FREEWAY CAPACITY

INVESTIGATION OF SOME UNRESOLVED ISSUES IN FREEWAY CAPACITY

BY

KWAKU AGYEMANG-DUAH, BSc(Hons)

A Thesis
Submitted to the School of Graduate Studies
in Partial Fulfilment of the Requirements
for the Degree
Master of Arts

McMaster University

April, 1991

MASTER OF ARTS (1991) McMASTER UNIVERSITY
(Geography) Hamilton, Ontario

TITLE: Investigation of Some Unresolved Issues in Freeway Capacity

AUTHOR: Kwaku Agyemang-Duah, BSc(Hons). (Kumasi, Ghana)
M.A. (McMaster University)

SUPERVISOR: Professor Fred L. Hall

NUMBER OF PAGES: viii, 97

ABSTRACT

Three issues related to the capacity of uninterrupted flow facilities are addressed using 52 days of detector data from one freeway, the Queen Elizabeth Way (QEW) in Mississauga, west of Toronto. The first issue is that of a possible capacity drop in the bottleneck after the queue forms upstream. The results show that there is a drop, of 98 passenger cars per hour per lane (pcphpl).

The second issue tackled is the distributional characteristics of queue discharge flow. Daily average morning peak flow rates and the peak 15-minute data were examined. The frequency histogram is not close to a Normal one. The mean and the median, contrarily, are very close to each other. The use of the mean value was therefore deemed appropriate. The distribution of speed was also looked at. At a distance of 1.5 kilometres from the bottleneck, the observed average travel speed of vehicles discharging from an upstream queue was 74 km/hour.

The third issue focuses on the definition of capacity, conceptually and numerically. Thirty-second, 5-minute and peak 15-minute flow data were used. Capacity flows were observed under two different conditions, free-flow and congested, or forced-flow. The findings that evolved from the data analysis indicate that if capacity refers to the highest flows sustained for at least 15 minutes, as suggested in the Highway Capacity Manual (HCM), then it is conceptually sound to define capacity as pre-queue high flow. The practical implications of the capacity concept, however, make it very useful and equally valid to define capacity as the maximum queue discharge flow.

Using the mean values of 30-second flow data, the pre-queue capacity is 2,306 pcphpl which is rounded off to 2,300 pcphpl; for queue discharge flow, it is 2,200 pcphpl. (These values were weighted by the duration whereas the capacity drop value above was not).

ACKNOWLEDGEMENTS

It is good to give thanks to God for the strength and direction in preparing this report. Profound thanks are due to Professor Fred L. Hall who supervised this work alongside his tight schedule with critical acumen. It has been worthwhile studying under him. I wish also to express my gratitude to Professor P.D.M. Macdonald of the Department of Mathematics and Statistics, McMaster University, for his advice on the statistical analysis of the data. A special word of thanks to Lisa Hall and Shi Yong, whose FORTRAN programmes, encouragement and comments were valuable to the completion of the thesis. It is just nice working with you all. To Mr. Mark Fox of the Freeway Traffic Management System at Mississauga, I say thank you for the timely provision of the traffic data. And to the many who assisted in one way or the other but their names are not mentioned here, accept a "blanket" thank you.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

CHAPTER ONE

INTRODUCTION

1.1 STATEMENT OF PROBLEM

Freeways, the only type of transport facility that provides uninterrupted flow conditions, continue to be one of the most widely used transport systems in inter- and intra-city travel. In 1984, the freeway system in the United States, for example, carried about 560 billion vehicle kilometres of travel, approximately 20 per cent of the nation's total, although it constitutes only one per cent of all paved urban and rural highways (McShane and Roess, 1990). This underscores the need for a good understanding of capacity, a concept fundamental to the practical design, operation and management of freeways.

The current debate over the freeway capacity concept makes it still of much interest after so many years of research, dating as far back as the 1930s. But the existing unresolved issues that have prompted the current debate about the capacity concept point to a neglect in this area of research for a long time. This neglect may have very serious consequences for freeway construction and control and hence overall transport policy.

Evidence of this neglect is the continued acceptance for more than four decades of 2,000 passenger cars per hour per lane (pcphpl) as the numerical value of the capacity of a basic section of a freeway (Highway Capacity Manual (HCM), 1950, 1965, 1985). Continued acceptance of the 2,000 value will mean that there has been no effect of the changes in the very factors that are said to affect roadway capacity. In other words, improved geometric designs, vehicle downsizing and the increasing driver experience with freeway driving are implied to have had no impact at all on freeway capacity during the past 40 years. Recent research on multi-lane rural highways has led to change in the value given for capacity on those roads, which has recently been said to be 2,200 pcphpl (Reilly *et al*, 1990). It is clearly time for similar new work on freeways.

The debate on even where to measure capacity is another indicator of a truly neglected, yet vital concept. This may suggest that in the past, the place to look for capacity has been taken for granted. Hence, there is a lack of any good description of where the data were collected for most earlier capacity studies. One aspect of the puzzle over the place to look for capacity is the set of conditions that one has to look for in capacity studies. One key condition is certainly not discussed in most studies : the existence of sufficient demand which is critical to traffic counts. Clearly, this condition cannot be found just anywhere on a freeway and hence the place to look for capacity cannot be just any "point" (HCM, 1985, p.1-3).

Another unclarified issue about capacity relates to the question of capacity drop. Wattleworth (1963) puts forward two conflicting theories about capacity drop. One theory supports a drop in capacity when a queue forms upstream. The other theory does not. Where such a drop can be found is also not certain. Given these contentious issues that are so fundamental to the concept, any current definition of capacity may not be very useful either conceptually or numerically, notwithstanding what is given in the HCM.

A possible reason for this state of affairs is the paucity of data. It is therefore appropriate to expand the data base with information that reflects current thinking about the concept. Apart from expanding the data base, this current study introduces two new approaches in capacity analysis. The first approach is a close comparison of techniques used to identify queue presence, an important indicator of the existence of sufficient demand. This approach is a shift from the speed-flow relationship method which has some serious weaknesses (Persaud and Hurdle, 1988). The other approach deals with the analysis of potential capacity drop. This second approach is a statistical test on the distribution of differences in daily mean flows, before and after a queue, to augment the comparisons of the two daily means by using *t* statistics.

1.2 PURPOSE OF THE STUDY

Three questions are addressed in the study. The first ıt the existence of two capacities in the Thirty-second flow data are used to examine the before (pre-queue) and after an upstream queue ɛue discharge). The second question focuses on :ional characteristics of pre-queue high flows and :ge flows. Daily mean flows and the peak 15- ; were used. Five-minute speed data were used to Level-of-service (LOS) that has been defined for ɔws. The third issue is the definition of he analysis of the nature of the distribution and the capacity drop issue are important to define m a conceptual viewpoint and to determine the lue for capacity.

ATION OF REPORT

:er two, the background of the study is presented. ntially a critical review of the existing notion ɔt of capacity. Chapter three is a discussion of l. It covers a description of the study site, the .on procedures, the time interval for analysis and ɔsition. Data analysis and the methods used for :ed in chapter four. Issues covered in this chapter include determination of the presence of a queue upstream, investigation of any drop in high flows at the onset of a queue and capacity distribution over time. Chapter five contains the discussions of the findings from the analysis and the conclusions.

CHAPTER TWO

STUDY BACKGROUND

In this chapter, a review of the traditional notion of the concept of capacity and how it has led to the current debate over the concept are presented. The four main points discussed are the definition of capacity, its numerical value, capacity drop as queue forms and operating conditions associated with capacity flows.

2.1 DEFINITION OF CAPACITY

The definition of "capacity" has evolved since 1950. The 1950 HCM simply refers to capacity as "a generic expression pertaining to the ability of a roadway to accommodate traffic" (p.5). Perhaps the most critical aspect in the definition of capacity comes when the Manual refers to "practical" capacity as the " maximum number of vehicles that can pass a given point on a roadway ...without the traffic density being so great as to cause unreasonable delay, hazard or restriction of the drivers' freedom to manoeuvre" (p.7). The definition makes implicit the idea of absolute as opposed to relative capacity of uninterrupted flow facilities and is thus very deterministic.

The 1965 HCM, however, avoids this element of determinism when it defines capacity as "the maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway in one direction ... during a given time period under the prevailing roadway and traffic condition" (p.5). Where no time frame is specified, the Manual presumes capacity as an "hourly volume".

There was a slight change in the wording of the 1985 definition: "maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions" (p.1-3). Further in the Manual, capacity was defined as "the maximum sustained 15-minute rate of flow" (p.3-3). The distinctions are the removal of the hourly volume which is replaced by hourly rate and the addition of control conditions, which refer to the various traffic control devices, rules and regulations on a particular facility. For uninterrupted flow facilities, such controls are limited to ramp metering and traffic regulations.

Four main points from the definition of capacity are worth discussing: the term "reasonable expectation"; the maximum number of cars recorded; the place to look for capacity; and the time period for capacity measurement. The discussion on the term "reasonable expectation" focuses on its implication for freeway capacity. For the notion of capacity to be of any practical importance, it should connote something achievable on a daily basis and not necessarily the highest flow rate, which is not likely to be repeatedly achieved. It is possible to record a very high traffic volume that can be described as an isolated, unusual case and therefore of limited importance regarding the concept of capacity. In the capacity analysis of any facility, attention should be focused on the high flow rates that are repetitive. This is aptly put by McShane and Roess (1990, p.192-3):

> capacity is defined on the basis of "reasonable expectancy". That is, a stated capacity for a given facility is a rate of flow that can be repeatedly achieved during every peak period for which sufficient demand exits.

The quotation brings into focus the portion of the capacity distribution (the mean, the median or the mode, for example) that will be reasonable to adopt, bearing in mind its repetitiveness.

The 1950 HCM adoption of the word "generic" implicitly acknowledges the need for some flexibility in the usage of capacity. People's perception of traffic laws and regulations and geometric parameters vary in time and space and so does the rate of flow. There is, however, the tendency to interpret "generic" to mean that capacity as defined in the Manual is applicable to all facilities everywhere. One such case is provided by McShane and Roess (1990,p.193):

> The defined capacity of a facility is the maximum rate of flow which the traffic engineer may be reasonably assured of being achieved day in and day out

anywhere in North America.

The flaw here with respect to the space dimension of the concept is that it disregards the importance of capacity of a type of a facility or of a specific facility.

Another element worthy of mentioning is the time unit used in the definition of capacity. The problem this presents relates to sustainability of flows for long periods. While both the 1950 and 1965 editions of the Manual emphasise traffic counts for at least one hour, the 1985 uses only 15-minute counts of sustained flows. The replacement of the hourly volume by hourly rate based on 15 minutes of traffic counts leaves open the question of whether those high flows can be sustained for a longer period.

The dilemma associated with the assurance of sustained high flows based on 15 minutes is well highlighted by Hall and Agyemang-Duah (1991): 10 out of the 20 cases of traffic data analyzed showed high pre-queue flow rates of between 2,220 passenger cars per hour per lane (pcphpl) and 2,420 pcphpl lasting for or more than 15 minutes. These high pre-queue flow rates could be defined as capacity since they qualify given the definition in the 1985 HCM. The question, however, is whether such flows can be sustained for long periods.

One important but missing qualification in the definition of capacity in all three editions of the HCM is the existence of sufficient demand. Capacity is said to be measured at any "point" or "given section of a lane or roadway". This is true but creates a problem that in a way accounts for some of the present unresolved issues of the concept.

The 1965 HCM talks about four limiting measures that can govern observed volumes. These are the demand placed on the roadway by motorists; the capacity at a point of observation; a point upstream; and a point downstream of a bottleneck (p.5) Translating these limiting measures to find out where to look for capacity shows how hard it is to identify the place to look for capacity. If capacity is to be measured only under certain "ideal" conditions at a particular point or section of the roadway, such a location may not be found everywhere on the freeway. The ideal conditions referred to include regular road users, traffic exclusively passenger cars, a twelve-foot minimum lane width and six-foot lateral clearance (HCM, 1985). These conditions are not sufficient to make it easy to identify the "right" place to look for capacity. It is interesting to note that while the HCM has defined a value for capacity of a "point" on the roadway, there are some sections in the Manual that deal with capacity of specific sections of the freeway.

The requirement for the existence of sufficient demand was included in the quotation above from McShane and Roess (1990, p.192-3). Sufficient demand has two important implications for capacity. One implication is the place to measure capacity. There are two possible ways to find the presence of sufficient demand. One is the presence of a queue, which indicates excess and hence sufficient demand. But the cause of the queue should be ascertained first, because if, for instance there is an incident at an entrance ramp or a lane, the demand that will be created is only in excess of the reduced capacity and not capacity of an unobstructed roadway.

Sufficient demand may also be possible in the absence of a queue. But how this can be known is difficult. The indicators to identify that sufficient demand exits without a queue may not be easy to define. There will therefore be a high degree of personal judgement as to when to take capacity measurement. This is true especially in capacity studies that are based on observed volumes, manual counting and/or photographic methods. Thus earlier capacity studies, for example, the one by Greenshields (1934) which were not based on queue discharge flows or detector data but one or a combination of any of the three methods mentioned are likely to be unreliable.

Clearly, the location to look for capacity cannot be just any "point" on the freeway. Despite the fact that the place to take the measurement is critical for capacity analysis, very little attention has been given to it.

The other implication of the requirement for sufficient demand is the definition of capacity conceptually. One school of thought is of the view that the assurance of sufficient demand for any considerable length of time is when there is a queue. Using data collected in a bottleneck for capacity analysis implies defining capacity as queue discharge flows. Thus Hurdle and Datta (1983) defined capacity as "the average flow through a bottleneck when a steady supply of cars is assured by the existence of an upstream queue". (p.133). Hall and Hall (1990) also support such a view but state it differently: "If a particular location operates in a queue, it must obviously be governed by something that is happening downstream. The place to look for capacity operation, then, is downstream, beyond the queue" (p.7).

The logic underlying this school of thought would have been unquestionably justified had the HCM maintained its "hourly volume" as used in the 1950 edition. The replacement of the hourly volume by the hourly flow rates based on only 15 minutes of traffic count (HCM, 1985), however, undermines the logic for defining capacity as a queue discharge flow. The reason is that it has empirically been shown that higher pre-queue flow rates for more than 15 minutes are possible (Hall and Agyemang-Duah, 1991). Using the maximum sustained 15-minute flows as a criterion in such a case, capacity may be defined as the maximum pre-queue flows.

2.2 THE NUMERICAL VALUE OF CAPACITY

There are two reasons to discuss the numerical value of capacity. The first one is the recent proposal by the HCM to increase the capacity of multi-lane rural highways to 2,200 pcphpl. Chin and May (1991) have, however, demonstrated that under ideal conditions the capacity of multi-lane highways is the same, whether rural or urban. There is, therefore, no justification for limiting the proposed change to multi-lane "rural" highways. The second reason is to analyze the logic behind the lack of change in the numerical value of capacity since 1950 in the wake of changes in traffic control and conditions and traffic composition and vehicle size.

As with the definition of capacity, the three editions of the Manual do not differ much on the numerical value of capacity of a multi-lane highway. There is a continuing consensus on the 2,000 pcphpl. There are, however, some differences in the way it is presented in each of the three editions.

The 1950 Manual states that the largest number of vehicles that can pass a point in a single traffic lane, under ideal conditions is between 2,000 and 2,200 (p.36). Further under the same heading and on the same page, it went on to say that "the basic capacity of multi-lane roads is 2,000 passenger cars per hour per lane". The 1965 Manual gives both a range and an exact value: "the largest number of vehicles that can pass a point one behind the other on a single lane,... averages between 1,900 and 2,200". Later in the Manual, the capacity value was precisely stated as 2,000 pcphpl. The 1985 Manual also gives the 2,000 figure, which is considered to be a national average, a value representative of all freeways.

The term "national average" implicitly means that the freeway capacity of 2,000 pcphpl is achievable on at least most freeways irrespective of the type. The validation of this assertion may be disputed in view of the fact that most capacity studies are based on very limited data with respect to the period for which traffic data were collected. For instance, Greenshields (1934) used less than two hours of data obtained from pictures of moving traffic in isolated cases; Roess, McShane and Pignataro (1979) based their analysis on a "limited number of pilot field surveys" (p.7); Hurdle and Datta (1983), confined their study to three weekdays of morning traffic data. Perhaps, the only recent and extensive studies on freeway capacity are the ones by Hall and Agyemang-Duah (1991) in which 20 good-weather, incident-free weekdays of morning traffic data were used; Chin and May (1991) who used 131 hours of detector data; and Urbanik, Hinshaw and Barnes' work (1991) which involved data from seven different highways in four U.S. cities. Thus talking about "national average" based on very limited data seems not to be very meaningful. As Hall and Agyemang-Duah noted, taking the highest point of the distribution may not be repeatedly achieved most of the time. Hence a "national average" supported by a weak data base needs to be "confirmed" with more data.

Four deductions can be drawn from the numerical value of capacity. First, the continued use of 2,000 pcphpl for the capacity of a multi-lane facility suggests that after four decades, improved roadway, traffic and control conditions are yet to have any impact on freeway capacity. Freeway traffic management systems have been justified, among other things, as increasing capacity (Ministry of Transportation, Ontario, 1990). Nevertheless, freeway capacity has been said by several editions of the HCM to be the same for forty years. Thus it makes no sense to stress prevailing roadway, traffic and control conditions as critical determinants of capacity numerically. It would be interesting to investigate the extent of change in any of the parameters affecting traffic flow on the freeway and then compare the results with the 2,000 pcphpl as given in the Manual.

The second deduction relates to changes in the size of vehicles and the impact on freeway capacity. Dramatic changes have occurred in both vehicle composition in the traffic stream and in vehicle sizes. Compact and sub-compact cars now comprise a greater percentage of traffic on freeways as compared to standard and intermediate cars. More recent data on vehicle dimensions seem to indicate that vehicle sizes have reduced over the past two decades. Woods (1983) reported that the large - small car ratio was 3:1 in 1975; in 1980, it was 1.2:1 and it was projected to be 0.3:1 as from 1985.

