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Analysis of a Ramp Metering Application for Mitchell Freeway

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Declaration

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgment has been made.

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

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ABSTRACT

Freeways are known as a key constituent of road infrastructure particularly in populated areas. Increasing traffic demands have resulted in several issues with freeway traffic. Today, it is well known that proper management of freeway traffic is essential in order to provide a safe and reliable level of service and to optimise freeway productivity. For this purpose, a number of ramp management strategies have been developed. Nowadays, ramp metering is one of the most common methods used for managing freeway traffic. It consists of installation of traffic signals on freeway entry ramps and controlling freeway traffic by limiting the entrance rate of traffic. Regardless of the selected ramp metering technique, it is vital to design the ramp meters in an appropriate manner. Traffic simulation modelling is a useful tool which allows analysis of traffic networks at different levels of detail.

In this study traffic micro-simulation modelling is used for analysis of the ramp metering in Mitchell Freeway which services the northern corridor of Perth, Western Australia. Compared to other traffic simulation techniques, microscopic simulation methods are regarded as the most competent approaches particularly in situations where detailed simulation of vehicle-road and vehicle-vehicle interactions is required.

The study focuses on morning peak hours when the southbound lanes of the freeway (from Hepburn Avenue to Graham Farmer Freeway) experience the highest traffic flow. After defining the geometrical scope of the study, different sets of data were obtained to be used as input for the model or as benchmarks. The data include traffic flows from links and cordons, SCATS data from intersection of the freeway ramps, and peak hour sub-area matrix.

The model of Mitchell Freeway was then built in Commuter, a nano and micro-simulation software and Q-Paramics, a micro-simulation package. The origin-destination matrix was estimated using Q-Paramics and Commuter packages. Before incorporating ramp metering in the model, the model is calibrated against observed freeway traffic counts and then validated against observed freeway travel times.

After confirming the constructed base model was a valid representation of Mitchell Freeway, ramp metering was added and studied in the model. Ramp meters were installed on the entry ramps of Mitchell Freeway section under study and relevant parameters were calibrated. The model ramp meters are based on ALINEA ramp metering algorithm. Finally, results of the base case and the case with ramp metering were compared and analysed to reveal potential advantages and disadvantages of ramp metering for Mitchell Freeway.

In general, the results show that currently Mitchell Freeway capacity is not optimally used, particularly in northern parts. Micro-simulation shows that proper ramp metering can potentially make use of the freeway more efficiently by increasing mean flow in different sections of the freeway and by prevention of queues. Furthermore, ramp metering was showed to decrease travel time and increase travel time reliability for freeway users. It was also revealed that ramp metering has a positive effect on reduction of fuel consumption and associated air pollution. Finally, a number of recommendations for future works are presented.

DEDICATION

To my wonderful husband, Amin, and my beloved family

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Notations and Abbreviations

ABS	Australian Bureau of Statistics
ALINEA	Asservissement Linéaire d' Entrée Autoroutière, i.e., Linear feedback control of a motorway on-ramp
BRM	Bottleneck Metering Rate
CBD	Central Business District
CO2	Carbon dioxide
DIT	Department of Infrastructure and Transport
GEH	Geoffrey E. Havers
GFF	Graham Farmer Freeway
GPS	Global Positioning System
HERO	Heuristic Ramp-Metering Coordination
HRM	Highway Capacity Manual
LMR	Local Metering Rate
LOS	Level of Service
LQ	Linear Quadratic
MRWA	Main Roads Western Australia
NO	Nitric oxide
O-D	Origin Destination
pc	Passengers Car
ROM	Regional Operations Model
SCANDI	Surveillance Control and Driver Information
SCATS	Sydney Coordinated Adaptive Traffic System
SMR	System Metering Rate
TCS	Traffic Control Site
VDS	Vehicle Detectors Sites
WA	Western Australia

1 Introduction

1.1 Introduction to Ramp Metering

A transport network forms a major part of a country's infrastructure. A well-developed and effective transport network is vitally important to the economy. An under-developed road network not only imposes huge costs to the country, but also makes everyday life difficult. With the recent population growth, which has led to increasing travel demands, the importance of road networks and the necessity for their development and optimum usage has become even more pronounced.

Among the various components of road transport infrastructure, freeways are recognised as a key constituent of road networks, particularly in populated metropolitan areas. Their role is to manage traffic by removing traffic loads from nearby arterial roads. As a result of population growth in recent decades, demands on freeways have increased considerably. This has led to a number of issues with freeway traffic management, with two key areas being congestion and reduction in safety. The increased frequency of these issues highlights the need for improved management practices of freeway traffic.

Today it is widely accepted (Burley and Gaffney 2010) that providing a safe and reliable level of service that maximises freeway productivity can only be achieved with proactive management of freeway traffic. Over the last few decades a number of ramp management strategies have been developed with the aim of controlling entry ramp traffic entering freeways, with the objective of balancing freeway demand and capacity. By default, on ramps are uncontrolled on the unmanaged freeways. Ramp metering is one such management strategy that is being commonly used today.

Ramp metering consists of installation of traffic signals on entry ramps and controlling the impact on freeway traffic by limiting the entrance rate of ramp traffic. In response to different traffic conditions, a number of ramp metering algorithms have been devised over the last few decades. The simplest category of ramp metering is called fixed-time ramp metering algorithms in which no real-time measurement and feedback is carried out. These methods are based on historical data. Due to lack of real-time data, these methods may, at times, overload or under-utilise the freeway.

A more sophisticated category of ramp metering is called reactive ramp metering. Unlike the first category, these methods adjust the entry traffic rates based on the changing traffic flow on the freeway. Ramp signals operate dynamically, adapting to varying freeway traffic flow rates in order to prevent impairment of traffic flow on the freeway mainline. Such methods include two different sub-categories: (a) local ramp metering algorithms and (b) coordinated ramp metering algorithms. Over the last few decades a number of different methods have been developed in each of these sub-categories.

Nonetheless, regardless of the selected ramp metering technique, it is essential to design the ramp meters in an appropriate manner. Without careful and appropriate designs, ramp metering methods can result in a deterioration of traffic situation in the network. This can for example happen due to excessive diversion of freeway traffic to nearby arterial roads. If arterial roads do not have sufficient capacity for the diverted traffic from freeway, new and undesirable traffic issues may arise in those parts of the network.

Traffic simulation modelling is a popular technique normally used for the design and analysis of traffic networks at different levels of detail. Among various simulation methods, micro-simulation approaches are more appropriate in situations where very detailed simulation of vehicle-road interaction events is required. Ramp metering modelling requires this level of detail, and thus the micro-simulation technique is the approach that has been adopted in this study.

1.2 Significance of Study

Perth, the capital of Western Australia, has experienced the highest population growth of any Australian State capital city (ABS 2012b). There are three freeways in the Perth metropolitan area that handle most of the city's traffic flow. These are the Mitchell, Kwinana and Graham Farmer Freeways.

Relevant traffic and transport planning design studies are essential if Perth's growing traffic demand it to be managed effectively. This study focuses on optimised operation of Mitchell Freeway -the only freeway connecting Perth Northern suburbs to Perth central business district- by micro-simulation of the effect of ramp metering on that freeway.

In order to maximise freeway productivity and optimise its throughput and travel time, it is essential to manage freeway traffic in an appropriate manner. Ramp metering is a key technique among other freeway management strategies. Several field applications have proved the considerable positive effect of ramp metering on management of freeway traffic. However, due to some drawbacks of inappropriate designs, ramp metering may even deteriorate network traffic. Therefore, it is critical for traffic and transport professionals to pay special attention to proper design of ramp metering in order to achieve the expected goals(Burley and Gaffney 2010).

As well as reducing congestion, appropriate ramp meters have proved to reduce traffic incidents rate on freeways (Burley and Gaffney 2010; Wu 2001). This in turn prevents the consequential deteriorating impact of accidents. By improving the freeway operational performance, delays caused by traffic entering the freeway are reduced thus total road network travel time is also decreased. Ramp metering smooths freeway traffic by redistributing traffic along the freeway. As well as providing more reliable travel times for passengers, ramp metering can significantly improve the safety of freeway journeys by reducing the likelihood and frequency of on-ramp related accidents.

The points outlined above serve to demonstrate the importance of ramp metering and its design as a key technique in managing freeway traffic. In this study, an attempt is made to evaluate possible effects of ramp metering on Perth's Mitchell Freeway. Study results provide useful information which could be considered in design of future ramp meters for the freeway to ensure this important Perth freeway is used in an efficient manner.

1.3 Study Objectives

This study has a number of objectives. The main goal of the study is to analyse ramp metering effects on Mitchell Freeway traffic. The study focused only on morning peak hours. During this time the southbound lanes of the freeway experience the highest traffic flow mainly due to commuters travelling from Perth's northern suburbs to the central business district. Therefore, in this study only southbound freeway traffic is modelled.

Before starting the analysis on the effect of ramp metering on freeway traffic, the study required the building of an appropriate base model. For this purpose, the different parameters of the base model need to be calibrated, and the model validated against observed travel time of actual traffic on the freeway.

In the next phase, the goal was to build a valid model of ramp metering on the freeway. To do so, ramp metering based on ALINEA algorithm was included in the base model and the relevant algorithm parameters calibrated. This provided a platform for each the main objective of this study –an analysis of different ramp metering strategies on freeway traffic. For this purpose, the effects of ramp metering on the freeway traffic were analysed against a number of criteria: freeway capacity, travel time duration, travel time reliability, fuel consumption and air pollution. Comparison of the base model with the ramp metered model based on these criteria delivered valuable information about potential impacts of ramp metering on different aspects of the Mitchell Freeway traffic. This information may then be considered in the design of any future ramp metering for the freeway.

1.4 Research Methodology

The approach used in this study is traffic simulation modelling. It is a useful tool for evaluating models which may be too complicated for analytical or numerical approaches. Moreover, using traffic simulation one can assess different scenarios for a traffic system, a task which is difficult or impossible in majority of field tests.

Real-life ramp metering presents is complex and thus challenging to implement successfully. Such complex situations normally require detailed simulation in order to take into account different factors in the model. Compared to other traffic simulation techniques, microscopic simulation methods are regarded as the most competent approaches particularly in situations where detailed simulation of vehicle-road and vehicle-vehicle interactions is required. This category of simulation the movement of vehicles is traced through a network. Behaviour of individual vehicles is also modelled using a combination of special car-following, vehicle performance, and lane changing algorithms.

In this study micro-simulation is employed for modelling and evaluation of ramp metering on the Mitchell Freeway. After defining the geometrical scope of study, different sets of data were obtained to be used as input for the model or as benchmarks. Traffic flows from links and cordons were used as input to two traffic simulator packages for estimation of the demand matrix as well as calibration of the model in later stages. Furthermore, SCATS data from intersections of freeway ramps are used for confirming data consistency as well as an estimation of missing data. The peak hour sub-area matrix of the study area was obtained from Main Roads Western Australia (MRWA) and was modified to suit the model requirements.

The model of Mitchell Freeway was then built in Commuter, a nano and micro-simulation software and Q-Paramics, a micro-simulation package. The origin-destination matrix was estimated using Q-Paramics and Commuter packages. As the study focuses on morning peak hours, only traffic in southbound of freeway was modelled. Before incorporating ramp metering in the model, the model is calibrated against observed freeway traffic counts and then validated against observed freeway travel times.

After confirmation that the constructed base model was a valid representation of Mitchell Freeway, ramp metering was added and studied in the model. Ramp meters were installed on the entry ramps of Mitchell Freeway section under study and relevant parameters were calibrated. The model ramp meters are based on ALINEA ramp metering algorithm. Finally, results of the base case and the case with ramp metering were compared and analysed to reveal potential advantages and disadvantages of ramp metering for Mitchell Freeway.

1.5 Summary

Ramp metering is one of the most common methods used for optimisation of freeway usage by controlling flow through freeway entry ramps. Appropriate design of ramp metering is essential to guarantee the objectives are met. Micro-simulation modelling is a useful tool for studying potential impacts of ramp metering on freeway traffic. The main goal of this study is to analyse ramp metering effects on traffic of Perth's Mitchell Freeway, the only freeway connection Perth's northern suburbs to the City's central business district. This is achieved by conducting micro–simulation modelling from the base modelling of freeway, and then including ramp metering in the model. Ramp metering effects were studied by comparison of different parameters in the base model and ramp metered model.

Chapter 2 provides a back ground on theory of ramp metering. Starting with an introduction to ramp metering on freeway, history of ramp metering around the world and in Australia is briefly reviewed. A description of different categories of ramp metering is then presented followed by a brief overview of traffic simulation modelling and its suitability for ramp metering.

Chapter 3 describes the scope of the study, different data used in this study and various steps undertaken for building firstly the base model of Mitchell Freeway, and secondly including ramp metering in the model. These steps include modification of peak hour sub-area matrix of study area, choosing modelling period, coding the model, origin-destination matrix estimation by simulation, warming up of the model, validating the base model, coding mainline detectors for ramp metering, and calibration of relevant parameters.

Chapter 4 initially discusses some limitations of the model based on which the results are obtained. It then covers an explanation of the results of modelling from three different aspects: capacity, travel time reliability, and fuel consumption and air pollutions. It will be shown how there parameters indicate improvement of freeway traffic after installation of ramp metering. Then, focus is made on other effects of the ALINEA ramp metering on Mitchell freeway traffic which may not necessarily be positive.

Finally, Chapter 5 includes a conclusion of the study as well as recommendations for future studies in this area.

2 Theoretical Background

2.1 Introduction to Freeway Traffic and Ramp Metering

Transport infrastructure is a term used to describe facilities and installations which are required for movement of people and goods from one location to another. Road networks are important components of this infrastructure that need to be developed in parallel to other sectors of the economy. The considerable growth in population in particular which has resulted in increased traffic demands, highlights the importance of optimising the use of road networks.

Freeways are one of the early solutions that were introduced to meet the increasing traffic demands. Governments and local authorities spend significant amount of money for maintaining, upgrading, and developing their freeway systems. Freeways are actually the key components of road network in almost all metropolitan areas and large cities around the world. They carry significant portions of traffic in such areas. For example, Melbourne's freeway and tollway network carry about 30% of arterial roads traffic while they cover only 7% of the arterial road length (Burley and Gaffney 2010).

In early days it was thought that freeways can provide unrestricted access for road users without introducing flow interruptions by traffic lights. However, the rapid growth in traffic demand resulted into ever increasing congestions. This proved that the initial optimistic postulation about freeways was wrong (Wu 2001).

Traffic congestion occurs in situations where too many vehicles want to use a common transportation route which has limited capacity. In other words, when travel demand approaches the capacity of the road, traffic congestion takes place (Papageorgiou et al. 2003; Wu 2001). The increase in traffic demand has led to severe congestions including both recurrent congestions which occur over the daily rush hours and non-recurrent congestions due to for example incidents

(Papageorgiou et al. 2003; Skabardonis, Varaiya, and Petty 2003). As it appears from the definition, location and time of recurrent congestions are normally predictable. Furthermore, delays due to this type of congestion can be as a result of fluctuations in demand, physical layout of the freeway, and the manner in which the road is operated.

On the other hand, non-recurrent congestions may take place due to temporary reduction of capacity caused by car crashes or other road incidents. Delays of this form depend on the nature of the incident. For example, an accident will probably cause more delay than a vehicle stopped on the shoulder of the highway (Skabardonis, Varaiya, and Petty 2003).

Occurrence of congestions on freeway urged management of freeway traffic to prevent such undesirable events. Today it is well recognised that providing a safe and reliable level of service which maximises the productivity of freeway and delivers optimum throughput and travel time, requires efficient use of freeway by appropriate management of its traffic (Burley and Gaffney 2010). Controlling traffic of freeway as one of the most important transportation infrastructures can include different control systems which are used in freeway networks:

- Ramp metering which is carried out by installation of traffic lights at on-ramps or freeway intersections.
- Link control which can include a number of different options such as lane control, variable speed limits, warnings for congestion, and keep-lane instructions.
- Driver information and guideline systems which can be based on using roadside variable message signs or two-way communication with equipped vehicles for example for route choice (Jacobson et al. 2006; Papageorgiou et al. 2003).

According to (Burley and Gaffney 2010), the most effective traffic management tool for controlling freeway in order to achieve high level of efficiency and reliability is controlling the access to freeway with coordinated ramp signals. Ramp metering is one of the most common freeway management approaches used in major metropolitan areas. It is implemented by installation of traffic signals on freeway on-ramps. This aims at reducing delays in freeway and improving travel safety. These are achieved by smoothing the traffic flow at the merging areas (which increases the effective freeway capacity and reduces rate of accident), and decreasing the traffic entering the freeway which reduces the volume to capacity ratio (Gordon 2009).

As previously mentioned, one type of congestions occurring on freeways is recurrent congestion. There are four main reasons for freeway recurrent congestion: queues extending from freeway off-ramps to the mainline, bottlenecks (e.g. lane drops), entering traffic surpassing existing traffic, and interruption of mainline flow by platooned entering traffic. Freeway recurrent congestion caused by the last three causes can be prevented, moderated, or reduced by controlling ramp access to the freeway mainline (Wu 2001). Freeway on-ramps can be controlled by different strategies one of the most common of which is ramp metering. This indicates the significance of ramp metering in freeways to prevent or mitigate different recurrent congestions on freeways.

If in a metered ramp the metering rate is below the average arrival rate at the ramp, it is called restrictive a ramp metering. In this case a queue will build up along the ramp which will result in additional delay to the entering vehicles. Therefore, some of arriving vehicles will choose alternate routes, thereby decreasing the demand volume at the on-ramp merger with the freeway mainline. This eventually reduces the volume-to-capacity ratio at the merge as well as its downstream which will in turn lead to a reduction of delay in freeway mainline. On the other hand, if the metering rate is chosen to be equal to the average vehicle arrival rate, it is called non-restrictive ramp metering. In such a case shorter queues will be formed and normally no route diversion will occur (Gordon 2009).

2.2 History of Ramp Metering

As previously mentioned, freeways were initially conceived as the flawless solution for traffic of large metropolitan cities. However, soon after introducing them, issues such as freeway congestion and safety problems emerged. As a result of considerable growth in demand, speed, and congestion, vehicle collisions became more widespread on freeways. More frequent road accidents not only caused periodic deterioration of freeway traffic, but also signified safety of travels as one of the most important issues in freeways.

This encouraged extra effort to be made on investigating the relationships between freeway capacity and demand and the consequent effect of demand-capacity relationships on freeway congestion and safety. Developing such relationships led to better understanding of freeway flow which in turn motivated traffic and transport professionals to devise different methods for efficient management of freeway traffic. In the United States for example, ramp management strategies are now commonly used by traffic and transport authorities across the country. According to (Jacobson et al. 2006), by 2006 ramp metering systems had been deployed in 26 metropolitan areas across the United States.

Ramp metering is one of the key techniques developed as a result of these efforts. Following the United States, other counties around the world have started to implement ramp metering in their freeway networks. Table 2-1 shows a history of some early applications of ramp metering.

Table 2-1 History of some applications of ramp metering in US and UK

Location (Year)	Description
Chicago, Illinois (1963)	The first ramp meters which were manually controlled in the field by traffic enforcement.
Los Angeles (1967)	The first known ramp closure
Minneapolis/St. Paul, Minnesota (1970)	The first implementation of bus by-pass lanes at metered ramps which initially was controlled on a fixed-time basis.
Detroit (1982)	Ramp meters were a part of Michigan Department of Transportation's Surveillance Control and Driver Information (SCANDI) system.
Lincoln Tunnel (tunnel under the Hudson River, connecting Weehawken, New Jersey and Manhattan, New York City)	Lane changes were permitted in the tunnel and bottleneck flows were frequently experienced at the foot of the upgrade in the tunnel.
M6 J10 near Walsall, UK (1986)	The first trial ramp metering in UK

The increase in application of ramp metering around the world has also indicates the necessity for proper understanding of the required tools for effective implementation of the techniques in order to ensure expected goals are achieved.

2.2.1 Ramp Metering in Australia

Parallel to several other counties around the world, more focus has recently been made on ramp metering in Australia. So far, ramp metering has been used in a number of Australian freeways. In several occasions the technique was used merely in order to manage merging flow issues at particular ramps, rather than the whole state of freeway traffic. Table 2-2 lists some examples of application of ramp metering in major Australian cities.

Table 2-2 Some examples of application of ramp metering in Australia

Location	Description
Sydney	M4 Western Motorway (Wallgrove Road on-ramp); the M5 East Motorway (Kingsgrove Road on-ramp); and the Citywest Link to Anzac Bridge
Melbourne	The Eastern Freeway and the Monash Freeway
Brisbane	Pacific Motorway (on some on-ramps)
Melbourne	M1 (major freight route to the Port of Melbourne) Upgrade (62 ramp meters that are coordinated using HERO by a 20s interval time)

Currently, there are a number government funded projects on planning and applying strategies for management of freeway traffic, including ramp metering, across Australia. These are defined under National Smart Managed Motorways Program by the Australian Government Department of Infrastructure and Transport. The objective of the Program is to increase efficiency of motorways using smart infrastructure technologies to improve real-time management of major Australian motorways. This is obtained by reducing congestion and improving the negative effects of traffic demand in major metropolitan areas. Investigation and implementation of innovative solutions including ramp metering is part of the projects.

The Australian Government has guaranteed funding to state governments for initial projects in Sydney, Melbourne, Brisbane, and Perth. These projects were announced in 2011-2012 budgets and are expected to be completed by June 2014. Government has allocated $60 million for Managed Motorway projects to 2014-15 (DIT 2012). Current projects under this program are listed in Table 2-3.

Table 2-3 Current projects under National Smart Managed Motorways Program

Location	Description
Sydney	M4 (Western Motorway) – feasibility/project development funding for the introduction of a managed motorway system, including ramp metering and potential freight prioritisation, on this existing motorway.
Melbourne	M1 West Gate Freeway (Western Ring Road to Williamstown Road) – upgrade this section to a level 3 Intelligent Transport System (ITS).
Brisbane	Gateway Motorway (Nudgee to Bruce Highway) – Introduce pole mounted variable speed limit signs, ramp signaling, travel time signs and variable message signs
Perth	Feasibility and trials of technology including ramp metering, for example, Urbsol have recently completed a ramp metering design analysis for Main Roads Western Australia using VISSIM.

2.3 Ramp Metering Techniques

As stated before, ramp metering is one of the most common freeway management approaches used in major metropolitan areas. It controls ramp access to the freeway by traffic signals on freeway entrance in order to regulate entering flow rate to the freeway. This control is based on different methods to optimise the freeway and ramp flows. A schematic of ramp metering is shown in Figure 2-1. Ramp metering is applied to regulate flow in three areas: freeway on-ramp, freeway-to-freeway connector, and freeway mainline.

Ramp metering controlling approaches could be generally classified into two categories: fixed-time ramp metering and reactive ramp metering. The latter can be local reactive ramp metering or coordinated reactive ramp metering. Following sections provide a review of different types of ramp metering techniques.

