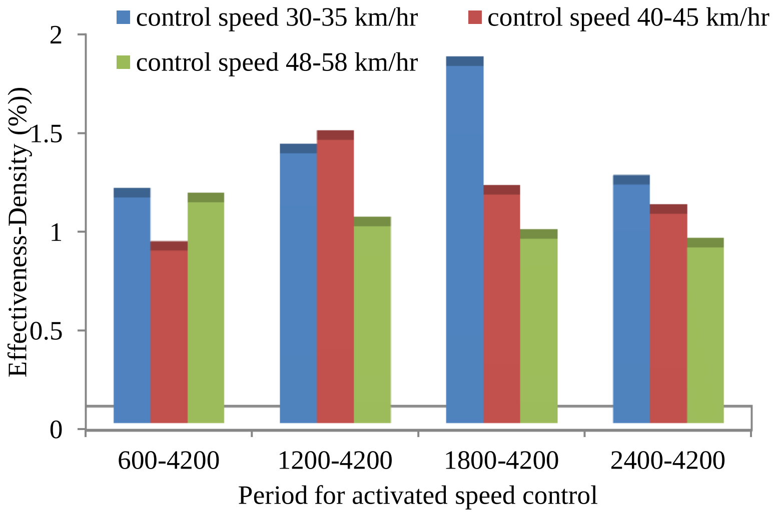
The efficiency of Scenario 3 is shown in Figure B.13 (a), (b), (c). All sub-categories of Scenario 3 achieved a similar improvement because the limited capacity of the road and the analysis being carried out during the period of maximum congestion. The improvement was about 9% in travel speed, 7% in flow and less than 2% in traffic density.



(a)



(b)



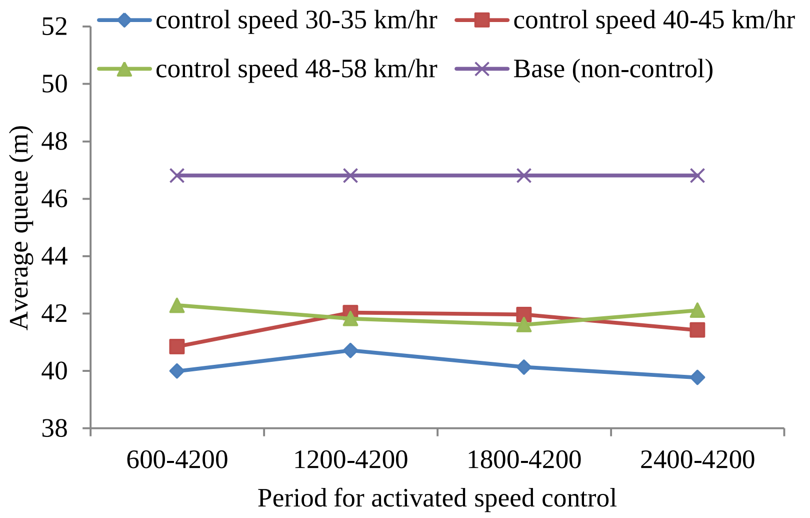
(c)

Figure B.13 The efficiency of Scenario 3 on speed, flow and density

B.6 Evaluation of Scenario 4

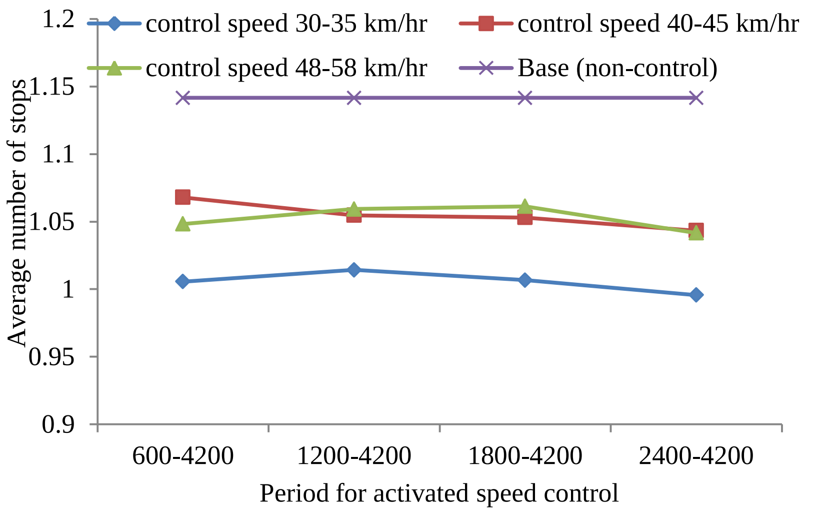
B.6.1 Evaluation of intersection parameters