The trend in vehicle dimensions is similar to that of car composition. Citing Woods (1983) again, the average length of cars was down by about 2.6 per cent in 1980, a decrease from 579 centimetres (cm) (228 inches) in 1960 to 564cm (222 inches) in 1980. A projected further reduction of 7.6 per cent brought the mean car length to 521cm (205 inches) (1985). There has also been a similar reduction in vehicle width from an average of 213cm (84 inches) in 1965 to a projected 183cm (72 inches) in 1990, about 14.3 per cent reduction. These facts are also substantiated by a similar scenario in the car market. The sale of large cars was down to 13 per cent of total sales and compact and sub-compact car sale jumped to 48 and 39 per cent respectively between 1970 and 1980 (Burtch *et al*,1983). In view of these changes in vehicle sizes and traffic composition on freeways, it is either unacceptable to maintain the 2,000 pcphpl and/or its basis is wrong.

2.3 CAPACITY DROP

There are some studies that have reported a drop in the maximum flow once there is a queue while others have indicated no such drop, making the capacity drop issue a contentious one. Wattleworth (1963) aptly states the contention surrounding capacity drop as follows:

> The question of whether or not the flow downstream of a freeway bottleneck decreases when congestion sets in is currently the subject of much discussion in engineering circles. Research findings support both the yes and no answers to this question.

> Several studies... suggested that perhaps the question did not have a simple yes or no answer (p.15).

The key issue for critical analysis with respect to a drop or no drop in capacity when there is a queue as highlighted in the above remark is the underlying logic. According to one theory, congestion at a bottleneck reduces capacity. Two criticisms can be levelled against this theory. Firstly, its premise is wrong: it does not address capacity in a bottleneck. The second criticism relates to the place to look for capacity. If in the presence of sufficient demand as indicated by a queue (assuming there is no incident downstream and all other criteria set out in the HCM are met), operations downstream are said not to reach capacity, what then will be the best indicator to determine when to measure capacity?

There are other studies on capacity drop that are based on similar wrong premises. One case is provided by Edie (1961) who investigated some aspects of vehicular traffic flow. Focusing on the sudden change in traffic stream from free-flowing to congested conditions, Edie concluded that so far as there is some interference in even the lightest traffic on the roadway, it is reasonable to expect some drop off in flows at some point with an increase in volume, and hence two capacities for "stable" and "unstable" flows. In a graphical presentation, Edie came out with a discontinuous parabolic speed-density curve, the upper left hand portion showing operations under free-flow conditions and the negatively sloped lower part of the curve representing congested conditions (see Figure 1). Ceder and May (1976) who also investigated some traffic flow models concluded that there exits a two-regime phenomenon in traffic flow, one with higher capacity than the other.

The two cases presented above are, however, not directed to capacity flows. Rather the two cases focus on the circumstances leading to queue formation and their subsequent impact on traffic flow phenomena; a description of the visual appearance of congested and uncongested operations. Banks (1990) cited a few such cases, which all describe the initiation of queuing at a constriction on the freeway.

According to the HCM, capacity flows occur only at the low density, left-hand side of the curve where level-of-service (LOS) *E* is defined. Assuming that LOS *E* as defined for capacity operations for uninterrupted flow facilities in the Manual is correct, the so-called drop in capacity as shown by the lower, right-hand part of the curve cannot be true since that indicates flow within the queue which cannot be defined as capacity flows. One reason is that capacity as defined in the HCM implicitly connotes stable speeds; there is no question of discontinuity resulting from slow-and-go driving conditions which characterise breakdown operations.

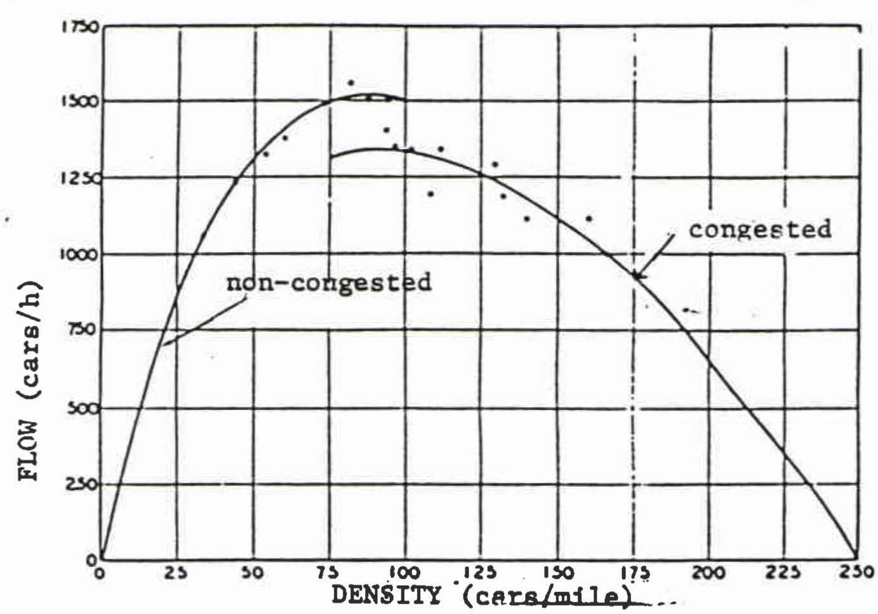

FIGURE 1. DISCONTINUOUS FUNCTION SHOWING TWO CAPACITIES
SOURCE: EDIE,1961.

Wattleworth (1963) puts the issue of capacity drop in its right perspective:

> When this condition occurs, the vehicles from the high density reservoir upstream of the bottleneck cannot be fed to the bottleneck as fast as they could pass through it.
>
> In this case the flow rate at the bottleneck decreased when congestion set in. However, the decreased flow rate was not due to a decreased "capacity" at the bottleneck, but to the inability of the upstream section to supply vehicles at the capacity rate of the bottleneck (p.17).

"Condition" in the quotation refers to the presence of a queue.

Accepting the two-capacity hypothesis raises the question of how significant, numerically, is the drop in flow rate to warrant the separation of capacity under free-flow conditions and capacity as queue discharge flows. Banks (1990) presented 30-second data, demonstrating a capacity drop in the presence of a queue. The difference in the mean flows before and after the queue was found to be 70 to 75 vehicles per hour per lane. Hall and Agyemang-Duah (1991), who also based their analysis on 30-second values, reported a weighted mean difference of 361 vehicles per hour across three lanes between pre-queue and queue discharge flows. Hall and Hall (1990), using 5-minute traffic data, contrarily, found no drop in flows when there was a queue. This raises the possibility of the time interval for analysis affecting capacity drop. Validating the existence of a capacity drop and the extent of the drop calls for more data. The analysis presented in chapter four involving more than 50 days should help resolve this issue.

2.4 OPERATING CONDITIONS UNDER CAPACITY

One other issue arising from the discussion of the numerical value and capacity drop is the operating condition, especially the low speeds that are said to be associated with capacity flows and the implication for the location of earlier capacity studies. Some of the models put forward to explain the low speeds characterising capacity operations can simply be dismissed as untenable. One such model is the deterministic speed-density model which assumes an inverse linear relationship between speed and density (Greenshields, 1934). Prigogine (1961), realising the logical flaw in such a model, developed the "desired" speed-density hypothesis, which states that for lower density there is a "free speed" distribution and at "critical concentration" speeds are low. In other words Prigogine sees speed as "relaxing to a desired level" in the traffic stream as density reduces from high to low. But other factors have an equally important impact on speed. The flaws in Greenshields and Prigogine's analyses are the presumption of a "common" driver behaviour and neglect of the influence of ramps and other control conditions on the roadway which necessitate that vehicles slow down irrespective of road density. It is not clear how the hypotheses accommodated the obvious unpredictable driver behaviour that in turn is affected by features such as trip purpose, and environmental conditions. Most importantly, speed ceases to be a function of flow when vehicles are discharging from an upstream queue. Rather downstream speed depends on the distance of the point of measurement from the head of the queue (Persaud and Hurdle, 1988).

The operating conditions under capacity flows quickly bring into focus the question of where data were collected. The discussion on the numerical value and capacity drop has shown that operating conditions upstream of the bottleneck are simply in queue. The suggestion that capacity flows on freeways occur at speeds in the order of 50 km/hour (HCM, 1985) is additional evidence to support the contention that most earlier capacity studies used data from upstream of the bottleneck.

The literature review has shown that the freeway capacity concept should connote something achievable on daily basis and should acknowledge some variation in the numerical value both in time and in space. The inclusion of "reasonable expectation" in the 1965 HCM implicitly indicates that there is no continent-wide notion of the capacity concept.

The replacement of hourly volume by hourly flow rate has changed the logic underlying the capacity drop and the meaning of capacity conceptually. In capacity analysis, the focus should be on the numerical value of the average capacity drop.

Validating the definition of capacity as the maximum 15-minute pre-queue flow rate requires more data, information on the place of measurement, and the quality of data used.

More importantly, the review has also shown that perhaps the one major source of the current debate on freeway capacity has been a lack of clarity about the place to look for capacity. Knowledge about the place to take capacity measurement will help dispel the "myth" surrounding the operating conditions, especially the low speeds that have been said to characterise capacity flows on the roadway. Adding the condition of sufficient demand to the set of criteria given in the HCM will help resolve some of the contentious issues surrounding freeway capacity.

CHAPTER THREE

DATA ACQUISITION AND REDUCTION

Many of the unresolved issues surrounding the concept of capacity seem to stem from an inadequate description of the location of the data collection. The study location has a great influence on the nature of the data for the analysis. Likewise, serious misinterpretation can result from unsuitable data and analytical procedures. This chapter presents an account of the data used in the analysis. There are three main things that are covered in this chapter: the description of the study site; data acquisition procedures; and data reduction.

3.1 THE STUDY SITE

Arising from the literature review, two criteria were used for site selection. One is the existence of sufficient demand upstream of a bottleneck. The second criterion is that only a site outside the influence of ramp and/or weaving manoeuvres (as defined in the HCM) was considered. Such a site should be 150 metres (500 feet) upstream and 760 metres (2,500 feet) downstream of an entrance junction; 760 metres upstream and 150 metres downstream of exit ramps; and 150 metres upstream and downstream of a merge point.

The freeway used in the study is the Queen Elizabeth Way (QEW) in Ontario. It has six lanes, three in each direction.

Each lane is 3.66 metres (12 feet) wide. The shoulder lanes are wide enough for any stalled vehicle to pull off safely. The posted speed limit is 100 km/hour.

The QEW has two Freeway Traffic Management Systems (FTMSs), one on the Skyway at Burlington and the other at Mississauga. The freeway bottleneck on the Skyway is beyond the instrumented system, which makes it difficult to collect data. This makes the Mississauga FTMS, which has five major bottlenecks, the choice. Since queues upstream of four of the bottlenecks within the Mississauga FTMS coalesce during the morning peak period (Hall and Hall, 1990), the one furthest downstream is the only possibility for the study. The section of the Mississauga FTMS which is of interest in this study is therefore the stretch from Cawthra Road to just beyond Dixie Road at Mississauga (Figure 2).

Earlier studies on traffic flow on the freeway were concentrated on the section between Cawthra and Dixie Roads. For instance, the work by Hall and Hall focused on station 23 (formerly station 22). Since that time, the FTMS has been extended eastward and re-numbered with four additional stations including station 25. Out of the four detector stations that are included in this section, two (stations 22 and 24) are single-loop stations, which record no speeds and therefore are not suitable in this study when it comes to the analysis of the relationship between capacity operations and level-of-service (LOS). Station 23 has been used in a previous and similar study. This, in addition to the fact that station 23 is 800 metres downstream of an entrance ramp, makes station 25 appropriate for the study. Station 22, upstream of Cawthra Road, where there is a major interchange ramp, experiences recurrent congestion during the morning rush hours as a result of excessive demand created by commuters to Toronto. Station 22, which is 2250 metres upstream of station 25, was therefore used as an indicator of the existence of sufficient demand.

The FTMS, which covers about 19 kilometres of the QEW, is of particular interest in this study because it is the main source of data. It has such devices as a closed circuit television monitoring system, changeable advisory information signs, ramp metering signals and computerised vehicle detectors.

The computerised vehicle detectors are made up of either one or two diamond-shaped induction loops beneath the road surface of each lane. For double loop detectors, occupancy and vehicle counts are independently made by the upstream and downstream loops. Direct measurement of speeds is also possible. The vehicle counts and the speed and occupancy measurements are all updated at 30-second intervals.

The ramp metering signals are located on the eastbound access ramps and operate for about three hours during the morning peak period. The signals are set manually at the rate of five to ten seconds cycle time depending on traffic

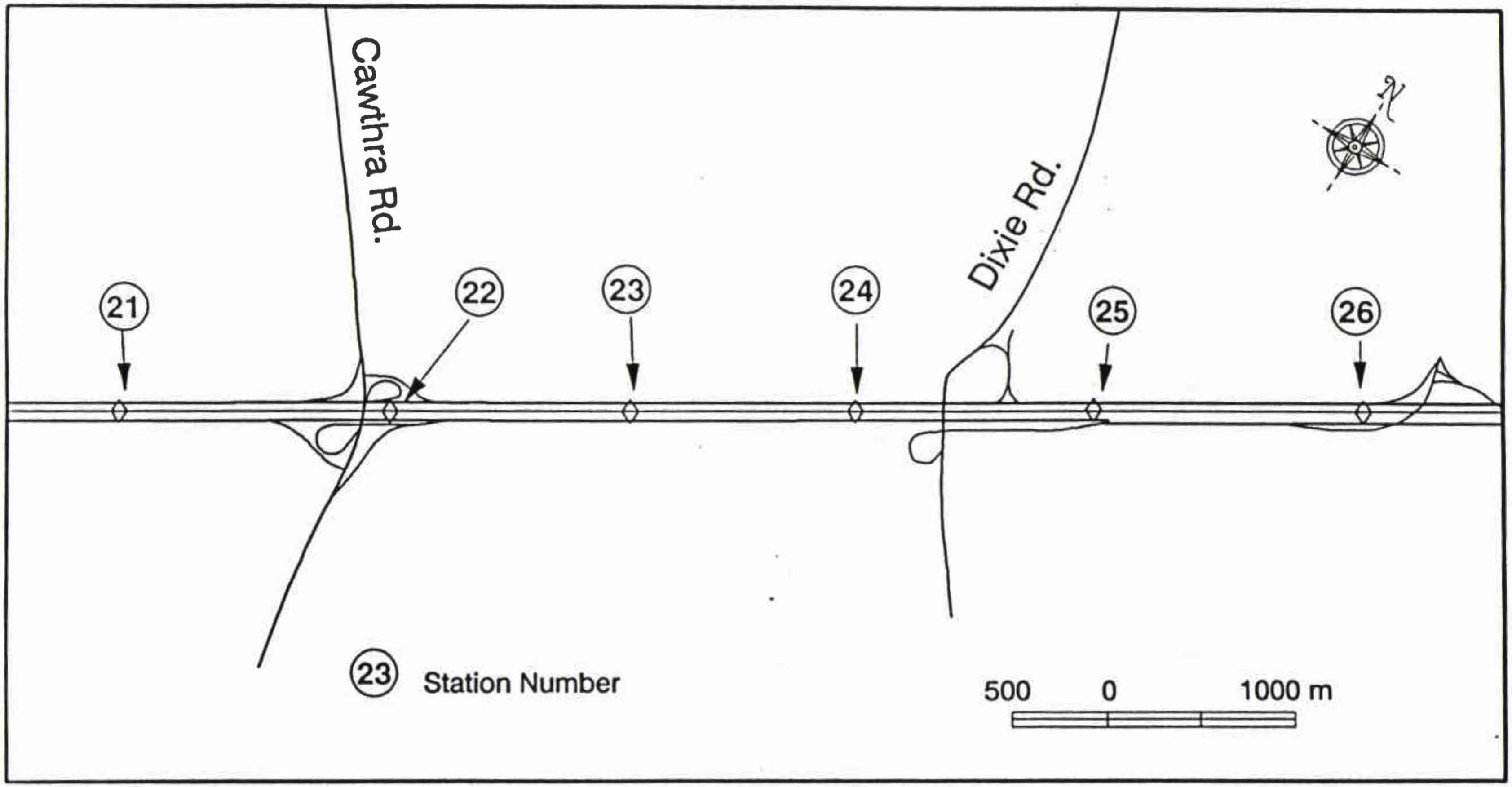

FIGURE 2. STUDY SECTION: QEW, MISSISSAUGA, SOURCE: HALL & AGYEMANG-DUAH, 1991.

conditions. The ramp metering controls vehicles entering the freeway and thus reduces the shock waves likely to be produced by such merging in the traffic stream.

The FTMS operates by transmitting data about the roadway and traffic conditions from the detectors through a communications system which are then displayed on a computer terminal at the control centre. The FTMS alerts the operators at the control centre when there is an incident on the roadway. A freeway traffic incident is defined as any obstruction to the free flow of traffic on the roadway. After assessing the situational needs through the closed circuit television, the operators transmit advisory information to motorists through the changeable message signs; and can call in whatever assistance is needed.

3.2 DATA ACQUISITION PROCEDURES

The data required for this study were traffic volume, vehicle speeds and occupancies. Traffic volume is the recorded number of vehicles and was used in the analysis of capacity drop and capacity distribution over time. Speed as used in the report refers to the arithmetic average speed recorded each 30 seconds as vehicles cross over the double loop detectors. Speed is important in assessing the operational conditions under capacity flows. Occupancy, defined as the percentage of time that vehicles are present over the detector, was useful in the identification of upstream queue presence at station 22.

The traffic data collected by the FTMS are saved regularly in weekly summary files on tapes. A weekly summary file contains all 30-second volume, occupancy and speed (where available) data for each day in one week from all detector stations. Data are for only weekdays when regular commuters use the freeway and were available from April 25, 1990 through August 24, 1990. The driver population on the freeway is therefore one that is quite familiar with the roadway conditions. For convenience, all dates are written in a numerical form, beginning with the year, the month and the day of the month. For example, April 25, 1990 is written as 900425.

Three forms of data were required for the analysis: 30-second, 5- and 15-minute data. (Section 3.3 treats the reasons for choosing 30-second and 5-minute time intervals). To get the 30-second values for a specific station (stations 22 and 25 in particular in this study) a FORTRAN programme which reads the traffic data from the summary files on the tape by lane for a particular station was used. Another FORTRAN programme was also used that calculates the volume and weighted speed for each lane and across all the three lanes for the 5-minute values. The output files from the two FORTRAN programmes were imported into a spreadsheet for subsequent analysis. The time reported in all the output files were the ending times for an interval.

