Fundamental relationships for freeway traffic flows

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This report presents findings of a study of fundamental characteristics of freeway traffic flows based on analysis of individual vehicle data collected on the Eastern Freeway in Melbourne under both saturated and unsaturated conditions using a two-loop presence detection system. The aim of the study was to assess data collection and analysis methods, and to develop analytical models to describe the relationships between traffic flow parameters. The traffic flow parameters considered include flow rate, speed, density, spacing, gap length, vehicle length, headway, occupancy and space time, gap time, vehicle passage time and occupancy ratios. Travel speeds were also measured using an instrumented car. Six analytical traffic flow models are presented including various single-regime and two-regime models. Model calibration results are summarised and the parameters representing the traffic characteristics at the survey site are given. Graphs showing fundamental relationships with measured data and model predictions are presented.

About the authors

Rahmi Akçelik

Dr Akçelik is a leading scientist in the area of traffic management with over 180 technical publications. He is the author of the SIDRA software package. Dr Akçelik represents ARRB TR at the AUSTROADS Traffic Management Reference Group. He is a member of the Signalised Intersections Subcommittee of the US Transportation Research Board (TRB) Committee on Highway Capacity and Quality of Service, and the TRB Committee on Traffic Signal Systems. Dr Akçelik graduated as a Civil Engineer from Istanbul Technical University (1968), and received a Ph.D. in Transportation Engineering from Leeds University, England. After working as a traffic engineer-planner with the National Capital Development Commission, Canberra, he joined ARRB TR (1979). Dr Akçelik was awarded the 1999 Clunies Ross National Science and Technology award.

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Ron Roper joined ARRB Transport Research in 1988. He has been involved in development of software and electronic hardware, field data collection methods and operation of productivity tools for asset management. Ron has wide experience in data analysis and processing using dedicated analysis packages, spreadsheets and purpose written applications. Prior to joining ARRB TR, Ron worked for the Federal Airports Corporation and its predecessors for twenty years in the field of electrical and electronic systems.

Mark Besley

Mark Besley joined ARRB TR in 1980 after studying applied mathematics at Monash University. Mark has been involved in many aspects of research on traffic operations including incident detection, traffic network models, fuel consumption estimation, traffic data collection and analysis, as well as software development, user support and training. He has made a significant contribution to the development and support of the SIDRA computer package for intersection analysis.

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Executive Summary

This research report presents findings of a study of fundamental characteristics of freeway traffic flows. The study was carried out under the ARRB TR research project RC 7082 *Reassessment of Fundamental Speed-Flow Relationships for Freeway Traffic Control* funded by AUSTROADS (Project NRUM 9712). The results given in earlier reports WD R 98/044, WD R 98/045 and Contract Report RC 7082 produced under this project are superseded by this research report.

The aim of the project was to carry out field measurements of freeway traffic flow characteristics under both free-flowing (unsaturated) and congested (saturated) conditions, and to develop improved analytical models to describe the relationships between the traffic flow parameters. The results of the report are useful for practical applications including:

(i)   design of freeway facilities with adequate capacity and level of service, and

(ii)   development of improved methods for the purposes of

   • incident detection,

   • ramp metering,

   • traffic monitoring, and

   • driver information.

A related ARRB TR research project RC 7057 *Fundamental Relationships for Adaptive Control* funded by the Roads and Traffic Authority of New South Wales (RTA NSW Project No. 6352 1/8) focused on traffic flow relationships at traffic signals, described in full in Research Report ARR 340 "Fundamental Relationships for Traffic Flows at Signalised Intersections". The two projects for freeways and traffic signals represent uninterrupted and interrupted flow conditions, respectively. They have much in common in terms of establishing data collection and analysis methodology for fundamental traffic flow relationships and developing analytical models.

Information on freeway traffic flow characteristics has been largely based on overseas work, and mainly US publications, e.g. the Highway Capacity Manual. The aim of the ARRB TR freeway research project was to study the applicability of overseas data and models to Australian conditions, and to extend the work on fundamental relationships for traffic flow at traffic signals to better understand current traffic flow characteristics on freeways.

