URBAN TRAVEL TIME MODELS: VANCOUVER (BC) CASE STUDY

by

CHRISTOPHER K. NUTAKOR

B.Sc. The University of Science and Technology, Ghana, 1989

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE STUDIES

Department of Civil Engineering

We accept this thesis as conforming

to the required standard

....

....

THE UNIVERSITY OF BRITISH COLUMBIA

October, 1992

In presenting this thesis in partial fulfilment of the requirements for an advanced degree at the University of British Columbia, I agree that the Library shall make it freely available for reference and study. I further agree that permission for scholarly purposes may be granted by the head of my department or by his or her representatives. It is understood that copying or publication of this thesis for financial gain shall not be allowed without my written permission.

Department of Civil Engineering

The University of British Columbia

Vancouver, Canada

Date October, 1992

ABSTRACT

Travel time survey data obtained from the City of Vancouver's Engineering Department have revealed that, travel times in the City of Vancouver have remained fairly constant over the past three decades, although traffic volumes and the number of traffic control devices have increased. This pattern of travel times is contrary to the predicted travel time behaviour of traditional travel time models. The traditional travel time models generally predict travel time as increasing with increasing traffic volume. This thesis investigated the reasons for the observed travel time trends. It also investigated the validity of three traditional time models, using data collected on a few arterial streets in the City of Vancouver. The results from the research indicated that, the observed travel time trends are principally due to increases in vehicle speeds and increases in the capacity of the street network.

The three traditional travel time models which were investigated for validity are the BPR model, the GVRD model and the Davidson model. None of the three models investigated provided a good fit for data collected on the arterial streets. Based on the data collected on one of the arterials, revised forms of the models were developed. The revised models were validated against data collected on three other arterials in the City of Vancouver. In all cases, the validation process proved satisfactory.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ACKNOWLEDGEMENTS

The author wishes to express his heartfelt thanks to Professor Frank Navin for his supervision and careful guidance throughout the duration of this thesis. Thanks are also due to Professor Gerry Brown for his constructive criticisms and Professor W.F. Caselton for reading through this thesis.

Finally, the author would like to express his gratitude to his fellow graduate students; Paul Deleur, Tarek Sayed, Mohsen Ghazel and Norman Stang for their encouragement and contributions.

God bless you all.

Chapter 1

INTRODUCTION

1.1 Defining the problem

Travel time, is a central factor for planning the transportation system of any municipality. It is defined as: The length of time it takes to move from a given origin to a given destination under prevailing roadway traffic and control conditions at some suitable time. The efficient management of any transportation system needs reliable travel time information. Ideally, the information would have to be continous, however, it is not possible to obtain travel time information on a continual basis by field measurements because of the high cost. As a result, analysts have developed procedures for predicting travel time from a few traffic parameters. These travel time estimating procedures have evolved from simple mathematical equations to complex simulation models. Nevertheless, researchers have had difficulty calculating consistent values for travel time under varying road conditions, due to the fact that no single satisfactory theory of urban traffic flow is available that relates the many known factors.

Travel time surveys in the City of Vancouver over the past three decades have revealed that, travel times on most arterial streets, as well as on some streets in the central business district (CBD) have remained fairly constant or shown slight improvements, although traffic volumes on the streets and the number of traffic control devices have increased. These travel time trends are contrary to the predicted travel time behaviour of traditional travel time models. The traditional models generally predict travel time as increasing with increasing traffic volume. These state of affairs have raised two questions:

(i) Why have the travel times in the City of Vancouver remained constant?

(ii) Are the old travel time models still valid or do they need to be modified?

The objective of this research is to find answers to these two questions. The research was conducted using traffic flow and travel time data collected on a few arterial streets in the City of Vancouver during the summer of 1992, and historic data available from the City of Vancouver Engineering Department.

1.2 Uses of travel time information

Travel time is probably the single most important variable in Transportation Engineering and Planning. Its uses can be summarised as follows:

(i)Transportation efficiency: Travel time data can be used to develop sufficiency ratings, congestion indices or other measures of route efficiency for use in programming traffic improvements.

(ii) Effects of improvements: Travel time data is useful in evaluating the effectiveness of specific traffic improvements on a " before and after" basis, such as the effect of parking prohibitions, traffic signal improvements, one-way streets, or turn prohibitions.

(iii) Traffic congestion and delay: Average speeds and the amount, location, duration, frequency and causes of delay in traffic stream help define problem locations where design or operational improvements may increase mobility.

(iv) Trends in mobility: The data can be used to evaluate the conditions of traffic mobility and the level of service as it changes over a period of time, due to transportation improvements and/or traffic growth.

(v) Transportation models: Trip assignment to proposed new traffic or transit facilities are based on relative travel times as well as other factors to predict mode split. It is therefore very useful as an input into traffic assignment algorithms and also for traffic signal control coordination.

(vi) Economic analyses: Travel time data can be used in cost/benefit analysis, such as transit scheduling, estimating gasoline consumption and as a very useful guide for the location of industries.

1.3 Factors affecting travel time

Travel times of vehicles in cities are influenced by many factors, and as a result travel time from a given origin to a given destination cannot be a single fixed quantity. The travel time varies depending on a number of prevailing conditions and influencing factors such as: traffic volumes, traffic composition, traffic control devices, times of day, weather and other factors.

(i)Traffic volumes: All other factors being equal, travel times increase with increasing flow. As such travel times of vehicles during peak hours are expected to be higher than during non peak hours.

(ii)Traffic composition: Traffic composition can significantly affect travel times of vehicles. Average travel times over a section of a route are likely to be high when the traffic composition has a high percentage of heavy vehicles. This is due to the fact that heavy vehicles are generally slow and also occupy a larger road space than all other vehicle types. As a result they reduce the capacity of the streets and may increase travel times.

(iii)Traffic Control devices: The presence of traffic control devices such as traffic signals, stop signs and yield signs delay vehicles and therefore increase the total travel time of a vehicle from a given origin to a given destination.

(iv)Effect of time of day: There are several reasons why travel times may depend on the time of day. These reasons include: altered network characteristics due to bus lanes or parking restrictions which may apply for only part of the day, changes in signal linking plans, changes in the proportion of turning movements at junctions, changes in pedestrian activity affecting the probability of being stopped at protected crossing facilities and also volume change effects.

(iv)Effect of weather: Factors such as heavy rain and snow have adverse effects on visibility of drivers and general road surface condition, and therefore effect on travel time.

(v)Other factors: Other factors such as road geometry, driving styles of drivers, vehicle characteristics, age and sex of drivers also affect travel time.

1.4 Measurement of travel time

Several techniques have been developed for measuring travel times. The choice of any particular measurement technique depends on the purpose of the study, the required degree of accuracy and the availability of equipments for the study. The techniques are divided into four categories as summarised below.

(i)Test Vehicle Method: In this method of obtaining travel time information, a series of runs are made through the section to obtain representative travel times. This could be achieved by adopting either one of two driving technique strategies; the 'floating car' technique or the 'average speed' technique. In the floating car technique, the driver attempts to approximate the median speed by passing as many vehicles as pass him. However, there are errors associated with this method, especially on multi-lane highways during periods of congested flow and on roads with very low volumes. The second 'driving strategy' is the average speed technique, in which the driver travels at a speed that in his own opinion is representative of the speed of the traffic at every point in time. Data is recorded by an observer in the vehicle or by a mechanical recorder. The use of an observer with two stop-watches is the most common method. The observer starts the first stop watch at the beginning of the test run and allows it to run continuously, recording the cumulative lapsed time at successive control points and delay points along the route. The second stop watch is used to determine the length of individual time delays at each delay point. The time, location and cause of the delay is recorded on forms or by voice recording equipment. It is possible for a driver alone to obtain the desired information by using voice-recording equipment and a stop watch mounted on the dashboard of the vehicle thus eliminating the need for an observer. However, this procedure can be hazardous due to the driving task.

(ii) The license plate method: In the license plate method, travel time information can be obtained by stationing one or more observers at each entrance and exit of the study section to record the time and license number of each vehicle as it passes the observation point. The numbers are matched later and the travel time of each vehicle is determined. The equipment used in this technique consists of synchronized stopwatches and recording forms or voice recorders without audible time signals.

(iii) Direct observation and timing method: The direct observation and timing method of vehicles is only employed over short road sections which are such that the observer can see both the entrance and exit points.

(iv) An interview technique: This is useful when a large amount of information is needed with little expense for field observations. For example, employees of private establishments or municipal agencies are asked to record their travel time to and from work on a particular day.

Chapter 2

LITERATURE REVIEW

2.1 Travel time prediction

In general terms, travel time prediction can be defined as: An automatic computation of travel times from classical travel time parameters. The knowledge of travel times as discussed previously is a critical component of information in traffic control systems, not only for drivers but also for traffic control managers, as it provides them with information on operational characteristics of roadways. It also serves as an important tool for watching over the evolution of traffic quality with time. The need for travel time information and the high cost associated with travel time measurements has led to the development of procedures for predicting travel time. Travel time prediction procedures have been very useful in Traffic Engineering and can be broadly classified into two groups: (i) Prediction Models and (ii) Prediction Algorithms. Some of these models and algorithms are applicable to traffic guidance systems and also useful for traffic control strategy in direct measurement and control of travel time on intersecting roads. This strategy enables traffic control managers to realize intended travel time policies. Most of them have also been used as inputs into traffic assignment algorithms used in urban transportation planning.

2.2 Travel time algorithms

2.2.1 Travel time prediction using vehicle sensor data

This travel time prediction method which utilizes vehicle sensor data was developed by T. Oda (1990). The method involves the division of the objective road section into several subsections to collect vehicle sensor data. The vehicle sensors are set at each subsection of the objective road section. Data collected through the vehicle sensors are traffic volumes, occupancy times and mean vehicle lengths. Using the vehicle sensor information obtained in each subsection, the subsection traffic conditions are determined and then future traffic conditions are predicted based on the traffic volume and occupancy time recorded from the past to the present. The method of prediction employed is an auto-regressive model with lead time. In the travel time prediction process, the mean length of the vehicles is computed with changing traffic conditions; in otherwords it is not assumed to be constant. This is an improvement on the conventional method which assumes that the vehicle length is constant despite changes in traffic conditions. This new method was applied to a road section in Chiba Perfecture, Japan, and the results showed that the new method is 5.6% more accurate than the conventional method. The mean travel time on each subsection is calculated as the ratio of the length of the subsection to the mean speed over the section. The mean speed is calculated as shown by Equation 2.1.

$$S = LV/C \tag{2.1}$$

where:

S = mean vehicle speed in meters per second

L = mean vehicle length in meter per vehicle

C = occupancy in seconds per minute

V = traffic flow rate vehicles per minute

The total travel time over the objective route is then obtained by totalling the travel times calculated over the subsections.

2.2.2 Travel time prediction for route guidance

The travel time prediction algorithm developed by G. Hoffman and J. Janko (1988), is a basic part of Berlin's LISB guidance and information system which is an individual dynamic guidance system for motorised vehicles. Although in a dynamic system routes are recommended at regular intervals, it is not sufficient to base quickest routes on travel times just at the moment of recommendation. This is due to the fact that the links that a vehicle will pass on its journey may change their travel times between the time of recommendation and the time of passing. The recommendation therefore has to be based on the travel time a vehicle has to expect at the moment it is travelling along this link, hence the development of the prediction algorithm. The travel time prediction process employed in the LISB Guidance strategy is in four steps: evaluation and validation of measured travel times, development of standard profiles, continuation of standard profiles and travel time prediction algorithm. Each of the steps is as described in the following discussion:

(i) Evaluation and validation of measured travel times

In the prediction process, the travel time values gathered on a link, with their time of entry into the link during the same time interval, are used to build an arithmetical mean. However, these values cannot be used without being checked, since some of these values may not be reasonable. For example very low travel times which are caused by exceeded speed limits are increased to a threshold. Also high travel time values which are considered to be caused by the driver's action are excluded from the analysis, while those considered not to be caused by the driver's action are included. For example high travel times due to exceptionally high volumes, weather and road surface conditions are considered.

(ii) Development of standard profiles

The second step in the prediction process is the development of long-term standard profiles which form the basis of the whole prediction process. Each link in the road network, the time of day divided into short time intervals (usually 5mins) and the weekday are the usual characteristics of differentiation for developing the travel time standard profiles. Information regarding weather or road surface conditions are only taken into account to explain strong variations in different profiles. Therefore, an analysis should be made to determine if variations in road surface conditions such as dry, wet or icy have significant influences on the travel time profiles. Since knowledge of travel time profiles will not be known at the beginning for each link and for each day of the week an arbitrary standard profile based on the average speed on the links is established for the different time periods during the weekdays and week-ends.

(iii) Continuation of standard profiles

Starting with a first provision of different levels of constant travel times for the different links of the network, a learning system has been brought into operation. In order to reduce the influence of stochastic variations an exponential smoothing algorithm is used as:

$$T_{ln} = \alpha T_{on} + (1-\alpha) t_{ln} \qquad (2.2)$$

where:

T_{ln} = new value of standard profile of link l in time interval n.

t_{ln} =value of standard profile of preceding day of operation of link l in time interval

T_{on} = average travel time in link l in time interval n of the preceding day of operation

α = a weighting parameter which is chosen such that, on one hand short range effects as vacation time or road surface conditions during the winter weather should influence the new travel time standard profile and on the other hand, single disturbances that will not occur on the following day of operation should not influence the travel time standard profile significantly. A value of $\alpha = 0.25$ was used in the LISB experiment.

(iv)Travel time prediction

The estimation of the expected travel time on a downstream link is based on the following information:

(a) The updated travel time standard profile of each link for this particular day of operation, and

(b) The travel time data of this day for each link up to the last 5-minute-interval before the prediction is started.

As a first step; a parameter d_{ln} is defined as:

$$d_{ln} = t_m / t_{ln} \tag{2.3}$$

where:

t_m is average travel time value measured on link l in time interval n and t_{ln} as defined earlier is the corresponding value of the actual travel time standard profile.

As some of the prediction cycles will not own travel time values for each link some travel times will not reflect the real situation; the parameter d_{ln} defined above is then smoothed exponentially by a weighting factor β. A value of $\beta = 0.20$ was used in the LISB guidance experiment. The smoothening relation is given as:

$$d_{sln} = \beta_{dln} + (1-\beta)_{dsln-1} \tag{2.4}$$

If there are no more travel times for one or more time intervals n, then the value for d_{ln} is set to 1.0 so that d_{sln} is also smoothed to 1.0. Further, to compensate for changes in the neighbourhood of a particular link, a mean d_{mln} of the smoothed ratios d_{srn} of all adjacent links is computed as:

$$d_{mln} = (1/a)\sum_{r=1}^{a} d_{srn} \tag{2.5}$$

From the values of d_{sln} and $d_{mln},$ an indicator D_{ln} is calculated as:

$$D_{\mathrm{ln}} = 0.5(d_{sln} + d_{mln}) \tag{2.6}$$

The prediction equation is therefore finally given as:

$$T_{inp} = D_{\mathrm{ln}} \times t_{\mathrm{ln}} \tag{2.7}$$

where T_{inp} is the best estimator for the expected travel time.