Days for which data were available but were rejected are presented in Table 3.1, along with the reason in each case. Rejections occurred because of weather, the location of incidents, the working condition of the detectors or information on general traffic operations between 5:30 A.M. to 10:00 AM. This left for analysis 52 days of good weather and no incidents.

3.3 DATA REDUCTION

The three main concerns in the data reduction are the time interval (defined as the traffic-counting interval), treatment of missing data and truck composition. Earlier capacity studies used various time intervals. Hurdle and Datta (1983), used 2-minute counts while Hall and Hall (1990) based their analysis on 5-minute values. Banks (1990), however, adopted a 30-second interval to analyze flow processes in a bottleneck. These studies based on different time intervals produced different results. For instance, Hall and Hall reported no drop in flows when there was a queue while Banks noted some differences in pre-queue and queue discharge flow rates, although statistically marginal.

Clearly, the choice of time interval will be greatly influenced by the purpose of the study. In this study, 30-second interval was used in the analysis of capacity drop. It was also important in determining the correct time for the beginning and ending of a queue in a flow process which calls

Table 3.1 Days with rejected data.

Day	Reason(s) for rejection
900516	Morning rain showers.
900517	Morning rain showers.
900521	Public holiday.
900529	Morning rain showers.
900427	Intermittent queues upstream
900605	Intermittent queues upstream
900615	Load of bricks on lanes 2 and 3 at 8:32 A.M., east of Dixie Road. Cleared at 9:10 A.M.
900618	Lane 3 closed for testing of changeable message sign NO.3. Thunderstorm in the morning.
900621	Showers in the morning.
900625	Series of eastbound incidents from 7:31 A.M. to 11:50 A.M.
900629	Thunderstorm in the morning; poor visibility.
900702	Public holiday.
900710	Intermittent queues upstream
900712	Eastbound incident (location not specified) at 6:52 A.M. Clearance time not specified.
900717	Intermittent queues.
900718	Stalled car on lane 1 at 7:28 A.M., east of Cawthra.
900727 to 900818	Malfunctioning of the detectors. (Station not specified)
900818 to 900820	Low flow rates due to insufficient demand upstream
900823	Intermittent queues upstream

Source: Operators' Logbook (FTMS Control Centre, Mississauga) Ministry of Transportation, Ontario, 1990.

for the shortest possible time interval.

The choice of the 5-minute interval was to reduce the random variation in the data set at 30-second intervals. The peak 15-minute values for each day as suggested by the HCM were also looked at.

The problem of missing data was particularly serious with the aggregation of data into 5-minute values. For the 30-second values, missing data were indicated by -1. In the case of the 5-minute values, missing data were identified by the difference in the number of data points used to calculate the volume and speed values and the total number of the data points in the interval, namely 10. In general, the criterion used was that if more than two points in a 5-minute interval were missing, the whole 5-minute period was left out in the analysis. Otherwise missing data were taken to be the average within the 5-minute interval.

Ideally, freeway capacity is measured in passenger cars. It was therefore deemed important to determine the traffic composition for the subsequent adjustment of the numerical value for capacity. This was necessary because the FTMS does not separate truck counts from the data.

Two choices were available to deal with such traffic data: the use of a fourth-degree polynomial, which gives an approximation of the truck percentage at different times of the day as developed by Hurdle and Datta (1983), or manual counting of trucks. The second choice was adopted and on May

29, 1990, manual counting of trucks at 5-minute intervals was carried out (Table 3.2). To get a good picture and to reduce variation in the truck distribution over time, the truck counts were aggregated at 30-minute intervals. At this interval, there was very little variation in the truck distribution over time. Since the truck distribution did not differ much over time, the use of an average figure was considered reasonable. A truck percentage of 6.0 per cent was therefore used for the truck correction. Although there was a morning rain shower on that day, the 6.0 per cent for the truck adjustment was used as the rain only slowed down the rate of flow and not the volume.

Table 3.2. Truck* percentage as against total volume.
Station 25. May 29, 1990

End time	@volume	+truck	truck %	At 30-minute interval volume	truck	truck %
6.35	465	17	3.66			
6.40	410	32	7.80			
6.45	406	48	11.82			
6.50	413	18	4.36			
6.55	464	28	6.03			
7.00	432	43	9.95	2590	186	7.181
7.05	416	11	2.64			
7.10	424	25	5.90			
7.15	410	39	9.51			
7.20	450	17	3.78			
7.25	442	30	6.79			
7.30	462	42	9.09	2604	164	6.298
7.35	451	17	3.77			
7.40	459	26	5.66			
7.45	447	37	8.28			
7.50	468	10	2.14			
7.55	459	24	5.23			
8.00	453	34	7.51	2737	148	5.407
8.05	473	16	3.38			
8.10	439	34	7.74			
8.15	471	49	10.40			
8.20	466	17	3.65			
8.25	460	34	7.39			
8.30	464	49	10.56	2773	199	7.176
Total	10240	648				
Average	446	28	6.32			
Std.dev.	22	12	2.78			

* heavy vehicles with an average weight-to-horsepower ratio of 200 lb/hp (HCM,1985, 3-14)
@ detector data on May 29,1990; total across all the three lanes
\+ manual counting on May 29, 1990; total across all the three lanes

CHAPTER FOUR

DATA ANALYSIS

The data analysis presented in this chapter covers three main areas. These are the determination of upstream queue presence, capacity drop, and the distribution of queue discharge flows.

4.1 DETERMINATION OF UPSTREAM QUEUE PRESENCE

There are two reasons for knowing when there is a queue in capacity analysis. The first one relates to the analysis of a potential drop in capacity as a queue forms upstream, which requires the separation of free-flow and forced flow conditions. This is translated into when each of the two periods begins and ends. The period before a queue forms when high flows exist is referred to here as the pre-queue or transition period.

Determining the period of queue presence also has a very important implication for the numerical value of capacity. Upstream queue presence (in the absence of any incident downstream) is a good indication of the existence of sufficient demand, a necessary pre-condition in capacity analysis. The time interval when vehicles discharge from a queue is defined as the queue discharge period.

In this section, a description of the methods considered for determining the beginning and the end of queue is presented. The relative advantages and/or disadvantages of each method, which led to the selection of one, are also discussed.

Traditionally, vehicle speeds, either in queue or downstream, have been the key parameter in determining queue presence. Low speeds in queue have been associated with congested operations. For instance, Hurdle and Datta (1983) in determining the beginning of congestion used speeds of less than 32 km/h. Banks (1990), Chin and May (1991) and Urbanik, Hinshaw and Barnes (1991), contrarily, used downstream speeds to determine the time to measure capacity. Using speeds only may, however, be unreliable. The reason is that apart from possibly being a function of traffic flow, speeds are also a reflection of general driver behaviour which is not only determined by the traffic conditions but also other several factors. An error in determining the beginning of congested operations may have serious implications for testing the two-capacity hypothesis regarding the average drop in flow rates as a queue forms. For instance, in the Hall and Hall paper (1990), the time of queue presence which was associated with low speeds was later detected to be incorrect (Hall and Agyemang-Duah, 1991): about 40 minutes of operation each day declared as queue discharge flows were in fact pre-queue flows.

In this present work, evidence of queue presence was looked for at the location where the queue forms. The use of a speed drop downstream in the bottleneck was also investigated. The upstream queue presence was determined from traffic data at station 22 (Figure 2), a single-loop station that does not record vehicle speed but only volume and occupancy.

It is possible to determine the beginning and end of congestion from volume and occupancy data. The rationale is that high traffic volume moving at low speeds, a common characteristic of congestion, results in high occupancies. By identifying the relationship between traffic flows and occupancy, the queue start and end times can thus be determined.

The averages of volume and occupancies across all three lanes were used to identify the queue presence. Three different approaches were considered. These are the ratio of occupancy to flow; use of a flow-occupancy boundary function; and visually observing the process of queue build-up and dissipation on a micro-computer. The procedures involved in each of the methods are discussed below. The queue start times given by each of the methods are compared with the time of a speed drop at stations 23 and 25. This comparison will also test the reliability of the downstream speed drop as a criterion to identify the time for capacity measurement. If it is consistent with other methods, it can help to confirm them.

The occupancy-flow ratio method

The occupancy-flow ratio method is based on the relationship between flow and occupancy. The logic is that a period of congestion is marked by a rise in occupancies and a simultaneous drop in flow. This is the approach used in the work by Hall and Agyemang-Duah (1991). Based on this logic, the ratios of occupancy to flow were plotted against time. As in Figure 3, the period of congestion is marked by high ratios. The occupancy-flow ratio curve is relatively flat, hovering around 1.0 till about 6:25. This represents the period before congestion. After about 6:25, the curve shoots up, remaining well above the 1.0 value till about 9:00 when the ratio starts tailing off, becoming flat once again, marking the beginning of post-queue flow.

The critical task with this method was to select a threshold value as the criterion for determining the start and end of congestion. Two values, 1.0 and 1.1, were evaluated by means of a sensitivity analysis in terms of sustainable high flows for each. Two problems were found to be associated with the value of 1.0. One of the problems is that flows of 5,800 or more did not stay consistently for at least three minutes, implying that the queue was not formed yet. The choice of the ratio 1.0 would have meant the inclusion of some very low flow rates, below 5,500 vehicles per hour, an indication that there was not sufficient demand at that time.

A ratio of 1.1, continuing for three minutes or more was

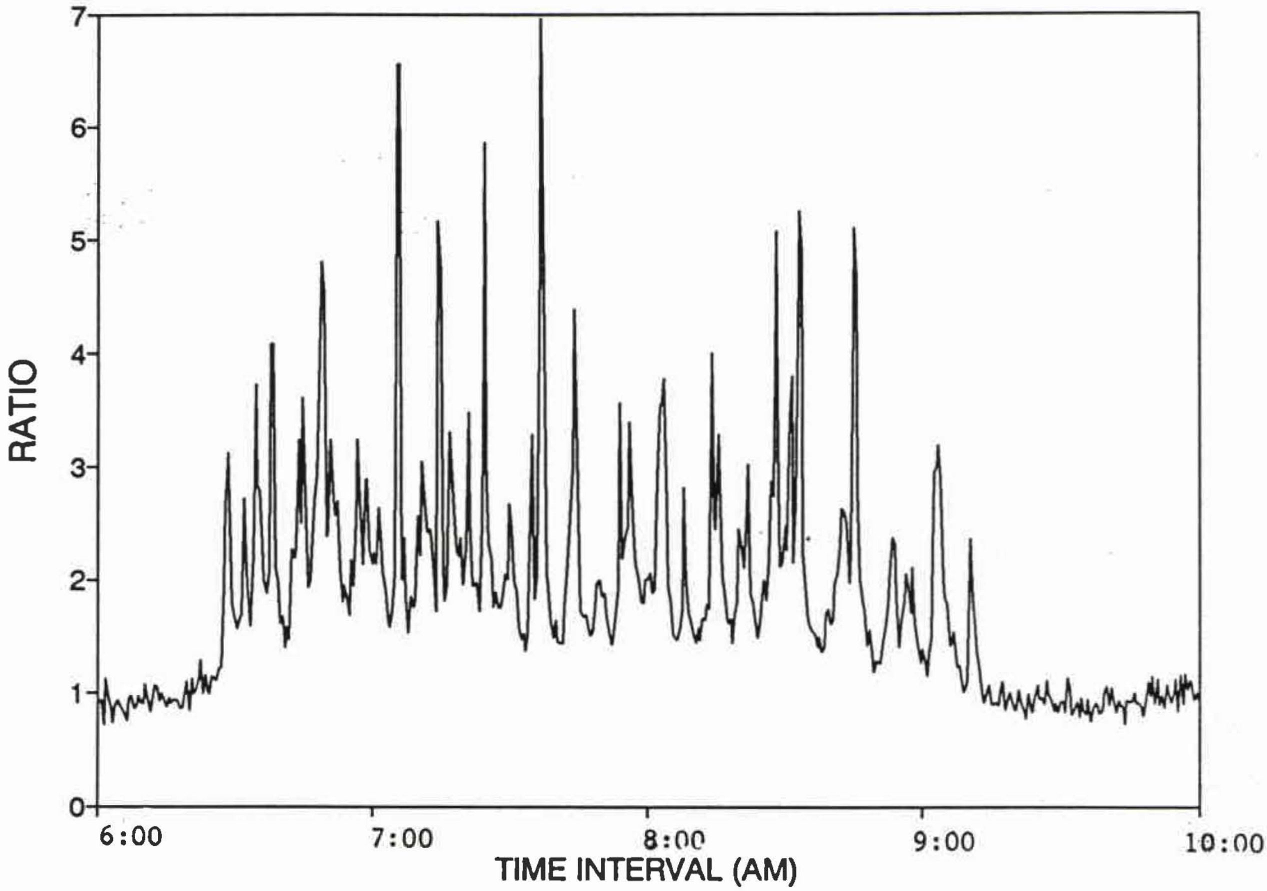

FIGURE 3. OCCUPANCY-FLOW RATIO PLOT. 30-SECOND AVERAGE VALUES. STATION 22. MAY 9, 1990; 5:30 A.M. - 10:00 A.M.

found to be a reasonable threshold value to choose for the identification of queue start times but not for queue end times. The reason is that even before the ratio falls below 1.1, flow rates were generally very low, below 5,500 vehicles. This implies using a different method to determine the end times of congestion. But combining two different methods to achieve the same purpose will necessitate one more task: the exploration of the nature of the complementary relationship between them. This is no mean task. Another disadvantage of this method is that it was not possible to determine the beginning of transition to congestion from the ratios although transition end time coincides by definition with queue start time. A method that can be used to determine at least both the start and end times of the queue was needed.

The flow-occupancy boundary function method

The flow-occupancy boundary function was also based on the notion that the congested period is characterised by much higher occupancies and a slight drop in flows. The function, which defines a threshold uncongested flow, is of a quadratic form:

$$\mathbf{Flow = a*occupancy - b*occupancy^2 - c}$$

where *a, b,* and *c* are calibrated constants determined by an iterative method using 30-second data from station 22.

Graphically, the flow-occupancy relationship has an upside down *v*-shaped form (Figure 4). Congested data are found on the right-hand side of the plot and uncongested data points are to the left-hand side. The main task was to define a boundary that separates congested flow from uncongested. This meant calibrating the quadratic function to determine the values of the parameters *a*, *b*, and *c*. Based on a previously calibrated function for this particular station (station 22), the calibrated values were manually increased or decreased depending on both the shape and position of the boundary line relative to the two types of data. If the plotted function line was too low in position, the constant value *c* was increased. Alternatively it was decreased when the line was too high. Similarly, the values of the parameters *a* and *b* were either increased or decreased depending on whether the function line tapered off or curved off sharply. With this "hand-crafted" technique, the final version of the quadratic equation with the calibrated values was :

$$\textbf{Flow} = \textbf{1.2*occupancy} - \textbf{0.014*occupancy}^2 - \textbf{2.5}$$

As in Figure 4, the data points below the boundary line represent congested flows, some of which occur at the same occupancies as uncongested flow (between 10 and 25 per cent). It should be noted that the boundary line does not start from zero on the occupancy scale (*x*-axis). Investigation into

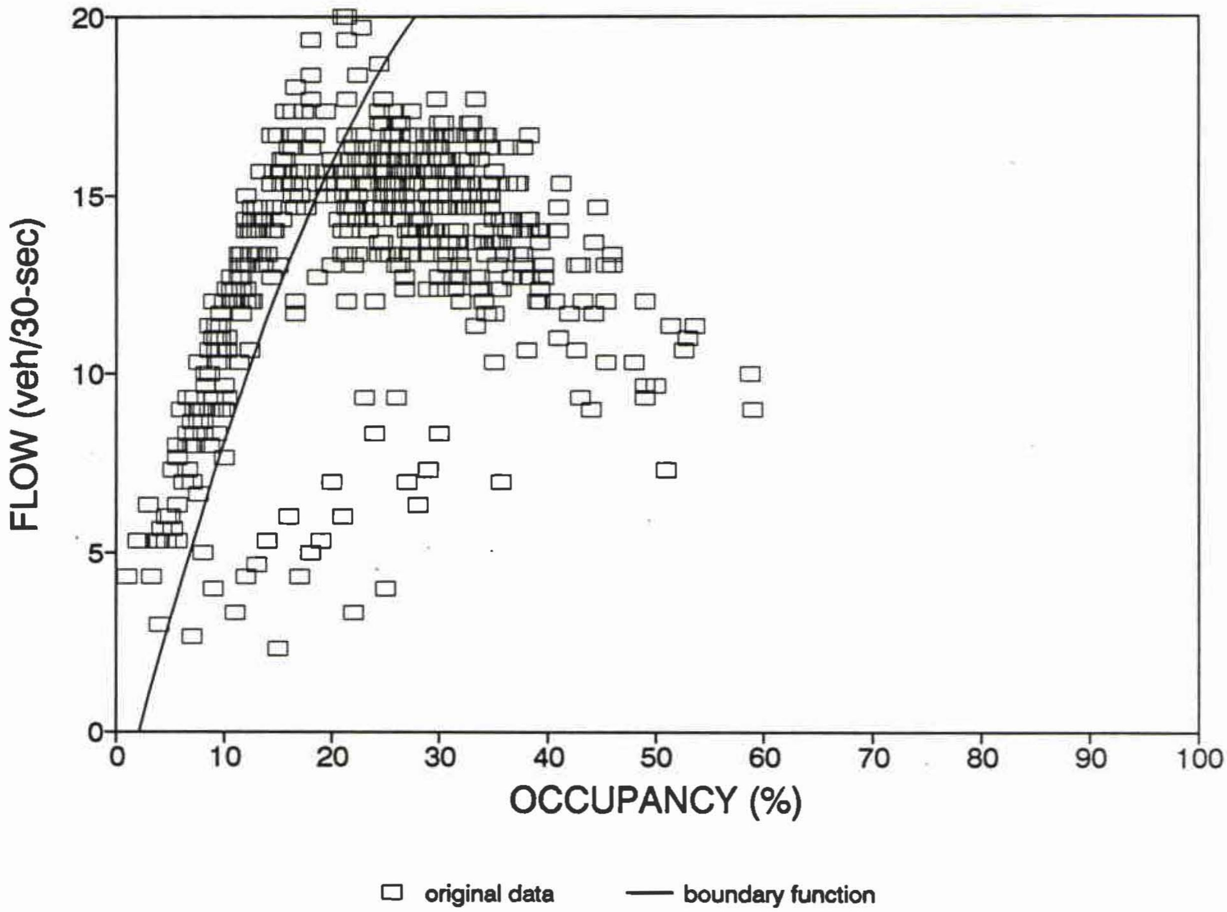

FIGURE 4. FLOW-OCCUPANCY BOUNDARY FUNCTION. 30-SECOND AVERAGE VALUES; STATION 22. MAY 9, 1990; 5:30 A.M.-10:00 A.M.

the nature of the data quality indicates that empirically, zero flow can be associated with occupancies up to three.

