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Ramp Metering Methodology: The Ottawa-Carleton Case.

By

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University of Science & Technology,
Kumasi, Ghana.*

*A Thesis Submitted to the
Faculty of Graduate Studies & Research
in Partial Fulfilment of The Requirements
For The Degree of*

Master of Engineering

*Department of Civil and Environmental Engineering,
Carleton University
Ottawa, Canada
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by

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in partial fulfilment of the requirements for the degree of
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Dedicated to my parents Mr & Mrs JEK Armah

ABSTRACT

Many large cities are using or planning to use advanced traffic control systems to improve the efficiency of traffic movement in their travel corridors. Ramp metering is a very effective technique, which releases traffic volume entering urban freeways, especially at peak periods, in measured quantities, such that *stop and go*, and erratic traffic conditions are eliminated and freeway travel times are reasonably predictable. Although its success in most cities where it has been used has resulted in its increasing use, the benefits have been limited to the freeway users, due to lack of co-ordination, co-operation & communication of metering strategies with area-wide traffic management activities in the travel corridor.

This thesis presents a ramp metering methodology, evaluates its impacts in the corridor, and ensures that all intersection approaches on alternate routes and its crossing streets operate at selected levels of average stopped delay or better.

The methodology was demonstrated in the Ottawa Queensway freeway corridor, using optimization and simulation techniques. Finally, guidelines for the implementation of the methodology through an advanced traffic management system are described.

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Glossary & Abbreviations

Glossary

Average Stopped Delay	The total number of stopped vehicles at an intersection approach for a time period divided by the total number of vehicles crossing the same intersection approach for the same time period.
Average Travel Speed	The total distance, divided by the average travel time of a given road network.
Average Travel Time	The total time taken to move from one point to another, including stops, and all delays encountered.
Best Metering Rates	A term used by the author for the maximum metering rate which minimises corridor travel time and, at the same time not violating established threshold capacities for all intersection approaches.
Corridor	An uninterrupted access controlled facility (i.e., a freeway) and all other surface routes in its vicinity affected by the freeway.
Latent Demand	The number of vehicles desiring to use a road facility.
Mainline	A controlled access facility (same as freeway).
Mainline Lane 1	The freeway lane adjacent to the right shoulder.
Measures of Effectiveness	Parameters like average stopped delay, corridor total travel, etc used to describe the service quality of a facility.
Occupancy	Occupancy as used in this thesis refers to detector occupancy: It is the percentage of time a loop detector is turned on
Optimised Metering Rates	In this study, it means metering plan based solely on linear programming method.
Off Line Metering	Metering rates based on historical traffic conditions.

On Line Metering	Metering rates based on real time traffic conditions.
Platoon	A moving queue of vehicles or pedestrians.
Ramp Gore	The intersection between the left edge of a ramp pavement and the right edge of a freeway pavement.
Threshold Capacity/delay	An established capacity/delay value usually lower than its maximum capacity/delay value.
Total Travel Time	The sum of the travel times of all vehicles in a given section of a road network, measured in vehicle-minutes or vehicle hours.

Abbreviations

ACS	Advanced Control Systems
AFMS	Advanced Fleet Management System
ATCS	Advanced Traffic Control System
ATIS	Advanced Traveller Information System
ATMS	Advanced Traffic Management Systems
AVI	Automatic Vehicle Identification
AVL	Automatic Vehicle Location
EB	Eastbound
CBD	Central Business District
CCTV	Closed Circuit Television system
CMS	Changeable Message Signs
FTMS	Freeway Traffic Management System
IVHS	Intelligent Vehicle Highway System
LCS	Lane Control System
LT	Left Turn
MTO	Ministry of Transportation of Ontario
NB	Northbound

NEMA	National Electrical Manufacturers Association
RMOC	Regional Municipality of Ottawa Carleton
RT	Right Turn
SB	Southbound
ST	Straight Through
TAC	Transportation Association of Canada
TCS	Traffic Control System
UTCS	Urban Traffic Control System
VDS	Vehicle Detection System
VMS	Variable Message Signs
WB	Westbound

CHAPTER 1

INTRODUCTION

1.1 Background.

Urban and suburban development and land use patterns have induced numerous vehicle trips, which use major arterials, highways & freeways. Convenience, privacy, comfort, freedom and flexibility, offered by the private automobile, have encouraged automobile use despite rising costs of ownership and maintenance. In many North American cities, peak period congestion, and its undesirable social and economic repercussions are already felt. In the Los Angeles region for instance, \$3 billion / year is estimated to be caused by congestion, of which 63% occurs on freeways [Johnston et al, 1990]. In a study by Hawks & Lomax, about 1.2 billion hours of delay, 1.3 billion gallons of wasted fuel and more than \$9 billion in road user costs, in the U.S.A, are caused by congestion annually [Hawks & Lomax, 1989]. Another estimate by the same author, indicates that the yearly cost of congestion for some 39 selected cities in the US was \$41 billion. A report from the US Secretary of Transportation to Congress revealed that the yearly cost of congestion is over \$100 billion in lost productivity alone, 40,000 fatal accidents and over 5 million injuries [US DOT, 1992]. Urban congestion is therefore socially, economically and politically undesirable and all efforts to improve it are desirable.

There are three main ways to relieve congestion.

- Increase road capacity by adding more lanes (supply side measure).
- Manage the existing capacity more efficiently (traffic management).
- Reduce demand (demand management)

Supply management measures have been used for many decades. When more lanes are added, in many cases in a few months, if not weeks or days, the facility returns to its former state of congestion mainly because of the increased number of vehicles desiring to use the facility. This is due to latent demand, and diversion from other routes. The end result is that increasing volume of traffic is attracted to the improved facility. Every city has its own acceptable congestion level. If a facility is improved to offer a level of service better than this level, attraction of more vehicles returns it to its original state of congestion [Spielberg, 1982]. The degree of congestion in any city depends on a number of factors including the type of transportation system, population, car ownership, land use patterns, income, the quality of public transit, etc. Right of way constraints in major urban areas and recent emphasis on environmental quality are diminishing the opportunity to add supply in an urban area. For a new project or facility expansion to take place, the following factors have to be studied [Morlok, 1978; US Army Corps of Engineers, 1992; IBI Group, 1993]:

- displacement of households, families, industries and businesses;
- land use Patterns: whether the facility will affect land development in terms of increasing imbalance in urban growth or cause an increase or decrease in property values;

- historic, monumental, and aesthetic losses;
- air Quality: CO, CO₂, HC, NO_x, and particulate matter produced before and after construction;
- noise levels before, during and after construction; its impacts on residential, commercial, recreational, educational, religious & other facilities;
- impact on wild life migration, aquatic and terrestrial plant species;
- effects on ground water, wells, aquifers, surface water elevations, catchment areas, ponds, etc;
- effects of facility construction on temporary road diversion, noise, air pollution, fuel and oil spills to adjacent waters; sedimentation & erosion caused by vibrations from machines and all other waste produced during construction activities.

To make matters worse, in spite of the escalating of the already prohibitive cost of capacity expansion, budget allocation to transportation agencies is decreasing. Even if supply management solutions are affordable and right-of-way is not a problem, should enough freeways be built to completely eliminate congestion? If for argument's sake the answer is a yes, can enough freeways be built to eliminate congestion? It appears that a better solution to congestion is through efficient management of existing facilities and modal shifts to public transit.

Urban streets, designed as interrupted flow facilities, are controlled by stop signs, yield signs, signalised intersections, etc, for an orderly movement of traffic. Freeways, on the other hand, are designed for uninterrupted traffic flow with limited access/egress

points to serve high speed traffic between major locations with the hope that traffic would operate efficiently. Increased inter and or intra-urban trips have made vehicles seeking reduced travel times converge onto freeway entrance ramps and compete for downstream mainline lane 1¹ gaps in a disorderly manner with the result that there is a breakdown of the free flowing upstream traffic [Wainright, et. al. 1972]. Unlike arterials, where there could be alternative routes to use, forced flow freeway condition lends itself to no route choice until the next exit.

Intelligent Vehicle Highway System (IVHS) technology, which features application of advanced sensors, computers and communication technologies to get a reliable and real time information on vehicle and roadway conditions, seems to be a very promising technique to improve the efficiency of surface transportation. Apart from being in its early stages of technical development, non technical issues need to be resolved before IVHS can be fully implemented [US DOT, 1993; Chen, et. al. 1990].

- IVHS technologies in North America will require the participation and partnership of both private and public organisations in terms of capital and technical expertise. For these organisations to be comfortable and make maximum contributions without any reservations, the level of protection from tort, liability claims and damages should be clearly defined rather than the present "Rule of Reason" to settle disputes.
- IVHS technologies will take over some of the driving tasks: In the advanced vehicle control mode for instance, vehicle headways, gaps, side clearance between

¹ Mainline lane 1 refers to the freeway lane adjacent to the right shoulder.

cars, braking etc will be out of the driver's control. If a collision occurs who will be responsible, the driver, the vehicle manufacturers, or the transportation agency? Will private agencies get involved in such an uncertain technology with costly risks? If they do, can they afford to pay for the sky rocketing insurance premiums?

- The success of IVHS technologies requires developers and manufacturers to work hand in hand to install and operate such systems. By involving a lot of participants with varying degrees of technical expertise it is not clear how copyright and other intellectual rights of these professionals will be resolved, when part of the project is financed by a public agency.
- Industry standards will have to be resolved internationally such that an European or a Japanese vehicle can be used in a North American environment and vice versa. This will prevent the proliferation of different technologies performing the same functions, such as the *Beta* and *VHS* video cassette recorders, when finally the former dropped out of the market.
- Technologies like the Automatic Vehicle Identification, where movements of all vehicles will be well documented and monitored, are likely to raise motorist concerns about their privacy. Such information, on each motorist's day to day activities may be abused (i.e., accessed by other organisations and used for other undesirable purposes).
- Are College and University graduate students being prepared to meet the challenges of these complicated and advanced technologies which require the full

understanding of their technical aspects?

In view of the above issues which are yet to be resolved, the full realization of IVHS technologies to cope with urban and suburban congestion is premature. It is expected that a significant time lag will occur (most likely by the turn of the century) before all IVHS technologies may be implemented. Meanwhile urban corridor congestion will have to be dealt with. Urban & suburban freeways in concentrated populated regions are increasingly using advanced detection, surveillance, control & advisory measures to reduce congestion. These methods are popularly known as Freeway Traffic Management System (FTMS), or urban traffic control systems. Unlike the more advanced forms of IVHS, these strategies are one way "on line" or "off line" communications between the transportation system and the driver. Given suitable alternate routes, ramp metering is a very effective strategy to maintain an efficient movement of traffic on freeways [Jacobson et al, 1989; McCormick Rankin, 1986; Blumentritt et al, 1981; US DOT 1976; Cocoran et. al., 1991].

1.2 What is ramp metering?

The Oxford Reference Dictionary defines metering as an instrument for measuring the amount of a substance supplied or used, time spent, distance travelled etc. Ramp metering can therefore be explained simply as device for recording and controlling the amount of vehicles passing through a ramp. Metering is usually undertaken with a purpose: to know if the amount of a measured substance is sufficient or not. If required quantities are obtained, the logical decision is to shut it off. Ramp metering therefore is a means of

measuring and controlling the amount of traffic entering the freeway using a ramp traffic signal. Metering is undertaken to maintain a stable and steady traffic flow rate of the downstream traffic at all times. A successful metering strategy maximizes throughput, reduces side swipe and rear end collisions, improves travel time predictability, and discourages short trips from using the freeway. It also breaks up platoons of vehicles discharged from a nearby surface street traffic signal and allows vehicles to merge onto the freeway in an orderly manner. [Wainright, 1972 et al; Blumentritt, et al 1981; US DOT, 1976; Robinson & Doctor, 1989].

1.3 Case Studies

Ramp meters are in use in more than 20 cities in North America, Japan, Britain, Germany, and some other European cities. Fig 1-1 indicates cities in North America implementing some form of metering. Ramp metering in some locations are not easy to evaluate because they form an integral part of other freeway management activities such as driver information systems, changeable message signs, incident management programs etc. These case studies were carefully selected at locations where ramp metering alone was installed at the time. Where other freeway improvements were installed, they are mentioned.

- In Portland, Oregon, metering 16 ramps in fixed time mode in 1981 increased average freeway speed during peak periods from 26 km/h to 72 km/h; travel time reduced from 20 to 9 minutes. Fuel consumption, reduced by 2077 litres of gasoline per weekday.

Figure 1-1 Ramp Metering Systems in North America

Source: Robinson, J & Doctor, M 1989

Side swipes and rear end collisions reduced by 43% [Robinson & Doctor, 1989]

- In St Paul, Minnesota, 6 metered ramps, installed in 1971 in traffic responsive mode on a 5 mile section of I-35E, after 14 years of installation, increased peak hour speeds, from 60 to 69 km/h and reduced peak period collisions from 24 to 38% whilst peak period volumes increased by 24%. [Robinson & Doctor 1989].
- In Seattle, Washington, metering I-5, north of CBD (17 SB AM ramps and 5 NB PM ramps) was undertaken in 1981. After six years, mainline volumes during peak periods increased 86% NB and 62% SB. Travel time reduced on the six mile section from 22 to 11 minutes and peak period collisions reduced by 39% [Robinson & Doctor, 1989].
- In Denver, Colorado, a pilot program was conducted in 1981 on NB I-25. After 18 months metering 5 ramps on 4.7 kilometre section in local traffic responsive mode, travel speeds increased 57%, travel time reduced 37% and side swipe and collisions declined 5%. These positive results led the transportation agency to meter more ramps and install a centralised computer to control and coordinate all metered ramps. Evaluation in 1989 indicated that speeds were lower than the 1983 condition (traffic responsive metering) but higher than 1981 conditions (pre metering). Although collisions were not much different and speeds reduced, the central computer control indicated significant benefits because the NB I-25 increased from 6200 vph in 1981 to 7350 vph in 1989. On one occasion in 1987, when the central metering system was turned on an hour later, traffic was the worst than it had been for years [Cocoran & Hickman, 1991].

- In Detroit, Michigan, metering 28 ramps on I-94 in 1982 increased speeds by 8%, reduced collisions by almost 50%, and typical peak volumes on the three EB lanes increased from 5600 to 6400 vph in 1985 [Robinson & Doctor, 1989].
- In Austin, Texas, Local traffic responsive strategies on 3 ramps along a 4.2 kilometre segment of NB I-35 on am peak increased vehicle throughput by 7.9% and a 60% increase in mainline speeds [Mardsen, 1981].
- In Atlanta, Georgia, installation of ramp signals on the Tenth street northbound on ramp of the I-75 in 1967, reduced rear end collisions by about 90% over a 12 month evaluation period [Thomas, 1969].
- In Canada, the first Freeway traffic control was implemented in 1974, by MTO along a 16 km stretch of Queen Elizabeth Way (QEW), Mississauga. The system included loop detectors, incident confirmation by Closed Circuit Television (CCTV) system with remote cameras with pan and zoom capabilities and ramp metering. It yielded [MTO, 1991]:
 - 22% reduction in collisions;
 - 45% increase in average speeds;
 - 21% reduction in overall delay.

Increasing traffic on other urban/suburban freeways and the encouraging results of the system, led to the initiation of the FTMS program in the early 80's which promoted designs and implementation of the Burlington Skyway FTMS and Highway 401 FTMS popularly known as COMPASS. Completed feasibility studies on the Gardiner Lakeshore Corridor and expansion of the Skyway are all

clear evidence of the past and present successes of freeway traffic control [MTO, 1991; IEI Group, 1991].

- In Montreal, freeway traffic control has been recommended for the Bonaventure/Champlain Bridge Corridors & the A25/Metropolitaine/Decarie/Ville Marie Corridor [SNC/DELUC, 1988].
- In the Ottawa Carleton Region, two consultant studies done for MTO in August 1987 and 1993 recommended freeway traffic control to ease peak period congestion in the most heavily travelled sections. Ramp metering was recommended for eastbound Richmond, Pinecrest, Woodroffe and Maitland on-ramps; and westbound p.m. St Laurent, Alta Vista and Nicholas on-ramps [IBI, 1987 & 1993; MTO 1990].

1.4 Problem Definition

There have been many studies done on ramp metering methodologies. The concept of metering is to improve the function of the freeway by restricting on ramp volumes at peak periods and allowing these vehicles to enter at a rate which the facility can handle. It smooths the on ramp merge into the freeway allowing vehicles to enter the freeway in an orderly function.

Most metering strategies have been implemented with varying degrees of success through surveillance and control, with the sole objective of improving the mainline traffic stream. There is however a general lack of study on its effect on the surface streets in terms of arterial travel time. All the case studies just discussed quantified improved

freeway flows, decreased freeway travel times, etc, but very few quantified its impacts on alternative routes. The response from the public to metering therefore varies from totally acceptable, where in some cases the public actually requested more ramps to be metered, to highly unacceptable where in an extreme case these metered signals were shot by a gun [Robinson & Doctor, 1989; Spielberg, 1982].

Furthermore, although ramp controls have improved traffic in almost all areas used, single and multi occupant vehicles are equally delayed. To maintain a stable traffic flow as well as meet another objective of encouraging car pools and other Higher Occupancy Vehicle (HOV) plans and programmes, HOV's should be given priority treatment where feasible. Such innovations have been identified by a growing number of municipalities and other jurisdictions, including the Regional Municipality of Ottawa-Carleton (RMOC).

The key to successful metering is a detailed analysis of freeways as well as all adjacent road network which diverted traffic will use. To do this, a careful identification of goals and objectives is needed. To increase acceptability of metering, control strategies should promote an efficient movement of traffic in the corridor as well as minimize excessive delays to any intersection approach.

1.5 Research Objectives and Scope

This study included a detailed analysis of both Highway 417, a freeway, and its adjacent surface streets, to determine the optimal ramp metering strategy. The purpose of this research was to

- develop a methodology for optimal ramp metering from a corridor-wide perspective,
- determine the impacts of metering on the corridor, and
- describe the conceptual and system aspects of implementing the methodology for ramp metering from a corridor-wide perspective.

1.6 Thesis Outline:

The thesis is divided into eight chapters and two appendixes. Following this introductory chapter, chapter 2 describes the framework of research. The structure of research and a brief overview of existing methodologies are discussed.

Chapter 3 provides a brief introduction to IVHS, FTMS, and a detailed treatment of ramp metering. The inter - relationship between these elements of traffic management and control are discussed. The purpose of the chapter is to acquaint the readers who may be unfamiliar with the present state-of-the-art.

Chapter 4 begins with a detailed treatment of the methodology and its application to the Ottawa Queensway. The chapter mainly deals with Corridor Control Module.

Chapter 5 deals with Freeway and Arterial Control Modules. Ramp metering strategies, arterial signal optimisation to cater for increased traffic on alternate routes are treated. TRAF-NETSIM and other computer simulation models are also discussed including data sources, and analysis.

Effects of traffic on the freeway and surface routes are discussed in Chapter 6. Chapter 7 deals with verification of simulation models through field studies. Travel time

studies were carried out and compared to simulated results to check their accuracy.

In Chapter 8, system implementation is discussed. Conclusions and recommendations can be found in chapter 9, followed by references and Appendixes.

CHAPTER 2

RESEARCH FRAMEWORK

From the case studies in the previous Chapter, it could be seen that most ramp metering evaluations did not cover any detailed studies of adjacent surface streets. They were mainly based on the assumption that the diverted traffic would not adversely affect traffic on these routes. This chapter is devoted to evidence of research need, and an outline of research methodology.

2.1 Research Need

In a feasibility study conducted on the Ottawa Queensway in 1987, it was mentioned that *"...From a total of 3100 ons¹ distributed across the 7 ramp locations, 230 diversions represent a total diversion of approximately 7% of on ramp traffic. It was determined after discussion with RMOC that these volumes were not large enough to significantly change the operation of arterial roadways"* [IBI 1987]. Vehicle delays were estimated for interchanges such as Richmond, Pinecrest, Woodroffe and Maitland.

¹ 3100 ons, stands for 3100 on ramp traffic.

Evaluation of metering was based on freeway and arterial travel time studies, O.D. data and forecasted ramp queueing delay.

The question is how much is this going to affect the eastbound, westbound, northbound and southbound approach traffic on surface streets, does it minimise corridor total travel time?

In the Denver case, benefits to ramp metering were analyzed for local responsive and system coordination ramp metering by measuring freeway traffic speeds and volumes before and after implementation, but its impacts on alternate routes were not assessed [Cocoran & Hickman, 1991].

In a real time metering algorithm developed for the Seattle area, it included a ramp queue adjustment logic as the main restriction on its impacts on alternate routes. Before and after studies were conducted with respect to ramp delays, ramp signal violations, freeway volumes, freeway travel time studies, and freeway accidents [Jacobson et al 1989].

Existing ramp metering strategies are mainly based on providing a stable flow on the mainline; satisfying the minimum and maximum metering rates; and satisfying the maximum queue length criteria.

Pretimed metering strategies for instance, are developed using historical demand /capacity volumes to establish metering rates or by optimization through Linear Programming [Blumentrit, 1981; Masher et. al., 1975; Institute of Transportation Engineers, 1992; Wattleworth, 1967; Wang, 1972; Pignataro, 1973; May, 1974].

Wattleworth, for example, developed pretimed metering strategies using linear programming by maximising total freeway traffic subject to volumes of each section operating at threshold capacities but did not study impacts on surface routes [Wattleworth, 1967].

Wang, used a strategy similar to Wattleworth except that he used a dynamic programming strategy, with a non-linear objective function to minimise delays, queue times for each vehicle, and average travel time, at peak period [Wang, 1972].

Yuan used a quadratic programming method with linear constraints and a linear objective function to maximize mainline flows at or below capacity and to increase metering rates if the rate of change of entrance ramp queue length is positive. Where increased rates will create forced flow conditions on the mainline, metering rates are selected to minimise ramp queue lengths, and upstream metering rates are reduced to avoid freeway congestion [Yuan, et. al., 1971].

Local responsive control strategies are based on local volume / capacity ratio or occupancy strategies, with a queue detector option to limit ramp queues. The micro-processor based controller is fed with user input control logic, which control "on line" metering rates.

Integrated or segment-wide traffic responsive metering strategies are similar to the locally responsive strategies except that all locally responsive controllers are coordinated through a central computer controller which determines the metering rate of each location on a system-wide basis. In the Denver case, for example, all the algorithms were aimed at improving mainline congestion and minimising ramp queues [Ayland, et. al., 1991;

Cocoran & Hickman, 1991].

From all these methodologies and strategies, although freeway traffic improved, very little documentation on its effects on alternate routes exists. This means that if entrance ramp queues were within certain limits, metering rates were assumed to be adequate. Every solution has its own merits and demerits; therefore to evaluate the benefits of metering, it is very important to consider the diversion routes as well as all their connecting streets. Because diverted traffic will likely require fine tuning of traffic signals, traffic moving in the opposite direction must also be examined.

A successful metering strategy is the one that maximizes mainline traffic throughput and also prevents negative impacts on alternate routes. The need to develop a methodology to analyze the impacts of metering on a corridor-wide basis has been identified by a number of transportation agencies and professionals including *Transportation Research Circular's Research Statements*, Dunn et. al., & May, et. al. [Dunn et al 1992; Transportation Research Circular, 1990, & 1983; May, et. al., 1982]. This study introduces a methodology to address these issues.

2.2 Measures of Effectiveness:

The research is intended:

- to reduce travel time in the corridor, and
- to suggest means to attain an efficient movement of traffic in the corridor;

Based on the above objectives, the following performance measures were used:

Objective	Performance Measure
Travel time	Vehicle minutes Seconds per vehicle
Efficient movement of traffic in the Corridor	Vehicle kilometres of travel at an acceptable Level of Service.

2.3 Research Approach

Two studies have been conducted by other researchers on the most travelled section of Highway 417 (Moodie Drive to Walkley Rd & Highway 417/17 interchange to Champlain Street) to see if the introduction of freeway traffic management strategies is feasible. Both reports recommended ramp metering [IBI 1993; IBI Group, 1987]. This study is not intended to replicate such studies from a benefit cost analysis perspective. It is aimed at developing a methodology for optimal metering, and establishing its feasibility from an operational analysis point of view, by taking into consideration all alternative routes and connecting roads. It is intended to address some of the issues that the two previous studies did not cover. For practical analysis, the Highway 417 corridor was selected as a case study. The intention of the Ministry of Transportation of Ontario (MTO) to meter some ramps on the mainline, makes it suitable for such analysis [IBI Group, 1987 & 1993; MTO, 1990]. In order to achieve the purpose of the study, a research framework was developed and used, as shown in Figure 2-1. The problem definition, covered in Chapter 1, is followed by the concepts of ramp metering technology and operational factors. The present state-of-the-art is described and the interrelationship of ramp metering and other IVHS technologies are defined. At the next stage, a methodology was developed for

establishing the best metering plan.

The developed methodology was tested, using Highway 417 as a case study. For simulations and analyses, **FREFLO**, **NETSIM**, & **PASSER-II-87** computer models were used. To verify the accuracy of these models, and to illustrate the application of the methodology, simulated results were compared with field studies. In the final stage, recommendations and conclusions were compiled. This thesis document, reports all parts of the research framework.

Figure 2.2 is a flow chart to illustrate the procedures involved in establishing "best metering plans". The methodology for this purpose is formulated in a logical order that a transportation agency could follow. The following are major activities that form a part of the overall research methodology. These are:

- Capacity Analysis Module;
- Freeway Control Module;
- Surface Street Control Module;
- Effectiveness Module;

2.3.1 Capacity Analysis Module

To determine if metering will be feasible, the present traffic condition in the corridor should be known. Capacity analysis was therefore conducted on the Queensway from Highway #7 & 17 interchange to highway 417 & Walkley interchange by using 1992 traffic volumes. Sections experiencing congestion at peak periods were identified and a detailed analysis was undertaken to see if metering was feasible.

Figure 2-1 Research Framework

Figure 2-2 Methodology For Establishing Best Metering Plans.

Traffic congestion can be mitigated through a number of solutions; when these solutions complement each other, benefits are maximised. In view of this, the concepts and methodologies have been formulated to reinforce ride share and increased use of the public transit, using ramp metering and HOV by-pass lanes.

Capacity analysis of alternate routes was also conducted to determine whether these could handle any extra traffic. Based on both analyses, a decision is taken to control or not to control freeway ramps. If surface routes are operating close to capacity, either minor geometric modifications can be undertaken to accommodate anticipated diverted traffic, or other transportation system management strategies are to be explored.

2.3.2 Freeway Control module

Freeway control module is activated only if there is residual capacity on surface streets. The procedure involves formulation of metering strategies to suit the problem at hand. Initial metering rates are carried out to minimise freeway travel time.

2.3.3 Surface Street module :

After initial metering of traffic entering the freeway, surface street control module comes into play. Cycle lengths, phase timing and sequence, and offsets are all revised for all signalised intersections to optimize increased traffic in the network, using **PASSER 2-87** software package. Capacity analysis is undertaken on the alternate routes, to examine the impacts of metering plans. If the arterial threshold level of service is not satisfied for any

intersection approach, "optimised metering rates"² are increased in small amounts. The procedure is repeated a number of times until all approach threshold LOS are satisfied.

2.3.4 Effectiveness Module

For each metering rate, arterial, freeway and corridor performance measures were obtained. Corridor performance measures were obtained by adding freeway and arterial travel times, total travel times and vehicle throughput, etc.

The metering plan which minimised corridor travel time, as well as satisfied the threshold LOS for all approaches was considered to be the best metering strategy.

2.4 Current Models Available:

Having defined research objectives and scope, existing models were reviewed to determine their suitability. Review of past and present simulation models have been done by some researchers including Yagar, Aerde, May and Leiman [Aerde & Yagar, 1988a, 1988b, 1987a & 1987b; May, 1987; Leiman et. al., 1991]. These revealed a number of freeway and corridor models including the **TRAF, CORQ, FREQ, INTRAS, FRECON, SCOT, MACK, KRONOS, TRAFFICQ, DYNEV, CONTRAM, SATURN, and MICRO-ASSIGNMENT** models. Following is a brief discussion and critique of integrated models.

² "Optimised Metering Rates" in this study refer to initial metering rates obtained using linear programming approach with an objective function of maximising freeway traffic input.

2.4.1 TRAF

TRAF is a family of simulation models integrated to represent the traffic environment. The main components are **NETSIM**, **FREFLO**, **NETFLO 1**, **NETFLO 2**, **FRESIM**, and **ROADSIM**. Out of these, **NETFLO 1**, **NETFLO 2**, and **FREFLO** (known as **CORFLO**) are combined to simulate traffic in a corridor.

These models, however don't completely replicate the corridor traffic conditions because they don't fully interact with each other: vehicles are stored in a *vehicle holding area*, which is demarcated artificially, from one sub-network to the other. Traffic spillback from one sub-network to the other are not modelled on a continuous basis [US DOT April, 1993; Aerde and Yagar, 1987 & 1988]

2.4.2 CORQ

CORQ simulates a corridor network using O.D. data to estimate traffic queues on freeways to determine ramp metering rates and their impacts on surface streets. Its major weaknesses are that, it is not user friendly, traffic assignment is directional, and traffic signals on surface streets are not modelled.

2.4.3 FREQ

FREQ simulates freeways and an alternate street using O.D. data from ramps. Metering rates are determined using a linear programming technique. Both priority and non priority freeway traffic could be modelled. The main disadvantages are that uninterrupted traffic environment are modelled very approximately and signals are not optimized. Furthermore

the model diverts traffic to only one alternate route and therefore traffic assignment is not undertaken. Based on the model's assessment, Aerde and Yagar had this to say [Aerde & Yagar, 1987a]:

*"Finally it appears that **FREQ** consists of a collection of a highly detailed extensions which have been added to a fundamentally weak base, consequently, many of these extensions provide only marginal or insignificant overall contributions as they are based on a modelling structure which inherently has a large error component already built in".*

2.4.4 INTRAS

INTRAS stands for **INtegrated TRAffic Simulation** model. It is a micro-stochastic model mainly used to simulate freeway incidents. Vehicles are modelled individually to respond to freeway and alternate surface street environment, using a *time-stepping* procedure. It does not model traffic signals, no traffic assignment is undertaken and diversion techniques are not based on O.D. data [Aerde & Yagar, 1987b].

2.4.5 FRECON

FRECON is a corridor traffic model based on a modified **FREFLO** model. Alternate routes are simulated using simple travel time analysis.

2.4.6 SCOT

The **SCOT** is a corridor traffic model with two main composite models: the **DAFT** which is based on micro-modelling of vehicle platoons, and the **UTCS-1** micro-model which uses a car following analogy.

Its major weaknesses are similar to that of other composite models such as **CORFLO**, described earlier. The model is no longer supported.

2.4.7 INTEGRATION

The INTEGRATION model simulates corridor traffic, taking into consideration interactions between freeway and arterial traffic, dynamic traffic assignment, and the changing route selections due to congestion. The entire corridor is considered as one system [Aerde & Yagar, 1988b]. A summary of some integrated models is provided in Table 2.1.

Some of the strengths of these integrated models have not been mentioned. The author has no intentions to be biased in its discussion but to emphasize the point that all existing integrated models cannot replicate a continuous dynamic equilibrium within the corridor. Each has its own strengths and weaknesses. The reasons are mainly due to the fact that:

- corridor models have been developed to meet the needs of an urban traffic system. That is most urban traffic control strategies are localised and operate in isolation. Consequently these models have been developed to simulate local conditions without the thought of a system integration,
- most integrated models, which include the SCOT, TRAF and TRAFLO have been developed and combined together.