The report presents an introduction to the fundamental traffic flow parameters and their relationships. The traffic flow parameters considered include flow rate, speed, density, headway, occupancy time, space time, gap time, vehicle passage time, spacing, space (gap) length, vehicle length, and time and space occupancy ratios. The relation between fundamental parameters for interrupted and uninterrupted traffic flows is explained.

The basis of the work was the measurement of headway and speed for *individual vehicles* at a point on the freeway. Other traffic parameters such as flow rate, density,

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spacing and occupancy ratios were derived from these parameters. The method used for collecting data on the Eastern Freeway in Melbourne is described, and issues related to data collection and analysis are discussed in detail.

A permanent VicRoads two-loop presence detection system was linked to VDAS traffic counters for the purpose of this research. The detection system and the characteristics of the freeway survey site are described. Data covered the full range of traffic conditions from free-flowing to congested traffic (individual vehicle speeds were in the range 10 - 132 km/h, and average speeds for 5-min intervals were in the range 36 - 103 km/h). Travel speeds were also measured during the survey period using an instrumented car. Comparison of speed data from the two survey methods indicated that data collected by the two-loop detection system was generally acceptable.

To obtain parameters describing the characteristics of freeway traffic streams, data for individual vehicles were aggregated for periods of 20 s, 1 min, 2 min, 5 min and 15 min. Aggregate data were always derived from individual vehicle data, not from the aggregate data for a shorter period. The data aggregation method used in this study aimed to achieve consistency between speed and all time-based and distance-based parameters. The report recommends that the data collection and analysis method described should be employed for future analysis of freeway traffic flows.

There is a vast amount of literature on fundamental traffic relationships for uninterrupted traffic flow offering empirical as well as theoretical models based on car-following, hydrodynamic and kinetic theories of traffic flow. However, these models fail to provide a complete description of the speed, flow and density characteristics observed in practice for a full range of flow conditions.

The report presents six analytical traffic flow models including single-regime and two-regime models. A single-regime model is a single function that applies to both saturated and unsaturated flow conditions. A two-regime model consists of two functions that apply to saturated and unsaturated flow conditions separately. The models are presented as speed - flow rate relationships from which other relationships such as speed - density, space time - speed and occupancy ratio - flow rate can be derived.

Model calibration results are given for the Eastern Freeway site. A constrained non-linear regression analysis method was used for model calibration. Regression analyses were carried out using data aggregated for 5-min intervals. This interval was chosen as the most appropriate interval for the purpose of evaluating alternative models. Several regression options were tried for each model with different model parameters specified. Results for all models and all regression options are presented.

By inspecting the graphs of predicted traffic flow parameters against measured values for all models, it was found that Models 4 and 5 together as a two-regime model ("Model 4+5") provided the best estimates of traffic characteristics for the survey site.

In addition to giving best predictions overall, Model 4+5 combination was preferred because:

(i)   Model 4 is applicable for both undersaturated and oversaturated conditions, including allowance for initial queued demand at the start of the analysis period,

(ii)   Model 4 has more general applicability as it has been used to model uninterrupted movements in the SIDRA software package, and a US study has shown that it has desirable characteristics for traffic assignment purposes, and

(iii)   in Research Report ARR 340, Model 4 is shown to be applicable to uninterrupted flows at traffic signals and Model 5 is shown to be applicable to queue discharge flows at traffic signals, thus both Models 4 and 5 apply to both uninterrupted and interrupted facilities.

Graphs showing fundamental relationships for all traffic flow parameters with measured data and model predictions are presented for Model 4+5.

The data used for the results given in the report include some heavy vehicle data (about 3 per cent). The effect of this is expected be minimal due to the low percentage of heavy vehicles. The report recommends further work on the effect of heavy vehicles on fundamental relationships. Sites with a high percentage of heavy vehicles are needed for this purpose.

This study used data from a single survey site with the purpose of in-depth assessment of data collection and analysis methods, and evaluation of alternative models. The report recommends that the method established in this study should be applied to collect and analyse data at a large number of sites to establish the range of traffic parameters possible on Australian freeways. The VicRoads driver information system routinely collects large amounts of data that could be used for this purpose.