2.3.1 Fixed Time Ramp Metering Method

Fixed-time ramp metering category is the simplest method of ramp metering which are based on simple static models. They are normally derived for specific times of the day based on constant historical demand of the area. No measurement is carried out for determining this demand in real-time. In this method, a freeway with several on-ramp and off-ramps is subdivided into different sections such that there is one on-

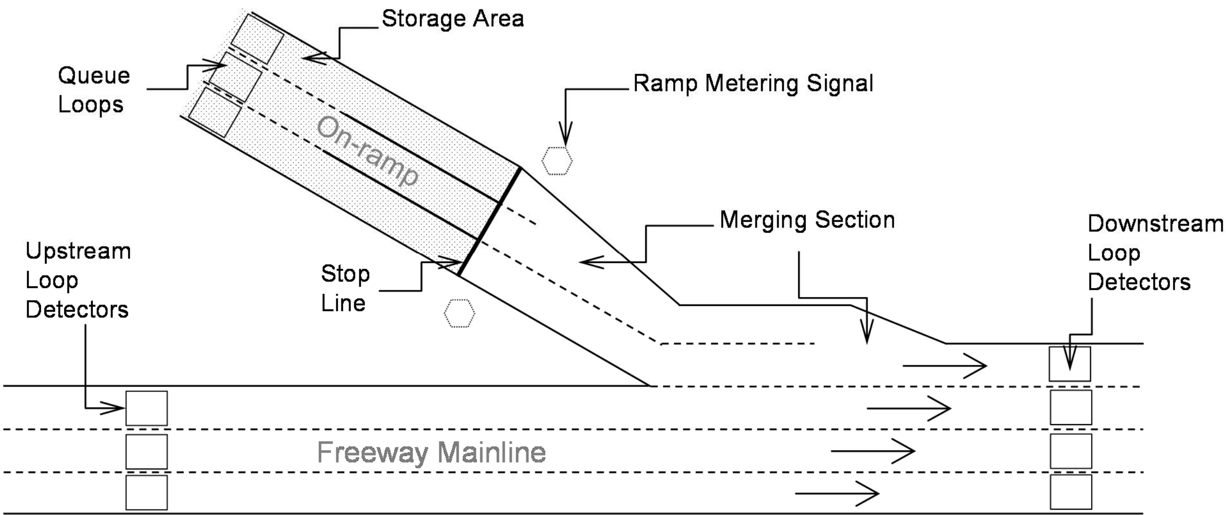


Figure 2-1 A schematic of ramp metering

ramp associated with each section. Then the following equation can be written on ramp volume and mainline flow:

$$q_i = \sum_{i=1}^{j} \alpha_{ij} r_i \qquad \text{Equation 1}$$

q_j = the mainline flow of section *j*

r_i = the inflow rate (in vehicles per hour) of section *i*

$\alpha_{ij} \in [0,1]$ = the known portion of vehicles that enter the freeway in section *i* and do not exit the freeway upstream of section *j*.

In order to prevent congestion, the Equation 2 must be met for the capacity of section *j* (q_{capj}):

$$q_j \le q_{capj} \qquad \forall j \qquad \text{Equation 2}$$

There is also a further constraint for the demand (d_j) and the ramp capacity at on-ramp *j* ($r_{j,\max}$):

$$r_{j,\min} \le r_j \le \min\{r_{j,\max}, d_j\} \qquad \text{Equation 3}$$

This method was first introduced by (Wattleworth 1965).

Equation 4 is to define the necessary criterion for maximising the number of vehicles. This would be equivalent to minimising the total time spent. And Equation 5 is to maximise the total travelled distance (Δ_j is the length of section j).

$$\sum_j r_j \rightarrow Max \qquad \text{Equation 4}$$

$$\sum_j \Delta_j q_j \rightarrow Max \qquad \text{Equation 5}$$

Furthermore, Equation 6 is a criterion to balance the ramp queues:

$$\sum_j \left(d_j - r_j\right)^2 \rightarrow Min \qquad \text{Equation 6}$$

The above formulations for fixed time ramp metering algorithm are then implemented in linear programming problems which can be solved using computers.

Fixed time ramp metering algorithms are based on historical data rather than real-time demand data of the area. However, this is not necessarily true in majority of real situations. Examples of such situations are areas whose demands are not constant in a particular time of the day, or areas whose demands may vary over different days. This fact is the major drawback for this category of ramp metering methods. Moreover, flow portions q_j in above equations are assumed to be constant. However, in reality these are not constant. They may vary due to drivers' response to any change in signal settings or events such as traffic incidents.

From the above discussion it become clear that ramp fixed time methods are not usually optimised methods for ramp metering. Due to factors mentioned above, they may over-utilise or under-utilise the freeway which is in contradiction with the main goal of freeway ramp metering (Papageorgiou et al. 2003; Papageorgiou and Kotsialos 2000).

2.3.2 Reactive Ramp Metering Methods

Another category of ramp metering strategies are reactive types. As opposed to fixed-time ramp metering methods, in these approaches the ramp metering response is a reaction to the observed real-time traffic conditions. It has been shown that dynamic operation of ramp signals that adapt to varying traffic of the freeway improve traffic flow and minimise flow breakdown more efficiently compared to methods based on fixed-time values (Burke 1972). In the following sections two different reactive ramp metering approaches are explained: Local and coordinated control methods.

2.3.2.1 Local Ramp Metering Algorithms

In this category of reactive ramp metering algorithms the ramp signal operates independently hence it does not react to the conditions experienced by adjacent on-ramps. Such isolated ramp signals on a freeway without any connection to other upstream ramp signals operate under local control ramp metering (Burley and Gaffney 2010). There are a number of different local ramp metering algorithms available. Some of the common approaches are introduced below.

- Demand-capacity

Demand-capacity is a feed forward algorithm which does not use output of the last iteration as the input for the next one. In this method the downstream occupancy is measured in the first step. Congestion is assumed to occur when the downstream occupancy is higher than the critical occupancy of its corresponding bottleneck. In such an event, the ramp meter switches to the minimum on-ramp volume value to eliminate the congestion. Otherwise, the metering rate would be largest value of the minimum rate, and the difference between downstream and upstream occupancy (Scariza 2003).

- Precent-Occupancy

Precent-occupancy is another feed forward algorithm. In this method congestion is identified by upstream detectors. Then the flow rate is estimated based on Equation 7:

$$r_k = K_1 - K_2 O_{in(k-1)} \qquad \text{Equation 7}$$

where r_k is inflow rate, K_1 is the capacity flow, K_2 is the slope of the straight line which approximates the uncongested section of fundamental diagram, and $O_{in(k-1)}$ is the last upstream occupancy (Hadj-Salem, Blosseville, and Papageorgiou 1990; Scariza 2003).

- ALINEA

ALINEA is an isolated, feedback-base ramp metering algorithm (Papageorgiou, Hadj-Salem, and Blosseville 1991; Papageorgiou, Hadj-Salem, and Middelham 1997). As a feedback strategy to control ramp metering, it uses its output as an input for the next iteration. Because of this ability ALINEA is stronger than previously mentioned algorithms. As the current study is focused on ALINEA, its linear quadratic (LQ) feedback function is described in more detail.

The algorithm is based downstream occupancy (O_{out}) of the entry ramp (on-ramp). O_{out} is measured by detector loops at the end of each time interval (T). This time period is suggested to be chosen between 20 to 60 seconds (Smaragdis, Papageorgiou, and Kosmatopoulos 2004) or 30 to 40 seconds (Azalient 2011).

The difference between measured downstream occupancy for the last time interval, $O_{out}(k-1)$, and the optimal set downstream occupancy O^* is used for calculation of the rate of the current interval $r(k)$:

$$r(k) = r(k-1) + K_R\left[O^* - O_{out}(k-1)\right] \qquad \text{Equation 8}$$

$r(k-1)$ is the last calculated metering rate, $K_R \rangle 0$ is the regulator parameter and the ALINEA function is not very sensitive to its value (Scariza 2003) and k = 1, 2,…

Figure 2-2 shows the flow-occupancy fundamental diagram. The downstream occupancy (O^*) is typically set slightly lower than the critical occupancy (q_{cr}) (Scariza 2003). This keeps downstream flow close to (but lower than) q_{cr}.

If $O_{out} \langle O^*$, then metering rate will increase in the next time interval by decreasing the cycle time of the ramp signal. On the other hand, if $O_{out} \rangle O^*$, then metering rate will be reduced in the next time interval by increasing the cycle time of the ramp signal. Nevertheless, the changes in metering rate should be in the range $[r_{\min}, r_{\max}]$ in which $r_{\min}$ is a minimum admissible metering rate and $r_{\max}$ is the

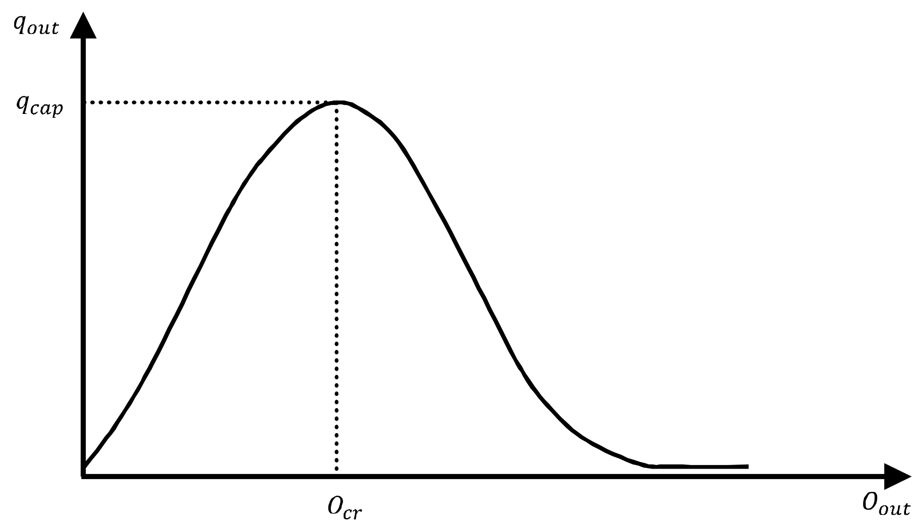


Figure 2-2 The flow-occupancy fundamental diagram

ramp's capacity. Therefore, when for example the calculated metering rate becomes lower than $r_{\min}$, the rate will be fixed to $r_{\min}$.

In two situations the ramp metering will be deactivated. Firstly, when the calculated ramp flow rate is higher than $r_{\max}$ and secondly, when the queue detectors occupancy exceeds a pre-set occupancy. The latter situation indicates the on-ramp storage is going to be fully occupied; hence there is a chance of queue building back to the arterial roads.

- ALINEA-Q

The improved version of ALINEA algorithm presented by (Smaragdis and Papageorgiou 2003) is ALINEA-Q algorithm. This algorithm not only is able to control the queue, but also can regulate the entering flow of freeway. To do so, it computes two different flow entrance rates. The first one, $r(k)$, is based on the downstream occupancy and is calculated by a procedure similar to that of ALINEA algorithm. The second one, $r'(k)$, is based on the length of the queue built up on entry ramp storage. This rate prevents growth of queue beyond the maximum tolerable queue length. Equation 9 is the formula used for calculation of $r'(k)$:

$$r'(k) = -\frac{1}{T}\left[\hat{w} - w(k)\right] + d(k-1) \qquad \text{Equation 9}$$

where $r'(k)$ is the minimum rate for controlling the ramp queue, $\hat{w}$ is maximum tolerable queue length, $w(k)$ is number of the vehicles in ramp storage at current time interval (k), T is the length of time interval, and $d(k-1)$ is the number of vehicles entering the ramp storage. The final output rate $R(k)$ of this algorithm is the maximum of the calculated rates mentioned above. So:

$$R(k) = \max\left\{r(k), r'(k)\right\} \qquad \text{Equation 10}$$

In this method, instead of loop detectors which measure occupancy of a point, the video detectors are used to measure $w(k)$ for the area of interest. If the algorithm was based on point measurements, it could not manage the queue before a special point. But by measuring and applying the current queue length after each interval it can smoothly prevent formation of a queue on the ramp storage. This is due to the fact that it considers the entire situation of ramp storage in detail rather than just a point from it. Therefore, it results in a more efficient flow rate for the vehicles released to the freeway.

2.3.2.2 Coordinated Ramp Metering Algorithms

As opposed to the local ramp metering algorithms, in coordinated ramp metering algorithms ramp signals in a freeway operate within an interconnected and coordinated ramp metering system. Nevertheless, ramp signals of a coordinated system may operate based on a local control algorithm when they are switched on for the first time or when downstream on-ramps are able to manage entering traffic flows without assistance of their upstream ramps for managing queues (Burley and Gaffney 2010). There are a number of such algorithms available and only some of the common ones are introduced in the following sections.

- METALINE

METALINE is actually an enhanced version of ALINEA ramp metering algorithm. The equation for this coordinate algorithm is similarly a further extension of ALINEA equation by vectorisation of terms. In METALINE, vectors of occupancy and two control gain matrices are used in order to return a vector of metering rate. Following is the equation used for obtaining the rate of current interval (Scariza 2003):

$$r(k) = r(k-1) - K_1(O(k) - O(k-1)) - K_2(O(k) - O_{cr}) \qquad \text{Equation 11}$$

- Flow

Flow is another area-wide coordinated ramp metering which calculates the metering rate based on the network condition as well as local capacity (Jacobson, Henry, and Mehyar 1989). This algorithm includes three modules. In the first module which is called local metering rate (LMR) predetermined rates are selected from a look up table base on the upstream occupancy for each on-ramp. This table was created on the basis of historical flow-occupancy relation. In the second module, the freeway is divided into some sections based on bottleneck locations which are determined using historical data. The congestion in each section is assumed to be mitigated by an influence zone that includes one or more on-ramps associated with the section. Here, the metering rate is called Bottleneck Metering Rate (BMR). It is calculated for each section only if the downstream occupancy of the section exceeds a threshold value, and the entering flow to the section is higher than its exiting flow.

BMR reduces the total rate of vehicles entering to the section by lowering the vehicle entering rate of influence zone associated with the section. This reduction is equal to the difference between exited flow and entered flow to the section which is called section storage. This reduction is distributed between on-ramps associated with the influence zone by considering weights that are computed based on on-ramp's distance from bottleneck and its demand. Afterwards, the minimum of LMR and BMR is selected as the system metering rate (SRM). Finally, the queue override strategy adjusts SRM in the last module.

Hasan, Jha, and Ben-Akiva (2002) compared the performance of ALINEA and Flow ramp metering algorithms. Their results show that Flow algorithm delivers better results in scenarios with downstream bottlenecks. Also, travel time reduction by Flow algorithm was larger than ALINEA in high levels of demand. However, it was reported that results of these two algorithms were similar in other aspects.

- HERO

Heuristic ramp metering coordination scheme (HERO) (Papamichail and Papageorgiou 2008; Papamichail et al. 2010) is implemented by generic software to regulate freeway on-ramps inflow. HERO also uses a modified version of ALINEA feedback regulator to manage ramp inflows. ALINEA is actually the local component of HERO's overall approach which includes several other components. Figure 2-3 illustrates how HERO scheme coordinates the local ALINEA blocks. Each ALINEA block may be installed locally for each on-ramp, while they are all connected to HERO. HERO is instead located in a central control room. There are dual connections available to submit the real time data to HERO and receive the control decision made from it. As shown by the figure, the real time data required by HERO is collected from three points at entrance, middle and exit of each on-ramp and from one or more (if required) mainline downstream bottlenecks of each on-ramp. The measured data is then modified by Data Processing module. Based on the

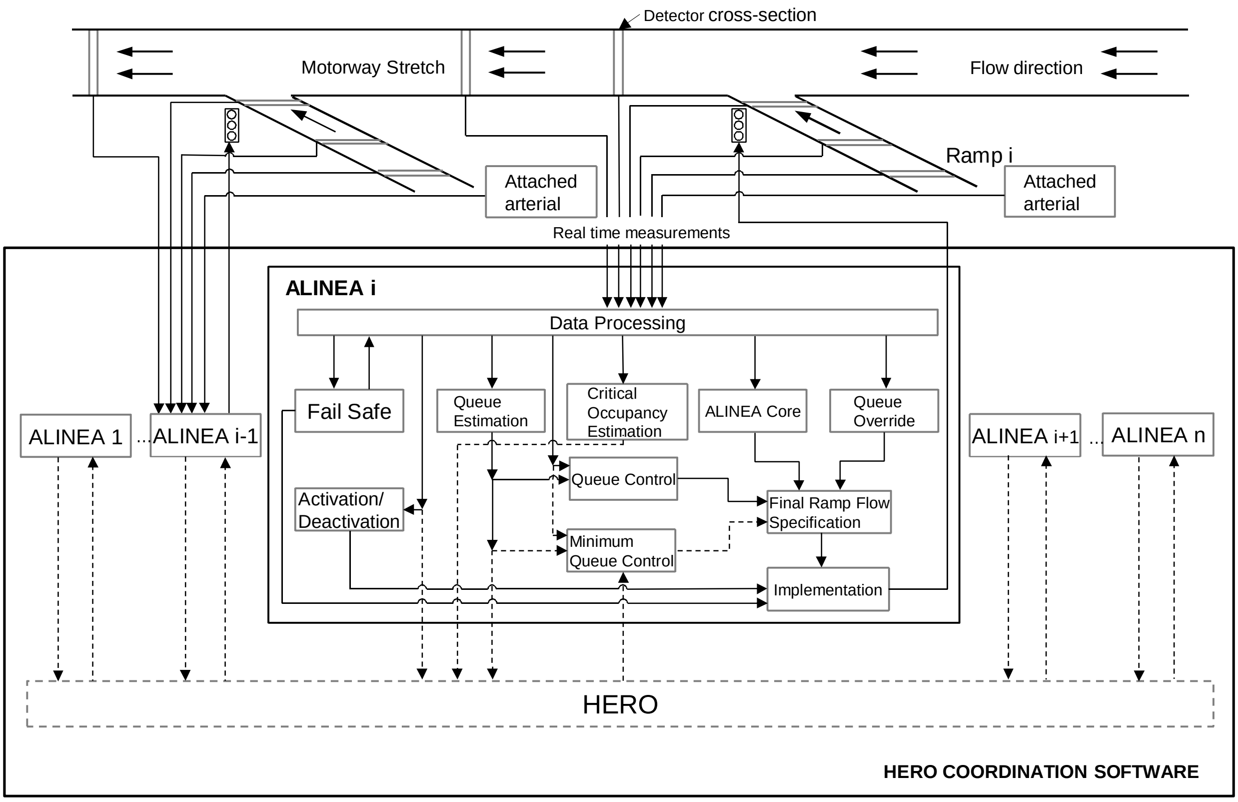


Figure 2-3 Modular structure of HERO coordination software

Source: Papamichail et al., 2010

current traffic situation at the freeway mainline, signal control of freeway entrance is activated or deactivated by the corresponding module. In ALINEA Core module, the freeway throughput rate is calculated based on ALINEA algorithm. If the critical occupancy is not available for ALINEA, it is estimated by Critical Occupancy Estimation module.

Controlling freeway entrance using ALINEA approach, there is always a chance to build up a queue on the on-ramp storage. To estimate the queue length, a Kalman filter approach (Smaragdis, Papageorgiou, and Kosmatopoulos 2004) is used in Queue Estimation module using data observed from detectors installed on on-ramp. The estimated queue length should be between zero and on-ramp's storage capacity. Furthermore, queue spill-over to the arterial road is prevented by applying a queue over ride or a queue control strategy. For more details on HERO algorithm the reader is referred to Papamichail and Papageorgiou (2008), and Papamichail et al. (2010).

2.4 Positive and Negative Effects of Ramp Metering

The main objectives of ramp metering are to optimise freeway throughput, travel speed, travel time reliability, and increase safety (Burley and Gaffney 2010). Although in the first look it may appear that ramp metering unquestionably improves traffic of freeway, there are also some drawbacks associated with this technique which may result in negative impacts on freeway traffic.

For this purpose, ramp metering is required to be applied appropriately in order to deliver expected positive effects. There are several advantages attained by proper implementation of ramp metering in freeways. For instance, delays will be reduced for users of high-volume freeways. Also, chance of traffic incidents in freeway mainline can be significantly reduced thus the deteriorating impacts of such events on freeway traffic will be eliminated. Furthermore, freeway throughput is increased at critical times and locations. Overall road network travel time is also improved by ramp metering. This is gained by enhanced operation of freeway which reduces delays of traffic entering the freeway. Ramp metering also results in impartial usage of road network. This includes redistribution of traffic in accordance with the capacity of freeway as well as discouraging use of freeway for short trips particularly over periods of high flow.

Improved safety is another positive effect of ramp metering. This is achieved by safer management of merging traffic (hence reducing risk of traffic incident) and establishing more stable speeds for freeway travels which means reducing stop-start conditions on the freeway. Moreover, as a result of efficient traffic conditions obtained by appropriate implementation of ramp metering fuel consumptions and associated carbon emissions are reduced (Burley and Gaffney 2010).

Despite all the positive effects mentioned above, inappropriate application of ramp metering can result in some negative effects. Some of the most important issues of concern with improper implementation of ramp metering are as follow (Wu 2001):

- Ramp metering may result in potential divergence of freeway trips to adjacent arterial roads. However, if the arterial roads do not have sufficient capacity for the diverted traffic from freeway, new traffic issues may occur in those parts of the network. In such a case, ramp metering creates a negative effect on the traffic of network.
- Ramp metering generally benefits longer trips while discouraging short trips to be made in the freeway. This causes extra costs for motorists with shorter trips. In other words, it can be said that generally suburban commuters benefit from the ramp metering while the situation is opposite for the residents close to central business district, where usually most ramp meters are located.
- There is possibility of queues to form on on-ramps of a metered freeway. In such a case, queues that extent back to the nearby arterials roads can undesirably disturb the network traffic.
- Last but not the least, stop-and-go conditions and vehicle queuing on the metered ramp may result in increased local emissions near the ramp meter.

2.5 Traffic Simulation

Traffic simulation refers to the mathematical modelling of different transportation systems such as roundabouts, freeway junctions, and arterial roads, using relevant computer software. Simulations are conducted to assist in planning, design, and operation of transportation systems. Using simulation one can examine models which may be too complicated for analytical or numerical approaches. Furthermore, although by field testing one can assess overall situation of a traffic system, calibrating different control parameters is very difficult. On the other hand, it is possible to examine different traffic plans and scenarios using traffic simulation.

Due to abovementioned factors traffic simulation is considered as the preferred choice in many traffic studies. In fact, traffic simulation is known as one of the most important disciplines in Traffic Engineering and Transportation Planning. A variety of transportation authorities and consulting companies use simulation approaches for management of transportation networks.

There are a number of traffic simulation software available. Approaches used by these packages can be categorised in four general groups: macro-simulation, meso-simulation, micro simulation and nano-simulation approaches. These methods are briefly discussed in the following paragraphs.