2.3 Travel time models

2.3.1 Travel time prediction model on congested roads

A method for predicting travel times on congested roads has been developed by Usami *et. al.* (1983). The application of the method involves the division of the road section being considered into several subsections. The total travel time over the road section is then computed as the sum of the predicted travel times over the subsections. The travel time estimation procedure on each subsection involves, dividing the number of the vehicles present in the subsection, ***E*** by the traffic volume, ***Q(veh/s)***, where the number of vehicles, ***E***, is given as the ratio of the section length, ***L(m)***, to average space headway, ***H(m/veh)***. The equation for predicting the travel time, ***T(sec)*** for the subsection of the road is expressed as :

$$T=\sum_i \frac{L_i}{H_i}\frac{1}{Q_i} \qquad (2.8)$$

To avoid errors that may be introduced by the constant average space headway, assumed in the above formula, the equation was modified by letting the inverse of H_i, or the traffic density, ***K(veh/m),*** be a linear function of the traffic volume Q as follows:

$$K_i = K_m - aQ_i \qquad (2.9)$$

The prediction equation is therefore written as:

$$T = K_m \sum_i \frac{L_i}{Q_i} - a \sum_i L_i \qquad (2.10)$$

where:

K_m and a are constants having pre-assigned values

Q_i = traffic volume in each subsection

L_i = subsection queue length

2.3.2 The BPR travel time model

The Bureau of Public Roads (BPR)(1964), developed a travel time model which is usually employed as an input into capacity restraint traffic assignment algorithms. The capacity restraint method of traffic assignment is an iterative process. The most common method is to assign the trips on to the network and adjust the speed or travel time on the link after each assignment, according to some speed-volume relationship, to minimise the imbalance of volume on the link. One such speed-volume relationships commonly employed is the BPR model which is given as:

$$T_n = T_o[1 + 0.15(V/C)^4] \qquad (2.11)$$

where:

T_n = the link travel time at the assigned volume

T_o = base travel time at zero volume

C = the practical capacity of the link

V = volume of traffic on the link

It has been found that a reasonable balance in volume of trips on the links can be obtained after three or four assignments.

2.3.3 Davidson's travel time model

Davidson (1966), developed a travel time model which, like the BPR model is applicable as an input into traffic assignment algorithms. This model has a quasitheoretical base from queueing theory, under an hypothesis that a length of continuous road can be represented as a sequential queueing system. In its original form, the model was based on the idea that if the total travel time is considered to consist of service time ***(To)*** plus a delay, then for a delay in a queue with random arrivals and random service, the mean arrival time through the queuing system is given as:

$$T=T_o[1+q/(s-q)] \qquad (2.12)$$

Davidson argued that as traffic flow on a road was not truly a single continuous queueing situation, the above relation could be modified to include an adjustment parameter j to yield:

$$T = T_o[1 + jq/(s-q)] \qquad (2.13)$$

where:

T_o = the free mean link travel time

s = link capacity and

j = a delay parameter which may be assumed to be a function of link type and environment

T = the travel time on the link at the flow q

Davidson's relationship has been used in several cases. It has been used in travel time flow studies in Toronto and Brisbane as reported in Blunden(1971). It has also been used by Taylor (1977) to fit data collected on four lane arterial roads in Australia.

Typical parameter values for the Davidson's model as reported by Blunden (1971) are as reported in Table 2.1.

conditions (Road-type)	Mean free flow travel time T_0(min/mile)	Delay parameter *(j)*	sat.flow (veh/hr)
motorways	0.8-1.0	0-0.2	2000/lane
multi-lane hwys	1.5-2.0	0.4-0.6	1800/lane
feeder and collector roads	2.0-3.0	1.0-1.5	1800/total width

Table 2.1 Flow parameters for various road types

Source: Blunden (1971), Table 3.9

2.3.4 The GVRD travel time model

The Greater Vancouver Regional District (GVRD) has also developed a model, which is incorporated in the capacity restraint traffic assignment algorithm of the district. The model is basically an improvement upon the BPR model. The assignment algorithm was developed from EMME2; a transportation planning program developed in the University of Montreal.

The GVRD travel time model for arterial streets is generally given as:

$$T=T_0[1+0.6(V/CL^{1.05})^4] \tag{2.14}$$

where:

T = the link travel time at the assigned volume

To = base travel time at zero flow

V = volume of traffic on the link

C = practical capacity per lane

L = number of lanes

2.3.5 Haase's travel time model

Haase (1968), proposed a freeway travel time model which takes into account the arrival of a vehicle on- ramp queue and its departure from the off-ramp. This model is of the form given below:

$$T_i = n_i[1/q_1 - 1/q_0] + d/u + n_i[1/q_2 - 1/q_1] \qquad (2.15)$$

where:

T_i = total trip time for the ith car

n_i = the ith car to arrive on ramp queue

q_0 = the average arrival rate at the on-ramp queue

q_1 = the average departure rate from the off-ramp queue

u = effective steady-state velocity of the N cars

d = distance travelled on the freeway

Haase's model is based on the assumption that, the total trip time of vehicles is influenced by the presence of other vehicles using the facility. In the model shown in Equation 2.15, total trip time will be low, if the average departure rate of vehicles from the off-ramp queue exceeds the average arrival rate at the on-ramp queue and vice versa.

2.3.6 Smeed's travel time model

Smeed (1968), discussed a special situation in which drivers delay starting their trips, in order to minimise travel time. He has developed a model that includes the fraction of the central business area devoted to streets in predicting travel time. This model is expressed as :

$$T = t/2 + \frac{(7.409/10^6)A^{1/2}}{[1 - n/33tfA^{1/2}]^{1/3}} \qquad (2.16)$$

where:

T = average journey time measurement from the time the first vehicle enters the central business district (CBD)

t = period over which entries to the CBD are spread

n = area of CBD in square feet; and

f = fraction of CBD area devoted to streets

The development of Smeed's model, is based on the idea that traffic speed usually decreases as flow increases. It therefore assumes that, the travelling times of each member of a group of vehicles, using the same road system will in general be less, when they spread the period over which they begin their journey times than when they concentrate this period as closely as possible. The model also assumes that all the vehicles entering the central business district street travel at the same average speed and that the area of the central business district has effect on the vehicles speeds.

2.4 The usable models

For the purposes of this research, not all the models discussed above are usable. The amount of data required for working with some of the models and algorithms is enormous and expensive and thus beyond the scope of this study. The research is therefore limited to the study of three models namely; BPR model, Davidson's model and the GVRD model.

2.5 Travel time studies in the City of Vancouver

Vancouver is Canada's major seaport on the Pacific Ocean and an active participant in the rapidly developing Pacific Rim Region. The metropolitan area known as the 'Lower Mainland' consists of 13 municipalities which have an aggregate population of 1.5 million. The core of this region is the central area of the City of Vancouver with a population of 415,000. The City of Vancouver serves 1.4 million person trips per day or 40 percent of 3.5 million trips within the metropolitan Vancouver area. Included within these trips are auto, transit and walking trips. Through trips are not included in the total. In spite of a high level of transit usage, the automobile is the predominant means of travel.

For the effective planning of the transportation system of any municipality, there is the need for occasional travel time studies. The City of Vancouver has on the average conducted a travel time study once in every ten years. The results from these occassional travel time studies has been very useful especially in the following areas:

(i) For determination of general trends in mobility

(ii) To serve as a guide for the evaluation and design of transportation improvements.

The available travel time data for the city are for the years of 1961, 1963, 1977 and 1988.

2.5.1 Travel time survey procedure

For all the years that travel time studies have been conducted in Vancouver, the surveys were conducted in the spring (March/April) in order to relate directly to one another and also because traffic volumes are normally close to the annual average at that time of the year. In each year of study three main time periods were surveyed. These were the morning peak periods between 0700-0900 hours; mid-day 1300-1500 hours; and the evening peak periods between 1600 and 1800 hours. An additional early morning sample was taken between 0400-0600 hours, so that peak period travel times could be compared to the minimum or fixed delay conditions occuring in the very light traffic period at night.

The travel time measurement technique used in all the studies was the test vehicle method. As described earlier, the test car was driven over each route at a speed that in the opinion of the driver was representative of the average speed of the traffic stream at the time of each run. In addition to the driver a recorder with a stop watch rode in the test car. The recorder started the watch at the beginning of the test run and recorded the time at various points along the route. A minimum of three runs was obtained on each route in each time period except 0400-0600 hours which was sampled once.

The measurement of stopped time delay; the delay due to traffic signals and congestion at traffic signals entails stationary observation at the subject intersection. The causes of delay were categorized in the data collection process to assist in remedying problems. The principal categories of delay considered include; delay due to congestion at traffic signals, delay due to pedestrians and delay due to stop signs. The stopped delay on a link was taken as the delay at the downstream intersection of the link.

2.5.2 Comparison of travel time data for Vancouver

Table 2.2 shows travel time data for some streets in Vancouver outside the downtown peninsula during the p.m. peak hour over a period of time. The information from the table indicates that travel time over the years have generally remained fairly constant or have shown slight improvements over time, although traffic count data have shown that traffic volumes have been increasing over time. Similar pattern was exhibited for the a.m. peak period and the mid-day travel time data as well. Table 2.3 also shows the travel time data for some streets in the Central Business District of Vancouver during the p.m. peak period over a period of time. The travel time trend over the years shown in this table is similar to that of Table 2.2. The mid-day and the a.m. peak trends were also similar in this case.

Graphical comparison of the travel times over the years during the p.m. peak period is shown in Figure 2.1. This comparison is for the years of 1961, 1966,1977 and 1987-88. The comparison is made by drawing five-minute isochrones with the common origin at the intersection of Granville Street and Georgia Street. This comparison confirms the trends identified in Tables 2.2 and 2.3. That is travel times have been surprisingly consistent over the past three decades, although these were years of rapid expansion of the population and number of automobiles on the City's roads.

STREET	Distance (mile)	Year/Travel time				
		1988	1977	1968	1966	1960
		min.	min.	min.	min.	min.
BROADWAY						
Boundary - Alma	7.41	20.20	20.19	22.96	23.56	24.05
Alma - Boundary	7.41	23.65	24.69	25.64	23.59	25.26
MAIN						
Marine - Prior	4.57	11.72	12.74	13.24	12.94	12.72
Prior - Marine	4.57	16.82	15.78	13.38	16.04	13.22
OAK						
19th - 70th avenue	3.18	8.22	9.37	10.84	9.58	8.25
70th-19th avenue	3.18	7.83	7.85	8.99	8.06	7.28
41ST						
Marine - Kingsway	7.29	19.15	18.91	20.59	19.76	19.17
Kingsway - Marine	7.29	17.22	18.39	19.11	18.44	18.63

Table 2.2 Average travel time comparisons for some major streets in Vancouver outside the Downtown peninsula

Source: City of Vancouver's Engineering Department

STREET	Distance (mile)	Year/Travel time				
		1988	1977	1967	1965	1959
		min.	min.	min.	min.	min.
ABBOT						
Pender-Cordova	0.13	-	0.64	2.10	0.82	0.84
Cordova-Water	0.07	0.33	0.19	0.70	0.49	-
BURRARD						
Pacific-Hastings	0.98	5.43	5.35	3.73	4.12	4.53
Hastings-Pacific	0.98	4.67	4.70	4.58	4.37	4.46
DAVIE						
Thurlow-Richards	0.43	1.75	2.50	3.21	3.40	3.46
Richards-Thurlow	0.43	3.04	3.77	2.22	2.78	2.82
GEORGIA						
Cadero-Beatty	1.15	7.16	7.24	5.24	6.41	5.60
Homer-Cadero	0.97	6.12	6.23	7.87	7.26	5.84

Table 2.3 Average travel time comparison for some streets in Vancouver within the Downtown peninsula

Source: City of Vancouver's Engineering Department

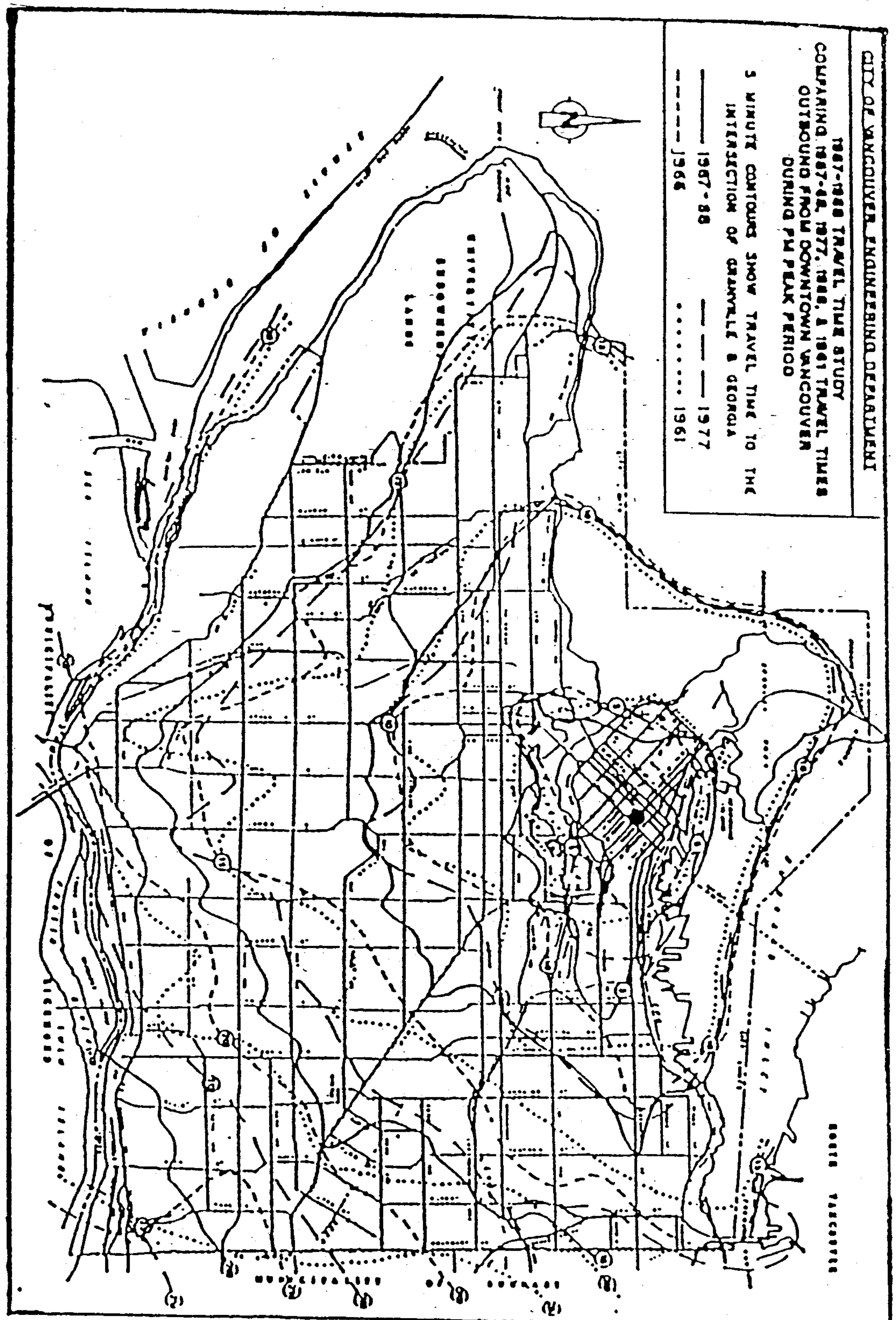

Figure 2.1 Source: City of Vancouver's Engineering Department

Chapter 3

RESEARCH METHODOLOGY

The research procedure was in three main sections. The first section involved testing the usable models. The second section involved the modification of the models, by fitting them with observed data and estimating the parameters that best fitted the observed data, and the third section was the validation of the revised models.