Using a database that includes a large number of freeway sites, methods could be developed to predict traffic flow parameters from freeway geometric and traffic characteristics, e.g. taking into account factors such as number of lanes, lane width, lateral clearance, grade, driver behaviour, and proportion of heavy vehicles in the stream, similar to the method used in the US Highway Capacity Manual. Research into lane utilisation on different freeway segments (basic segment, ramp junctions and weaving areas) is recommended to take into account different flow rates in different lanes on a given section of freeway.

The report recommends further research on saturated traffic flows on freeways covering a wide range of low speed conditions. A data collection method with higher accuracy levels than the two-loop presence system is needed for this purpose. Accuracy of loop data is not sufficient for refining data under saturated conditions to allow for accelerations and decelerations. VDAS treadle detectors were not considered to be strong enough to endure the high speed, high volume traffic conditions on freeways. It may be possible to overcome such problems using new detector technologies.

A summary of traffic flow parameters for the survey site on the Eastern Freeway are given below (based on 5-min aggregation period):

Free-flow speed, v_f (km/h)	101
Speed at maximum flow rate, v_n (km/h)	90
Maximum flow rate, q_n (veh/h)	2500
Average headway corresponding to maximum flow rate, h_n (s)	1.44
Spacing at maximum flow rate, L_{hn} (m/veh)	36.0
Density at maximum flow rate, k_n (veh/km)	27.8
Occupancy time at max. flow rate, t_{on} (s)	0.25
Space time at maximum flow rate, t_{sn} (s)	1.19

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Time occupancy ratio at max. flow rate, O_{tn}	18 %
Time occupancy ratio at jam density, O_{tj}	42 %
Jam spacing, L_{hj} (m/veh)	15.0
Jam density, k_j (veh/km)	66.7
Average vehicle length, L_v (m/veh)	4.4

Frequency distributions for individual vehicle headways indicated that there was a high proportion of small headways at this site, for example, the frequency of headways equal to or less than 1.0 s was 28 per cent.

The capacity characteristics observed at the Eastern Freeway site as indicated above appear to be in line with those specified in the US Highway Capacity Manual and found in recent studies in New Zealand. The maximum flow rate for the Eastern Freeway site is 2500 veh/h based on 5-min aggregation period and approximately 2300 veh/h based on 15-min aggregation period. While the 15-min aggregation period is appropriate for capacity and level of service analysis, investigation of the most appropriate aggregation period for incident detection and ramp metering purposes is recommended.

Jam spacing and jam gap length are important parameters that determine the nature of fundamental traffic relationships. Generally, the predicted jam spacing was high compared with a jam spacing of 7.0 m per light vehicle observed at signalised intersections as discussed in the accompanying Research Report ARR 340. An inspection of individual vehicle data indicated that a lower-bound value of jam spacing for freeway traffic flows is probably 10 m/veh. Further investigation of the prediction of jam spacing is recommended.

NOTATIONS AND BASIC RELATIONSHIPS

a	Acceleration rate (m/s^2)
c	Average cycle time at traffic signals (seconds) $c = r + g$
g	Average effective green time at traffic signals (seconds)
h	Headway, or headway time: the time between passage of the *front* ends of two successive vehicles at an observation point (seconds/veh) (alternatively, this can be defined as the time measured between passage of the *rear* ends of two successive vehicles; with both definitions, headway is associated with the following vehicle) $h = 1 / q = L_h / v = t_o + t_s = t_g + t_v$ (q in veh/s, v in m/s) $h = 3600 / q$ (q in veh/h)
h_a	Average arrival headway (seconds/veh) measured at a point upstream of the back of queue at the arrival flow rate q_a (veh/h) $h_a = 3600 / q_a$
h_n	Minimum headway (seconds/veh) corresponding to the maximum flow rate, q_n (veh/h) $h_n = 3600 / q_n$
k	Density (concentration): number of vehicles per unit distance $k = q / v = 1 / L_h$ in veh/m, or $k = 1000 / L_h$ in veh/km
k_j	Jam density: number of vehicles per unit distance at zero speed, i.e. for a stationary queue $k_j = 1/ L_{hj}$ in veh/m, or $k_j = 1000 / L_{hj}$ in veh/km
k_n	Density (veh/km) corresponding to the maximum flow rate $k_n = q_n / v_n$ (q_n in veh/h and v_n in km/h)
L_a	Length of a road section (km or m)
L_h	Spacing (m/veh): the headway *distance* as measured between the *front* ends of two successive vehicles (sum of space length and vehicle length) (alternatively, this can be defined as the distance measured between the *rear* ends of two successive vehicles; with both definitions, spacing is associated with the following vehicle) $L_h = v\, h = v / q = L_s + L_v$ (q in veh/s, v in m/s)