For similar reasons, Reiss et al performed traffic simulation in the Long Island corridor in New York, using the SCOT model. Because the model could not optimize traffic signals, signal timing plans were carried out using the TRANSYT model [Reiss et. al., 1991].

For a successful integrated model, a common modelling approach to both interrupted and uninterrupted traffic, taking into consideration traffic diversion and

Table 2.1 Comparison of Integrated Models (Summarised from Aerde et. al., 1987a)

Model	Assignment Technique	Traffic Flows		Model Type	Major Purpose	Critique	Desirable Features
		Freeways	Alternate Routes				
TRAF	TRAFFIC	Fluid conservation	NETFLO 1 NETFLO 2	Deterministic Macroscopic & Microscopic	All networks (1 direction)	No interaction of sub-models	Composite network
CORQ	Incremental, iterative	Step function travel time	Implicit travel time calibration	Macroscopic	Queuing in freeway corridor	Directional traffic assignment	Auto network performance ratings
FREQ	Diversion of ramp queue	HCM (speed-volume)	Simple Travel time	Macroscopic	Freeways and evaluate priority and non priority lanes	Diversion not the same as assignment	Linear programming optimization
INTRAS		Vehicle specific time stopping simulation		Stochastic, microscopic	To study freeway incidents	No O-Ds	
FRECON	Diversion due to ramp metering	Modified from FREFLO	Simple Travel time	Macroscopic	Freeway (1 direction)	Diversion not same as assignment	Adaptive discretization of step size
SCOT	UTCS-1: turning movements DAFT: minimum path	DAFT platoon based speed-density	UTCS-1 individual vehicles	Microscopic and macroscopic	Test real time control policies for corridor	Model component incompatibilities	Composite network
INTEGRATION	Static/dynamic & deterministic/stochastic traffic assignment	Models individual vehicles	Analyze traffic signals and ramp meters	Mesoscopic	Simulates corridor traffic as one system	Cannot optimize traffic signals.	Traces each vehicle's trip from Origin to destination on link by link basis.

congestion is required. Aerde et. al. suggested that true integrated models required the optimization of the entire system, not individual subsystem: therefore adaptation and combination of contemporary models do not dynamically simulate such systems [Aerde & Yagar, 1988b].

Although the **INTEGRATION** model dynamically simulates traffic in the corridor, it was not selected because the intention of the author is to select models such that a reasonable control of stopped delays at all signalized intersection approaches and a conservative traffic assignment is achieved.

For this research, the following computer simulation models were used:

- **TRAF NETSIM**, for arterial analysis,
- **FREFLO** for freeway simulation,
- **PASSER II-87** for signal progression and optimisation, and
- **LP-88**, to optimize ramp metering rates.

These models were combined to,

- conduct a detailed simulation of the alternate route with the view to satisfying threshold delays for all signalized approaches,
- optimize traffic signals to respond to diverted traffic in both directions, and simulate freeway traffic conditions, and
- determine metering rates on a system wide basis, taking into consideration its impacts on all signalised intersection approaches.

Because the combination of these models cannot accurately replicate real traffic conditions in the corridor, a limited and conservative approach have been adopted. A brief

introduction to **TRAF-NETSIM** can be found in chapter 4, page 78 whereas **FREFLO** & **PASSER II-87** are described in chapter 5, page 108

2.5 Why Simulation?

Computer simulation was chosen for the following reasons:³

- Testing various traffic scenarios with combinations of different timing plans, variable metering rates, & amount of traffic diversion, is not practical in the field. This activity would disrupt traffic and require enormous time, money and effort. By using a real freeway for such studies would also cause intolerable levels of delay, and decrease vehicular and pedestrian safety. The impact of such experiments in the field with indiscriminate testing of scenarios would be totally rejected by the public.
- By using simulation, a number of scenarios are tested and evaluated; hence only the most promising ones are tested in the field. This eliminates the need to test undesirable scenarios in the field.
- It is inexpensive and some Measures of Effectiveness (MOE) that would have been very difficult to obtain in the field (e.g., average stopped delays, density, occupancy, total travel time, in vehicle hours etc) are easily obtained.

If traffic diversion studies were conducted in the field, it is easy to imagine what would happen: police officers required to divert traffic, the number of field staff required to carry out speed, density, delay and other intersection studies, fine tuning of signals, data

³ Source: **TRAF User Reference Guide, Version 4 1993, page 1**

reduction from all these studies etc. Apart from cost, time and effort, will it be acceptable to create such scenarios? With the use of simulation, field conditions were created in the laboratory and a number of scenarios were tested.

To verify the level of accuracy of simulation models, travel time studies were performed on both the Ottawa Queensway and Carling Avenue.

CHAPTER 3

FREEWAY TRAFFIC: CONTROL & FLOW

Previous chapters made mention of IVHS, FTMS, and ramp metering. The purpose of this chapter is to give a brief overview of IVHS, a detailed description of the current metering strategies, concepts and philosophies, and their interrelationship.

3.1 Ramp Metering and IVHS

Freeway traffic management makes effective use of the freeway through advanced traffic control systems and driver information means. It is a step in the right direction towards Intelligent Vehicle Highway Systems (IVHS). IVHS systems use advanced sensors, computers and communications to get reliable, precise, and real time information on vehicle and road conditions. Thus they provide accurate advisory, guidance and control strategies to manage traffic [Transportation Association of Canada, 1992; Roos, 1991]. The concept is to integrate the driver with smart cars and smart transportation systems to

Figure 3-1 IVHS & FTMS

foster a more efficient performance. Figure 3-1 is a flow chart showing how ramp metering and freeway traffic management system is tied into the IVHS. The major types of IVHS are

- Advanced Traveller Information Systems (ATIS)
- Advanced Fleet Management Systems (AFMS)
- Advanced Traffic Management System (ATMS)
- Advanced Command and Control Systems (ACS)
- Advanced Public Transportation Systems (APTS)
- Advanced Rural Transportation Systems (ARTS)

3.2 Advanced Traveller Information Systems.

Advanced traveller information systems will provide travellers and drivers with precise information on traffic, roadway conditions, route sequence, schedules, and transportation modes before and during the course of travel in real time, through a broadcast media in vehicles or buildings. Travellers will know the best, safest, and fastest routes; the most convenient mode; and will be able to predict their travel time. Motorists will also know the condition of their route choices, and other valuable information in real time required to select the best route, time and mode of travel. Vehicles will receive dynamic route guidance from their origins to destinations. Components of the system include variable message signs, electronic maps, in-vehicle route guidance system, bus stop displays, and an in- vehicle hazard warning systems (e.g., for slippery roads) [Transportation Association of Canada, 1992; Roos, 1991].

An example of such a system is TRAVTEK, a three year test project by the American Automobile Association, FHWA, Florida Department of Transportation, General Motors, and the city of Orlando. The system provides:

- information on traffic congestion,
- motorist services ("electronic yellow pages"),
- tourist information, and
- route guidance to 100 vehicles which were equipped with in vehicle navigation equipment.

3.3 Advanced Fleet Management Systems

Advanced fleet management systems will be mainly used by trucking and emergency vehicles. With such technologies as Automatic Vehicle Location (AVL), Automatic Vehicle Identification (AVI), Weigh in Motion (WIM), Automatic Vehicle Classification, Automatic Clearance Sensing (ACS), two way communications between fleet operator and vehicles will be possible. Vehicles can be located and monitored through satellite communications or roadside communications via beacons used in conjunction with infra-red or microwave transmissions. Parcels will be traced and monitored continuously whilst pick up and delivery times will be reduced. Truck inspection, axle weights, taxation, safety standards, customs clearance and other vehicle inspection will be transparent and effectively enforced with little or no delays.

An example of such a system is the Advantage I-75 demonstration project, currently under construction, which will improve the efficiency of motor carrier operations

from Florida to Michigan with minimum stops at inspection and weigh-in-stations. The system will eventually be linked to the AVION project.

AVION is the Canadian counterpart of Advantage I-75, which would use similar technologies such as weigh-in-motion, electronic data document processing, etc to keep a close watch on truck movements, and strict enforcement of standards and regulations, on Highway 401, from Windsor to Montreal.

3.4 Advanced Transportation Management Systems

The ATMS is the advanced form of the present Freeway Traffic Management System with the following capabilities:

- ▣ Effective and efficient response to incidents with very little or no false alarms,
- ▣ Traffic in the entire urban area will be effectively and efficiently monitored,
- ▣ Coordination and integration of urban traffic control signals, freeway ramp metering, transit control systems, traffic and travel information systems and all other transportation related actions,
- ▣ Transportation agencies and transportation related organisations will operate in harmony, with common goals and objectives without fragmentation or duplication of control and management strategies,
- ▣ It will be able to predict where and when congestion will occur, hence execution of dynamic traffic assignment based on aggregate origin and

destination information supplied by all motorists at the beginning of their trips will be possible.

The AVI, AVL, & advanced incident detection using image processing technologies will be featured prominently. Dynamic traffic assignment models, automatic toll collection and parking management will significantly increase efficiency in the transportation system. It could be inferred that if a full scale implementation of IVHS technologies takes place, then a number of transportation policies and strategies such as road pricing (e.g., peak period pricing), enforcement of HOV lanes, a better traffic control on a corridor-wide basis, etc, can be implemented with very little and sometimes no obstacles.

3.5 Advanced Vehicle Control Systems

This is the most advanced form of technology, where some driving tasks such as collision and skid avoidance, vehicle headways, lane changes, vehicle warnings, adaptive cruise control and other safe manoeuvres are partially or fully executed by the vehicle, with the help of sensors. Shorter headways, closer lateral lane clearance, and speed controls are possible through either vehicle transponders buried in the pavement or radar transmitters. In the advanced mode, automated lanes on freeways characterised by high speeds, shorter headways and automatic braking, will increase capacity. According to Johnston et. al., vehicle headways of 0.3 secs and operating speeds of 160 km/h, resulting in 12000 vehicles per hour per lane (vphpl) will be possible. As compared with existing capacity of 1800 vphpl, this represents a very high increase in capacity [Johnston et al, 1990].

3.6 Freeway Traffic Management Systems

The lower order of ATMS includes Advanced Traffic Control Systems (ATCS) & Freeway Traffic Management Systems (FTMS). A number of strategies have evolved from FTMS but the major ones are detection, detection and advisory, and detection and control systems. Table 3.1 shows components, merits and demerits of each strategy. Specific components of each depend on a number of factors including traffic characteristics and pattern, geometry and network configuration, objectives and policies of the transportation agency, the nature of the problem, and the last but not the least, available funds. Of all the freeway traffic management strategies, ramp metering was chosen for research because studies done confirm that it has yielded the most positive results [McCormick Rankin, 1986; Bluementritt, 1981; Masher et. al., 1975; Robinson & Doctor, 1989]. Table 3-2 is a summarised form of a study done on the QEW to evaluate the total effectiveness of various strategies on all sections of the freeway from Ford drive to Humber River Road. The parameters measured were reduced accidents, reduced weaving intensity, safe merging, improved Level of Service, and increased speeds (see Table 3-2). It confirms the accepted fact that ramp metering is an important technique of FTMS.

3.7 Types of Metering

There are five major types of metering

- ▣ Ramp Closure
- ▣ Pretimed Metering

Table 3.1 Modified Evaluation of Strategies, Reported by McCormick Rankin, 1986.

Strategy	Methods	Advantages	Disadvantages	Comments
Monitoring and detection	<ul style="list-style-type: none"> • Detector Loops • CCTV Cameras • Call Boxes • Emergency Vehicles 	<ul style="list-style-type: none"> • Automatic detection of incidents and confirmation of incidents 		Considered to be necessary for effective operation of other strategies.
Freeway Ramp Control	<ul style="list-style-type: none"> • Ramp metering 	Maintains predetermined level of service on the freeway. Incident management	Requires sufficient ramp storage and safe stopping sight distance. Requires sufficient acceleration distance	Motorist interprets time in queue to be longer than delay on the mainline without control system.
Freeway Diversion	<ul style="list-style-type: none"> • Changeable Message Signs • Media • Highway advisory Radio • Lane Control Signs 	Allows maximum use of available road-way capacity. Reduces energy and time losses due to congestion	Requires out of way travel Requires adequate capacity on alternative routes/freeways.	Diversion potential relates directly to the number of alternative freeway/surface routes available, their driver acceptance and ramp capacity.
Corridor Optimisation	<ul style="list-style-type: none"> • Changeable Message Signs • Special Signs • Highway Advisory Radio • Ramp Metering • Integration with Municipal TCS 	Allows maximum use of available corridor roadway capacity. Reduces energy and time losses due to congestion.	Requires some out of way travel Requires adequate capacity on alternative routes Platooning from signalised intersection to on ramp may require ramp metering	Relies on strong relationship between adjacent road signing and freeway traffic performance Relies on strong relation between authorities and inter municipal TCS
Express Collector Balancing	<ul style="list-style-type: none"> • Changeable Message Signs • Highway Advisory Radio • Special Signs • Lane Control Signs 	Allows maximum use of available freeway capacity; Reduces energy and time losses due to congestion.	May require more weaving movements.	
Motorist Advisory	<ul style="list-style-type: none"> • Changeable Message Sign • Highway Advisory Radio • Special Signs • Lane Control Signs • Media 	Advises freeway user of traffic conditions ahead enabling decision on what actions should be taken.	Distracts motorists from concentrating on driving	Message should not cause driver indecisiveness or sudden actions.
Lane Control	<ul style="list-style-type: none"> • Lane Control Signs 	Provides motorists with real time information regarding local traffic conditions. Reduces energy and time losses due to congestion. Improves Maintenance work.	Must be visible to drivers at regular and frequent intervals.	Can be advisory, regulatory or diversionary.

Table 3.2 Evaluation of Total Strategy Effectiveness in Reducing Problems: WB QEW

Peak/Off Peak	Ford Drive	Winston Churchill Blvd	Winston Churchill Blvd	Erin Mills Pkwy.	Erin Mills Pkwy	Mississauga Rd	Huronario St.	Huronario St	Cawthra Rd.	Dixie Rd.	Dixie Rd.	Highway 427.	Highway 427	Kipling Ave.	Kipling Ave.	Islington Ave.	Islington Ave.	Parklawn Rd.	Parklawn Rd.	Lakeshore Blvd	Lakeshore Blvd.	Humber River Rd.
Strategy																						
Monitoring Detection	3/3							9/3	12/3	6/6	6/6	6/6	6/3	6/3	3/3	3/3	3/3	6/3	6/3	6/3	6/3	
Freeway Entry Control	3/1							9/1	12/1	6/1	6/1	7/1	4/1	3/1	3/1	3/1	3/1	6/1	6/1	6/1	6/1	
Freeway Diversion	1/2							4/2	3/2	2/1	2/1	0/1	0/1	0/1	0/1							
Motorist Advisory	3/3							5/3	6/3	3/3	3/3	3/3	4/3	1/1	1/1	1/1	1/1	4/3	4/3	4/3	4/3	
Lane Control	3/3							5/3	6/3	3/3	3/3	3/3	4/3	1/1	1/1	1/1	1/1	4/3	4/3	4/3	4/3	

Effectiveness: 12.....Highest Effectiveness; 1.....Lowest Effectiveness.
Peak/Off Peak6/3

Source: McCormick Rankin, 1986.

- ▣ Traffic Responsive Metering
- ▣ Merge Control
- ▣ Integrated Ramp Control

3.7.1 Ramp Closure

Ramp closure is the simplest, the most effective and restrictive method of ramp control. The control procedure simply restricts all traffic from entering the freeway by manual or remote closing. It is the least popular method but in some cases the best control strategy and yields positive results. It is used when:

- ▣ there is totally inadequate storage space for ramp vehicles wanting to enter the freeway and would cause interference to surface streets,
- ▣ upstream mainline traffic is operating at capacity whereas alternate routes are well below capacity,
- ▣ sub-standard ramp geometrics with inadequate acceleration lanes, and restricted visibility, and
- ▣ close ramp spacings, where weaving vehicles are causing a serious congestion and shockwave propagation.

Ramp closure has been used successfully in Houston, Los Angeles, Osaka, Tokyo, Ottawa (permanent closure) and Toronto (Gardiner Expressway) [Wainwright, 1972; US DOT, 1976; Robinson & Doctor, 1989]. For maximum effectiveness, it is important to conduct origin destination studies to determine the traffic pattern, bottleneck sections, and what ramps have to be closed for maximum effectiveness.

3.7.2 Pretimed Metering

The first ramp metering system involved police officers to stop and release vehicles one by one using a stop watch and a ringer to determine the metering rate.

Pretimed metering now operates in a similar way as a pretimed intersection signal: traffic discharge is predetermined with the help of historical and projected data. Depending on volume variation, there can be more than one metering plan. These metering plans are based on historical traffic variations. The major components are noted below and shown in Fig 3-2:

- ▣ Traffic Signals,
- ▣ Vehicle detectors (optional),
- ▣ Advance Warning Flashers,
- ▣ Controller Cabinet.

Because metering plans are static, it is insensitive to upstream and or downstream traffic variation. Traffic signals and the controller cabinet are the essential components. The traffic signal controls metering rates, advanced warning signs are placed prior to the signals to warn motorists of metering and to give them an option to either use the freeway or the surface route. It is simple, easy to install, relatively inexpensive and very suitable for traffic patterns with predominantly recurring congestion and low frequency of incidents. Because metering plans are "off line", control algorithms could be formulated to any degree of sophistication. Although control strategies are not responsive to real time traffic conditions, metering rates are consistent, making ramp queues and waiting times predictable. Motorists are encouraged to divert as they are aware of the expected ramp

Fig 3.2 Pretimed Metering Source: Bluementritt et al 1981

Figure 3.3 Local Responsive Metering Source: Bluementritt et al, 1981.

conditions. Where the objective of metering is only to break platoons of vehicles discharged from a signalised intersection, substandard geometrics, or to reduce rear end collisions and chaos at ramp bottlenecks, its use is highly recommended [Blumentritt, 1981; Masher et al 1975; ITE handbook, 1992].

3.7.3 Traffic Responsive Metering

Metering rates are based on prevailing upstream and downstream traffic. The major components are

- ▣ traffic signals,
- ▣ vehicle detectors (upstream, downstream, check in, merge, and queue detectors),
- ▣ Controller Cabinet,
- ▣ Advance Warning Flasher.

On line traffic volumes, occupancy and speed parameters from both upstream and downstream detectors, are used to calculate real time metering rates based on local conditions, hence unanticipated traffic conditions are taken care of. Metering rates vary, depending on the prevailing traffic conditions. Passage detectors confirm the entry of a ramp vehicle into the merge section and merge detectors ensure that the lead vehicle has merged into the mainline. The main disadvantage of this method is that metering plans are determined under local traffic conditions.

3.7.4 Merge Control

The concept of merge control is to afford maximum utilisation of acceptable gaps in mainline lane that ramp vehicles would merge [Wainright, 1972; Pignataro, 1973]. It is intended to maximize the number of ramp vehicles merging safely onto the mainline, hence reduce disruptions to freeway traffic caused by ramp bottlenecks. It doesn't calculate metering rates. Ramp traffic is guided in a way that it arrives at ramp merge section when there is an acceptable gap. The system operates according to the following guidelines:

- ▣ detection of acceptable ramp merge gap,
- ▣ projection of the time this gap is made available at the point of entrance ramp, and
- ▣ releasing ramp vehicles in time to merge into these gaps.

Two types of merge control strategies exists:

- ▣ Gap acceptance merge control,
- ▣ Moving Merge.

3.7.4.1 Gap Acceptance Merging

Gap acceptance injection times a ramp vehicle's release such that there is an acceptable gap by the time it reaches the merge section. A number of vehicle detectors are placed in mainline lane 1 to measure vehicle speeds and gaps and compute the times prospective acceptable gaps would be available at merge point. The progressive gap speed is taken into account based on the distance from the ramp to the merge point, the vehicle is

released in time to merge into these gaps.

Its major elements are:

- traffic signals
- vehicle detectors (gap/speed, queue, check in, check out, merge and slow vehicle detectors).

Its advantages are:

- ideal for entrance ramps with substandard geometrics, and
- safer merging of vehicles, with significant reduction in rear end collisions.

The disadvantages are:

- high construction and maintenance costs,
- metering rates, are not much different from pretimed and local responsive types,
- violation rates are higher: wide variation in metering rates makes drivers either impatient or think signals are mal-functioning,
- it requires several vehicle detectors, and its control algorithms are complicated,
- ramp vehicles released to merge in a progressive acceptable gap are not continuously guided; sometimes mainline lane changes render these gaps occupied.

3.7.4.2 Moving Merge (Pacer System)

It is similar to gap acceptance system, except that ramp vehicles are guided continuously into the merge point. By keeping pace with a positive indication on a string of signals, or in a band of light, the vehicle is placed in a proper merging position for entering a safe available gap in the mainline lane 1 traffic. The speed of moving merge signal is

determined by the mainline lane 1 acceptable gap speed.

Major components of the system are the same as the gap acceptance type, except that there is a continuous set of signals at about 600mm intervals. Furthermore, because the paths of acceptable gaps are monitored continuously, more vehicle detectors are required. The pacer system was developed to alleviate situations where acceptable gaps were not available at merging point to on ramp released vehicles because of vehicles changing from other freeway lanes to occupy identified acceptable gaps.

3.7.5 Integrated Ramp Control

All ramp control strategies discussed so far operate independently on local upstream and or downstream traffic conditions. The concept of an integrated ramp control is to link a series of ramps together for proper co-ordination. The control concept is to maintain mainline volumes at or below a threshold capacity on a system wide basis, rather than on a section by section basis. It could be applied to pretimed, local responsive, or moving merge to achieve maximum utilisation of the entire study area.

In addition to the basic components of pretimed, traffic responsive, and merge control, it requires an element to monitor the whole system. A control centre is required to receive information from vehicle detectors and traffic controllers and send the best control decision logic to each ramp. The main elements include all the hardware and software of the local responsive mode, a central computer and a communications system, to collect data and override local control metering rates with optimal system-wide rates under some predetermined constraints, if necessary.

As the hardware components increase in number, control and communication equipments play a major role in the system. This is evident as more computer resource allocations are required for sophisticated signal control algorithms as well as increased occupancy, speed, volumes and other constraint computations. A central high speed, high memory computer where all data from each ramp are sent for analysis and decision making, through a two way communication system becomes a necessity [Blumentritt, 1981; Masher et al, 1975; US DOT, 1976; Pignataro, 1973].

3.8 Components of Freeway Traffic Management System:

A freeway traffic management system includes all or any combination of

- ▣ Freeway Ramp Metering
- ▣ Closed circuit TV surveillance
- ▣ Highway Advisory Radio
- ▣ Changeable Message signs (CMS)
- ▣ Emergency Vehicle Patrols
- ▣ Lane Control System

The selection of freeway traffic management components depends on the predominant type of congestion (recurring or non recurring), mainline and surface street configuration, traffic patterns, and its relative change with time.

On Highway 401 in Toronto for instance, where immediate response to incidents is very vital due to the heavy traffic in its urban section (over 300,000 everyday users), the system consists of [MTO, 1993]:

- ▣ 32 closed circuit tv cameras with pan and zoom capabilities,
- ▣ 850 vehicle detectors,
- ▣ changeable message signs,
- ▣ emergency MTO patrols and standby tow trucks,
- ▣ coaxial cable communications,
- ▣ traffic operations centre, and
- ▣ a traffic and road information system (TRIS) delivering up to minute information to drivers, emergency patrols, Ontario Provincial Police staff, radio and television stations, and fire stations, via telephones, faxes, and video signals.

In the case of Highway 401, ramp metering strategy was not necessary, because both express and collector lanes are uninterrupted controlled access facilities. Therefore emphasis are placed on detection and advisory to make maximum use of available corridor capacity.

A Proposed Freeway Traffic Management system for the Ottawa-Carleton Region is shown in Figure 3.4.

3.8.1 Control Hardware:

Vehicle Detectors

A number of vehicle detectors exist but the main ones are

- ▣ Ultrasonic detectors
- ▣ Magnetometers
- ▣ Inductive Loop Detectors

Figure 3-4: Proposed FTMS For The Ottawa-Carleton Region

Source: IBI Group, 1993

- ▣ Microwave detectors
- ▣ Video image processing

Most vehicle detectors are activated whenever a vehicle comes within its area of sensitivity through magnetic, sonic microwave or radar technologies. Inductive loop detectors are the most widely used vehicle detection devices. These consist of a flexible cable, turned a number of times and laid in rectangular or diamond shaped saw cut slots made in the wearing course. Magnetic fields are generated when a vehicle traverses its area of influence. These signals are transmitted to the traffic control centre via control cabinet substations. Detectors can be used to measure speed, volume, and occupancy with 5% error [Masher, DP et al, 1975; Michalopoulos et al 1993]. Speed, occupancy, and volume are measured on line. Loops are usually placed in each lane. Disruption of traffic during maintenance and false congestion detection alarms are their major disadvantages. Because of these set-backs, the Ministry of Transportation is actively testing microwave detectors, which would be mounted on overhead structures above or beside the freeway. Initial reports are very encouraging [MTO, 1993].

Inductance loops, as well as sonic, microwave, and magnetometer detectors are based on the principle of discrete detection. In many locations where there is a higher probability of non-recurring congestion on the freeway, a closed circuit television system is used to confirm congestion detected by loops [MTO, 1991; Ayland et. al., 1991; IBI Group, 1991].

Video image processing techniques, by virtue of continuous detection on an area-wide basis, are likely to take over from loops. Initial tests conducted in Minnesota

indicate 95% accuracy to detect volumes, speed, & occupancy measurements in all weather. About 80% of incidents could be detected by a single camera within a 3km range with only 0.65 false alarms per day! This means a single camera could replace a number of loops [Michalopoulos et al 1993]. The Municipality of Metropolitan Toronto is presently using the Autoscope system on the Gardiner-Lakeshore corridor, to manage traffic at locations where the use of loops is not possible, in its corridor rehabilitation.

3.8.1.1 Controllers

Controllers are devices that send messages to the traffic signals, instructing them when to turn green, amber, or red. The main types are

- Dial-and-cam unit, controlled by a timer
- Digital Controller
- Central Controller
- Microprocessor based controller

Dial-and-cam unit are electromechanical controllers with dials, cams, switches, synchronous motor, and timing gears. The dials are used to set different timing plans. Very limited timing plans can be delivered. This type of controller can be used for pretimed metering where metering rates are static.

Recent controllers include hardware and software systems with microprocessors, detector amplifiers and conflict monitors. The architecture is based on a standard hardware system into which a number of software programs with the help of menu driven assistance, can be easily entered to perform a variety of traffic control functions. Signal timing plans

could also be put into "*silicon chips*" and assembled in a microprocessor unit. Detector amplifiers found in controllers, pick up loop signal, and transmit them to regional or central controllers. The most widely used controllers in North America are the NEMA type 170 and NEMA II controllers.

3.8.1.2 Communications

Communication is essential when data are to be transferred from mainlines or ramps to a central control point. Communication links could be

- leased telephone lines,
- coaxial cable,
- fibre optic.

These communication links can be rented, leased or owned by the transportation agency. At present, the traffic management department of MTO is engaged in a pilot project, evaluating the cost effectiveness of sending compressed video images through telephone lines on Highway 401 and Sheppard Avenue interchange [MTO, 1993].

3.9 Metering Concepts

Pretimed, traffic responsive and integrated ramp controls operate on the concept of fluid flow through a tank. That is volume input should be equal to volume output, otherwise there is storage in the tank (see Figure 3-5a & b).

Figure 3-5 a & b Principles of Fluid Flow.

Figure 3-6 Input Output Flow Illustration.

Case 1: $Q_{in} > Q_{out}$ (Figure 3-5a)

$$Y_s = Q_{input} - Q_{out} \quad (3-1)$$

Case 2: $Q_{out} \Rightarrow Q_{in}$ (Figure 3-6b)

$$Y_s = Q_{input} - Q_{out} = 0 \quad (3-2)$$

To prevent storage, Q_{in} for case 1 will have to be decreased by an amount equal to Y_s , the rate of storage.

Applying the same concept to ramp control, mainline input plus on ramp volume should be equal or less than off ramp volume plus mainline output volume. This is expressed mathematically as:

$$(q_{input} + q_{on}) \leq (q_{out} + q_{off}) \dots \text{No Storage} \quad (3-3)$$

$$(q_{input} + q_{on}) > (q_{out} + q_{off}) \dots \text{Storage Condition} \quad (3-4)$$

$$q_a = q_{on} - (q_{on} + q_{input} - q_{out} - q_{off}) \quad (3-5)$$

Where

q_a Allowable on ramp vehicles.

q_{input} , q_{on} , q_{off} , & q_{out} are all illustrated in Figure 3-6

The amount of vehicles allowed at an entrance ramp is found by subtracting the amount of storage on mainline from the entrance ramp volume.

As stated earlier freeway ramp control is the most positive method of freeway traffic management system. This is explained by Newell's theory, as reported by Pretty, who discovered that where mainline traffic was congested, almost half of the downstream mainline lane 1 traffic are ramp traffic [Pretty, 1984]. If the freeway is already at a LOS E or worse, this extra 50% ramp traffic worsens the state of congestion and increases queue build ups upstream of the bottleneck. Ramp control is introduced to reduce the percentage of downstream mainline lane 1 traffic originating from on ramps. To account for minor fluctuations in traffic, mainline sections must be maintained slightly below its capacity (threshold capacity) for pretimed metering [Payne, 1973; Jacobson et. al., 1989]. A successful metering philosophy is the one that will introduce metering before the peak period to induce diversion in space, time and modal shifts and also to ensure a smooth transition from maximum metering rates to minimum rates. It should also be incorporated in new urban freeway construction, as is being planned for the proposed Highway 407 in Toronto [MTO, 1990 & 1992;].

3.10 Control Strategies

The principal control strategies are [ITE, 1992]

- Demand Capacity Control, and
- Occupancy Control

Figure 3.7: Volume Occupancy Plot For Calculation of Entrance Ramp Metering as Reported in ITE 1992 Handbook.

Figure 3.8 Flow, Speed, & Density Relations For Uninterrupted Flow.

Source: Highway Capacity Manual, 1985

Metering rates for demand capacity control depends on real time or historical upstream mainline traffic and downstream capacity. If the upstream traffic is more than the downstream capacity, then the permissible on ramp traffic is set equal to the difference between these mainline volumes on condition it doesn't violate the minimum and maximum metering rates, as well as maximum queue length for each ramp. Metering rates for occupancy control are based on on-line upstream or downstream detector occupancies. These percentage occupancies are converted into traffic volumes and permissive metering rates determined in a similar way as that for demand capacity. Figure 3-7 is a volume occupancy plot from Chicago Area Expressway. From the figure, as lane volume increases, its corresponding occupancy increases as well. This is logical because within facility capacity, vehicle occupancies are directly proportional to traffic volume. Without metering, lane capacity of about 33 vehicles per minute (1980 vph) occurs at an occupancy of about 19%. Any further increase in capacity results in erratic, unstable, stop and go, and forced flow conditions. If metering begins at an occupancy of about 14%, each lane can accommodate up to 36 vpm (2200 vph) at a steady flow rate.