Scenario 4 was applied to test the effect of increasing the control speed distance on the performance parameters for intersection 6. The effect of this variable is shown in Figure B.14 (a), (b), (c) and (d). It shows that Scenario 4 produced favourable reduction in average queue length, number of stops, delay and stopped delay when compared to the base line condition. Scenario 4 raised the level of service by reducing vehicle delay and enhancing traffic safety through reducing rear-end collisions.



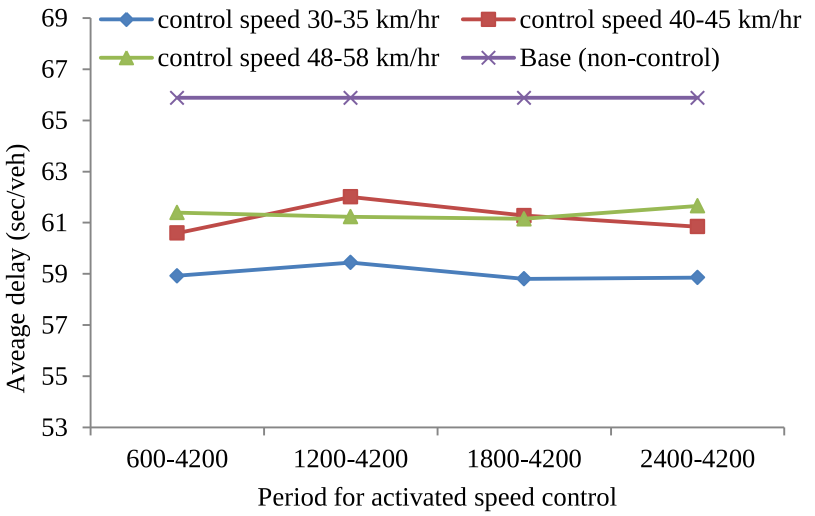
control speed 30-35 km/hr
control speed 40-45 km/hr
control speed 48-58 km/hr
Base (non-control)
Average queue (m)
52
50
48
46
44
42
40
38
600-4200
1200-4200
1800-4200
2400-4200
Period for activated speed control

(a)



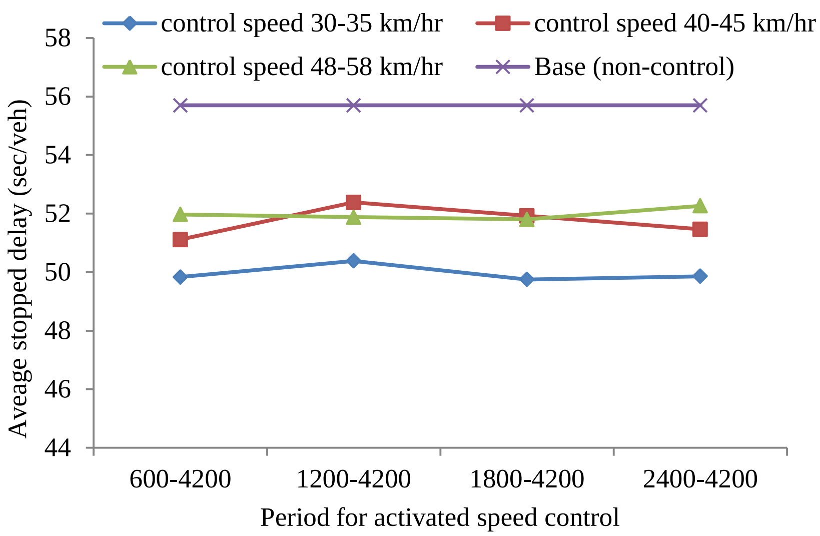
control speed 30-35 km/hr
control speed 40-45 km/hr
control speed 48-58 km/hr
Base (non-control)
Average number of stops
1.2
1.15
1.1
1.05
1
0.95
0.9
600-4200
1200-4200
1800-4200
2400-4200
Period for activated speed control