L_{hj} Average jam spacing (m/veh), which is the sum of vehicle length and average space (gap) length for vehicles in a stationary queue

$$L_{hj} = L_{sj} + L_v$$

L_{hjLV} Average jam spacing for *light vehicles* (m/veh)

L_{hjHV} Average jam spacing for *heavy vehicles* (m/veh)

L_{hn} Average spacing (m/veh) at the maximum flow rate, q_n

$$L_{hn} = v_n h_n = v_n / q_n = L_{sn} + L_v \text{ (q_n in veh/s, v_n in m/s)}$$

L_p Effective detection zone length (m) (typically within $\pm$ 0.5 m of the detector loop length)

L_s Space (gap) length (m/veh): the following distance between two successive vehicles as measured between the *back* end of the leading vehicle and the *front* end of the following vehicle

$$L_s = L_h - L_v = v \, t_g$$

L_{sj} Average jam space (gap) length for vehicles (average distance between vehicles in a stationary queue) (m)

L_{sn} Space (gap) length (m/veh) corresponding to the maximum flow rate, q_n

L_v Average vehicle length (m/veh) for the actual traffic mix

$$L_v = (1 - p_{HV}) L_{vm} + p_{HV} L_{vHV}$$

L_{vHV} Average vehicle length for heavy vehicles (m/HV)

L_{vLV} Average vehicle length for light vehicles, or passenger car units (m/LV or m/pcu)

L_y Distance between two detectors (m)

m_c Delay parameter in Model 4

m_q, m_v Parameters in Model 5

n_a Number of vehicles observed in a road section of length L_a

N_i Initial queued demand as observed at the start of a flow period used in the extended form of Model 4 (vehicles)

O_s Space occupancy ratio: the *proportion of road space* (length) covered by vehicles

$$O_s = L_v / L_h = k \, L_v$$

O_t Time occupancy ratio: the *proportion of time* in an analysis period when the detector at a given point along the road is occupied by vehicles

$$O_t = t_o / h = (L_v + L_p) / L_h \text{ (with presence detection)}$$

pcu	Passenger car units (used to allow for the effect of heavy vehicles)
p_{HV}	Proportion of heavy vehicles (HVs) in the stream, e.g. $p_{HV} = 0.05$ means 5 % HVs and 95 % LVs (light vehicles, or pcus)
p_{LV}	Proportion of light vehicles in the traffic stream
q	Flow rate (veh/s or veh/h): number of vehicles per unit time passing (*arriving* or *departing*) a given reference point
q_a	Arrival (demand) flow rate (veh/s or veh/h): the average number of vehicles per unit time arriving at a point upstream of the back of queue
q_d	Departure flow rate (veh/s or veh/h)
q_n	Maximum flow rate (veh/h) (for uninterrupted traffic, this is the capacity)
q_s	Departure flow rate (veh/h) under saturated conditions
Q_e	Capacity of a lane (veh/h): maximum arrival flow rate that can be serviced *under prevailing flow conditions*
	$Q_e = s\,g\,/\,c$ for interrupted traffic (where s is in veh/h)
	$Q_e = q_n$ for uninterrupted traffic
r	Average effective red time at traffic signals(seconds)
s	Saturation flow rate at interrupted traffic facilities (veh/h)
t_g	Gap time (seconds/veh): the time between the passage of the *back* end of leading vehicle and the *front* end of the following vehicle as measured at a given point along the road, e.g. at a passage detector, or at the leading edge of a presence detector loop; this is the time taken to travel the space (gap) length, L_s
	$t_g = L_s\,/\,v = h - t_v$ (v in m/s)
t_o	Detector occupancy time (seconds/veh): duration of the period when the detection zone is *occupied* by a vehicle (sum of the vehicle passage time and the time to travel the detection zone length) at speed v (m/s)
	$t_o = h - t_s = t_v + L_p\,/\,v = (L_v + L_p)\,/\,v$
t_{on}	Occupancy time (seconds/veh) corresponding to the maximum flow rate, q_n
t_s	Space time (seconds/veh): the duration of the time between the detection of two consecutive vehicles when the presence detector loop is not occupied (this is equivalent to the gap time less the time taken to travel the effective detection zone length, L_p) at speed v (m/s)
	$t_s = h - t_o = t_g - L_p\,/\,v = (L_s - L_p)\,/\,v$
t_{sn}	Space time (seconds/veh) corresponding to the maximum flow rate, q_n
	$t_{sn} = h_n - t_{on}$