In macroscopic simulations vehicles are not simulated individually. Using continuum equations these approaches consider traffic as a combined flow. Compared to other approaches, macroscopic models generally require less data input and simpler coding. As a result they deliver outputs with lower levels of detail (Jones et al. 2004). Vehicle movements are regularly simulated as packets in a network. The time step used here is in the order of one to several seconds. In order to manage movement of a vehicle platoon in the network links, an analytical model (such as platoon dispersion model) is employed. This approach is useful for design and optimisation of networks (Luk and Tay 2006).

On the other hand, in microscopic simulation methods, movement of a vehicle is traced through a network. Such models use a combination of particular car-following, vehicle performance, and lane changing algorithms to model behaviour of individual vehicles (Jones et al. 2004). This is performed over the simulation time at small time increments normally in the other of a fraction of a second. Therefore, using microscopic simulation it is possible to carry out a detailed simulation of vehicle-road interactions under the influence of a control measure. Although this method is applicable to a variety of situations, it is computationally expensive and more effort is required for calibration of such models. Generally, optimisation of model parameters in this method is challenging (Luk and Tay 2006).

Meso-simulation (also called hybrid simulation) technique is a type of simulation that has features of both macro and microscopic models (Wu 2001). This technique combines a detailed microscopic simulation of important components of a model (e.g. intersection operations) with appropriate analytical models (e.g. speed-flow relationships for traffic assignment). It is also possible to interface a micro-simulation model with a real-time signal control system such as SCATS (Sydney Coordinated Adaptive Traffic System) (Luk and Tay 2006).

Nano-simulation is the most recent simulation approach. The term “nano-simulation” refers to the process of modelling each person through all modes of a trip. Such a high level of detail allows one to measure different parts of a single trip. It this way it is possible to apply different cost values to each part. These can include time and distance for driving a private vehicle or riding on public transport and associated fares, time and distance for walking, price of parking and carbon emissions. In such an approach, the trip cost is based on all parts of a trip regardless of the trip mode (Azalient 2011). One example of available nano-simulation packages is Commuter.

2.5.1 Ramp Metering and Micro-Simulation

In most of simulations that have been performed as a part of evaluation studies, macroscopic traffic simulators were used. On the other hand, field data indicate that there is a complex traffic pattern in and around merging areas (Cassidy and Bertini 1999). Actually, traffic flow in merging areas is a result of a complex interaction between freeway mainline and ramp traffic. This complex behaviour depends on different factors such as directional demand, driver behaviour, and road geometry. As previously mentioned, in macroscopic traffic simulation traffic flow in the network is coarsely represented. This reduces its capability for modelling interactions between individual vehicles in the model. Therefore, it is not appropriate to use macroscopic simulators for modelling and examining the ramp control algorithms (Hasan, Jha, and Ben-Akiva 2002).

On the other hand, the purpose of this study is to evaluate the effect of ramp metering in freeway which is a complex situation requiring detailed simulation. Microscopic simulation methods are competent in carrying out a detailed simulation of vehicle-road interactions under the influence of a control measure. Therefore, it was decided to use micro-simulation for modelling and evaluation of ramp metering in Mitchell Freeway.

2.5.2 Simulation Software

In this study two different traffic simulation packages were used: Q-Paramics (Paramics) and Commuter (Azalient 2011). In the following these two packages are introduced briefly. Details of modelling procedure are covered in the next chapter.

Quadstone Paramics (Q-Paramics) is a commercial microscopic traffic and pedestrian software. It can be used to design efficient, economical, driver and pedestrian friendly transportation infrastructure. The package allows operation assessment for current and future traffic conditions. Paramics models are scalable and it is able to simulate scenarios with different levels of complexity (*Quadstone Paramics*).

Commuter is a traffic simulation package with micro and nano-simulation capability. In nano scale it is capable of modelling door-to-door trips made by people. In such a nano-simulation all segments of the trip are modelled. These can include walking segments (i.e. from parking to office), self-driven segments (i.e. from driveway to city centre parking) and public transport segments (i.e. suburban stations to city centre) (Azalient 2011). It is also possible to use Commuter in the micro-simulation level for modelling vehicle-vehicle and road-vehicle interactions. As will be discussed in the next chapter, micro-simulation capability of the software is used in this study

2.5.3 Strength and Limitations of Traffic Simulation Modelling

While in some situations traffic simulation can be considered an appropriate approach, it may not be deemed proper for another situation. This is due to intrinsic limitations and strengths associated with traffic simulation modelling.

Simulation modelling has several strengths which make this approach interesting for studying different situations. Some of these strengths are listed below:

- In some situations traditional analytical approaches may not be appropriate which makes simulation modelling the appropriate tool.
- Using simulation modelling one can examine different scenarios and situations that do not really exist today.

- A valid simulation model can provide valuable information revealing the most important variables in network traffic and details on how they interrelate.
- Situations which can be potentially unsafe can be simply examined by simulation modelling without any risk to system users.
- Taking advantage of simulation one can model base conditions for impartial comparison of improvement alternatives of a network system.
- By performing sensitivity analyse one can study the effects of changes on the operation of a system.
- Interacting queuing processes can be modelled by simulation.
- Demand can be varied over time and space.
- Unusual arrival and service patterns which are not in accordance with traditional mathematical distribution models can be efficiently modelled by simulation (May 1990).

Nevertheless, there are some potential issues with simulation modelling which need to be considered before this method is employed for evaluation of traffic networks. These are as follows (May 1990; Stanescu 2008; Wu 2001; Algers et al. 1997):

- First of all, there may be easier and more traditional approaches to solve the problem than using simulation modelling.
- Some simulations can be computationally expensive thus may become time consuming.
- Generally, considerable input characteristic and data are required for simulation models. Obtaining such data may by difficult or in some situations impossible.
- Normally, simulation models require verification, calibration and validation that depending on the considered criteria may take a lot of time.
- Due to lack of proper documentation the simulation model may be difficult to use by non-developers.
- A proper simulation is not doable unless the system details are completely understood by the modeller. Some users may apply

simulation models without correctly knowing what they present or what the simulation assumptions and limitations are.

- To manage movements of vehicles in the model, the majority of simulation models use simple car following and lane changing algorithms. However, such simple algorithms are not capable of modelling congested conditions realistically.
- Normally, simulation models are used parallel with other models (i.e. assignment models). Although all such models require common inputs (i.e. origin-destination data), the input data for each model usually needs to be in a different format. Converting the format of data to suit requirement of each model and re-entering data require extra effort and time.
- After generating results from traffic simulations, different simulated cases require to be ranked. The criteria used for evaluation of different cases are some kind of performance indicators. To be consistent in ranking different cases, it is essential to carefully define some standard sets of performance indicators and procedures.

2.6 Summary

Since freeways are a key component of traffic network particularly in metropolitan areas, their management with the aim of their optimum usage very important. Ramp metering as one the most efficient tools for managing freeways, regulate freeway's inflow in a controlled manner.

The very first ramp meters were used in the United States in 1960s. Since then, several methods have been introduced for metering freeway ramps. Ramp metering has also attracted attention in Australia and there are currently a number of active projects on this subject.

Ramp metering algorithms can generally be divided into two main categories: fixed-time ramp metering algorithms and reactive time ramp metering algorithms. Each algorithm has its own advantages and disadvantages. ALINEA is the one of the most successful algorithms for ramp metering which is based on real-time data to locally control the traffic flow rate entering the freeway.

Compared to the field studies, simulation can be are more effective tool for studying and examining traffic of a network. Simulation also provides flexibility in evaluating and comparing different scenarios. Out of different traffic simulation methods, micro-simulation suits best the modelling of ramp metering on freeways. This is due to detailed level of simulation in this method which is required in modelling ramp metering.

3 Modelling

3.1 Introduction

Whilst it is possible to evaluate the overall performance of traffic control system through field testing, calibrating control parameters is almost impossible. This is because of uncontrolled factors such as weather, lighting, incidents and changes in traffic demands. It is also impractical to change the physical attributes of the system such as detector and stop-line locations. Substantial expense and the time consuming nature of field tests also adds to these complexities. Since the end of the last century, simulation tools have been introduced that allow analysts to assess a range of traffic plans and scenarios. Simulation also lets designers evaluate their designs and calibrate design parameters.

As mentioned in the previous chapter, different authors have conducted simulation of ramp metering for different traffic and transport networks. The main purpose of this current research is to develop a case study of ramp metering in a local network by means of simulation. This chapter covers the different steps undertaken for ramp metering modelling of Mitchell Freeway in Perth Western Australia. This includes data gathering, building the model, O-D demand estimation, designing ramp metering and calibrating its parameters. Simulation results however, are presented in a separate chapter.

3.2 Scope of Study

Western Australia (WA) is the largest state in Australia in terms of area. According to the Australian Bureau of Statistics (ABS 2012a), WA has experienced the highest population growth in the 2011-2012 period. This was mainly due to its rich mining and petroleum resources. The capital of Western Australia is Perth with an estimated population of 1.83 million (as of June 2011) which corresponds to 78% of the state's total population (ABS 2012b).

Although major mining, petroleum and agricultural export industries are located in rural areas of the state, Perth is the main economic and administrative centre for business and government. Its unique situation as the capital city of the state has also created some exceptional opportunities for development of several other businesses. These all have led to a considerable population growth in Perth. For example, from 2001 to 2011, the population of Greater Perth increased by 380,100 people (26% growth). This was the fastest growth of all capital cities in Australia (ABS 2012b).

With the significant increase in population and extension of the metropolitan area of the city, it is very important to appropriately develop and upgrade traffic and transport infrastructure in order to address the growing traffic demands.

There are three freeways and nine metropolitan highways in Perth's road network. Freeways include Mitchell Freeway, Kwinana Freeway, and Graham Farmer Freeway. Connecting different suburbs to Perth Central Business District (CBD), they have a key role in handling the traffic of the city. The map in Figure 3-1 shows Perth CBD, the surrounding areas and geometry of the three abovementioned freeways. As shown above Mitchell Freeway is the main route connecting northern suburbs to Perth CBD. As a result, in the morning of typical weekdays the traffic in Mitchell Freeway southbound reaches a peak. This is as a result of commuters heading to mainly Perth CBD, Graham Farmer Freeway, or Kwinana Freeway.

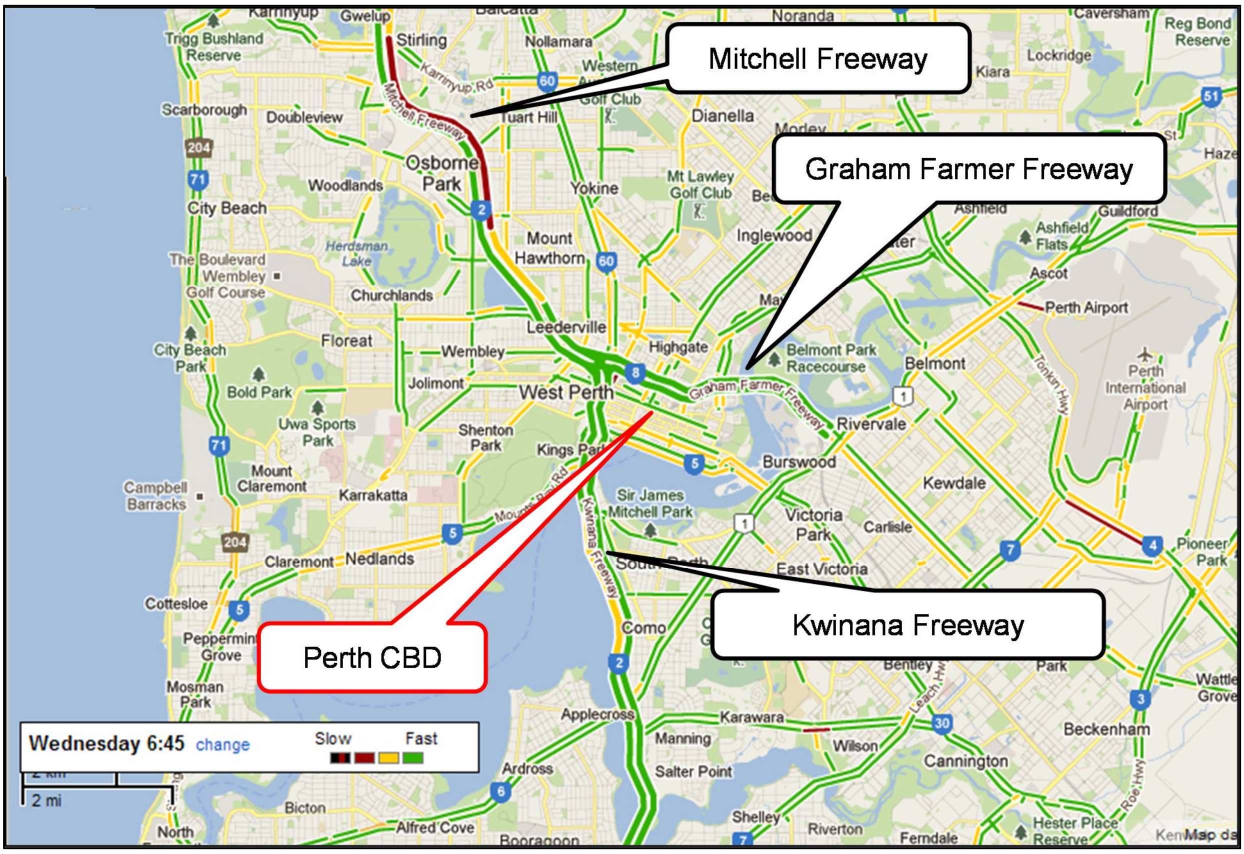


Figure 3-1 A map of Perth CBD and sourranding suburbs showing three freeways of the city
Source: Google Maps

In this study, a section of Mitchell Freeway from Hepburn Avenue to Graham Farmer Freeway (GFF) was considered as the geometric scope of the project, Figure 3-2 shows the extent of the study area. As ramp metering operation is most common during critical peak hours, a morning rush hour was decided as the appropriate time period of this study.

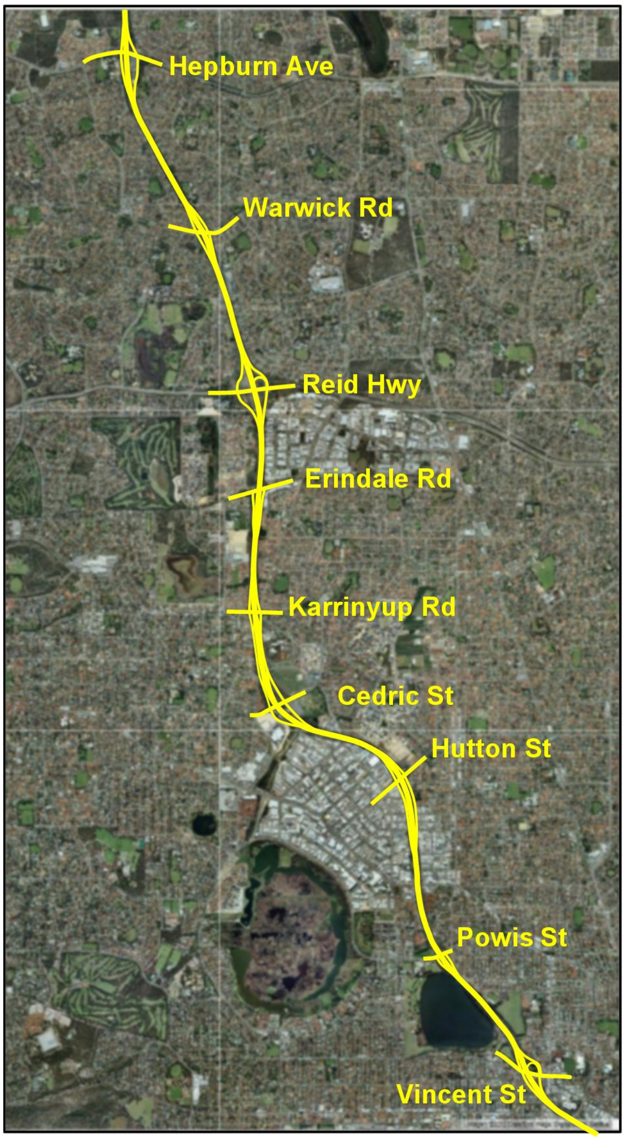


Figure 3-2 A map of Mitchell Freeway from Hepburn Avenue to Graham Farmer Freeway

3.3 Data Obtained from Main Roads Western Australia

This section outlines the data sources and treatment used in this study. All of these traffic data was obtained from Main Roads Western Australia (MRWA), the main government authority in charge of implementing the state's policies on the main roads and road access.

3.3.1 Flow and Speed Data

In simulation modelling, one of the core roles of link flows, cordon volumes and turn counts is to assist with matrix estimation which is often defined as an underspecified optimisation problem and as such these form one of the key inputs to this project.

There are a number of Vehicle Detector Sites (VDS) installed along Mitchell Freeway and its ramps, continuously gathering data such as traffic flow, density and speed. While dealing with MRWA to obtain the data for this research, the author was advised that unfortunately there were some gaps in the data due to coverage and operational problems. This may be due to different factors such as road construction works or technical problems with the data acquisition systems.

Generally speaking, the more complete the VDS data, the more reliable the estimated demand matrix for the network is. Therefore, it is desirable to obtain data from the most recent period during which most, if not all, VDS data are available. After searching through different data sets available in the MRWA database, the period with the most available VDS data for Mitchell Freeway and its ramps was identified. This period is from Monday 22 August 2011 to Friday 26 August 2011. The data comprised average flow and speed data in 15 minute bins on a per lane basis for the Mitchell Freeway southbound as recorded by 31 VDSs.

As a sample, Figure 3-3 shows a map of one of the VDSs (VDS number 310) located on Cedric Street off-ramp (lane 1 and lane 2), Karrinyup Road on-ramp (lane 3 and lane 4), and Mitchell Freeway (lane 5 - lane 8). As can be seen from the detector locations, each lane can be recorded separately.

It is now worth noting some of the observed data issues. Under uncongested conditions, the highest observed traffic volumes can be generally taken to represent infrastructure demand. However, once the flow has broken down due to insufficient capacity this can no longer be assumed to be true.

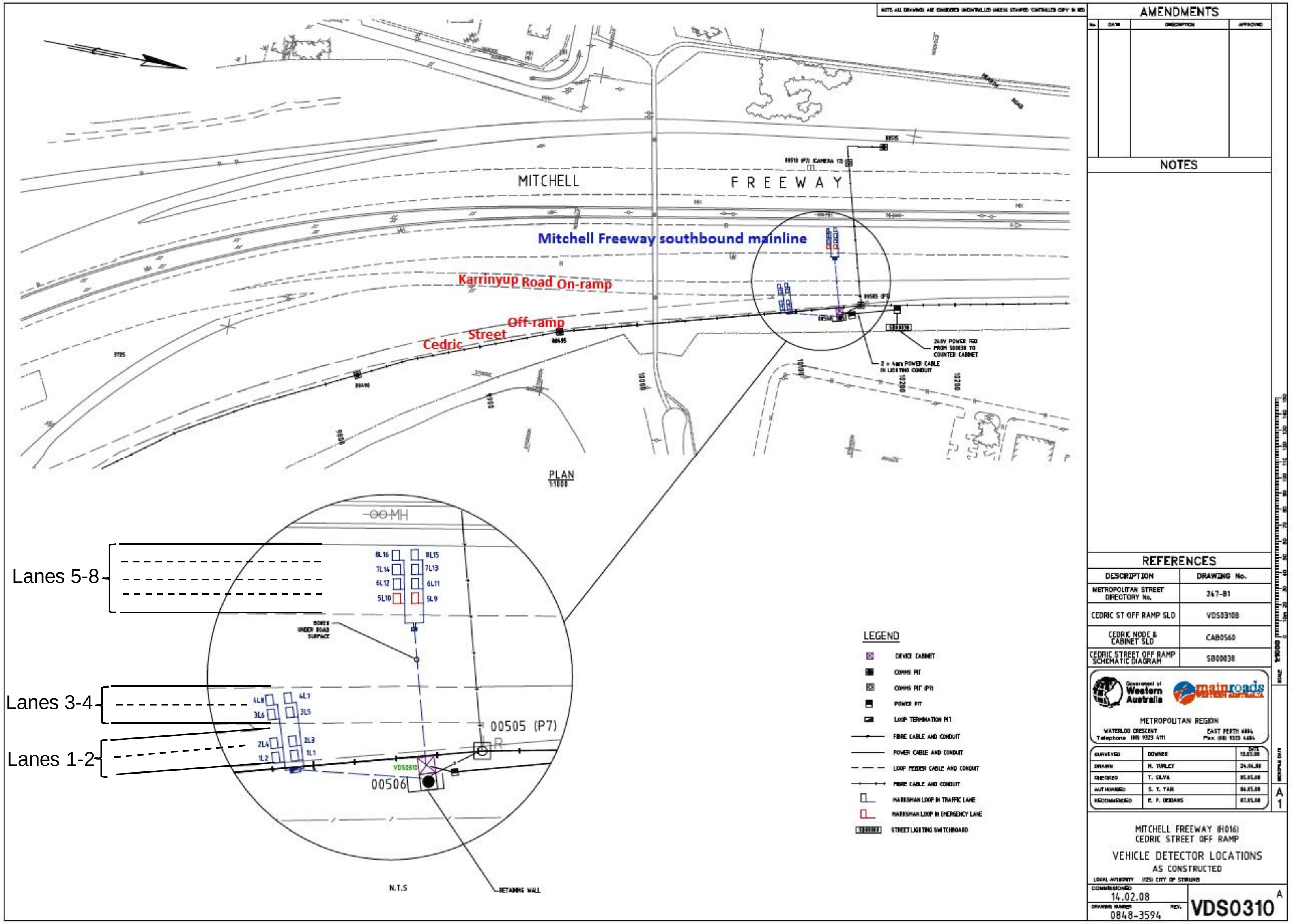


Figure 3-3 VDS 310 located on Mitchell Freeway, Karrinyup Road on-ramp, and Cedric Street off-ramp

A reduction in the observed volume following an apparent peak in flow can represent conditions where demand is higher but the capacity is simply insufficient to meet that demand. Once the system collapses under pressure it becomes less efficient and less capable of processing traffic. This is illustrated by the traditional speed-flow relationship shown in Figure 3-4. Under congested conditions observed data essentially represents network equilibrium as opposed to the true demand for the infrastructure.