(b)



control speed 30-35 km/hr
control speed 40-45 km/hr
control speed 48-58 km/hr
Base (non-control)
Aveage delay (sec/veh)
69
67
65
63
61
59
57
55
53
600-4200
1200-4200
1800-4200
2400-4200
Period for activated speed control

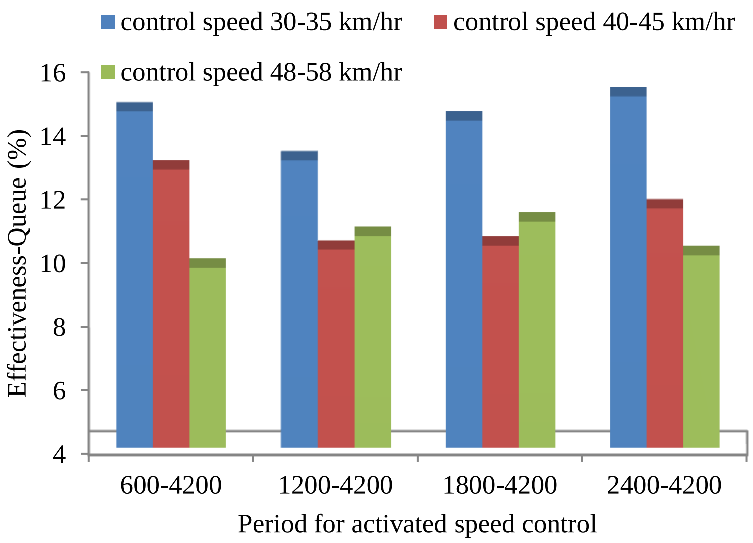
(c)



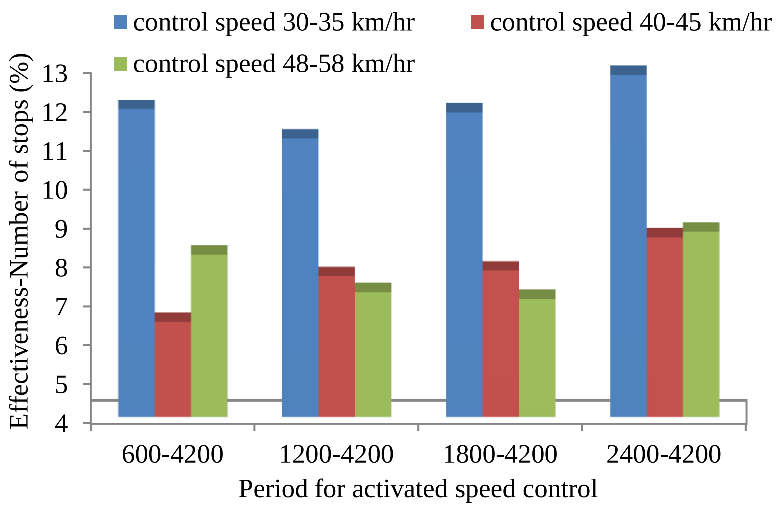
(d)