The above equation was used in a FORTRAN programme which calculates the beginning and end times of congestion. The programme has a built-in persistence-check of six 30-second intervals to calculate the congestion start and end times. The choice of six persistence-check intervals was derived from the logic of the McMaster Incident Detection Algorithm (Persaud, Hall and Hall, 1990), which has only two. While the main function of the McMaster Incident Detection Algorithm is early detection of congestion, whether recurrent or incident-related, the concern in this study is the detection of queue presence. The confirmation of the beginning and end times of congested operations was deemed very important. One therefore needs to be as conservative as possible to prevent any false declaration of congestion, hence the six intervals.

The queue start and end times used with this method are not those at the end of the sixth interval. Since the persistence check intervals were to confirm the queue presence, determining the actual queue start times means working backwards to the first interval. Hence, three minutes were subtracted from the calculated times. But before these times could be used, the visual method was looked at to compare the two results since they are both based on the same logic.

The visual method

A visual observation of the process of queue formation on a micro-computer, using detector data consisting of 30-second volume and occupancies from station 22 was adopted as a third method. This was made possible with "Macspin", statistical analysis software that helps discover important patterns in statistical data. This software makes it possible to display the data points on a micro-computer screen in time sequence, to control the rate at which they are displayed, and to move back and forth in time within the data. It also allows one to select a point of the plot and to display the values of the variables, such as flow, occupancy, and time. The steps involved in this method are explained below.

A scatter plot of flow versus occupancy (averaged across all three lanes) was plotted as in Figure 5a. It is clear from Figure 5a that there are two parts of the plot apparently separated by a gap. To the left-hand side are the uncongested data that occurred at relatively lower occupancies than the congested section to the right-hand side of the plot. Then, only that portion of the plot was displayed that contains the uncongested branch of the curve occurring before the onset of congestion (Figure 5b). By moving forward in time, comparatively low flow points at higher occupancies start appearing (Figure 5c), an indication of possible queue presence. This slow, forward movement in time was continued till the data points stayed congested (Figure 5d). The data

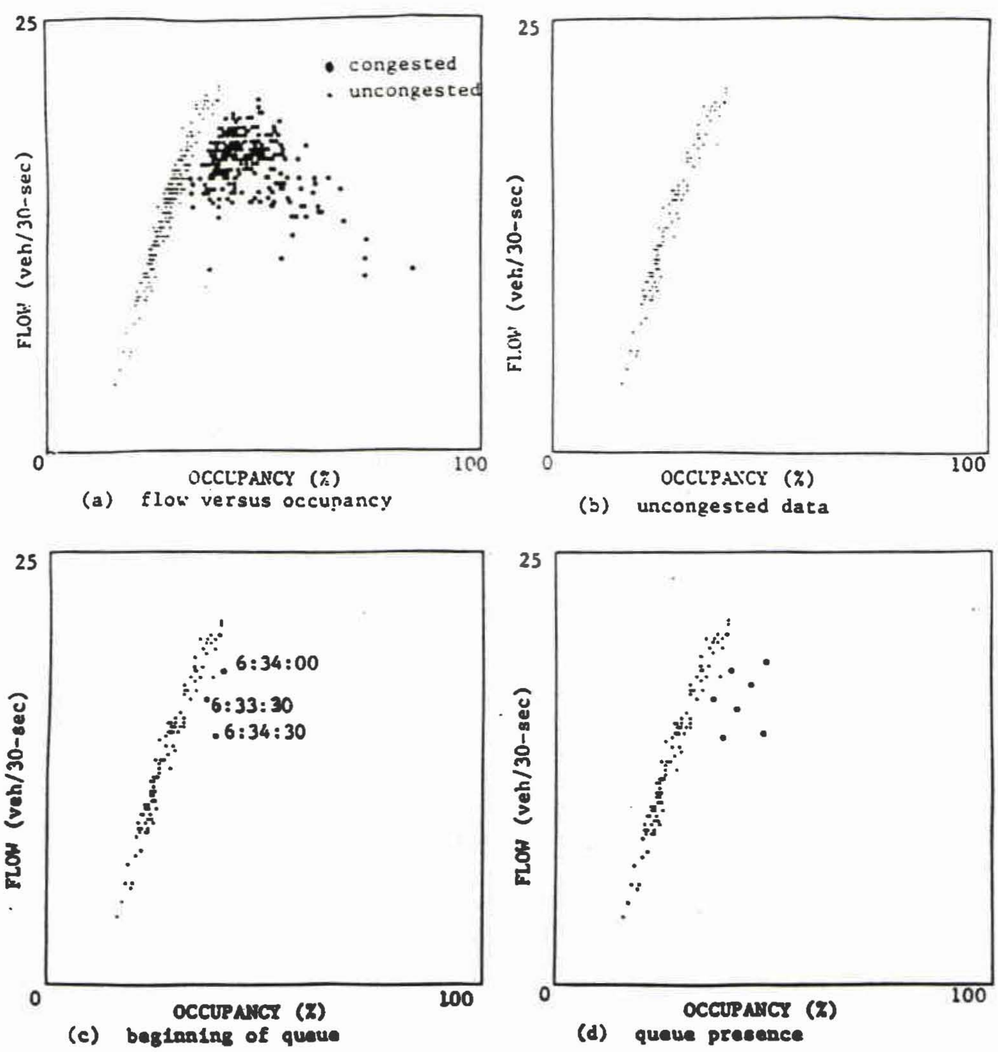

FIGURE 5. DETERMINING QUEUE START TIME BY THE VISUAL METHOD AT STATION 22; MAY 11, 1990.

points that stayed congested were selected, and the variable values of each point looked at. The data point with the earliest time in this selection was considered to mark the beginning of congested conditions and that time was taken as the queue start time. The end of congestion was identified in the same way as for queue start time.

Determining the start of transition to congestion involved the same process as finding the queue start time but the focus was on only the uncongested branch of the curve. Inspection of the data points was restricted to pre-queue high flows at consecutive intervals that indicate the existence of sufficient demand. This was complemented by an inspection of the flow data at station 25 when comparatively low flow data points (5,500 vehicles per hour or below) appear on the screen. Working backwards in time with station 25 flow data, when a sharp drop in flow separating any period of relatively high flows was detected, a two-tailed significance test of differences in mean flow was done for the two periods. (The choice of test depended on whether the variances were equal or not). If the difference in mean flow is significant at the 95 per cent confidence level, the end time just before the drop was taken as the pre-queue start time. For instance on 900704 (Table 4.1), the pre-queue start time was tentatively set at 6:25:00. But there was a period of comparatively high flows before this time. A *t*-test for the two periods 6:22:30 to 6:24:30 and 6:25:00 to 6:36:00 indicated that the difference

Table 4.1 Determining pre-queue start time at station 25 on 900704 using t-test.

time	End interval	900704 flow/h
6	19.00	4680
6	19.30	6000
6	20.00	6120
6	20.30	6120
6	21.00	5400
6	21.30	4200
6	22.00	4320
*6	22.30	7080
6	23.00	5520
6	23.30	6960
6	24.00	6600
6	24.30	3840
6	25.00	6120
6	25.30	5880
6	26.00	7200
6	26.30	8280
6	27.00	6960
6	27.30	5640
6	28.00	6360
6	28.30	6960
6	29.00	6480
6	29.30	6960
6	30.00	7800
6	30.30	5280
6	31.00	6240
6	31.30	6360
6	32.00	6360
6	32.30	6600
6	33.00	6840
6	33.30	6720
6	34.00	6120
6	34.30	6000
6	35.00	5640
6	35.30	6240
#6	36.00	6360

* Pre-queue start time
\# Pre-queue end time
Note: on this day pre-queue end time did not coincide with arrival of queue discharge.

in mean flows in the two periods is insignificant. The two periods were therefore taken as one period. The pre-queue start time was set again at 6:22:30 and a *t*-test for the periods 6:19:30 to 6:22:00 and 6:22:30 to 6:36:00 showed a significant difference in mean flow. The pre-queue start time was therefore 6:22:30. When relatively low flows (5,500 vehicles per hour or below) were observed for four 30-second consecutive intervals or more followed by a period of comparatively high flows till the start of the queue, the end time of such low flows was taken as the pre-queue start time (Table 4.2).

It is said that a change from uncongested to congested conditions in traffic flow may create a condition that cannot be characterised as pre-queue or queue discharge as a result of what has been described as shock waves (Lighthill and Witham, 1964; Walker, 1989). Evidence of such condition was therefore sought in determining the arrival of queue discharge at station 25. An examination of the flow data at station 25 showed that on some days, at the onset of the queue, flow dropped below 5,000 vehicles/hour in the next two minutes or the flow was characterised by alternating highs (up to 6,200 vehicles/hour) and lows (below 5,000 vehicles/hour) tab 4.1for up to about six minutes (Table 4.3). All such flow data were not included in the analysis.

Table 4.2. Determining pre-queue start time at station 25 based on low flows on 900509

time	End interval	900509 flow
6	11.00	4680
6	11.30	4680
6	12.00	4560
6	12.30	4560
6	13.00	5280
6	13.30	5040
*6	14.00	6840
6	14.30	7080
6	15.00	6240
6	15.30	6000
6	16.00	6720
6	16.30	6480
6	17.00	5760
6	17.30	7680
6	18.00	6360
6	18.30	8280
6	19.00	5880
6	19.30	6120
6	20.00	4800
6	20.30	5400
6	21.00	6840
6	21.30	5400
6	22.00	7080
6	22.30	6600
6	23.00	6480
6	23.30	6360
6	24.00	5400
6	24.30	7680
6	25.00	7800
6	25.30	6960
6	26.00	5880
6	26.30	6120
6	27.00	6240
#6	27.30	6600
$6	28.00	6720

* Pre-queue start time
# Pre-queue end time
$ Queue discharge start time

Table 4.3. Example of fluctuations in flow rates at station 25 at the onset of a queue at station 22.

time	End interval	900601 flow rate	900622 flow rate	900704 flow rate	900716 flow rate
6	30.00	6720	5160	7800	7800
6	30.30	6840	6120	5280	6960
6	31.00	6840	*6480	6240	6120
6	31.30	6720	5640	6360	6360
6	32.00	6360	4560	6360	6480
6	32.30	6120	6840	6600	7680
6	33.00	6720	4680	6840	5880
6	33.30	6240	4680	6720	5880
6	34.00	7080	5640	6120	5760
6	34.30	6120	5760	6000	7320
6	35.00	6480	5640	5640	6360
6	35.30	7320	5160	6240	5760
6	36.00	5520	#6120	*6360	5520
6	36.30	5520	6240	5040	6960
6	37.00	7200	6720	5880	6480
6	37.30	6600	5760	4560	6480
6	38.00	6000	5640	5640	5760
6	38.30	6000	5520	6240	*5760
6	39.00	6480	5640	5880	4920
6	39.30	5880	5520	4800	5880
6	40.00	5760	5880	6120	4800
6	40.30	6360	5640	5640	#5880
6	41.00	6720	4800	4800	6360
6	41.30	6000	6600	4560	6120
6	42.00	5400	4920	#5520	6840
6	42.30	5400	6480	5400	6000
6	43.00	5760	5640	6120	6480
6	43.30	5640	6120	5160	6960
6	44.00	*6600	6240	6600	5760
6	44.30	5040	5640	6000	6000
6	45.00	4920	5760	5760	6720
6	45.30	#6720	5400	5760	5520
6	46.00	5280	5520	4680	6000
6	46.30	5640	5040	6120	7320
6	47.00	6120	6000	6480	6120

* Pre-queue flow ends

\# Queue discharge flow starts

Comparison of the flow-occupancy boundary function and the visual methods

Before making a decision as to which method to adopt in the analysis two main issues are discussed. These are method limitation and speed drop downstream.

Method Limitation

Although the two methods represent the same logic, there was one basic difference in the procedures used in each of the two methods. While one general function was used to run the FORTRAN programme in the occupancy-flow boundary function method, determining the queue period with the visual method was done separately for each day. In other words, the peculiarities of operations on each day were taken into account in the visual method but this was not the case in the occupancy-flow boundary method. The function when used in the McMaster Incident Algorithm is continuously updated, but this was not possible given the short time segments worked with for each day. A visual observation of the process of queue formation on the micro-computer screen seems to be intuitvely appealing. This makes the visual method a preferred choice given the two methods. Also the queue start times given by the visual method were almost always earlier than those given by the function line (Appendix 1).

Speed drop downstream of the queue

Banks (1990), Chin and May (1991) and several other researchers have indicated that once a queue forms upstream, there is a sharp drop in speed downstream. The logic behind this proposition is that speed is taken as a function of flow. Empirical studies in traffic flow theory do not support this idea or deny it. What is important, however, is that when vehicles are discharging from an upstream queue, speeds become a function of the distance from the observation point to the head of the queue (Persaud and Hurdle, 1988). The volume of vehicles may have very little impact on speed when vehicles are discharging from a queue upstream. An attempt was made to pick out the speed drop time on its own at the downstream station. Generally, the fall off in speed was not so sharp as is commonly believed. Rather it was a gradual change (Appendix 2). It was not easy to pick the time of upstream queue. In this study the presence of the upstream queue was further tested by comparing the queue start times as found at station 22 with the times of the speed drops at downstream stations 23 and 25, which record vehicle speeds.

To conduct this test involved the coordination of time between the stations, taking into consideration the travel time between them. Station 22 is 800 and 2250 metres away from stations 23 and 25 respectively. To find the travel time between stations 22 and 25, for instance, one has to calculate the arithmetic average of vehicle speeds. In doing this, two decision criteria were made: one, only speeds of flows in the transition to congestion (at station 25) were considered; and two, it was assumed that speeds do not change on flat terrain like the study site in the absence of congestion and any incident. With these criteria, the calculated travel time between stations 22 and 25 was found to be 1.5 to 1.6 minutes. Since the data were collected at 30-second intervals, it makes sense to add one and one-half minutes to the congestion start and end times at station 22 to get the time of arrival of queue discharge flow at station 25. It takes about 30 seconds for operations at station 22 to be felt at station 23, based on the calculated travel time.

A comparison of queue start times at station 22 given by the flow-occupancy boundary function method and speeds at station 23 shows that in general, there was no instantaneous, precipitous drop in speed. Despite the absence of a precipitous drop in speed as would have been expected, the fact still remains that the queue start times at station 22 given by the flow-occupancy boundary function do not compare well with the times of speed reduction at station 23. For instance, the expected speed drop time at station 23 on 900426 based on queue start time at station 22 would be 6:38:30 (Appendix 1). Nevertheless, the speed drop at station 23 began roughly at 6:27:00 (Appendix 3). At station 25, (compare Appendix 2) speeds were even higher, making it difficult to identify the queue start time using speed only.

Truly, when vehicles are discharging from an upstream queue, speed may not be a function of flow. This was the case for more than half of the days that data were available.

The queue start times given by the visual method were also compared with the times of the speed drops at stations 23 and 25. The results show that for more than half of the days that data were available, the times when speeds dropped (although only a small amount) at these two stations were comparable with queue start times at station 22. The rate of reduction in downstream speed was therefore not a major decision criterion in the choice between the two methods but rather the consistency in the time of a drop in speed with the expected time of queue discharge flow at station 25 was used. This makes the visual method a preferred one. The adjusted arrival and end times of queue discharge flow at station 25 are presented in Appendix 4.

This comparison has one significant implication for capacity studies: once vehicles are discharging from a queue, speeds cease to be a function of flow. Researchers must therefore take extra precaution if they are to rely on speed drop downstream to determine when to measure capacity.

The procedures used in each of the methods, the speed drop times, and the method limitations make the visual method the preferred method. It was therefore adopted in the analysis.

4.2 STATISTICAL ANALYSIS OF THE TWO-CAPACITY HYPOTHESIS

This section presents the statistical analysis of the hypothesis that pre-queue flow is greater than queue discharge flow. There were two main steps in this analysis. The comparison of standard deviations for the pre-queue flow period and queue discharge period by means of an F-test was done first. The reason is that this test provides the basis for determining which test statistic to use in the analysis of the statistical significance of differences in means in the two periods. The two test statistics applicable in this case were the Student-t test and the approximation to t, the former for the comparison of means of independent samples with equal variances, and the latter for samples with unequal variances. The two samples (pre-queue and queue discharge) were considered independent because of their different operating conditions, which are not related in any way.

The F-test

The F-test, which tests for equality of variance, was based on the standard deviations of 30-second flow data in the two time periods. In using the F-test, it was assumed that the two time periods (pre-queue and queue discharge) were independent and that flows within them have a normal distribution. The F-test calculates the variance ratio, with the larger of the two variances as the numerator. A one-sided test criterion at the one and five percentage significance levels was used.

The F-test results (Table 4.4) show that at the one per cent level, in about 85 per cent of the cases (44 out of the 52 days), there was no significant difference in the variances. Only in about 15 per cent (8 days) were the variances significantly different. The picture at the five per cent level was little different from that at the one per cent level: only a small increase (from 15 to 27 per cent) in the number of cases that have statistically unequal variance.

These results indicate that the two time periods have common variances, which makes the Student-t test more appropriate than the t-estimate. Running only the Student-t test was, however, inappropriate for days where the variances were unequal. It was considered good to run one-tailed tests for both of the test statistics concerning means. Running the two tests also has its own problem since in all cases one of the two tests is inappropriate. For instance, running the Student-t test for 900810, which has significantly unequal variances would be inappropriate and may bias the end results. To deal with this problem, the "correct" test for each day based on its F-test result at the five per cent level was identified. These are discussed in the sub-section following.

Table 4.4. Comparison of sample standard deviations, pre-queue flow and queue discharge flow (QDF).