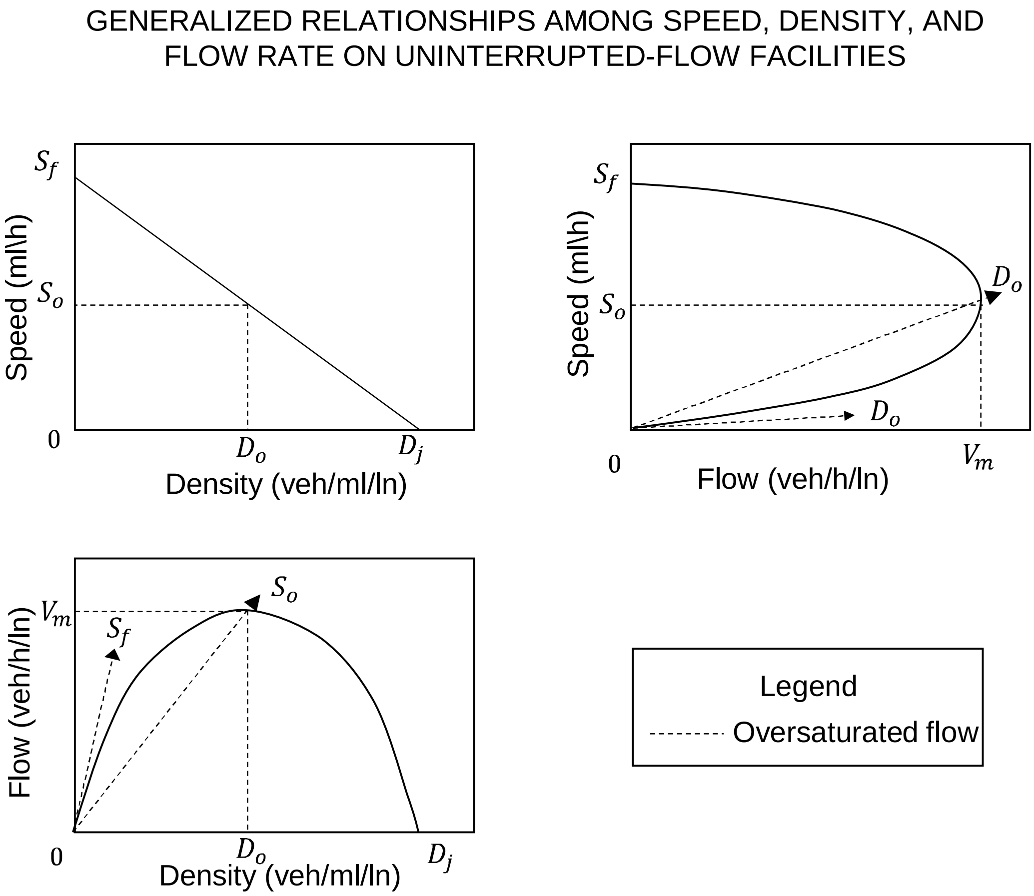


Figure 3-4 Speed, flow and density relationship

Source: (HCM 2000)

3.3.2 SCATS (Sydney Coordinated Adaptive Traffic System)

SCATS is an advanced computer system originally developed by the New South Wales Roads and Traffic Authority. This system monitors the traffic signals and volumes of traffic in real time. Traffic demand and traffic flow data are recorded by sensors installed in the traffic lanes at the stop lines for the traffic signals. The monitored data is used to coordinate and optimise adjacent traffic signals with the ultimate aim of easing traffic congestion and improving the traffic flow. SCATS is recognised as one of the most advanced urban traffic control systems in the world (MRWA).

SCATS data obtained for this study are from the intersections at the ramp terminals (on and off) and from neighbouring sites. This data serves 2 key purposes:

- To assess the consistency of the VDS data
- To backfill data gaps where VDS data was not available or corrupted

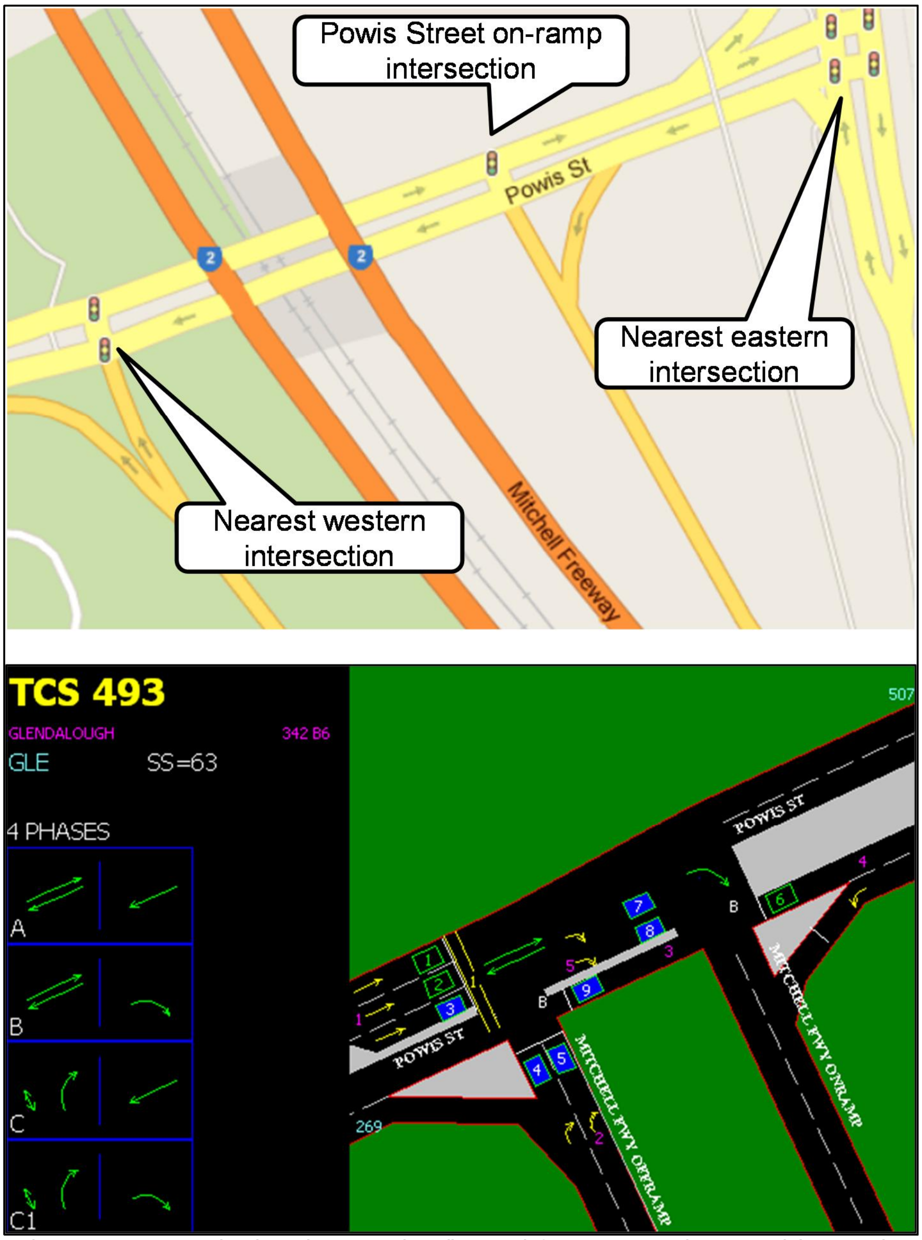


Figure 3-5 A map (top) and a snapshot (bottom) from SCATS (TCS 493) installed on Powis Street intersection

Source: Main Roads Western Australia

As an example, Figure 3-5 shows a map (top) and a snapshot of the SCATS Traffic Control Site (TCS) layout from the Central Manager for the Mitchell Freeway / Powis Street ramp. The bottom snapshot shows that sensors 7 and 8 record Powis Street eastbound to the on-ramp. However, as the Powis Street westbound to on-ramp lane is a "Give Way" intersection, there are no sensors installed to detect the traffic data. As a result, the real throughput of Powis on-ramp is not reflected by the SCATS data. Some of the freeway intersections studied here had similar configurations. Therefore, some of the SCATS data for this area was not helpful.

3.3.3 Peak Hour Sub Area Matrix of Study Area

Generally, a “pattern” matrix is used for estimation of the demand matrix. This pattern matrix can be based on an old estimated matrix for area of study or cordoned from a larger strategic model which includes the area of study.

The pattern matrix used here is a peak hour sub-area matrix from Main Roads Regional Operations Model (ROM). ROM is an 1160 zone strategic model built on the CUBE / VOYAGER platform.

For modelling purposes, two zones are defined for each arterial road on the interchanges with Mitchell Freeway: the western zone and eastern zone. A correlation table was developed that related the strategic level zones from the ROM model to the simulation model zone system. Figure 3-6 provides a schematic of one of the Mitchell Freeway interchanges and zones. In this figure, B1, B2, and B3 are the zones in the original pattern matrix which guide traffic flow to the same

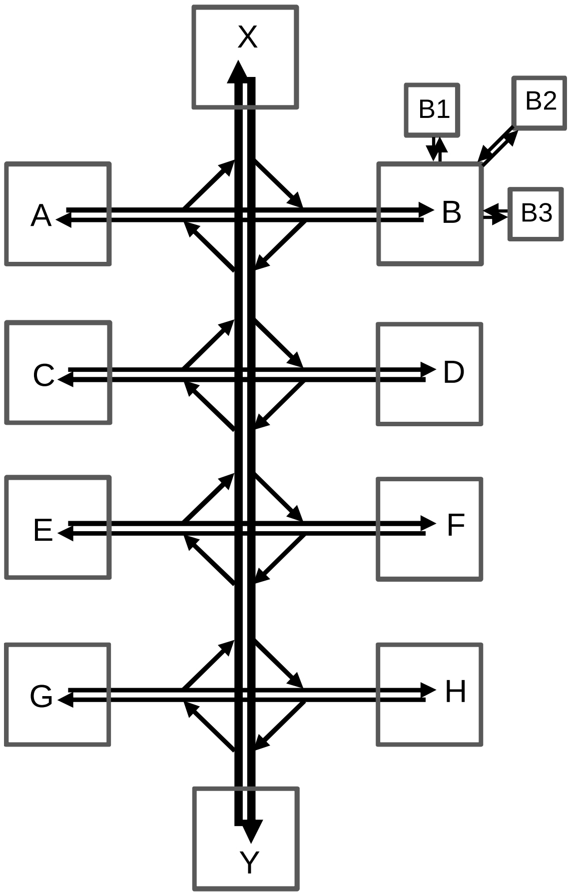


Figure 3-6 A schematic of original pattern matrix zones, and newly defined western and eastern zone pairs

interchange of freeway. As all of these zones are located in the newly defined zone B, they are all combined and defined as the zone B in the modified pattern matrix. The same process was followed for defining all new zones. Totally, 20 zones were defined including nine western zones, nine eastern zones, the top zone and the bottom one. In Figure 3-6 zones A, C, E and G are the newly defined western zones for their corresponding interchanges. Similarly, zones B, D, F and H are the eastern zones of interchanges. Zone X is the top zone and Zone Y is showing the bottom one. As mentioned above, the travel between each western and eastern zone pairs such as zone A and B is assumed to be zero.

Given focus of the study was on the flows entering and exiting the freeway itself no traffic is modelled across the arterial roads (east to west and vice versa).

As it is assumed that no travel is occurred between each zone pair, all travels from each western zone (origin zone) to its southern destination zones need to be carried out from the origin zone via its eastern zone. For example, any travel from zone C in (see Figure 3-6) to its southern zones (E, F, G or H) must occur from zone C to zone D, to enter the freeway southbound on-ramp and reach the destination. To implement this assumption into the modified pattern matrix, all southward travels starting from each western zone (for example zone C) are assigned to its corresponding eastern zone (for example zone D). Furthermore, as the focus of this study is solely on the freeway southbound, all northward travels from all western and eastern zones are assumed to be zero. Appendix 7.1 shows the final modified pattern matrix which is matching the newly defined zones. Later it will be seen that this matrix is used as the pattern matrix for estimation of O-D matrix.

3.4 Building the Base Model

To analyse the effect of ramp metering on Mitchell Freeway, it is required at the first step to construct a base model upon which other analyses can be conducted. Such a model is valid only when it suitably represents the real conditions of the freeway. That is why calibration of the model and its verification are essential. In the following sections the steps taken for construction of a valid bade model are explained. These include selecting appropriate modelling period, coding the model, estimation of origin-destination matrix by simulation, defining time windows, determination of traffic profile, choosing suitable vehicle motion algorithms, and finally calibration and validation of the model.

3.4.1 Modelling Period

As mentioned before, traffic in Mitchell Freeway southbound reaches a peak in the morning of typical weekdays. Thus, the most appropriate time for studying current situation of the network is modelling it for the AM (morning) peak hour with the highest existing demand of the freeway southbound. To determine modelling time period, traffic counts data of 14 link VDSs located on the Mitchell Freeway mainline is studied.

Due to the relatively long length of the model (more than 16 km), the peak hour of different sites of the model were expected to be different. To pick only one period as the AM peak hour, it was decided to focus on the freeway section of interest as a whole. Therefore, in order to estimate the peak hour, the total flow of different VDSs in the network was taken into account. Figure 3-7 shows a graph of total flow, average speed, and space mean speed for each 15 minute interval. As the

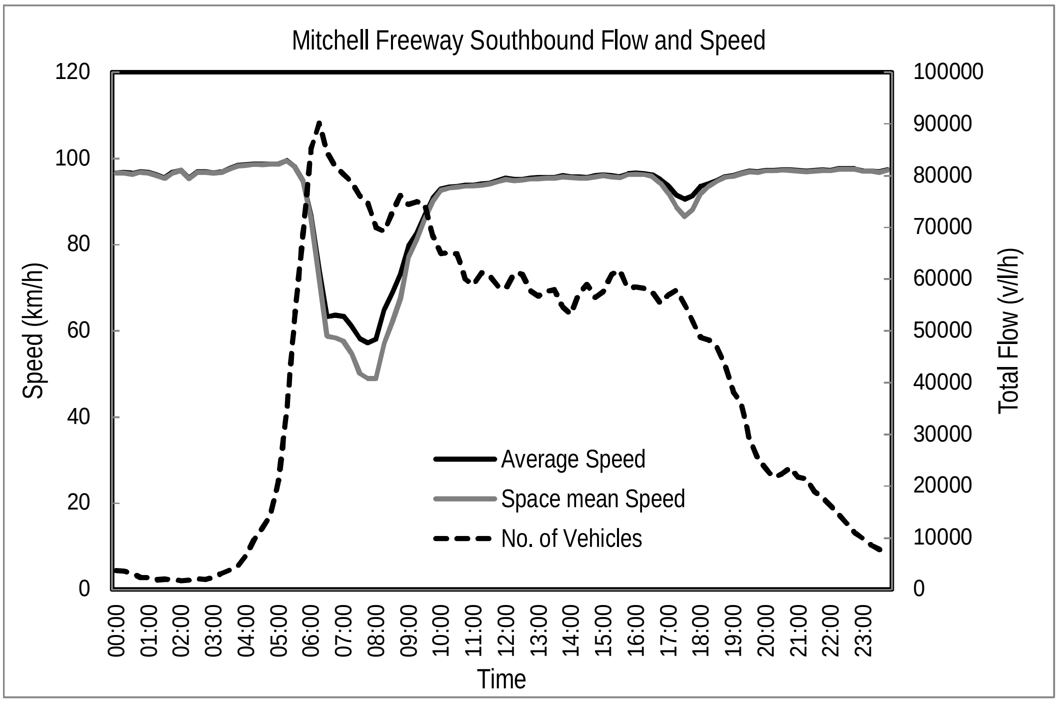


Figure 3-7 Total flow, average and space mean speeds versus time

plot shows the maximum number of vehicles was counted by detectors at 6:15AM. However, the minimum speed occurred during the 7:30-8:00 am period at about 50 km/h. This indicates the smaller number of vehicles detected by the system is not necessarily due to a lower demand. It is rather as a result of lower vehicle speeds. Hence, it is not cautious to simply obtain the peak hour demand using available data. Determination of the precise peak hour demand is however, not in the scope of current study. As a reasonable assumption, the highest flow of the freeway was taken as the maximum demand of the model. Therefore, the assumed one-hour modelling period is from 6:15 am to 7:15 am. Nonetheless, it should be noted that this is not the real peak hour of the freeway. As will be described in later sections, after the validating the model for this time period, the demand will be increased to study the state of the freeway in higher demands.

3.4.2 Time Windows

Normally, it is required to define a number of time windows in a simulation model. In Commuter, this is implemented using “Term” option in which start and finish time of the terms are specified. Defining terms (time windows) offers the ability to specify different conditions for specific time periods of the model. In the current study, the total simulation period was from 5:30 am to 8:15 am. This total time was divided into three smaller time windows as follows:

1- Warm up period from 5:30 am to 6:15 am. This is explained in detail in the next section.
2- AM simulation period: from 6:15 am to 7:15 am which is the main time period of this study. Most simulation data is also acquired during this period.
3- Final period: from 7:15 am to 8:15 am. Some of the vehicles released to the freeway in AM simulation period may have not finished their journey by the end of AM simulation period. Therefore, the final period was defined to allow all the vehicles to complete their journeys. Over this period not more vehicles enter the freeway.

3.4.3 Warm-up

One of the important steps before running the model is to define the warm up period. In reality, before 6:15 am, which is the start of AM simulation period, there are some vehicles already travelling in the freeway. However, at the start of the model there is no traffic present in the links. To account for existence of vehicles prior to AM simulation period in the model, a warm up period was defined to establish the initial flow in the model.

Usually, the warm up period is taken to be shorter than the simulation time. In the current model by considering the size of the model and after examining different values, the warm-up period length was set to 45 minutes, from 5:30 am to 6:15 am. This was to ensure the whole network is affected by the warm up demand thus making the situation as close as possible to the real network situation.

Generally speaking, warm up demand is set to a value slightly smaller than the actual demand of the main simulation. As mentioned above, the length of the warm up period is 45 minutes which is 75 % of AM simulation period length. If a demand equivalent to the AM simulation period is to be set for the warm up period, it should be the same proportion (75%) of the AM simulation demand. Figure 3-7 indicates that in the last 15 minutes before the simulation time (6:00 am – 6:15 am), the observed counts data of the network is still large. Therefore, an attempt was made to choose a warm up demand as close as possible to the equivalent AM simulation period. Examining a number of values, it was found out that a warm up demand equal to 74% of AM simulation demand generates a flow as high as that of observed data. The calibrated warm up period length and demand were then input to the model.

3.4.4 Coding the Model

The model of the Mitchell Freeway from upstream of Hepburn off-ramp to downstream of Vincent Street towards Graham Farmer Freeway was coded in Commuter (a micro and nano-simulation package) and Q-Paramics (a micro simulation package) considering left side drivers.

The majority of land in Perth is relatively flat and as such the vertical dimension of the freeway mainline has been disregarded in this study. In order to code the horizontal alignment an overlay was prepared using imagery from the

NearMap database. Figure 3-8 shows how the model coincides with this overlay around the Vincent Street exit.

The geometric map described before was used for importing geometry (e.g., length, width, curvature, and number of lanes of each link) of the model area to the software. The overlay map was then scaled to the real geometry of the area. The coded model contains freeway mainline, on-ramp and off-ramp link, as well as 20 zones whose definition is explained in the next section. Based on reality speed limit of freeway mainline and ramps are defined 100km/h and 80 km/h, respectively. Only the southbound of the coded model is used for modelling the freeway AM rush hour. As described before, there is no traffic considered in the northbound of modelled freeway and all the travels are occurring in the southbound. Figure 3-9 is a snapshot of the coded model in Commuter showing freeway mainline, intersecting arterial roads, off-ramps and on-ramps, and location and number of zones in the model. The blue cursor shows the scale of the model.

Figure 3-8 Using overlay to code geometrical information of area around Vincent Street interchange with Mitchell Freeway

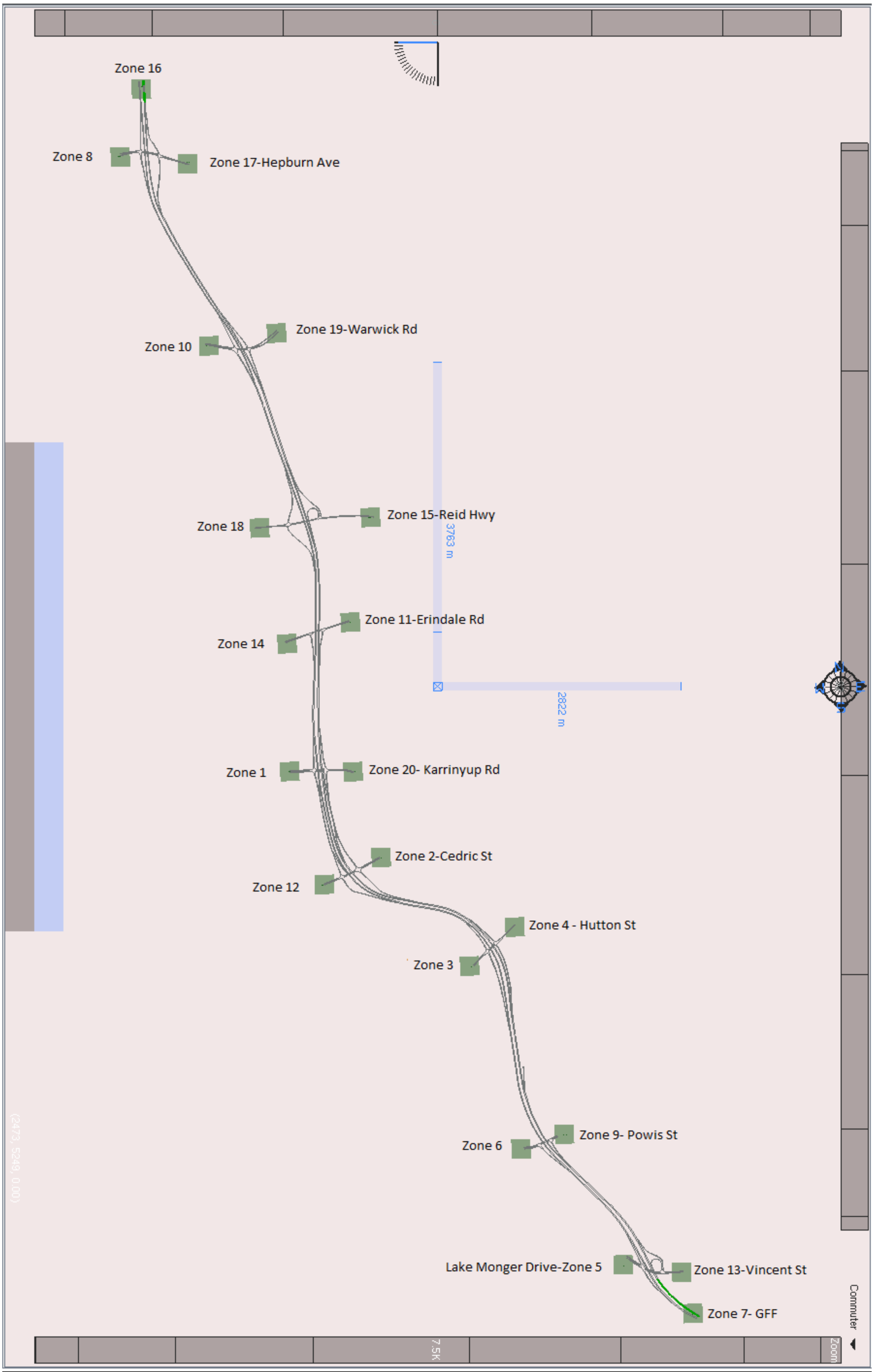


Figure 3-9 Coded model in Commuter and zone's location

3.4.5 Traffic Profile

By default of the software, the estimated demand of zones is released with a constant rate. However, in reality the zone demands are not necessarily released with a constant rate. For example, as normally most travellers desire to reach CBD by the start of the morning business hour, in the first half of the study time more travels occur from the top zone (which is farthest from Perth CBD) compared to a zone which is close to CBD.