3.1 Testing the applicable models

The applicable models, namely the GVRD model, the BPR model and the Davidson model, were all tested using traffic flow and average travel time data collected on links of four arterial streets in the City of Vancouver. The links are 41st Avenue to 49th Avenue on Oak Street, 57th Avenue to 49th Avenue on Oak Street, Fraser Street to Clark Street on 12th Avenue and 16th Avenue to King Edward Avenue on Arbutus Street. The reason for choosing two links on Oak Street was to investigate whether data collected on links of the same street produce similar results with the models as compared to data from other streets.

3.1.1 Description of the data collection sites

The link between 49th Avenue and 41st Avenue on Oak Street has three driving lanes in the northbound direction with an exclusive left turn lane. Two of the three driving lanes serve through traffic and the third lane which is the curb lane serves right turning traffic and through traffic. For vehicles turning left however, an exclusive left turn lane is provided. Data were collected on this link in the northbound direction, during the morning peak period. Traffic volume counts were conducted at a section which is approximately 400 metres upstream of the intersection of 41st Avenue on Oak Street.

For the section between 57th Avenue and 49th Avenue on Oak Street, data was collected during the morning peak period in the northbound direction. The traffic volume counts were conducted at a section located at about 300 metres upstream of the intersection of 49th Avenue on Oak Street. This section consists of three driving lanes, two of which serve through traffic and the third one, which is the curb lane serves both through and right turning vehicles. An exclusive left turn lane is provided at the intersection of 49th Avenue on Oak Street in the northbound direction.

The data collection between Clark Street and Fraser Street on 12th Avenue was in the Eastbound direction during the evening peak period. Traffic volume observations were made at a distance of about 200 metres upstream of the intersection of Clark Street on 12th Avenue. This section also has two driving lanes in the Eastbound direction, one of which serves through traffic and the other serves through and right turning traffic. At the intersection of Clark on 12th Avenue an exclusive left turn lane is provided for vehicles approaching the intersection in the Eastbound direction and wishing to turn left.

The section between 16th Avenue and King Edward Avenue on Arbutus Street has two driving lanes in the northbound direction with an exclusive left turn lane. One of the two driving lanes, similar to the case on 12th Avenue, serves through traffic with the other serving both through and right turning traffic. The data collection was in the northbound direction during the morning peak period with traffic flows observed at about mid block locations.

Table 3.1 shows the signal timing plans that were in operation on the study links in the direction of the approach intersections, during the period data were collected.

Site	Direction	Time of day	Cycle time at approach intersection (seconds)	Green time at approach intersection (seconds)	% Green time at approach intersection
Oak St. 41st-49th	Northbound	Morning peak period	75	43	57
Oak St. 49th-57th	Northbound	Morning peak period	75	46	61
12th Ave. Clark-Fraser	Eastbound	Evening peak period	55	26	49
Arbutus St. 16th-King Edward	Northbound	Morning peak period	65	32	49

Table 3.1 Signal timing plans in operation at the various approach intersections

Source: City of Vancouver's Engineering Department (May, 1992)

3.1.2 Data collection procedure

Data required for testing the models were traffic flow and travel time data. Data were collected under clear weather conditions on all the four links described previously. No parked vehicles were observed along any of the four links during the period data were being colleced. The data collection methodology was the simultaneous observation of short period traffic counts and measurement of the average cruising speed of the traffic stream using the test vehicle technique. The driver of the test vehicle drove each time with the traffic stream at a speed which in his opinion was representative of the average speed of the traffic stream. Speed measurements which appeared to be affected by the downstream or the upstream traffic signals were ignored. These were speeds when the test vehicle was either accelerating or deccelerating. An observer riding in the test vehicle with the driver recorded the observed cruising speed of the test vehicle each time it plied the test section. Another observer conducted traffic volume counts of continuous traffic streams of which the test vehicle was part and recorded the time in seconds using a stop watch. At the end of each run through the test section, the recorder in the test vehicle reported the observed cruising speed and this was recorded against the measured traffic flow before the next run. The travel times were calculated using the speed data collected. Table 3.2 shows the data collected on the link between 41st Avenue and 49th Avenue on Oak Street. The predicted travel times corresponding to the measured flows are also shown in Table 3.3. The observed data and the corresponding predicted data for the other three links are included in Appendix A.

The capacity values and free flow speed values used in the prediction models are those currently being used by the GVRD for the links in traffic assignment. The capacity value used for all the links is 1067 pcph per lane and the free flow speed value used is 50 km/hr.

Traffic flow (veh/hr)	speed (km\hr)	Travel time (secs/km)	Traffic flow (veh/hr)	Speed (km/hr)	Travel time (secs/km)
480	72	50.0	978	67	53.7
780	65	55.4	1632	58	62.1
850	60	60.0	2184	49	73.5
1100	65	55.4	2640	42	85.7
1350	62	58.1	2520	34	105.0
1400	63	57.1	2040	50	72.0
1600	55	65.5	1800	55	65.5
1750	59	61.0	600	68	52.9
1850	55	65.1	1392	60	60.0
2100	47	76.6	2100	50	72.0
2200	51	70.6	1800	62	58.1
2250	48	75.0	640	64	56.3
2300	50	72.0	840	60	60.0
2350	49	73.5	2100	49	73.5
2400	47	76.6	2292	47	76.6
2450	44	81.8	2400	42	85.7
2500	42	85.7	1680	56	64.3
2550	41	87.8	2650	41	90.0
2600	40	90.0	2700	40	100.0

Table 3.2 Traffic flow and speed data, collected between 41st Avenue and 49th Avenue on Oak Street, with computed travel times (May 8, 1992)

Traffic Flow (veh/hr)	Predicted Travel Times (seconds/km)			Traffic Flow (veh/hr)	Predicted Travel Times (seconds/km)		
	GVRD	BPR	Davidson		GVRD	BPR	Davidson
480	72.1	72.0	78.4	2550	124.9	86.9	233.4
780	72.4	72.1	83.7	2640	129.1	88.1	252.0
850	72.6	72.2	85.1	2700	133.6	89.3	274.5
1100	73.7	72.5	91.0	978	73.0	72.3	87.6
1350	75.8	73.1	98.6	1632	80.0	74.2	109.4
1400	76.5	73.8	100.3	2184	97.2	79.1	48.2
1600	79.6	74.1	108.5	2640	128.3	87.8	248.0
1750	82.9	75.1	116.1	2520	118.7	85.1	209.5
1850	85.6	75.8	122.1	2040	92.1	77.6	136.4
2100	94.5	78.3	142.0	1800	84.2	75.4	119.0
2200	99.1	79.6	152.8	640	72.2	72.0	80.4
2250	101.7	80.3	159.1	1380	76.2	73.2	99.6
2350	107.3	81.9	173.9	1800	84.2	75.4	119.0
2400	110.4	82.8	182.8	600	72.2	72.0	80.4
2450	113.7	83.7	192.8	850	72.6	72.2	84.9
2500	117.3	84.7	204.4	2100	94.5	78.3	142.0
2550	121.0	85.8	217.7	2292	103.0	80.8	163.2
2400	110.4	82.8	182.8	1680	81.2	74.6	112.3

Table 3.3 Predicted travel times, using the old travel time models, with traffic flow data collected between 41st Avenue and 49th Avenue on Oak Street

3.1.4 Testing the effectiveness of the models and test results

To investigate the effectiveness of the models in predicting travel time, the observed and the predicted data for each of the links were graphically compared as shown in Figures 3.1, 3.2, 3.3 and 3.4. The graphs indicate that the predicted data have significant deviations from the observed data for all the locations considered. None of the graphs representing the predicted data are reasonably within the observed data (scatter diagrams). Instead, all the models overestimated the link travel times. An indication that, none of the prediction models fitted the data observed on any of the links.

To provide stronger evidence that the models poorly fitted the observed data, a hypothesis test was performed to test the equality of the means of the observed and the predicted travel times by the models for each link. The test procedure involved testing a null hypotheses, that the means of the observed and the predicted travel times are equal against an alternative hypotheses that they are not equal. The null hypotheses is given as:

$$H_o\text{: } \mu_1 = \mu_2 \qquad (3.1)$$

The alternative hypothesis is therefore given as:

$$H_o\text{: } \mu_1 \neq \mu_2 \qquad (3.2)$$

If X and Y are the means and S_X and S_Y are the standard deviations of the sampled observed and predicted travel times respectively, then a random variable can be defined as:

$$Z_1 = \frac{|X - Y|}{\sqrt{S_X^2/n_1 + S_Y^2/n_2}} \qquad (3.3)$$

n_1 and n_2 are the sample sizes of the observed and the predicted travel times respectively.

The Z_1 value computed from the above equation is compared with a value Z_2 read from standard tables at a defined significant level(α). If the values of Z_1 and Z_2 are such that Z_1 is greater than Z_2, the null hypothesis is rejected in favour of the alternative hypothesis; and if Z_1 is less than Z_2 the null hypothesis is accepted and the alternative hypothesis is rejected. The acceptance of the null hypothesis signifies that the model being tested is a good fit for the observed data and vice versa . The tests were conducted at three different significant levels of 2%, 5% and 10%. The lower the significance level of the test the more crude the test; in otherwords the greater the chance for a prediction model to be judged as a good fit for the observed data. The means and the variances of the observed and the predicted data are as summarised in Table 3.4, with the test results in Table 3.5. The results from Table 3.5 demonstrate that the null hypothesis be rejected for all the models even at a significance level as low as 2%. This provides a strong enough evidence that all the models being tested poorly fitted the observed data.

The next stage of the research was calibration of the model parameters to fit the observed data on all the links.

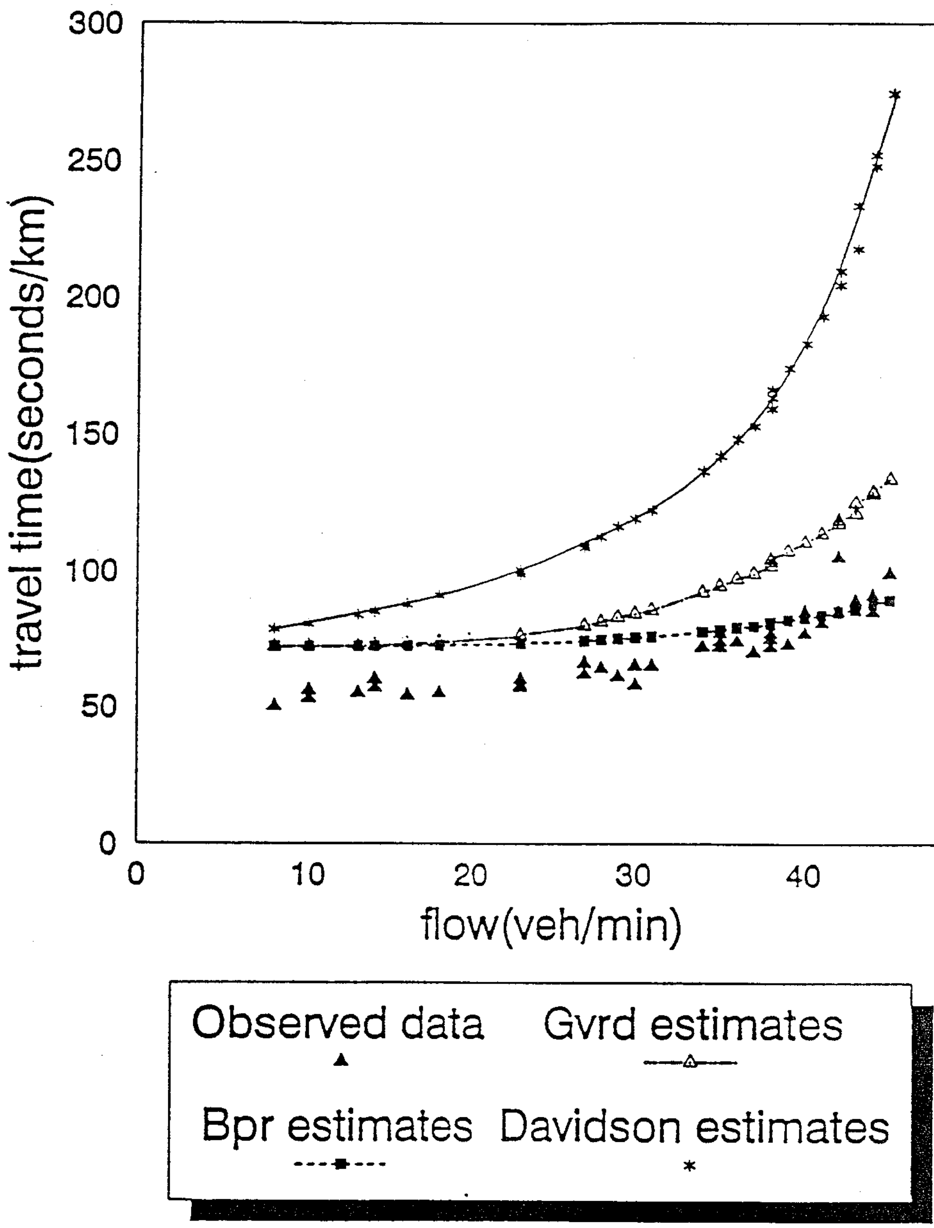

Figure 3.1 (Testing the models) Fitting the old prediction models with observed data, collected between 41st Avenue and 49th Avenue on Oak Street