Figure B.14 The evaluation of Scenario 4 in term of intersection properties

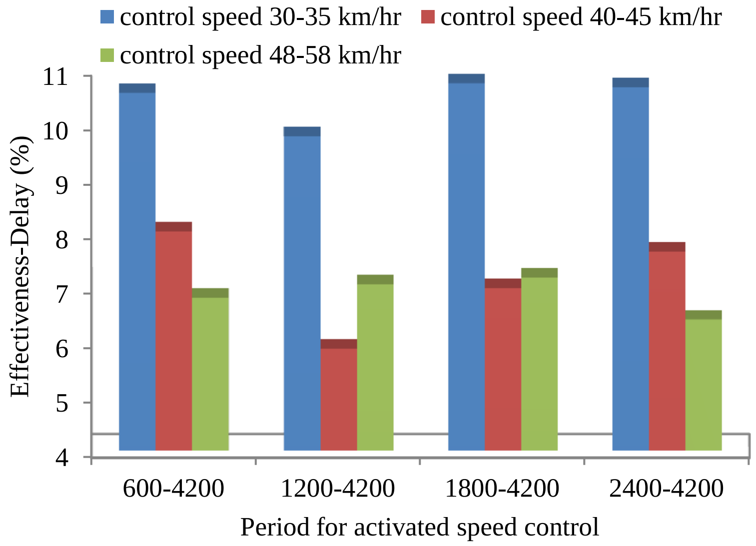
The efficiency of Scenario 4 is shown in Figure B.15 (a), (b), (c), (d). This figure shows that all control speed limits of Scenario 4 achieved a good traffic management for the GAR through the peak hour period. Since the control distance was increased, a control speed limit range of 30-35 km/hr with an activation period of 2400- 4200s produced the best intersection properties (15% improvement in average queue length, 13% improvement in average number of stops, about 11% improvement in delay and stopped delay).



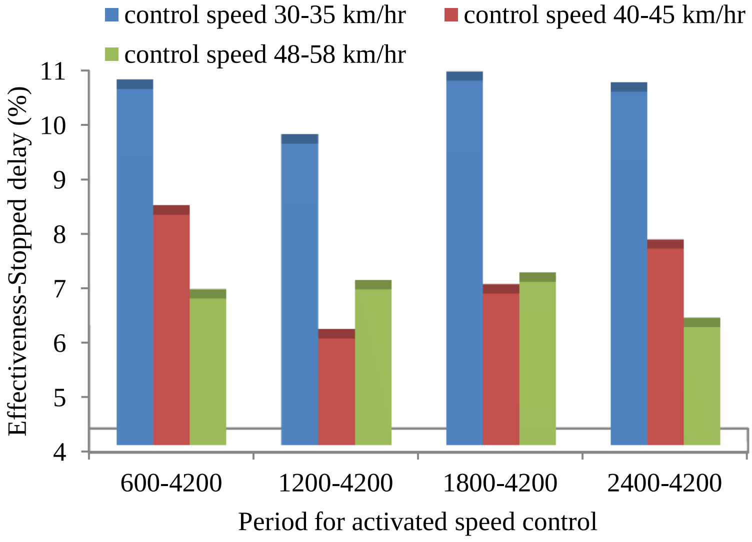
(a)



(b)



(c)

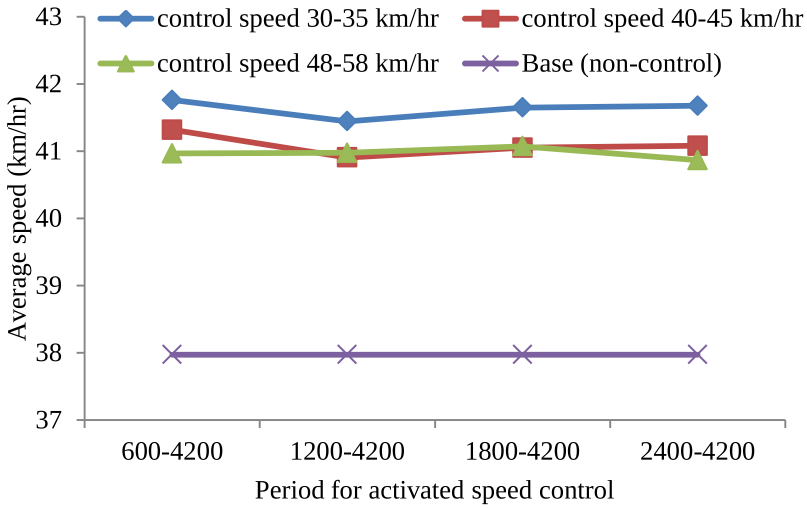


(d)