In order to increase resemblance of the simulations to the reality, the rate of demand release for each zone of the model was selectively defined. The estimated origin-destination table in Section 3.4.7 is considered as the one-hour flow between each zone. It was noticed by the author that the observed travel times between some zones were quite long (about 30 minutes). Therefore, it was decided to define a profile with four equal periods of 15 minutes length. Such a profile was constructed based on the observed count data.

Table 3-1 contains the obtained profile data which was then applied to the base model. As expected, the percentage of the volume in the zones farther from CBD is higher during the early modelling times. On the other hand, the zones close to CBD have larger number of released vehicles in the late modelling times. It was understood that using a profile is very important in current model. Particularly, as the model is relatively long, the rate by which vehicles are released could considerably change the congestion area and its intensity.

Table 3-1 Model profile

Zone\Time	6:15-6:29	6:30-6:44	6:45-6:59	7:00-7:15
Karrinyup Road (West)	0.25	0.25	0.25	0.25
Cedric Street (East)	0.23	0.25	0.26	0.27
Hutton Street (West)	0.25	0.25	0.25	0.25
Hutton Street (East)	0.22	0.25	0.24	0.30
Lake Monger Drive	0.25	0.25	0.25	0.25
Powis Street (West)	0.25	0.25	0.25	0.25
From GFF	0.25	0.25	0.25	0.25
Hepburn Avenue (West)	0.25	0.25	0.25	0.25
Powis Street (East)	0.22	0.25	0.24	0.30
Warwick Road (West)	0.25	0.25	0.25	0.25
Erindale Road (East)	0.23	0.25	0.25	0.27
Cedric Street (West)	0.25	0.25	0.25	0.25
Vincent Street	0.19	0.24	0.27	0.30
Erindale road (West)	0.25	0.25	0.25	0.25
Reid Highway (East)	0.30	0.26	0.24	0.20
North of Hepburn Avenue	0.25	0.25	0.26	0.24
Hepburn Avenue (East)	0.30	0.26	0.20	0.24
Reid Highway (West)	0.25	0.25	0.25	0.25
Warwick Road (East)	0.30	0.26	0.20	0.24
Karrinyup Road (East)	0.23	0.25	0.26	0.27

3.4.6 Vehicle Motion

Size of the vehicles is an important factor in determining the behaviour of vehicles. Having different car following algorithms, this fact is suitably taken into account in Commuter. In the current model, three types of vehicle are used: small car, medium car and large car.

There are two main car following algorithms in Commuter: Wiedemann algorithm and Fritzsche algorithm. As the Wiedemann algorithm is suitable for free flow situations, it was used for the freeway mainline. However, a number of parameters are required to be defined in this algorithm. It was found out that running the model with default parameter values did not result in a realistic behaviour of the model. Different values were then selected based on a trial and error approach to obtain a realistic result from the model. Figure 3-10 shows the final parameters chosen for vehicle movement in the freeway mainline.

The Wiedemann algorithm was also initially used for freeway ramps and arterial links. However, it was revealed that this algorithm is not a particularly good candidate for links releasing vehicles in the model. Examination of the Fritzsche car following algorithm proved that this algorithm deliver more realistic results for ramps and arterial links. Therefore, it was selected as the preferred approach for vehicle movements in the ramps and the arterials roads of the model.

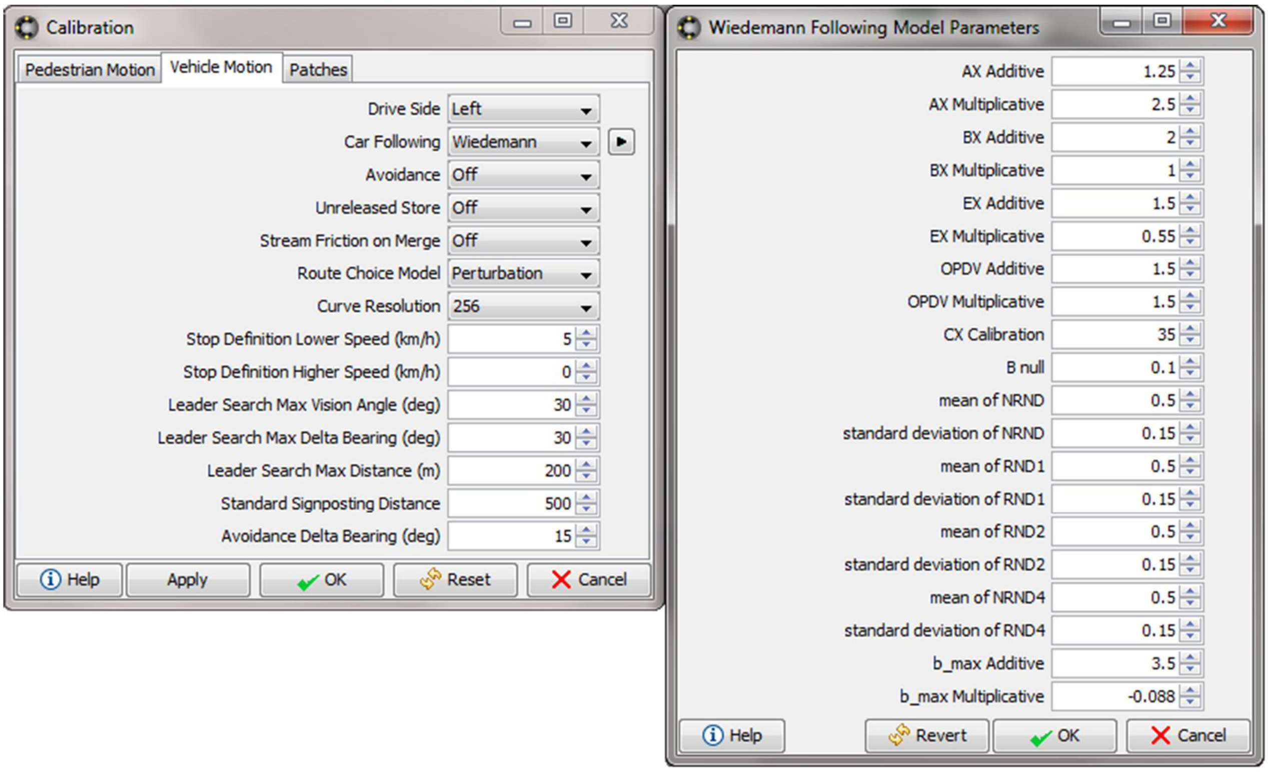


Figure 3-10 Vehicle motion parameters

3.4.7 Origin-destination Matrix Estimation by Simulation

Estimation of traffic demand is one of the most important and generally problematic tasks in traffic simulation studies. Under congested condition this is particularly complex. This is especially the case in large free flow networks such as freeways.

For estimation of demand matrix a range of data (explained in previous section) was used:

- VDS data which was obtained from Main Roads freeway data collection system (See Section 3.3.1)
- Intersection volumes from ramp terminals which were collected from the Main Roads SCATS system (See Section 3.3.2)
- Peak hour demand matrices obtained from Main Roads Western Australia's Regional Operations Model (See Section 3.3.3).

Demand estimation for micro-simulation models can be undertaken in a number of ways. However only few software packages offer a means that directly employs the network to be simulated in the task. External estimation using methods in these instances is possible however, this process becomes more focussed on the demand with less concern for the actual capacity of the network. This fact can invariably lead to a disagreement between the two. In the current study a number of approaches were considered.

3.4.7.1 Quadstone Paramics Estimator

The codded model in Q-Paramics Modeller was used to estimate origin-destination matrix in Paramics Estimator. Although Paramics was not the simulation engine chosen for the project, its ability to estimate traffic using the modelled network was felt to be advantageous and any resulting matrix would have a higher probability of being suitable for use in the Commuter model. Estimator is described by the software developer as undertaking an "underspecified simultaneous multivariate optimisation using confidence weights". The program essentially uses the modelled network and a range of observed data to search a very large problem space for probable solutions to the demand problem.

The inputs for the Estimator include the coded model in Modeller, the pattern matrix, the observed count data including cordon, link and turn counts and their confidence weights. Three different estimation methods used Q-Paramics Estimator: periodic normalisation, incremental, and combined method.

In the case of periodic normalisation, all trip values are normalised at the end of each calculation period. By appropriately adjusting the trip value, the difference between the modelled and observed values on all associated flows is minimised. In the normalisation process, the ratio of modelled to observed values is calculated for all associated flows. In the next step mean of the logarithm of ratios is obtained. The exponent of this mean is then multiplied by the trip value. This normalisation process is repeated iteratively after every calculation period and will finally find a solution in which all observed flows are satisfied.

In the incremental method, trip values are continuously adjusted during the calculation period. Every time a vehicle passes a flow element, the trip value of the vehicle's present O-D is checked. In case the modelled flow value is below the observed value and outside the confidence interval, the O-D trip value is increased by the present increment for the vehicle. Furthermore, the increment value for the vehicle is increased up to the maximum increment value. Similarly, if the modelled value is above the observed value and outside the confidence interval, then the O-D trip value is decreased by the present increment of the vehicle. Furthermore, the increment for the vehicle is decreased. In the case the new value is zero, the vehicle is removed from the network.

Finally, the combined method uses both Periodic Normalisation and Incremental algorithms for estimation process (Paramics).

During estimation process, each of abovementioned methods was repeated several times. Matrix estimated after each calculation cycle was evaluated by static GEH. The matrix with the lowest GEH was chosen as the best option. If after a number of cycles no improvement in the result was observed, it was assumed that the result is constrained within a local optimum value. In such a situation, another method was chosen for estimation of O-D matrix. Finally, when it was judged that no improvement in the result is likely to occur, the best obtained matrix was assigned to the model. Using Q-Paramics Analyser, the best obtained matrix was then compared to the observed count data to find out if the obtained matrix is acceptable. GEH values of all the links and cordon counts were used as the main criteria to evaluate suitability of obtained matrices. It was found out that none of the obtained matrices were close enough to the observed count data.

It is now worth pointing out two key issues with this approach:

- One of the main issues with Estimator is the ability to model a period of time that exceeds 1 hour in duration. With the scale of the network in question many trips were observed to have travel times that exceeded 20 -30 minutes. This means at any given time the matrix being tested by the program would represent a hybrid demand of two time periods and there was no way to guarantee the resultant demand would match the counts.
- Commencing the exercise with an empty network also led to large oscillations in the next matrix early in the process as the program would start scaling up the magnitude in an attempt to match the counts. This frequently led to an over-specification in volumes and gridlock conditions.

3.4.7.2 Commuter

The problems identified above meant the use of Estimator would be problematic and as such less than suitable for the task. Thus Commuter package was used for O-D estimation.

Commuter has an undirected demand modelling capability that allows for an unlimited timeframe to be modelled. Therefore, the model could be allowed to run until all traffic in the system had completed its journey. For undertaking the estimation, cordon traffic volumes were identified from the Main Roads VDS data source and used as targets for traffic release from each zone. Furthermore, on and off ramp volumes were used to steer the estimation process. In the next step, flow proportions of each interchange were defined for the program. Thus, for the interchanges of the freeway and arterial roads in the model, 100 precent of each on-ramp's flow was released in the freeway mainline. On the other hand, for each off-ramp the proportion of mainline and off-ramp flow was calculated based on the observed traffic counts.

Table 3-2 shows calculated proportions at each off-ramp interchange for 6:15 am to 7:15 am traffic volume.

Table 3-2 Proportion of the flow for the mainline and off-ramp links

Zone	Location	VDS	Off-ramp	Total	Portion to off-ramp	Portion to mainline
ZONE 17	Hepburn Ave off-ramp	400	88	4779	0.02	0.98
ZONE 19	Warwick Rd off-ramp	380	112.25	6464	0.02	0.98
ZONE 15	Reid Hwy off-ramp	360	1131	6469	0.17	0.83
ZONE 20	Karrinyup Rd off-ramp	330	327.5	6624	0.05	0.95
ZONE 2	Cedric St off-ramp	310	833	5810	0.14	0.86
ZONE 4	Hutton St off-ramp	290	708.5	6655	0.11	0.89
ZONE 13	Vincent St off-ramp	250	668.75	6767	0.10	0.90

By running the model in “undirected demand” mode, unlike the normal case in which the trips are generated based on a pre-determined O-D matrix, travels between each zone were recorded by the software. These were then used to build the estimated O-D matrix. The final table obtained after simulation time of one hour was taken the primary estimated matrix. Running the model with this matrix would result the same cordon traffic counts as the observed data. However, because there are always several matrices leading to the same set of traffic counts, manual refinement was used to remove illogical trips and to better shape the resultant matrix to match the peak hour patterns obtained from the Main Roads Model. To do so, assuming a GEH factor of 5, a range was defined for the total value of each row equivalent to each on-ramp’s flow, and the flow from the top zone. Similarly, the same range limit was defined for the total value of each column corresponding to each off-ramp’s flow and the flow to the bottom zone. Afterwards, the following process was repeated until the closet result to the pattern matrix was obtained:

For each cell of the current estimated matrix, the ratio of the cell value to the summation of all matrix cells is calculated. The same ratios are obtained for all cells of the pattern matrix. The ratios are then compared and if the cell ratio of the current estimated matrix is less than the same cell of the pattern matrix, the current matrix cell value is increased until a ratio close enough to that of the pattern matrix is reached. Similarly, if the cell ratio of current estimated matrix is more than the same cell of pattern matrix, current matrix cell value is decreased until its ratio reaches close enough to that of the pattern matrix. During this process it is ensured that the total values of the corresponding rows and columns remain within the defined range. This is achieved by selective modification of other cells in the corresponding row or column while considering their ratios. If for instance, the total row value needs to be reduced, the values of cells whose ratios are larger than that of pattern matrix are reduced as required.

Although the result of this iterative method may not necessarily be the optimum, it provided the best matrix compared to the results obtained in previous section by Q-Paramics Estimator. Therefore, the matrix obtained in this section was used as a reliable base matrix for next stages of the modelling. It is worth mentioning that estimation of the best possible matrix, which is by itself a tedious task, was not a focus of this study. Table 3-3 shows the final O-D matrix obtained in this section.

Table 3-3 Estimated origin-destination matrix

Zones	Karrinyup Road (West)	Cedric Street (East)	Hutton Street (West)	Hutton Street (East)	Lake Monger Drive	Powis Street (West)	Toward GFF	Hepburn Avenue (West)	Powis Street (East)	Warwick Road (West)	Erindale Road (East)	Cedric Street (West)	Vincent Street	Erindale road (West)	Reid Highway (East)	North of Hepburn Avenue	Hepburn Avenue (East)	Reid Highway (West)	Warwick Road (East)	Karrinyup Road (East)	SUM
Karrinyup Road (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Cedric Street (East)	0	0	0	196	0	0	640	0	0	0	0	0	100	0	0	0	0	0	0	0	936
Hutton Street (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hutton Street (East)	0	0	0	0	0	0	720	0	0	0	0	0	79	0	0	0	0	0	0	0	799
Lake Monger Drive	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Powis Street (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
From GFF	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hepburn Avenue (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Powis Street (East)	0	0	0	0	0	0	899	0	0	0	0	0	59	0	0	0	0	0	0	0	958
Warwick Road (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Erindale Road (East)	0	310	0	94	0	0	456	0	0	0	0	0	27	0	0	0	0	0	0	48	935
Cedric Street (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Vincent Street	0	0	0	0	0	0	484	0	0	0	0	0	0	0	0	0	0	0	0	0	484
Erindale road (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Reid Highway (East)	0	50	0	87	0	0	249	0	0	0	0	0	25	0	0	0	0	0	0	30	441
North of Hepburn Avenue	0	352	0	186	0	0	1989	0	0	0	0	0	235	0	681	0	90	0	126	120	3779
Hepburn Avenue (East)	0	62	0	127	0	0	793	0	0	0	0	0	73	0	313	0	0	0	38	36	1442
Reid Highway (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Warwick Road (East)	0	74	0	39	0	0	635	0	0	0	0	0	53	0	287	0	0	0	0	35	1123
Karrinyup Road (East)	0	14	0	112	0	0	729	0	0	0	0	0	65	0	0	0	0	0	0	0	920
sum	0	862	0	841	0	0	7594	0	0	0	0	0	716	0	1281	0	90	0	164	269	11817

3.4.8 Calibration and Validation of the Base Model

All the parameters discussed in previous sections were varied in such a way to make the model behaviour as close as possible to a reference. This reference is essentially the actual data of the freeway traffic. During this process, two main criteria were used to evaluate the resemblance between the model and the observed freeway traffic. These two include the observed traffic counts data and the recorded travel time collected using the floating-car technique. Calibration and validation of the model using these two criteria are discussed in the following sections.

3.4.8.1 Calibrating against the Observed Traffic Counts

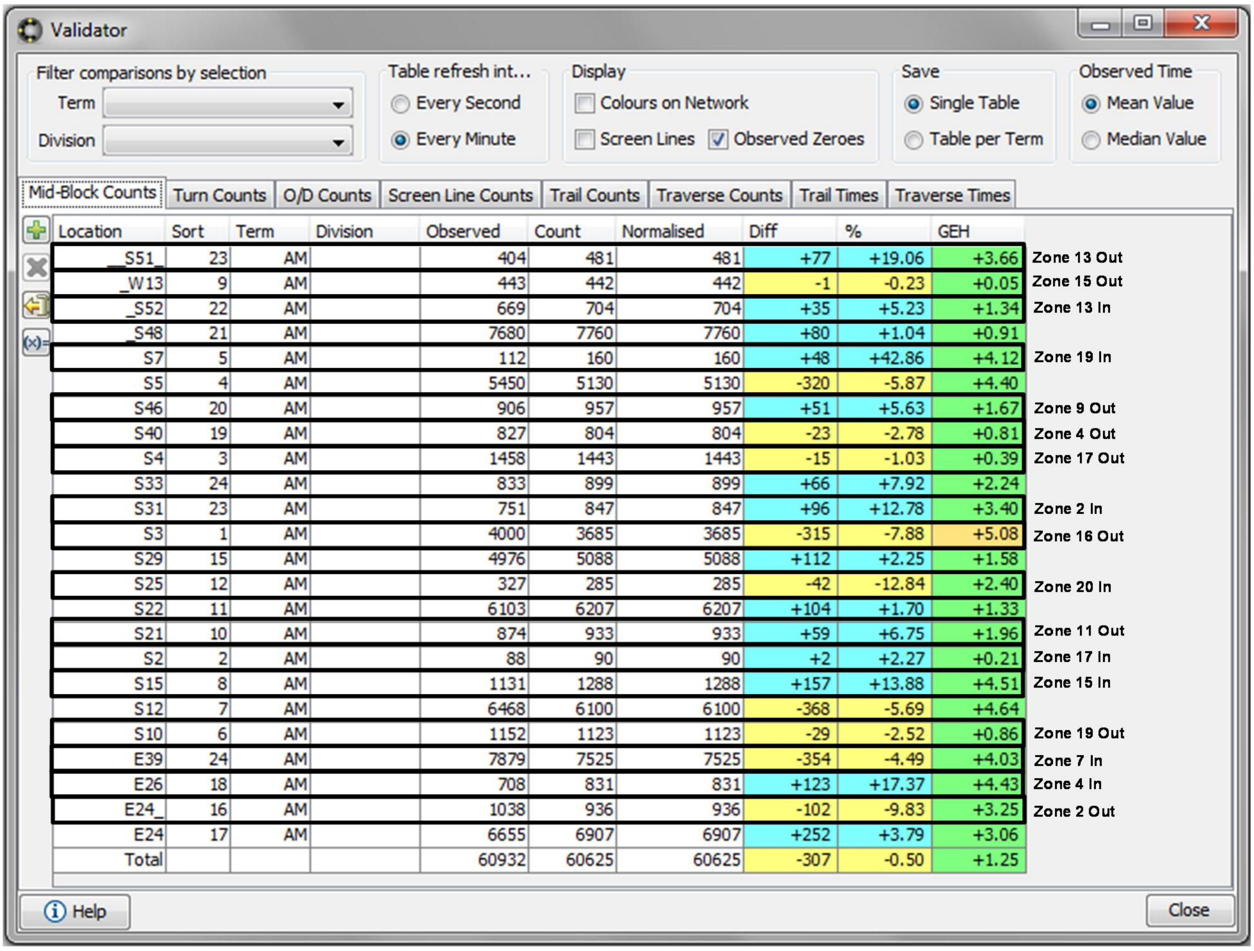
A common approach for calibration of simulation models is to compare different traffic count sets including cordon, links and turn traffic counts obtained from simulation to those observed in the real traffic of the network. For comparison of different sets of data in this study a popular tool, GEH Statistics, is used at the indicator. Named after its creator (Geoffrey E. Havers) GEH is used in traffic modelling to evaluate how closely observed and modelled flows fit. GEH is obtained from the following formula:

$$GEH = \sqrt{\frac{2(M-C)^2}{M+C}} \qquad \text{Equation 3-1}$$

where M is the traffic flow obtained from the traffic model and C is the observed traffic flow. The smaller the GEH value, the model fits betters to the reality. In the calibration procedure, it was assumed that 85% of flows in the model should have GEH of 5 or less.

Based on availability and reliability of traffic data, a total of 24 cordons and links flow were considered for validating the model. Figure 3-11 shows the report of the final values of traffic counts from Commuter. This data is obtained at the end of the AM simulation period for the 24 cordons and links. The figure shows both observed and model count data. Furthermore, cordon counts are shown as the link counts corresponding to the link of off-ramp or on-ramp of the cordon. For clarity, cordons are specified by thick black lines. The zones corresponding to each cordon are quoted in their corresponding row.