DAY	PRE-QUEUE Std.dev.	QDF Std.dev.	F-test	Are std. dev. different at significance level of: 1%	5%
900425	900	563	1.60	no	yes
900426	672	608	1.11	no	no
900430	760	566	1.34	no	no
900501	947	596	1.59	yes	yes
900502	761	580	1.31	no	no
900503	728	558	1.30	no	no
900504	1034	566	1.83	no	yes
900508	578	545	1.06	no	no
900509	802	578	1.39	no	no
900511	867	584	1.48	no	yes
900514	682	677	1.01	no	no
900515	1015	627	1.62	yes	yes
900518	680	602	1.13	no	no
900522	806	639	1.26	no	no
900524	927	643	1.44	no	yes
900525	583	566	1.03	no	no
900528	656	572	1.15	no	no
900530	610	538	1.13	no	no
900601	618	763	1.23	no	no
900604	1026	651	1.58	yes	yes
900606	474	599	1.26	no	no
900607	724	547	1.32	no	no
900608	633	631	1.00	no	no
900611	564	579	1.03	no	no
900612	771	593	1.30	no	no
900614	490	581	1.19	no	no
900620	633	573	1.10	no	no
900622	523	566	1.08	no	no
900626	852	547	1.56	yes	yes
900627	740	618	1.20	no	no
900628	855	574	1.49	yes	yes
900703	642	575	1.12	no	no
900704	830	543	1.53	no	yes
900706	903	592	1.53	yes	yes
900709	762	585	1.30	no	no
900711	544	598	1.10	no	no
900713	870	588	1.48	no	no
900716	798	598	1.33	no	no
900719	832	635	1.31	no	no
900723	805	629	1.28	no	no
900724	617	783	1.27	no	no
900725	692	610	1.13	no	no
900726	791	592	1.34	no	no
900809	814	581	1.40	yes	yes
900810	1015	607	1.67	yes	yes
900814	745	656	1.14	no	no
900815	802	777	1.03	no	no
900816	928	634	1.46	no	yes
900817	661	622	1.06	no	no
900821	644	698	1.08	no	no
900822	722	643	1.12	no	no
900824	744	607	1.23	no	no

Significance tests for the comparison of mean flows: pre-queue and queue discharge

In running the significance tests, the means ($\bar{x}_1$ for pre-queue flow and $\bar{x}_2$ for queue discharge flow) and the variances (s_1 and s_2) based on the sample sizes (n_1 and n_2) were computed. The means were calculated as the average 30-second flow data in each of the two periods on each day. The test criteria were set at 95 and 99 per cent for a one-tailed test as was used in the *F*-test. Because all the tests were one-tailed, the hypothesis is rejected only if the values of the test statistics fall into the right-hand tail of their sampling distribution. Once all these were set, the values of the test statistic were computed.

The Student-*t* test used for large samples (greater than 30) with equal variance (King, 1969) was of the form:

$$t = \frac{\bar{x}_1 - \bar{x}_2}{\sigma * ((1/n_1 + 1/n_2))^{1/2}}$$

with $n_1 + n_2 - 2$ degrees of freedom;
σ is the population variance.

Because the population variance was not known, the pooled variance which best estimates it was computed for the Student-*t* test as follows:

$$s_p^2 = \frac{(n_1 - 1)*s_1^2 + (n_2 - 1)*s_2^2}{n_1 + n_2 - 2}$$

where s_p is the pooled variance.

For the *t*-estimate, the equation used was (Nie *et al*, 1975) :

$$t = \frac{\bar{x}_1 - \bar{x}_2}{(s_1^2/n_1 + s_2^2/n_2)^{1/2}}$$

with degrees of freedom (*d.f.*) :

$$d.f. = \frac{((s_1^2/n_1) + (s_2^2/n_2))^2}{((s_1^2/n_1)^2/(n_1 - 1)) + ((s_2^2/n_2)^2/(n_2 - 1))}$$

Table 4.5 contains a summary of results of both tests as well as for the "correct" *t*. Tables 4.6 and 4.7 present the details of the results of the two tests for each day.

Table 4.5 Number of days showing a significant difference (yes) or not (no) in mean flow before and after queue.

Test type :	Student-*t*		*t*-estimate		"Correct" *t*	
Significance level :	1%	5%	1%	5%	1%	5%
YES	28	35	23	32	26	32
NO	24	17	29	20	26	20

At the five per cent level there is a difference in the mean flows in the two time periods, pre-queue and queue discharge in both tests as well as in the "correct" t in about 62 - 67 per cent of the cases (32 - 35 days).

The significance tests at the one per cent level, however, give mixed results. The difference in mean flows was not found to be significant using the t-estimate (29 as against 23 days). It was an even split for the "correct" t. The Student-t test result gives weak evidence in differentiating between the two time periods. While it is clear that there is a significant difference in mean flow, pre-queue to queue discharge, at the five per cent level, it cannot be confidently stated that this is or is not the case at the one per cent level. The trend in the daily mean flow rates, however, indicates a strong preponderance of pre-queue high flows, which shows that in 47 of the 52 cases pre-queue flow is greater than queue discharge flow (Appendix 5). The significance tests results do not give any definite answer to the capacity drop issue, and is therefore a very blunt tool in this case. A different approach was tried, making use of the daily difference between pre-queue and queue discharge flow values.

Table 4.6. A one-tailed Student-t test to compare the mean flow, pre-queue and queue discharge at station 25

			Significance	
DAY	t-test	d.f.	1%	5%
900425	0.959	310	no	no
900426	5.674	387	yes	yes*
900430	2.842	390	yes	yes*
900501	3.881	375	yes	yes
900502	5.698	372	yes	yes*
900503	3.141	357	yes	yes*
900504	-2.742	334	no	no
900508	6.862	367	yes	yes*
900509	2.809	351	yes	yes*
900511	1.860	330	no	yes
900514	2.875	350	yes	yes*
900515	3.221	312	yes	yes
900518	0.207	300	no	no*
900522	3.875	364	yes	yes*
900524	0.681	305	no	no
900525	3.801	326	yes	yes*
900528	5.197	370	yes	yes*
900530	0.599	365	no	no*
900601	2.779	315	yes	yes*
900604	3.006	367	yes	yes
900606	-2.462	325	no	no*
900607	1.064	349	no	no*
900608	5.357	323	yes	yes*
900611	-0.522	346	no	no*
900612	1.325	363	no	no*
900614	3.559	348	yes	yes*
900620	-0.040	326	no	no*
900622	2.018	359	no	yes*
900626	2.511	301	yes	yes
900627	6.619	376	yes	yes*
900628	5.306	343	yes	yes
900703	-0.082	309	no	no*
900704	0.198	297	no	no
900706	3.427	278	yes	yes
900709	4.895	260	yes	yes*
900711	1.413	297	no	no*
900713	1.521	284	no	no*
900716	3.585	310	yes	yes*
900719	2.239	307	no	yes*
900723	-1.732	275	no	no*
900724	1.882	268	no	yes*
900725	1.393	243	no	no*
900726	3.170	282	yes	yes*
900809	2.906	307	yes	yes
900810	2.194	294	no	yes
900814	2.532	334	yes	yes*
900815	3.886	375	yes	yes*
900816	1.999	332	no	yes
900817	1.056	304	no	no*
900821	2.342	328	yes	yes*
900822	3.544	343	yes	yes*
900824	0.450	252	no	no*

*appropriate test based on the F-test results at 5% level.

Table 4.7. A one-tailed t-estimate to compare the pre-queue and queue discharge mean flow at station 25.

			Significance	
DAY	t-est.	d.f.	1%	5%
900425	0.658	29	no	no*
900426	5.207	31	yes	yes
900430	2.241	40	no	yes
900501	2.753	49	yes	yes*
900502	2.825	39	yes	yes
900503	2.534	37	yes	yes
900504	-1.584	12	no	no*
900508	6.544	42	yes	yes
900509	2.137	29	no	yes
900511	1.389	45	no	no*
900514	2.857	30	yes	yes
900515	2.340	55	yes	yes*
900518	0.188	40	no	no
900522	3.319	74	yes	yes
900524	0.529	52	no	no*
900525	3.714	42	yes	yes
900528	4.653	41	yes	yes
900530	0.541	40	no	no
900601	3.246	60	yes	yes
900604	2.089	36	no	yes*
900606	-3.027	22	no	no
900607	0.846	33	no	no
900608	5.344	40	yes	yes
900611	-0.535	23	no	no
900612	1.060	29	no	no
900614	4.127	27	yes	yes
900620	-0.037	25	no	no
900622	2.162	26	no	yes
900626	1.799	38	no	yes*
900627	5.694	34	yes	yes
900628	4.052	61	yes	yes*
900703	-0.075	36	no	no
900704	2.834	24	yes	yes*
900706	2.666	64	yes	yes*
900709	4.257	83	yes	yes
900711	1.539	16	no	no
900713	1.079	17	no	no
900716	2.887	43	yes	yes
900719	1.793	29	no	yes
900723	-1.460	50	no	no
900724	2.300	26	no	yes
900725	1.239	8	no	no
900726	2.584	48	yes	yes
900809	2.261	42	no	yes*
900810	1.528	43	no	no*
900814	2.313	61	no	yes
900815	3.799	71	yes	yes
900816	1.48	37	no	no*
900817	1.008	45	no	no
900821	2.496	44	yes	yes
900822	3.246	50	yes	yes
900824	0.392	30	no	no

*appropriate test based on the F-test results at 5% level.

Confidence interval for the "true" mean of the average flow difference

The distribution of the difference of daily mean flows (which is defined as pre-queue mean flow less queue discharge mean flow) is plotted in Figure 6, which gives a visual impression of the nature of the distribution that serves as the basis for this analysis. In preparing this figure, the difference of means for each day was computed and these were grouped in cells of 200 vehicles given the maximum and minimum values in the distribution. The frequency for each cell was plotted against the interval midpoint.

It is clear from Figure 6 that the distribution is negatively skewed and that the "true" mean is unlikely to be zero. The confidence interval for the estimate of the mean (Freund, 1984) was done by the expression :

$$\bar{\bar{x}} - Z_{\alpha/2} * \frac{s}{n^{1/2}} < u < \bar{\bar{x}} + Z_{\alpha/2} * \frac{s}{n^{1/2}}$$

where: $\bar{\bar{x}}$ is mean difference (the average of difference of daily means)

$Z_{\alpha/2}$ is the z-values at the 95 or 99 per cent degree of confidence

s is the standard deviation of the difference of means

u is the estimated "true" mean difference

n is the sample size

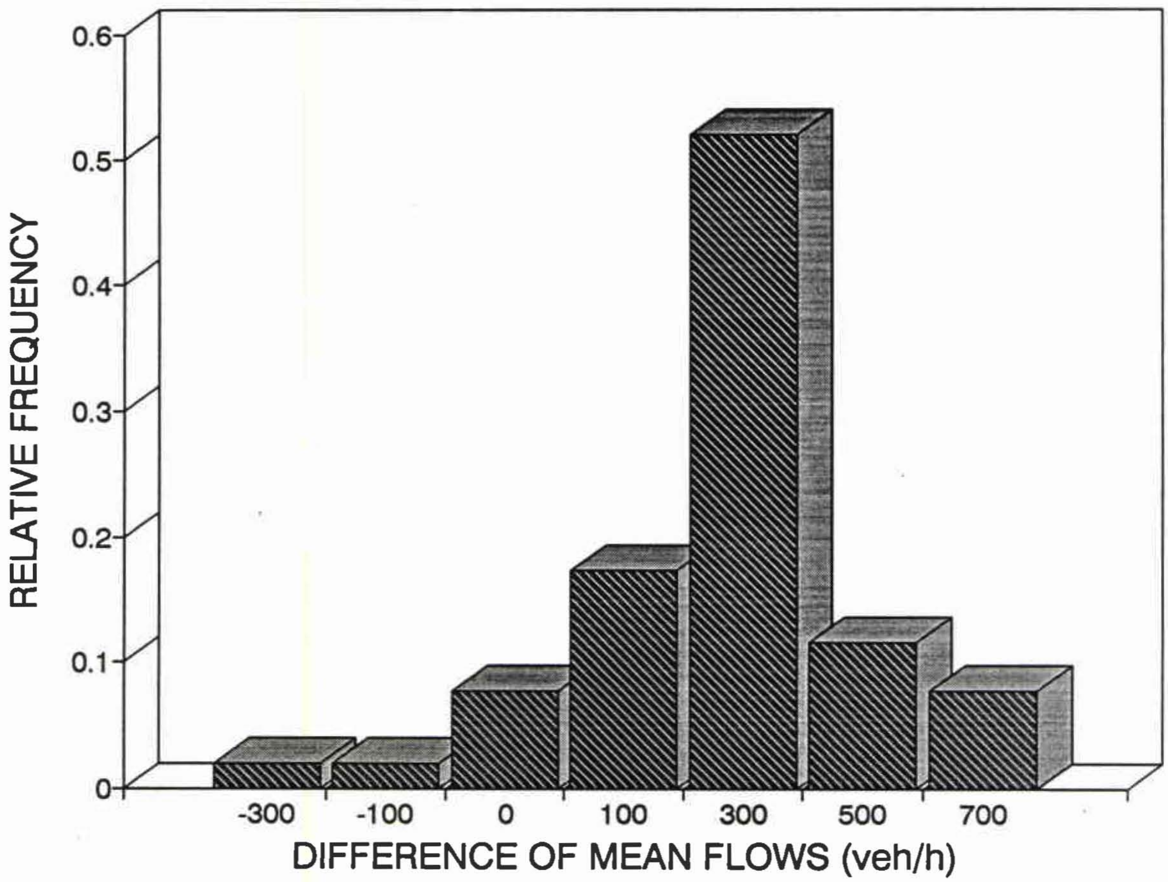

FIGURE 6. FREQUENCY DISTRIBUTION OF DIFFERENCES OF MEAN FLOWS.

With a mean difference of 269, a standard deviation of 232, and a sample size of 52, the interval that contains the "true" mean difference at the 95 per cent confidence level is 206 to 332 vehicles per hour. At the 99 per cent confidence level, the interval boundary estimates are 186 and 352. Because neither interval includes zero, it can be stated that the pre-queue flow is significantly different from the queue discharge flow. In other words, there is a drop in flow as the queue forms upstream of at least 186 vehicles per hour. The best estimate of the drop is 269 vehicles per hour over three lanes, or 90 vehicles per hour per lane.

4.3 <u>DISTRIBUTIONAL CHARACTERISTICS OF QUEUE DISCHARGE FLOWS</u>

The distributional behaviour of queue discharge flows is presented in this section for both the daily average values and the peak 15-minute flow rates on each day. The average flow rates and standard deviations for each day were calculated using 5-minute data. The means of the daily average flow and of the 15-minute flow were also computed and compared. Histograms of the distribution of the two sets of data were plotted to give a visual impression of the nature of the distribution in each case.

Daily average flow rates

The hourly flow rate at the 5-minute intervals was computed by the sum of the volume counts across all the three lanes multiplied by 12. For convenience and consistency, the start and end of the queue discharge period excluded any 30-second queue discharge flow values for which not all of the 5-minute interval was queue discharge flow. For example, in Appendix 4, the start and end times of queue discharge on 900607 were 6:32:30 and 9:12:30 respectively. For the 5-minute analysis, the queue discharge period for this day was taken to be from 6:40:00 to 9:10:00. (Note that 6:40:00 is the end time interval and the flow value is the sum total between 6:35:30 and 6:40:00 inclusive). Appendix 6 is a summary of the daily average queue discharge flow rate, the standard deviation and average travel speed for each day.

A frequency histogram was made using the daily average flows which were grouped in intervals of 100 vehicles per hour. The frequency in each interval was plotted against the interval midpoint (Figure 7). The distribution is positively skewed, with a Pearsonian coefficient of 0.028. There is a difference of only one between the mean and the median values; 6,057 as against 6,056 vehicles per hour.

Observed peak 15-minute flow rates

Both pre-queue flows lasting for 15 minutes or more and the peak 15-minute queue discharge flow rate on each day as suggested in the HCM for capacity analysis were compared. On no occasion was the daily peak 15-minute queue discharge flow rate less than 6,000 vehicles per hour; only one case for pre-queue flow was less than 6,000 (on 900703), and that was 5981 vehicles per hour over three lanes.

Following the same procedures as used in the frequency distribution of the daily average queue discharge flow data, a histogram of the maximum 15-minute flow rates was made on the same graph as in Figure 7 for easy comparison. The nature of the distribution is similar to the daily average queue discharge flows with higher mean (6287) and median (6286) values. The degree of skewness is also close to that of 5-minute data, only 0.020.

Table 4.8 shows for the 52 days, the mean, the standard deviation, and the median, the maximum, and the minimum values of the daily average queue discharge flow rate (Appendix 6), and the daily maximum 15-minute flow rate (Appendix 7). The mean of the peak 15-minute flow rates is higher than the daily average flow data, as is its standard deviation.

The peak 15-minute queue discharge flows were also compared with the pre-queue flows that meet the HCM requirement of 15-minute period. There were 34 such cases. (The peak 15-minute queue discharge flows for only these 34 days were compared). The frequency distribution of these values (Figure 8), shows the distributions are close ones. The comparison highlights the problem posed by the 15-minute sustainable flow rate used in the definition of capacity in the HCM. This problem is discussed in chapter five.