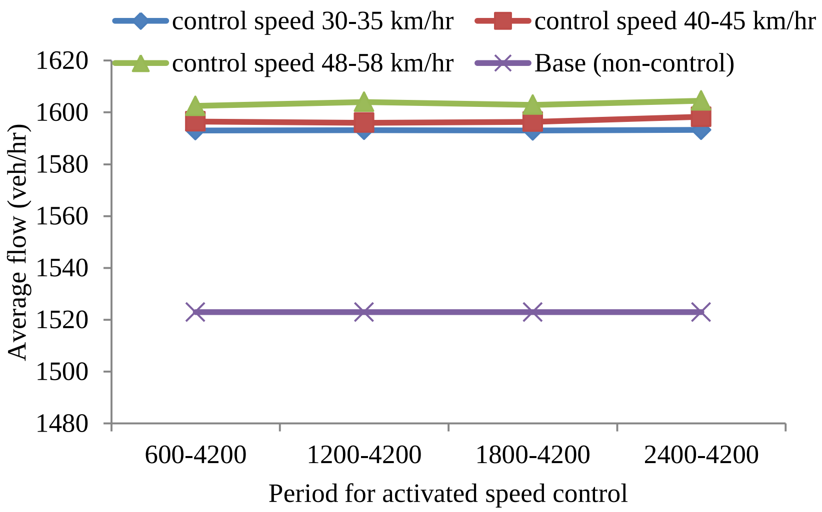
Figure B.15 The efficiency of Scenario 4 in terms of intersection parameters

B.6.2 Evaluation of macroscopic traffic characteristics

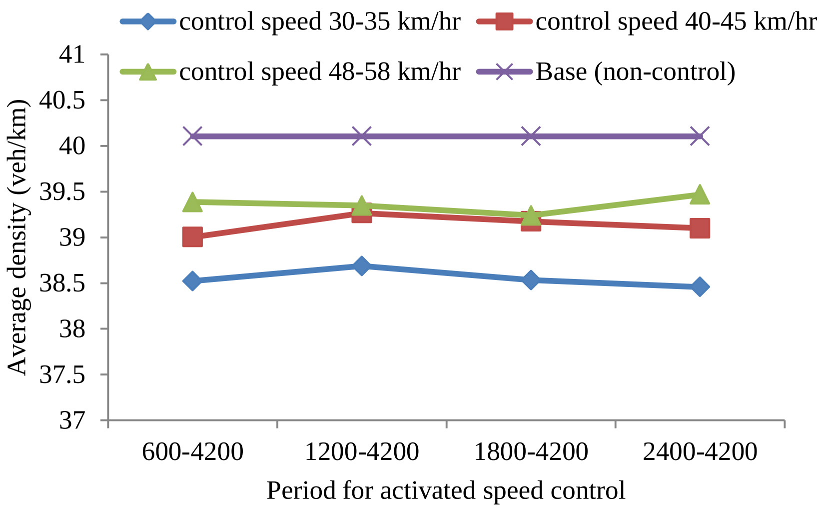
Figure B.16 (a), (b), (c) shows the influence of Scenario 4 on the macroscopic traffic parameters for the congested link. It shows that speed, flow, and density were improved after applying VSL. The speed and flow were increased and the traffic density was reduced after using VSL.