As the report shows, calculated GEH values are reasonably low for all links and cordons except for the S3 link which shows a GEH slightly larger than five. This corresponds to the link of the freeway mainline from Hepburn off-ramp interchange to Hepburn on-ramp interchange. To study link flows in more detail, statistics of all link flows from 24 counts were evaluated. The calculations lead to an average GEH



Location	Sort	Term	Division	Observed	Count	Normalised	Diff	%	GEH	
__S51_	23	AM		404	481	481	+77	+19.06	+3.66	Zone 13 Out
_W13	9	AM		443	442	442	-1	-0.23	+0.05	Zone 15 Out
_S52	22	AM		669	704	704	+35	+5.23	+1.34	Zone 13 In
_S48	21	AM		7680	7760	7760	+80	+1.04	+0.91	
S7	5	AM		112	160	160	+48	+42.86	+4.12	Zone 19 In
S5	4	AM		5450	5130	5130	-320	-5.87	+4.40	
S46	20	AM		906	957	957	+51	+5.63	+1.67	Zone 9 Out
S40	19	AM		827	804	804	-23	-2.78	+0.81	Zone 4 Out
S4	3	AM		1458	1443	1443	-15	-1.03	+0.39	Zone 17 Out
S33	24	AM		833	899	899	+66	+7.92	+2.24	
S31	23	AM		751	847	847	+96	+12.78	+3.40	Zone 2 In
S3	1	AM		4000	3685	3685	-315	-7.88	+5.08	Zone 16 Out
S29	15	AM		4976	5088	5088	+112	+2.25	+1.58	
S25	12	AM		327	285	285	-42	-12.84	+2.40	Zone 20 In
S22	11	AM		6103	6207	6207	+104	+1.70	+1.33	
S21	10	AM		874	933	933	+59	+6.75	+1.96	Zone 11 Out
S2	2	AM		88	90	90	+2	+2.27	+0.21	Zone 17 In
S15	8	AM		1131	1288	1288	+157	+13.88	+4.51	Zone 15 In
S12	7	AM		6468	6100	6100	-368	-5.69	+4.64	
S10	6	AM		1152	1123	1123	-29	-2.52	+0.86	Zone 19 Out
E39	24	AM		7879	7525	7525	-354	-4.49	+4.03	Zone 7 In
E26	18	AM		708	831	831	+123	+17.37	+4.43	Zone 4 In
E24_	16	AM		1038	936	936	-102	-9.83	+3.25	Zone 2 Out
E24	17	AM		6655	6907	6907	+252	+3.79	+3.06	
Total				60932	60625	60625	-307	-0.50	+1.25	

Figure 3-11 GEH of links and cordon counts

value of 1.25.

Cumulative GEH statistics is shown in Table 3-4. As can be seen from the table, 96% of the sites had a GEH of 5 or less. A graphical presentation of the GEH data is also provided in a histogram in Figure 3-12.

Table 3-4 Cumulative GEH summary

GEH	Frequency	Percentage	Cumulative %
1	6	0.25	0.25
2	5	0.21	0.46
3	2	0.08	0.54
4	4	0.17	0.71
5	6	0.25	0.96
6	1	0.04	1
More	0	0	1
SUM	24		

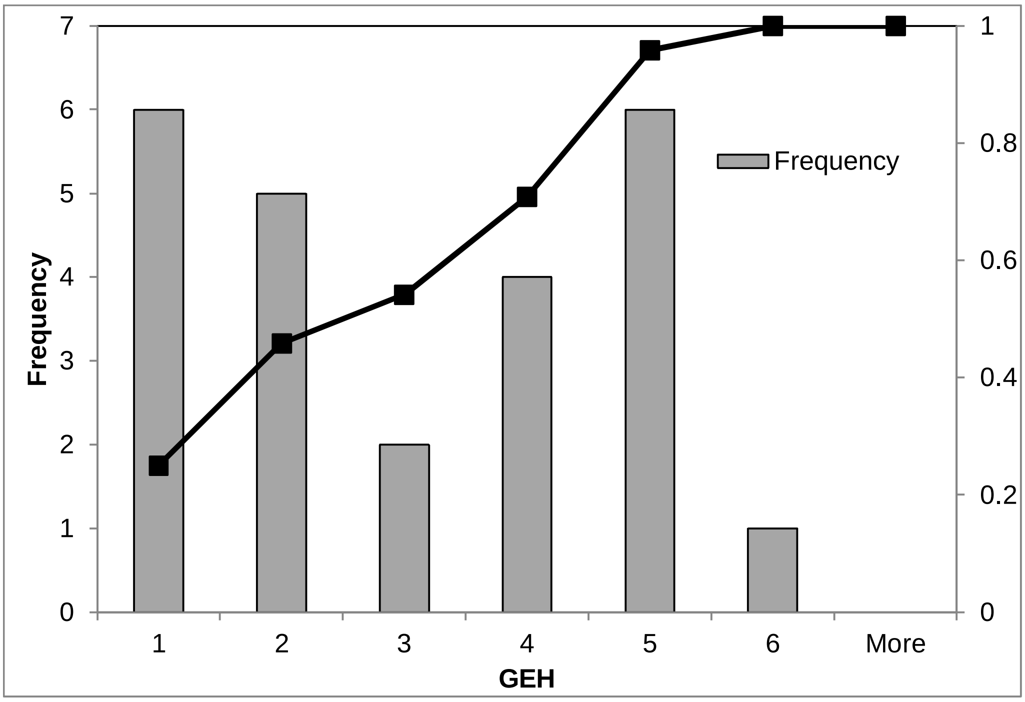


Figure 3-12 Cumulative GEH summary

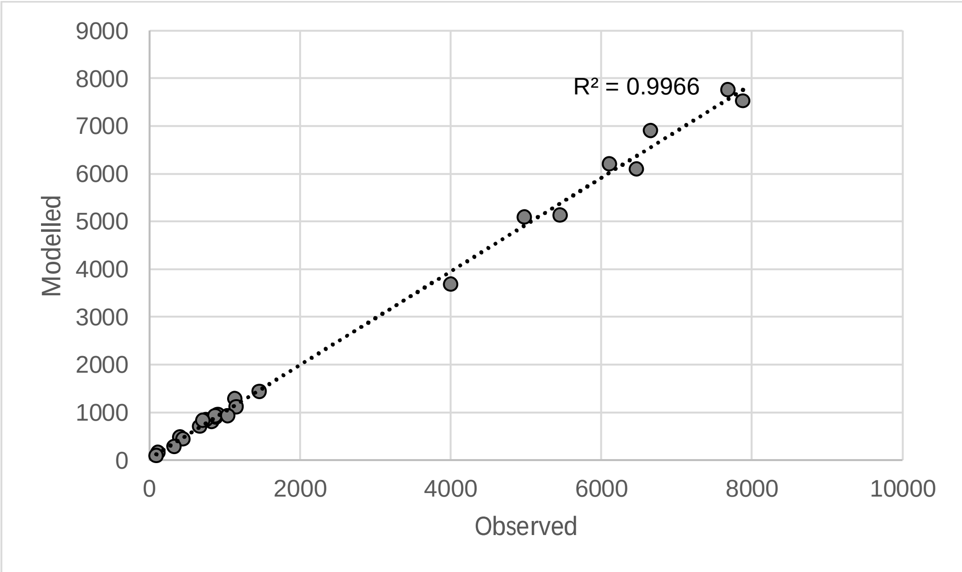


Figure 3-13 Modelled versus observed link flows

A plot of modelled traffic volumes versus observed traffic volumes is presented in Figure 3-13. The figure shows that the model performs well in terms of reproducing the observed traffic volumes of the freeway (note the almost 45^o line slope and R^2 = 0.9966). At this stage calibration of the model is accepted accurate enough.

3.4.8.2 Validating against Floating-car Technique

Floating-car technique is a method used to measure an average travel time along a route. In this method, the vehicle is required to move or "float" with a speed equal to the average of stream of vehicles. It is therefore important for the driver of the vehicle survey to realise the mean speed of stream of vehicles on the road and maintain the speed at an appropriate level. The average travel time of the link measured by this method is dependent on the level of the traffic demand at the time of the survey (Luk, LLoyd, and Yoo 2009). It is obvious that the measured average speed in a route varies even for the same period of the survey times. Therefore, in order to reduce the uncertainty of the traffic data, floating-car survey is normally carried out on mid-week days including Tuesdays, Wednesdays, and Thursdays (Luk, LLoyd, and Yoo 2009).

In this study a GPS Logger was used to the record real travel time and speed over different non-school holiday mid-week days. The data was recorded over the

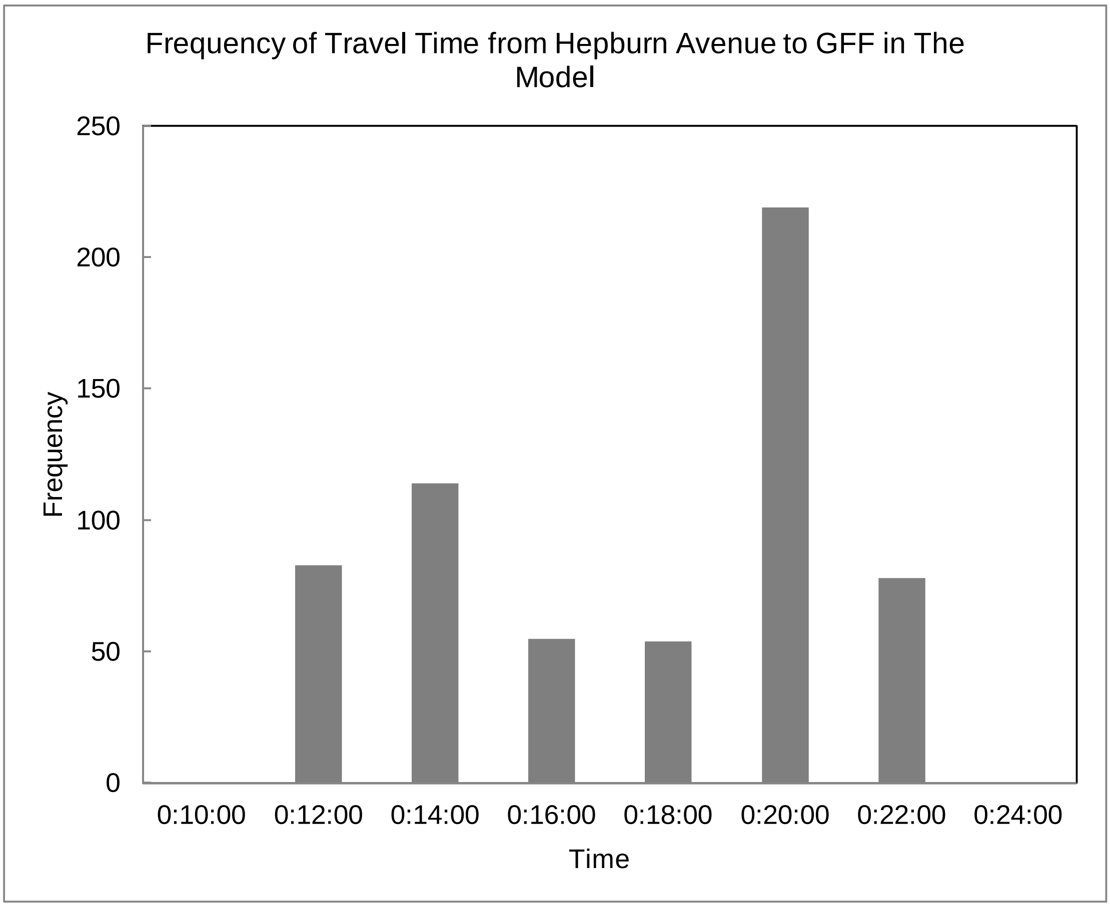


Figure 3-14 Frequency of travel time from Hepburn Avenue to GFF in the model

6:15 am to 7:15 am period, surveying from Hepburn Avenue on-ramp interchange to the end of the model boundary on Mitchell Freeway. In order to check whether the peak time starts before this period or not, some data was recorded even before this period. Nevertheless, measurements showed that the peak hour does not occur before 6:15 am.

Recorded average travel times were then compared to the values obtained from the model. The comparison showed the average travel times computed from simulation model to be very close to the observed travel times. The average real travel time was 16 minutes and 33 seconds and the same value in the simulation model was 16 minutes and 30 seconds. Furthermore, the range of observed travel times was from 12 minutes and 3 seconds to 19 minutes and 33 seconds. Figure 3-14 shows the frequency of simulated travel times of vehicles presented in discrete time intervals. These travel times are related to the vehicles that depart after 6:15 am and arrive before 7:15 am in the simulation. Comparing the observed travel times and the simulation travel times it is concluded that the travel time range of the model reasonably matches that of the observed data.

Calibrating the model against traffic volume data as well as validating against surveyed travel time of the longest route, form Hepburn off-ramp interchange to GFF, at this stage the model was considered a valid model for simulating the mentioned Mitchell Freeway section from 6:15 am to 7:15 am. It can now be used for studying traffic behaviour of the freeway. The next section discusses installation of ramp meters in the model.

3.5 Ramp Metering Modelling

As mentioned before, a variety of ramp metering algorithms have been developed over the last decade. This study was only limited to the ramp metering algorithms available in the simulation package. There are two predefined local ramp metering algorithms in Commuter: ALINEA and ALINEA-Q. The former algorithm was initially coded in the base model and it was then examined to find out if it was required to be upgraded to ALINEA-Q. The simulation results revealed however that during the simulation time of one hour, storage capacities of ramps were never exceeded by the traffic inflow. Similar results were obtained by running the model with even higher demands. Thus, the original ALINEA algorithm was considered to be suitable for modelling purposes and was used for freeway ramp metering study.

One issue that may arise after installation of ramp metering is that the freeway trips could be diverted to the nearby arterial roads (Wu 2001). This is mainly caused by the added waiting time of the ramp metering before the vehicles can enter the freeway. This in turn, makes alternative routes more interesting for drivers causing traffic diversion. It is recalled that ALINEA algorithm is a method which locally controls the entering traffic of the freeway ramps. In the current model, there is not necessarily a bottleneck after each freeway on-ramp. Therefore, in the first look it may appear that some ramps are not required to be metered. However, there is the risk of divergence of freeway trips from the metered on-ramps to those adjacent ramps which are not metered. As such, it was decided to meter all of the model on-ramps to be able to manage the freeway traffic globally. The next two sections contain the design and calibration procedures of the ramp metering in the model.

3.5.1 Designing Ramp Metering

Ramp metering based on good control algorithms may not necessarily lead to better traffic conditions if the installations are based on inadequate designs. Examples are available in which ramp signals installed on freeway ramps did not deliver expected results due to inappropriate design, insufficient detailed analysis, and lack of understanding of ramp metering principles. Such weaknesses not only may not result to any benefit from ramp metering, but also in the worst case they may deteriorate the freeway traffic (Burley and Gaffney 2010). This fact signifies the key role of proper design of ramp metering. This section contains the steps followed for designing the ramp metering.

3.5.1.1 Coding the Mainline Detectors

In the past, the entry ramp arrival flow rate normally was calculated based on vehicles entering the gaps in the left lane of the freeway mainline. However, it is the total traffic flow and density across the whole freeway that determines a technically effective control and results in optimum mainline and on-ramp flow. Therefore, traffic flow across all lanes of the freeway mainline is measured by the latter approach (Burley and Gaffney 2010). Burley and Gaffney (2010) referred to Euramp Handbook of ramp metering (Papageorgiou and Papamichail 2007) to demonstrate that on different days even under similar environmental circumstances, flow capacities may vary in merging areas. Thus, traffic flow breakdown takes place at different flow capacities. Furthermore, Keen, Schofield, and Hay (1986) showed the likelihood of capacities variations is higher in hostile weather conditions. On the other hand, even under dissimilar weather conditions, occupancy at which flow breakdown occurs is almost steady. Therefore, instead of using flow rate and speed, the occupancy measured by detectors was used in the current work to optimise the throughput.

In order to measure occupancy on each on-ramp downstream, aligned detectors were set up in each lane of the model separately. Each detector measures the occupancy of its corresponding lane at the end of the defined interval time for that link. The average occupancy across all lanes of the freeway mainline would be the input to the ramp metering algorithm. However, the detectors should be installed downstream of each ramp where it is possible to quickly and easily identify mainline congestion (Hasan 1999). Therefore, the detectors distance from the ramp entrance is a key parameter for model calibration. The calibrated distance for the current model is presented in Section 3.5.3.

3.5.1.2 Identification of Bottlenecks

The main reason for using ramp metering in freeways is to prevent traffic flow breakdown. According to (Burley and Gaffney 2010), traffic breakdown is referred to the condition where the free-flow traffic speed is considerably reduced with a continued loss of throughput. There are several recognised reasons for flow breakdown. Amongst all, bottlenecks are some of the most common causes of traffic flow breakdown. A bottleneck is a fixed location with a traffic flow capacity lower than its upstream capacity. There are several types of bottlenecks which affect traffic flow capacity and can potentially cause flow breakdown. These are as follows:

- Traffic merging from an entry ramp
- Traffic merging caused in a lane drop, for instance narrowing from four to three lanes
- Significant lane changing manoeuvres that need to be performed over a short distance.
- Traffic queues that form at an off-ramp and extend back into the freeway. These may block freeway’s left lane or slow down the traffic prior to exiting freeway.
- Locations in the mainline where geometric features force vehicles to reduce speed. These may include steep upgrades, tight radius curves, restricted widths, or limited sight distance.

Those locations along a freeway section where flow breakdown first occurs are defined as “critical bottlenecks”. Thus a critical bottleneck is the location that reaches its traffic flow capacity first (Burley and Gaffney 2010). A potential bottleneck on the other hand is the one which is not necessarily active. When flow breakdown occurs due to the flow exceeding the capacity, a potential bottleneck may turn into an active bottleneck (Burley and Gaffney 2010).

Traffic flow breakdown may take place for reasons other than bottlenecks. In such a case, flow breakdown may happen at any location along a freeway. One of the factors could be the difference in speed of vehicles. For instance, a truck moving with lower speed than average vehicles' speed may cause flow breakdown. An accident or even an object in a freeway is another possible reason. Some driver behaviours also can slow down the traffic flow. Examples are "rubber necking", to look at an incident, presence of police or enforcement activities, and sudden actions such as rapid braking due to a driver's distraction. Other causes may include road works, lower speed limits on special occasions, and suddenly releasing a large number of vehicles with very high density over a short time (Burley and Gaffney 2010).

In the first step of design of ramp metering in the model, it is essential to identify the fixed bottlenecks in the model. Precise identification of bottlenecks in a freeway is by itself a tough and time consuming procedure and is out of the scope of this study. Nonetheless, the bottlenecks in the model where identified based on the main bottleneck reasons mentioned above. All merging lanes in the freeway section under study were identified and considered as bottlenecks. Furthermore, it was noticed that weaving takes place in the freeway section from Warwick Road on-ramp to Reid Highway off-ramp interchange. Table 3-5 shows identified bottlenecks and their causes.

Table 3-5 Bottlenecks identified in the model

No.	Location	Reason
1	Between Warwick on-ramp and Reid Hwy off-ramp	Weaving
2	Reid Hwy on-ramp	Merging
3	After Karrinyup and Cedric off ramp	Merging
4	Karrinyup on-ramp	Merging
5	Between Hutton on-ramp and off-ramp	Merging
6	Hutton on-ramp	Merging
7	Vincent on-ramp	Merging

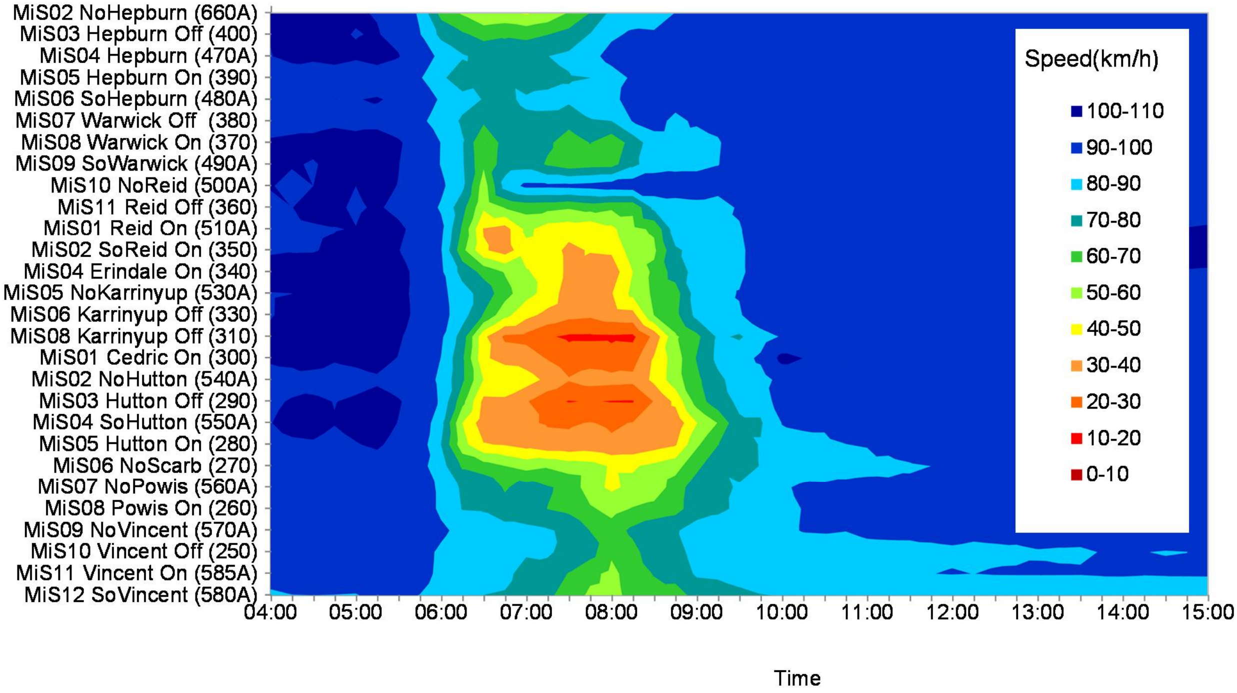


Figure 3-15 A contour plot showing average speed recorded in different VDS stations during weekdays

Figure 3-15 shows a contour plot of average speeds recorded at different VDS stations over weekdays. This figure is based on information which is not complete throughout the freeway as they were obtained in limited VDS locations. Nevertheless, this figure shows the speed reduction as a result of at least some of the bottlenecks. For example, the bottlenecks between Warwick Road on-ramp and Reid Highway off-ramp, after Karrinyup Road on-ramp and Cedric Street off-ramp, and between Hutton Street on-ramp and off-ramp are almost apparent by VDS data from stations 500A, 310, and 550A, respectively.

3.5.1.3 Bottleneck Capacity

The next step after determining the bottlenecks is to estimate their capacities. According to HCM (2000), the capacity of a freeway is defined as the maximum flow rate that can reasonably be expected to traverse a facility under prevailing roadway, traffic, and control conditions. Brilon, Geistefeldt, and Regle (2005) state that at a rate of around 1700 vehicles per hour per lane, there is about 5% chance of flow breakdown happening (see Figure 3-16). At a rate of 2000 vehicles per hour per lane the probability is in the order of 50 to 60%. Also, it was shown that the capacity is not consistent along the freeway (Brilon, Geistefeldt, and Regler 2005).