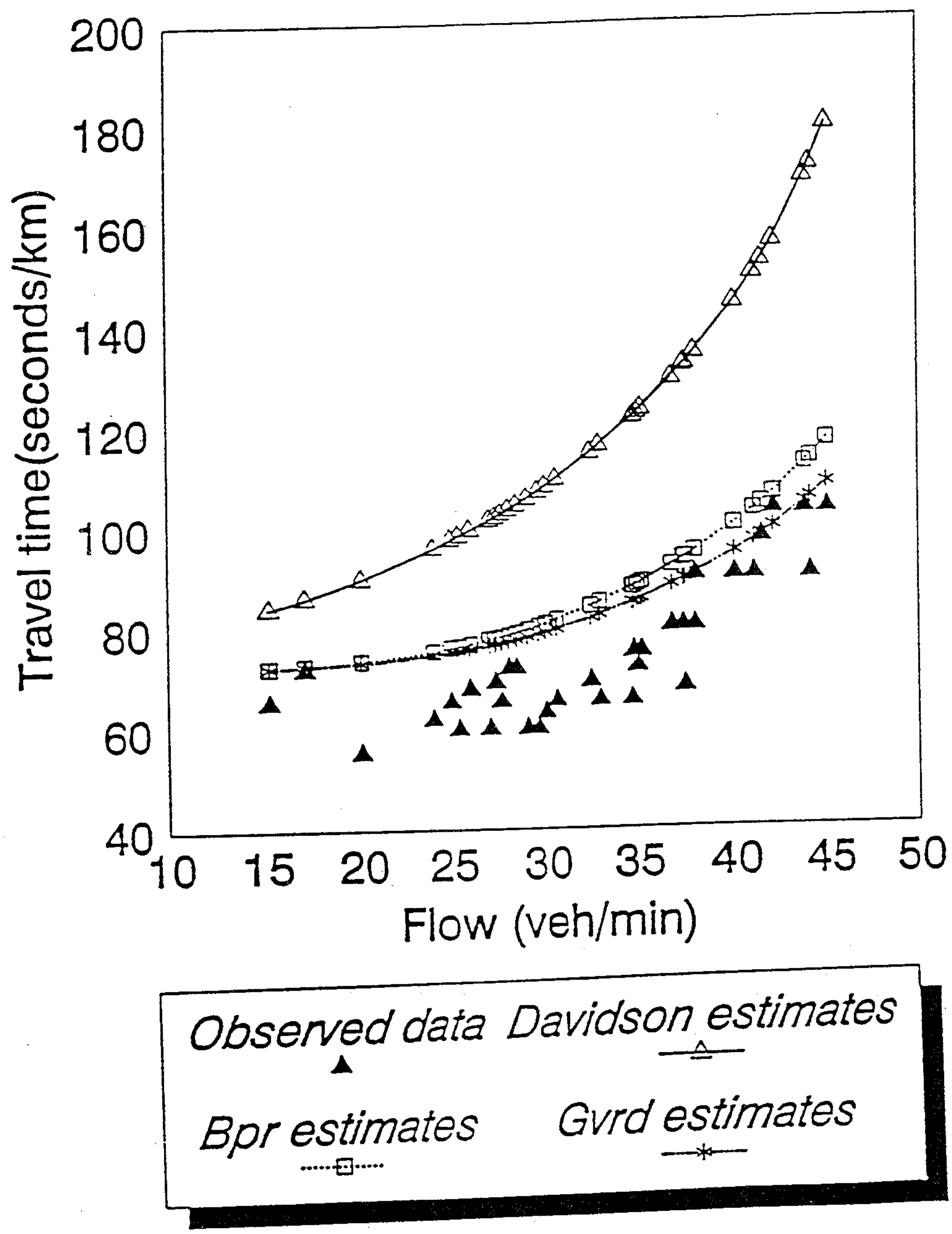

Figure 3.2 (Testing the models) Fitting the old prediction models with observed data, collected between 49th Avenue and 57th Avenue on Oak Street

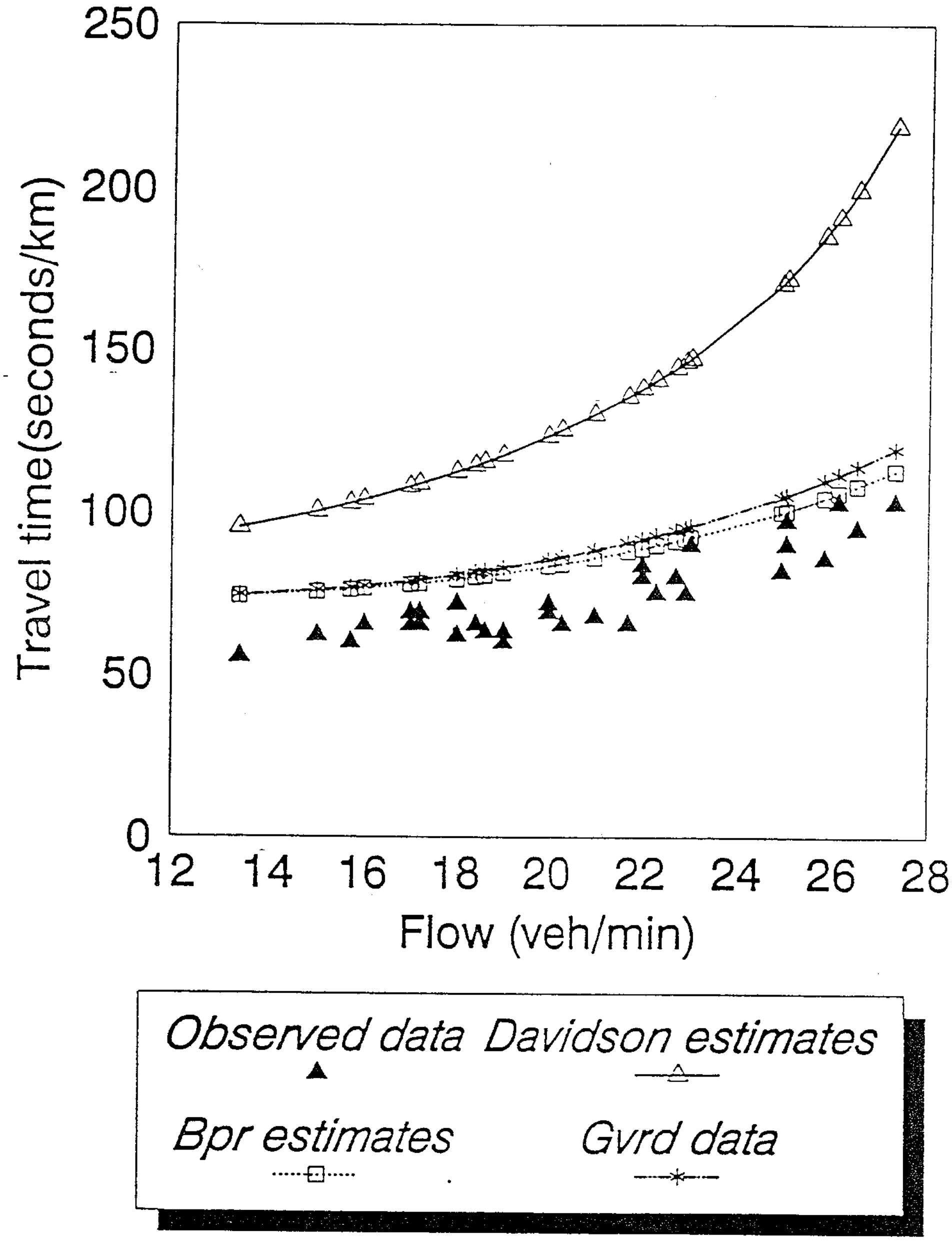

Figure 3.3 (Testing the models) Fitting the old prediction models with observed data, collected between Clark Street and Fraser Street on 12th Avenue

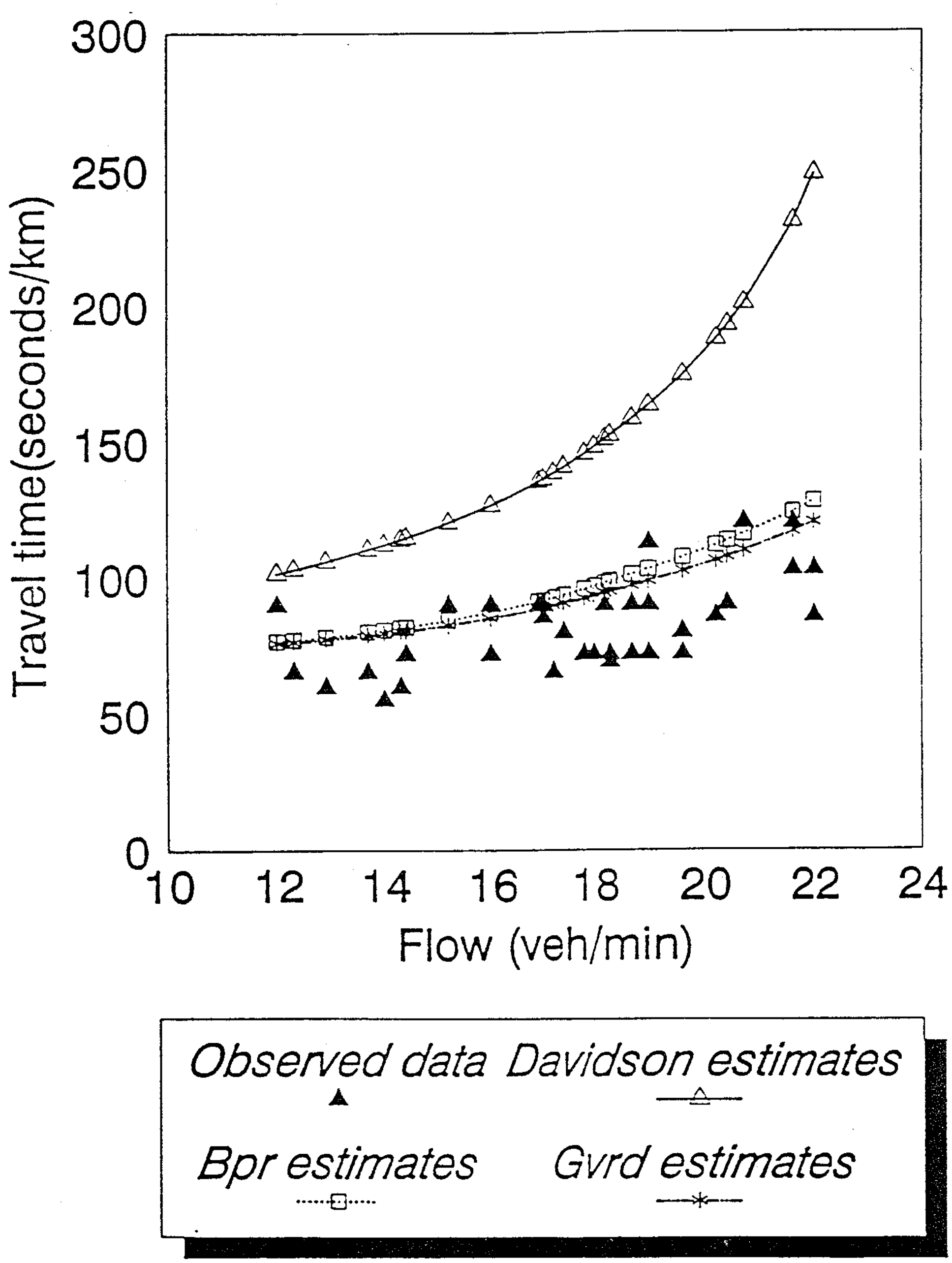

Figure 3.4 (Testing the models) Fitting the old prediction models with observed data, collected between 16th Avenue and King Edward Avenue on Arbutus Street.

Site	Parameter	Observed	GVRD Model	BPR Model	Davidson Model
Oak Street 41st-49th	Mean(s/km)	70.7	94.3	78.3	144.7
	Variance$(s/km)^2$	182.5	362.9	128.6	2876.2
	Sample size(n)	38	38	38	38
Oak Street 49th-57th	Mean(s/km)	74.5	84.7	87.7	120.5
	Variance$(s/km)^2$	180.4	96.9	149.1	601.1
	Sample size(n)	32	32	32	32
Arbutus St. 16th-King Edward	Mean(s/km)	82.6	94.7	98.4	155.1
	Variance$(s/km)^2$	252.1	151.1	204.5	1536.5
	Sample size(n)	35	35	35	35
12th Ave. Clark-Fraser	Mean(s/km)	69.9	87.5	77.4	135.2
	Variance$(s/km)^2$	166.4	109.2	176.9	986.8
	Sample size(n)	32	32	32	32

Table 3.4 Values of the means, variances and sample sizes for the observed and predicted data with the old models

Models	Sites/Z_1 values				Z_2 at α=10%	Z_2 at α=5%	Z_2 at α =2%
	12th Ave. Clark-Fraser	**Oak St.** 41st-49th	**Oak St.** 49th-57th	**Arbutus St.** 16th-King Edward			
Davidson	10.88	8.26	9.31	10.14	1.645	1.960	2.326
GVRD	6.00	6.24	3.48	3.60	1.645	1.960	2.326
BPR	3.29	3.22	4.14	4.36	1.645	1.960	2.326

Table 3.5 Results from testing the old travel time models

3.2 Modification of the models

The three models being investigated can be rewritten as shown below.

The GVRD model as:

$$T = T_o[1 + z(V/CL^{x})^{y}] \qquad (3.4)$$

the BPR model as:

$$T = T_o[1 + \alpha(V/C)^{\beta}] \qquad (3.5)$$

and the Davidson model as:

$$T = T_o[1 + jq/(s - q)] \tag{3.6}$$

The modification of the models involved the estimation of the parameters $\boldsymbol{n}$, $\boldsymbol{x}$, $\boldsymbol{y}$, α, $\boldsymbol{\beta}$ and $\boldsymbol{j}$ which fitted the field data. The values of capacity and free flow travel time were directly measured in the field.

3.2.1 Determination of Capacity values

According to the Highway Capacity Manual, capacity is defined as "the 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."

The capacity of arterial streets like the cases being considered are significantly influenced by factors such as: the arterial environment, the interaction between vehicles and the effect of traffic signals. The arterial environment includes geometric characteristics of the facility and adjacent land uses; the number of lanes and lane width, type of median, access point density and spacing between signalised intersections, existence of parking, level of pedestrian activity and speed limit. The presence of traffic signals also considerably reduce the capacity of arterial streets. The reason being that the traffic signals force the vehicles to stop and to remain stopped for a certain time and then release the vehicles in platoons. Thus the delays and speed changes result in reduction of capacities.

The determination of the capacity of links of arterial streets are typically based on the use of an ideal maximum traffic flow rate that is adjusted to reflect site- specific conditions that may not be ideal. Regardless of the specific procedure used in arterial capacity analyses; saturation flow is used as the base flow rate. Accurate estimation of arterial capacities is therefore based on accurate estimation of saturation flows.

The highway capacity manual gave an estimate of saturation flow rate under ideal conditions as 1800 passenger cars per hour green per lane (pcphgpl) which can be adjusted for site specific conditions. The capacity of an arterial link is then given as:

$$C_i = S_i \times (g/C)_i \qquad (3.7)$$

where:

C_i = capacity of lane group or approach i, in vph

S_i = adjusted saturation flow rate for lane group or approach i, in vphg; and

$(g/C)_i$ = green ratio for lane group or approach i

Measurement of saturation flow rates by various researchers and research groups have however demonstrated that, saturation flow rates vary widely depending on some other local conditions. These conditions include weather conditions and unusual traffic mixes which are not considered in the Highway Capacity Manual methodology. In otherwords, the ideal value of 1800 pcphgpl might not be applicable to some areas. This is evidenced by some saturation flow studies conducted under non ideal conditions which gave values higher than the ideal value of 1800 pcphgl. For example, Webster *et. al.* (1966), reported a value of 1850 pcphgl for an average site with an effective approach width of 10 feet. Miller (1969), also reported a value of 1810 pcphgl for the same approach width. All these approach widths are less than the ideal width of 12 feet. Studies by research groups such as the City of Edmonton in conjunction with the University of Alberta, the University of Kentucky, and the Australian Road Research Board confirmed the high degree of variability of saturation flows. Thus for this studies, it was deemed appropriate to determine saturation flow rates using local data. The study procedure is described in the next section.