(a)



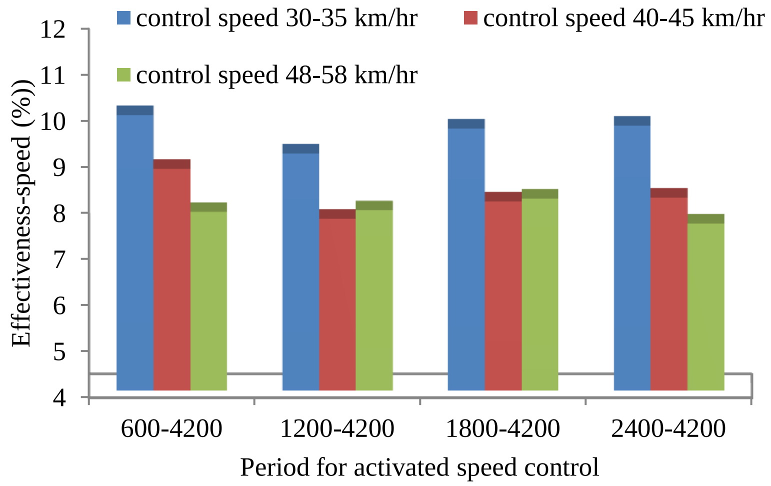
(b)



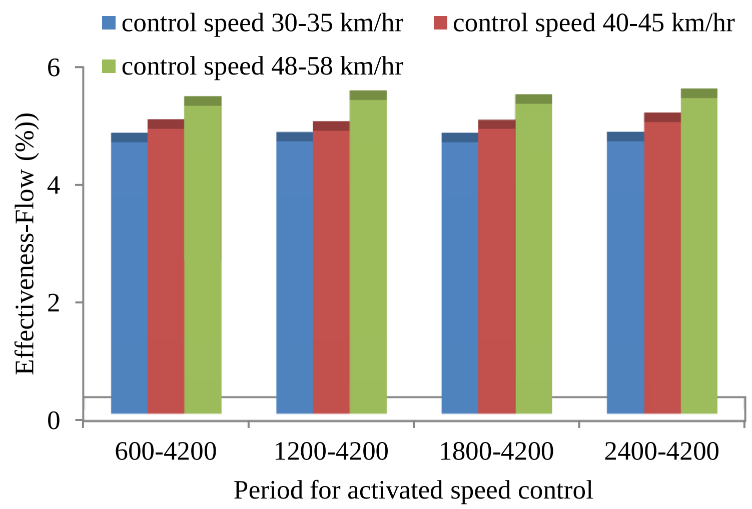
(c)

Figure B.16 The influence of Scenario 4 on speed, flow, and density

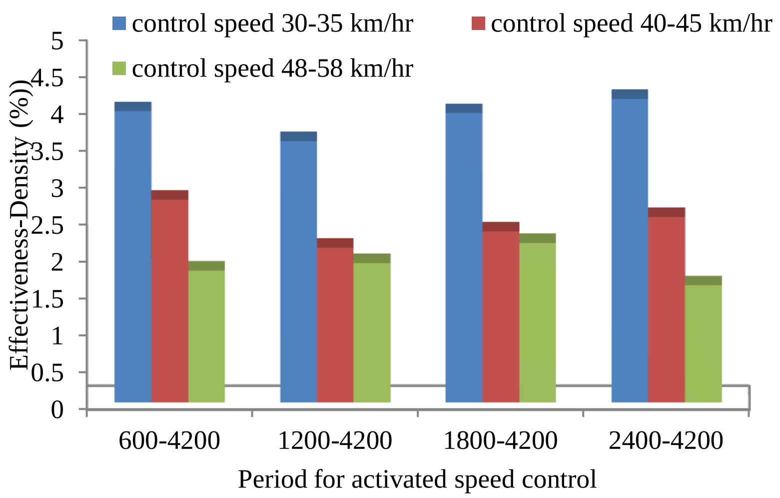
Figure B.17 shows the efficiency of Scenario 4 on the speed, flow, and density of the congested link. A 7-10% improvement in average travel speed, about 5% improvement in average flow and about 4% improvement in average traffic density were achieved by using VSL.