t_v — Vehicle passage time (seconds/veh): time it takes for the vehicle length (from front end to back end) to pass a given point at speed v (m/s) (this can be considered to be the occupancy time, $t_o \approx t_v$, when the detection zone length is negligible, $L_p \approx 0$).

$$t_v = L_v / v$$

T_f — Duration of a demand flow (analysis) period (hours)

T_L — Travel time between two VDAS passage detectors for the leading end of a vehicle (seconds)

T_T — Travel time between two VDAS passage detectors for the trailing end of a vehicle (seconds)

u — Green time ratio for interrupted traffic facilities

$$u = g / c$$

v — Speed (m/s or km/h): distance travelled per unit time

$$v = L_h / h = q\, L_h = q / k$$

v_a — Average speed (m/s or km/h)

v_{ac} — Average approach cruise speed (km/h) as measured at a point upstream of the back of queue under uninterrupted conditions: this is the average cruise speed of the arrival stream at arrival flow rate q_a

v_d — Average speed for interrupted traffic including the effect of delays at traffic interruptions (km/h)

v_f — Free-flow (zero-flow) speed in the uninterrupted speed-flow model (average speed under near-zero flow conditions) (km/h)

v_L — Speed based on time measurements for the leading end of the vehicle (km/h)

$$v_L = 3.6\, L_y / t_L$$

v_n — Speed at the maximum flow rate, q_n (km/h)

v_{of} — Free-flow (zero-flow) speed for interrupted traffic (average speed under near-zero flow conditions) (km/h)

v_Q — Average travel speed when the demand flow rate equals the capacity for interrupted traffic (km/h)

v_T — Speed based on time measurements for the trailing end of the vehicle (km/h)

$$v_T = 3.6\, L_y / t_T$$

v_u Average *uninterrupted* travel speed (km/h or m/s) (at the signal stop line, this is the average speed during the unsaturated part of the green period) ($v_u = v_{ac}$)

x Degree of saturation: the ratio of arrival (*demand*) flow rate to capacity

$x = q_a / q_n$ for uninterrupted traffic ($Q_e = q_n$)

$x = q_a / Q_e$ for interrupted traffic

z A parameter used in Model 4

$z = x - 1 + 2 N_i / (q_n T_f)$

($z = x - 1$ for $N_i = 0$)

1   INTRODUCTION

This research report presents findings of a study of fundamental characteristics of freeway traffic flows. The study was carried out under the ARRB TR research project RC 7082 *Reassessment of Fundamental Speed-Flow Relationships for Freeway Traffic Control* funded by AUSTROADS (Project NRUM 9712).

The results given in earlier reports WD R 98/044 (Akçelik 1998), WD R 98/045 (Akçelik, Roper and Besley 1998) and Contract Report RC 7082 (Akçelik, Roper and Besley 1999) are superseded by this research report.