Occupancy is a better indicator of freeway traffic conditions because measured volumes lower than mainline capacity could occur in forced flow as well as free flow conditions [Transportation Research Board, 1985; ITE Handbook, 1992]. Furthermore capacity of a road facility is not fixed but depends on prevailing roadway and traffic conditions. From flow, speed and density relations shown in Figure 3-6 for instance, flow rate V_1 occurs under two different flow conditions, illustrated by A & B.

This concludes the prologue of this thesis. The previous chapters, including this

one, have identified the current pitfalls in freeway ramp control as well as the current state of the art of metering. Subsequent chapters propose modifications and additions to existing methodologies to address these problems such as,

- co-ordinating ramp signals with alternate traffic signals,
- conducting detailed analysis of the impacts of metering on alternate routes, and
- introducing a methodology to determine metering rates, considering freeway traffic conditions as well as surface street traffic delays.

CHAPTER 4

RAMP CONTROL STRATEGIES AND TACTICS: CAPACITY ANALYSIS

This Chapter, and Chapter 5 present a methodology for evaluating ramp control strategies and tactics. Here, a brief background of the Queensway is presented, followed by the first step of the methodology: Capacity Analysis Module.

4.1 The Ottawa Queensway: Introduction and Need

Highway 417 is an east west uninterrupted controlled access facility. It is the only continuous east west link through the study area. It is a Provincial Freeway which serves both intra city and inter city traffic in the Ottawa-Carleton Region. The section of this freeway of interest to this research, namely the Ottawa Queensway, is defined by the Eagleson Road Interchange & Highway 417 and 17 Interchange.

Urban land use patterns, increasing growth in satellite cities, especially Orleans and Kanata, and the preservation of the green belt areas, have all precipitated increased

trips to and from the CBD during morning and afternoon peak periods. In Kanata for instance, its population is expected to increase from the present 42,000 to 100,000 in the early years of the next century [Ray, R 1994].

Highway 417, popularly known as the Queensway, was constructed in 1966, from Richmond to Montreal Rd for uninterrupted traffic flow [IBI Group, 1987]. Various expansion and rehabilitation works have been executed for more than a decade which have improved its traffic flow. Present and future conditions indicate that it is and will continue to experience both recurrent and non-recurrent congestion, characterised by erratic traffic flow, stop and go traffic, increased travel times, driver frustration and other undesirable attributes at peak periods.

With increasing traffic, decreasing budget allocations to transportation agencies, and increasing environmental awareness, it has become necessary to examine ways to efficiently manage traffic flow at minimum cost.

4.2 Capacity Analysis Module.

For this methodology to be useful and successful, the following functional requirements are necessary:

- The freeway traffic is operating at an undesirable LOS, manifested by low speeds, erratic stop and go operation, high densities, etc.
- There is an alternative route, operating below capacity, where some traffic could be diverted;
- The freeway on ramps have adequate ramp storage capabilities.

The first step of the methodology is a two stage process to investigate if the first two requirements are satisfied through:

- Freeway Capacity Analysis;
- Arterial Capacity Analysis;

Figure 4.1 is a flow chart to illustrate Capacity Analysis Module.

4.2.1 Capacity Analysis: Highway 417

The purpose of freeway capacity analysis is to identify problem areas on the Queensway. Analysis was undertaken on the basic freeway sections of the Queensway. Section by section mainline traffic data was not available; therefore traffic volumes on all sections of Highway 417 were estimated based on 1992 ramp counts and data from a mainline permanent count station located about 1.5 km west of Highway 7 and Highway 417 interchange in the months of July and August, 1992. The Ministry of Transportation of Ontario (MTO) provided the above noted information. Mainline hourly volumes of each section of the freeway were obtained by addition and subtraction of entrance and exit ramp volumes, for each hour. Tables 4.1 & 4.2 provide a summary of 1992 ramp counts from 06:00 to 10:00 and 14:00 to 18:00 by either loop detectors or pneumatic tube counters obtained from MTO.

Figure 4.2 indicates estimated mainline and ramp volumes for eastbound (EB) traffic during the a.m. and p.m. peak hours (hourly volumes). The 1992 screenline traffic volumes, obtained from the Regional Municipality of Ottawa-Carleton (RMOC) were compared with calculated volumes and these are also shown. Although all traffic volumes

Figure 4.1: Capacity Analysis Module

Table 4.1: 1992 Highway 417 Westbound Ramp Volumes (vph)

Ramp	IC	Date	6-7am	7-8am	8-9am	9-10am	2-3pm	3-4pm	4-5p	5-6pm
66 56	7/17	Jul 24 '92	167 7	303 4	305 9	405 7	601 13	731 12	945 14	868 13
61 16	Carp Rd	Aug 14 '92	148 16	254 49	260 36	199 44	280 58	316 79	403 93	384 94
36 26 61	Terry Fox	Jul 24 '92	7 35 128	11 76 221	17 54 219	19 62 157	26 115 310	17 130 363	32 198 555	19 234 473
36 26 61	Eagleson	Jul 24 '92	20 19 431	53 36 802	52 46 949	64 46 614	111 76 912	94 141 1193	161 203 1447	121 105 1351
36 26 61	Moodie Dr	July 24 '92	20 23 344	43 47 747	60 74 879	41 56 518	86 124 555	162 101 621	215 140 667	152 160 575
36 26 61	Richmond	July 24 '92	98 14 204	172 48 330	179 52 339	113 35 393	312 36 668	387 77 827	433 77 1056	458 50 1063
36 26 61	Pinecrest	July 24 '92	63 88 186	62 244 520	78 260 779	79 217 569	113 265 781	125 293 1068	189 410 1336	181 315 1136
16 63 62 88	Woodroffe	Jul 24 '92	278 227 72 13	325 388 139 57	276 416 209 71	340 322 198 43	502 471 342 5	574 659 319 20	482 814 342 48	403 754 370 51
16 61	Maitland	Aug 14 '92	144 217	367 508	476 511	402 413	528 585	656 698	848 771	723 670
66 67	Carling	Aug 14 '92	139 375	314 930	389 1080	347 798	529 961	712 997	838 1145	652 869
61 16	Parkdale	Aug 21 Aug 14	360 175	716 350	627 345	523 299	543 446	661 537	466 515	697 468
16	Rochester	Jul 24	182	240	285	196	310	311	552	376
16 61	Bronson Ave.	7/24/92	170 521	355 1056	462 1011	329 703	464 782	583 770	590 770	533 688
36	Lyon	July 24	111	322	424	388	667	793	1009	857

Table 4.1: (continued) 1992 Highway 417 Ramp volumes (vph) Westbound

Ramp	IC	Date	6-7	7-8	8-9	9-10	2-3	3-4	4-5	5-6
66 62	Metcalf	Jul 24	338 97	809 556	844 648	534 497	520 337	617 405	622 387	484 300
76	O'Connor	Jul 24	96	339	481	370	532	712	808	664
43 36 62 64	Nicholas	Jul 24 Jul 24 Jul 26* Jul 26*	158 153 720 128	384 286 1641 343	487 360 1377 270	316 293 1018 214	413 506 972 283	490 552 1092 282	585 628 1179 334	463 502 253 316
36 26 61	Riverside	Aug 28 Jul 24 Jul 24	347 215 581	696 531 882	761 762 708	538 650 508	661 620 531	851 781 591	833 810 658	630 613 612
36 26 31	St Laurent	Jul 24 Jul 24 Jul 24	321 118 265	636 225 561	682 349 452	503 375 378	722 451 491	836 537 429	797 549 432	755 448 363

Table 4.2: 1992 Highway 417 Eastbound Ramp Volumes (vph)

Ramp	IC	Date	6-7	7-8	8-9	9-10	2-3	3-4	4-5	5-6
55	7/417	Jul 24	490	648	416	311	369	413	470	395
51	Carp Rd	Jul 24	53	90	72	46	47	62	57	46
35		Jul 24	97	178	152	108	129	156	155	146
25		Jul 24	91	176	154	104	111	99	122	119
51	Terry Fox	"	44	148	111	109	97	113	145	123
35		"	3	15	21	12	18	10	12	14
25		"	267	495	417	251	208	258	256	210
51	Eagleson	"	80	167	155	69	89	98	79	94
35		"	382	647	664	415	359	492	580	480
25		"	446	759	685	444	401	428	469	487
51	Moodie	"	98	257	262	175	192	176	200	179
35		"	34	82	103	133	157	396	492	334
25		"	215	436	330	274	326	398	384	377
51	Richmond	Aug 14	74	140	152	182	219	262	299	288
35		"	158	271	226	203	372	348	339	365
25		"	267	721	632	343	376	416	403	341
51	Pinecrest	Aug 14	139	273	379	245	329	348	418	441
35		Aug 14	190	310	317	278	267	334	275	283
25		Jul 24	455	1033	954	600	518	615	599	551
77	Woodroffe	"	35	48	38	5	20	77	49	40
51			205	392	396	344	353	379	467	414
35			59	198	189	169	248	223	203	234
25			390	983	965	559	477	515	519	481
51	Maitland	"	225	537	713	396	343	416	408	376
15		"	283	623	799	584	547	632	678	559
57	Carling	"	267	562	572	454	504	525	508	480
55		Aug 14	316	826	1076	787	876	958	983	810
65		"	127	244	246	219	269	280	307	268
51	Parkdale	Jul 24	354	718	592	457	427	451	416	378
15			315	582	671	514	684	870	868	619
51	Rochester	Jul 24	219	580	663	353	343	400	462	318
51	Bronson	"	360	618	730	497	518	587	573	604
52	Kent	"	303	882	1046	607	437	484	516	416
75	Metcalfe	"	322	785	1058	903	1449	1753	2170	1681
35	Nicholas	"	606	846	747	693	1104	1399	1561	1228
45			277	431	483	380	536	653	640	574
54			80	245	295	178	201	255	245	240
52			246	640	672	539	531	566	600	572

Table 4.2: (contd) 1992 Highway 417 Eastbound Ramp Volumes (vph).

Ramp	IC	Date	6-7	7-8	8-9	9-10	2-3	3-4	4-5	5-6
51 35 25	Riverside	Jul 24	793 90 93	1468 162 225	1322 192 348	1069 172 203	1313 278 344	1469 472 561	1541 404 611	1165 306 434
51 35 25	St Laurent	"	524 68 57	1222 97 79	1254 114 125	1004 126 168	1291 298 185	1332 335 251	1346 426 316	1002 414 231

were not measured on the same day, due to relatively low variation in traffic during the weekdays, these estimates can be considered to be satisfactory. The next step was to determine the level of service of a.. the sections of the study section using the Highway Capacity Manual (1985) methodology, based on the following data and assumptions¹.

- 3.66m lanes
- 1.83m shoulder widths and lateral clearances. (On some sections of the freeway the clearance to the median barrier was less than 1.83m but this doesn't affect the capacity as indicated by HCM)
- 7% trucks and 1% buses (obtained from 1992 RMOC traffic characteristics).
- A peak hour factor of 0.9 was assumed, based on RMOC fifteen minute counts, which varies from 0.8 to 0.9.
- Freeway and ramp lanes were obtained from MTO aerial photographs and through visual observations.

¹ Source Highway Capacity Manual, 1985 page 3-10 to 3-17

Mainline Counts By Pneumatic Tube Detectors From RMDC or Loop Detectors From either RMDC or MTD

1559 Mainline Volumes Estimated From Ramp Counts

Figure 4.2 Highway 417 EB Traffic Volumes On A Typical Weekday in July/August 1992 From 07:00 to 08:00 Hours 8

Mainline Counts By Pneumatic Tube Detectors From RMDC or Loop Detectors From either RMDC or MTD

1225 Mainline Volumes Estimated From Ramp Counts

Figure 4.3 Highway 417 EB Traffic Volumes On A Typical Weekday in July/August 1992 From 16:00 to 17:00 Hours

- ▣ Drivers are regular users of the facility.

Figures 4.2 & 4.3 are eastbound am and pm mainline hourly volumes estimated from ramp counts. Hourly traffic volumes were converted into peak flow rates and the level of service (LOS) at all sections of the Queensway was estimated by the following equations:²

$$SF = \frac{\text{Mainline Volume}}{PHF} \quad (4-1)$$

To find the level of service, the effective v/c ratio of the section is computed as

$$\frac{V}{C} = \frac{SF}{C_j N f_w f_p f_{HV}} \quad (4-2)$$

where

SF Service flow rate,

c_j Capacity under ideal conditions (2000Pcphpl),

N Number of lanes in one direction,

f_w A factor for lane width and lateral clearance, and

f_{HV} A factor to adjust for the effect of heavy vehicles,

PHF Peak hour factor,

f_p A factor to adjust for the effect of driver population.

Figures 4.4 & 4.5 are graphical representation of the LOS for both EB & WB Queensway. Sections with a LOS D or worse are sections of the freeway which would be considered for metering.

² Source: Highway Capacity Manual page 3-19

Freeway Level of Service Highway 417, Eastbound 07:00 To 08:00

Estimated from 1992 MTO ramp counts.

Figure 4.4 Level of Service for am Peak EB Basic Freeway Sections

Freeway Level of Service Highway 417, Eastbound 16:00 To 17:00

Estimated from 1992 MTO ramp counts.

Figure 4.5 Level of Service of pm Peak EB Basic Freeway Section.

Freeway Level of Service Highway 417, Westbound 07:00 To 08:00

Estimated from 1992 MTO ramp counts.

Figure 4.6 Level of Service for WB Basic Freeway Sections, AM Peak

Freeway Level of Service Highway 417, Westbound 16:00 To 17:00

Freeway Sections

Estimated from 1992 MTO ramp counts.

Figure 4.7 Level of Service for WB Basic Freeway Sections, PM Peak

Existing peak period is directional: Eastbound traffic from Pinecrest to Kirkwood interchange, westbound traffic from Orleans to Vanier are very heavy in the morning, whereas opposing traffic is relatively free flowing. For maximum effectiveness of metering, it is essential to trace the path of all vehicles in the study section. Eastbound Highway 417 from Pinecrest to Carling was selected for further analysis because it was the only section for which origin and destination (OD) data were available.

4.3 Capacity Analysis on Carling Avenue

The purpose of arterial analysis is to find out:

- ▣ if there are candidate routes available for diversion, and
- ▣ the present state of peak period traffic on these routes.

It is only when a comparison of both arterial and freeway present traffic conditions are made that meaningful and realistic strategies can be adopted.

Fig. 4.8 is part of a regional map of the study section. Two streets were identified as alternate routes for vehicle diversion, namely Carling Ave and Baseline Rd. Carling Ave was chosen as an alternative route because it provided better entrance and exit links with the Queensway.

Fig 4.9 is a network diagram of the surface street. It is made up of links and nodes. The nodes represent signalized intersections and the links represent the street sections (not to scale). The EW Carling Ave from Pinecrest to Kirkwood, including all signalised crossing streets, are shown. Capacity analysis of the entire network was done with the help of a computer simulation package called TRAF-NETSIM. Before

Figure 4.8 Map of The Study Section.

describing network analysis, a brief introduction of this simulation package follows.

4.3.1 The TRAF-NETSIM Software

Traf-Netsim, formerly known as the Urban Traffic Control System 1 (UTCS-1), is a microscopic simulation model developed in the early seventies and has been used by many traffic and transportation research engineers. Its original purpose at that time was to evaluate the significance of the first and second generation computer-based urban traffic control strategies by careful study. Today, its accuracy has precipitated its use in a number of other purposes [Fertis, et. al., 1976].

Vehicles are modelled individually, based on car-following, queue discharge and lane switching decision rules. Interrupted traffic, unlike freeway traffic, involves a number of elements which affect traffic flow. These are traffic volume, bus volume and operations, parking manoeuvres, pedestrians, orderly movement of traffic at-grade intersections, traffic signs and signals and many more. Netsim simulates each vehicle's movement with respect to these elements.

When vehicles enter a network, each is allocated a lane. These lane assignments depend on whether the vehicle would turn right or left at subsequent intersections. Right turners are allocated to the right lane, most left turners are assigned to left lane and through vehicles are assigned to either through lanes or lanes with the least occupancy. The speed of the first vehicle on a link is dependent on vehicle characteristics (i.e., whether it is a bus, car, or a truck), the free flow speed of the subject link and the safe stopping distance. Speed and acceleration of second, third, fourth, fifth, and other vehicles depend

2

PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

PRECISIONSM RESOLUTION TARGETS

on their own characteristics, the behaviour of the preceding vehicle, the slope of the link, driver characteristics and traffic flow on immediate adjacent lanes. There are four classes of vehicles specified in the model, namely private auto, truck, bus, and car pool. Vehicles in the same class may not behave in the same way due to different driver characteristics. In order to account for this, each vehicle's speed is obtained by multiplying the specified free flow speed by a percentage. There are ten percentage values (known as random numbers 1 to 10) generated randomly by the model and each represents a driver characteristic. The default values, ranging from 75 to 125%, are assigned to a vehicle as it enters the network, randomly. This means that two or more vehicles with the same characteristics would be travelling at different free flow speeds. Gap acceptance for left turns, right turn on red (RTOR) queue discharge headways, start up and lost times, are all assigned randomly in a similar way, with transparent default values [Ferlis, 1976; US DOT 1993; Wong, 1990].

When a vehicle approaches an intersection, it either stops, slows down or continues its speed depending on whether it is controlled by a stop sign, yield sign, or a traffic signal. If it is a stop sign, it comes to a stop, looks for an acceptable gap, and then proceeds. At yield signs, it will not stop if there is an acceptable gap. At traffic signals, it continues at its speed on a green signal. When it turns amber, it stops only if its position is further than the safe stopping distance. Otherwise it proceeds through the intersection. When it changes from red to green, the vehicles move through the intersection under the following decision variables:

- position of the vehicle in the queue,

- discharge headway of each vehicle,
- vehicle characteristics, and
- driver characteristics.

Vehicles react to lane closures, obstruction or parked vehicles by changing lanes.

As the first vehicle moves towards an obstruction, it starts to look for an acceptable gap in the adjacent lane to make a lane change. If there are not any gaps within 1.5 meters from the obstruction, it stops and looks for one. As long as the first vehicle moves, following vehicles will follow until it stops, then the immediate vehicle after the leader will attempt to make a lane change. Most vehicles will always move back to their preferred lane after avoiding an obstruction. At signalised intersections, turning vehicles are modelled in detailed terms. During a green signal, left turn vehicles will make the turn on protected green or wait for acceptable gaps on permissive green. When the signal turns amber the left turn leader is allowed to make the turn. The model assigns a probability of 0.97, 0.77, & 0.37 for second, third, and fourth vehicle respectively to make the turn. Left turns are also affected by pedestrian volume. A right turn on red vehicle will wait for an acceptable gap [US DOT, 1993; Wong, 1990].

If downstream lanes are full, upstream vehicles will either wait for a downstream space or block the intersection.

TRAF-NETSIM also models pretimed, semi-actuated and fully-actuated signals, as well as transit buses.

4.3.2 Why TRAF-NETSIM ?

Netsim was chosen for simulation because:

- it has a number of features not available on TRANSYT 7-F, for instance it could model bus flow, and other transit operations accurately,
- it is a network simulation model,
- it has been tested and validated by a number of research organisations for different traffic patterns with encouraging results, which prompted the Federal Highway Administration (FHWA) to integrate it into the **TRAF** system, an integrated set of simulation models, which represent the traffic environment, and
- it is readily available at Carleton University.

4.3.3 Case Studies of NETSIM applications:

Chang, Cohen, et. al. used **NETSIM** simulation model to compare **MAXBAND**, **MAXBAND 86** and **TRANSYT-7F** timing plans [Chang et. al., 1988].

Rathi used **NETSIM** to evaluate and test various levels of traffic metering in a congested Central Business District (CBD) in a typical peak period [Rathi, 1991].

Sadegh et. al. compared **SIGOP III**, **PASSER II-84**, and **TRANSYT-7F** performance measures using **NETSIM** because it was assessed to be reliable for micro-simulation [Sadje et. al., 1987].

Yauch et. al. used **NETSIM** to evaluate the effects of drawbridge openings on adjacent signalised intersections, using the bus flow cards to model boat demand for bridge openings and concluded that:

"NETSIM proved to be a powerful tool in the evaluation of the effects of drawbridge openings on adjacent signalised intersections. The program, although data intensive, provided a realistic simulation of varying traffic conditions. Unlike the simulation mode of TRANSYT-7F, NETSIM permits the building of queues over one cycle length, thus clearly indicating when and where congestion is likely to occur" [Yauch et. al., 1988].

From these case studies and many others not reported here it appears that NETSIM is a very reliable network simulation model. Table 4.3 compares NETSIM with TRANSYT-7F, PASSER-II & SOAP computer programs.

Table 4.3: Comparison of TRAF-NETSIM with Other Traffic Engineering Programs

Program Features	Traf-Netsim	Transyt-7F	PasserII	Soap
Simulates isolated intersections	Yes	Yes	Yes	Yes
Simulates fixed time signals	Yes	Yes	Yes	Yes
Simulates actuated signals	Yes	No	No ³	No
Simulates stop and yield signs	Yes	Yes	No	No
Simulates network	Yes	Yes	No	No
Simulates intersection with different cycle lengths	Yes	No	No	No
Simulates saturated conditions	Yes	No	No	No
Simulates pedestrians	Yes	No	No	No
Simulate buses	Yes	Yes	No	No
Simulates lane closures	Yes	No	No	No
Simulates parking	Yes	No	No	No
Simulates any combination of the above features	Yes	No	No	No
Optimises timing plans	No	Yes	Yes	Yes

Source: TRAF User Reference Guide, Version 4.0 1993.

4.3.4 Data Sources

The following data were obtained or assumptions were made in order to conduct a detailed network analysis:

³ Approximation only

- The 1992 am peak period intersection volume counts for left turns (LT), right turns (RT), and straight through (ST), for all intersections from Richmond to Kirkwood were obtained from RMOC (see appendix for intersection counts),
- Distance between signals were scaled from RMOC 1:500 regional plans,
- Left turn pockets, right turn pockets and number of lanes in each link were all estimated on site.
- The topography of the area is relatively flat; road geometrics are also straight, therefore zero grades were assumed for all links.
- 60 Km/hr free flow speed,
- In order to simulate existing traffic behaviour during the peak period, signal timing plans were obtained from RMOC Signals Department,
- 2.2 seconds discharge headway,
- Light pedestrian traffic was assumed for all intersections,
- Bus volumes, bus routes, and bus paths were obtained from OC Transpo Planning Dept; bus stop locations were estimated on site. Bus paths were divided into five routes for simplicity.

Vehicle specifications used for the model are indicated in Table 4.4.

Fleet Component	Maximum Acceleration (km/h/sec)	Maximum Velocity (km/h)	Length (meters)
Private Auto	8.85	120	5.18
Truck	4.82	97	10.36
Bus	3.23	80	14.33

Source: TRAF User and Reference Guide Version 4.0

Figure 4.9 NetSim Network Configuration

4.3.5 Data Preparation for first NETSIM Run:

TRAF-NETSIM, is data hungry and requires experience and engineering judgement for some data input. There are a lot of options and default values which provide varying levels of accuracy. Netsim networks consist of links and nodes which represent one directional segments of streets and intersections respectively. Entry node numbers range from 8000 to 8999 whilst internal nodes range from 1 to 750. Figure 4.9 is a network diagram used to enter all **NETSIM** data.

Intersection count data from Pinecrest to Kirkwood avenue were converted into percentages and entered into each link. Left turn pocket, right turn pocket, upstream node number, downstream node number, number of lanes in all pockets & links were all specified on a link by link basis. Traffic volumes were specified for entry links 8100 to

Record Number ? 1	NETSIM Card Options Menu	
Title (00)	Phase Operations (47)	Amber Phase (144)
Identification (01)	Pedestrian Oper. (48)	Accept. Gaps (145)
Run Control (02)	Entry Link Volume (50)	Pedestrian Delay (146)
Time Spec. (03,04)	Source/Sink Volume (51)	Free-Flow Speed (147)
Output Options (05)	Load Factor (52)	Short Term Dur. (148)
Link Names (10)	Short Term Events (54)	Link Distributions (149)
Link Chars. (11)	Long Term Events (55)	Bus Dwell Times (150)
Turn Movements (21)	Parking Activity (56)	Delimiter (170)
Cond. Turn Move. (22)	Vehicle Type (58)	Bus Station (185)
Signal Control (35,36)	Environment. Table (60)	Dwell Time (186)
Approach Config. (43)	Link Aggreg. (90)	Bus Path (187)
Act. Signal Coord. (44)	Turn Calibration (140)	Bus Route (188)
Traffic Movements (45)	Spill back & Misc. (141)	Bus Flow (189)
Act. Cont. Det. (46)	Side Gaps (142,143)	Final (210)

F6-Explain, F7-Print, F8-Help, F9-Default, F10-Accept, ESC, CR, BS, HOME, END
Enter (A)dd, (M)odify, (D)elete, or ESC.

Figure 4.10 Netsim Input Main Screen⁴

⁴ Source: TRAF User Reference Guide Version 4

8321. Simulation runs were conducted for the peak fifteen minutes. NETSIM data input structure consists of cards and record numbers. Fig 4.10 is the main NETSIM input screen. Run control parameters were first entered in card 02. An initialization time of 30 mins, a simulation period of 840 seconds, an English data input and a metric data output were specified as illustrated by Figure 4.11. Bold Characters in Figures 4.11 through 4.16 indicate entered data.

Run Control Card (R) Card 02
Additional Case Study ? (Y)es or (N)o: N
Type Of Simulation (N)ETSIM analysis, (D)ata check only: N ⁵
Initialization Time (min): 30
Environmental Data Code: 0
Input Units (E)nglish, (M)etric: E
Output Units (S)ame as input, (E)nglish, (M)etric, (B)oth: M
Simulation Start Time (hrs & min): 07:45
Signal Transition Code:
Random Number Seed For Traffic Stream:
General Random Number Seed:

F6-Explain, F7-Print, F8-Help, F9-Default, F10-Accept, ESC, CR, BS, HOME, END

Enter data or ESC.

Figure 4.11 Input Data For Run Control Card⁶

⁵ All characters shown in bold letters are input data into NETSIM

⁶ Source: TRAF User Reference Guide, Version 4

Because only the peak 15 mins of simulation was undertaken, only one time period was selected for card numbers 03, & 04. Duration of simulation of 840 seconds instead of 900 seconds was used because a simulation period should be a multiple of cycle lengths. Existing timing plans of RMOC are based on 120 secs for the study section; hence 840 sec was used.

Time Period Specifications (R)		Card 03,04
Time Interval:120		
Time Period No. 1:840	Time Period No. 2:	
Time Period No. 3:	Time Period No. 4:	
Time Period No. 5:	Time Period No. 6:	
Time Period No. 7:	Time Period No. 8:	
Time Period No. 9:	Time Period No.10:	
Time Period No.11:	Time Period No.12:	
Time Period No.13:	Time Period No.14:	
Time Period No.15:	Time Period No.16:	
Time Period No.17:	Time Period No.18:	
Time Period No.19:		

F6-Explain, F7-Print, F8-Help, F9-Default, F10-Accept, ESC, CR, BS, HOME, END
Enter data or ESC.

Figure 4.12 Time Period Input Specifications For Netsim Run⁷

⁷ Source: TRAF User Reference Guide, Version 4

Upstream node, downstream node, link length, left and right turn pockets, pedestrian traffic, link grade and discharge headways were all entered for each link direction in separate record numbers, on card number 11.

Intersection turn movements for each node were converted into percentages and entered in card number 21, as shown in Figure 4.14

(N)ext or (P)rev: Urban Link Characteristics (R,O) Card 11		
Upstream Node No.:1		
Downstream Node Number.:2		1 3
Link Length (ft.):853	Fill in	3
Left Turn Pocket - Length:100	for link	\ /
No. of Lanes:1	(ij)	51 - j - 40
Right Turn Pocket - Length:		
No. of Lanes:		i
No. of Full Lanes:2		
Start-Up Lost Time (1/10):25		Channelization Code
Grade (%):0		Lane 1:0 Lane 4:
Discharge Headway (1/10 sec):22		Lane 2:0 Lane 5:
Discharge Code:		Lane 3: Lane 6:
Speed (mph):37		Lane 7:
Right Turn On Red ? (Y)es, or (N)o:N		
Pedestrian Traffic: (N)one, (L)ight, (M)edium, or (H)eavy:L		
	Alignment Data	
Upstream Lane No.:1	Downstream Lane No.:1	

F6-Explain, F7-Print, F8-Help, F9-Default, F10-Accept, ESC, CR, BS, HOME, END

Enter data or ESC.

Figure 4.13 Netsim Input Data For Link 1-2⁸

⁸ Source: TRAF User Reference Guide Version 4

(N)ext or (P)rev:		Turning Movement (R,O)			Card 21
Upstream Node	Downstream Node	Left Turn % or Vol.	Through % or Vol.	Right Turn % or Vol.	Diagonal % or Vol.
2	3	1	98	1	
3	4	3	97		

F6-Explain, F7-Print, F8-Help, F9-Default, F10-Accept, ESC, CR, BS, HOME, END

Enter data or ESC.

Figure 4.14 Typical Input Data For Turning Movement Traffic⁹

Existing intersection timing plans for the network were entered on a node by node basis. Figure 4.15 is a pretimed signal plan for Carling/Alpine intersection, which indicates a two phase signal with 10 secs eastbound leading green followed by 95 secs of eastbound and westbound permitted green. The second phase is 15 secs of permitted north and southbound green. Approaches 2&4 are eastbound and westbound Carling Ave and approaches 53 & 52 are south and northbound Alpine street. Protected, permitted, and combined phasing were all input in sequence for all signalised intersections. Signal transition green was left blank because it is not required for one timing plan.

⁹ Source: TRAF User and Reference Guide Version 4.0

(N)ext or (P)rev:		Sign or Pre-timed Signal Timing (R)			Card 35,36	
Node Number:3		X-coord:		Y-coord:	Transition Green:	
Upstream Node Of						
Offset:38		2	53	52	4	
Interval No.	Duration	Approach 1	Approach 2	Approach 3	Approach 4	Approach 5
1	10	1	2	2	2	
2	95	1	2	2	1	
3	15	2	1	1	2	
4						
5						
6						
7						
8						
9						
10						
11						
12						

F6-Explain, F7-Print, F8-Help, F9-Default, F10-Accept, ESC, CR, BS, HOME, END

Enter data or ESC.

Figure 4.15 Signal Timing Plan For Carling/Alpine Intersection.¹⁰

¹⁰ Based on 1994 RMOC signal timing plans

Entry link volumes were entered for all entry links. A sample entry link input screen is shown in Figure 4.16 for entry links 8106, 8107 and 8108.

(N)ext or (P)rev:		Entry Link Volumes (O)			Card 50	
Node	Upstream Node	Downstream Volume	Entry Trucks	Percent Car pools	Percent	
8106	52	104	6			
8107	55	254	6			
8108	54	3	6			

F6-Explain, F7-Print, F8-Help, F9-Default, F10-Accept, ESC, CR, BS, HOME, END

Enter data or ESC.