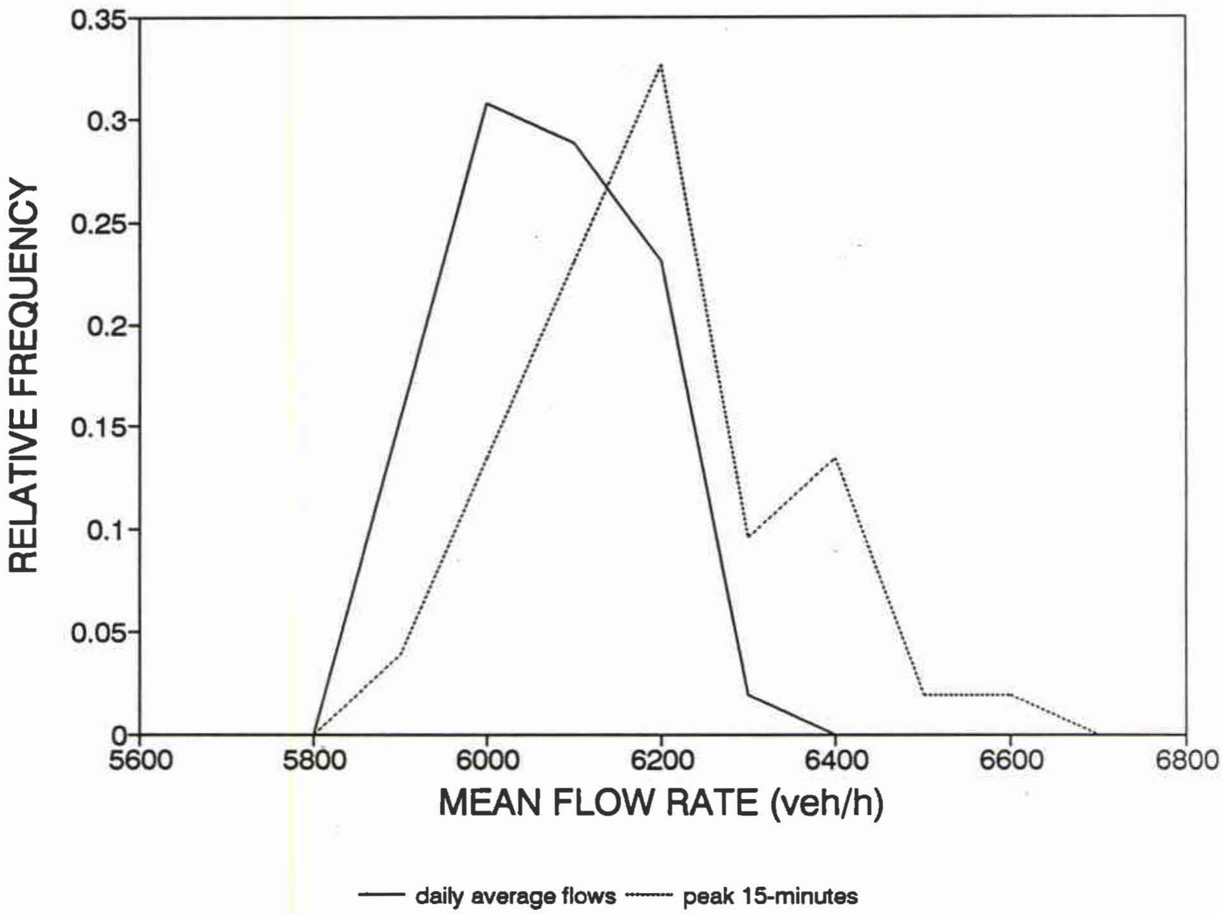

FIGURE 7. FREQUENCY DISTRIBUTIONS OF 52 DAYS OF QUEUE DISCHARGE FLOW DATA FOR DAILY AVERAGE FLOWS AND THE PEAK 15-MINUTE FLOW DATA, STATION 25.

Table 4.8 Mean, standard deviation, median, maximum and minimum values of daily average queue discharge flow rate and daily maximum 15-minute flow rate*.

	QUEUE DISCHARGE	
	Daily average	Max. 15-min.
Mean	6057	6287
Standard deviation	109	147
Median	6056	6286
Maximum flow rate	6307	6656
Minimum flow rate	5854	6008

*All figures are expressed in vehicles per hour.

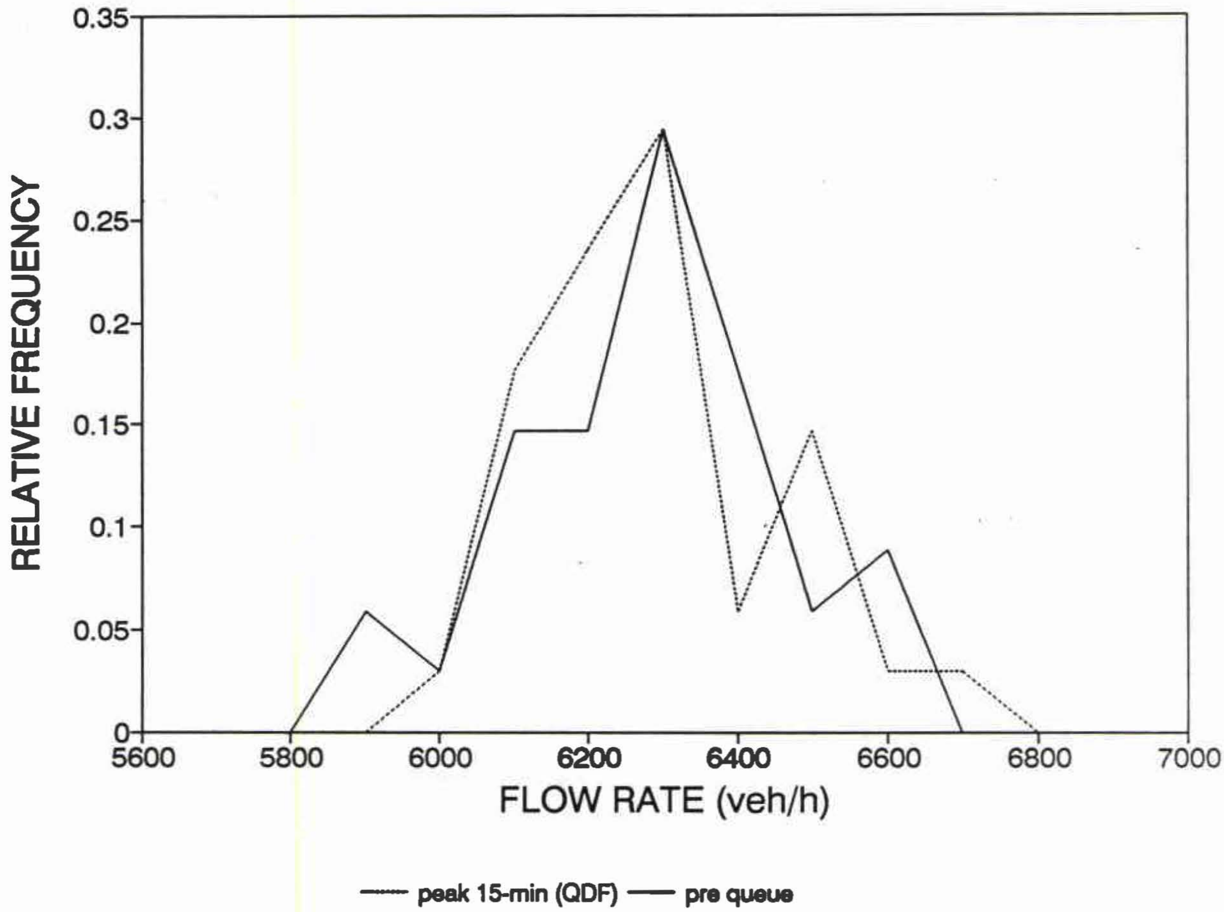

FIGURE 8. PRE QUEUE FLOWS (15 MINUTES OR MORE) AND THE PEAK 15-MINUTE QUEUE DISCHARGE FLOW RATES COMPARED, STATION 25.

CHAPTER FIVE

DISCUSSIONS AND CONCLUSIONS

The discussion covers three main areas. They are the validation of capacity drop in a bottleneck, the definition capacity (conceptually and numerically), and the level-of-service concept (LOS) for capacity flows. Conclusions are also presented.

5.1 THE VALIDATION OF CAPACITY DROP IN THE BOTTLENECK

The statistical analysis of the two-capacity hypothesis in section 4.2 supports the HCM assertion of a drop in capacity when a queue forms upstream. The HCM and the findings in this study, however, differ with respect to the underlying logic, and hence with the location on the freeway where the drop can be seen. Flow and occupancy data on the same day for the same time period but collected from two different locations (upstream and downstream of a bottleneck) are used in this discussion.

Figure 9 presents data from station 22, just upstream of a major entrance ramp, on 900511 from 5:30 AM to 10:00 AM. There are two portions of the graph: the left-hand side represents uncongested data which occur at relatively lower occupancies than the congested data found in the right-hand portion. There is a slight drop in volume just above an occupancy of 20 per cent. Figure 9 looks like the

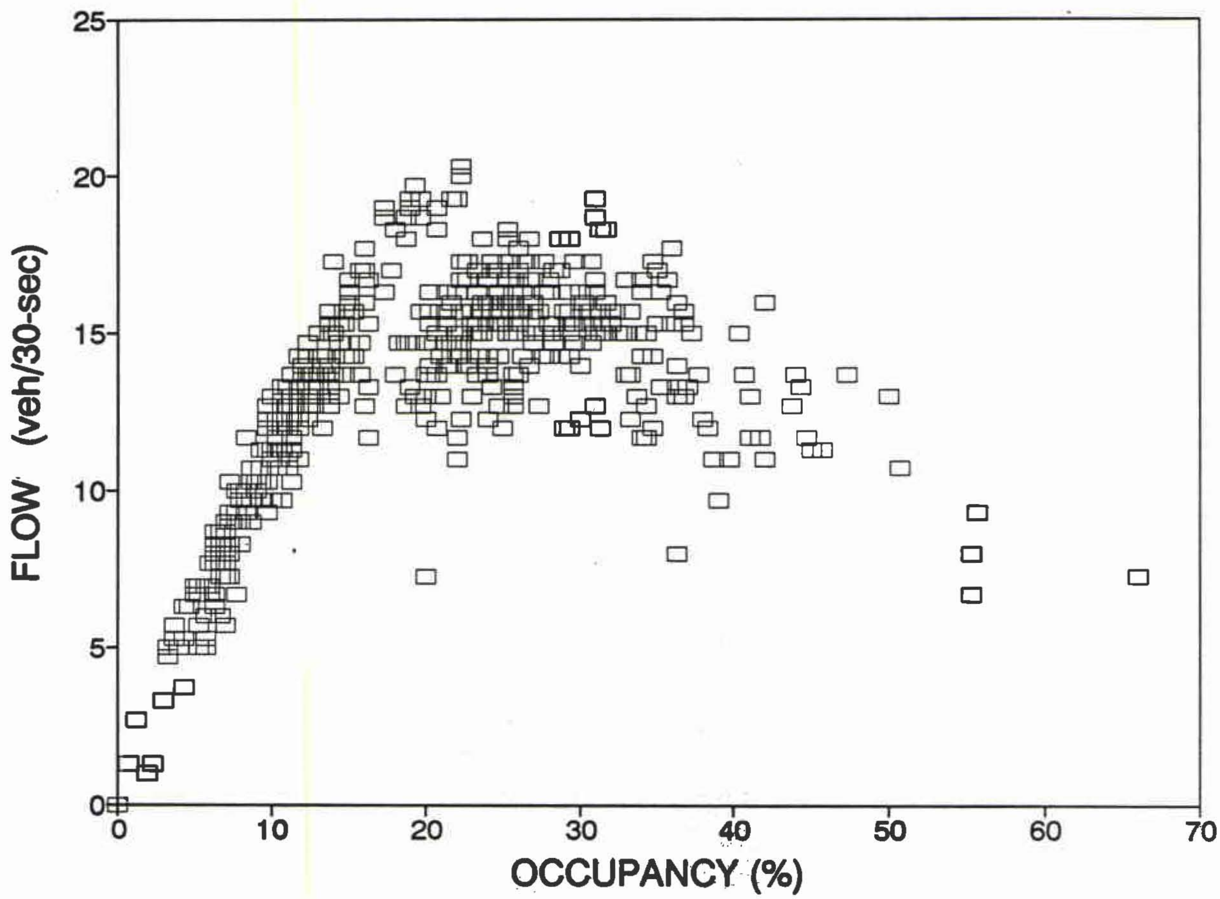

FIGURE 9. FLOW-OCCUPANCY PLOT. 30-SECOND AVERAGE VALUES; STATION 22. MAY 11, 1990; 5:30 A.M.-10:00 A.M.

"discontinuous function" shown in Figure 1 which essentially represents the HCM version of capacity drop. However, my interpretation of Figure 9 is that at higher occupancies with comparatively lower volumes, the station does not operate at capacity level because of the presence of congestion.

Figure 9 is contrasted with Figure 10, which displays data for the same day and the same time period, and on the same scale as in Figure 9, but are from station 25 **in** the bottleneck. The data in Figure 10 cover three different operating conditions: pre-queue, queue discharge and post-queue (defined as the period after queue dissipation). The pre-queue and post-queue data occur at lower occupancies. In terms of volume, the pre-queue data are higher than the queue discharge data which in turn generally lie above the post-queue data. In general, there is one similarity in Figures 9 and 10: uncongested data points occur at lower occupancies in both pre-queue and queue discharge periods. The two graphs, however, differ significantly in shape, the distinguishing feature being the disappearance in Figure 10, of the right-hand section of the graph in Figure 9. Note that station 25 operates at capacity because of the continuous supply of vehicles from the reservoir of traffic at station 22.

The inference drawn from this comparison is that capacity drop per the HCM is based on data collected in a queue. Capacity drop as described in the HCM is therefore founded on a mistaken premise and hence is flawed. It even contradicts

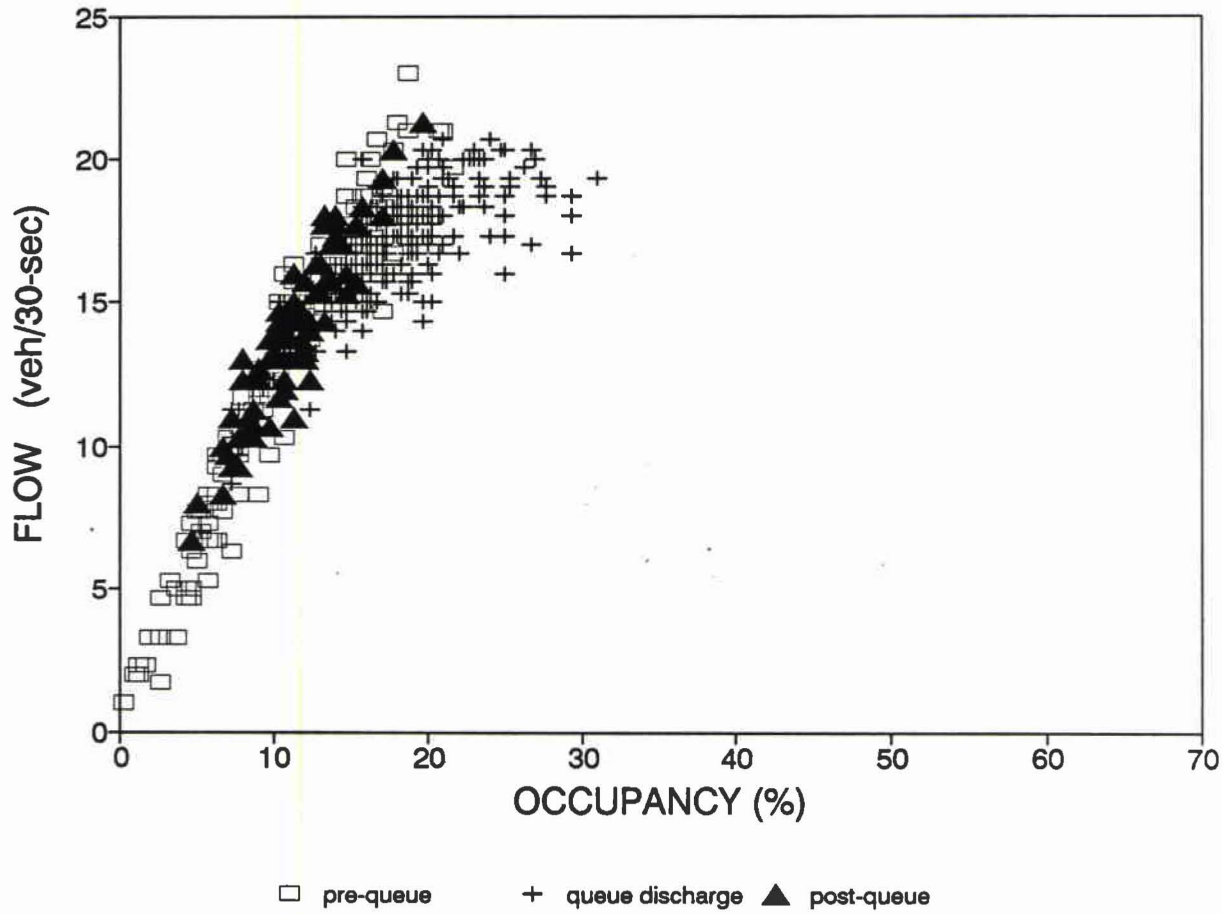

FIGURE 10. FLOW-OCCUPANCY PLOT. 30-SECOND AVERAGE VALUES; STATION 25. MAY 11, 1990; 5:30 A.M.-10:00 A.M.

the HCM characterisation of capacity as sustainable flow which implicitly rules out any unstable flow condition, a common feature of operations in a queue. The place to look for capacity drop and hence capacity is **in the** bottleneck. The most obvious explanation for the drop in flow is that the congestion created by the pool of upstream traffic does not allow vehicles to get through the bottleneck at its capacity rate.

One important thing that arose during the analysis of capacity drop is the method used. Past research on capacity drop has concentrated on the comparisons of the mean flow rate before and after the queue, looking at each day separately, which involves the Student-*t* test or its approximation. The choice of any one depends on the *F*-test results.

The analytical problem which seems to be glossed over in the literature is how to test when there are a number of days of data. Is it appropriate to compare the daily mean flow before and after the queue? Or should one conduct the test on the distribution of the difference between the two means? If the concern regarding the capacity drop issue is how significant is the drop, then it appears to be right to focus the analysis on the difference. The conclusions of some past studies on capacity drop (for instance Banks 1990), have been based on a "majority rule decision" according to the number of cases that were significant or not in the test statistic results. The rate of the drop in flow was determined by the difference in mean flows, pre-queue and queue discharge. Hall and Agyemang-Duah did this and found a difference of about 5 - 6 percent. Hurdle and Datta (1983), Banks, and Urbanik and Barnes, among others were not specific on the extent of the drop. Although the difference in the two figures is negligible, it is important to use the appropriate procedure to determine how large is the drop, in this case, the use of the distribution of difference of means.