The aim of the project was to carry out field measurements of freeway traffic flow characteristics under both free-flowing (unsaturated) and congested (saturated) conditions, and to develop improved analytical models to describe the relationships between the traffic flow parameters. Such analytical models of freeway traffic flow are useful for practical applications including:

(i)   design of freeway facilities with adequate capacity and level of service, and

(ii)   development of improved methods for the purposes of

- incident detection,
- ramp metering,
- traffic monitoring, and
- driver information.

ARRB TR research project RC 7057 *Fundamental Relationships for Adaptive Control* funded by the Roads and Traffic Authority of New South Wales (RTA NSW Project No. 63521/8) focused on traffic flow relationships at traffic signals, described in full in Research Report ARR 340 "Fundamental Relationships for Traffic Flows at Signalised Intersections" (Akçelik, Besley and Roper 1999). The two projects for freeways and traffic signals represent uninterrupted and interrupted flow conditions, respectively. They have much in common in terms of establishing data collection and analysis methodology for fundamental traffic flow relationships and developing analytical models.

Information on freeway traffic flow characteristics has been largely based on overseas work, and mainly US publications, e.g. the Highway Capacity Manual (TRB 1998). The aim of the ARRB TR freeway research project was to study the applicability of overseas data and models to Australian conditions, and to extend the work on fundamental relationships for traffic flow at traffic signals to better understand current traffic flow characteristics on freeways.

An introduction to the fundamental traffic flow parameters and their relationships is presented in *Section 2*. The traffic flow parameters considered include flow rate, speed, density, headway, occupancy time, space time, gap time, vehicle passage time, spacing, gap (space) length, vehicle length, and time and space occupancy ratios. The relation between fundamental parameters for interrupted and uninterrupted traffic flows is explained.

A detailed discussion of the data collection and analysis methods is presented in *Section 3*. The basis of the work was the measurement of headway and speed for *individual vehicles* at a point on the freeway. Other traffic parameters such as flow rate, density, spacing and occupancy ratios were derived from these parameters using the equations given in *Section 2*. The method used for collecting data on the Eastern Freeway in Melbourne is described, and issues related to data collection and analysis are discussed in detail.

A permanent VicRoads two-loop presence detection system was linked to VDAS traffic counters for the purpose of this research. Travel speeds were also measured during the survey period using an instrumented car. Comparison of speed data from the two survey methods is presented.

To obtain parameters describing the characteristics of freeway traffic streams, VDAS data for individual vehicles were aggregated using the method described in *Section 2* for periods of 20 s, 1 min, 2 min, 5 min and 15 min. Aggregate data were always derived from individual vehicle data, not from the aggregate data for a shorter period. The data aggregation method used in this study aimed to achieve consistency between speed and all time-based and distance-based parameters.

There is a vast amount of literature on fundamental traffic relationships for uninterrupted traffic flow offering empirical as well as theoretical models based on car-following, hydrodynamic and kinetic theories of traffic flow. However, these models fail to provide a complete description of the speed, flow and density characteristics observed in practice for a full range of flow conditions.

Section 4 presents six analytical traffic flow models including single-regime and two-regime models. A single-regime model is a single function that applies to both saturated and unsaturated flow conditions. A two-regime model consists of two functions that apply to saturated and unsaturated flow conditions separately. The models are presented as speed - flow rate relationships from which other relationships such as speed - density, space time - speed and occupancy ratio - flow rate can be derived using the fundamental relationships given in *Section 2*.

Model calibration results for the Eastern Freeway site are given in *Section 5*. A constrained non-linear regression analysis method was used for model calibration. Regression analyses were carried out using data aggregated for 5-min intervals. This interval was chosen as the most appropriate interval for the purpose of evaluating alternative models. Several regression options were tried for each model with different model parameters specified. Results for all models and all regression options are presented. Graphs showing fundamental relationships for all traffic flow parameters with measured data and model predictions are presented for Model 4+5 (combination of Model 4 for unsaturated conditions and Model 5 for saturated conditions).

Section 6 summarises the main findings of the study, and recommends further research on various aspects of the results presented in this report.