Figure 4.16 Sample Entry Link Volume Entered Into NETSIM¹¹

Bus paths were broken down into 5 routes:

- Route 1 was allocated for eastbound buses entering Pinecrest intersection (entry link 8100) and leaving to the Carlingwood mall (link)

¹¹ Based on 1992 Intersection Counts From RMOC

- Route 2 was allocated to eastbound buses entering at Carlingwood intersection and going through Kirkwood Ave.
- Route 3 was allocated to westbound buses entering at Churchill intersection and exiting at Maitland Ave.
- Route 4 buses entered westbound Carling at Maitland and made an exit at Carlingwood mall.
- Route 5 was designated for buses using westbound Carling at the Carlingwood intersection and went through Pinecrest.

Before NETSIM proceeds with simulation, it first checks all input data for completeness & consistency with the help of the pre-processor component of the model. When errors are detected it exits from the program indicating the errors which halted its execution.

Secondly it fills the network with vehicles until equilibrium is reached. This is known as "initialisation time". It is important for network to reach equilibrium before simulation begins. As a rule of thumb, initialisation time should be the time taken for a vehicle to traverse the study section [US DOT, 1993].

4.3.6 Output From NETSIM

The following measures of effectiveness are generated by NETSIM:

- ▣ throughput in vehicle kilometres,
- ▣ vehicle trips,

- ▣ travel time: move time, delay time and total time in vehicle-minutes and seconds per vehicle,
- ▣ stopped delay in seconds per vehicle,
- ▣ average link volumes in vehicles per hour,
- ▣ average speed in km/h or mph,
- ▣ signal phase failures, and
- ▣ queue length

For interrupted traffic flow, travel time is the most important performance measure. Network analysis was therefore based on:

- ▣ travel time (measured by the total time taken to traverse a given segment),
- ▣ intersection stopped delay, and
- ▣ arterial travel speed (measured by the total distance between the street segment divided by the total time taken to travel).

Figures 4.15 is a printout of some of the MOEs generated by NETSIM.

Intersection levels of service and arterial levels of service were all determined using tables 4.5 and 4.6 from the Highway Capacity Manual ¹².

¹² Source: Highway Capacity Manual 1985, Table 9-1 & 11-1

ALL EXISTING SUBNETWORKS REACHED EQUILIBRIUM
CUMULATIVE NETSIM STATISTICS AT TIME 7:14: 0

ELAPSED TIME IS 0:14: 0 (840 SECONDS), TIME PERIOD 1 ELAPSED TIME IS 840 SECONDS

LINK	VEHICLE KN	VEHICLE TRIPS	VEHICLE MINUTES		RATIO MOVE/ TOTAL	MINUTES / KM		SECONDS / VEHICLE		STOP		AVERAGE VALUES	
			MOVE TIME	DELAY TIME		TOTAL TIME	DELAY TIME	TOTAL TIME	DELAY TIME	TIME	TIME	STOP'S (%)	VOLUME VPH
(8100, 1)	56.42	216	57.1	123.3	0.32	3.20	2.19	49.9	34.1	23.0	22.0	67	925
(1, 2)	109.46	346	110.8	65.6	0.63	1.61	0.60	30.6	11.4	1.9	1.7	17	929
(2, 3)	65.93	366	66.8	44.9	0.60	1.69	0.68	18.3	7.4	1.7	1.5	10	1482
(3, 4)	52.06	400	52.7	89.2	0.37	2.73	1.71	21.3	13.4	4.3	3.5	37	1568
(4, 5)	65.91	412	66.7	49.2	0.58	1.76	0.75	16.9	7.2	1.2	1.5	8	1714
(5, 6)	177.81	404	180.1	218.3	0.45	2.24	1.23	59.2	32.4	14.7	12.3	54	1765
(6, 7)	238.26	397	241.3	222.4	0.52	1.95	0.93	70.1	33.6	9.0	6.5	43	1731
(7, 8)	15.40	77	15.6	44.3	0.26	3.89	2.88	46.7	34.5	29.5	26.1	57	1701
(8, 9)	44.41	74	45.0	19.8	0.69	1.46	0.45	52.5	16.1	4.5	7.1	21	329
(9, 10)	34.77	79	35.2	12.3	0.74	1.37	0.35	36.1	9.3	1.0	2.4	17	317
(10, 11)	13.28	83	13.4	10.2	0.57	1.78	0.77	17.1	7.4	2.6	4.0	18	338
(11, 12)	10.40	80	10.5	16.4	0.39	2.59	1.58	20.2	12.3	7.3	8.6	33	355
(12, 13)	13.31	74	13.5	6.0	0.69	1.46	0.45	15.8	4.9	1.1	1.1	6	342
(13, 14)	26.88	84	27.2	48.0	0.36	2.80	1.79	53.8	34.3	20.7	20.2	80	317
(14, 15)	29.42	114	29.8	10.8	0.73	1.38	0.37	21.4	5.7	0.0	0.0	0	359
(15, 16)	91.10	255	92.2	46.9	0.66	1.53	0.51	32.7	11.0	3.7	3.5	25	488
(16, 17)	102.07	249	103.4	83.4	0.55	1.83	0.82	45.0	20.1	10.7	10.5	32	1092
(17, 18)	73.94	247	74.9	165.6	0.31	3.25	2.24	58.4	40.2	20.1	17.8	62	1067
(18, 19)	174.03	232	176.2	242.4	0.42	2.41	1.39	108.3	62.7	31.1	25.0	68	1058
(19, 20)	36.53	122	37.0	58.1	0.39	2.60	1.59	46.8	28.6	18.4	17.7	54	994
(20, 21)	27.87	68	28.2	56.3	0.33	3.03	2.02	74.6	49.7	29.3	29.6	77	449
(21, 22)	27.70	77	28.0	15.4	0.65	1.57	0.56	31.9	12.0	3.0	4.0	77	522
(22, 23)	51.57	60	52.2	12.4	0.81	1.25	0.24	64.6	12.4	3.9	6.4	20	291
(23, 24)	7.97	57	8.1	2.8	0.74	1.36	0.35	11.4	2.9	0.2	0.2	5	329
(24, 25)	12.17	61	12.3	12.4	0.50	2.03	1.02	24.4	12.2	7.8	7.5	42	257
(25, 26)	21.34	73	21.6	30.9	0.41	2.46	1.45	45.0	26.5	21.6	21.1	87	261
(26, 27)	38.10	125	38.6	12.4	0.76	1.34	0.33	24.5	6.0	0.1	0.0	0	312
(27, 28)	16.15	53	16.4	45.0	0.27	3.80	2.79	69.5	51.0	39.5	38.4	94	299
(28, 29)	8.84	29	9.0	2.0	0.82	1.24	0.22	22.6	4.1	0.0	0.0	0	535
(29, 30)	6.10	20	6.2	5.7	0.52	1.94	0.93	35.5	17.0	11.6	11.6	90	231
(30, 31)	2.36	8	2.4	0.6	0.80	1.27	0.26	22.5	4.6	0.0	0.0	0	85
(31, 32)	0.91	3	0.9	0.2	0.79	1.28	0.26	23.3	4.8	3.3	3.3	66	34
(32, 33)	1.82	6	1.8	0.2	0.88	1.15	0.14	20.8	2.5	0.0	0.0	0	12
(33, 34)	2.74	9	2.8	4.8	0.36	2.78	1.76	50.8	32.3	29.7	29.7	88	12
(34, 35)	16.40	54	16.6	3.3	0.83	1.22	0.20	22.1	3.7	0.0	0.0	0	25
(35, 36)	3.35	11	3.4	3.5	0.49	2.07	1.06	37.8	19.3	15.5	15.5	100	38
(36, 37)	1.19	4	1.2	0.4	0.77	1.31	0.30	23.5	5.4	0.0	0.0	0	231
(37, 38)													42
(38, 39)													47
(39, 40)													17

* AVERAGE QUEUE AND STOP TIME ARE COMPUTED AS TOTAL QUEUE TIME OR TOTAL STOP TIME DIVIDED BY

Figure 4.17. Netsim Output For Existing Conditions

Table 4.5 Level of Service Criteria For Signalised Intersections

Level of Service	Stopped Delay per Vehicle (sec)
A	< 5.0
B	5.1 to 15.0
C	15.1 to 25.0
D	25.1 to 40.0
E	40.1 to 60.0
F	> 60.0

Source: Highway Capacity Manual 1985

Table 4.6 Level of Service for Arterial Class II

Level of Service	Average Travel Speed (Km/h)
A	> 48
B	> 39
C	> 29
D	> 23
E	> 16
F	< 16

Source: Highway Capacity Manual 1985

Except Richmond which operates at a LOS D, all eastbound intersection approaches are operating at a LOS C or better; Alpine, Croydon, Connaught, Lincoln Fields, Woodroffe South, Carlingwood, Iroquous and Broadview eastbound approach have an average stopped delay of 10 sec/veh or less (LOS B). This means traffic moves with little or no stops which is mainly due to very good progression (see Figure 4.18).

Despite efficient traffic progression, Pinecrest-Richmond, Woodroffe South-Fairlawn, & Clyde-Churchill links are travelling at lower speeds (18 km/hr or less see Figure 4.19). This is due to high left turns, ranging from 142 to 250 vehicles per hour (see appendix A) which interferes with through traffic. Apart from these links, all other eastbound approach traffic travel at a speed of 28km/hr or higher (LOS D or better).

Intersection Stopped Delay Eastbound Carling Ave

Based on 1992 RMOC intersection counts and existing timing plan

Figure 4.18 Level of Service for Eastbound Carling

Link Travel Speed Eastbound Carling

Based on 1992 RMOC intersection counts and existing timing plan

Figure 4.19 Link Travel Speed for Eastbound Carling.

Cumulative travel Time Existing Eastbound Carling

Based on 1992 RMOC intersection counts and existing timing plan

Figure 4.20 Cumulative Eastbound Travel Time.

Intersection Stopped Delay Westbound Carling Ave

Based on 1992 RMOC Intersection counts and existing timing plan

Figure 4.21 Average stopped Delays for Westbound Carling.

Cumulative eastbound travel time from Pinecrest to Kirkwood is 12.8 minutes (766.9 secs) (see Figure 4.20). For the westbound traffic, Woodroffe South & Broadview intersections experience an average stopped delay of 26.1 and 29.6 secs/veh respectively (LOS D), but all other intersections exhibit a stopped delay of 20.2sec/veh or better (LOS C or better, see Figure 4.21). Cumulative travel time on westbound Carling from Churchill to Richmond is 8.3 minutes. The entire eastbound traffic is travelling at an average speed of 25 km/h, which corresponds to a LOS D whereas the entire westbound traffic is travelling at an average speed of 32 km/h, a LOS C.

From both arterial and freeway capacity analyses it could be concluded that:

- whereas the eastbound Queensway between Pinecrest and Maitland is experiencing a combination of near capacity to forced flow conditions, Carling Avenue traffic in the same direction enjoys a LOS D or better,
- during the peak 15 mins, despite its 16 signalized intersections, cumulative travel time on Carling Avenue is less than its freeway counterpart, and
- therefore, this section of Carling Avenue can accommodate additional traffic from the freeway.

Cumulative Travel Time Westbound Carling Ave

Based on 1992 RMOC intersection counts and existing timing plan

Figure 4.22 Cumulative Travel Time, WB Carling.

CHAPTER 5

RAMP CONTROL STRATEGIES AND TACTICS:

CORRIDOR ANALYSIS

In Chapter 3 the main types of ramp metering and other freeway traffic management strategies were discussed. From Chapter 4 it was revealed that the mainline section just upstream of the bottleneck section was experiencing queue build ups and congestion whilst the section of Carling Avenue east of Pinecrest & west of Kirkwood Ave was operating at a level of service C, which makes it a suitable route for diversion. This Chapter deals with freeway and arterial control modules.

A successful metering strategy is one that ensures a balanced level of service for both the freeway and the alternative route. In other words, the total travel time in the corridor should be at least equal to or lower than the pre-metering conditions.

5.1 Entrance Ramp Control Feasibility

Although most of the case studies proved that ramp control is successful, it does not in any way suggest that ramp control is feasible on all freeways. Unlike traffic signals, freeway ramp control signals have no specific warrants for their installation. The Canadian Manual on traffic control devices (1976) makes no mention of guidelines or

warrants for their installation. The Manual on Uniform Traffic Control Devices [US DOT 1988] contains some guidelines, not warrants, for ramp control signals. Implementation of ramp control strategies should therefore be based on a detailed engineering analysis on the major factors affecting the feasibility of metering such as:

- bottleneck analysis,
- traffic diversion analysis (amount of diversion, candidate routes for diversion, and impacts of diversion),
- geometric design factors,
- enforcement analysis,
- public acceptance analysis,
- accident analysis,
- benefit-cost and cost-effectiveness analysis.

This part of the study covers bottleneck analysis, geometric design and traffic analysis of both the freeway and arterial. Bottleneck analysis involves identifying sections of the freeway where demand is higher than capacity and deciding what control measures would best improve flow. In Chapter 4, freeway bottlenecks were identified, based on demand/capacity criterion. Bottlenecks could also be due to other exogenous and endogenous factors such as horizontal and vertical alignment, the condition of the pavement, and effect of weather conditions such as rain, snow, etc

5.2 Metering Alternatives:

For on ramp metering, three options ought to be considered. These are

- ▣ Single entry metering,
- ▣ Platoon metering, and
- ▣ Two lane metering.

For single entry metering, only one vehicle is allowed per green. Vehicles merge smoothly with the mainline and violation rates are the lowest when this alternative is used.

In platoon metering, two or three vehicles are allowed to enter the freeway per cycle length. The main reason for platoon metering is to achieve higher ramp throughput, especially for a single lane metering.

Two lane metering requires at least two ramp lanes. Two vehicles, side by side, are allowed entry per cycle length.

Single entry metering was chosen for the Ottawa Queensway for the following reasons:

- ▣ ramp volumes of the section were not high enough to justify platoon or two lane metering, and
- ▣ it is better to use one alternative than to switch from one alternative to the other to avoid higher violation rates and driver confusion.

5.3 Metering rates

Minimum and maximum metering rates were determined, based on the minimum cycle length of a ramp signal to allow only a vehicle to be discharged per green (4 secs) and

the maximum tolerable waiting time for a motorist (20 secs). Experience from a number of locations indicate that when ramp signals remain on red for more than 20 secs, violation rates are higher because motorists are either impatient or they think of the signals as malfunctioning [Blumentritt et. al., 1981; US DOT 1976; ITE Handbook 1992].

$$\text{Min. Metering Rate} = \frac{3600\text{secs}}{20\text{secs}} = 180\text{vph} \quad (5-1)$$

$$\text{Max. Metering Rate} = \frac{3600\text{secs}}{4\text{secs}} = 900\text{vph} \quad (5-2)$$

Metering rates were based on demand/capacity criterion for two reasons :

- Pretimed strategies were used.
- Occupancy metering requires a historical relationship to be established between occupancy and mainline volume; there are no such data available in the Ottawa-Carleton region at present as there are no vehicle detectors installed in the Queensway to conduct such studies.

5.4 Control Strategies

In Chapter 4, the four major metering methods were discussed. Pretimed metering was chosen for Highway 417 using the following elimination procedure:

- Moving merge and gap acceptance control strategies are not cost-effective due to numerous vehicle detectors, and other hardware components required. Moreover, Pinecrest, Maitland, Woodroffe and Carling ramps

offer adequate ramp geometrics.

- ▣ Experience in many locations has revealed that there is a diminishing rate of return, with respect to cost, when control hardware and software for locally actuated and integrated centrally computer controlled metering are used [Wainright 1972; Jacobson et al 1989 Bluementritt et al 1981; Cocoran & Hickman, 1991]. It is better to install pretimed because it could easily be upgraded to either local responsive or integrated systems if or when necessary.

5.5 Experiment Set Up

The following procedure were adopted for corridor analysis. Existing freeway traffic (1992 condition) was simulated with the **FREFLO** simulation model, using the data as discussed in the previous chapter (see section 5.5.1 for a description of the **FREFLO** model). Ramp metering rates were optimised using linear programming. Out of the traffic diverted from the mainline, 20 queued on the ramps and the rest were diverted to Carling Ave. The diverted volumes were all assumed to use EB Carling Avenue to Kirkwood. Signal timing plans for the network were optimized to suit the increased volumes by using **PASSER II-87** computer software. The new timing plans were entered into **TRAF-NETSIM** to simulate these new volumes. Optimized metering rates were increased by very small amounts, repeatedly, until the threshold level of service for all intersection approaches were satisfied. For every metering increase, corresponding traffic on the alternate route, in this case Carling Ave, was adjusted accordingly. Performance measures

Figure 5.1 Flow Chart For FREFLO Runs

for both directions of the corridor were recorded. Each time traffic was increased on alternate routes, traffic signals were fine-tuned to optimize traffic flow. Figure 5.1 is a flow chart of the experiment set up and analysis. A brief introduction to **FREFLO** and **PASSER II-87** is provided in the following sections.

5.5.1 FREFLO Model

FREFLO is the successor to the **MACK** simulation model, developed by HJ Payne. It is a macro simulation model, which models flow on freeway networks on an aggregate basis. It is also a subset of **TRAF**, a family of simulation programs integrated by the **FHWA**.

The model is based on the principle of fluid flow conservation: the volume of a section of the freeway is the sum of input volume, the entrance ramp volume and the exit ramp volumes. During free flow conditions, the linear flow speed and density relations apply. As the freeway sections approach capacity, drivers tend to adjust their speed to the equilibrium speed density relationship and finally tend to slow down. The conservation equation is expressed as [Payne & Isaken 1973; Rathi et al 1987; Payne, 1979]

$$\rho_j^{n+1} = \rho_j^n + \frac{\Delta t}{L_j \Delta x_j} [q_{j-1}^n + f_j^{on,n} - q_j^n - f_j^{off,n}] \quad (5-3)$$

The normal speed-density relation:

$$q_{j+1}^{n+1} = \rho_j^n u_j^n \quad (5-4)$$

The dynamic speed-density relation:

$$u_j^{n+1} = u_j^n - \Delta t \left(u_j^n \left[\frac{u_j^n - u_{j-1}^n}{\Delta x_j} \right] + \frac{1}{T} \left[u_j^n - u_e(\rho_j^n) + v \frac{\rho_{j+1}^n - \rho_j^n}{\Delta x_j} \right] \right)$$

Where

n	Current point in time;
j	Current section;
l_j	Number of lanes along section j
Δx_j	Length of section j ;
ρ_j^n	Density of section j for the current point in time;
u_j^n	Space mean speed of section j ;
q_j^n	The rate at which vehicles pass section x_j for the current point in time, in vehicles per hour per lane per hour per lane
$f_j^{\text{on},n}$	On ramp volume
$f_j^{\text{off},n}$	Off ramp volume
T	The relaxation time coefficient
v	Anticipation coefficient
$(u_j^n - u_{j-1}^n)/\Delta x$	Represents convection that is upstream vehicles at section $j-1$ will travel at that speed as they enter section j
$u_j^n - u_e(\rho_j^n)$	Represent the tendency of drivers to adjust their speeds to the equilibrium speed density relationship
$(\rho_{j+1}^n - \rho_j^n)/\Delta x_j$	The effect of drivers changing their speed in anticipation of changing density.
Δt	Incremental time, t .

Freeway sections are divided into links and nodes where nodes are ramp junctions, or and links are distance between nodes. Before simulation begins, the preprocessor checks the input data for completeness and logic. If there are any errors, it terminates and exits the program indicating the errors that halted execution. Its initialisation period is done the same way as NETSIM [US DOT 1993].

FREFLO have been used by a number of agencies and transportation professionals including Demenstry, Bluementritt, et. al. A survey of freeway simulation models conducted by May, et. al. also revealed **FREFLO** & **FREQ** as the two main candidate models which could adequately simulate freeway traffic [May, Adolf et. al., 1982]. Furthermore, it has been recognised by the FHWA and incorporated into the TRAF family of simulation programs

5.5.2 Model Calibration:

Calibration of the model involves collecting field data on speed flow and density to select the type of speed-density relationships and determine its coefficients. For accurate results, data needs to be collected on a link-by-link basis.

Speed density calibration requires data collection on the study section continuously from free flow condition to forced flow condition. A sample size of at least 50 to 100 is required [McShane & Roess, 1990]. Depending on the quality of data, it could be more. The next step is to conduct a multiple regression analysis using the dependent variable

(speed) and the independent variable (density) in a polynomial form, similar to the expression established by the alternative forms of the model:

$$Speed = K_1 + K_2 \rho 10^2 + K_3 (\rho 10^{-2})^2 + K_4 (\rho 10^{-2})^3 \quad (5-6)$$

$$Speed = \frac{10^4}{K_1 + K_2 (\rho 10^2) + K_3 (\rho 10^{-2})^2 + K_4 (\rho 10^{-2})^3} \quad (5-7)$$

Where

$K_1, K_2, K_3, K_4,$ are constants to be established by multiple regression,

ρ density in km/hr or mph

Such calibration could be best undertaken in a freeway environment where continuous speed, volume and occupancy data are collected automatically. In the case of the Ottawa Queensway where there is only one permanent mainline volume counting station, 1.5 km west of highway 417/7 interchange, it is obvious that automatic freeway data generation is not possible. Given the enormous time, resources and effort for other methods, the video camera technique posed the next alternative, but camera positions to capture all the basic freeway sections of the study section were not attainable.

For the purposes of this study, speed-density equation (5-6) was assumed and the following default values, calibrated for a typical freeway, were used:¹

K_1 172;
 K_2 -597;
 K_3 896;
 K_4 -495;

¹ Source: TRAF User Reference Guide, Version 4 1993

Fig 5.2 Typical Speed Density Relations Used By FREFLO.

Making these assumptions means that traffic in the study section will behave in the same way as other freeways used to calibrate the model. The accuracy of the model relative to the conditions of the Queensway was tested by conducting travel time studies, reported in Chapter 7.

5.6 PASSER II-87

PASSER II-87 stands for **Progression Analysis & Signal System Evaluation Routine 1987** version. It is a computer model that maximizes arterial progression and at the same time, minimizes intersection delay. It calculates signal times, phase sequence, cycle lengths (within 10% of the maximum input cycle length) and signal offsets through a number of iterations and selects a timing plan which maximizes bandwidth progression and minimizes arterial travel delays on a two way or one way street [Texas Transportation Institute, 1988; Messer et. al., 1973].

The main input data required are traffic volumes (LT, ST, RT), cycle length, number of lanes, distance between signals, minimum phase time, phase sequence and progression speed. Based upon these intersection movements, possible signal phasing options are recommended by the model, for each intersection, for selection by the analyst. Ideal saturation flow of 1800 vphgpl is assumed as the default value, which could be increased or decreased to 2400 & 1500 vphgpl, respectively. Saturation flow rate is estimated by assuming an ideal saturation flow rate and multiplying by a number of factors to adjust for specific roadway and traffic conditions, as recommended by the HCM [Transportation Research Board, 1985].

Saturated flow rate is given by

$$S = S_0 N f_w f_{HV} f_g f_p f_{bb} f_a f_{RT} f_{LT} \quad (5-8)$$

Signal phases are based on demand/capacity concept as analyzed by Webster and reported in Messer et. al., (1973):

$$g_{im} = \frac{D_{im}}{S_{im}} C + L_{im} \quad (5-9)$$

The average stopped delay for random arrival is

$$d = 0.38 C \frac{[1 - g/C]^2}{[1 - g/C(X)]} + 173 X^2 [(X-1) + \sqrt{(X-1)^2 + (16X/c)}]$$

where

S	Saturated flow rate
S_0	Ideal saturation flow rate
N	Number of lanes
f_w	Adjustment factor for lane width
f_{HV}	Adjustment factor for heavy vehicles
f_g	Adjustment factor for grade
f_p	Adjustment factor for the presence of a parking lane
f_{bb}	Adjustment factor for the blocking effect of buses
f_a	Adjustment factor for area type
f_{RT}	Adjustment factor for right turns in the lane group
f_{LT}	Left turn adjustment factor
d	Average stopped delay in secs/veh
C	Cycle length in secs
g/C	the ratio of the green time over cycle length
X	v/c ratio for the lane group
c	Capacity of lane group

g_{im}	Green time for movement m at intersection i
D_{im}	Traffic demand for movement m at intersection i
S_{im}	Degree of saturation (the ratio of volume to capacity)
L_{im}	Lost time

It uses the National Electrical Manufacturers Association (NEMA) phase numbering system, illustrated in Figure 5.3. The NEMA convention assigns numbers to different traffic movement. Figure 5.4 shows the **PASSER II** volume input screen.

Existing arterial offsets could also be simulated to evaluate existing performance measures

Its analysis is based on HCM's methodology for signalised intersections.

Figure 5.5 presents a sample of **PASSER II-87** optimised timing plans.

5.6.1 Reasons for choosing PASSER II-87

This software is popular and user friendly. The following factors led to its choice:

- ▣ It is used by RMO for signal optimisation
- ▣ It is used by The Metropolitan Toronto for signal optimization
- ▣ **PASSER II** maximum bandwidth solution has been widely accepted and applied throughout many cities in USA and Canada.

5.7 Freeway Traffic Optimisation

5.7.1 Traffic Optimisation Using Linear Programming.

Integrated pretimed control strategy was used to establish initial metering rates using a linear programming technique. Linear programming is a method to optimize a system

Figure 5-3: NEMA Movement Designation²

Figure 5.4: Passer II-87 Volume Input Screen

²

Source: Passer 2-87 Micro computer users Guide, 1988

(BEST.SOLN)

TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

**** BEST SOLUTION CONTINUED.... NEMA PHASE DESIGNATION ****
 *** INT. 3 37.6 SEC OFFSET ART ST PHASE SEQ IS LT 5 LEADS (2+5)
 Alpine 34.2 % OFFSET CROSS ST PHASE SEQ IS LT 3 LEADS (3+8)

CONCURRENT PHASES	ARTERIAL STREET				CROSS STREET			
	2+5	2+6	1+6	TOTAL	3+8	4+8	4+7	TOTAL
PHASE TIME (SECS)	10.1	84.9	.0	95.0	.0	15.0	.0	15.0
PHASE TIME (%)	9.2	77.2	.0	86.4	.0	13.6	.0	13.6
----- MEASURES OF EFFECTIVENESS -----								
PHASE (NEMA)	5[4]	6	1[4]	2	3[4]	4	7[4]	8
PHASE DIRECTION	EBLTPM	WBTHRU	WBLTPM	EBTHRU	NBLTPM	SBTHRU	SBLTPM	NBTHRU
PHASE TIME (SEC)	10.1	84.9	.0	95.0	.0	15.1	.0	15.0
V/C-RATIO	.03	.22	.10	.55	.10	.06	.00	.47
LEVEL OF SERVICE	A	A	A	A	A	A		A
DELAY (SECS/VEH)	2.8	.0	4.5	2.3	43.8	44.9	.0	48.6
LEVEL OF SERVICE	A	A	A	A	D	D		D
QUEUE (VEH/LANE)	.0	.0	.0	1.4	.2	.1	.0	1.1
STOPS (STOPS/HR)	5.	694.	5.	503.	16.	9.	0.	75.
MINIMUM DELAY CYCLE								
53 SECS								

(BEST.SOLN)

TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

**** BEST SOLUTION CONTINUED.... NEMA PHASE DESIGNATION ****
 *** INT. 4 48.4 SEC OFFSET ART ST PHASE SEQ IS LT 5 LEADS (2+5)
 Croydon St 44.0 % OFFSET CROSS ST PHASE SEQ IS LT 7 LEADS (4+7)

CONCURRENT PHASES	ARTERIAL STREET				CROSS STREET			
	2+5	2+6	1+6	TOTAL	4+7	4+8	3+8	TOTAL
PHASE TIME (SECS)	18.1	68.8	.0	86.9	.0	23.1	.0	23.1
PHASE TIME (%)	16.5	62.5	.0	79.0	.0	21.0	.0	21.0
----- MEASURES OF EFFECTIVENESS -----								
PHASE (NEMA)	5[6]	6	1[1]	2	3[1]	4	7[4]	8
PHASE DIRECTION	EBLTPP	WBTHRU	WBLTPM	EBTHRU	NBLTPM	SBTHRU	SBLTPM	NBTHRU
PHASE TIME (SEC)	18.1	68.8	.0	86.9	.0	23.1	.0	23.1
V/C-RATIO	.09	.22	.00	.61	.00	.07	.71	.01
LEVEL OF SERVICE	A	A		B		A	C	A
DELAY (SECS/VEH)	3.8	.0	.0	2.4	.0	37.9	49.1	38.2
LEVEL OF SERVICE	A	A		A		D	D	D
QUEUE (VEH/LANE)	.1	.0	.0	1.4	.0	.3	3.0	.0
STOPS (STOPS/HR)	19.	545.	0.	476.	0.	25.	207.	3.
MINIMUM DELAY CYCLE								
54 SECS								

Figure 5.5: A Sample of Passer II-87 Best Timing Plans

under a number of constraints. The basic components of a linear programming problem are the objective function (what we want to optimize), and the constraints. It has been observed that most drivers regard travel time as the most important factor and perceive cost as secondary, because out of pocket differences between alternative routes is apparently not realised [Pretty, 1984]. Therefore travel time is the single most important measure of effectiveness. But the total travel time in a system, in vehicle-hours or minutes, could be minimised either by decreasing the number of vehicles on the facility or decreasing individual travel time. Traffic volume in a system therefore directly affects freeway travel time. For a fixed input facility, maximising the total traffic input, is equivalent to minimising the travel time. This is proved mathematically as follows:

Figure 5.6a illustrates traffic volume in a system plotted against time period t_1 to t_2 , whilst Figure 5.6b illustrates traffic on a freeway section. From Figures 5a & b, the following expressions are derived:

$$T = Vt \quad (5-11)$$

$$T = \int_{t_1}^{t_2} Vt \, dt \quad (5-12)$$

$$\text{But } V_t = V_0 + It - Ot \quad (5-13)$$

$$T = \int_{t_1}^{t_2} [V_0 + It - Ot] \, dt \quad (5-14)$$

Figure 5.6a Traffic Volume versus Travel Time

Figure 5.6b Input & Output Traffic.

$$T = V_0 (t_2 - t_1) + \int_{t_1}^{t_2} I t \, dt - \int_{t_1}^{t_2} O t \, dt \quad (5-15)$$

Putting k_1 for $V_0(t_2 - t_1)$; k_2 for $\int_{t_1}^{t_2} I t \, dt$;

$$\text{Total Travel Time} = k_1 + k_2 - \int_{t_1}^{t_2} O t \, dt \quad (5-16)$$

Where

- V Volume of traffic,
- V_0 Initial volume of traffic in the system
- V_t total number of vehicles in the system in a time t ,
- t time slice,
- I traffic input,
- O traffic output,
- T total travel time

Therefore at peak periods, given that only recurring congestion is present and traffic input is fixed, minimising the total travel time is equivalent to maximising traffic in the system. For this study, the objective function will be taken to maximize the total traffic input into the system. The main constraint is to maintain the traffic flow on all sections of the freeway below its capacity.

The concept of freeway ramp metering using mathematical programming is not new. It was first used by Wattleworth on the Congress Street Expressway in Chicago [Wattleworth, 1967]. Subsequent studies were done by Brewer et al, Ovaici et al, May etc [Brewer et. al., 1969; Ovaici et. al., 1975; May, 1974] utilising various objective functions such as "*maximize traffic*" on the freeway, "*minimise total travel time*",

"maximize the number of persons served", "minimise person-hrs", etc. Linear programming methodology as used in this research is similar to the one used by Wattleworth but the difference is that ramp queue lengths are replaced by the condition that all surface streets in the corridor should not operate beyond an established threshold average stopped delays. Two initial metering strategies have been developed for this research study; "Priority" and "Non Priority" strategies. Non priority formulation is similar to the methodology developed by Wattleworth. For the priority strategy, it is based on providing exclusive lanes for Higher Occupancy Vehicles (HOV).