(a)



(b)



(c)

Figure B.17 The efficiency of Scenario 4 on the intersection properties

B.7 Comparison and optimisation of VSL scenarios

The maximum efficiency for each proposed scenario is shown in Figure B.18 and Figure B.19**.** Figure B.18 shows that the efficiency of VSL under different scenarios produced significant improvement for intersection 6 in terms of queue length, number of stops, delay and stopped delay. The most appropriate traffic operating system was produced by Scenario 4 because the high traffic volume required more flow constraint than could be achieved by increasing the control distance. Also traffic volume has a great influence on the performance of VSL applications and this performance is reduced when the traffic volume increases and vice versa. For example, the performance of VSL in the WB direction (Figure 5.28) was higher than the performance in the EB direction because the traffic volume in the WB direction (1332 veh/hr) was lower than the traffic volume in the EB direction (1523veh/hr). The maximum improvement was 15% in average queue length, 13% in average number of stops, and 11% in average delay and average stopped delay for Scenario

4. These results show a 13-15% increase in traffic safety and an 11% enhancement in service at the intersection.

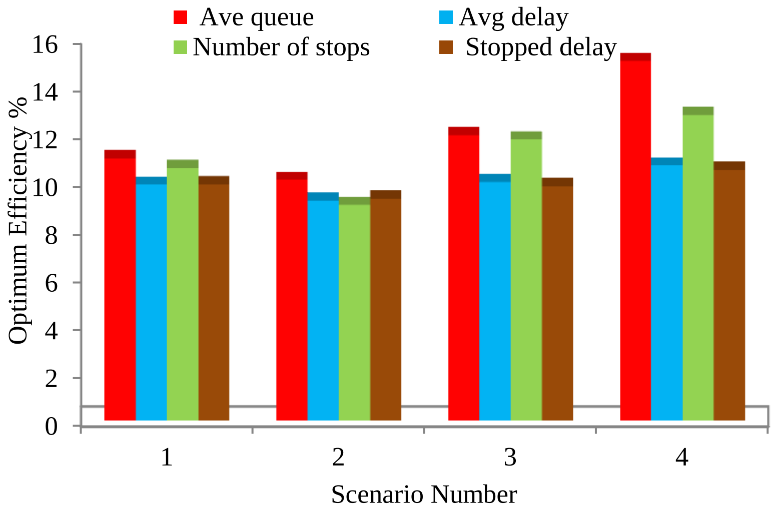


Figure B.18 Optimum efficiency in term of the intersection characteristics

Figure B.19 shows the maximum efficiency in term of macroscopic traffic parameters under different scenarios. It shows a steady improvement in speed and flow for all VSL scenarios. The optimum efficiency for speed and flow were 8-10 % and 5-7 % respectively. The maximum efficiency for traffic density was less than 4 %. The similarity in results between the scenarios may due to the limited capacity of the road and the analysis being carried out during the period of maximum congestion.