5.2 THE DEFINITION OF CAPACITY, CONCEPTUALLY AND NUMERICALLY

The discussion on the definition of capacity is based on two elements. These are the notion of sustainable flow and the utility of the concept of capacity. The HCM uses a 15-minute period in its definition of sustainable flow as capacity. The adoption of this criterion in this discussion will mean there are two capacities. One capacity is pre-queue which can last for 15 minutes or more (about 65 per cent of the time or 34 days), and the other is queue discharge, which can be sustained for up to three hours. In all but one of the 34 cases, the pre-queue flow rate was higher than queue discharge.

It has been contended that capacity should be referred to as the highest flow observed (HCM, 1985). If a minimum counting interval of 15 minutes is to be used, and if capacity is the flow rate that can be repeatedly achieved, then defining capacity as the maximum sustained pre-queue flow is conceptually sound. This definition of capacity as the high pre-queue flow may, however, be of limited utility for freeway management. For capacity defined as pre-queue flow to be useful, it will mean, for instance, preventing congestion on the freeway for more than 30 minutes, the maximum observed time period for pre-queue high flows, which seems to be a remote possibility. Capacity defined as high queue discharge flow seems to be a more useful and practical concept in freeway control. This is because when there is a breakdown of the freeway, it is the queue discharge rate that will determine the time to recovery and not the pre-queue flow rate.

The determination of the numerical value of capacity for any given distribution is equally difficult whether capacity is defined as pre-queue flow or queue discharge. The practice in estimating the numerical value of capacity has been to select the mean value. This is not strange since almost all previous capacity studies have been based on very limited data. The problem of what percentile or proportion of the distribution to select for the numerical value arises when there is a distribution as in the current study. Should the 15th or 85th percentile, for example, be chosen? An indifference to the choice of any of the mean, mode, or the median is only appropriate when the distribution is a Normal one. Reliance on the normality of the distribution may have its own special problem as came out in this study. The closeness of all the three measures will make one believe that the distribution is a near-Normal one. Nevertheless, the graphical presentation of the frequency distribution of the daily average flow rates and the peak 15-minute queue discharge flows is to the contrary (Figure 7). The distribution is clearly skewed but how this should be interpreted is not certain. Despite these problems, the important thing is that the portion selected should be the one the engineer or the planner or the freeway traffic operator can reasonably be assured of achieving on a daily basis. For the observed daily average flow rates and the peak 15-minute queue discharge flows, it does not really matter if the mean, or the median or the mode is selected since the difference among the three is very small, a little over 50 and 60 vehicles per hour respectively. The engineer or the planner does not run a risk of a system failure if any of the measures is chosen.

Certainly, whether capacity is defined as pre-queue flow or queue discharge, and whether the mean, median or the modal value is to be selected to represent capacity, sound professional judgement is important in the application of the capacity concept. But to be consistent with the HCM, and given the closeness of all the three measures, the mean value is used in the numerical definition of capacity.

5.3 <u>CAPACITY AND LEVEL-OF-SERVICE (LOS) CONCEPT</u>

The operating characteristics under capacity flows found in this study contradict the LOS under which capacity is said to operate according to the HCM. Operations at capacity are said to be "extremely unstable" and the average travel speed is estimated to be 50 km/hour.

In the case of pre-queue high flows, the characterisation of capacity flow in the HCM is inconsistent with what was found in this section of the QEW. The average travel speed was about 80 km/hour at a flow rate of more than 2,000 vehicles per hour. For queue discharge, the average speed will depend on how far the point of measurement is from the point of the constriction (Persaud and Hurdle, 1988). Figure 11 illustrates the relationship between speed and flow on 900425 at station 25. The graph covers three different flow conditions denoted by three symbols : empty squares, plus (+) signs and asterisks (*) for pre-queue, queue discharge and post-queue respectively. Daily average speed and flow data when vehicles were discharging from a queue (Appendix 6) for all the 52 days were also plotted (Figure 12). (The x-axis in Figure 12 was not to the same scale as in Figure 11; the data points would have clustered almost at one portion of the graph if the same scale was to be used). Figures 11 and 12 deviate from the HCM curve under high flows (Figure 13). The reason for the observed discrepancy may be that the data behind the HCM curve were taken within a queue (although there is no background information provided about location of the data for the HCM curve). The 50 km/hour average travel speed in the HCM is most likely to be found in a queue, but operations are not at capacity within the queue. Because of a restrictive downstream environment, vehicles are forced to slow down, reducing the flow rate.

It would have been a good idea to compare speed data from stations 22 and 25. This was not possible because station 22 (which is single loop) does not record speed. At station 25, a distance of about 1.5 kilometres from the head of the queue, the average travel speed of vehicles discharging from the queue was about 74 km/hour, also at a rate of about 2,000 vehicles per hour per lane. Little evidence was found of the downstream shock waves which might have caused the momentarily sharp drop in flows on some days at the onset of a queue upstream. In about 35 per cent (18 days) of the cases, flow dropped below 5,500 vehicles per hour across all three lanes for up to two minutes when a queue formed upstream; the only exception was one day when high and low flows alternated for about six minutes. These few, short-lived cases are not enough to associate capacity flows with disruption waves as described in the HCM.

It is specifically stated in the HCM that shock waves travel in the opposite direction of traffic (p.3-2). (There are reports that shock waves can be stationary or can travel downstream, see Walker, 1989). Upstream of the bottleneck,

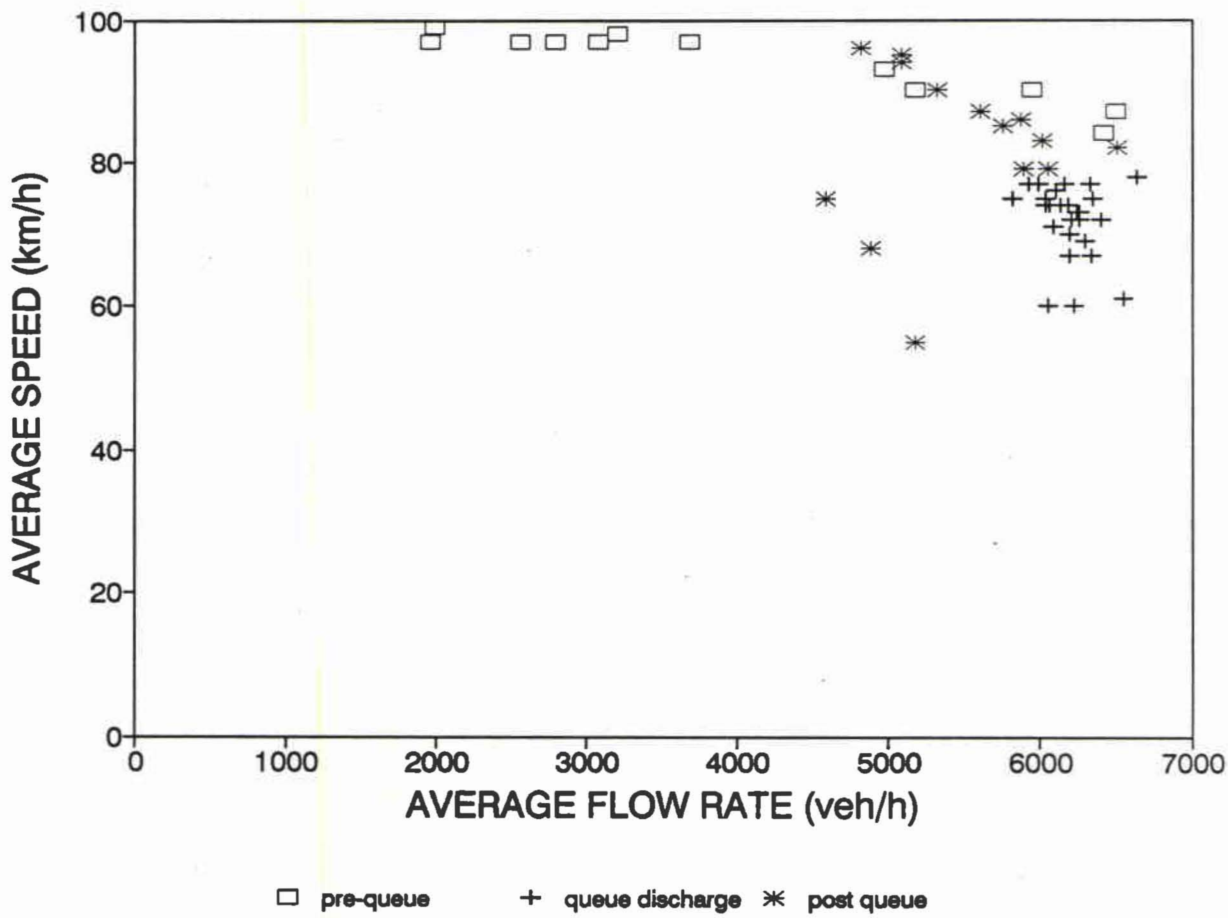

FIGURE 11. SPEED-FLOW RELATIONSHIP. 5-MINUTE VALUES; STATION 25. APRIL 25, 1990; 5:30 A.M.-10:00 A.M.

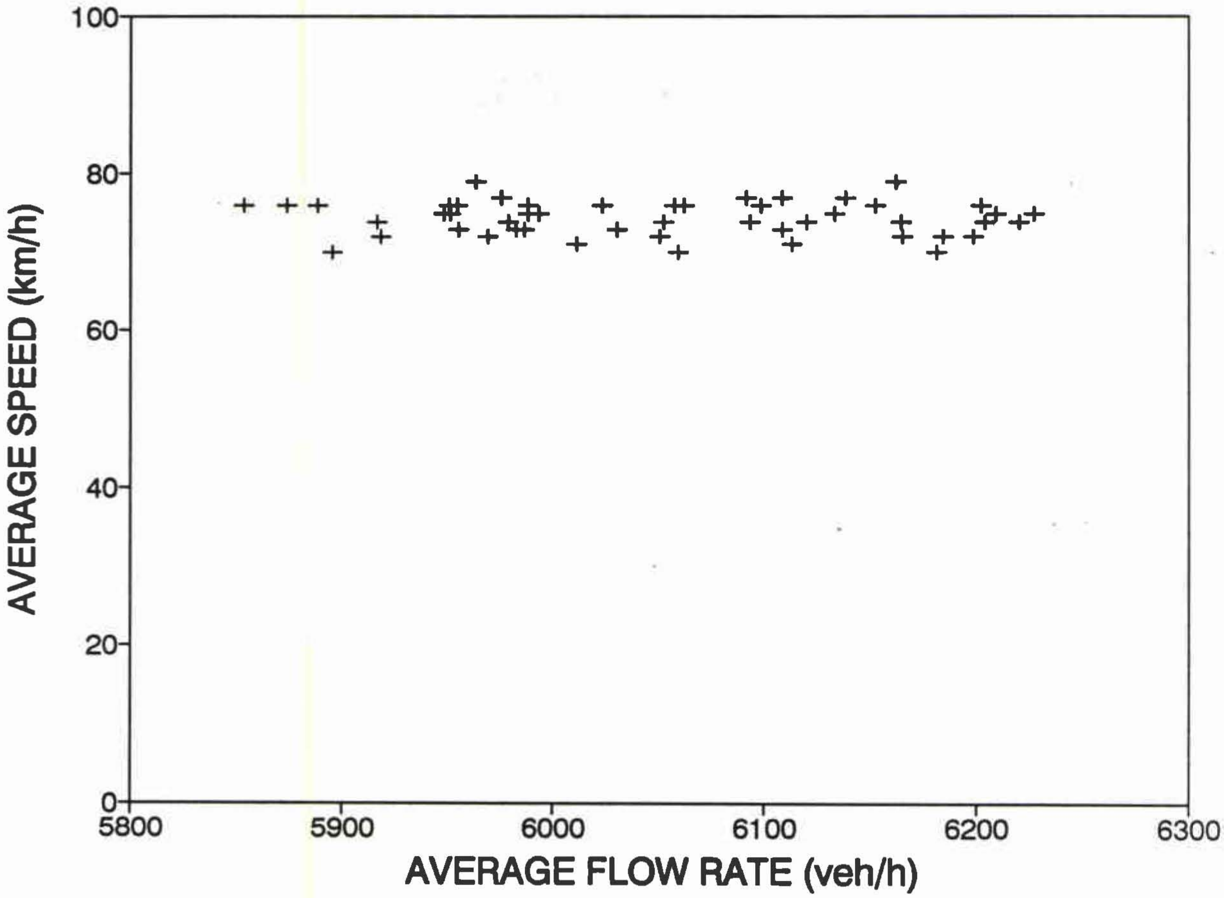

FIGURE 12. SPEED-FLOW RELATIONSHIP. DAILY AVERAGE VALUES; STATION 25.

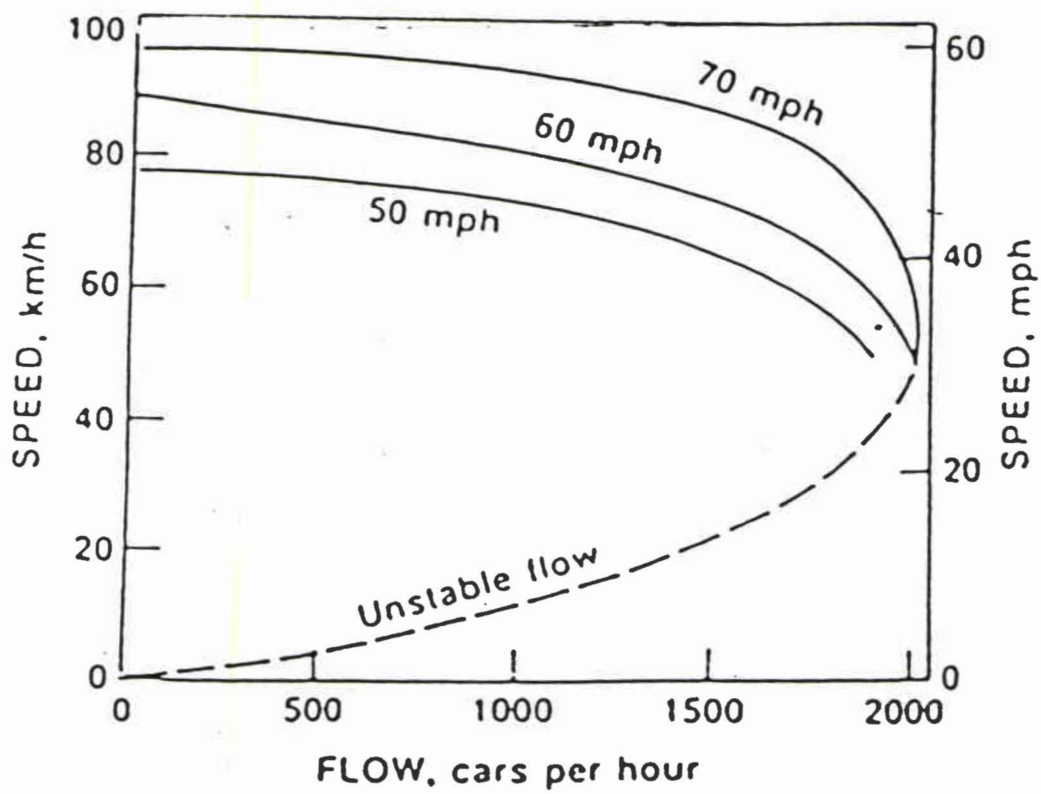

FIGURE 13. SPEED-FLOW CURVES FROM THE HIGHWAY CAPACITY MANUAL, 1985.

two types of flow condition are identified : free-flow and congested flow. Under free-flow condition, capacity may be reached without the development and subsequent propagation of shock waves. When the shock waves eventually develop (due to increased concentration of vehicles) there is a breakdown on the system and conditions quickly change from free-flow to congestion. Under the congested conditions, the freeway does not operate at capacity and the shock waves (according to the HCM) sweep through the upstream traffic.

5.4 CONCLUSIONS

More than 50 days of traffic volume, occupancy and speed data were used in the investigation of the capacity of a section of the QEW. The method of data collection (by loop detectors) made possible not only this large sample size which is perhaps the largest in any capacity study but also ensured a high degree of reliability in volume counting and occupancy and speed measurements. Despite the large sample size, the validation of the results with data from other freeways is important. Hence one has to be cautious in generalizing some of the findings because capacity certainly differs in time and may well vary across different systems (space) as well.

The analysis of the 30-second flow data indicates that capacity under free-flow condition in the bottleneck is greater than the capacity when vehicles are discharging from an upstream queue. The conclusion that there is a drop in capacity was determined by the nature of the distribution of differences between pre-queue and queue discharge mean flows, and calculation of the confidence interval that contains the "true" mean difference. The mean difference of capacity flows before and after a queue forms upstream was found to be 98 pcphpl, after taking into account the truck percentage of six and a passenger car equivalent value for trucks of 1.5 (Reilly *et al*, 1991). The drop in capacity is important to freeway control in two respects; first, it gives an idea of what is happening at a specific section of the freeway, in this case, station 22. Secondly, the drop in capacity implies that if congestion can be delayed or prevented, the best use of the freeway's capacity can be made.

Both the pre-queue and queue discharge capacities satisfy the HCM criterion of 15-minute sustainable flow. Pre-queue capacity flows lasting for at least 15 minutes were observed in about 65 per cent of the cases investigated with the longest lasting for 30 minutes. Queue discharge capacity was sustained for 100 minutes or more on all the 52 days used and even for three hours on one day. Clearly, the capacity when vehicles are discharging from a queue has a more "reasonable expectation" of being sustained on a daily basis than pre-queue capacity.

The speed characteristics of the observed flows suggest that there was no precipitous drop in speed at capacity. The average travel speed at capacity when there was no queue was about 80 km/hour. When vehicles were discharging from an upstream queue, at a distance of about 1.5 kilometres from the queue the average travel speed was found to be about 74 km/hour. Clearly, speeds remained well above the 50 km/hour as stated in the HCM.

Regarding the numerical value of capacity, the mean of the weighted daily average flow rates is used. When the truck percentage of 6 and passenger-car equivalent value of 1.5 for a level ground are applied, the capacity under free-flow conditions is 2,306 pcphpl which is rounded off to 2,300 pcphpl. For queue discharge capacity, this drops to 2,200 pcphpl. (The difference between these two figures is 106. This is higher than the 98 pcphpl drop in capacity because of the weighting by duration of flows here, which does not enter in estimating the mean difference in pre-queue and queue discharge flows). These figures compare very well with the capacity of 2,200 pcphpl for multi-lane rural highways (Reilly *et al*, 1990). Although these figures are only 10 -15 per cent over the 2,000 pcphpl as stated in the H.C.M., they point out the need to update the numerical value of capacity to reflect the changes in the roadway and traffic conditions, vehicle downsizing and driver behaviour.