5.7.2 Origin Destination Information

The pattern of network and freeway traffic is highly dependent on O.D. information and influences:

- the propensity of a motorist to choose a particular route in response to his/her perceived value of travel time, cost, inconvenience, comfort, and safety, when there are competing routes, and
- the effects of an on ramp control.

True corridor control strategies may be unsuccessful unless there is a complete understanding of underlying O.D. patterns.

5.7.3 Ramp Metering Optimisation (See Figure 5.7a).

$$\text{Maximise } X_j \qquad (5-17)$$

Figure 5.7a Non Priority Metering

Figure 5.7b Priority Metering.

Subject to

$$\sum_{j=1}^n P_{jk} X_j \leq C_k \quad k=1 \dots m \quad (5-18)$$

$$X_j \leq D_j \quad j=1 \dots n \quad (5-19)$$

$$X_j \geq 0 \quad (5-20)$$

$$M_{jmax} \geq X_j \geq M_{jmin} \quad (5-21)$$

$$X_j \leq R_j \quad (5-22)$$

Where

P_{jk}	=	Fraction of vehicles entering input j and going through k
X_j	=	Traffic volume at Input j
C_k	=	Capacity at section k
D_j	=	Traffic demand at j
R_j	=	Ramp capacity at j
M_{jmin} & M_{jmax}	=	Minimum and maximum metering rates at j

Equation 5-17 states the objective function, in this case, to maximize traffic input on all ramps. Equation 5-18 is a capacity constraint, which ensures that the total freeway traffic at critical sections does not exceed its threshold capacity. Equation 5-19 ensures that entrance ramp volumes are not greater than its ramp demand. Equation 5-20 ensures that

ramp and mainline traffic input cannot be negative. Equation 5-21 are constraints for maximum and minimum metering rates, and the purpose of equation 5-22 is to maintain input volumes below ramp capacity.

5.7.4 Priority Ramp Optimization.

In priority³ ramp optimization, some form of priority is given to classes of vehicles such as car pools, van pools, and buses. Priority optimization could be done by maximizing person throughput or person kilometres. The methodology developed here is to give priority to HOV through providing exclusive lanes on entrance ramps, to guarantee that priority vehicles experience no delays on ramps. The next section develops an optimized metering methodology, by taking priority vehicles into consideration.

5.7.4.1 Priority Metering With Exclusive Lanes (See Figure 5.7b)

$$\text{Maximise } \sum_{j=1}^n (X_{jp} + X_{jnp}) \quad (5-23)$$

Subject to

³ Priority optimization, as used here, refers to providing ramp by-pass lanes for transit and other Higher Occupancy Vehicle lanes

$$\sum_{j=1}^n (P_{jpk}X_{jp} + P_{jnpk}X_{jnp}) \leq C_k \quad (5-24)$$

For $k = 1 \dots m$

$$X_{jp} = D_{jp} \quad (5-25)$$

$$M_{jmax} \geq X_{jnp} \geq M_{jmin} \quad (5-26)$$

$$X_j \leq R_j \quad (5-27)$$

$$X_{jnp} \leq D_{jnp} \quad (5-28)$$

Where

- X_{jnp} Total number of non priority vehicles entering ramp j
- X_{jp} Total number of priority vehicles entering ramp j
- P_{jnpk} Fraction of non priority vehicles entering input j and going past section k
- P_{jpk} Fraction of priority vehicles entering input j and going past section k
- D_{jp} Demand of priority vehicles at input j
- M_{jmin} M_{jmin} and R_j are all defined earlier

By equating priority vehicle input to its demand, all priority vehicles are allowed to enter the freeway without ramp control. To ensure that no multi occupant vehicle which qualifies as a priority vehicle is delayed, ramp bypass lanes are entered into the linear programming model as pseudo ramps.

Equation 5-23 is the objective function: to maximize both priority and non priority vehicles on all ramps. A capacity constraint, to prevent the sum of the total freeway

vehicles from exceeding a specified capacity is enforced by Equation 5-24. Equation 5-25 ensures that all priority vehicles are allowed to enter the mainline (through ramp bypass lanes). Equation 5-26 has been explained earlier.

5.8 Application of Ramp Optimization to the Queensway.

In the preceding chapters the methodology was covered for estimating ramp and mainline volumes on both EB & WB sections of the freeway. At this stage, we will apply the concept of ramp optimisation to the Queensway. A non priority type of strategy is intended here.

The O.D.⁴ information was taken from a forecast based on MTO license plate surveys [IBI Group, 1987] and were converted into percentages as shown in table 5.1. Table 5.2 indicates volumes crossing various sections, (i.e. % volumes going through a particular section).

Where two or more entry ramps were present at an interchange, it was necessary to determine the O.D. paths of each ramp. For this work, it was assumed that the number of vehicles from ramp *i* to *j* was directly proportional to the aggregate traffic from interchange *I* to destination *J*.

⁴ In this thesis, Origin-Destination (O.D.) implies the on-ramp location (origin) and off ramp location (destination)

Table 5-1a O.D. Surveys for am Peak, Converted into Percentages

Origin	Destination (% vehicles)				
	Pinecrest	Woodroffe	Maitland	Carling	Beyond Carling
Mainline	6.22	7.41	8.4	6.87	71.1
Pinecrest		8.6	6.4	7.7	77.3
Woodroffe			7.1	8.4	84.5
Maitland				9.3	90.7

Source: IBI Group, 1987

Table 5-1b Percentage vehicles going past various ramps (am peak period).

Origin	% Vehicles going past			
	Pinecrest	Woodroffe	Maitland	Carling
Mainline	93.8	86.37	77.97	71.1
Pinecrest		91.4	85.0	77.3
Woodroffe			92.9	84.5
Maitland				90.7

The next stage of the process was to identify sections with potential bottlenecks. The critical sections of the bottlenecks are C_1 & C_2 . Our main constraints were to restrict traffic flow rate at the potential bottlenecks from exceeding a level of service D. The problem was formulated in the following manner:

$$\text{Maximise } X_1, X_2, X_3, X_4, X_5, X_6, X_7, X_8 \quad (5-29)$$

Subject to

$$0.711X_1 + 0.773X_2 + 0.773X_3 + 0.845X_4 + 0.845X_5 + 0.907X_6 + X_7 + X_8 \leq 6524$$

$$0.863X_1 + 0.914X_2 + 0.914X_3 + X_4 + X_5 \leq 6524$$

$$0.780X_1 + 0.85X_2 + 0.85X_3 + 0.929X_4 + 0.929X_5 + X_6 \leq 5262$$

$$X_1 = 4868$$

$$345 \geq X_2 \geq 190$$

$$800 \geq X_3 \geq 190$$

$$220 \geq X_4 \geq 190$$

$$800 \geq X_5 \geq 190$$

$$692 \geq X_6 \geq 190$$

$$800 \geq X_7 \geq 190$$

$$244 \geq X_8 \geq 190$$

$X_1, X_2 \dots X_8$ are traffic inputs into the mainline as illustrated in Figure 5.8. The coefficients of $X_1, X_2, \dots X_8$ in equation 5-30 are percentage traffic entering these inputs which proceed past c_1, c_2 or c_3 . The minimum and maximum metering rates were taken as 190 vph & 800⁵ vph (see Chapter 2). 345, 220, 692 and 244 are existing on ramp volumes (Figure 5.9a). The assumption here is that diverted vehicles will use Carling Ave

⁵ In this study 800 vph is used instead of 900 vph because experience indicates that it is not possible to achieve the latter

Threshold Capacity at Basic Freeway Sections.

Figure 5.8: Ottawa Queensway Study Section.

and will not enter any downstream ramps, hence existing ramp traffic is taken as maximum metering rates. The next step is to convert all the inequalities into equations by adding a slack variable. These equations were solved using a simplex method. The initial simplex tableau is shown in Table 5-2. The simplex method uses the Gauss Jordan Complete elimination procedure to solve a group of linear equations where the number of variables in a group of equations are more than the number of equations in the same group. It (simplex) is a systematic method for obtaining an optimal solution for a linear program. The last row represents the objective function, whereas the fifth row, through to the last but one row are the constraints. The solution is found by performing a number of iterations, such that none of the figures in the last row is negative [Gliksman, 1963]. The optimal solution was obtained by a computer program known as Linear Programming LP88 available at Carleton University.

Fig 5.9a shows existing ramp volumes and Figure 5.9b are the optimised metering rates obtained from linear programming.

The metering plan was entered into FREFLO simulation model to evaluate the effect of ramp metering on the freeway. Corresponding traffic due to metering, on Carling Avenue was also simulated on Carling Avenue using Traf Netsim micro simulation model.

5.8.1 FREFLO First Run:

Figure 5.10 shows freeway network configuration for FREFLO input data. Data entry involved:

- ▣ Setting up run control parameters such as initialisation time, duration of

Table 5-2 Initial Simplex Tableau

[illegible]

Figure 5.9a Existing Traffic

Figure 5.9b Optimised Metering Rates

simulation, time period, specifying input data in metric etc.

- ▣ Defining link distances, number of lanes, entrance and exit ramps, percentage exits, lane capacities, etc
- ▣ Entry link volume input for nodes 8400, 8402, 8403, 8405, 8406, 8408, 8410 and 8411 were all entered.
- ▣ A capacity of 1890 vphpl was used, based on capacity analysis performed in Chapter 4
- ▣ At exit ramps movement percentages were estimated as the ratio of exit volumes to the mainline volume just upstream of its node multiplied by 100;
- ▣ Traffic was simulated for the morning peak 15 min flow rate at time intervals of 60 seconds.
- ▣ Initialization time of 10 minutes was used.

Subsequent simulation runs were executed for Highway 417 and Carling Ave in the following manner:

- ▣ Metering rates for each interchange began with the "optimised rates"⁵ determined from linear programming,
- ▣ Corresponding ramp diversion was estimated by deducting metering rates from the total ramp demand,

⁵ "Optimised Metering Rates" in this thesis refers to metering rates determined by the Linear Programming Technique.

Figure 5.10: FREP/O Network Configuration

- ▣ If the optimised metering rates did not satisfy threshold level of service for each intersection approach, ramp metering rates were increased by 30 vph for each interchange and alternate route network traffic adjusted accordingly. The procedure was repeated a number of times until all intersection approach threshold LOS were satisfied,
- ▣ For each volume change on Carling Avenue, traffic signals were optimised with **PASSER II-87** as discussed before. Figure 5.8 is a flow chart illustrating revised signal plan methodology,
- ▣ For the purposes of this experiment, metering rates were increased until maximum allowable metering rates were achieved,
- ▣ Movement percentages for each node and for each **NETSIM** run were adjusted to maintain constant left and right turns, using a computer spreadsheet package known as **QUATTRO PRO** (except for nodes 40-2, 41-8 & 43-12 where right turn traffic was adjusted to reflect traffic on NB Richmond, NB Woodroffe, & NB Maitland).

Figure 5.11: Signal Optimisation Flow Chart

Table 5-3 shows metering strategies for each simulation run.

In the next chapter, simulation results and the experiment will be discussed.

Table 5-3: Freeway Simulation Runs

Simulation Run	Ramp Metering Rates						
	8402-101 ⁶	8403-102	8405-201	8406-202	8408-301	8410-401	8411-402
Fwy0a ⁷	345	1148	220	1092	692	826	244
Fwy1a ⁸	345	778	190	190	190	800	244
Fwy2a	345	808	195	215	220	800	244
Fwy3a	345	838	200	240	250	800	244
Fwy4a	345	868	205	265	280	800	244
Fwy5a	345	898	210	290	310	800	244
Fwy6a	345	900	215	315	340	800	244
Fwy7a	345	900	220	340	370	800	244
Fwy8a	345	900	220	370	400	800	244
Fwy9a	345	900	220	400	430	800	244
Fwy10a	345	900	220	430	460	800	244
Fwy11a	345	900	220	460	490	800	244
Fwy12a	345	900	220	490	520	800	244
Fwy13a	345	900	220	520	550	800	244
Fwy14a	345	900	220	550	580	800	244
Fwy15a	345	900	220	580	610	800	244
Fwy16a	345	900	220	610	640	800	244
fwy17a	345	900	220	640	670	800	244
fwy18a	345	900	220	670	692	800	244
fwy19a	345	900	220	700	692	800	244
fwy20a	345	900	220	730	692	800	244
fwy21a ⁹	345	900	220	760	692	800	244
fwy22a	345	900	220	790	692	800	244
fwy23a	345	900	220	810	692	800	244
Fwy24a	345	900	220	840	692	800	244
Fwy25a	345	900	220	870	692	800	244
Fwy26a	345	900	220	900	692	800	244

⁶ On Ramp Entry Links. See FREFLO link Configuration

⁷ Existing Traffic on Entrance Ramps

⁸ Optimised Metering Rates Based On Linear Programming.

⁹ "Best metering rates"

CHAPTER 6

SIMULATION RESULTS

6.1 Arterial Measures Of Effectiveness

Selected MOEs used for analysis can be found in appendix B. Link travel time, average stopped delays, and total travel on all links were some of the outputs generated directly from NETSIM. Cumulative travel time of the sub-network, for each direction was obtained by link aggregation.

$$C_{iu} = Fwy_{iu} + Art_{iu} + R_w \quad (7-1)$$

Where

C_{iu}	=	Corridor total travel time (veh-mins)
Fwy_{iu}	=	Total freeway travel time (veh-mins)
Art_{iu}	=	Total arterial travel time (veh-mins)
R_w	=	Ramp waiting time (veh-mins)

The term R_w is added to adjust for vehicles queued on an entrance ramp as a result of

metering. From a scaled 1:5000 drawing from IBI Group (1987) drawings, Pinecrest, Woodroffe, Maitland and Kirkwood Ramps were taken to be 225 m or more in length. Using an acceleration lane length of 100 m, and an average vehicle spacing of 6.10 m, a maximum ramp queue of 20 vehicles per ramp was obtained. The total ramp waiting time was obtained by the following expression:

$$R_w = V_q \times D_s$$

Where

V_q Number of queued vehicles

D_s Duration of simulation.

For convenience, graphs for Figures 6.1 through 6.9 included average metering rates of 430vph ("optimised metering rates"), 550 vph, 646 vph, 702 vph, and the existing traffic.

6.1.1 Eastbound Carling Avenue

Travel time on Carling Avenue increased from pre-metering condition of 12.8 minutes to 15.7 mins for "optimised metering rates", a 23% increase (Figure 6.1).

At "optimised metering" rates, Richmond, Clyde, & Kirkwood eastbound intersection approach delays, increased significantly, when compared to existing conditions. Eastbound Richmond, for instance increased from 31 sec/veh to 56.9 sec/veh, whilst Clyde and Kirkwood increased from 10 sec/veh to 57.8 sec/veh; and 25 to 42.6 sec/veh respectively (Figure 6.2). Apart from Clyde and Fairlawn intersections, where there was a pronounced change in link travel speed, all others did not show much change in speeds between existing and optimised metering rates (9 km/h to 7 km/h for Richmond, 31 to 26 km/h

Ramp Metering & Cumulative Travel Time Eastbound Carling Ave

Figure 6.1 Cumulative Travel Time Eastbound Carling

Ramp Metering & Intersection stopped Delay Eastbound Carling Ave

Based on 1992 RMOC intersection counts and existing timing plan

Figure 6.2 Eastbound Intersection Approach Stopped Delays

Ramp Metering & Link Travel Speed Eastbound Carling Ave

Based on 1992 RMOC Intersection counts and existing timing plan

Average Metering Rates
 Existing 430vph 550 vph 640vph 702 vph
Figure 6.3 Eastbound Link Travel Speeds

for Woodroffe South intersection, and 35 km/h to 27 km/h for Maitland Ave; (see Figure 6.3). This means that at "optimised metering rates", eastbound arterial progression was still efficient. The significant increase in eastbound stopped delays is mainly due to long delays of left turn traffic, at Richmond, Clyde and Kirkwood, caused by high left turn volumes, which interfere with their respective through traffic. As "optimised metering rates" were increased, these approach delays reduced substantially especially for Kirkwood, Clyde, Woodroffe and Richmond (Figure 6.2). Increased metering rates means lesser traffic diversion to Carling Ave and therefore reduced delays.

For the entire eastbound sub-network, the following observations were made when the existing conditions were compared to the "optimised metering rates" (Table 6.1):

- reduction in arterial speed from 25 km/h to 20 km/h,
- increased total travel time from 61 to 94 veh-hrs, a 54% increase,
- increased traffic throughput from 1556 vehicle-km to 2035 vehicle-km, a 31% increase,
- increased cumulative travel time from 12.8 mins to 15.7 mins, a 22% increase.

Table 6.1: Effect of "Optimised Metering Rates" on Eastbound Carling

	Travel Speed (km/h)	Total Travel Time (veh-hrs)	Total Travel (veh-km)	Cum. Travel Time (mins)
Existing Condition	25	61	1556	12.8
"Optimised Rates"	20	94	2035	15.7

6.1.2 Westbound Carling Ave.

Clyde, Woodroffe South, and Richmond are the only intersections with a significant increase in average stopped delays when optimised metering rates were used (17.7 to 33.2 sec/veh for Clyde, 26.1 to 44.9 sec/veh for Woodroffe South, and 20.2 to 38.1 sec/veh for Richmond; see Figure 6.4). All other intersections experienced 24 sec/veh or less in vehicle stopped delays.

The highest range in link travel speeds from 30 km/h to 21 km/h, occurred at Fairlawn intersection, otherwise all other travel speeds changed slightly (see Figure 6.5). Traffic throughput, measured in veh-km, for each link, also changed slightly, in the range of 5 veh km or less except for Maitland-Iroquois section, which changed from 48 to 39 km/h (see Figure 6.6).

For the entire westbound sub-network, the following was observed when existing traffic conditions were compared with optimised metering rates (Table 6.2):

- Vehicle throughput reduced from existing 322.3 veh-km to 314.3 veh-km, a 2% reduction.
- Travel time increased from 640.9 veh mins to 723.5 veh mins, a 13% change, and cumulative travel time increased from 8.3 mins to 9.4 mins.
- Arterial travel speed reduced from 32 km/h to 28 km/h (a reduction of less than 2%), corresponding to a LOS C.

Although ramp control strategies were only applied to the eastbound direction, westbound traffic was affected slightly because of green time reallocation to reflect all

Ramp Metering & Intersection Stopped Delay Westbound Carling Ave

Based on 1992 RMOC Intersection Counts

Average Metering Rates
 Existing 430vph 460vph 500vph 550vph

Figure 6.4 Westbound Intersection Stopped Delay

Ramp Metering & Link Travel Speed Westbound Carling Ave

Based on 1992 RMOC Intersection counts and existing timing plan

Figure 6.5: Westbound Link Travel Speed

Ramp Metering & Vehicle Throughput Westbound Carling Ave

Based on 1992 RMOC Intersection counts and existing timing plan

Metering Rates

Existing
 430vph
 550 vph
 646vph
 702 vph

Figure 6.3 Westbound Link Total Travel

approach demands (see Equation 5-9).

Table 6.2: Effect of "Optimised" Metering Rates on Westbound Carling

	Travel Speed (km/h)	Total Travel Time (veh-hrs)	Total Travel (veh-km)	Cum. Travel Time (mins)
Existing Condition	32	10.7	322	8.3
"Optimised Rates"	28	12.1	314	9.4

6.1.3 Southbound & Northbound Approaches:

Except for Churchill Ave which operated at a LOS E, all other SB approach stopped delays were at a LOS D or better, when the "optimised metering rates" were used (Figure 6.7). Southbound link travel speeds at Croydon, Clyde & Churchill, operated within a LOS E, but all other Link travel speeds were at a LOS B or better (Figure 6.8). For the Northbound approaches, all other intersections except Richmond and Maitland Avenue operated at a LOS D or better. Northbound Richmond experienced the worst intersection stopped delay (LOS F) under "optimised metering plans (Fig 6.9). On the average, North & South bound approaches are not significantly affected even at the lowest metering rates, except NB Richmond and SB Clyde.

6.2 Freeway Measures of Effectiveness

The benefits of ramp control had a very positive impact on the eastbound Queensway. At "optimised metering rates" the following was observed (Table 6.3):

- reduced cumulative travel time from 13.4 mins to 6.1 mins, a 54% reduction,
- reduced total travel time from 308 veh-hrs to 141 veh-hrs, a 54% reduction,
- reduced throughput on the freeway from 8680 to 8634 veh-km, (less than 1%).

Ramp Metering & Intersection Stopped Delays. Southbound Approaches

Based on 1992 RMOC Intersection Counts

Average Metering Rates

Existing
 430vph
 550 vph
 646vph
 702 vph

Figure 6.7: Southbound Intersection Approach Delay.

Ramp Metering & Link Travel Speed. Southbound Approaches

Based on 1992 RMOC Intersection Counts

Figure 6.8 Southbound Link Travel Speed.

Ramp Metering & Intersection Stopped Delay. Northbound Approaches

Based on 1992 RMOC Intersection Counts

Average Metering Rates

Existing
 430vph
 550 vph
 646vph
 702 vph

Figure 6.9 Northbound Intersection Stopped Delay.

Table 6.3: Effect of "Optimised Metering Rates" on Eastbound Queensway

	Travel Speed (km/h)	Total Travel Time (veh-hrs)	Total Travel (veh-km)	Cum. Travel Time (mins)
Existing Condition	29	308	8680	13.4
"Optimised Rates"	63	141	8634	6.1

6.3 Corridor Analysis:

Impacts of metering would be incomplete, unless its effects on the corridor are considered. Corridor MOEs were broken into three sub components:

- ▣ WB Corridor, comprising of WB Queensway from Kirkwood interchange to Pinecrest interchange and WB Carling Avenue from Churchill to Pinecrest street.
- ▣ EB corridor, consisting of EB Queensway From Pinecrest Interchange to Kirkwood interchange plus EB Carling Avenue from Pinecrest Street to Kirkwood Avenue.
- ▣ Combined corridor MOEs consisting of the sum of EB and WB corridors.

6.3.1 Eastbound Corridor:

Existing conditions offered the lowest traffic throughput in the eastbound corridor of 10236 veh-km. At "optimised metering rates", throughput increased to 10669 veh-km, an increase of 4% (Figure 6.10).

Corridor travel times are highest at pre metering conditions of 369.5 veh-hrs; optimised metering rates (430vph) reduced the total travel time to 235.5 veh-hrs, a

Ramp Metering & Corridor Total Travel Eastbound Corridor

Figure 6.10: Eastbound Corridor Total Travel

Ramp Metering & Corridor Total Travel Time Eastbound Corridor

Figure 6.11: Eastbound Corridor Total Travel Time

savings of 129 veh-hrs (35% reduction). See Figure 6.11. The reason for this significant reduction is that, even though arterial total travel time increased, the freeway carries 82% of the total traffic. Therefore, the net effect was reduced corridor travel time. As metering rates increased from the optimised metering plans, its corresponding corridor total travel time increased.

6.3.2 WB Corridor:

Westbound corridor traffic change was negligible. The following observations were revealed when "optimised metering rates" were compared with pre metering conditions:

- reduction of traffic throughput of only 8 veh-km (less than 1% change). See Figure 6.12,
- A change of 1.4 veh-hr (less than 3% change see Figure 6.13).

These findings indicate that the effect of eastbound mainline on the westbound traffic is very minimal. Furthermore the change in performance measures were very small. Therefore, its impacts on the westbound corridor is negligible.

6.3.3 Combined Corridor

Comparison between pre-metering and metering conditions also revealed that corridor traffic behaved in a manner similar to the eastbound corridor. That is,

- vehicle throughput increased from 14685 veh-km to 15110 veh-km, when optimised metering plans were introduced (see Figure 6.14), and

Ramp Metering & Corridor Total Travel Westbound Corridor

Figure 6.12 Total Travel For Westbound Corridor

Ramp Metering & Corridor Total Travel Time Westbound Corridor

Figure 6.13 Total Travel Time For Westbound Corridor

Ramp Metering & Combined Corridor Total Travel

Based on 1992 RMOC Intersection counts

Figure 6.14 Corridor Total Travel.

Ramp Metering & Combined Corridor Total Travel Time

Based on 1992 RMOC Intersection counts

Figure 6.15 Corridor Total Travel Time

- total travel times, reduced from existing 421.7 veh-hrs to 289 veh-hrs, when optimised metering rates were introduced (Figure 6.15).

6.4 Disadvantages of "Optimised Metering Rates"

"Optimised metering rates" reduced corridor travel time, and increased total traffic throughput in the corridor, but eastbound Richmond, and Clyde approaches operated in the upper range of LOS E, manifested by longer stopped delays. Intersections such as Clyde, Churchill, Richmond and Maitland experienced very high stopped delays.

Although "Optimized metering" plans produced minimum corridor total travel time, it was achieved at the expense of severe delays at a limited number of locations, especially at eastbound and southbound Clyde, northbound Maitland and eastbound Richmond approaches.

6.5 Suggested Alternative Method

To avoid severe delays on all surface street approaches, threshold delays are introduced at all intersection approaches. The establishment of these threshold values should complement other activities like ride share, HOV lanes, etc in the corridor. The best metering rates from this methodology were formulated to respond to the disadvantages mentioned in section 6.4 by using the following criteria:

- minimize corridor total travel time, and
- all intersection approach delays should not be greater than its threshold value.

Establishing threshold delays depends on a number of factors including:

- the objectives and policy of the transportation agency under its jurisdiction,
- the acceptable level of congestion in the region, and
- the severity of congestion on both freeway and the adjacent surface street network.

For this study, the threshold average stopped delays for each approach is taken as 45 sec/veh which corresponds to an upper range of LOS E. The best average metering rate which satisfies these two conditions occurred at a metering rate of 674 vph.

6.5.1 Comparison of the "Best" with "Optimised" Rates.

The following observations were made when the best metering rates were compared with the optimised rates (Table 6.4 & 6.5);

- Reduced cumulative travel time from 15.7 mins (optimised) to 13.2 mins for eastbound Carling but increased eastbound freeway travel time from 6.1 mins (optimised) to 9.3 mins (Figure 6.16)
- Increased traffic throughput in the corridor from 15110 veh-km to 15408 veh-km (Figure 6.17),
- Increased total travel time in the corridor from 289 to 352 veh-hrs (Figure 6.18),
- The average stopped delays in all links were less than 26 secs/veh (LOS C), except eastbound Richmond, which experienced 44.8 secs/veh.

Even though the best metering rates increased corridor total travel time, as well as

Table 6.4 Comparison of Optimized & Best Metering Rates

	Existing	Optimized	Best
Cum. travel time, EB Carling (mins)	12.8	15.7	13.2
Cum. travel time, WB Carling (mins)	8.3	9.4	8.7
Cum. travel time, EB Queensway (mins)	13.4	6.1	9.3
Traffic throughput, EB Carling (veh-km)	1556	2036	1927
Traffic throughput, WB Carling (veh-km)	322	314	312.9
Traffic throughput, EB Queensway (veh-km)	8680	8634	9042
Traffic throughput, EB Corridor (veh-km)	10236	10669	10969
Traffic throughput, WB Corridor (veh-km)	4449	4441	4439
Traffic throughput, Combined Corridor (veh-km)	14685	15110	15409
Total Travel Time, EB Carling (veh-hrs)	61.1	94.4	74.6
Total Travel Time, WB Carling (veh-hrs)	10.7	12.1	10.9
Total Travel Time, EB Queensway (veh-hrs)	308.4	141.1	225
Total Travel Time, EB Corridor (veh-hrs)	370	236	300
Total Travel Time, WB Corridor (veh-hrs)	52.1	53.5	52.4
Total Travel Time, Combined Corridor (veh-hrs)	422	289	352

Table 6.5 EB MOE For The Best (NV21A) & Optimized (NV1A) Metering Rates

NV21A						
LINK	VEH KM	VEH MIN	11 (SECS)	DEL (S/V)	SPEED	CUM. TT (MIN)
RICH	69.19	598.00	158.10	44.80	7.00	2.64
ALPINE	116.28	202.10	32.90	1.40	35.00	3.18
CROY	68.81	119.10	18.10	1.90	35.00	3.50
CONN	53.62	152.80	22.30	4.20	21.00	3.87
LF	67.51	117.80	16.70	1.90	34.00	4.15
EDGE	188.82	368.40	51.50	8.80	31.00	5.00
W. S	254.47	549.00	77.70	9.90	28.00	6.30
FAIRLAWN	70.98	100.80	11.00	0.90	42.00	6.58
CAL. SC	63.30	81.30	16.50	1.00	44.00	6.86
IROQUOIS	45.32	68.30	12.80	1.80	40.00	7.07
MAIT	280.26	466.60	85.90	14.60	36.00	8.50
BROADVIEW	135.96	201.20	31.80	2.60	41.00	9.03
CLYDE	151.67	411.20	66.10	20.70	22.00	10.14
CHURCHILL	102.43	195.10	69.30	22.70	16.00	11.30
KIRK	258.79	638.90	111.10	25.90	24.00	13.15
TOTAL	1927.41	4476.20	789.00			
NV1A.OPT						
LINK	VEH KM	VEH MIN	11 (SECS)	DEL (S/V)	SPEED	CUM TT MIN
RICH	67.67	617.00	166.80	56.30	7.00	2.78
ALPINE	122.75	203.50	31.60	1.60	36.00	3.31
CROY	73.86	126.00	18.40	1.90	35.00	3.61
CONN	57.01	185.00	25.30	5.10	19.00	4.04
LF	72.47	136.60	19.10	1.90	32.00	4.34
EDGE	191.02	374.50	51.80	9.20	31.00	5.20
W. S	249.66	570.90	82.30	19.70	26.00	6.57
FAIRLAWN	70.80	103.90	11.70	0.70	40.60	6.87
CAL. SC	63.33	90.40	11.10	1.30	42.10	7.15
IROQUOIS	44.62	62.80	11.80	1.10	42.60	7.35
MAIT	302.62	663.10	113.60	30.20	27.40	9.23
BROADVIEW	173.12	214.50	38.80	3.60	33.00	9.88
CLYDE	168.48	964.90	140.80	57.80	11.00	12.23
CHURCHILL	114.13	365.50	57.60	16.80	19.00	13.19
KIRK	264.04	888.30	151.10	42.60	13.00	15.71
TOTAL	2035.58	5666.70	942.50			

Comparison of Metering Rates Eastbound Corridor

Figure 6.16 Cumulative Travel Time Comparison

Comparison of Metering Rates Combined Corridor Total Travel

Figure 6.17 Comparison of Total Travel.

Comparison of Metering Rates Combined Corridor Total Travel Time

Figure 6.18 Total Travel Time Comparison.

increased cumulative travel time on the freeways, (when compared to the "optimised rates,) traffic throughput increased, the average stopped delays at all intersection approach are monitored continuously and therefore maintained at or below threshold levels. Furthermore, cumulative travel time for the eastbound Carling reduced.