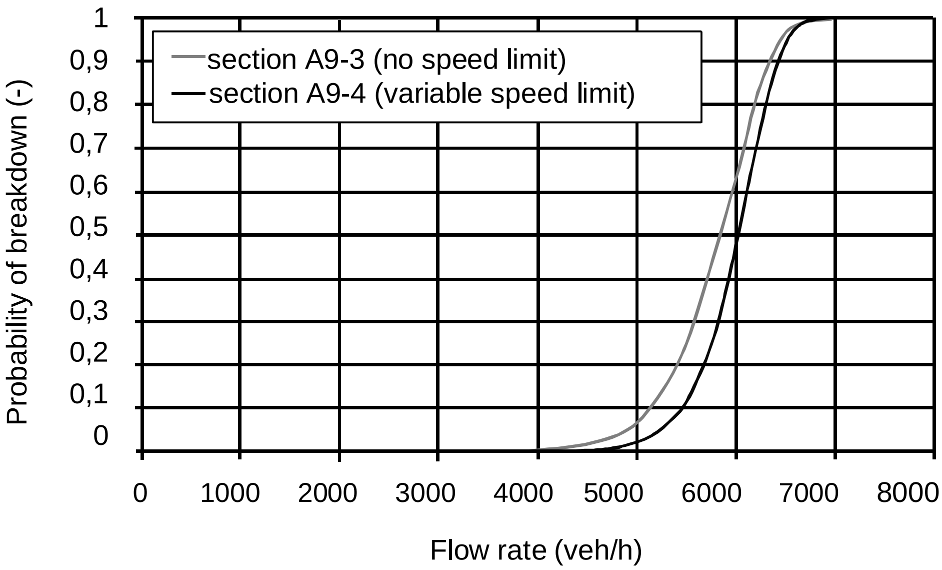


Figure 3-16 Capacity distribution for a 3-lane freeway

Burley and Gaffney (2010) presented different levels of service (LOS) for different values of speed and flow (with the unit of vehicle per hour per lane).

Table 3-6 shows the table they provided for this purpose. This table shows that for a speed of 100 km/h, which is the speed limit of the study area in the current study, the maximum flow leading to the defined high LOS (a density of smaller than 16 vehicles per kilometre) is 1500 vehicles per hour per lane. Flow breakdown generally occurs at densities of 22 pc/km (passengers car per kilometre) to 28 pc/km (Burley and Gaffney 2010).

Table 3-6 Traffic flow relationship

Flow - Headway - Speed - Density - Spacing

Flow (pc/h/lane) ->	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400	2500
Headway (s) ->	3.60	3.27	3.00	2.77	2.57	2.40	2.25	2.12	2.00	1.89	1.80	1.71	1.64	1.57	1.50	1.44
Speed (km/h) v	Density (pc/km)															
30	33.3	36.7	40.0	43.3	46.7	50.0	53.3	56.6	60.0	63.3	66.6	70.0	73.3	76.6	80.0	83.3
40	25.0	27.5	30.0	32.5	35.0	37.5	40.0	42.5	45.0	47.5	50.0	52.5	55.0	57.5	60.0	62.5
50	20.0	22.0	24.0	26.0	28.0	30.0	32.0	34.0	36.0	38.0	40.0	42.0	44.0	46.0	48.0	50.0
60	16.7	18.3	20.0	21.7	23.4	25.0	26.7	28.4	30.0	31.7	33.4	35.0	36.7	38.4	40.1	41.7
70	14.3	15.7	17.1	18.6	20.0	21.4	22.9	24.3	25.7	27.1	28.6	30.0	31.4	32.9	34.3	35.7
80	12.5	13.8	15.0	16.3	17.6	18.8	20.0	21.3	22.5	23.8	25.0	26.3	27.5	28.8	30.0	31.3
90	11.1	12.2	13.3	14.4	15.6	16.7	17.8	18.9	20.0	21.1	22.2	23.3	24.4	25.6	26.7	27.8
100	10.0	11.0	12.0	13.0	14.0	15.0	16.0	17.0	18.0	19.0	20.0	21.0	22.0	23.0	24.0	25.0

Legend: Density (pc/km)

	Density (pc/km)	LOS
white	< 16	(LOS A, B, C)
light grey	16 -22	(LOS D)
medium grey	22 - 28	(LOS E)
dark grey	> 28	(LOS F)

Note: High Flows (greater than 2200 veh/h) would only be achieved within short flow periods

Source: (Brilon, Geistefeldt, and Regler 2005)

Capacity of bottlenecks in this study was chosen by taking into account both of the two abovementioned criteria. Table 3-7 summarises the number of lanes in each bottleneck and their assumed capacities.

Table 3-7 Bottlenecks' capacities

Bottlenecks Location	Reason	No. of lane	Capacity	
			1500 v/h/l	1700 v/h/l
Warwick Rd on ramp - Reid Hwy off ramp	Weaving	4	6000	6800
Reid Hwy on ramp	Merging	3	4500	5100
After Karrinyup Rd and Cedric St off ramp	Merging	3	4500	5100
Karrinyup Rd on ramp	Merging	3	4500	5100
Between Hutton St on ramp and off ramp	Merging	3	4500	5100
Hutton St on ramp	Merging	3	4500	5100
Vincent St on ramp	Merging	4	6000	6800

3.5.1.4 On-ramp Storage Capacity

As ALINEA is an isolated (local) ramp metering algorithm, each bottleneck in the model is controlled by metering its one or two immediate upstream ramp meters. The metering rate based on its corresponding bottleneck(s) control the throughput rate of the freeway. As this rate is not the same as the demand rate of the on-ramp, it is necessary to determine the values of storage capacity for each on-ramp. The vehicles waiting behind the stop-line use this storage before entering the freeway. To design the ramp-storage storage capacity, the number of lanes and length of total storage are required to be defined. For each on-ramp it is required to determine the value of storage capacity. For this purpose the number of lanes and length of total storage should be defined. To do so, the guidelines provided by Burley and Gaffney (2010) were followed for designing the ramps based on one vehicle per lane per cycle. Table 3-8 shows the requirements of lanes at the stop line as well as the ramp storage.

To use this guideline table, two inputs are required: the indicative ramp layout and the ramp design flow. Starting with the former, there are three possible types of the indicative ramp layout: single lane merge, added lane entering the freeway or two lane merge, and added lane entering the freeway plus a merging lane.

Table 3-8 Lanes at the stop line and Ramp Storage Requirement

Indicative Layout (5)	Ramp Design Flow (5)	Total Storage Required (Lane metres)	Ramp Storage (3) and Cycle Time (7) relative to the number of lanes at the Stop Line							
			1 Lane		2 Lanes		3 Lanes		4 Lanes	
			Average Storage per Lane (m)	Average Cycle Time (s)	Average Storage per Lane (m)	Average Cycle Time (s)	Average Storage per Lane (m)	Average Cycle Time (s)	Average Storage per Lane (m)	Average Cycle Time (s)
Single lane merge (6)	200	113	113	18.0						
	300	170	170	12.0						
	400	227	227	9.0						
	500	283	283	7.2	142	14.4				
	600	340	340	6.0	170	12.0				
	700	397			198	10.3				
	800	453			227	9.0				
	900	510			255	8.0	170	12.0		
	1000	567			283	7.2	189	10.8		
	1100	623			312	6.5	208	9.8		
	1200	680			340	6.0	227	9.0		
added lane entering the freeway or Two lane merge	1300	737					246	8.3	184	11.1
	1400	793					164	7.7	198	10.3
	1500	850					283	7.2	213	9.6
	1600	907					302	6.8	227	9.0
	1700	963					321	6.4	241	8.5
	1800	1020					340	6.0	255	8.0
Added lane entering the freeway plus a merging lane	1900	1077							269	7.6
	2000	1133							283	7.2
	2100	1190							298	6.9
	2200	1247							312	6.5
	2300	1303							326	6.3
	2400	1360							340	6.0
	2500	1417							354	5.8
	2600	1473							368	5.5
	2700	1530							383	5.3
	2800	1587							397	5.1
	2900	4643							411	5.0
	3000	1700							425	4.8

Note:

1. Max wait / vehicle (min): 4
2. Storage per vehicle (m): 8.5
3. Average storage per lane assumes lanes of equal length. Not application with auxiliary lanes at the stop line.
4. No. vehs / green / lane: 1
5. Ramp layout and ramp design flow are subject to the bottleneck capacity on mainline.
6. A single lane merge layout may be satisfactory for higher flows, e.g., a ramp flow of 1600 veh/h with mainline of 2400 veh.h on a two lane freeway mainline.
7. Cycle times lower than values in black are generally not appropriate as an average cycle over the design hour. Cycle time in orange may be appropriate at ramps with spare mainline merge capacity.
8. Designs with average cycle times outside the limits in this table shall be approved by the Executive Director – Network and Asset Planning.

Source: (Burley and Gaffney 2010)

Based on the current conditions of individual on-ramps, their indicative layout was decided between the three available options. There was however no data available on the second required input, the ramp design flows. Therefore, the highest observed flow was used as an indicator of the design flow for each ramp. To compare this to the values presented in the guideline, the highest observed flows of the ramps were rounded to the closest ramp design flow in Table 3-8. Finally, the number of lanes and their length were obtained from the guideline table. Table 3-9 shows for each on-ramp the highest observed flow, assumed ramp design flow and indicative layout as well as number of lanes and their length. For majority of the on-ramps there were two possible designs available. The design closest to the current freeway conditions was used for each on-ramp and was applied in the model. The selected design of each on-ramp is highlighted by its grey cells in the table.

Table 3-9 Ramps design

Bottlenecks Location	Upstream on ramp(s) and its max flow		Ramp metering			
	Zone	Highest Observed flow (v/h)	Considered Ramp design flow	Inductive layout	No. of lanes, Storage (m)	
No bottleneck	Hepburn (Zone 17)	2189	2200	Added lane entering	4lanes- 312m	
Warwick on ramp - Reid off ramp	Warwick (Zone 19)	1372	1400	Added lane entering	3lanes- 164m	4lanes-184m
Reid Hwy on ramp	Reid (Zone 15)	509	600	Single lane merge	1lane- 340m	2lanes- 170m
After Karrinyup and Cedric off ramp	Erindale (Zone 11)	1417	1500	Added lane entering	3lanes- 283m	4lanes-213m
Karrinyup on ramp	Karrinyup (Zone 20)	1442	1500	Added lane entering	3lanes- 283m	4lanes-213m
Hutton on ramp - off ramp	Cedric (Zone 2)	1314	1400	Added lane entering	3lanes- 164m	4lanes-184m
Hutton on ramp	Hutton (Zone 4)	952	1000	Single lane merge	2lanes- 283m	3lanes- 189m
No bottleneck	Powis (Zone 9)	1299	1400	Added lane entering	3lanes- 164m	4lanes-184m
Vincent on ramp	Vincent (Zone 13)	918	1000	Single lane merge	2lanes- 283m	3lanes- 189m

3.5.2 Demand Scaling

One of the key issues with estimating traffic under congested conditions is the inability to differentiate between the demand and equilibrium and the drop in observed flows that occurs as a result of system breakdown. Scariza (2003) showed that the ramp metering can be beneficial only at the high levels of demand where congestion occurs. On the other hand, for low demands ramp metering causes a reduction in the traffic flow efficiency because of unnecessary vehicle stops before entering the freeway. Recognising this issue, the estimated matrices were incrementally globally scaled up in order to better reflect likely traffic volumes under congested conditions.

To increase the demand, it was scaled by 105, 110, 115, 120 and 125 precent. For each case the model was run and the results were analysed. The analysis revealed that demands of 125% or larger are not suitable for the current model as they exceed the model capacity. Finally, the scaled demand of 123% was considered as appropriate. This incremental scaling lead to a 23% increase in estimated demands meaning traffic loaded onto the network is 23% higher than the estimated demand matrices. Given the focus of this study is on the improvements likely from implementing ramp metering, the impact of this simplified approach was considered marginal since the same traffic demands are loaded on to both the network with and without the metering which suggests that for comparison purposes the impact could be ignored. On the other hand, the scale of the warm up period demand of the model was not varied.

3.5.3 Ramp Metering Calibration

The ramp metering designed in previous steps need to be calibrated. For this purpose there are a number of ramp metering parameters in Commuter that need to be varied. The following parameters have been calibrated for each ramp metering:

- First Cycle Time: This is the initial length of signal cycle in seconds. The cycle length is the sum of the red and green times of the signal. The default value of this parameter is 20 seconds.
- Minimum Cycle Times: The minimum length of the cycles in seconds. The maximum flow that can pass the meter is controlled by this value. By default the cycle is set to a minimum of five seconds.

For instance, if only one vehicle per cycle is chosen, then the maximum flow through the meter is 720 vehicles per hour.

- Maximum Cycle Time: This is the maximum length of the cycle in seconds. The minimum flow that can pass the meter is controlled by this value.
- Interval: This is the length of time between the calls to the logic module. If a short interval length is chosen the meter will become more responsive to variations in the main flow. But at the same time the output value will become less stable and more prone to oscillations between large and small values. The call interval is also used as the period over which the ALINEA logic computes the percentage occupancy.
- Target Occupancy and K_R, ALINEA algorithm parameters (Azalient 2011).

All of these parameters were initially changed to see their effect on the result. Based on observations, some of them were changed back to their default values. Table 3-10 shows the final values used after parameter calibration.

Table 3-10 Parameter calibration

Meter Location	Vehicle per Cycle	Per Green	1st Cycle	Min Cycle	Max Cycle	Interval	Target Occupancy	K_R
Hepburn Avenue On-ramp	4	1	5	2	20	30	0.13	65
Warwick Road On-ramp	3	1	5	2	20	30	0.15	70
Reid Highway On-ramp	2[1]	1	5	5	20	20	0.16	70
Erindale Road On-ramp	4	1	5	2	20	30	0.10	65
Cedric Street Off-ramp	4[2]	1	5	5	20	30	0.16	70
Karrinyup Road On-ramp	4	1	8	5	20	30	0.16	70
Cedric Street On-ramp	4	1	5	2	20	30	0.16	70
Hutton Street On-ramp	2* [3]	1	5	2	20	30	0.15	70
Powis Street On-ramp	4	1	8	2	20	30	0.20	70
Vincent Street On-ramp	3	1	8	2	20	30	0.20	70

[1] In Commuter, the default number of vehicles per lane per green of ramp metering signals on links with 2 lanes is 1 or 2. In this ramp, this number was set to 2 (1 vehicle per lane per green)

[2] Cedric Street off-ramp has the same design as Karrinyup Road on-ramps because they have similar demands.

[3] Here "*" means that vehicles per green is set to its default value in Commuter. This means number "2" is not fixed for this link (with two lanes) and 2-4 vehicles per two lanes per cycle could enter the freeway.

Parallel to the calibration of the mentioned parameters, the detector locations are also calibrated. Table 3-11 shows the result of this calibration. Each detector is placed downstream of its corresponding on-ramp interchange. Most detectors are placed immediately after each determined merging area or lane drop.

Table 3-11 Detectors location

Meter Location	Approximate Distance from Ramp Entrance (m)	Location
Hepburn Avenue On-ramp	24	After adding on-ramp lane
Warwick Road On-ramp	197	In weaving area
Reid Highway On-ramp	120	After merging on-ramp lane
Erindale Road On-ramp	217	After adding on-ramp lane
Cedric Street Off-ramp	70	After merging Cedric off-ramp and Karrinyup on-ramp
Karrinyup Road On-ramp	280	After merging on-ramp lane
Cedric Street On-ramp	1000	After a lane drop
Hutton Street On-ramp	272	After merging on-ramp lane
Powis Street On-ramp	37	After adding on-ramp lane
Vincent Street On-ramp	100	After merging on-ramp lane

3.6 Summary

Different steps taken in traffic simulation of this study were discussed in this chapter. Ramp metering modelling of a section of Perth's Mitchell Freeway was the focus of this study.

Different data sets were obtained for the purpose of modelling. They include geometrical information, flow and speed data, SCATS data, and peak hour sub-area matrix of the study area. The model was then coded in Commuter and Q-Paramics to estimate O-D matrix. A decision was also made on the modelling period.

In the next step the demand matrix was estimated with both simulation packages and the most reliable matrix was selected to be used as the base demand of the model. The model was then validated against observed flow and travel time data. In the next step, ramp meters were designed for each ramp. In order to examine the capacity performance of freeway, the demand used in the base model was scaled by 123%. ALINEA was used as the preferred ramp metering algorithm. Thereafter, relevant ramp metering parameters were calibrated in such a way to maximise throughput and minimise travel time.

The results of ramp metering simulation and the comparison between freeway with and without ramp metering are discussed in the next chapter.

4 Results and Discussion

The built model described in the previous chapter was validated and therefore was considered reliable enough for evaluation of freeway ramp metering. This chapter includes the results of modelling Mitchell Freeway without ramp metering and with ALINEA ramp metering algorithms. Before explaining the results, limitations of the modelling which need to be considered when interpreting the results, are presented. Then, the results of the models are compared using different criteria. These include positive as well as negative effects of modelled ramp metering on the Mitchell Freeway traffic.

4.1 Limitations

Before discussing the results, it is important to take into account different limitations associated with the modelling in this study. Attempt is made in this section to point out the most important limitations.

The model was coded without considering vertical geometry data such as vertical curves. Therefore, possible changes in drivers' behaviour due to vertical geometry of the freeway are not taken into account by the model. This could include freeway mainline sections having geometric features such as steep upgrades or sight distance constraints which could cause vehicles to slow down.

There are traffic lights installed on arterial roads before each on-ramp of the Mitchell Freeway. For the sake of simplicity, these traffic lights are not considered in this study. This fact affects the pattern of entering vehicles to the freeway such as large uncontrolled platoons of traffic and their frequency.

Most of the micro simulation packages including Commuter may not be designed to model free flows. Also, Commuter is not a good candidate for proper modelling of freeway weaving. In order to overcome these limitations, the

Wiedemann car following algorithm seems to be more realistic for modelling vehicle movement along the freeway mainline. In addition, logical lane choice rules are considered in some links of the model in order to distribute evenly lane usage for each link.

It is worth mentioning that the Wiedemann algorithm is not suitable for vehicle movement on arterial roads and for releasing the demand in the model. Actually, it was observed that Wiedemann causes increasing unreleased number of vehicles especially for very high demand zones. In this situation the Fritzsche car following algorithm was selected as a suitable vehicle movement algorithm for arterial roads.

The abovementioned limitations associated with freeway modelling causes some parts of the model not to simulate the real freeway traffic behaviour accurately. Figure 4-1 compares the distance-time graph of a random vehicle in the model with an observed travel from Hepburn on-ramp to the end of the model area, recorded by GPS data logger. The figure also shows the lower and higher limits for the observed travel based on a GEH of 5. As the figure shows, although the observed travel time is almost the same as the modelled travel time (with about 45 seconds difference),

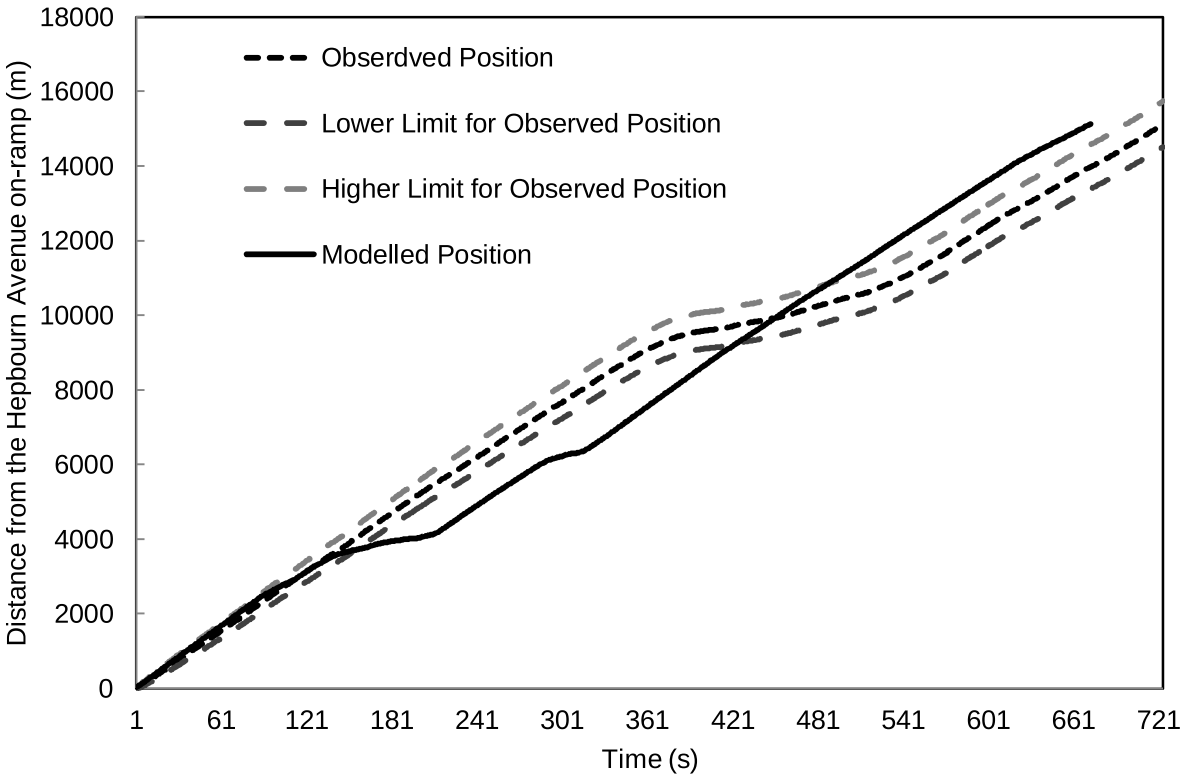


Figure 4-1 Comparison between the observed and the model

travelling speed is not always the same along the route. The modelled vehicle speed (corresponding to the slope of the lines) becomes less than the observed one at around travel distance of 4 km and then 6 km. On the other hand, the observed vehicle becomes slower than the modelled one after a travel distance of about 9.5 km. This trend continues for about one kilometre. In the remaining parts of the freeway, the model behaviour is close to that of reality based on this sample.

4.2 Capacity

Generally, it is expected that the capacity of the freeway is increased by installation of ramp metering. This is due to the fact that ramp metering optimises freeway traffic flow. In the current model also, it was observed that the capacity of the freeway was increased by ramp metering. The number of vehicles that completed their trips over the modelling period was recorded for freeway model with and without ramp metering. Table 4-1 shows these numbers for abovementioned scenarios. Comparison of the numbers indicates that more vehicles completed their trips in the models with ramp metered freeway. This shows the relative advantage of ramp metering in increasing the capacity of the freeway.