3.2.1.1 Measurement of Saturation flow

The method of saturation flow measurement adopted was the one recommended by the Highway Capacity Manual. The studies were undertaken by two people; one being assigned as the timer with a stop watch and the other observer. The crosswalk at the intersections was chosen as the reference point for the studies. The studies were conducted at the approach intersections of the four links during the same time periods that flow and speed data were collected on the links, but on different days. The timer started the stop watch at the beginning of the green light and the times that the fourth and the last vehicle in the queue crossed the stop line while the traffic signal was still green were recorded. This procedure was repeated for five different cycles. The average headway was then calculated from the collected data hence the saturation flow rate. The saturation flow was measured for the through lane and the curb lane which is shared by through and right turning vehicles. The left turning vehicles were not considered; since an exclusive left turning lane is provided at each of the intersections, as such the left turning vehicles did not influence the results of the studies. Tables 3.6 and 3.7 show detailed results of the studies at the intersection of 41st Avenue and Oak Street. The saturation flow rate for the through lane is calculated as :

$$\frac{3600}{1.609} = 2237pcphgl \tag{3.8}$$

For the shared lane the saturation flow rate is given as:

$$\frac{3600}{2.275} = 1582pcphgl \tag{3.9}$$

For the two through lanes and the one shared lane at this study location, with the proportion of green during the morning peak hours as 0.57, the capacity of the link or the approach lanes is calculated as :

$$(2\times2237+1582)\times0.57=3450\mathit{pcuph}\ \mathit{per}\ 3\mathit{lanes}=1150\mathit{pcphpl} \qquad (3.10)$$

The saturation flows and capacity values for the other three links are summarised in Appendix A, Table A.10

Cycles	Number of vehicles in queue (C)	Time of crossing reference point (seconds)		Headway per vehicle (A-B)/C (seconds)
		Fourth vehicle (B)	last vehicle (A)	
1	12	8.05	21.76	1.713
2	10	7.97	16.42	1.400
3	14	7.74	23.76	1.602
4	10	7.5	17.82	1.720
5	13	8.0	22.49	1.610
Mean headway = 1.609s				

Table 3.6 Results of saturation flow studies at the intersection of 41st Avenue on Oak Street for through lanes in the northbound direction during the morning peak period (May 13, 1992)

Cycles	Number of vehicles in queue (C)	Time of crossing reference point (seconds)		Headway per vehicle (A-B)/C (seconds)
		Fourth vehicle (B)	last vehicle (A)	
1	9	10.02	27.22	2.44
2	11	8.56	26.90	2.62
3	12	9.64	27.48	2.23
4	10	10.76	23.90	2.19
5	12	10.33	29.21	2.36
Mean headway = 2.275s				

Table 3.7 Results of saturation flow studies for shared lane(right and through) at the intersection of 41st Avenue on Oak Street in the northbound direction during the morning peak period (May 13, 1992)

3.2.2 Determination of free flow travel time

In testing the models the speed value used in calculating the free flow travel time was 50 km/hr, which is the value currently being used by the GVRD in traffic assignment for the test section. However, this value is the posted speed limit and as such is inappropriate for use in determing free flow travel time. The imposition of speed limit on streets is for reasons of safety and does not represent free flow speed of drivers. An accurate determination of free flow speed is therefore neccesary.

Free flow speed is defined as: " the speed adopted by drivers on a segment of road when influenced by local alignment but uninfluenced by other traffic."(Navin 1991). As the definition suggests, free flow speeds could be observed at late night or early morning before peak period conditions when traffic volumes are light. A representative study was therefore conducted on Oak street to determine the average free flow speed.

3.2.2.1 Measurement of free flow speed

The method adopted for the free flow speed measurement was the moving observer method using a test vehicle. The procedure involved following a lead vehicle with the test vehicle at late night, after 11 p.m. At this time of the night, the traffic volumes are significantly lower than during the day and the average time headway between vehicles could be as high as 30 minutes. The test vehicle followed the lead vehicle and simulated the speed of the lead vehicle by maintaining a fairly constant headway between it and the lead vehicle. The speedometer readings of the test vehicle while it was at cruising speed were recorded by an observer in the test vehicle. Speed values affected by traffic signals or by the presence of other vehicles were not recorded. The procedure was repeated with a number of vehicles for three nights and the results are as tabulated in Table 3.8. Speeds of some few vehicles considered to be travelling at unreasonably high speeds were ignored. The mean of the recorded speeds calculated as 62.2km/hr was adopted as the average free flow speed.

Lead vehicle	Free flow speed(km/hr)	Lead vehicle	Free flow speed(km/hr)
1	55	11	68
2	52	12	75
3	65	13	70
4	60	14	70
5	62	15	55
6	68	16	57
7	72	17	60
8	65	18	60
9	50	19	65
10	55	20	60
Average free flow speed = 62.2km/hr Standard deviation = 6.63km/hr			

Table 3.8 Results of free flow travel time studies on Oak Street in the northbound direction after 11 p.m. in May, 1992. Posted speed limit = 50km/hr

3.2.3 Estimation of the model parameters

The parameter estimation procedure employed to determine the model parameters for the three models under review is the method of least squares. This method involves estimating the values of the parameters which minimize the sum of the squares of differences between the observed and predicted travel times based on the observed flows.

3.2.3.1 Estimation of the parameters for the BPR model

The BPR model (Equation 3.5) could be linearised as shown below.

$$\ln\left(\frac{T}{T_o}-1\right) = \beta \ln V/C + \ln\alpha \qquad (3.11)$$

Using data collected on the link between 41st Avenue and 49th Avenue on Oak Street, the average free flow travel time ***(T_0)*** and the capacity value ***(C)*** determined for that link, a linear regression analysis of ***$\ln(T/T_0 - 1)$*** on ***$\ln V/C$*** was performed. The linear regression analysis which is based on the method of least square estimation produced the value of β equal to 4.03 and the value of $\ln\alpha$ equal to -0.65; from which α was determined as 0.52. The coefficient of regression, (R^2) was found to be 0.86, which is large enough for the estimated parameter values to be accepted as reasonable.

With the new values of α and β, a revised BPR model is derived as:

$$T = T_o[1 + 0.52(V/C)^{4.03}] \qquad (3.12)$$

3.2.3.2 Estimation of the parameter for the Davidson's model

The Davidson's model (Equation 3.6) could also be rewritten as :

$$\left(\frac{T}{T_o} - 1\right) = j\left(\frac{q}{s-q}\right) \qquad (3.13)$$

Using the same field data as for the BPR model, a linear regression analysis of ***(T/T*** $_0$ ***-1)*** on ***q/(s-q)*** produced the value of ***j*** equal to 0.22 with an R^2 value of 0.89.

With the new value of ***j***, the Davidson's model can be written in a new form as shown in Equation (3.14).

$$T = T_o[1 + 0.22(q/s-q)] \qquad (3.14)$$

3.2.3.3 Estimation of the parameters for the GVRD model

Like the two previous models, the parameters for the GVRD model were also estimated based on the method of least squares, but by solving a system of equations simultaneously. The GVRD model (Equation 3.4) like the BPR model can be written as:

$$\ln\left(\frac{T_i}{T_o} - 1\right) = y\ln V_i + \ln z - y\ln CL^x \qquad (3.15)$$

Also for a set of travel times T_i computed from observed flows and by putting $(T_i/To\text{-}1)$ as W_i, the sum of the squares of differences (U) between observed and predicted travel times can be written as:

$$U = \sum_{i}^{n} [W_{iV_i} - \frac{zV_i^{2y}}{(CL^x)^y}] \qquad (3.16)$$

For those parameters that minimize the value of U, the following partial derivatives of U with respect to the parameters hold.

$$\frac{\delta U}{\delta z} = \sum_{i}^{n} [W_i V_i - \frac{zV_i^{2y}}{(CL^x)^y}] = 0 \qquad (3.17)$$

$$\frac{\delta U}{\delta y} = \sum_{i}^{n} W_i V_i^y lnV_i - lnCL^x \sum_{i}^{n} W_i V_i^y - \frac{z}{(CL^x)^y} [\sum_{i}^{n} V_i^{2y} lnV_i - lnCL^x \sum_{i}^{n} V_i^{2y}] = 0 \quad (3.18)$$

The value of y was obtained by performing a linear regression analysis of $ln(T_i/T_0\text{-}1)$ on $lnVi$ using the same data used for the earlier models. The value of y was found to be 4.03 with the R^2 value as 0.86. The constant term expression from Equation 3.9 was also found to be -16.2 and therefore the constant term can be written as:

$$lnZ - ylnCL^x = -16.2 \qquad (3.19)$$

With the value of y known, Equations 3.18 and 3.19 were solved simultaneously for the remaining parameters. The estimates of x and z were found to be 1.12 and 0.64 respectively. The GVRD model can therefore be written in a new form as:

$$T = T_o[1 + 0.64(V/CL^{1.12})^{4.03}] \qquad (3.20)$$

3.3 Validation of the revised models

Validation is defined as "the extent to which any measuring instrument measures what it is intended to measure" Carmines and Zeller (1979). Validation of any measuring instrument is therefore very important in order to ensure that it sufficiently represents what it purports to measure.

There are several procedures for validating measuring instruments and the choice of any particular procedure or procedures depends on the measuring instrument being considered. Some of these validation methods are face validation, hypotheses validation and external validation. The method of validation considered applicable and adopted in this research is the external validation. External validation is achieved by comparing the measuring instrument which are the revised models in this case to other field data.

The three revised models shown in Equations 3.12, 3.14 and 3.20 were derived based on data collected on only the link between 41st Avenue and 49th Avenue on Oak Street. Therefore to verify their applicability to other arterial links, they were validated with data collected on the other three links which are, 57th Avenue to 49th Avenue on Oak Street, Fraser Street to Clark Street on 12th Avenue and 16th Avenue to King Edward Avenue on Arbutus Street.

3.3.1 The validation procedure and results

The basis for testing the validity of the revised models, was by the hypothesis test of the equality of the means of the predicted and the observed data as described under section 3.1.4. For the observed data from each of the sections, the corresponding predicted travel times (included in Appendix A) were computed using the various revised travel time models. The value of Z_1 was calculated for each case as in equation 3.1 and compared with values of Z_2 read from standard tables at different significant levels. The means and variances of the predicted data sets using the revised models are shown in Table 3.9. Also an attempt was made to fit the revised models with the observed data for each location. These are shown in Figures 3.5, 3.6 and 3.7.

The results from the validation of the models as revealed by the hypothesis test are shown in Table 3.10. The results from this test indicate that all the models proved valid at least at the 2% significant level of the test for all the sites investigated. For observed data from 12th Avenue between Fraser and Clark, the Davidson model fitted the data best at the 10% significant level of the test, with the GVRD model fitting the data least at the 2% significant level. The BPR model however provided the best fit for the observed data from Oak Street at the 10% significant level of the test, with the Davidson model being the least fitting this time and fitted the data at the 5% significant level. For the observed data on Arbutus Street, the BPR model was the best fitting model and fitted the observed data at the 10% significant level of the test, while the GVRD model fitted the data least but also at the 10% significant level. These patterns are better illustrated in Figures 3.5, 3.6 and 3.7.

The revised models could be used to estimate expected values and variances of travel times at particular periods on arterial links in the City of Vancouver. An example of the estimation procedure is included in Appendix B.

Site	Parameter	GVRD Model	BPR Model	Davidson Model
Oak Street 49th-57th	Mean(s/km)	68.90	70.68	75.19
	Variance$(s/km)^2$	71.57	96.77	75.52
	Sample size(n)	32	32	32
Arbutus St. 16th-King Edward	Mean(s/km)	77.50	80.11	87.47
	Variance$(s/km)^2$	111.50	143.51	193.03
	Sample size(n)	35	35	35
12th Ave. Clark-Fraser	Mean(s/km)	67.14	68.84	75.68
	Variance$(s/km)^2$	80.6	103.98	123.97
	Sample size(n)	32	32	32

Table 3.9 Values of the means, variances and sample sizes for the predicted data, with the revised models

Models	Sites/Z_1 values			Z_2 at α=10%	Z_2 at α= 5%	Z_2 at α= 2%
	12th Ave. Clark-Fraser	**Oak St.** 49th-57th	**Arbutus St.** 16th -King Edward			
Davidson	0.27	1.91	1.46	1.645	1.960	2.326
GVRD	2.04	1.00	1.47	1.645	1.960	2.326
BPR	1.32	0.37	0.66	1.645	1.960	2.326

Table 3.10 Results from validation of the revised models.