6.5.2 Limitations.

This methodology will likely overestimate all the MO. on the alternate route, hence overestimate the metering rates because traffic assignment was based on the assumption that all traffic would use Carling Ave. This traffic assignment technique is known as "all or nothing" assignment. In real life, northbound Richmond may not experience delays and travel times to this degree and other link volumes on eastbound Carling may not carry these traffic volumes for the following reasons:

- A realistic random variation of a motorist's route choice was ignored, and the effect of capacity restraint was not considered. Some trip makers, for instance would use Baseline Rd instead of Carling Avenue. Random variation of a motorist's route selection is explained by the fact that given two or more competitive routes, the user may not select the best route because the driver may either not be aware of the attractiveness of the non-chosen route or have other reasons for such a selection.
- When a motorist recognises this everyday ramp control, a different time may be chosen or even a change in mode may occur.
- Even if all diverted vehicles used Carling Avenue, they would not all enter

through Pinecrest, Maitland and Carling Avenue because not all these vehicles would proceed to these ramps before diverting.

- The other assumption that all diverted vehicles would use Carling at least beyond Kirkwood may not be valid. Metering would induce more short trips than long ones to divert, therefore not all diverted trips would travel to Kirkwood.
- The methodology is not responsive to the changing freeway O.D. patterns. At pre-metering conditions, the O.D. data is based on manifest demand. When metering strategies are introduced, reduction in freeway travel times may cause some latent demand, not accounted for, to use the freeway. As the AVI & AVC technologies advance and the traffic population of equipped vehicle probes increase, the static P_{jk} s (equation 5-12) could be updated to reflect the changing O.D patterns. Before an O.D update, the reliability of these P_{jk} s (which depends on the number of equipped vehicle probes using the freeway), could be determined by conditional probabilities, using Bayes Rule.

Whether vehicles would divert or not also depends on where these vehicles originated from and where they are going. If most vehicles are travelling to longer destinations, they may decide to use the freeway despite the increased ramp delays. The need to conduct an origin and destination survey is therefore very essential.

These limitations do not in anyway undermine the usefulness of the results. It simply indicates that given O.D. data in the area, the benefits of the metering rates would be further refined to maximize its benefits. The intent of this research was to present a

limited and realistic evaluation of benefits that such strategies could produce. Due to these limitations, the results indicate a conservative estimate of average metering rates, which means that the benefits due to metering would probably be higher.

Chapter 7

VERIFICATION OF SIMULATION MODELS

Travel time runs were carried out on the Ottawa Queensway and Carling Avenue. This chapter presents the procedure used for these studies and compares the field observations with simulated results.

7.1 Field Studies

The purpose of the field studies was to assess the accuracy of some of the MOEs generated by NETSIM and FREFLO. Although these simulation models have been well validated, it is important to conduct field tests to ascertain their validity on site-specific conditions.

Travel time studies were conducted on both the Ottawa Queensway and Carling Avenue during peak period on the dates shown in Tables 7-1 & 7-2.

Table 7-1 Travel Time Runs For Highway 417.

Date	Duration	Road Condition	Comments
Feb 7 th 1994	7:00 - 8:30	Bare	Four runs each, EB & WB
Feb 8 th 1994	7:00 - 8:30	Bare	Four runs each, EB & WB
Feb 9 th 1994	7:00 - 8:30	Snow Covered	One run each, EB and WB
Feb 10 th 1994	7:00 - 8:30	Bare	Four runs each, EB & WB
Feb 11 th 1994	7:00 - 8:30	Bare	Four runs each, EB & WB
May 30 th 1994	7:00 - 8:30	Bare	Four runs each, EB & WB
June 2 nd 1994	7:00 - 8:30	Bare	Four runs each, EB & WB

Table 7-2 Travel Time Runs For Carling Avenue.

Date	Duration	Road Condition	Comments
Feb 15 th 1994	7:00 - 8:30	Bare	Four runs each EB & WB
Feb 16 th 1994	7:00 - 8:30	Bare	Four runs each EB & WB
Feb 17 th 1994	7:00 - 8:30	Bare	Four runs each EB & WB
Feb 18 th 1994	7:00 - 8:30	Bare	Four runs each EB & WB
Feb 21 st 1994	7:00 - 8:30	Bare	Four runs each EB & WB
May 26 th 1994	7:00 - 8:30	Bare	Four runs each, EB & WB

A total of four runs per day, were performed for each direction of the Queensway, from Pinecrest interchange to Kirkwood interchange. On Carling Avenue, field studies were undertaken from Pinecrest/Carling intersection to Kirkwood/Carling intersection. Road surfaces were bare except on February 9th, when it was covered with snow and ice.

7.2 Procedure

The study required the use of:

- ▣ a test car,
- ▣ a stop watch, and
- ▣ an observer.

A 1988 VW fox, which was used as the test car, was driven on both eastbound and west bound Highway 417 and Carling Avenue at the speed of the traffic stream. Lane change or passing manoeuvres were made only for passing vehicles travelling far below the average traffic speed. In the case of the eastbound Queensway, there was very limited opportunity to pass other vehicles. Therefore following the car ahead was equivalent to average traffic stream speed. This procedure, known as the "average car method" was used because it was unsafe to use the "floating car method" on the freeway [ITE Handbook, 1992; Transportation Research Board, 1985; McShane & Roess, 1992]. Furthermore, the average car method has proved to be more reliable and accurate than the floating car method [ITE, 1992]. Cumulative travel times were recorded at the gore of both entrance and exit ramps with a battery powered digital stop watch, whilst distances were recorded from the odometer.

On Carling Avenue, cumulative travel times were recorded at intersections and other locations where the vehicle came to a complete stop. Arterial free flow speed was recorded at mid-blocks of signalised intersections, where it was not hindered by other vehicles. During data collection, the test car was a platoon leader and a platoon follower. Tables 7-3, 7-4, & 7-5 are sample data sheets of travel time runs for both the Ottawa Queensway and Carling Avenue.

Table 7-3: A Sample of Travel Time Run For Eastbound Carling Ave

Street Name:EB Carling Ave Date:Feb 15 th 1994 Time:7:51 Weather								
Cross Streets	Cum. Dist Km	Cum TT mins	Segment Trav Time	Stop Time secs	Seg Speed	Speed Limit	Ideal Travel	Segment Delay
Pinecrest	0.45	0	1.23		15	60		
Richmon	0.75	1.23	0.45	0:48	46	60		
Alpine	1.1	1.68	0.27		34	60		
Croydon	1.25	1.95	0.32		28	60		
Connau	1.4	2.27	1.03		26	60		
LF	1.85	3.30	0.30	0:02	30	60		
Edgewor	2.0	3.60	1.68		21	60		
Wo'ffe S.	2.6	5.28	0.32	33:00	80	60		
Fairlawn	2.8	5.60	0.15		28	60		
Carl. SC	3.0	5.75	0.20		45	60		
Iroquous	3.15	5.95	1.90		28	60		
Maitland	4.05	7.85	0.40	41:00	45	60		
Broad.	4.35	8.25	1.50		16	60		
Clyde	4.75	9.75	0.42	48:00	43	60		
Churchill	5.05	10.17	1.65		27	60		
Kirkwood	5.8	11.82		27:00				

Table 7-4: A Sample of Travel Time Run For Westbound Carling Ave

Street Name:WB Carling Ave			Date:Feb 15 th 1994 Time:7:51 Weather					
Cross Streets	Cum. Dist Km	Cum Tl mins	Segment Trav Time mins	Stop Time secs	Seg Speed	Speed Limit	Ideal Travel Time	Segment Delay
Churchill	0.3	0.433				60		
Clyde	0.75	1.417	0.984		27	60		
Broadvie	1.00	2.667	1.250		12	60		
Maitland	1.90	3.567	0.900		60	60		
Iroquous	2.10	3.750	0.183		66	60		
Carl. SC	2.25	3.917	0.167		54	60		
Fairlawn	2.95	4.670	0.753		56	60		
Wo'ffe	3.15	4.833	0.163		74	60		
Edgewort	3.20	5.033	0.200		15	60		
Lincoln	3.65	5.800	0.767		35	60		
Connau	3.80	6.033	0.233		38	60		
Croydon	4.00	6.250	0.217		55	60		
Alpine	4.30	7.817	1.567		11	60		
Rich	4.60	8.233	0.416		43	60		
Pine	5.10	8.750	0.517		58	60		

Table 7-5: Sample Travel Time Runs for EB Queensway.

Study Section: Eastbound Highway 417 Date: Feb 8 th 1994 Time: 8:00 Road Condition: Bare			
Ramp	Cumulative Distance (km)	Cumulative Travel Time (mins)	Av. Link Travel Speed (km/h).
Greenbank ramp # 51		0.000	
	0.2		
Greenbank ramp # 35		0.450	
	0.7		
Greenbank ramp # 25		1.283	21
	2.3		
Woodroffe ramp # 51		5.750	
	2.4		
Woodroffe ramp # 35		6.100	
	2.7		
Woodroffe Ramp # 25		6.817	27
	4.2		
Maitland Ramp # 51		10.167	
	4.3		
Maitland Ramp # 15		10.400	53
	5.5		
Kirkwood Ramp # 57		11.750	
	6.2		
Kirkwood Ramp # 55		12.350	
	6.7		
Kirkwood Ramp # 65		12.567	

7.3 Results From Field Studies

From travel time studies, it was observed that the peak 15 mins in the corridor was between 7:50 and 8:05 a.m. in the eastbound direction. Corresponding westbound field and simulated results were also used to compare off peak conditions. Out of a total of 49 runs, only travel time runs in the peak 15 mins of the hour were used for comparison. Table 7-6 shows selected travel time runs, used for comparison. The westbound Queensway travel times were consistent, ranging from 3.5 to 3.8 minutes, a 9% difference. For the eastbound direction, travel times ranged from 12.6 mins to 21.9 mins (a 74% change from the minimum to the maximum), suggesting that peak period freeway travel times are unpredictable, inconsistent and erratic. Such traffic patterns are due to high traffic volumes and densities, resulting in shockwaves upstream of mainline and ramp bottlenecks, causing a ripple effect in traffic flow. The end result is forced flow, increased stops, and longer travel times on the mainline.

On the Westbound Carling Avenue, travel times were between 8.5 and 9.5 mins (12% change between the lowest and the highest rates) whilst the eastbound direction recorded 10.4 to 12.5 mins (20% change between the highest and the lowest rates). These results indicate that travel time on eastbound Carling Avenue during the peak period is lower than its corresponding section on the Queensway. Furthermore travel time variation on Carling Ave is very low, when compared to the eastbound Queensway. Therefore one can predict the arterial travel time with a reasonable level of accuracy.

Eastbound and westbound travel times for both the Queensway and Carling Avenue were averaged and compared with NETSIM and FREFLO simulation models.

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of/de

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PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
NBS 1010a ANSI/ISO #2 EQUIVALENT

PRECISIONSM RESOLUTION TARGETS

Table 7-6: Selected Travel Times.

Date	Designation	Start Time	Cum Travel Time
Feb 7 th 1994	WB Queensway	7:57	3.767
Feb 8 th 1994	WB Queensway	7:50	3.533
Feb 10 th 1994	WB Queensway	8:00	3.783
May 30 th 1994	WB Queensway	8:01	3:750
June 2 nd 1994	WB Queensway	7:23	3.650
Feb 7 th 1994	EB Queensway	7:52	14.583
Feb 8 th 1994	EB Queensway	8:00	12.567
Feb 10 th 1994	EB Queensway	7:57	21.850
May 30 th 1994	EB Queensway	7:43	13.250
June 2 nd 1994	EB Queensway	7:53	12.917
Feb 14 th 1994	WB Carling	8:04	9.500
Feb 15 th 1994	WB Carling	7:55	8.50
Feb 16 th 1994	WB Carling	7:40	8.33
Feb 18 th 1994	WB Carling	8:02	8.75
May 24 th 1994	WB Carling		7.63
Feb 14 th 1994	EB Carling	7:54	10.67
Feb 15 th 1994	EB Carling	7:51	11.82
Feb 16 th 1994	EB Carling	7:50	12.52
Feb 18 th 1994	EB Carling	7:48	11.20
May 24 th 1994	EB Carling	7:50	10.43

7.3 Comparison of Simulation and Field Results:

7.3.1 FREFLO Model

Comparison of observed and simulated results for FREFLO are shown in Figures 7-1 &

7-2. An average westbound travel time run of 3.7 mins was obtained as compared to a

Travel Time Comparison Westbound Highway 417

Figure 7-1 Travel Time Comparison: WB Highway 417

Travel Time Comparison Eastbound Highway 417

Figure 7-2 Travel Time Comparison EB Highway 417

simulated 3.3 mins. This gives an error of 11%. An average eastbound travel time of 15 mins was obtained as compared to a simulated time of 13.4 mins indicating a 11% error. The model tends to underestimate cumulative travel times for both directions.

7.3.2 NETSIM Model.

An average eastbound travel time of 11.3 mins, compared with simulated time of 12.8 min yields an error of 12% for eastbound Carling Ave. For the westbound direction, an average travel time of 8.5 mins, compared with a simulated time of 8.3 mins gives a 3% difference (Figures 7-3 & 7-4). Therefore the model overestimated travel times at peak periods whilst it slightly underestimated cumulative travel time at off peak periods.

Based on these results it could be inferred that all performance measures generated by both NETSIM & FREFLO are within 12% error.

In general, both FREFLO & NETSIM models compared reasonably well with field results, given the fact that,

- the FREFLO model was not calibrated with site specific data,
- all intersections on Carling Avenue were modelled as "pretimed."

Travel Time Comparison Eastbound Carling Avenue

Figure 7-3 Travel Time Comparison EB Carling Ave.

Travel Time Comparison Westbound Carling Avenue

Figure 7-4 Travel Time Comparison WB Carling Ave.

CHAPTER 8

IMPLEMENTATION

8.1 Introduction

The purpose of this chapter is to give a brief description of how the methodology developed in this study could be implemented in the field. Because freeway network and surface street network are under different jurisdictions in most urban areas, implementation of freeway and arterial traffic management methods require a system integration. In such a case, strategies will be executed in the corridor with one accord in order to minimise corridor travel time, and make efficient and optimal use of resources.

8.2 System Integration

The concept of system integration implies :

- a complete elimination of provincial, municipal and federal boundaries in traffic management and control,
- to coordinate policies and programs in the corridor, and
- to cooperate and communicate in traffic control matters.

These attributes of integration would maximize the efficiency and effectiveness of traffic in the corridor. Integration attempts to converge all transportation systems in the corridor

and makes them work as a single system in order to maintain a balanced movement of traffic in the corridor. Localised traffic management systems cannot monitor traffic in the entire corridor, and therefore its benefits are localised. Sometimes policies of neighbouring agencies work against each other. Any system integration requires free flow of information between all the agencies. There are three Cs in system integration, namely:

- Co-ordination,
- Co-operation, and
- Communication.

For cost effectiveness, the existing urban and proposed freeway traffic control systems need not be discarded or changed, neither should the responsibilities of these agencies differ. The most economical method is to link both arterial and freeway control systems together, such that both control centres could monitor the entire corridor. This could be made possible through a centralised computer data base, linked to both systems to collect and disseminate corridor information, similar to the one used by the smart corridor in California, and as being planned in the Gardiner-Lakeshore Corridor Traffic Management System in Toronto [Kay, 1992; Kelman, 1992]

When corridor-wide policies, goals and objectives are developed and a complete knowledge of real time traffic in the corridor are made available, to all agencies, control strategies will be more acceptable, and mobility enhanced.

8.3 Methodology Application

The methodology, as discussed in Chapters 4 & 5 are best suited for "on line" metering.

As such, the mainline and alternate surface routes require vehicle detectors, traffic signal controllers, a central traffic control system, and communications systems.

8.3.1 Freeway Links

Field equipment for the freeway is not much different from the planned systems. Induction loop detectors, located upstream and downstream of ramp junctions as well as in between basic freeway sections collect traffic volume and occupancy data. The ramp controllers, located at interchanges,

- ☐ control adjacent ramp signals when the central control system is "off line",
- ☐ collect volume and occupancy data,
- ☐ perform some checks on the accuracy of loop information, and
- ☐ gather and transmit alarm and failure information.

A central control centre, equipped with central computers receives data from vehicle detectors and ramp controllers via communication lines, and sends metering plans to all ramps concerned.

8.3.2 Surface Streets:

Real time traffic adjacent to the mainline could be monitored, detected, and controlled by a traffic adaptive control system, similar to the **SCOOT** or **SCATS** system.

SCOOT, which stands for Split Cycle & Offset Optimisation Technique, was developed in Glasgow, United Kingdom, by a team from the Transport Road Research Laboratory (TRRL), the Ferranti, GEC, & Plessey traffic companies. It is a significant improvement

over the "First Generation" traffic control systems. First generation traffic control systems use "off line" signal timing plans from computer programs such as **TRANSYT**, **PASSER** etc, based on historical traffic data.

The **SCOOT** system develops timing plans based on real time traffic collected from loop detectors. Cycle splits, and offsets are adjusted automatically in small increments to respond to prevailing traffic [Hunt et. al., 1982; Clowes, 1982; Walmsley, 1982].

The **SCOOT** system is presently used by 25 cities in the UK, and other cities which include Hong Kong, Beijing, Sao Paolo, Red Deer (Alberta), Halifax, and Oxnard (California). Presently, the Municipality of Metropolitan Toronto is conducting a demonstration project on three distinctly different signal network areas, with different operational characteristics and complicated traffic operations using the **SCOOT** system [Quan, Greenough et. al., 1993]. The Regional Municipality of Ottawa-Carleton have also identified traffic adaptive control as its future traffic control system.

The main field equipment for a traffic adaptive control could include:

- vehicle detectors,
- communication unit,
- central controller.

Following is a brief description of the **SCOOT** system.

Loop detectors, used in the vehicle detection system, are located about 10 to 15 metres downstream of all signalised approaches on all surface street approaches in the corridor, as shown in Figure 3.1. This location enables the collection of traffic volumes

Figure 8.1 Layout of Vehicle Detectors

Figure 8.2: Principles of The SCOOT Traffic Model.

Source: Hunt et al 1982.

at free flow conditions. Based on traffic cruise speed, the status of the traffic signal, and saturation flow rate, vehicle occupancies, traffic volumes and queue lengths are estimated and predicted seconds ahead of time (see Figure 8.2). Traffic queues and occupancy provide an indication of delays at the intersection.

A few seconds before the end of each phase, the split optimizer either decides to decrease or increase the green time based on volume and queue predictions. Cycle lengths are updated any time there is a change in green times, by small increments, for at least 2.5 minutes. Signal offsets are also adjusted incrementally to minimize stops, delays and maintain a smooth signal transition [Hunt et al, 1982; Clowes, 1982; Walmsley, 1982].

8.3.3 The Algorithm

The innovative aspect of this ramp control system lies in its system integration, where ramp controls are coordinated with other corridor traffic control systems. Control algorithms for system integration are outlined in the flow chart shown in Figure 8.3. A description of the algorithm is provided next.

Mainline detectors are scanned by the microprocessors 60 times a second, to measure volume and occupancy data, which are then sent to the central control computer once every second. The condition of various sections of the mainline are assessed, to determine if these volumes and occupancies are greater or less than the established threshold capacity. Where mainline traffic is less than the threshold value, entrance ramps are not controlled. If on the other hand these values are greater than the established threshold values, metering rates are calculated and introduced using the linear

Figure 8.3: Algorithms For Corridor Traffic Control

programming technique as described earlier.

"On line signal timing plans using the SCOOT system are generated and introduced on all signalized intersections in the corridor. Each signalized intersection approach, by virtue of its geometry, number of lanes, traffic volumes, pedestrian volumes, etc would need threshold levels of delay established, based on current and predicted traffic conditions on the basic freeway sections. As the mainline LOS deteriorates, the threshold value for intersection stopped delays are increased up to a certain limit, where it is maintained even if mainline conditions continue to deteriorate. The critical approach level of delay influences metering rates. Where the latter is greater than the threshold values, metering rates are increased gradually. Signal timing are fine tuned to respond to reduced traffic on the surface streets, due to increased metering and critical approach delays compared with the threshold values. When critical approach delay is less than its threshold value, existing metering rates are checked if they are greater than the optimized rates. If yes, metering rates are reduced incrementally, and if no, metering rates are calculated using the linear programming technique.

8.3.4 Limitations

- The success of this methodology depends on establishing the changing threshold delay levels for each approach with respect to mainline traffic predictions. The purpose of the changing threshold values is to smoothen the time taken for diverted traffic to have an impact on signalised intersections.
- The algorithm depends solely on the accuracy of the vehicle detection system. The

accuracy of loop detectors as mentioned earlier is within 6% which is acceptable for the system. However, image processing techniques, such as the Autoscope technology, when fully developed will enhance the accuracy of the system.

- The threshold delay values do not take into consideration any origin destination data, therefore these values may be under or over estimated. However historical origin destination data may be used to estimate threshold values within a reasonable range.

8.3.5 Advantages

- The algorithm works with real time traffic conditions. The prevailing traffic signal timing plans respond to real time metering rates. The control of traffic in the entire corridor is coordinated to minimise corridor travel time as well as maintain a certain level of acceptable delay on the surface streets. Corridor travel time is therefore predictable, and mobility is improved in the corridor.
- The system does not rely on a traffic assignment procedure. The SCOOT system measures real traffic moving on links, making any traffic assignment unnecessary.
- Improved data for traffic and transportation planners become readily available. Real time traffic conditions could be made available to the motorist through a number of systems as in the form of changeable message signs on the freeways, arterial control signs, highway advisory radio and other media dissemination procedures. Such information could also be available at major transit transfer points to avoid routes with longer delays.

- All agencies communicate, coordinate, and cooperate effectively; hence all programs in the corridor are interrelated with common goals.

8.4 Application To The Ottawa-Carleton Region

The Ottawa-Carleton Region uses a decentralised urban traffic control system, with three levels of control as shown in Figure 8.4:

- Local Control,
- Regional Control, and
- Central Control System.

The local controllers, located close to each signalised intersection,

- ☐ monitor traffic at intersections with loop detectors within its jurisdiction,
- ☐ check system components for failure at the local level,
- ☐ send traffic data and all other relevant information to the regional controllers and
- ☐ send signal timing plans to traffic signals.

The regional controllers, located at Orleans, Kanata, Walkley, Britannia and Montreal Rd,

- ☐ provide traffic and equipment information status to the central control system, (via regional controllers) as well as carry instructions from the centralised control system to the local controllers,

Figure 8.4 The UTCS System In The Ottawa Carleton Region.

- communicate with all local controllers in their jurisdiction; these determine timing plans based on *time of the day*.

The central control system, located at Loretta Ave, (Ottawa):

- is capable of monitoring traffic at any intersection; therefore equipment failures are detected on time, and the proper response executed,
- is capable of controlling and overriding timing plans for any signalised intersection, via the regional controllers.

A set of eight local controllers communicate with their respective regional controller through a leased Bell Canada telephone line with the help of a high speed modem. Regional controllers are also linked to the central traffic control system in a similar manner.

The advantages of the system are:

- reduced cost of communication: the number of telephone lines leased from Bell Canada is reduced,
- although the local, regional, and central control systems are linked to each other, they can operate independently if any of them is out of order.

Its disadvantage is that field equipment is bigger, more expensive, more complex and require a higher level of protection.

To apply ramp metering as described in the methodology for the Ottawa-Carleton Region, the present loop detectors on Carling Ave and all its crossing streets will have to be re-

Figure 8.5 Proposed System Integration For The Ottawa Queensway Corridor.

sited within 15m downstream of signalized approaches as discussed earlier and shown in Figure 8.1. Local, regional and central control centres need not be modified. An additional system component required is a centralised computer data base, which could be located at the freeway traffic management centre to communicate with the latter, as well as the centralised urban traffic control centre (Figure 8.5). Existing control and responsibilities of both RMOC and MTO need not change. The only special feature is to develop algorithms, as described earlier, to override ramp metering rates based on threshold stopped delays.

Advanced sensors, computers and communication technologies are rapidly advancing, which makes it very uncertain what type of hardware and software that any advanced control system should use to be cost-effective and not be obsolete within a few years. Furthermore different agencies may be operating different types of systems which may not be compatible with each other.

The concept of system integration is not only to integrate the overall traffic control system but also to integrate its system components as well. The control system should be developed with an ultimate objective to make its system architecture flexible. The uncertainties in the type of hardware and software to use, could be reduced by a method where integrated transportation agencies could work together to select identical hardware in future as well as to develop a common or compatible software in order to have a complete control of their control systems and reasonably respond to changing technology. As the number of agencies are integrated, there will be more technical experts who can participate in such regional programs.

CHAPTER 9

CONCLUSIONS & RECOMMENDATIONS

9.1 Conclusions

Although the use of current ramp metering methodologies has improved the freeway traffic flow in most application areas, serious inefficiencies have resulted because their impacts have been addressed, in only a few locations, from a corridor perspective. Hence corridor travel time is not optimised. The objective of this research was to develop a methodology for ramp metering as well as to conduct a detailed investigation of its impacts in the corridor. As reported in the previous chapters, these have been accomplished. The impacts of metering were analyzed by using the Ottawa Queensway as a case study. Unlike other ramp metering feasibility studies which have been conducted [McCormick Rankin, 1986; Cocoran et. al., 1991; IBI Group 1987, 1991 & 1993; Ayland et. al., 1982; etc], this study carried out the impacts of metering on adjacent parallel routes and all streets crossing this route. Corridor analysis was also undertaken to determine whether metering improved the overall traffic conditions in the corridor.

From the research results, the following conclusions can be drawn:

- Unlike traffic signals, there are no warrants for ramp signals in order to determine their installation. Hence a methodology is needed to determine their feasibility,

such that there is an efficient movement of traffic, with no adverse effects especially on the alternate routes. Existing methodologies only looked at metering feasibility in terms of benefit cost analysis without conducting a full scale analysis on alternate routes. These are therefore opposed by the public, and some Municipal, City and County Transportation Agencies. This methodology provides a comprehensive analysis of alternate routes within a corridor, using operational analysis.

- Given residual capacity on alternate routes, freeway ramp control, when integrated and coordinated with arterial traffic control, yields the following results:
 - Reduced total corridor travel time is possible.
 - Increased traffic throughput in the corridor as well as traffic throughput in the direction of the peak traffic with metering is another useful outcome, even though the objective of metering was not to increase capacity of the freeway, but to maintain a stable flow. Improved traffic conditions meant that there was a stable and orderly movement of traffic on the mainline and the number of vehicles competing for space at on ramp merge and other bottlenecks has been reduced tremendously.
- Efficient progression is capable of causing only a very small change in performance measures of traffic travelling against the peak direction even when "optimised metering rates" are used.
- Although travel time on alternate routes were increased owing to traffic diversion,

the eastbound and total corridor travel times reduced remarkably.

- Metering rates under the sole objective of maximising freeway traffic input would yield undesirable impacts on alternate routes, whilst maximum metering rates would slightly improve freeway traffic flow. Therefore the best metering plan is the one that minimizes total corridor travel time and at the same time satisfies the threshold intersection approach stopped delays for all intersections in the corridor.
- A comparison between field studies and the FREFLO model indicated that during congestion, the model slightly underestimated travel time. The reason cannot be fully attributed to the model since certain speed density relation parameters were assumed due to lack of data. Field studies from link travel analysis confirmed the accuracy of TRAF-NETSIM simulation model.

9.2 Recommendations

From the research, the following recommendations could be made:

- The methodology reported here, for determining the best metering plans for effective and efficient movement of traffic in the corridor could be used by the profession to minimise corridor travel time and reduce opposition to ramp metering.
- The benefits and impacts of metering as analyzed, were estimated only for the peak 15 minutes of the peak hour. Further studies are needed to evaluate the impacts of metering for the entire peak period.

- Further studies are needed in the development of integrated dynamic simulation models, with a view to optimize traffic in a corridor as one system. Such models would be very useful for a comprehensive evaluation of diverted traffic, including dynamic traffic prediction and assignment, and provide accurate corridor analysis.
- There is the need to establish a methodology for selecting the type of ramp control strategy under budget constraints, addressing problems such as where to start, when to expand the system, and the required components for effective operation. Models to determine the best control strategy, given such constraints as available funds, would be very useful to traffic and transportation managers.
- Further studies are also needed to determine the criteria to select an integrated control system components (hardware and software components) as well as its cost effectiveness, bearing in mind the rapidly changing technology.
- Further studies are needed to quantify the impacts of metering on noise levels, fuel consumption and vehicle emissions in the corridor.
- Freeway, arterial and intersection level of service standards determined by the Highway Capacity Manual, although very useful, are not site-specific. More research is required to determine the tolerable level of congestion in the Ottawa-Carleton Region. Such a study would be used to decide on the threshold level of

service required at all intersection approaches.

- Real time traffic control strategies could be made more responsive to traffic, if the changing O.D. patterns could be accounted for. Rapid advancement in communication, computers and sensor technologies and the increased number of vehicle probes will make it possible to determine real time O.D. patterns. Further studies are required to determine dynamic O.D. patterns to reflect real time traffic.
- More research in latent demand, its key factors, and impacts on traffic growth is required.

A number of IVHS technologies, such as sensor technologies currently under development or available, open up more options and innovation for improved traffic flow. Camera detection devices could be used to identify vehicle types (HOV & non HOVs). Infrared sensors, relying on infrared heat image technologies will be capable of determining the number of people in a vehicle. These could be used to enforce HOV bypass lane enforcement during priority metering. Development of traffic progression and signal timing plans based on the number of occupants in a vehicle will be made possible to encourage the increased use of HOVs & HOV lanes.

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Appendix A

Intersection Count Data

Time: 07:00 to 08:00 Hours

Year: 1992

Carling/Pinecrest

Carling/Richmond

Carling/Alpine

Carling/Croydon

Carling/Connaught

Carling/Lincoln Fields

Carling/Edgeworth

Carling/Woodroffe South

Carling/Fairlawn

Carlingwood SC

Carling/Iroquois

Carling/Maitland

Carling/Broadview

Carling/Clyde

Carling/Churchill

Carling/Kirkwood

Appendix B

Measures of Effectiveness

SUMMARISED EASTBOUND CARLING Measure Of Effectiveness.

NVO.OUT ¹						
LINK	VEH KM ²	VEH MIN ³	TT SECS ⁴	DEL S/V ⁵	SPEED ⁶	CUM TT MIN
RICH	78.94	538.90	124.80	31.00	9.00	2.08
ALPINE	109.46	176.50	30.60	1.70	37.00	2.59
CROY	65.93	111.70	18.30	1.50	35.00	2.90
CONN	52.06	141.90	21.30	3.50	22.00	3.25
LF	65.91	115.90	16.90	1.50	34.00	3.53
EDGE	177.81	398.40	59.20	12.30	27.00	4.52
W. S	238.26	463.60	70.10	6.50	31.00	5.69
FAIRLAWN	60.38	233.30	46.30	21.10	16.00	6.46
CAL. SC	48.34	106.50	26.30	6.80	27.00	6.90
IROQUOUS	33.85	77.30	19.20	5.00	26.00	7.22
MAIT	183.96	319.30	89.50	19.50	35.00	8.71
BROADVIEW	91.10	139.10	32.70	3.50	39.00	9.25
CLYDE	102.07	186.70	45.00	10.50	33.00	10.00
CHURCHILL	73.94	240.50	58.40	17.80	18.00	10.98
KIRK	174.03	418.70	108.30	25.00	25.00	12.78
TOTAL	1556.04	3668.30	766.90			

NV1A.OUT						
LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN
RICH	67.67	617.00	166.80	56.90	7.00	2.78
ALPINE	122.75	203.50	31.60	1.60	36.00	3.31
CROY	73.86	126.00	18.40	1.90	35.00	3.61
CONN	57.01	185.00	25.30	5.10	19.00	4.04
LF	72.47	136.60	18.10	1.90	32.00	4.34
EDGE	191.02	374.50	51.80	9.20	31.00	5.20
W. S	249.66	570.90	82.30	15.70	26.00	6.57
FAIRLAWN	70.80	103.90	17.70	0.70	40.60	6.87
CAL. SC	63.33	90.40	17.10	1.30	42.10	7.15
IROQUOUS	44.62	62.80	11.80	1.10	42.60	7.35
MAIT	302.62	663.10	113.00	30.20	27.40	9.23
BROADVIEW	173.12	314.40	38.80	3.60	33.00	9.88
CLYDE	168.48	964.80	140.80	57.80	11.00	12.23
CHURCHILL	114.13	365.50	57.60	16.80	19.00	13.19
KIRK	264.04	888.30	151.40	42.60	18.00	15.71
TOTAL	2035.58	5666.70	942.50			

¹ NVO stands for existing conditions (based on 1992 intersection counts).