Table 4-1 Number of vehicles that completed their trips for different scenarios

Freeway Model	With ALINEA Ramp Metering	Without Ramp Metering
Number of Completed Trips Out of Total of 23279 (including Warm up Demand + 123% of Total Estimated Demand)	20657	20370

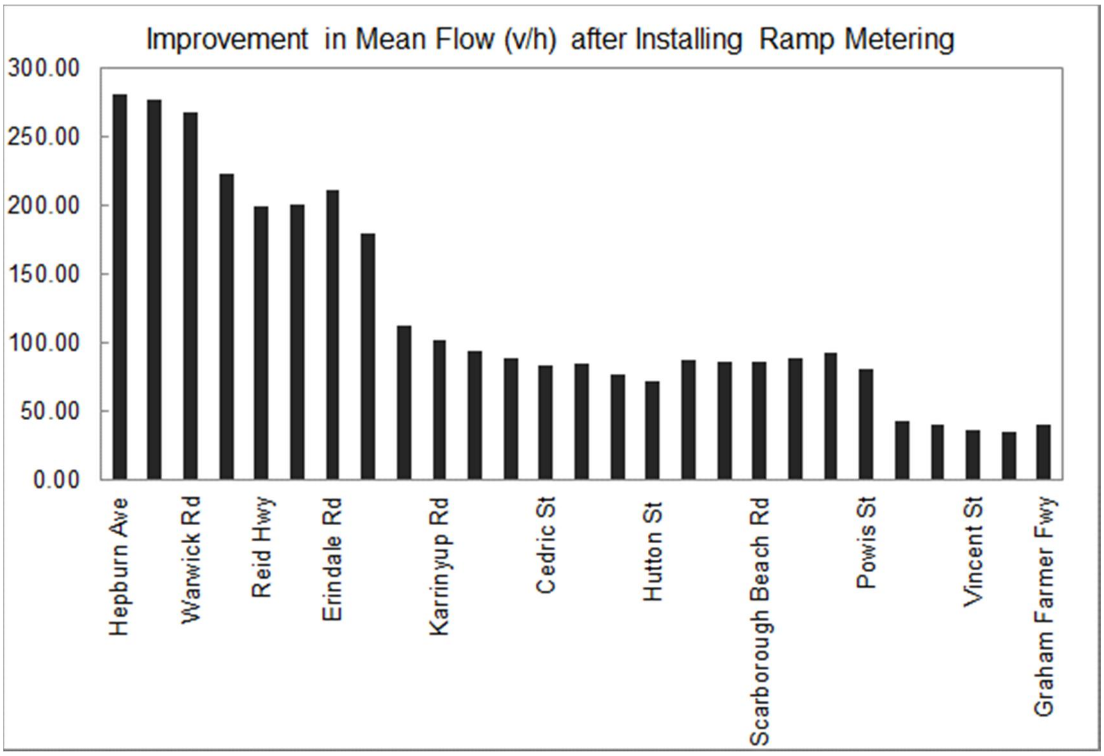


Figure 4-2 Improvement in mean flow (v/h) after installing ramp metering

In addition, more evidence to prove the rise in capacity of freeway after installing ramp metering is the increase in mean flow (vehicle per hour) in mainline links of the freeway. Figure 4-2 shows the flow improvement in each mainline link along the freeway from Hepburn Avenue to Graham Farmer Freeway. As it is shown by the figure, modelled mainline links experienced an increase of at least about 50 vehicles per hour in their flow after ramp meters are installed in the model. Moreover, the observed increase in number of vehicles in the northern parts of the freeway is more than that of the southern parts. This shows that before installation of ramp metering, capacity of northern parts of the freeway is not efficiently used.

At the end of the one hour modelling period for the freeway without ramp metering a queue of almost 900 meters length, equivalent to around 250 vehicles, was built up at the top zone of the model (north of Hepburn Avenue). Such a queue was not observed in the metered freeway models. Figure 4-3 and Figure 4-4 show the differences between the situation of upstream of Hepburn Avenue in the model with ALINEA ramp metering and without ramp metering, respectively. This again proves the advantage of ramp metering in Mitchell Freeway by eliminating queuing possibility.

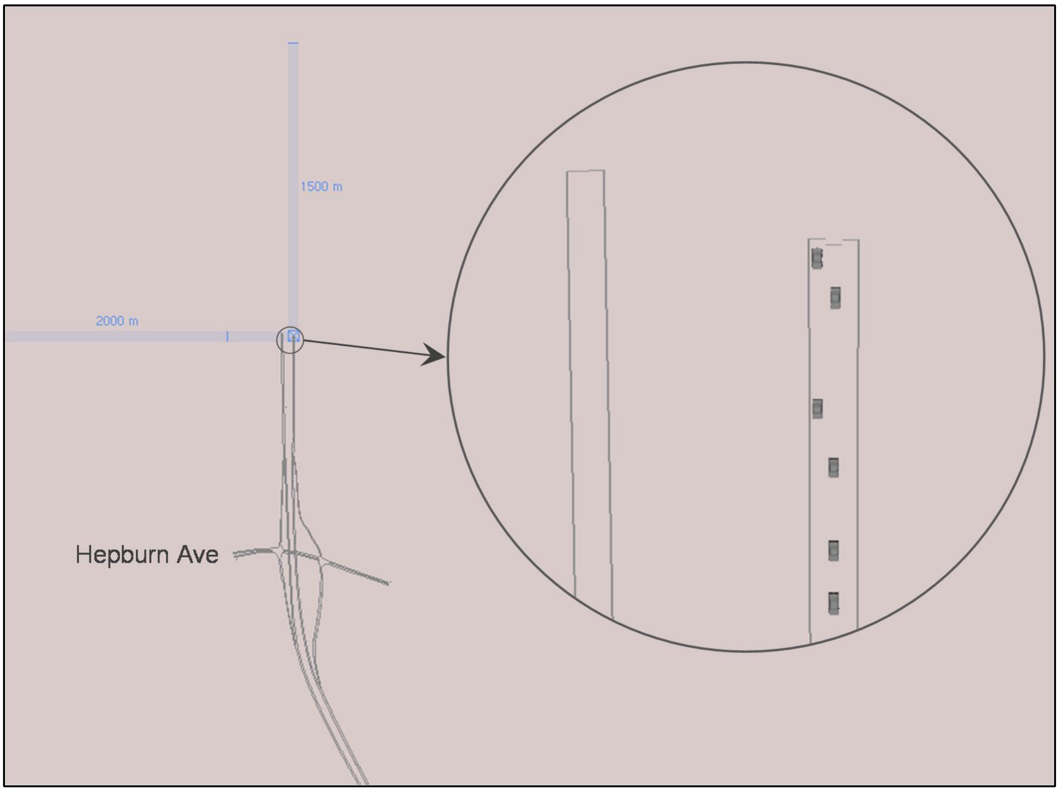


Figure 4-3 A snapshot of the top of the model with ALINEA ramp metering taken at the end of modelling period

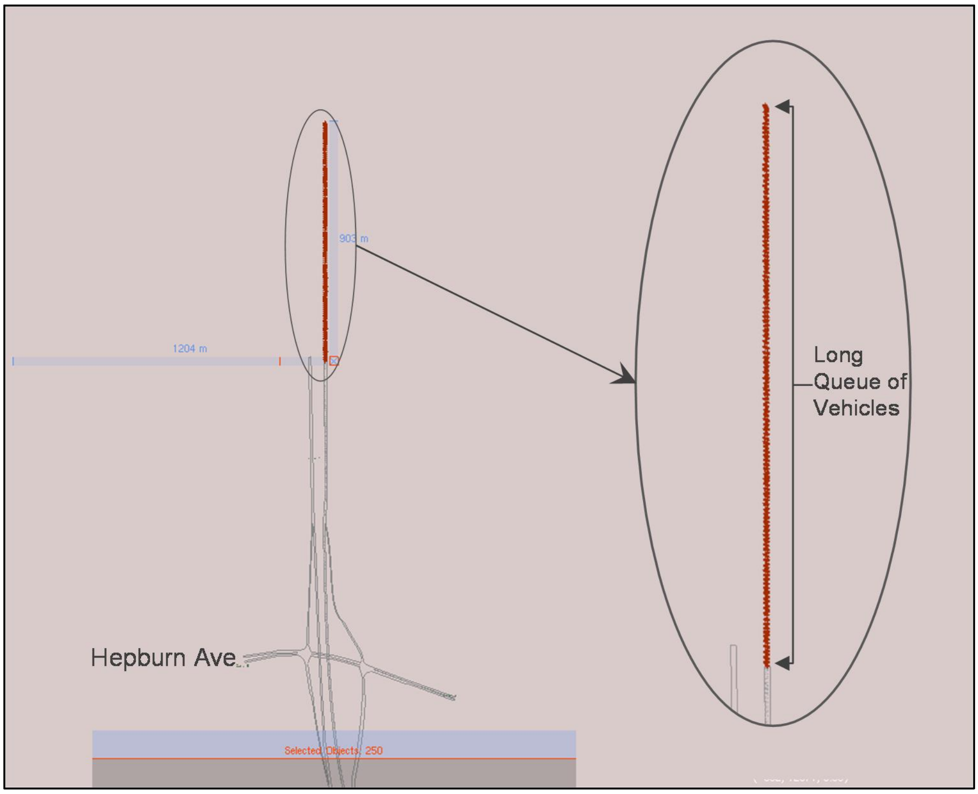


Figure 4-4 A snapshot of the top of the model without ramp metering taken at the end of modelling period

4.3 Travel Time Reliability

The other expected improvement of installing ramp metering is stabilisation of a reliable travel time for freeway travels. Data presented in Table 4-2 prove this improvement in the ramp metered freeway. First of all it is seen that the total completed trip drive distance is about 4,200 km more in the ramp metered model compared to the model without ramp metering. However, the average completed trips drive time is about 4 seconds shorter in the ramp metered model compared to the model without ramp metering. In the case of ALINEA ramp metering, an average of 4 second reduction in drive time (compared to the model with non-metered ramps) results in about 23 hours reduction in total travel time of 20,657 vehicles.

Moreover, the smaller standard deviation of travel time in the ramp metered model indicates more reliable travel times in this model compared to the model without ramp metering. As it is expected, the incomplete drive time is longer in the model without ramp metering.

Table 4-2 Average completed and incompleted drive didatnce and average travel time and its standars deviation

Freeway Model	With ALINEA Ramp Metering	Without Ramp Metering
Total Completed Trips Drive Distance (Km)	180,635	176,399
Average Completed Trips Drive Time	7 min and 20 sec	7 min and 24 sec
Travel Time Standard Deviation	4 min and 24 sec	4 min and 50 sec
Average Incomplete Trips Drive Time	7 min and 10 sec	8 min and 44 sec

4.4 Fuel Consumption and Air Pollutions

The number of vehicle stops recorded in the model without ramp metering was 94,875 while this number was 55,184 in model with ALINEA ramp metering This is an evidence of reducing the fuel consumption with ramp metering (Arnold Jr 1998; Sonesson 2000; Stevanovic et al. 2009).

Air pollution is also directly related to fuel consumption of vehicles. Table 4-3 shows the produced particulate air pollution (CO2, NO, and PM10) per vehicle per kilometre. As it can be seen in this table, installation of the freeway ramp metering has mitigated air pollution in the area. Although in the current model the effect is not tremendous, using more optimised ramp metering algorithm are expected to reduce emissions considerably.

Table 4-3 Completed trips' air pollution per vehicle per kilometres

Completed Trips' Particulate Air Pollution	With ALINEA Ramp Metering	Without Ramp Metering
CO_2 (kg)	0.1785	0.1790
NO (g)	0.38100	0.38106
PM10 (g)	0.0022	0.0023

4.5 Drawbacks of ALINEA Algorithm

In previous sections it was shown that ramp metering generally improved the traffic of Mitchell Freeway in terms of capacity of the freeway, travel time reliability, and fuel consumption. However, it was noticed that ALINEA algorithm has also some negative effects on modelled freeway traffic.

As mentioned before, ramp metering used in this study is a “local” ramp metering. This kind of ramp metering can reduce merging issues and improve freeway traffic where there is a high merging flow. There are however, some issues related to this type of ramp metering. Its functionality and ability to establish a global traffic balance along particularly long routes is limited compared to a coordinated control. For example, if bottleneck capacity of a ramp meter downstream is less than its upstream flow, local ramp metering does not function properly. Also, as the algorithm is local each ramp meter behaves merely based on its own upstream bottleneck. Therefore, it is possible that an upstream ramp meter produces a large

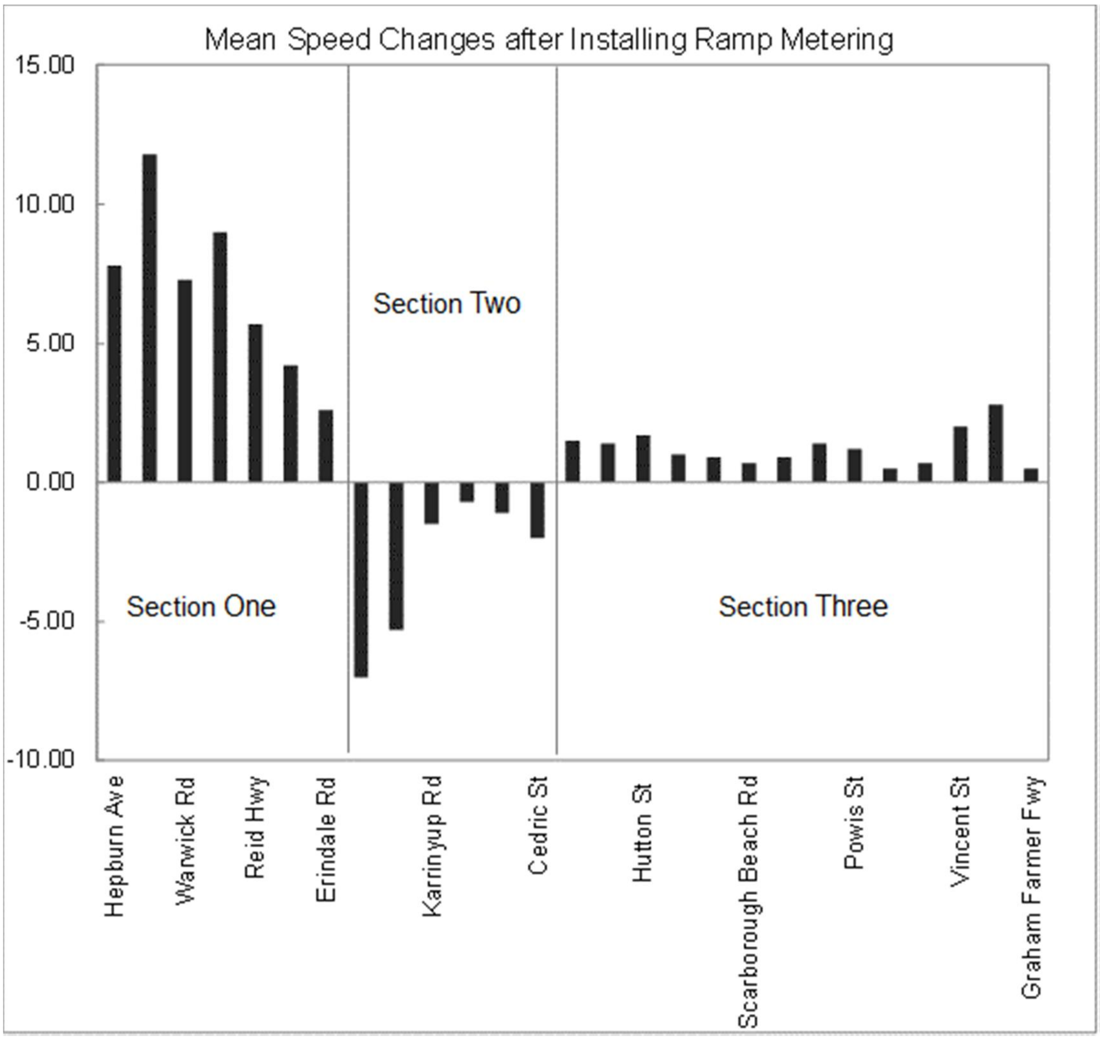


Figure 4-5 Change in link speeds (km/h) in the model with ramp metering compared to the the model without ramp metering

flow in freeway which may transfer all the loads to a downstream ramp meter managing a critical bottleneck. Furthermore, if there are congestions due to other bottleneck in the freeway, local ramp metering may not be able to maintain the optimum freeway throughput. Local ramp metering is also not recommended to be used in freeways with heavy traffic, high demand on-ramps, and very critical bottlenecks (Burley and Gaffney 2010).

Figure 4-5 shows the change in speed of all freeway mainline links in the model after installing ALINEA ramp metering in the model. In this figure the name of the arterial road refers to the mainline section of the freeway in the model opposite to that arterial road.

In Figure 4-5 freeway southbound is divided into three sections. Section One is from upstream of Hepburn off-ramp to Erindale on-ramp. Section Two is from the end of Section One to Cedric Street. The rest of the freeway is labelled as Section Three.

As it is seen in the figure, link speeds of Section One increased in the models with ramp metering compared to the model without one. This means that ramp metering algorithms work effectively in this section. However, it resulted in a decrease in link speeds of Section Two. This can be explained by the increase in capacity of Section One due to ramp metering which transfers more traffic to the next section (Section Two). Vehicles in Section Three of the freeway however, experienced higher speeds in the model with ramp metering compared to the non-metered model.

As mentioned before, the ramp metering method used here are local algorithms and may not be able to manage the downstream bottlenecks.

4.6 Summary

Despite the limitations of the modelling the freeway and simplifications associated with the modelling, comparison between the behaviour of the model with ramp metering and that without one shows a general improvement in traffic of Mitchell Freeway. The results show the models with metered ramps have more capacity, more reliable travel time and less fuel consumption and air pollution. Nevertheless, it was observed that application of ALINEA ramp metering on Mitchell Freeway may have some drawbacks. As ALINEA is a local ramp metering algorithm, it negatively affects some parts of the freeway traffic by reducing the speed of vehicles.

5 Conclusions and Recommendations

This study analysed the effects of ramp metering on the Mitchell Freeway. For this purpose, as a first step a base model of the freeway was built using micro-simulation modelling. Relevant parameters were calibrated, and the model was validated against recorded freeway travel times. Afterwards, ramp metering based on the ALINEA algorithm was implemented in the model and the model was run accordingly. The results were then compared to those obtained from the base model.

An analysis of the effects of ramp metering on the freeway was conducted considering different criteria: freeway capacity, travel time reliability, and fuel consumption and associated carbon emissions.

In order to examine the effect of ramp metering on freeway traffic, a number of vehicle parameters were measured. Values of such parameters from the base model and the ramp-metered model were then compared. First of all, vehicles completing their trip over the modelling period in the base model was compared to that of the ramp metered model. In the ramp metered model this number was shown to be larger than the base model although the difference is not significant. Simulations showed improvements in mean flow after installation of ramp metering. The northern parts of the freeway experienced the largest increase in mean flow, while parts close to the CBD experienced the least increase. Modelling also showed that at the end of modelling period a long queue of vehicles was formed at the top zone of the model while such a queue did not form in the ramp metered model.

In general, the results noted above indicate that currently Mitchell Freeway capacity is not optimally used, particularly in its northern parts. Micro-simulation shows that proper ramp metering can potentially make use of the freeway more efficiently by increasing mean flow in different sections of the freeway and by prevention of queues.

The effect of ramp metering on freeway travel times was also investigated. Simulation results showed the total completed trip drive distance was about 4200 km more in ramp metered model compared to the base model, while the average completed trips drive time is about 4 second shorter in the model with ramp metering. Considering the total number of vehicles (which is 20,657) the corresponding reduction in drive time is corresponding to a reduction of about 23 hours in total travel time of vehicles. Furthermore, standard deviations of travel time in the ramp metered model is reduced indicating more reliable travel times for users of a ramp metered freeway.

Finally, to examine the effect of ramp metering on fuel consumption and associated air pollution, the number of vehicle stops recorded by both models was compared. The comparison shows a reduction of about 42% in vehicle stops in the ALINEA ramp metered model compared to the base model without ramp metering. This is an indicator of significant reduction in fuel consumption and associated air pollution caused by excessive fuel consumption in the current configuration of freeway.

It is worth noting that the current study was conducted within limited time constraints and the results are based on the data and tools that were available. Therefore, the work has still the potential to be improved. For future studies in this area, following recommendations are suggested:

- The current study is focused on ALINEA algorithm which is a local ramp metering algorithm. For future studies on Mitchell Freeway ramp metering, it is recommended that the effects of coordinated ramp metering on Mitchell Freeway traffic be evaluated. Coordinated ramp metering algorithms are expected to deliver better results due to the fact that they control freeway traffic globally within the freeway physical scope.
- The other effect of ramp metering which is recommended for further study is diversion. Diversion was not considered in this study. By applying ramp signals queues may form which in turn may result in diversion of short journeys from the freeway to nearby arterial roads.

It is important to perform a system-wide study on diversion effects. Also, diversion may cause reduction in on-ramp demand (because drivers use arterial roads rather than the freeway) which may reduce the chance of queue formation on freeway on-ramps. The latter may help in even distribution of freeway demand between freeway on-ramps.

- In the study area of Mitchell Freeway, there are signals installed upstream of each freeway on-ramp entrance. This fact may affect the entrance pattern of vehicles to the on-ramps. The entrance pattern is a key factor in determining merging operations as well as queue formation behaviour. As a result, it is suggested that future work considers inclusion of signals upstream of each on-ramp entrance in order to take into account its possible effects on entrance pattern of vehicles.
- There are some limitations in Commuter package (Version 4) used in this study such as free flow simulation and lane distribution on the freeway mainline. Although these problems were fixed by adjusting the car-following algorithm and adding lane choice rules, it is recommended that more powerful packages or newer versions of Commuter with required improvements be used to avoid these simulation issues.
- Finally, to increase the validity of micro-simulation study results, an appropriate field study is also recommended. It might not necessarily be as large as the current network. However, such a study may be suitable for analysing different scenarios for smaller sub-area network.

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7 Appendix

7.1 Sub-area Matrix

Zones	Karrinyup Road (West)	Cedric Street (East)	Hutton Street (West)	Hutton Street (East)	Lake Monger Drive	Powis Street (West)	Toward GFF	Hepburn Avenue (West)	Powis Street (East)	Warwick Road (West)	Erindale Road (East)	Cedric Street (West)	Vincent Street	Erindale road (West)	Reid Highway (East)	North of Hepburn Avenue	Hepburn Avenue (East)	Reid Highway (West)	Warwick Road (East)	Karrinyup Road (East)	SUM
Karrinyup Road (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Cedric Street (East)	0	0	0	177	0	0	470	0	0	0	0	0	131	0	0	0	0	0	0	0	778
Hutton Street (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hutton Street (East)	0	0	0	0	0	0	539	0	0	0	0	0	100	0	0	0	0	0	0	0	639
Lake Monger Drive	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Powis Street (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
From GFF	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hepburn Avenue (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Powis Street (East)	0	0	0	0	0	0	1977	0	0	0	0	0	113	0	0	0	0	0	0	0	2090
Warwick Road (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Erindale Road (East)	0	574	0	149	0	0	524	0	0	0	0	0	155	0	0	0	0	0	0	88	1490
Cedric Street (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Vincent Street	0	0	0	0	0	0	652	0	0	0	0	0	0	0	0	0	0	0	0	0	652
Erindale road (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Reid Highway (East)	0	68	0	117	0	0	275	0	0	0	0	0	83	0	0	0	0	0	0	15	558
North of Hepburn Avenue	0	439	0	501	0	0	1568	0	0	0	0	0	493	0	943	0	211	0	123	85	4363
Hepburn Avenue (East)	0	69	0	166	0	0	537	0	0	0	0	0	154	0	379	0	0	0	7	7	1319
Reid Highway (West)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Warwick Road (East)	0	114	0	169	0	0	418	0	0	0	0	0	139	0	302	0	0	0	0	19	1160
Karrinyup Road (East)	0	17	0	250	0	0	818	0	0	0	0	0	208	0	0	0	0	0	0	0	1292
SUM	0	1281	0	1528	0	0	7777	0	0	0	0	0	1575	0	1623	0	211	0	130	215	14340