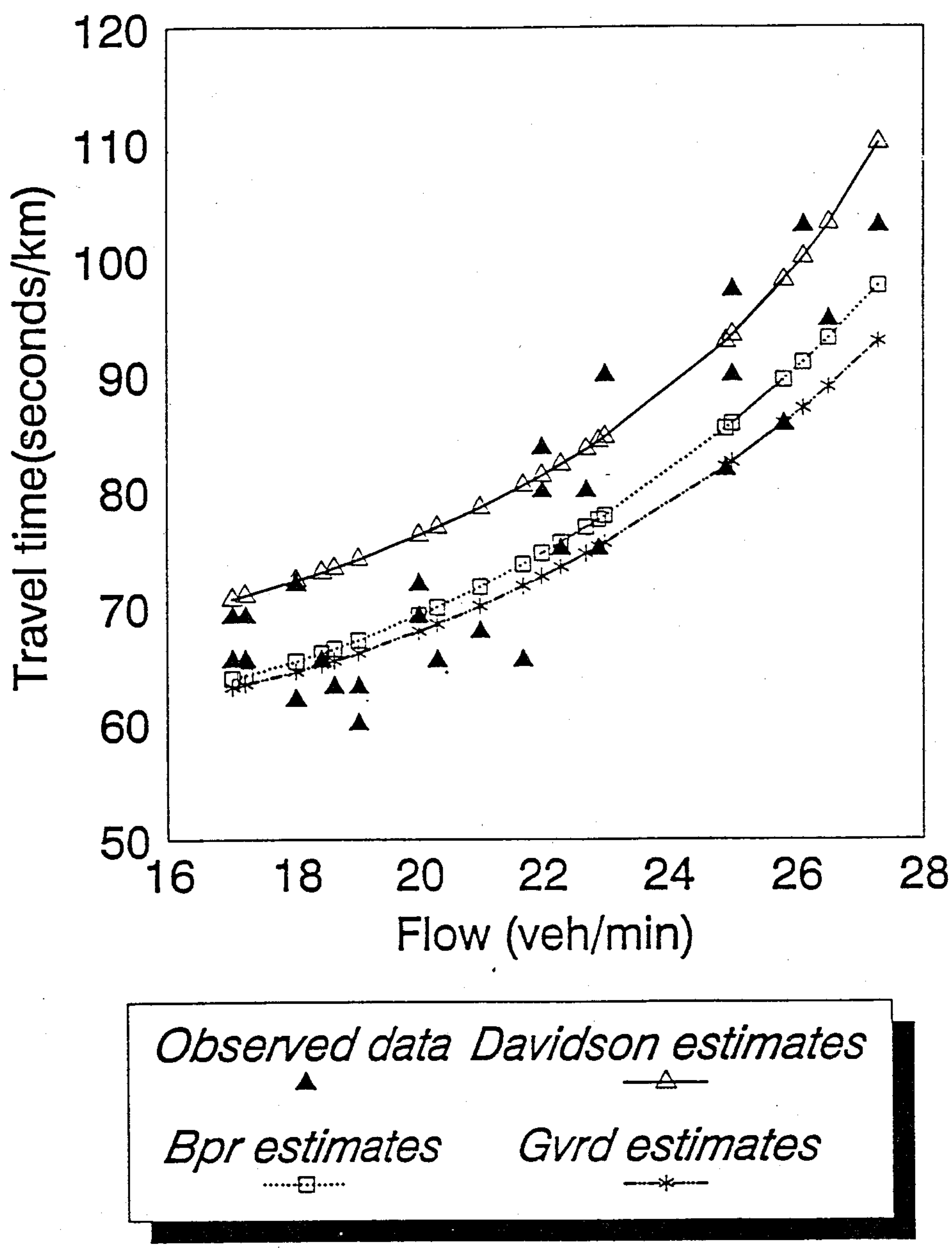

Figure 3.5 (Validating the revised models) Fitting the revised models with data collected on 12th Avenue, between Fraser Street and Clark Street

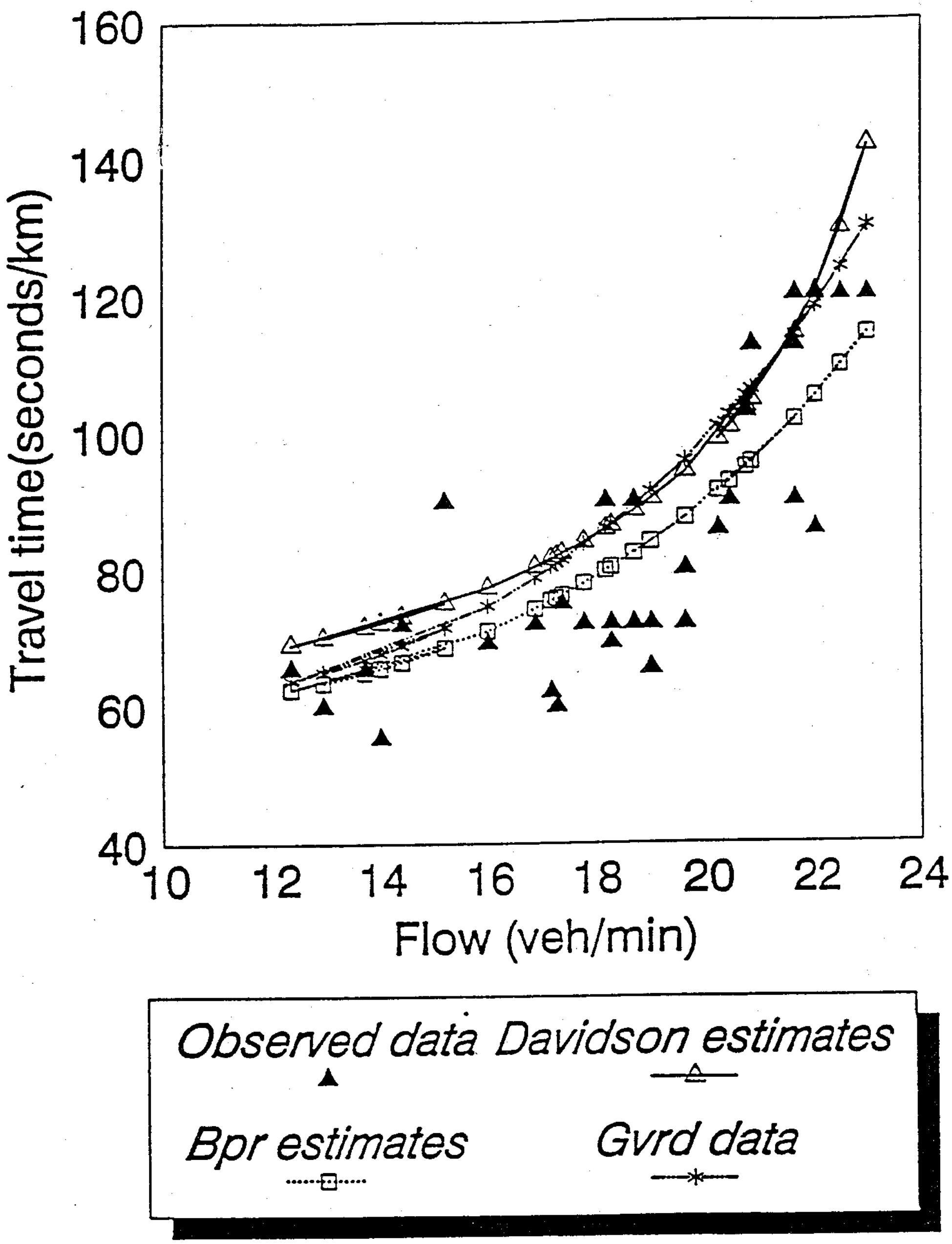

Figure 3.6 (Validating the revised models) Fitting the revised models with data collected on Arbutus Street, between King Edward Avenue and 16th Avenue

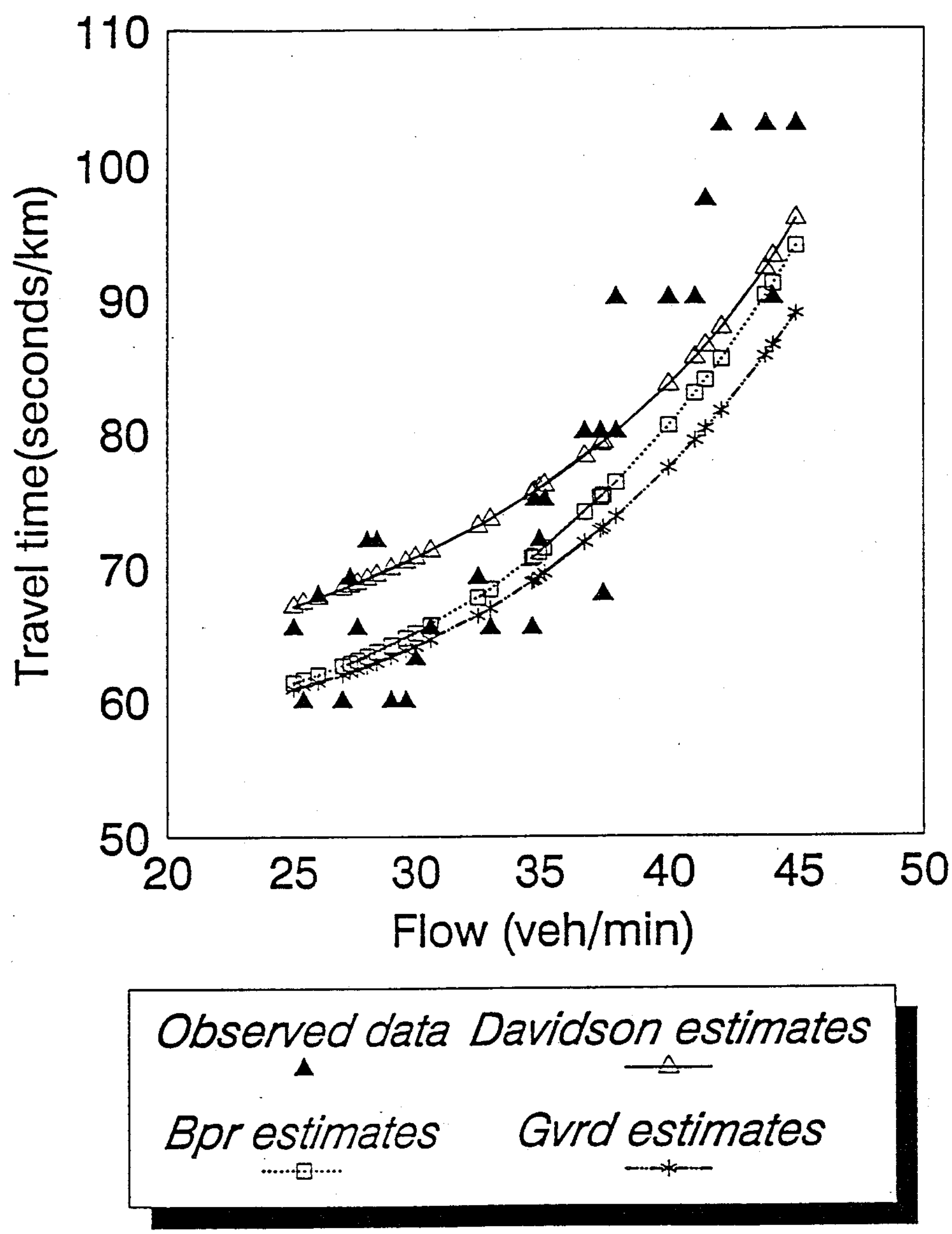

Figure 3.7 (Validating the revised models) Fitting the revised models with data. collected on Oak Street, between 49th Avenue and 57th Avenue Street

Chapter 4

DISCUSSION OF RESULTS

This chapter is in three main sections. The first section discusses capacities and free flow travel time as measured in the study. The second section offers explanations for the travel time trends, as observed in the City of Vancouver over the years, and the third section discusses the various travel time models investigated and the results produced.

4.1 Capacity and Free flow travel time

The saturation flows which were obtained from the study of through lanes for all the streets analysed were at least 2000 pcphgpl. This value is higher than the ideal saturation flow of 1800 pcphgpl reported by the Highway Capacity Manual although the former was under non-ideal conditions. This could be due to the fact that on average many more smaller cars now use the street system and also, as mentioned previously, some local factors not considered by the Highway Capacity Manual, might have influenced the results. As link capacities are determined based on saturation flows, the high values obtained for saturation flows resulted in high capacities. However, the capacity values used for all the street sections studied are not significantly different from the capacity values currently being used by the GVRD. Records have indicated that the GVRD has been updating the street capacities in their assignment algorithm over time.

The free flow travel time calculation was based on the average free flow speed measured in the field. The average free flow speed of about 62.2 km/hr (Table 3.8) is about 20 percent greater than the 50km/hr which is currently being used by the GVRD for most arterials (similar to the ones studied) in transportation planning and traffic assignment for the city. The average free flow speed of about 62.2 km/hr resulted in a lower free flow travel time of about 58 seconds per kilometre as compared to the 72 seconds per kilometer produced by using the old value of 50 km/hr. The relatively high value of the free flow speed, and for that matter, the low value of the free flow travel time, could be attributed to the improved technology which has led to the production of faster vehicles that are currently used on the street network.

4.2 The travel time patterns

The travel time patterns which have developed over the years in the City of Vancouver, as discussed previously have remained constant or shown slight improvements (Tables 2.2 and 2.3). This appears surprising, as there have been increases in traffic volumes and traffic control devices. The results from this research offer good explanations for the observed trends. The nature of the three travel time models being investigated showed that the higher the capacity values, the lower the link travel times. Also the lower the free flow travel times, the lower the link travel times. It is therefore clear that factors that contribute to an increase in free flow speed hence a reduction in free flow travel time and increase in capacities contribute to improvement in travel times. Therefore, although the traffic volumes and traffic control devices could increase, travel times could remain constant or even improve, so long as the speeds of vehicles and capacity of the street network increase. Some factors that contribute to increases in vehicle speeds and capacity of the street network in the City of Vancouver are discussed below.

4.2.1 Increase in vehicle speeds

The free flow speed studies that were conducted for this research has revealed that the average driver drives at a speed approximately 20 percent higher than the speed limit under free flow conditions. This indicates about 20 percent reduction in link travel time per kilometer as compared to driving at the speed limit value of 50 km per hour. Police reports (City Enginering Department 1989) have also shown that drivers are becoming more aggressive and have been driving faster. Also as mentioned earlier, the production of faster vehicles in recent times has also contributed to the high speeds. Another factor is that, because commuters still want to maintain their travel times despite the increase in traffic volumes and traffic control devices, they drive at higher speeds at any opportunity to make up for delays that they anticipate or they might have suffered previously due to traffic control devices or congestion.

4.2.2 Increase in capacities

Improvements in street capacities can significantly reduce travel times of vehicles. There have always been constant efforts by the City of Vancouver's Engineering Department to improve the capacity of the street network to safely accommodate the increasing traffic volumes. Some of the measures adopted to achieve this are summarised as follows:

(i) Arterial Development: The City of Vancouver's Engineering Department has always undertaken measures to improve its arterial system in order to increase capacities. Typical examples of these in the City of Vancouver in recent times are the Pacific Boulevard, Quebec Street, and Marpole By-pass. The development of these arterials have added significant capacities to these corridors.

(ii) Major left turn bay programs: Major left turn programs embarked upon by the City of Vancouver's Engineering Department have contributed immensely to capacity increases hence reducing travel times in the affected streets. A documented example of this is the improvement in travel time on Granville Street from 71st Avenue to 6th Avenue since 1977, which was partly due to the left turn bay improvements along Granville at 70th Avenue and at 41st Avenue.

(iii) Curb lane regulation programs: The removal of curb parkings along some street sections has contributed to capacity increases hence improving the travel times along the affected streets. An example documented by the City of Vancouver's Engineering Department is the improvement in travel time since 1977 along the Knight-Clark corridor which was partly due to the removal of curb parking along portions of this route.

(iv) Street structure and maintenance programs

Periodic maintenance of the street system such as the widening of roads and reconstruction of streets are some factors that provide additional capacities to streets in the City of Vancouver. Reports from the City's Engineering Department have shown that the improvement in travel time noticed between Kingsway Avenue and Marine Drive on Boundary Road from 1977 to 1988 could be attributed in part to the widening of Boundary Road southward from Kingsway Avenue.

(v) Computerised traffic signal management system

One of the major factors known to have contributed to travel time improvements in the City of Vancouver, especially in the last five years, is the installation of computerised traffic signal management system by the City's Engineering Department. This computerised system, which was installed in 1986 and includes all the 460 traffic signals in Vancouver, provides an efficient signal coordination and optimisation in the city. The effect of this is that the capacity of the street network is increased as the average stopped delay of the vehicles is reduced.