NV1 stands for traffic conditions based on optimised metering rates

From NV2...NV26, traffic was increased at the rate of 30 vph for Pinecrest, Woodroffe, Maitland, & Kirkwood interchange.

² Total Travel in Vehicle Kilometers

³ Total Travel Time in vehicle minutes

⁴ Travel Time in seconds

⁵ Average Stopped Delay in Seconds/Vehicle

⁶ All link speeds are in kilometers per hour

LINK	VEH KM	VEH MIN	TT SECS	DEL S/V	SPEED	CUM TT MIN	NV2A.OUT
RICH	72.24	650.80	164.80	61.00	7.00	2.75	
ALPINE	127.82	211.40	31.50	1.40	36.30	3.27	
CROY	75.48	134.80	19.30	2.00	33.60	3.59	
CONN	59.87	199.20	26.00	4.90	18.00	4.03	
LF	77.43	142.90	17.70	1.60	32.85	4.32	
EDGE	209.94	436.20	54.90	11.10	28.90	5.24	
W. S	250.26	617.80	88.90	19.80	24.30	6.72	
FAIRLAWN	74.38	107.60	17.30	0.30	41.50	7.01	
CAL. SC	70.90	98.50	16.60	1.00	43.20	7.28	
IROQUOUS	51.48	69.70	11.40	0.60	44.30	7.47	
MAIT	333.57	691.70	106.90	26.70	29.00	9.26	
BROADVIEW	179.83	349.10	41.50	4.70	30.90	9.95	
CLYDE	172.99	924.60	131.50	55.50	11.20	12.14	
CHURCHILL	119.22	460.40	69.40	22.80	15.50	13.30	
KIRK	283.54	717.30	113.90	29.30	23.70	15.19	
TOTAL	2158.95	5812.00	911.60				

LINK	VEH KM	VEH MIN	TT SECS	DEL S/V	SPEED	CUM TT MIN	NV3A
RICH	69.19	584.10	154.40	54.70	7.00	2.57	
ALPINE	125.60	205.90	31.20	1.30	37.00	3.09	
CROY	74.40	129.70	18.80	2.10	34.00	3.41	
CONN	59.48	215.30	28.30	6.30	17.00	3.88	
LF	75.03	145.00	18.50	2.10	31.00	4.19	
EDGE	189.70	881.70	122.70	33.70	13.00	6.23	
W. S	261.67	604.80	83.20	19.00	26.00	7.62	
FAIRLAWN	76.38	108.00	17.00	0.00	42.40	7.90	
CAL. SC	68.52	93.90	16.40	1.00	44.00	8.18	
IROQUOUS	47.98	72.00	12.60	1.60	40.00	8.39	
MAIT	312.94	611.20	100.70	22.40	31.00	10.06	
BROADVIEW	175.17	307.20	37.50	4.40	34.00	10.69	
CLYDE	155.36	1094.20	173.20	85.00	9.00	13.58	
CHURCHILL	105.41	475.10	81.00	29.60	13.30	14.93	
KIRK	264.04	657.60	112.10	27.80	24.00	16.79	
TOTAL	2060.87	6185.70	1007.60				

LINK	VEH KM	VEH MIN	TT SECS	DEL S/V	SPEED	CUM TT MIN	NV4A
RICH	64.31	526.10	168.10	54.40	7.00	2.80	
ALPINE	116.75	193.00	31.50	1.30	36.00	3.33	
CROY	69.17	133.40	20.80	2.40	31.00	3.67	
CONN	55.70	172.20	24.10	4.80	19.00	4.08	
LF	71.35	129.90	17.50	1.80	33.00	4.37	
EDGE	190.14	367.00	51.00	8.00	31.00	5.22	
W. S	250.26	582.80	83.80	13.50	26.00	6.61	
FAIRLAWN	71.58	102.60	17.20	0.20	41.80	6.90	
CAL. SC	61.53	85.50	16.60	1.10	43.20	7.18	
IROQUOUS	43.51	65.70	12.70	1.40	39.70	7.39	
MAIT	257.05	511.80	102.70	25.60	30.00	9.10	
BROADVIEW	154.46	284.80	39.40	4.40	33.00	9.76	
CLYDE	154.95	937.50	148.80	61.40	10.00	12.24	
CHURCHILL	105.13	435.10	74.40	25.80	14.00	13.48	
KIRK	251.29	658.00	117.90	25.70	22.90	15.44	
TOTAL	1917.18	5185.40	926.50				

LINK	VEH KM	VEH MIN	TT SECS	DEL S/V	SPEED CUM	TT MIN	NV5A
RICH	72.85	575.50	144.50	43.00	8.00	2.41	
ALPINE	118.83	186.80	29.90	1.30	38.00	2.91	
CROY	71.51	121.60	18.40	1.90	35.00	3.21	
CONN	57.27	163.90	22.40	4.00	21.00	3.59	
LF	72.47	126.70	16.80	1.30	34.00	3.87	
EDGE	191.46	354.40	48.90	7.60	32.00	4.68	
W. S	255.67	580.30	81.70	13.80	26.00	6.34	
FAIRLAWN	72.18	107.10	17.80	0.40	41.00	6.34	
CAL. SC	62.53	87.10	16.60	1.00	43.00	6.62	
IROQUOUS	44.68	72.60	13.60	2.00	37.00	6.84	
MAIT	298.32	559.30	96.70	19.60	32.00	8.43	
BROADVIEW	165.39	294.00	38.00	3.80	34.00	9.09	
CLYDE	168.89	930.00	135.40	54.10	11.00	11.35	
CHURCHILL	114.13	334.70	52.70	14.40	20.50	12.22	
KIRK	244.54	663.00	122.00	32.20	22.10	14.26	
TOTAL	2010.72	5157.00	855.40				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV6A
RICH	70.71	586.30	151.60	50.50	7.20	2.53	
ALPINE	117.92	186.70	30.10	1.40	38.00	3.03	
CROY	70.79	130.30	19.90	2.40	33.00	3.36	
CONN	56.49	189.00	26.10	4.90	18.00	3.79	
LF	70.23	126.70	17.30	1.70	33.00	4.08	
EDGE	188.38	378.20	53.00	9.30	30.00	4.97	
W. S	249.66	615.60	88.80	15.80	24.00	6.45	
FAIRLAWN	71.38	105.20	17.70	0.30	41.00	6.74	
CAL. SC	63.53	88.20	16.60	0.90	43.00	7.02	
IROQUOUS	44.62	61.10	11.50	0.90	44.00	7.21	
MAIT	277.68	499.10	92.70	19.60	33.00	8.76	
BROADVIEW	157.81	275.20	37.30	3.90	34.00	9.38	
CLYDE	161.92	653.30	99.20	35.30	15.00	11.03	
CHURCHILL	108.73	393.60	65.10	19.60	17.00	12.12	
KIRK	270.79	548.60	91.20	16.10	30.00	13.63	
TOTAL	1980.64	4837.10	818.10				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV7A
RICH	75.90	523.30	126.10	37.30	9.00	2.10	
ALPINE	119.11	196.30	31.30	1.40	36.40	2.62	
CROY	71.69	121.60	18.30	2.00	35.40	2.93	
CONN	55.18	156.60	22.20	5.00	21.10	3.30	
LF	69.91	123.60	17.00	1.70	34.00	3.58	
EDGE	195.42	377.00	51.00	8.70	31.00	4.43	
W. S	268.27	605.10	81.20	10.90	27.00	5.79	
FAIRLAWN	78.58	115.20	17.60	0.40	41.00	6.08	
CAL. SC	67.13	91.40	16.30	0.90	44.00	6.35	
IROQUOUS	47.42	68.50	12.10	1.20	42.00	6.55	
MAIT	288.86	543.00	97.00	21.60	32.00	8.17	
BROADVIEW	159.78	287.40	38.50	4.30	33.00	8.81	
CLYDE	161.10	747.10	114.10	43.80	13.00	10.71	
CHURCHILL	108.13	400.30	66.50	20.50	16.00	11.82	
KIRK	253.40	613.20	108.90	26.50	25.00	13.64	
TOTAL	2019.88	4969.60	818.10				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED		NV8A
RICH	74.37	596.00	146.50	39.50	8.00	2.44	
ALPINE	118.79	186.50	29.80	1.30	38.00	2.94	
CROY	71.51	119.90	18.10	1.90	36.00	3.24	
CONN	57.53	176.60	24.00	4.60	20.00	3.64	
LF	72.15	126.60	16.80	1.50	34.00	3.92	
EDGE	192.78	362.50	49.70	8.00	32.00	4.75	
W. S	253.86	530.80	75.30	10.50	29.00	6.00	
FAIRLAWN	71.98	101.40	16.90	0.40	43.00	6.29	
CAL. SC	63.13	88.20	16.70	1.10	43.00	6.56	
IROQUOUS	45.18	63.60	11.80	1.00	43.00	6.76	
MAIT	288.00	513.13	91.90	19.00	34.00	8.29	
BROADVIEW	155.06	256.70	35.40	3.80	36.00	8.88	
CLYDE	168.07	708.00	103.60	38.60	14.00	10.61	
CHURCHILL	109.93	377.80	61.80	18.40	18.00	11.64	
KIRK	250.54	622.70	111.90	26.60	24.00	13.50	
TOTAL	1992.88	4830.43	810.20				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV9A
RICH	63.40	551.60	159.10	58.40	7.00	2.65	
ALPINE	111.34	179.80	30.70	1.60	37.00	3.16	
CROY	66.29	111.50	18.20	1.80	36.00	3.47	
CONN	53.23	177.30	26.00	5.10	18.00	3.90	
LF	68.31	121.30	17.00	1.70	34.00	4.18	
EDGE	177.37	336.40	50.10	9.50	32.00	5.02	
W. S	240.66	542.30	81.10	14.40	27.00	6.37	
FAIRLAWN	69.78	100.50	17.30	0.20	42.00	6.66	
CAL. SC	63.33	90.00	17.00	1.10	42.00	6.94	
IROQUOUS	45.74	69.90	12.80	1.80	39.00	7.15	
MAIT	288.86	518.10	92.50	19.00	34.00	8.70	
BROADVIEW	159.01	280.20	37.70	4.60	34.00	9.33	
CLYDE	166.02	669.40	99.20	35.40	15.00	10.98	
CHURCHILL	112.33	403.10	64.50	19.30	17.00	12.05	
KIRK	236.29	634.00	120.80	33.20	22.00	14.07	
TOTAL	1921.96	4785.40	844.00				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV10A
RICH	69.19	604.30	159.70	43.00	7.00	2.66	
ALPINE	116.94	195.60	31.80	1.50	35.90	3.19	
CROY	69.35	114.70	17.90	1.70	36.30	3.49	
CONN	54.40	162.00	23.30	4.10	20.10	3.88	
LF	69.27	124.90	17.30	1.90	33.30	4.17	
EDGE	191.46	367.50	50.70	8.80	31.30	5.01	
W. S	258.07	605.50	84.50	13.20	25.60	6.42	
FAIRLAWN	72.78	107.90	17.80	0.50	41.00	6.72	
CAL. SC	62.73	84.50	16.10	1.00	45.00	6.99	
IROQUOUS	44.62	63.10	11.90	0.90	42.00	7.18	
MAIT	281.98	487.20	89.00	17.60	35.00	8.67	
BROADVIEW	148.46	249.60	36.00	3.70	36.00	9.27	
CLYDE	158.64	579.80	89.90	29.00	16.00	10.77	
CHURCHILL	110.53	389.80	63.40	18.70	17.00	11.82	
KIRK	257.29	639.80	111.90	26.70	24.00	13.69	
TOTAL	1965.71	4776.20	821.20				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV11A
RICH	70.41	569.00	147.80	45.20	7.00	2.46	
ALPINE	115.37	179.10	29.50	1.40	39.00	2.96	
CROY	69.35	118.30	18.40	1.90	35.00	3.26	
CONN	55.31	161.50	22.80	3.90	21.00	3.64	
LF	69.75	124.00	17.10	1.60	34.00	3.93	
EDGE	183.10	361.00	52.10	8.80	30.00	4.80	
W. S	240.66	527.00	78.90	13.00	27.00	6.11	
FAIRLAWN	69.78	98.50	16.90	0.30	43.00	6.39	
CAL. SC	63.33	88.70	16.70	1.10	43.00	6.67	
IROQUOUS	44.48	67.00	12.60	1.60	40.00	6.88	
MAIT	291.44	520.40	92.10	17.50	34.00	8.42	
BROADVIEW	152.36	245.00	34.40	3.80	37.00	8.99	
CLYDE	161.51	544.80	83.00	27.40	18.00	10.37	
CHURCHILL	107.83	394.20	65.70	20.20	16.00	11.47	
KIRK	249.04	623.40	112.70	27.10	24.00	13.35	
TOTAL	1943.72	4621.90	800.70				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV12A
RICH	72.54	584.20	147.30	42.10	8.00	2.46	
ALPINE	117.55	188.60	30.50	1.40	37.40	2.96	
CROY	70.79	117.60	17.90	1.90	36.10	3.26	
CONN	56.62	168.40	23.20	4.30	20.20	3.65	
LF	71.99	130.70	17.40	1.60	33.00	3.94	
EDGE	192.34	366.60	50.30	9.10	31.50	4.78	
W. S	252.66	625.60	89.20	14.10	24.20	6.26	
FAIRLAWN	72.38	105.60	17.50	0.40	41.00	6.56	
CAL. SC	65.93	90.40	16.40	1.00	44.00	6.83	
IROQUOUS	46.30	67.80	12.30	1.40	41.00	7.03	
MAIT	293.16	496.20	87.30	16.60	35.00	8.49	
BROADVIEW	155.24	249.80	34.50	3.50	37.30	9.06	
CLYDE	158.23	589.80	91.70	32.70	16.10	10.59	
CHURCHILL	105.13	403.90	69.00	22.30	15.60	11.74	
KIRK	237.79	566.30	107.20	26.00	25.20	13.53	
TOTAL	1968.65	4751.50	811.70				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV13A
RICH	71.63	563.80	144.00	46.30	7.60	2.40	
ALPINE	115.28	179.70	29.50	1.30	39.00	2.89	
CROY	69.71	114.70	17.80	1.70	37.00	3.19	
CONN	55.83	166.10	23.20	4.00	20.00	3.58	
LF	71.03	127.30	17.20	1.60	34.00	3.86	
EDGE	188.82	401.00	56.10	10.90	28.00	4.80	
W. S	246.06	568.70	83.20	16.10	26.00	6.18	
FAIRLAWN	73.58	105.90	17.30	0.40	42.00	6.47	
CAL. SC	64.93	90.10	16.60	1.00	43.00	6.75	
IROQUOUS	45.74	65.00	11.90	1.30	42.00	6.95	
MAIT	277.68	459.30	85.30	15.10	36.00	8.37	
BROADVIEW	147.07	232.70	33.90	3.30	37.90	8.93	
CLYDE	158.23	629.20	97.80	34.10	15.00	10.56	
CHURCHILL	105.43	271.40	46.30	10.50	23.00	11.34	
KIRK	246.04	650.00	118.90	28.90	23.00	13.32	
TOTAL	1937.06	4624.90	799.00				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	nv14a
RICH	71.93	604.80	153.80	44.00	7.00	2.56	
ALPINE	118.48	203.70	32.70	1.50	35.00	3.11	
CROY	69.89	123.90	19.20	1.80	34.00	3.43	
CONN	54.01	158.40	22.90	3.90	21.00	3.81	
LF	68.79	121.70	17.00	1.70	34.00	4.09	
EDGE	188.38	362.70	50.80	9.30	31.00	4.94	
W. S	263.47	647.70	88.50	13.60	24.00	6.42	
FAIRLAWN	74.18	106.50	17.20	0.40	42.00	6.70	
CAL. SC	62.53	84.30	16.10	0.90	45.00	6.97	
IROQUOUS	44.34	66.60	12.60	1.40	40.00	7.18	
MAIT	273.38	480.20	90.60	17.10	34.00	8.69	
BROADVIEW	146.48	236.30	34.60	3.60	37.00	9.27	
CLYDE	154.45	506.30	80.40	26.80	18.00	10.61	
CHURCHILL	107.23	362.60	60.80	17.30	18.00	11.62	
KIRK	271.54	686.00	113.70	26.40	24.00	13.52	
TOTAL	1969.08	4751.70	810.90				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV15A
RICH	68.89	542.30	144.00	40.30	8.00	2.40	
ALPINE	117.27	193.80	31.40	1.40	36.30	2.92	
CROY	69.71	122.00	18.90	2.00	34.30	3.24	
CONN	52.97	140.10	20.70	3.20	23.00	3.58	
LF	67.35	118.50	16.90	1.80	34.00	3.87	
EDGE	192.34	389.30	53.40	9.40	30.00	4.76	
W. S	262.87	590.80	80.90	11.70	27.00	6.10	
FAIRLAWN	74.18	104.60	16.90	0.30	43.00	6.39	
CAL. SC	65.93	89.90	16.30	1.00	44.00	6.66	
IROQUOUS	46.16	70.20	12.80	1.80	39.00	6.87	
MAIT	286.28	527.60	95.10	20.30	33.00	8.46	
BROADVIEW	136.83	208.40	32.70	3.10	39.00	9.00	
CLYDE	154.13	493.70	78.80	25.50	19.00	10.31	
CHURCHILL	102.13	414.00	72.90	24.30	15.00	11.53	
KIRK	246.04	756.90	138.50	33.30	20.00	13.84	
TOTAL	1943.08	4762.10	830.20				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV16A
RICH	71.02	620.20	159.70	46.30	7.00	2.66	
ALPINE	118.17	204.10	32.80	1.40	35.00	3.21	
CROY	69.71	123.20	19.10	2.10	34.00	3.53	
CONN	54.53	169.40	24.30	5.10	19.00	3.93	
LF	68.15	123.90	17.50	1.90	33.00	4.22	
EDGE	187.94	386.10	54.30	9.60	29.00	5.13	
W. S	232.26	518.70	80.40	11.40	27.00	5.47	
FAIRLAWN	65.58	96.10	17.60	0.90	41.00	6.76	
CAL. SC	58.93	82.50	16.70	1.10	43.00	7.04	
IROQUOUS	42.53	72.10	14.20	2.50	35.00	7.28	
MAIT	268.23	472.90	91.10	16.90	34.00	8.79	
BROADVIEW	132.60	208.80	33.80	3.30	38.00	9.36	
CLYDE	147.57	403.30	67.20	19.80	22.00	10.48	
CHURCHILL	98.53	324.00	59.10	17.00	18.00	11.46	
KIRK	249.04	548.80	99.20	17.80	27.00	13.12	
TOTAL	1864.79	4354.10	787.00				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV17A
RICH	72.54	593.60	149.70	46.00	7.00	2.50	
ALPINE	117.23	184.60	29.90	1.30	38.00	2.99	
CROY	70.07	117.90	18.20	2.00	36.00	3.30	
CONN	55.96	155.90	21.80	3.50	22.00	3.66	
LF	71.51	125.80	16.90	1.30	34.00	3.94	
EDGE	191.46	362.30	50.00	8.40	32.00	4.78	
W. S	248.46	551.80	80.00	12.00	27.00	6.11	
FAIRLAWN	71.38	103.50	17.40	0.70	41.00	6.40	
CAL. SC	64.13	89.90	16.70	1.10	43.00	6.68	
IROQUOUS	45.46	66.90	12.30	1.30	41.00	6.88	
MAIT	290.58	511.60	90.80	17.40	34.00	8.40	
BROADVI	139.88	211.12	32.40	2.10	40.00	8.94	
CLYDE	150.85	431.50	70.40	22.80	21.00	10.11	
CHURCHILL	98.23	333.80	61.10	17.40	18.00	11.13	
KIRK	228.04	580.20	114.50	32.00	24.00	13.04	
TOTAL	1915.78	4420.42	782.10				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV18A
RICH	72.54	588.50	148.40	46.90	7.00	2.47	
ALPINE	116.27	184.50	30.20	1.30	38.00	2.98	
CROY	70.25	122.00	18.80	1.70	35.00	3.29	
CONN	56.09	154.70	21.50	3.10	22.00	3.65	
LF	70.39	125.80	17.20	1.30	34.00	3.94	
EDGE	191.46	349.20	48.20	7.20	33.00	4.74	
W. S	261.07	522.20	72.00	9.50	30.00	5.94	
FAIRLAWN	74.38	109.20	17.60	1.00	41.00	6.23	
CAL. SC	65.73	91.50	16.60	1.00	43.00	6.51	
IROQUOUS	45.74	72.80	13.40	2.40	38.00	6.73	
MAIT	289.72	482.60	85.90	15.20	36.00	8.16	
BROADVIEW	141.15	221.20	33.60	3.00	38.00	8.72	
CLYDE	155.77	451.10	71.20	22.50	21.00	9.91	
CHURCHILL	102.13	361.60	63.60	18.30	17.00	10.97	
KIRK	237.79	651.70	123.40	32.30	22.00	13.03	
TOTAL	1950.68	4488.60	781.60				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV19A
RICH	68.58	595.70	158.80	47.40	7.00	2.65	
ALPINE	115.38	178.40	29.40	1.20	39.00	3.14	
CROY	68.09	111.30	17.70	1.90	37.00	3.43	
CONN	54.14	158.70	22.90	4.60	21.00	3.81	
LF	70.07	122.20	16.70	1.30	34.00	4.09	
EDGE	181.77	332.60	48.30	7.70	33.00	4.90	
W. S	233.46	472.00	72.70	10.60	27.00	6.11	
FAIRLAWN	67.38	97.00	17.30	0.60	41.70	6.40	
CAL. SC	60.53	85.70	16.90	1.20	42.00	6.68	
IROQUOUS	43.09	69.80	13.60	2.40	37.00	6.91	
MAIT	287.14	481.30	86.50	15.10	36.00	8.35	
BROADVIEW	138.46	208.10	32.30	2.50	40.00	8.89	
CLYDE	146.34	353.00	59.30	17.70	25.00	9.87	
CHURCHILL	100.93	422.00	75.10	25.70	14.00	11.13	
KIRK	220.53	516.30	105.40	27.90	26.00	12.88	
TOTAL	1855.89	4204.10	772.90				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV20A
RICH	72.54	574.20	144.80	40.40	8.00	2.41	
ALPINE	116.91	183.60	29.90	1.40	38.00	2.91	
CROY	70.25	118.30	18.20	1.90	37.00	3.22	
CONN	56.09	168.80	23.50	4.10	20.00	3.61	
LF	71.35	128.40	17.30	1.70	33.00	3.90	
EDGE	191.02	388.00	53.60	9.40	30.00	4.79	
W. S	249.06	542.00	78.50	10.60	27.00	6.10	
FAIRLAWN	71.38	104.80	17.60	0.80	41.00	6.39	
CAL. SC	62.73	87.40	16.60	1.00	43.00	6.67	
IROQUOUS	43.93	69.40	13.30	2.30	38.00	6.89	
MAIT	291.44	593.40	105.00	27.30	30.00	8.64	
BROADVIEW	131.65	193.10	31.50	1.50	41.00	9.16	
CLYDE	142.65	299.60	51.70	11.80	29.00	10.03	
CHURCHILL	97.63	368.90	67.90	21.80	16.00	11.16	
KIRK	236.29	628.00	119.70	31.40	23.00	13.15	
TOTAL	1904.92	4447.90	789.10				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV21A
RICH	69.10	598.00	158.10	44.80	7.00	2.64	
ALPINE	116.28	201.10	32.90	1.40	35.00	3.18	
CROY	68.81	119.10	18.70	1.90	35.00	3.50	
CONN	53.62	152.80	22.30	4.20	21.00	3.87	
LF	67.51	117.80	16.70	1.90	34.00	4.15	
EDGE	188.82	368.40	51.50	8.80	31.00	5.00	
W. S	254.47	549.00	77.70	9.90	28.00	6.30	
FAIRLAWN	70.98	100.80	17.00	0.90	42.00	6.58	
CAL. SC	63.30	87.30	16.50	1.00	44.00	6.86	
IROQUOUS	45.32	68.90	12.80	1.80	40.00	7.07	
MAIT	280.26	466.60	85.90	14.60	36.00	8.50	
BROADVIEW	135.96	201.20	31.80	2.60	41.00	9.03	
CLYDE	151.67	411.20	66.70	20.70	22.00	10.14	
CHURCHILL	102.43	395.10	69.30	22.70	16.00	11.30	
KIRK	258.79	638.90	111.10	25.90	24.00	13.15	
TOTAL	1927.41	4476.20	789.00				

LINK	VEH KM	VEH MIN	TT (SECS)	DEL S/V	SPEED	CUM TT MIN	NV22A
RICH	70.71	565.60	146.30	47.70	8.00	2.44	
ALPINE	115.68	189.20	31.10	1.60	37.00	2.96	
CROY	69.53	249.80	38.80	14.60	17.00	3.60	
CONN	53.75	165.50	24.00	4.50	20.00	4.00	
LF	67.51	121.30	17.20	1.60	33.00	4.29	
EDGE	180.89	344.90	50.30	8.00	32.00	5.13	
W. S	255.07	511.90	72.30	9.30	30.00	6.33	
FAIRLAWN	72.98	106.90	17.60	1.30	41.00	6.63	
CAL. SC	64.73	91.10	16.80	1.10	43.00	6.91	
IROQUOUS	46.30	76.10	13.80	2.60	37.00	7.14	
MAIT	286.26	522.30	94.10	16.60	33.00	8.71	
BROADVIEW	136.66	211.70	33.30	3.10	39.00	9.26	
CLYDE	143.47	322.40	55.30	15.70	27.00	10.18	
CHURCHILL	99.43	363.50	65.70	20.70	16.00	11.28	
KIRK	234.79	571.60	109.60	28.60	25.00	13.10	
TOTAL	1897.76	4413.80	786.20				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV23A
RICH	71.63	547.50	139.80	43.20	8.00	2.33	
ALPINE	117.58	192.70	31.20	1.50	37.00	2.85	
CROY	69.89	114.50	17.70	1.60	37.00	3.15	
CONN	55.96	159.40	22.20	4.10	21.00	3.52	
LF	71.35	127.60	17.20	1.50	34.00	3.80	
EDGE	189.25	366.20	51.10	8.90	31.00	4.65	
W. S	254.47	549.70	77.80	9.90	28.00	5.95	
FAIRLAWN	73.38	107.10	17.50	1.30	41.00	6.24	
CAL. SC	66.13	94.30	17.00	1.10	42.00	6.53	
IROQUOUS	46.30	75.20	13.70	2.50	37.00	6.75	
MAIT	288.86	504.20	90.00	17.00	34.00	8.25	
BROADVIEW	138.13	212.60	33.00	2.90	39.00	8.80	
CLYDE	144.70	325.70	55.40	15.60	27.00	9.73	
CHURCHILL	99.43	332.00	60.00	17.00	18.00	10.73	
KIRK	231.04	621.10	121.00	28.00	22.00	12.74	
TOTAL	1918.10	4329.80	764.60				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV24A
RICH	72.54	578.30	145.80	42.20	8.00	2.43	
ALPINE	118.90	186.40	30.00	1.30	38.00	2.93	
CROY	70.43	129.10	19.80	2.10	33.00	3.26	
CONN	56.49	163.70	22.60	3.30	21.00	3.64	
LF	71.67	122.80	16.40	1.20	35.00	3.91	
EDGE	195.86	368.20	49.60	7.50	32.00	4.74	
W. S	265.27	535.70	72.70	8.10	30.00	5.95	
FAIRLAWN	74.38	117.20	18.90	2.60	38.10	6.26	
CAL. SC	64.33	91.40	17.00	1.10	42.20	6.55	
IROQUOUS	45.60	77.40	14.20	2.90	35.00	6.78	
MAIT	287.14	481.80	86.50	14.70	36.00	8.23	
BROADVIEW	132.40	199.20	32.30	2.90	39.90	8.76	
CLYDE	145.52	369.80	62.30	20.20	23.60	9.80	
CHURCHILL	97.33	395.10	72.90	25.90	14.80	11.02	
KIRK	221.28	576.10	117.20	31.60	23.00	12.97	
TOTAL	1919.14	4392.20	778.20				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV25A
RICH	77.11	565.80	134.20	38.50	8.00	2.24	
ALPINE	121.00	204.20	32.10	1.50	36.00	2.77	
CROY	71.87	133.40	20.00	2.30	32.00	3.11	
CONN	55.83	167.40	23.40	4.30	20.00	3.50	
LF	70.87	119.60	16.20	1.50	36.00	3.77	
EDGE	189.70	375.80	52.30	10.10	30.00	4.64	
W. S	267.07	555.80	74.90	8.40	29.00	5.89	
FAIRLAWN	77.18	126.30	19.60	2.80	37.00	6.21	
CAL. SC	65.53	92.10	16.80	1.00	43.00	6.49	
IROQUOUS	46.30	72.10	13.10	2.20	39.00	6.71	
MAIT	279.40	488.10	90.00	17.30	34.00	8.21	
BROADVIEW	129.51	199.30	33.00	3.00	39.00	8.76	
CLYDE	144.29	403.30	68.70	20.80	22.00	9.91	
CHURCHILL	96.43	357.80	66.70	21.40	16.00	11.02	
KIRK	240.79	629.80	117.70	27.00	23.00	12.98	
TOTAL	1932.88	4490.80	778.70				

LINK	VEH KM	VEH MIN	TT SECS	DEL (S/V)	SPEED	CUM TT MIN	NV26A
RICH	73.15	593.90	148.50	43.90	7.00	2.48	
ALPINE	117.86	187.80	30.30	1.30	38.00	2.98	
CROY	70.97	123.30	18.80	2.30	35.00	3.29	
CONN	56.49	177.40	24.50	4.40	19.00	3.70	
LF	71.03	126.40	17.10	1.60	34.00	3.99	
EDGE	191.90	400.90	55.20	9.70	29.00	4.91	
W. S	252.06	505.90	72.30	8.90	30.00	6.11	
FAIRLAWN	71.73	104.70	17.50	1.30	41.00	6.40	
CAL. SC	65.53	91.80	16.70	1.10	43.00	6.68	
IROQUOUS	45.32	67.90	12.60	1.70	40.00	6.89	
MAIT	289.72	502.50	89.50	17.20	35.00	8.38	
BROADVIEW	138.14	207.60	32.30	2.40	40.00	8.92	
CLYDE	147.57	428.10	71.30	25.40	21.00	10.11	
CHURCHILL	103.33	450.10	78.30	26.80	14.00	11.42	
KIRK	228.04	599.80	118.40	28.60	23.00	13.39	
TOTAL	1922.84	4568.10	803.30				