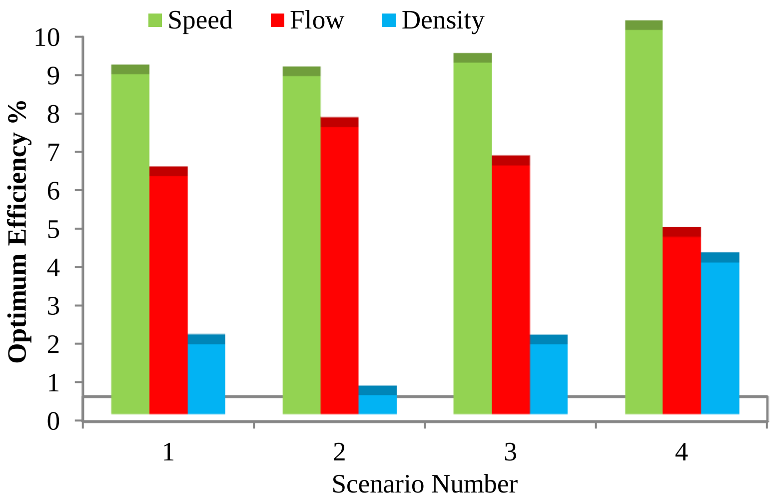


Figure B.19 Optimum efficiency in term of the link characteristics

B.8 Evaluation of total travel time

The total travel time (TTT) in the EB direction, with and without control, was studied in order to explore the influence of VSL application on TTT. TTT was considered for all vehicles starting at the entrance link in the EB direction until exiting at intersection 6. This is a distance of about 6.7 km. Figure B.20 shows that the TTT for the base condition (no-speed control) was 570 seconds whereas the use of various VSL scenarios produced TTTs of 587, 550, 548 and 560 seconds. Variation in TTT was due to the speed control limit and its applied distance for each scenario. The efficiency of VSL application on TTT is shown in Figure B.21. Scenarios 2, 3 and 4 reduced the TTT by 3%, 4% and 2% respectively. No improvement or negative results were obtained by applying Scenario 1. Scenario 1 caused an increase in TTT because it constrained the flow by applying a speed control range of 30-35 km/hr which was slower than the other scenarios. Also, the speed control was activated for 600-4200s which was earlier than in the other cases. Nevertheless, VSL application has little impact on TTT due to the constrained speed limits for non-congested links. These have a major effect on the final TTT calculation. The best traffic system for the congestion period should trade-offs between improvement in traffic characteristic and the total travel time.

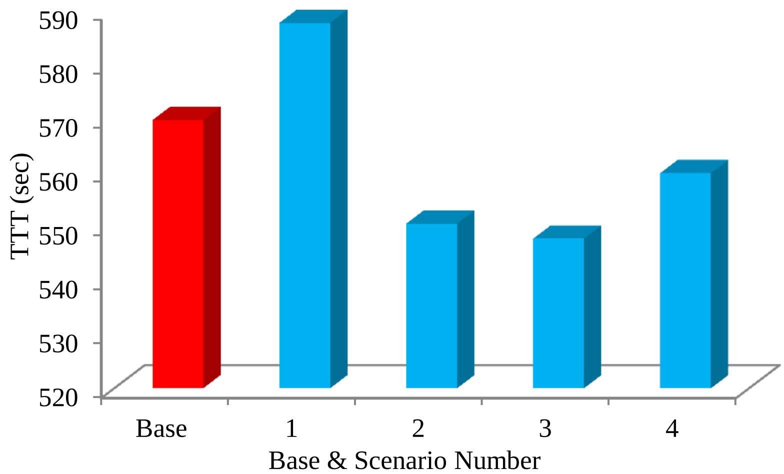


Figure B.20 TTT comparison

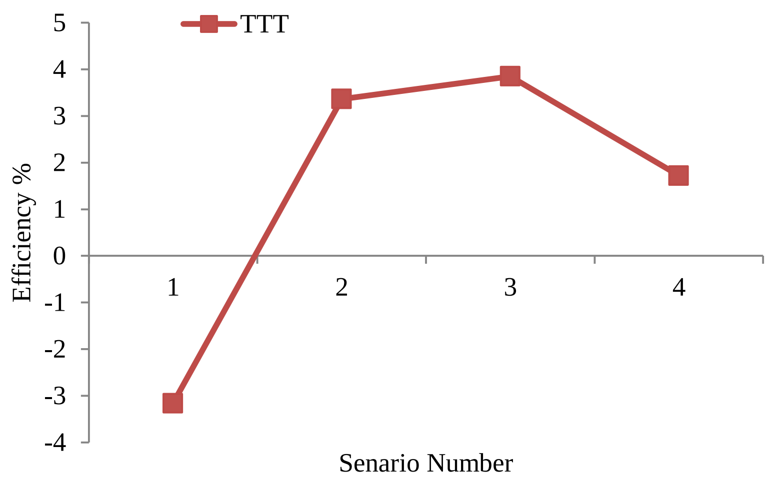


Figure B.21 The efficiency of VSL application on the TTT