4.3 The travel time models

The three travel time models namely the BPR model, the GVRD model and the Davidson model were first tested for their effectiveness in predicting travel time as discussed under section 3.1.4. The capacity value and free flow travel time used in testing the models are the values currently being used by the Greater Vancouver Regional District for the data collection site and as such, should be presumably correct. However, the test results demonstrated that the models as tested with the parameters currently in use proved unsatisfactory (Table 3.5). Detailed discussions on each of the models is given in the following sections.

4.3.1 The BPR model

The BPR model failed to satisfactorily duplicate the observed data within the levels of significance of the test. The values of α and β (Equation 3.5), which are normally used with this model and were used for the test of the model, are 0.15 and 4 respectively. When the model was fitted directly with observed data to estimate the parameters that best fit the observed data, the value of β was found to be 4.03 which is almost the same as the commonly used value of 4. However, the value of α was found to be 0.52, which is significantly higher than the the value of 0.15 used in testing the model. The newly estimated value of α appeared to fit Vancouver data for arterial streets better as confirmed by the validation test results (Table 3.10). The failure of the model to reasonably duplicate the observed data when tested might therefore be due to the inaccurate values of α of 0.15 and also the free flow travel time of 72 seconds per kilometer used, instead of 58 seconds per kilometer obtained by direct measurement. The capacity value obtained by direct measurement is almost the same as the one used in testing the model and as such could not have contributed to the poor results produced by the model when tested.

4.3.2 The GVRD model

The GVRD model (Equation 3.4) when fitted to the calibration data produced the values of x, z and y as 1.12, 0.64 and 4.03 respectively. The values of the estimated parameters are all sufficiently close to the old ones being used by the GVRD which are 1.05, 0.6 and 4 respectively. The validation test also proved satisfactory with the slightly modified form of the GVRD model and with a free flow travel time of 58 seconds per kilometer. The failure of the GVRD model to satisfy the goodness of fit test conducted under section 3.1.4 could be attributed solely to the inaccurate value of free flow travel time used.

4.3.3 The Davidson model

The Davidson model (Equation 3.6) produced a j value of 0.22 when fitted with the observed data. The model also proved successful with the same j value when validated against data from other sites (Table 3.10). The j value used when first testing the model was 0.5. This is the mean of the range of 0.4-0.6 recommended by Blunden (1971) for arterial streets. The value of the j parameter of 0.22 produced by the Vancouver data is therefore not within the recommended range for arterial streets.

Davidson's j parameter has been an object of investigation by several researchers for a long time. Menon *et. al.* (1974) investigated the relationship between the value of j and factors such as the number of signalised intersections per kilometer, the number of lanes, the lane width, and the environment. In particular, Menon could not find any meaningful relationship between the values of j and any of the factors mentioned above.

In this research, although the validation test results (Table 3.10) showed that, of all the sites considered, the model proved most successful with data from Oak Street (the same street from which data was collected for calibrating the j parameter), it cannot be concluded from this research whether or not factors peculiar to Oak Street affected the results. The failure of the model when it was first tested could therefore be attributed to an erroneous value of the j parameter used initially and also the erroneously high free flow travel time.

Chapter 5

FURTHER RESEARCH

The subject of this thesis is expandable in several directions.

Inavailability of data and the high cost involved with collecting current data have made it impossible for some of the travel time models and algorithms discussed in the literature to be investigated. As such, with availability of funds, the research could be expanded to include all the models and the algorithms.

In this study, the travel time models were investigated with data collected on only arterial streets. The research could therefore be expanded to include other classes of roads such as roads in the central business district and freeways.

More accurate results may be obtained if the arterial streets are classified and average free flow speeds measured seperately for each road class. The streets could be classified in terms of some common factors such as, number of lanes, lane widths and other environmental factors. This is likely to produce more accurate results. Also collection of more data in working with the models and comparing the results with those produced by this research is recommended in any future study on the subject of this thesis.

Chapter 6

CONCLUSION

Travel time trends over the past three decades in the City of Vancouver have been investigted in this study. The travel times have remained surprisingly constant, although traffic volumes and the number of traffic control devices have increased.

Three traditional travel time models have also been investigated for their validity in predicting travel time on arterial streets in the City of Vancouver, under present traffic conditions. The pertinent results of the research are summarised in the following discussions.

The fairly constant trend of travel times observed in the City of Vancouver over the years are principally due to increases in vehicle speeds and increases in the capacity of the street network. The increased vehicle speeds and street capacities tend to offset any deteriorations in travel times that might have resulted from the increased traffic vloumes and the increased number of traffic control devices. The increases in vehicle speeds could be ascribed to the advancement in technology, which has contributed to the production of faster vehicles, that are currently used on the street network. Another likely contributing factor to the higher vehicle speeds, is the increased aggressiveness of drivers as documented in the police reports. The increases in the capacity of the street network, are the results of the constant efforts made by the City of Vancouver's Engineering Department to improve the capacities of the streets. These efforts are seen in the form of development of the arterials, major left turn bay programs, curb lane regulation programs, street and structural maintenance programs and the installation of a computerised traffic signal management system. Also, increases in the number of smaller cars that are currently used on the street network have resulted in higher saturation flows, hence higher capacity values.

The three traditional travel time models investigated are the BPR model, the Davidson model and the GVRD model. The other travel time models and algorithms discussed in the literature could not be investigated due to data limitations.

The Davidson model (Equation 3.6), proved invalid with data collected on a few arterial streets in the City of Vancouver. The failure of the model to provide a good fit for the data is due to the j value of 0.5 and free flow travel time value of 72 seconds per kilometer used. Free flow travel time study conducted in the field produced a value of 58 seconds per kilometer. Also a j value of 0.22 provides a good fit for the Vancouver data. A revised form of the Davidson model developed for the arterial streets in Vancouver is given in Equation 3.14.

The BPR model (Equation 3.5) also failed to satisfy the goodness of fit test with the data collected in the City of Vancouver. Its failure can be attributed to the α value of 0.15 and also the erroneous value of free flow travel time used. However, an α value of 0.52 provides a good fit for data collected on arterial streets in the City of Vancouver. A revised form of the BPR model derived for arterial streets in the City of Vancouver is given in Equation 3.12.

The GVRD model (Equation 3.4) also did not provide a good fit with the Vancouver data. However, the constant model parameters, x, y, and z determined by direct calibration agree fairly well with the ones currently in use. The failure of the model can therefore be attributed to the erroneous value of free flow speed used when testing the model. A slightly modified form of the GVRD model as revealed by this research is as given in Equation 3.20.

It is recommended that the revised models and the new value of free flow travel time obtained from this research be considered for use in future traffic assignments on arterial streets in the City of Vancouver.

REFERENCES

1. Ang, A.H. and Tang, H.T. (1975) "Probability concepts in Engineering Planning and Design." Vol. 1, Chapter 4.

2. Beck, J.V. and Arnold, K.J. (1977). "Parameter Estimation in Engineering and Science." John Wiley and sons, New York.

3. Blunden, W.R. (1971). "Land-use/Transport system." Pergamon Press, New York.

4. Carmines, E.G. and Zeller, A.R. (1979). "Reliability and Validity assessment." Sage Publications, California.

5. City Engineering Department (1989). "Travel time study, 1987-88." Vancouver, B.C.

6. City Engineering Department (1980). "Travel time study, 1980." Vancouver B.C.

7. Fisk, C.S. (1988) " A transportation planning model for detailed traffic analyses." Transportation Research Series report No. 11, U.B.C., Dept. of Civil Engineering, Vancouver, B.C.

8. Fisk, C.S. (1989). "Link travel time functions for traffic assignment." Transportation Research Part B, Methodological, Vol. 25B, 103-113.

9. Gerlough, D.L. and Huber, M.J. (1975). "Traffic flow theory, A monogragh." Special Report 165, Transportation Research Board, National Research Council, Washington, D.C.

10. Haase, R.H. (1968). "Increasing freeway utilization for motorists." Paper presented at 34th national meeting of Operations Research Society of America, Philadelphia.

11. Henderson, D.M., Lam, J.K. and Rock Stephen (1988). "The City of Vancouver traffic management system."Institute of Transportation Engineers Journal, Vol. 58, No.6, 17-21.

12. Hoffman, G. and Janko, J. (1990). "Travel times as a basic part of the LISB Guidance strategy." IEE Conference Publication, No. 30, 1-8.

13. Hogg, R.V. and Ledolter,J. (1987). "Engineering statistics". Macmillan, London.

14. Institute of Transportation Engineers (1982). "Transportation and Traffic Engineering Handbook." Prentice Hall, Englewood Cliffs, New Jersey.

15. Lucas, C. and Davidson, K.B. (1973). "A method of measuring the effectiveness of a given assignment method and an an application to three different methods."Australia Road Research Journal, Vol. 5, No. 5, 92-99.

16. Lucas, C. and Davidson, K.B. (1974). "Quantitative evaluation of traffic assignment methods." University of Queensland, Dept. of Civil Engineering. Bulletin No. 17.

17. May D. Adolf (1990). "Traffic flow fundamentals." Prentice Hall, Englewood Cliffs, New Jersey.

18. Menon *et. al.* (1974)."Study of Davidson's flow travel time relationship." Proceedings[of the conference]. Australian Road Research Board. Vol. 7, part4, 5-21.

19. Miller, A.J. (1969). "On the Australian Road Capacity Guide." Highway Research Record, 289, 1-13.

20. Montgomery, F.O. and May, A.D. (1987). "Factors affecting travel times on urban radial routes." Traffic Engineering and Control, Vol. 28, No. 9, 452-458.

21. National Research Council. (1985). "Highway Capacity Manual." Transportation Research Board, Special Report 209, Wasington D.C., Chapter 9.

22. Navin, F.P.D. (1991). "Vehicle operations, traffic speed and delay models." For NCHRP 2-18. Research strategies for improving Highway user cost estimating strategies.

23. Navin, F.P.D. (1991). "Speed : Some definitions and observations." Proceedings of the Canadian Multidisciplinary Road Safety Conference VII, Vancouver, B.C.

24. Oda T. (1990). "Algorithm for prediction of travel time using vehicle sensor data." IEE Conference Publication No. 320, 40-44.

25. Smeed, R.J. (1968). "Some circumstances in which vehicles will reach their destinations earlier by starting later." Transportation Science Vol. 1, No. 4, 308-317.

26. Stokes W. Roberts. (1989). "Some factors afffecting signalised intersection capacity." Institute of Transportation Engineers Journal, Vol. 59, No. 1, 35-40.

27. Taylor, M.A.P. (1977). "Parameter estimation and sensitivity of parameter values in a flow-rate/travel time relation." Transportation Science, Vol. 11, No. 3, 275-292.

28. U.S. Department of Commerce, Bureau of Public Roads (1964). "Traffic assignment

manual." Office of Planning, Urban Planning Division.

29. Usami *et. al.* (1986). "Travel time prediction algorithm and signal operations at critical intersections for controlling travel time." IEE Conference Publication No. 260, 205-208.

30. Wardrop, J.G. (1952). "Some theoretical aspects of road traffic research." Excerpt part 11, Proceedings of the Institution of Civil Engineers."

Appendix A

TABLE OF RESULTS

Traffic flow (veh/hr)	speed (km\hr)	Travel time (secs/km)	Traffic flow (veh/hr)	Speed (km/hr)	Travel time (secs/km)
1638	35	102.86	1320	43	83.72
1590	38	94.74	1374	48	75.00
1566	35	102.86	1320	45	80.00
1500	40	90.00	1302	55	65.45
1500	37	97.30	1260	53	67.92
1548	42	85.71	1200	50	72.00
1494	44	81.82	1200	52	69.23
1380	40	90.00	1218	55	65.45
1362	45	80.00	1140	57	63.16
1338	48	75.00	1140	60	60.00
1116	57	63.16	1032	55	72.00
1104	55	65.45	1032	52	67.23
1080	50	72.00	960	55	65.45
1080	58	62.07	942	60	60.00
1020	55	65.45	900	58	62.07
1020	52	69.23	804	65	55.38

Table A.1 Traffic flow and speed data, collected between Fraser Street and Clark Street on 12th Avenue, with computed travel times (15/5/92)

Traffic flow (veh/hr)	speed (km\hr)	Travel time (secs/km)	Traffic flow (veh/hr)	Speed (km/hr)	Travel time (secs/km)
2694	35	102.86	2208	45	75.00
2640	40	120.00	2112	48	80.00
2622	35	102.86	2280	45	65.45
2520	35	102.86	2082	55	67.92
2484	37	97.30	2250	53	72.00
2460	40	90.00	2100	50	69.23
2400	40	90.00	1950	52	65.45
2280	40	90.00	1980	55	63.16
2244	45	80.00	1800	57	60.00
2088	48	75.00	1776	60	60.00
1740	60	63.16	1560	53	69.23
1656	55	65.45	1500	55	65.45
1680	50	72.00	1524	60	60.00
1620	60	62.07	1440	58	62.07
1836	55	65.45	1200	65	55.38
1638	52	69.23	1020	50	72.00
1704	50	65.45	912	55	65.45

Table A.2 Traffic flow and speed data, collected between 49th Avenue and 57th Avenue on Oak Street, with computed travel times (22/5/92)

Traffic flow (veh/hr)	speed (km\hr)	Travel time (secs/km)	Traffic flow (veh/hr)	Speed (km/hr)	Travel time (secs/km)
720	50	72.00	1212	42	85.71
1020	40	90.00	1140	32	112.5
1080	40	90.00	1320	35	102.86
1296	35	102.86	1020	40	90.00
1224	40	90.00	1140	40	90.00
1242	30	120.00	1320	42	85.71
1296	30	120.00	960	50	72.00
1176	40	90.00	1092	40	90.00
1122	50	72.00	1068	50	72.00
1122	40	90.00	1044	45	80.00
1176	45	80.00	858	60	60.00
1140	40	90.00	1032	55	65.45
1140	50	72.00	1014	50	72.00
1098	50	72.00	960	50	72.00
1098	55	65.45	738	55	65.45
912	40	90.00	840	65	55.38
864	50	72.00	822	55	65.45
774	60	60.00			

Table A.3 Traffic flow and speed data, collected between 16th Avenue and King Edward Avenue on Arbutus Street, with computed travel times (29/5/92)

Traffic flow (veh/hr)	Predicted travel times (seconds/km)			Traffic flow (veh/hr)	Predicted travel times (seconds/km)		
	Davidson Model	BPR Model	GVRD Model		Davidson Model	BPR Model	GVRD Model
1638	218.69	86.22	112.68	1320	138.00	77.96	89.05
1590	199.20	84.61	108.09	1302	135.51	77.64	88.13
1566	190.94	83.86	105.94	1260	130.15	76.94	86.13
1500	172.00	81.97	100.53	1200	123.43	76.06	83.61
1500	172.00	81.97	100.53	1200	123.43	76.06	83.61
1548	185.27	83.32	104.40	1218	125.34	76.31	84.33
1494	170.51	81.81	100.08	1140	117.60	75.30	81.44
1380	147.27	79.13	92.39	1140	117.60	75.30	81.44
1362	144.32	78.76	91.34	1116	115.48	75.03	80.67
1338	140.62	78.29	90.00	1104	114.46	74.90	80.30
1320	138.00	77.96	89.05	1080	112.50	74.65	79.59
1374	146.27	79.00	92.04	1080	112.50	74.65	79.59
960	104.00	73.65	76.72	1020	108.00	74.11	78.03
942	102.89	73.53	76.38	1020	108.00	74.11	78.03
900	100.42	73.27	75.64	1032	108.86	74.21	78.32
804	95.42	72.81	74.31	1032	108.86	74.21	78.32