SUMMARISED WB CARLING AVE MOES

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NVOA
CHURCHILL						
CLYDE	36.53	95.10	46.80	17.70	23.00	
BROADV	27.87	84.60	74.60	29.60	19.80	
MAIT	27.70	43.50	33.90	4.00	38.30	
IROQUOUS	51.57	64.60	64.60	6.40	48.00	
CARL SC	7.97	10.90	11.40	0.20	44.00	
FAIRLAWN	12.17	24.80	24.40	7.90	30.00	
WO. S	15.40	59.90	46.70	26.10	15.00	
EDGE	44.41	64.80	52.50	7.10	41.00	
L.F.	34.77	47.50	36.10	2.40	44.00	
CONN	13.28	23.60	17.10	4.00	34.00	
CROY	10.40	27.00	20.20	8.60	23.00	
ALPINE	13.31	19.50	15.80	1.10	41.00	
RICH	26.88	75.10	53.60	20.20	23.00	
TOTAL	322.26	640.90	497.70			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV1A
CHURCHILL						
CLYDE	35.33	130.60	66.40	13.20	16.00	
BROADV	23.77	53.60	55.30	24.00	27.00	
MAIT	26.62	58.50	47.40	16.50	27.00	
IROQUOUS	44.69	59.50	68.60	9.30	45.00	
CARL SC	6.71	10.40	13.00	0.80	39.00	
FAIRLAWN	11.57	33.50	34.70	17.10	21.00	
WO. S	14.60	86.10	70.80	44.90	10.00	
EDGE	48.01	69.00	51.80	6.00	42.00	
L.F.	35.65	47.00	34.80	3.10	41.00	
CONN	13.60	26.20	18.50	5.40	31.00	
CROY	10.66	23.60	17.20	6.60	27.00	
ALPINE	14.57	20.30	15.00	1.00	43.00	
RICH	28.48	105.20	70.90	38.10	16.00	
TOTAL	314.26	723.50	564.40			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV2A
CHURCHILL						
CLYDE	41.32	120.90	52.60	23.10	21.00	
BROADV	28.69	60.60	51.90	15.90	28.00	
MAIT	27.70	67.50	52.60	22.10	25.00	
IROQUOUS	49.84	67.10	69.40	8.60	45.00	
CARL SC	7.41	11.10	13.00	0.90	39.00	
FAIRLAWN	12.97	38.80	35.80	16.10	20.00	
WO. S	15.40	102.60	79.90	50.90	9.00	
EDGE	48.01	70.60	53.00	8.20	41.00	
L.F.	36.09	47.10	34.50	2.00	46.00	
CONN	14.56	25.70	17.00	4.00	34.00	
CROY	12.23	26.70	17.00	5.50	28.00	
ALPINE	16.74	22.70	14.60	0.70	44.00	
RICH	29.12	103.20	68.10	35.70	17.00	
TOTAL	340.08	764.60	559.40			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV3A
CHURCHILL						
CLYDE	33.53	85.40	45.80	16.70	23.60	
BROADV	23.77	45.70	47.30	15.90	31.00	
MAIT	24.46	49.00	43.30	12.20	30.00	
IROQUOUS	48.13	61.50	65.90	6.30	47.00	
CARL SC	7.41	11.00	12.40	0.60	41.00	
FAIRLAWN	12.77	44.80	42.00	21.40	17.00	
WO. S	15.40	59.00	46.00	25.60	16.00	
EDGE	51.61	70.60	49.20	5.50	44.00	
L.F.	37.85	49.70	34.70	2.50	46.00	
CONN	13.92	27.50	19.00	6.10	30.00	
CROY	10.53	25.00	18.50	7.70	25.00	
ALPINE	14.21	20.70	15.70	1.40	41.00	
RICH	24.52	76.10	53.10	20.30	22.00	
TOTAL	318.11	626.00	492.90			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV4A
CHURCHILL						
CLYDE	36.83	112.00	54.70	25.20	19.70	
BROADV	26.64	60.10	55.40	20.60	27.00	
MAIT	24.46	43.40	38.30	8.20	34.00	
IROQUOUS	46.40	58.30	64.80	4.90	48.00	
CARL SC	6.57	10.50	13.40	1.40	37.00	
FAIRLAWN	11.57	36.30	37.50	19.70	19.00	
WO. S	16.00	101.20	75.90	50.50	9.00	
EDGE	49.21	65.10	47.70	4.50	45.00	
L.F.	37.41	48.20	34.00	3.00	47.00	
CONN	14.08	26.00	17.80	4.20	32.00	
CROY	11.45	25.20	17.20	5.80	27.00	
ALPINE	15.48	21.60	15.00	1.10	43.00	
RICH	27.52	101.80	71.10	37.80	16.00	
TOTAL	323.62	709.70	542.80			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV5A
CHURCHILL						
CLYDE	35.93	102.60	51.30	23.80	21.00	
BROADV	23.36	46.90	49.40	16.50	30.00	
MAIT	24.46	44.40	39.10	8.10	33.00	
IROQUOUS	45.55	58.30	66.10	6.10	47.00	
CARL SC	6.99	10.90	13.00	1.00	39.00	
FAIRLAWN	12.57	40.20	38.30	20.80	19.00	
WO. S	14.20	95.10	60.40	50.70	9.00	
EDGE	49.21	68.40	50.10	5.30	43.00	
L.F.	39.17	53.20	35.80	2.00	44.00	
CONN	14.88	31.60	20.40	6.40	28.00	
CROY	11.58	22.50	15.20	4.10	31.00	
ALPINE	15.11	20.00	14.30	0.60	45.00	
RICH	28.80	102.90	68.60	34.70	17.00	
TOTAL	321.81	697.00	542.00			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV6A
CHURCHILL						
CLYDE	36.82	116.40	56.80	26.80	19.00	
BROADV	25.00	53.10	52.20	18.90	28.00	
MAIT	26.26	45.40	37.30	7.10	35.00	
IROQUOUS	48.13	62.50	67.00	6.80	46.00	
CARL SC	7.27	12.30	14.20	1.90	35.00	
FAIRLAWN	10.97	31.20	34.00	15.70	21.00	
WO. S	15.00	102.40	81.90	55.40	9.00	
EDGE	48.61	67.30	49.90	4.70	43.00	
L.F.	36.97	48.60	34.70	2.00	46.00	
CONN	14.24	31.50	21.30	7.50	27.00	
CROY	11.06	23.20	16.40	5.20	29.00	
ALPINE	14.93	20.00	14.50	0.80	45.00	
RICH	27.84	104.20	71.90	36.50	17.00	
TOTAL	323.10	718.10	552.10			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV7A
CHURCHILL						
CLYDE	35.93	108.60	54.30	25.20	20.00	
BROADV	25.82	65.00	61.90	26.80	24.00	
MAIT	24.46	42.60	37.60	7.00	34.00	
IROQUOUS	44.68	57.90	66.80	7.10	46.00	
CARL SC	6.71	10.30	12.90	0.90	39.00	
FAIRLAWN	11.37	35.50	37.40	19.00	19.00	
WO. S	16.20	104.70	77.60	51.80	9.30	
EDGE	49.21	68.00	49.80	5.50	43.00	
L.F.	36.53	48.50	35.00	2.40	45.00	
CONN	13.76	27.90	19.50	5.00	30.00	
CROY	10.79	22.10	16.00	4.70	29.00	
ALPINE	15.48	21.30	14.80	1.00	44.00	
RICH	28.80	105.00	70.00	35.60	17.00	
TOTAL	319.74	717.40	553.60			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV8A
CHURCHILL						
CLYDE	36.53	101.30	49.80	21.60	22.00	
BROADV	25.00	51.20	50.40	16.80	29.00	
MAIT	26.62	47.20	38.20	7.20	34.00	
IROQUOUS	47.27	60.70	66.20	6.50	47.00	
CARL SC	7.41	10.70	12.10	0.40	42.00	
FAIRLAWN	13.17	39.50	35.90	18.80	20.00	
WO. S	14.40	94.30	78.50	50.30	9.00	
EDGE	49.21	67.20	49.10	3.90	44.00	
L.F.	38.73	55.40	37.80	3.00	42.00	
CONN	14.24	23.30	15.70	2.10	37.00	
CROY	11.84	24.90	16.40	5.20	29.00	
ALPINE	15.84	21.70	14.80	0.70	44.00	
RICH	27.84	99.20	68.40	31.90	17.00	
TOTAL	328.10	696.60	533.30			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV9A
CHURCHILL						
CLYDE	34.83	95.60	46.60	19.60	23.00	
BROADV	24.18	51.80	52.70	18.40	28.00	
MAIT	26.26	47.40	38.90	7.50	33.00	
IROQUOUS	46.41	59.80	66.50	6.40	47.00	
CARL SC	6.85	10.10	12.30	0.80	41.00	
FAIRLAWN	12.17	42.40	41.70	23.60	17.00	
WO. S	14.60	93.70	77.00	47.90	9.00	
EDGE	49.21	70.10	51.30	5.50	42.00	
L.F.	38.73	52.70	35.90	2.50	44.00	
CONN	14.24	26.90	18.10	4.80	32.00	
CROY	11.19	22.40	15.60	4.60	30.00	
ALPINE	15.84	21.20	14.40	0.70	45.00	
RICH	29.12	106.90	70.50	36.00	16.00	
TOTAL	325.63	701.00	541.50			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV10A
CHURCHILL						
CLYDE	38.33	125.30	58.70	27.60	18.00	
BROADV	27.87	69.10	61.00	25.00	24.00	
MAIT	24.82	44.80	38.90	8.10	33.00	
IROQUOUS	44.69	56.60	65.50	5.10	47.00	
CARL SC	6.85	10.20	12.50	0.90	40.00	
FAIRLAWN	12.17	38.60	38.00	20.10	19.00	
WO. S	15.80	100.40	76.30	48.30	9.00	
EDGE	48.01	68.40	51.30	5.80	42.00	
L.F.	36.09	47.50	34.80	2.10	46.00	
CONN	13.60	27.60	19.50	4.90	30.00	
CROY	10.92	22.60	16.20	4.40	29.00	
ALPINE	14.93	20.70	14.90	1.10	43.00	
RICH	27.52	104.60	73.00	37.80	16.00	
TOTAL	321.60	736.60	560.60			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV11A
CHURCHILL						
CLYDE	35.63	114.00	57.50	27.80	18.80	
BROADV	25.41	51.00	49.30	16.40	30.00	
MAIT	27.34	49.00	39.40	8.70	33.00	
IROQUOUS	47.27	60.60	66.10	6.40	47.00	
CARL SC	6.71	10.20	12.70	0.60	39.00	
FAIRLAWN	11.51	34.50	35.70	17.90	20.00	
WO. S	14.00	91.60	78.50	48.60	9.00	
EDGE	46.81	67.00	51.50	5.50	42.00	
L.F.	36.97	51.30	36.70	2.50	43.00	
CONN	13.76	25.50	17.80	4.10	32.00	
CROY	11.45	24.20	16.50	5.10	28.00	
ALPINE	16.02	22.00	14.80	0.70	44.00	
RICH	28.80	104.70	69.80	34.20	17.00	
TOTAL	321.68	705.60	546.30			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV12A
CHURCHILL						
CLYDE	37.13	105.60	51.10	22.40	21.00	
BROADV	25.41	49.60	48.00	14.90	31.00	
MAIT	26.26	47.90	39.30	8.10	33.00	
IROQUOUS	43.83	55.40	65.20	5.40	48.00	
CARL SC	6.71	9.90	12.30	0.70	41.00	
FAIRLAWN	11.17	37.90	40.60	21.70	18.00	
WO. S	14.20	91.60	77.40	51.10	9.00	
EDGE	47.41	66.30	50.40	5.70	43.00	
L.F.	36.09	46.30	33.90	2.40	47.00	
CONN	13.44	23.20	16.60	3.80	35.00	
CROY	11.06	23.30	16.50	5.20	28.00	
ALPINE	15.29	21.00	14.80	0.80	44.00	
RICH	27.52	101.00	70.50	34.70	16.00	
TOTAL	315.52	679.00	536.60			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV13A
CHURCHILL						
CLYDE	37.43	137.40	66.00	33.20	16.00	
BROADV	24.18	48.50	49.40	16.70	30.00	
MAIT	25.90	45.10	37.60	7.00	34.00	
IROQUOUS	48.13	63.30	67.90	7.70	46.00	
CARL SC	7.27	11.10	12.90	0.40	39.00	
FAIRLAWN	12.17	30.80	30.20	13.20	24.00	
WO. S	14.00	84.50	72.40	46.60	10.00	
EDGE	43.10	62.90	52.40	5.90	41.00	
L.F.	33.45	43.80	34.60	2.30	46.00	
CONN	12.48	22.60	17.40	4.30	32.00	
CROY	9.88	19.80	15.60	5.00	30.00	
ALPINE	14.21	19.90	15.10	1.20	43.00	
RICH	28.16	106.90	72.90	37.70	16.00	
TOTAL	310.36	696.60	544.40			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV14A
CHURCHILL						
CLYDE	35.63	87.90	44.30	17.50	24.00	
BROADV	25.41	57.00	55.20	20.20	27.00	
MAIT	24.82	42.20	36.70	6.30	35.00	
IROQUOUS	44.68	55.90	64.50	4.70	48.00	
CARL SC	6.29	9.40	12.50	1.00	40.00	
FAIRLAWN	10.77	33.50	37.20	19.00	19.00	
WO. S	15.60	91.30	70.20	47.10	10.00	
EDGE	48.61	67.30	49.80	4.20	43.00	
L.F.	36.97	50.90	36.40	2.90	44.00	
CONN	13.60	23.30	16.40	3.10	35.00	
CROY	11.45	22.40	15.30	4.10	31.00	
ALPINE	15.48	21.80	15.20	1.40	43.00	
RICH	26.56	103.20	74.60	37.90	15.00	
TOTAL	315.87	666.10	528.30			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV15A
CHURCHILL						
CLYDE	39.23	119.10	54.50	24.80	20.00	
BROADV	27.46	65.40	58.60	22.30	25.00	
MAIT	25.18	44.50	38.20	7.30	34.00	
IROQUOUS	47.26	60.80	66.40	6.40	47.00	
CARL SC	6.71	11.40	14.20	2.00	35.00	
FAIRLAWN	11.17	34.10	36.50	18.80	20.00	
WO. S	15.60	91.50	70.40	48.90	10.00	
EDGE	46.21	64.10	49.90	5.30	43.00	
L.F.	36.09	48.40	35.40	2.90	45.00	
CONN	13.12	23.00	16.80	4.10	34.00	
CROY	10.92	22.20	15.80	4.30	30.00	
ALPINE	14.75	20.30	14.80	1.10	44.00	
RICH	27.52	102.90	71.80	36.90	16.00	
TOTAL	321.22	707.70	543.30			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV16A
CHURCHILL						
CLYDE	37.43	100.10	48.00	20.00	22.00	
BROADV	27.05	64.20	58.30	22.80	25.00	
MAIT	24.46	41.70	36.60	6.60	35.00	
IROQUOUS	47.26	59.00	64.40	5.20	48.00	
CARL SC	7.27	11.00	12.70	0.70	40.00	
FAIRLAWN	11.97	36.00	36.00	18.10	20.00	
WO. S	15.00	91.30	73.00	46.50	10.00	
EDGE	42.61	59.10	50.00	4.80	43.00	
L.F.	33.01	43.10	34.50	2.10	46.00	
CONN	12.16	21.60	17.00	4.00	34.00	
CROY	9.88	19.60	15.50	4.00	30.00	
ALPINE	14.03	19.40	14.90	1.10	44.00	
RICH	25.60	93.40	70.10	37.70	16.00	
TOTAL	307.73	659.50	531.00			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV17A
CHURCHILL						
CLYDE	35.33	93.30	47.40	20.10	22.70	
BROADV	25.41	51.00	49.30	16.40	30.00	
MAIT	25.90	46.00	38.30	8.00	34.00	
IROQUOUS	49.85	65.40	67.70	8.10	46.00	
CARL SC	7.83	11.90	12.80	0.90	39.00	
FAIRLAWN	13.17	43.70	39.70	21.60	18.00	
WO. S	16.80	114.20	81.50	52.40	9.00	
EDGE	47.41	69.90	53.10	6.60	41.00	
L.F.	35.65	47.20	35.00	2.20	45.00	
CONN	13.60	24.00	16.90	3.50	34.00	
CROY	10.79	22.40	16.20	5.00	29.00	
ALPINE	14.03	19.40	14.90	0.80	43.00	
RICH	27.52	100.00	69.70	34.00	17.00	
TOTAL	323.29	708.40	542.50			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV18A
CHURCHILL						
CLYDE	36.53	127.70	62.80	30.60	17.20	
BROADV	23.77	42.50	43.90	12.60	34.00	
MAIT	25.90	46.60	38.80	8.00	33.00	
IROQUOUS	51.57	71.70	71.70	10.80	43.00	
CARL SC	7.55	12.80	14.20	1.60	36.00	
FAIRLAWN	12.57	36.10	34.30	16.00	21.00	
WO. S	15.40	95.10	74.10	44.60	10.00	
EDGE	41.41	59.70	51.90	6.00	42.00	
L.F.	32.13	43.50	35.80	2.80	44.00	
CONN	12.16	22.90	18.10	4.70	32.00	
CROY	9.88	20.30	16.00	5.60	29.00	
ALPINE	14.21	19.50	14.80	1.10	44.00	
RICH	28.48	106.50	71.80	37.10	16.00	
TOTAL	311.56	704.90	548.20			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV19A
CHURCHILL						
CLYDE	34.73	80.60	41.70	15.30	25.80	
BROADV	25.82	49.80	47.40	14.30	31.00	
MAIT	27.70	47.80	37.20	6.50	35.00	
IROQUOUS	54.15	74.80	71.20	10.00	43.00	
CARL SC	7.97	13.50	14.20	1.40	35.00	
FAIRLAWN	12.57	39.30	37.50	18.20	19.00	
WO. S	16.40	108.30	79.30	47.90	9.00	
EDGE	43.21	63.30	52.80	6.80	41.00	
L.F.	32.13	42.90	35.20	3.00	45.00	
CONN	12.16	23.40	18.50	5.10	31.00	
CROY	9.75	19.80	15.80	4.90	30.00	
ALPINE	14.57	19.80	14.60	0.80	44.00	
RICH	28.48	106.30	71.60	36.60	16.00	
TOTAL	319.64	689.60	537.00			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV20A
CHURCHILL						
CLYDE	35.93	102.30	51.10	22.60	21.10	
BROADV	22.13	42.60	47.40	14.80	31.00	
MAIT	25.54	60.40	51.10	18.80	25.00	
IROQUOUS	45.55	64.20	72.70	12.00	43.00	
CARL SC	6.99	11.80	14.20	1.70	36.00	
FAIRLAWN	12.57	36.40	34.70	17.20	21.00	
WO. S	15.80	88.50	67.20	44.10	11.00	
EDGE	44.41	64.90	52.60	7.00	41.00	
L.F.	34.77	46.90	35.60	2.10	45.00	
CONN	13.12	28.30	20.70	6.70	28.00	
CROY	10.27	21.80	16.60	5.40	28.00	
ALPINE	14.57	20.60	15.30	1.10	42.00	
RICH	27.84	103.40	71.30	35.50	16.00	
TOTAL	309.49	692.10	550.50			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV21A
CHURCHILL						
CLYDE	36.53	95.50	47.00	18.70	23.00	
BROADV	27.46	51.40	46.00	14.40	32.00	
MAIT	27.70	49.40	38.40	8.20	34.00	
IROQUOUS	50.70	66.70	67.90	7.50	46.00	
CARL SC	7.13	11.20	13.20	1.50	38.00	
FAIRLAWN	12.17	39.60	38.90	20.40	19.00	
WO. S	16.20	84.90	62.90	39.80	11.00	
EDGE	42.01	57.50	49.30	5.00	44.00	
L.F.	32.13	43.20	35.50	3.10	45.00	
CONN	12.00	23.40	18.70	4.90	31.00	
CROY	9.88	19.70	15.50	4.10	30.00	
ALPINE	13.67	19.30	15.20	1.20	43.00	
RICH	25.28	94.30	71.60	39.10	16.00	
TOTAL	312.86	656.10	520.10			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV22A
CHURCHILL						
CLYDE	35.93	99.10	49.50	21.50	22.00	
BROADV	24.18	47.20	48.00	15.10	31.00	
MAIT	26.26	45.90	37.70	6.90	34.00	
IROQUOUS	51.57	68.30	68.30	8.10	45.00	
CARL SC	7.69	12.00	13.10	1.10	39.00	
FAIRLAWN	12.97	40.60	37.50	19.30	19.00	
WO. S	16.00	72.70	54.50	32.40	13.00	
EDGE	45.61	65.60	51.80	6.00	42.00	
L.F.	35.65	48.60	36.00	2.10	44.00	
CONN	13.12	23.90	17.50	3.60	33.00	
CROY	10.79	50.30	36.40	22.00	13.00	
ALPINE	15.48	30.10	21.00	4.50	31.00	
RICH	28.16	122.70	83.70	47.30	14.00	
TOTAL	323.41	727.00	555.00			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV23A
CHURCHILL						
CLYDE	36.83	100.20	48.90	20.60	22.10	
BROADV	24.18	48.50	49.30	16.50	30.00	
MAIT	26.26	44.90	36.90	6.40	35.00	
IROQUOUS	53.29	73.00	70.60	10.30	44.00	
CARL SC	7.83	12.80	17.70	1.60	37.00	
FAIRLAWN	13.17	42.50	38.60	19.90	19.00	
WO. S	15.60	72.20	55.50	32.70	13.00	
EDGE	43.81	61.30	50.30	5.60	43.00	
L.F.	32.57	46.00	37.30	3.10	43.00	
CONN	12.32	24.10	18.80	4.60	31.00	
CROY	9.75	18.70	14.90	4.20	31.00	
ALPINE	13.67	18.50	14.60	0.80	44.00	
RICH	26.88	96.90	69.20	33.40	17.00	
TOTAL	316.16	659.60	518.60			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV24A
CHURCHILL						
CLYDE	35.03	86.50	44.40	17.60	24.00	
BROADV	26.23	54.50	51.00	16.80	29.00	
MAIT	25.54	45.30	38.30	7.30	34.00	
IROQUOUS	52.43	72.20	71.00	9.90	44.00	
CARL SC	7.69	13.30	14.50	2.10	35.00	
FAIRLAWN	11.97	38.80	38.80	18.80	19.00	
WO. S	15.80	70.80	53.70	31.20	13.00	
EDGE	44.41	61.70	50.00	5.10	43.00	
L.F.	33.01	45.80	36.60	2.90	43.00	
CONN	12.32	25.40	19.80	5.10	29.00	
CROY	9.75	18.50	14.80	4.40	32.00	
ALPINE	14.75	19.80	14.50	0.90	45.00	
RICH	28.16	103.60	70.60	34.00	16.00	
TOTAL	317.09	656.20	518.00			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV25A
CHURCHILL						
CLYDE	37.13	106.50	51.50	22.10	21.00	
BROADV	26.64	59.50	55.00	21.00	27.00	
MAIT	24.82	43.90	38.20	7.90	34.00	
IROQUOUS	48.98	63.00	66.30	7.10	47.00	
CARL SC	7.13	11.30	13.30	1.40	38.00	
FAIRLAWN	11.97	35.80	35.80	17.20	20.00	
WO. S	15.80	65.50	49.80	28.80	15.00	
EDGE	48.01	65.60	49.20	5.00	44.00	
L.F.	36.09	51.30	37.60	2.90	42.00	
CONN	12.96	24.80	18.30	4.80	31.00	
CROY	10.66	19.60	14.30	3.40	33.00	
ALPINE	14.93	20.90	15.10	1.40	43.00	
RICH	26.56	94.80	68.60	35.90	17.00	
TOTAL	321.68	662.50	513.00			

LINK	VEH KM	VEH MIN	TT SECS	STOP S/V	SPEED	NV26A
CHURCHILL						
CLYDE	34.13	92.20	48.50	20.90	22.00	
BROADV	22.54	41.10	44.80	13.50	33.00	
MAIT	25.18	44.80	38.40	8.00	34.00	
IROQUOUS	51.57	72.50	72.50	10.70	43.00	
CARL SC	7.69	12.90	14.10	1.70	36.00	
FAIRLAWN	12.37	39.00	37.70	18.50	19.00	
WO. S	15.00	69.80	55.80	32.80	13.00	
EDGE	44.41	62.50	50.60	4.60	43.00	
L.F.	33.89	47.20	36.80	3.10	43.00	
CONN	12.48	24.60	18.90	5.10	31.00	
CROY	9.88	18.30	14.40	3.80	33.00	
ALPINE	15.11	20.90	14.90	1.00	44.00	
RICH	27.20	97.20	68.60	30.40	17.00	
TOTAL	311.45	643.00	516.00			

EASTBOUND QUEENSWAY AND EASTBOUND CORRIDOR PERFORMANCE MEASURES

	EB QUEENSWAY			EASTBOUND CORRIDOR		
	VEH KM	VEH HRS	TRV. TIME	VEH KM	VEH HRS	AV TRVT
FWY0A	8680.20	308.41	803.00	10236.24	369.54	13.08
FWY1A	8633.70	141.06	366.00	10669.28	235.51	10.90
FWY2A	8696.20	146.20	376.00	10855.15	243.07	10.73
FWY3A	8754.90	151.37	387.00	10815.77	254.47	11.62
FWY4A	8834.30	153.02	388.00	10751.48	239.44	10.95
FWY5A	8857.20	163.10	412.00	10867.92	249.05	10.56
FWY6A	8897.50	165.04	415.00	10878.14	245.66	10.28
FWY7A	8915.00	168.86	425.00	10934.88	251.69	10.36
FWY8A	8934.50	172.08	431.00	10927.38	252.59	10.34
FWY9A	8964.50	173.37	433.00	10886.46	253.13	10.64
FWY10A	8968.40	178.03	444.00	10934.11	257.63	10.54
FWY11A	8996.30	179.53	446.00	10940.00	256.56	10.39
FWY12A	8993.50	184.72	460.00	10962.15	263.91	10.60
FWY13A	9019.30	186.31	463.00	10956.36	263.39	10.52
FWY14A	8993.00	193.68	484.00	10962.08	272.88	10.79
FWY15A	9015.90	195.32	486.00	10958.98	274.69	10.97
FWY16A	9068.00	206.28	511.00	10932.79	278.85	10.82
FWY17A	8997.40	204.27	510.00	10913.18	277.94	10.77
FWY18A	8998.00	218.77	546.00	10948.68	293.58	11.06
FWY19A	9013.30	220.00	548.00	10869.19	290.07	11.01
FWY20A	9027.00	222.46	551.00	10931.92	296.59	11.17
FWY21A	9041.90	225.03	558.00	10969.31	299.63	11.23
FWY22A	9055.10	227.92	564.00	10952.86	301.48	11.25
FWY23A	9064.00	228.58	565.00	10982.10	300.74	11.08
FWY24A	9075.20	231.36	571.00	10994.34	304.56	11.24
FWY25A	9088.00	232.44	574.00	11020.88	307.29	11.27
FWY26A	9092.00	235.29	579.00	11014.84	311.43	11.52

EASTBOUND CORRIDOR & COMBINED CORRIDOR PERFORMANCE MEASURES

	WB CORRIDOR			COMBINED CORRIDOR	
	VEH KM	VEH HRS	AV TRVT	VEH KM	VEH HRS
FWY0A	4448.86	52.14	11.59	14685.10	421.68
FWY1A	4440.86	53.52	12.71	15110.14	289.03
FWY2A	4466.68	54.20	12.62	15321.83	297.27
FWY3A	4444.71	51.89	11.52	15260.48	306.36
FWY4A	4450.22	53.29	12.35	15201.70	292.73
FWY5A	4448.41	53.08	12.33	15316.33	302.13
FWY6A	4449.70	53.42	12.50	15327.84	299.08
FWY7A	4446.34	53.42	12.53	15381.22	305.11
FWY8A	4454.70	53.07	12.19	15382.08	305.66
FWY9A	4452.20	53.14	12.32	15338.66	306.27
FWY10A	4448.20	53.74	12.64	15382.31	311.37
FWY11A	4448.28	53.22	12.41	15388.28	309.78
FWY12A	4442.12	52.78	12.24	15404.27	316.69
FWY13A	4436.96	53.07	12.37	15393.32	316.46
FWY14A	4442.47	52.56	12.11	15404.55	325.44
FWY15A	4447.82	53.26	12.36	15406.80	327.94
FWY16A	4434.33	52.45	12.15	15367.12	331.30
FWY17A	4447.89	53.27	12.34	15363.07	331.21
FWY18A	4438.16	53.21	12.44	15386.84	346.79
FWY19A	4446.24	52.95	12.25	15315.43	343.02
FWY20A	4436.00	53.00	12.48	15367.92	349.59
FWY21A	4439.46	52.40	11.97	15408.77	352.03
FWY22A	4450.01	53.58	12.55	15402.87	355.06
FWY23A	4442.76	52.45	11.94	15424.86	353.19
FWY24A	4443.69	52.40	11.93	15438.03	356.96
FWY25A	4448.28	52.50	11.85	15469.16	359.79
FWY26A	4438.05	52.18	11.90	15452.89	363.61

AVERAGE METERING RATES & METERING PLANS

SIMULATION RUN	PINE	PINE	WO	WO	MAIT	KIRK	KIRK	AVERAGE METERING RATE
	8402 101	8403 102	8405 201	8406 202	8408 301	8410 401	8411 402	
NV0	345	1148	220	1092	692	826	244	
NV1	345	778	190	190	190	800	244	430
NV2	345	808	195	215	220	800	244	448
NV3	345	838	200	240	250	800	244	466
NV4	345	868	205	265	280	800	244	484
NV5	345	898	210	290	310	800	244	502
NV6	345	900	215	315	340	800	244	514
NV7	345	900	220	340	370	800	244	526
NV8	345	900	220	370	400	800	244	538
NV9	345	900	220	400	430	800	244	550
NV10	345	900	220	430	460	800	244	562
NV11	345	900	220	460	490	800	244	574
NV12	345	900	220	490	520	800	244	586
NV13	345	900	220	520	550	800	244	598
NV14	345	900	220	550	580	800	244	610
NV15	345	900	220	580	610	800	244	622
NV16	345	900	220	610	640	800	244	634
NV17	345	900	220	640	670	800	244	646
NV18	345	900	220	670	692	800	244	656
NV19	345	900	220	700	692	800	244	662
NV20	345	900	220	730	692	800	244	668
NV21	345	900	220	760	692	800	244	674
NV22	345	900	220	790	692	800	244	680
NV23	345	900	220	810	692	800	244	684
NV24	345	900	220	840	692	800	244	690
NV25	345	900	220	870	692	800	244	696
NV26	345	900	220	900	692	800	244	702

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