REFERENCES

1. Banks, J.H. 1990. "Flow process at a freeway bottleneck". A paper presented at the 69th Annual Meeting of Transportation Research Board, Washington, D.C.

2. Burtch, T.M., R.W. Sanderson, and P.S. Irwin. 1983. "Vehicle downsizing and roadway design in Canada". ITE Journal, February edition. pp.14-19.

3. Ceder, A. and A.D. May. 1976. "Further evaluation of single- and two-regime traffic flow models". Transportation Research Record 567. Washington, D.C. pp.1-15.

4. Chin, H.C and A.D. May. 1991. "An examination of the speed-flow relationship at the Caldecott Tunnel". Presented at the 70th Annual Meeting of Transportation Research Board, Washington, D.C.

5. Edie, L.C. 1961. "Car-following and steady-state theory for noncongested traffic". Operations Research 9. pp.66-76

6. Freund, J.E. 1984. Modern elementary statistics. 6th edition. Prentice-Hall.

7. Greenshields, B.N. 1934. "A study of traffic capacity". Proceedings, 14th Annual Meeting of the Highway Research Board. pp.448-474.

8. Hall, F.L. and K. Agyemang-Duah. 1991. "Freeway capacity drop and the definition of capacity". Presented at the 70th Annual Meeting of Transportation Research Board, Washington, D.C.

9. Hall, F.L. and L.M. Hall. "Capacity and speed-flow analysis of the QEW in Ontario". Presented at the 69th Annual Meeting of the Transportation Research Board, Washington, D.C.

10. *Highway Capacity Manual: Practical Applications of Research. 1950*. Highway Research Board.

11. *Highway Capacity Manual 1965*. Highway Research Board, Special Report 87.

12. *Highway Capacity Manual 1985*. Transportation Research Board, Special Report 209.

13. Hurdle, V.F. and P.K. Datta. 1983. "Speeds and flows on

an urban freeway: some measurements and a hypothesis". Transportation Research Record 905. pp.127-157.

14. King, L.J. 1969. Statistical analysis in geography. Prentice-Hall.

15. Lighthill, M.J. and G.B. Witham. 1964. "On kinematic waves: II. A theory of traffic flow on long crowded roads". Highway Research Board, Special Report 79. pp.8-35.

16. McShane, W.R and R.P. Roess. 1990. Traffic Engineering. Prentice Hall.

17. *Ministry of Transportation*, QEW - Mississauga Freeway Traffic Management Systems, Ontario, Canada. 1990.

18. Nie, N.H. *et al*. 1975. Statistical package for the social sciences. 2nd edition. McGraw Hill.

19. Persaud, B.N., F.L. Hall and L.M. Hall. 1990. "Congestion identification aspects of the McMaster Incident Detection Algorithm". Presented at the 69th Annual Meeting of Transportation Research Board, Washington, D.C.

20. Persaud, B.N. and V.F. Hurdle. 1988. "Some new data that challenge some old ideas about speed-flow relationship". Transportation Research Record 1194. pp.191-198.

21. Prigogine, I. 1961. "A boltzmann-like approach to the statistical theory of traffic flow". Theory of traffic flows: Proceedings. ed R. Herman. Elsevier, Amsterdam. pp.158-164.

22. Reilly, W.R. *et al*. 1991. *Capacity and level-of-service. Procedures for multi-lane rural and suburban highways.* Final Report.

23. Roess, R.P., W.R. McShane and L.J. Pignataro. 1979. "Freeway level-of-service: a revised approach". Transportation Research Record 699. pp.7-18.

24. Urbanik II, T., W. Hinshaw and K. Barnes. 1991. "Evaluation of high volume Urban Texas Freeways". Presented at the 70th Annual Meeting of Transportation Research Board, Washington, D.C.

25. Walker, J. 1989. "The amateur scientist: how to analyze the shock waves that sweep through expressway traffic". Scientific America. August edition.

26. Wardrop, J.G. 1952. "Some theoretical aspects of road traffic research". Proceedings of the Institute of Civil Engineering Part III 1(2). pp.325-362.

27. Wattleworth, J.A. 1963. "Some aspects of macroscopic freeway traffic flow theory". Traffic engineering 34:2. pp.15-20.

28. Woods, D.L. 1983. "Small car impacts on highway design". ITE Journal. April edition. pp.20-24.

APPENDIX 1

Examples of queue start times given by the flow-occupancy boundary function and the visual methods at station 22.

Date	Boundary function		Visual (Macspin)	
900425	6	41.00	6	26.30
900426	6	38.30	6	30.30
900430	6	30.00	6	30.00
900501	6	46.30	6	33.00
900502	6	46.00	6	31.00
900503	6	42.00	6	29.00
900504	6	21.30	6	21.00
900508	6	34.00	6	31.00
900509	6	26.30	6	25.30
900511	6	33.00	6	33.00
900514	6	32.00	6	28.00
900515	6	51.00	6	36.00
900518	6	35.00	6	35.00
900522	6	38.30	6	40.30
900524	6	48.30	6	35.30
900525	6	49.30	6	33.00
900528	6	59.30	6	32.30
900530	6	33.30	6	32.30

APPENDIX 2
Speed characteristics at station 25

time	interval	900425 spd	900426 spd	900514 spd	900524 spd	900525 spd
6	20.00	92	79	87	92	85
6	20.30	89	69	87	88	90
6	21.00	84	77	85	90	89
6	21.30	80	84	83	90	85
6	22.00	87	84	85	89	86
6	22.30	88	90	80	92	88
6	23.00	85	85	85	88	84
6	23.30	84	82	88	89	79
6	24.00	85	79	85	96	79
6	24.30	80	75	86	98	83
6	25.00	82	81	84	89	87
6	25.30	90	80	82	89	87
6	26.00	86	81	85	97	90
6	26.30	80	81	88	95	95
6	27.00	84	83	88	90	92
6	27.30	85	87	81	94	94
6	28.00	90	84	83	85	88
6	28.30	81	85	84	86	86
6	29.00	78	84	86	91	91
6	29.30	78	85	81	94	92
6	30.00	76	81	80	87	89
6	30.30	79	82	75	91	84
6	31.00	79	79	75	90	84
6	31.30	73	81	76	86	84
6	32.00	78	80	75	79	81
6	32.30	76	78	71	82	81
6	33.00	80	80	74	87	81
6	33.30	76	84	79	87	87
6	34.00	80	79	81	89	82
6	34.30	72	77	78	83	80
6	35.00	75	74	77	80	81
6	35.30	76	77	69	87	81
6	36.00	73	76	79	85	78
6	36.30	74	73	82	90	74
6	37.00	81	77	77	88	71
6	37.30	81	85	75	89	74
6	38.00	80	81	72	83	77
6	38.30	77	79	73	72	73
6	39.00	80	81	78	78	75
6	39.30	79	73	81	84	76
6	40.00	76	72	74	86	78

APPENDIX 3
Speed characteristics at station 23.

time	interval	900425 106	900426 avg.spd	900514 avg.spd	900524 avg.spd	900525 avg.spd
6	20.00	106	108	99	106	102
6	20.30	108	102	95	105	100
6	21.00	99	104	107	109	108
6	21.30	99	103	102	104	100
6	22.00	107	89	105	101	100
6	22.30	100	84	104	104	102
6	23.00	106	89	102	111	101
6	23.30	104	92	98	112	102
6	24.00	92	91	100	102	106
6	24.30	92	103	98	106	103
6	25.00	93	91	100	72	104
6	25.30	113	96	95	68	107
6	26.00	95	108	96	102	108
6	26.30	101	104	87	108	104
6	27.00	104	101	83	101	103
6	27.30	93	92	79	104	103
6	28.00	89	86	74	106	105
6	28.30	92	95	73	111	108
6	29.00	88	86	72	103	107
6	29.30	90	84	65	103	98
6	30.00	86	90	67	94	97
6	30.30	84	95	54	89	91
6	31.00	85	96	52	97	95
6	31.30	90	93	45	99	101
6	32.00	82	94	42	102	96
6	32.30	-1*	94	40	99	95
6	33.00	79	92	55	95	96
6	33.30	82	90	57	94	91
6	34.00	82	93	57	103	92
6	34.30	74	97	55	96	96
6	35.00	70	95	56	97	91
6	35.30	64	88	58	95	91
6	36.00	52	90	53	97	85
6	36.30	48	86	59	100	95
6	37.00	45	84	61	93	88
6	37.30	46	83	60	95	78
6	38.00	52	85	57	99	72
6	38.30	52	81	53	90	47
6	39.00	41	80	41	92	48
6	39.30	48	78	46	87	55
6	40.00	48	82	51	85	57

* no speed was recorded

APPENDIX 4

Start and end times and durations (in minutes) of pre-queue and queue discharge periods, station 25.

	PRE-QUEUE			QUEUE DISCHARGE		
DATE	START	END	DURATION	START	END	DURATION
900425	6:14:00	6:28:00	14	6:28:00	8:50:00	142
900426	6:18:00	6:32:00	14	6:32:00	9:32:30	180.5
900430	6:13:00	6:31:30	18.5	6:31:30	9:29:00	177.5
900501	6:12:00	6:34:30	22.5	6:34:30	9:20:30	166
900502	6:13:00	6:32:30	19.5	6:32:30	9:20:00	167.5
900503	6:13:30	6:30:30	17	6:30:30	9:13:00	162.5
900504	6:16:30	6:23:00	6.5	6:24:00	9:05:30	161.5
900508	6:14:30	6:32:30	18	6:32:30	9:19:00	166.5
900509	6:13:30	6:27:30	14	6:27:30	9:10:00	162.5
900511	6:14:00	6:34:30	20.5	6:35:00	9:00:00	145.5
900514	6:16:00	6:29:30	13.5	6:29:30	9:12:00	162.5
900515	6:13:00	6:37:30	24.5	6:37:30	8:50:00	132.5
900518	6:19:30	6:36:30	17	6:38:00	8:50:30	134
900522	6:12:00	6:42:00	30	6:42:00	9:15:00	153
900524	6:14:30	6:37:00	22.5	6:37:00	8:48:00	131
900525	6:17:00	6:34:30	17.5	6:34:30	9:01:00	146.5
900528	6:16:00	6:34:00	18	6:34:00	9:22:00	168
900530	6:16:30	6:34:00	17.5	6:34:00	9:20:00	166
900601	6:22:00	6:44:00	22	6:45:30	9:03:30	138
900604	6:18:30	6:34:00	15.5	6:34:30	9:21:30	167.5
900606	6:20:30	6:30:00	9.5	6:31:00	9:05:30	154
900607	6:17:00	6:32:30	15.5	6:32:30	9:12:30	160
900608	6:23:00	6:39:30	16.5	6:39:30	9:05:30	146
900611	6:21:00	6:31:30	10.5	6:31:30	9:15:00	163.5
900612	6:22:00	6:35:30	13.5	6:36:00	9:25:00	169
900614	6:22:00	6:33:00	11	6:33:00	9:16:30	163.5
900620	6:18:00	6:29:30	11.5	6:33:00	9:05:30	152.5
900622	6:19:30	6:31:00	11.5	6:36:00	9:25:00	169
900626	6:15:00	6:32:30	17.5	6:33:30	8:47:30	134

APPENDIX 4 (continued)

	PRE-QUEUE			QUEUE DISCHARGE		
DATE	START	END	DURATION	START	END	DURATION
900627	6:17:00	6:32:30	15.5	6:32:30	9:26:00	173.5
900628	6:16:00	6:42:30	26.5	6:43:00	9:09:00	146
900703	6:14:30	6:30:00	15.5	6:30:30	8:50:30	140
900704	6:22:00	6:36:00	14	6:42:00	9:00:00	138
900706	6:23:00	6:50:00	27	6:50:30	8:43:30	113
900709	6:18:00	6:48:30	30.5	6:48:30	8:29:00	100.5
900711	6:24:00	6:31:30	7.5	6:32:00	8:54:00	142
900713	6:23:00	6:31:30	8.5	6:32:00	8:46:30	134.5
900716	6:19:30	6:38:30	19	6:40:00	8:57:00	137
900719	6:19:00	6:32:30	13.5	6:34:00	8:55:00	141
900723	6:16:30	6:37:30	21	6:38:30	8:36:00	117.5
900724	6:25:00	6:35:30	10.5	6:35:30	8:40:00	124.5
900725	6:50:00	6:54:30	4.5	6:54:30	8:52:30	118
900726	6:22:00	6:42:30	20.5	6:43:30	8:45:00	121.5
900809	6:22:00	6:40:30	18.5	6:41:00	8:56:30	135.5
900810	6:22:00	6:42:00	20	6:42:00	8:50:00	128
900814	6:18:00	6:42:30	24.5	6:42:30	9:06:00	143.5
900815	6:17:30	6:44:30	27	6:44:30	9:26:00	161.5
900816	6:18:30	6:35:30	17	6:35:30	9:05:30	150
900817	6:21:00	6:39:30	18.5	6:39:30	8:54:00	134.5
900821	6:26:00	6:43:30	17.5	6:43:30	9:11:00	147.5
900822	6:20:30	6:41:30	21	6:41:30	9:13:00	151.5
900824	6:23:00	6:35:30	12.5	6:35:30	8:45:30	130

APPENDIX 5

Mean flow rates, standard deviations and sample sizes (count) for pre-queue and queue discharge, 30-second data, station 25.

	PRE-QUEUE			QUEUE DISCHARGE FLOW		
DAY	flow rate	std.dev.	count	flow rate	std.dev.	count
900425	6300	900	28	6186	563	284
900426	6651	672	28	5969	608	361
900430	6208	760	37	5920	566	355
900501	6389	947	45	5990	596	332
900502	6268	761	39	5921	280	335
900503	6424	728	34	6098	558	325
900504	5750	1034	13	6207	566	323
900508	6690	578	36	6030	545	333
900509	6467	802	28	6136	578	325
900511	6392	867	41	6198	584	291
900514	6338	682	27	5948	677	325
900515	6585	1015	49	6234	627	265
900518	6247	680	34	6224	602	268
900522	6238	806	60	5872	639	306
900524	6395	927	45	6319	643	262
900525	6559	583	35	6173	566	293
900528	6507	656	36	5978	572	336
900530	6123	610	35	6065	538	332
900601	6259	618	44	5912	763	276
900604	6367	1026	31	5992	651	335
900606	5703	474	19	6048	599	308
900607	6298	724	31	6185	547	320
900608	6745	633	33	6124	631	292
900611	5989	564	21	6057	579	327
900612	6333	771	27	6172	593	338
900614	6563	490	22	6121	581	327
900620	5943	633	23	5948	573	305
900622	6099	523	23	5854	566	338
900626	6387	852	35	6121	547	268
900627	6732	740	31	5952	618	347
900628	6503	855	53	6008	574	292
900703	5981	642	31	5990	575	280
900704	6407	830	28	5953	543	276
900706	6480	903	54	6136	592	226
900709	6354	762	61	5903	585	201
900711	6288	544	15	6065	598	284
900713	6318	870	17	6087	588	269
900716	6379	798	38	5991	598	274
900719	6347	832	27	6052	635	282
900723	6029	805	42	6220	629	235
900724	6326	617	21	5996	783	249
900725	6280	692	9	5990	610	236
900726	6430	791	41	6096	592	243
900809	6303	814	37	5994	581	271
900810	6198	1015	40	5946	607	256
900814	6235	745	49	5973	656	287
900815	6376	802	54	5930	777	323
900816	6462	928	34	6220	634	300
900817	6255	661	37	6139	622	269
900821	6457	644	35	6167	698	295
900822	6487	722	42	6106	643	303
900824	6159	744	25	6097	607	260
Wght.mean	6348			6055		

APPENDIX 6
DAILY AVERAGE QUEUE DISCHARGE FLOW.

Date	Average flow rate	std. dev	Avg. spd. (km/h)
900425	6185	178	72
900426	5956	264	73
900430	5917	174	74
900501	5987	216	73
900502	5919	223	72
900503	6094	184	74
900504	6205	182	74
900508	6024	195	76
900509	6134	172	75
900511	6199	228	72
900514	6012	244	71
900515	6228	244	75
900518	6221	245	74
900522	5874	375	76
900524	6307	244	74
900525	6166	184	72
900528	5970	160	72
900530	5896	168	70
900601	5976	313	77
900604	6031	305	73
900606	6060	179	70
900607	6182	233	70
900608	6121	187	74
900611	6051	213	72
900612	6165	250	74
900614	6114	166	71
900620	5949	174	75
900622	5854	190	76
900626	6109	220	73
900628	5989	170	75
900703	5979	201	74
900704	5955	199	76
900706	6153	228	76
900709	5889	200	76
900711	6063	187	76
900713	6092	208	77
900716	5983	277	73
900719	6053	254	74
900723	6210	187	75
900724	6058	334	76
900725	5989	190	76
900726	6099	207	76
900809	5994	247	75
900810	5952	244	75
900814	5964	287	79
900815	5951	334	76
900816	6203	245	76
900817	6139	250	77
900821	6163	301	79
900822	6109	242	77
900824	6099	259	76

APPENDIX 7
*PEAK 15-MINUTE FLOW RATE (QUEUE DISCHARGE)

Date	flow rate
900425	6336
900426	6420
900430	6076
900501	6240
900502	6216
900503	6228
900504	6380
900508	6192
900509	6348
900511	6592
900514	6256
900515	6656
900518	6500
900522	6008
900524	6516
900525	6516
900528	6160
900530	6256
900601	6324
900604	6308
900606	6232
900607	6340
900608	6244
900611	6296
900612	6480
900614	6256
900620	6044
900622	6116
900626	6228
900627	6080
900628	6144
900703	6088
900704	6160
900706	6372
900709	6060
900711	6328
900713	6328
900716	6280
900719	6376
900723	6316
900724	6456
900725	6176
900726	6304
900809	6256
900810	6076
900814	6216
900815	6332
900816	6476
900817	6500
900821	6328
900822	6376
900824	6292

* Hourly rate for maximum 15-minute volume