Table A.4 Predicted travel times, using the old travel time models, with traffic flow data collected between Clark Street and Fraser Street on 12th Avenue

Traffic flow (veh/hr)	Predicted Travel Times (seconds/km)			Traffic flow (veh/hr)	Predicted travel times (seconds/km)		
	Davidson Model	BPR Model	GVRD MOdel		Davidson Model	BPR Model	GVRD
720	101.79	76.88	76.20	1020	136.42	91.80	89.09
738	103.18	77.39	76.64	1032	138.58	92.83	89.91
774	106.15	78.54	77.62	1044	140.84	93.83	90.76
822	110.53	80.33	79.16	1068	145.66	95.92	92.56
840	112.32	81.09	79.81	1080	148.24	97.02	93.51
858	114.20	81.90	80.51	1092	150.94	98.16	94.49
864	114.84	82.18	80.75	1098	152.34	98.75	94.99
912	120.42	84.66	82.88	1098	152.34	98.75	94.99
960	126.86	87.57	85.38	1122	158.31	101.18	97.09
960	126.86	87.57	85.38	1122	158.31	101.18	97.09
1014	135.38	91.41	88.68	1140	163.20	103.12	98.75
1020	136.42	91.88	89.09	1140	163.20	103.12	98.75
1242	200.48	115.95	109.78	1140	163.20	103.12	98.75
1296	230.69	124.17	116.85	1140	163.20	103.12	98.75
1296	230.69	124.17	116.85	1176	174.26	107.27	102.32
1320	248.00	128.18	120.29	1176	174.26	107.27	102.32
1328	248.00	128.18	120.29	1212	187.43	111.83	106.24
1224	192.39	113.44	107.62				

Table A.5 Predicted travel times, using the old travel time models, with traffic flow data collected between 16th Avenue and King Edward Avenue on Arbutus Street

Traffic flow (veh/hr)	Predicted travel times (seconds/km)			Traffic flow (veh/hr)	Predicted travel times (seconds/km)		
	Davidson Model	BPR Model	GVRD Model		Davidson Model	BPR Model	GVRD Model
2694	179.05	116.57	107.93	2250	132.00	93.57	89.39
2640	171.00	113.07	105.12	2100	122.40	88.33	85.17
2622	168.52	111.96	104.22	1950	114.55	84.12	81.77
2520	156.00	106.05	99.46	1980	116.00	84.88	82.39
2480	152.13	104.13	97.91	1800	108.00	80.78	79.08
2460	149.68	102.90	96.91	1776	107.05	80.31	78.70
2400	144.00	99.97	94.55	1740	105.68	79.65	78.17
2280	134.18	94.75	90.34	1656	102.67	78.27	77.06
2244	131.58	93.34	89.20	1680	103.50	78.65	77.36
2088	121.71	87.96	84.87	1620	101.45	77.74	76.63
2208	129.10	91.99	88.12	1836	109.47	81.50	79.66
2112	123.10	88.71	85.47	1638	102.06	78.00	76.84
2280	134.18	94.75	90.34	1704	104.35	79.04	77.67
2082	121.38	87.78	84.72	1560	99.53	76.93	75.97
1200	90.00	73.71	73.38	1500	97.71	76.21	75.39
1020	86.23	72.89	72.72	1524	98.43	76.49	75.62
912	84.21	72.57	72.46	1440	96.00	75.57	74.88

Table A.6 Predicted travel times, using the old travel time models, with traffic flow data collected between 57th Avenue and 49th Avenue on Oak Street

Traffic flow (veh/hr)	Predicted travel times (seconds/km)			Traffic flow (veh/hr)	Predicted travel times (seconds/km)		
	GVRD Model	BPR Model	Davidson Model		GVRD Model	BPR Model	Davidson Model
1638	92.96	97.7	109.99	1338	73.47	75.57	82.32
1590	89.01	93.22	103.09	1320	72.65	74.64	81.39
1566	87.16	91.12	100.16	1374	75.22	77.55	84.32
1500	82.52	85.85	93.44	1320	72.65	74.64	81.39
1500	82.52	85.85	93.44	1302	71.86	73.74	80.51
1548	85.83	89.62	98.15	1260	70.14	71.80	78.61
1494	82.12	85.40	92.91	1200	67.96	69.33	76.23
1380	75.52	77.90	84.68	1200	67.96	69.33	76.23
1362	74.62	76.87	83.63	1218	68.59	70.03	76.91
1140	66.11	67.21	74.16	1020	63.18	63.89	70.76
1140	66.11	67.21	74.16	1032	63.43	64.17	71.06
1116	65.45	66.46	73.41	1032	63.43	64.17	71.06
1104	65.12	66.10	73.05	960	62.06	62.16	69.34
1080	64.52	65.41	72.36	942	61.76	62.27	68.95
1080	64.52	65.41	72.36	900	61.13	61.55	68.07
1020	63.18	63.89	70.76	804	59.99	60.26	66.30

Table A.7 Predicted travel times, using the revised travel time models, with traffic flow data, collected between Fraser Street and Clark Street on 12th Avenue

Traffic flow (veh/hr)	Predicted travel times (seconds/km)			Traffic flow (veh/hr)	Predicted travel times (seconds/km)		
	GVRD Model	BPR Model	Davidson Model		GVRD Model	BPR Model	Davidson Model
2694	88.87	93.90	95.94	2088	69.06	70.86	75.62
2640	86.46	91.10	93.09	2208	71.85	74.10	78.24
2622	85.68	90.19	92.21	2112	69.58	71.46	76.11
2520	81.59	85.43	87.77	2280	73.76	76.33	80.04
2484	80.26	83.89	86.40	2082	68.93	70.71	75.50
2460	79.41	82.89	85.53	2250	72.94	75.37	79.27
2400	77.38	80.53	83.52	2100	69.31	71.16	75.86
2280	73.76	76.33	80.04	1950	66.39	67.76	73.08
2244	72.78	75.19	79.12	1980	66.93	68.38	73.60
1800	64.07	65.07	70.76	1704	62.87	63.67	69.47
1776	63.76	64.70	70.42	1560	61.42	61.97	67.76
1740	63.30	64.17	69.94	1500	60.92	61.39	67.11
1656	62.34	63.05	68.87	1524	61.10	61.61	67.37
1680	62.60	63.35	69.17	1440	60.47	60.88	66.51
1620	61.98	62.62	68.44	1200	59.19	59.38	64.38
1836	64.58	65.66	71.28	1020	58.62	58.72	63.64
1638	62.16	62.83	68.65	912	58.39	58.46	62.33

Table A.8 Predicted travel times, using the revised travel time models, with traffic flow data collected between 49th Avenue and 57th Avenue on Oak Street

Traffic flow (veh/hr)	Predicted travel times (seconds/km)			Traffic flow (veh/hr)	Predicted travel times (seconds/km)		
	GVRD Model	BPR Model	Davidson Model		GVRD Model	BPR Model	Davidson Model
720	61.61	62.09	68.56	1140	80.98	84.07	90.33
1020	72.68	74.65	80.83	1320	99.50	105.1	120.38
1080	76.48	78.96	85.02	1020	72.68	74.65	80.83
1296	96.54	101.7	114.25	1140	80.98	84.07	90.33
1224	88.61	92.72	100.67	1320	99.50	105.1	120.38
1242	90.46	94.82	103.54	960	69.50	71.04	77.44
1296	96.54	101.7	114.25	1176	84.05	87.55	94.25
1212	87.42	91.37	98.91	1122	79.56	82.45	88.59
1122	79.56	82.45	88.59	1044	74.12	76.29	82.40
1176	84.05	87.55	94.25	858	65.31	66.29	72.96
1140	80.98	84.07	90.33	1032	73.39	75.45	81.60
1140	80.98	84.07	90.33	1014	72.34	74.26	80.46
1098	77.76	80.41	86.48	960	69.50	71.04	77.44
1098	77.76	80.41	86.48	738	61.98	62.52	69.05
1092	77.33	79.92	85.98	912	67.35	68.61	75.16
1068	75.67	78.04	84.11	864	65.52	66.53	73.19
840	64.71	65.61	72.29	774	62.83	63.48	70.10
822	64.15	64.98	71.66				

Table A.9 Predicted travel times, using the revised travel time models, with traffic flow data collected between 16th Avenue and King Edward Avenue on Arbutus Street

Site	Saturation flows(pcphgpl)		No. of shared and through lanes	Proportion of green at approach intersection	Average capacity per lane (pcphpl)
	Through lane	Shared lane (Right and Through traffic)			
Oak Street 49th-57th	2000	1680	3	0.61	1153
Arbutus Street 16th-King Ed.	2040	1920	2	0.49	970
12th Avenue Clark-Fraser	2200	2000	2	0.49	1012

Table A.10 Summary of Saturation flow study results and determined Capacity values

Appendix B

EXPECTED VALUES OF TRAVEL TIME AND VARIANCES USING THE REVISED MODELS

The probabilistic mean travel times and variances which commuters should expect while travelling on arterial links in the City of Vancouver could be estimated with each of the revised models using an approximate form of the Taylor series expansion. This is as described below.

For a general equation of the form :

$$g(X) = X_1 + X_2 + X_3 / X_4 \ etc \qquad (9.1)$$

where X_i are random variables that may be related, the expected value is given as:

$$E[g(X)] = E[g(\bar{X})] + 1/2 \sum_{i=1}^{n} \sum_{j=1}^{n} \frac{\delta^2 g(X)}{\delta X_i \delta X_j} Cov(X_i, X_j) \qquad (9.2)$$

and the variance is also given as follows:

$$Var[g(X)] = \sum_{i=1}^{n} \sum_{j=1}^{n} C_i C_j Cov(X_i, X_j) \qquad (9.3)$$

where:

$C_i = \delta g(X) / \delta X_i$

$$C_j = \delta g(X)/\delta X_j$$

A detailed discussion of the above formulae is found in Ang and Tang (1975) page 198. There are a number of important precautions that need to be considered when estimating the probabilistic mean travel times and variances using the revised models. One of the precautions is that the time period for which the estimates are being made must be well defined. This is due to the fact that the distribution of the random variable (V_i) is very important in the estimation process and it might differ markedly from one time period to another time period (e.g. peak and off peak conditions). Also link capacities change with the time of the day depending on which signal timing plan is in operation. More reliable estimates could be obtained if the analysis is made over short time periods and with relatively large sample sizes.

The Taylor series formula is used to estimate the probabilistic means and variances of the travel time using the revised models and with the traffic flow data collected between 41st Avenue and 49th Avenue on Oak Street. The results however apply to only the link from which the data were collected and the time period during which the data were collected which in this case is the morning peak period. The mean traffic flow rate is 30.58 vehicles per minute and the variance is 120.33 $(vehicles/\ minute.)^2$

B.1 Estimates using the BPR model

The revised BPR model is given as:

$$T_i = T_o[1 + 0.52(\frac{V_i}{C})^{4.03}] \qquad (9.4)$$

The link capacity value, C is 43 vehicles per minute and the free flow travel time T_o is

58

seconds per kilometer.

The expected value of the travel time is given as:

$$E(\overline{T}) = 58 + 0.52\frac{\overline{V}^{4.03}}{43^{4.03}} + 1/2\frac{\delta^2 T}{\delta V^2}Var(V) \qquad (9.5)$$

$$\frac{\delta T}{\delta V} = 3.14x10^{-5}\overline{V}^{3.03} \qquad (9.6)$$

$$\frac{\delta^2 T}{\delta V^2} = 9.52 \ x \ 10^{-5}\overline{V}^{3.03} \qquad (9.7)$$

By using equations ***9.7, 9.5*** and the mean value of ***V***, the expected value is obtained as *71.5 seconds per kilometer.*

The variance is given as:

$$Var(T) = \left(\frac{\delta T}{\delta V}\right)^2 \times Var(V) \qquad (9.8)$$

Using equations ***9.6, 9.8*** and the variance of ***V***, the variance of the link travel time is obtained as ***119.38*** *(seconds/kilometer)²*.

B.2 Estimates using the GVRD model

The revised GVRD model is given as:

$$T_i = T_o\left[1 + 0.64\left(\frac{V_i}{CL^{1.12}}\right)^{4.03}\right] \qquad (9.9)$$

L is the number of lanes = ***3***

C is the average practical capacity per lane = ***14.3*** *vehicles per minute per lane.*

The expected value of the travel time is given as:

$$E(\bar{T}) = 58 + \frac{0.64\bar{V}^{4.03}}{6.44x10^{6}} + 1/2\frac{\delta^{2}T}{\delta V^{2}}Var(V) \quad (9.10)$$

$$\frac{\delta T}{\delta V} = 2.32x10^{-5}\bar{V}^{3.03} \quad (9.11)$$

$$\frac{\delta^{2}T}{\delta V^{2}} = 7.03x10^{-5}\bar{V}^{2.03} \quad (9.12)$$

Using equations ***9.12, 9.10*** and mean value of ***V***, the expected mean link travel time is obtained as *77.9 seconds per kilometer*. Also, by using equations ***9.8, 9.11*** and the variance of ***V***, the variance of the link travel time is found to be *176.73 (seconds/ kilometer)^2*.

B.3 Estimates using the Davidson model.

The revised Davidson model is given as :

$$T_{i} = T_{o}[1 + 0.22\frac{V_{i}}{(C - V_{i})}] \quad (9.13)$$

C is the total link capacity = ***57.3*** *vehicles/minute*

$$E(\overline{T}) = 58 + \frac{12.76\overline{V}}{57.3-\overline{V}} + 1/2\frac{\delta^2 T}{\delta V^2}Var(V) \tag{9.13}$$

$$\frac{\delta T}{\delta V} = 12.76\frac{(57.3 + \overline{V})}{(57.3 - \overline{V})^2} \tag{9.15}$$

$$\frac{\delta^2 T}{\delta V^2} = 12.76[\frac{(57.3 - \overline{V})^2 - (6566.58 - 2\overline{V}^2)}{(57.3 - \overline{V})^4}] \tag{9.16}$$

Using equations ***9.16, 9.14*** and the mean value of ***V***, the expected mean link travel time is obtained as ***66.61*** *seconds/kilometer*. Also, by using equations ***9.15, 9.8*** and the variance of ***V***, the variance of the link travel time was found to be ***296.98*** *(seconds/kilometer)*^{2}.

The Davidson model gave the least expected value of the travel time as ***66.61*** *seconds/km.* However, it produced the highest variance as ***298.98*** *(s/km)*^{2}. The expected values obtained with the BPR model and the GVRD model are ***71.5*** *s/km and* ***77.95*** *s/km* with variances as ***119.38*** *(s/km)*^{2} *and* ***176.73*** *(s/km)*^{2} respectively.