2 TRAFFIC FLOW PARAMETERS AND RELATIONSHIPS

A general discussion of fundamental traffic flow parameters and their relationships is presented in this section. The traffic flow parameters considered include flow rate, speed, density, headway, occupancy time, space time, gap time, vehicle passage time, spacing, gap (space) length, vehicle length, and time and space occupancy ratios.

In *Section 2.1*, headway, occupancy time, space time, gap time, vehicle passage time, spacing, gap length, vehicle length and speed are discussed as parameters describing individual vehicle movements. This is the same material included in Research Report ARR 340 for fundamental traffic variables at the signal stop line (Akçelik, Besley and Roper 1999, Section 5).

The use of the individual vehicle parameters as aggregate (average) traffic stream parameters, and flow rate, density, time occupancy ratio and space occupancy ratio parameters are discussed in *Section 2.2*.

The relationships between the basic traffic flow parameters are discussed in *Section 2.3*. The relation between speed-flow models for interrupted and uninterrupted traffic flows is also explained in *Section 2.3*.

2.1 Basic Parameters Describing Individual Vehicle Movements

The following parameters can be used to describe road traffic flow characteristics:

(i)   Time-based traffic flow parameters: *headway (h), occupancy time (t_o), space time (t_s), gap time (t_g), vehicle passage time (t_v)*;

(ii)   Distance-based traffic flow parameters *spacing (L_h), space or gap length (L_s), vehicle length (L_v)*; and

(iii) Other traffic flow parameters that relate the time-based and distance-based parameters or are derived from them: *speed (v), flow rate (q), density (k), time occupancy ratio (O_t), and space occupancy ratio (O_s)*.

In this report, the following *units* of measurement will be used:

(i)   all time-based traffic flow parameters in seconds (s),

(ii)   all distance-based traffic flow parameters in metres (m),

(iii) speed in kilometres per hour or metres per second (km/h or m/s), flow rate in vehicles per hour or vehicles per second (veh/h or veh/s), density in vehicles per kilometre (veh/km), occupancy ratios as percentage values.

Other distance parameters such as the length of a road section (L_a) can be measured in kilometres or metres (km or m), and the duration of an analysis period (T_f) can be measured in hours or seconds (h or s).

The reader should be aware of differences between the terminology used here and those used in the literature, and should note the lack of uniformity in the use of terminology in the literature. In particular, note that:

- *headway* in this report is the "headway time",

- *headway* is sometimes used to mean "headway distance", i.e. *spacing* in this report,

- *space headway* is sometimes used to mean "spacing" (e.g. Drew 1968),

- *gap* is sometimes used to mean "headway time" (e.g. "gaps in a traffic stream" as referred to in the gap-acceptance theory),

- *gap and headway settings* used in actuated signal control terminology have very different meanings (see Akçelik 1997),

- *spacing* is sometimes used to mean "gap length" (e.g. AUSTROADS 1993),

- *concentration* is used to mean "density",

- *occupancy* is often used to mean *time occupancy ratio* (e.g Lowrie 1996), and sometimes to mean *space occupancy ratio*,

- the terms used in the US Highway Capacity Manual (TRB 1998, Chapter 2) for *time occupancy ratio* and *space occupancy ratio* are *occupancy in time* and *occupancy in space*, respectively.

Parameters *flow rate*, *density* and *occupancy ratios* are discussed in *Section 2.2* as aggregate traffic stream parameters. All other parameters are discussed in this section as measured in relation to individual vehicles, and their use as aggregate (average) traffic stream parameters is discussed in *Section 2.2*. The relationships described in this report apply to a *single lane* of traffic. Their extension to multi-lane traffic streams requires due attention to unequal lane utilisation.

The time-based, distance-based and related traffic flow parameters can be depicted by means of time-distance diagrams as seen in *Figures 2.1.1 to 2.1.4*. Constant and equal vehicle speeds are shown in these figures for the purpose of simplicity. More complicated cases that involve vehicle accelerations and different speeds for the leading and following vehicles are discussed in *Section 3*.

Figure 2.1.1 shows the time-distance traces of front and rear ends of two vehicles, and explains how traffic flow parameters headway, spacing and speed can be defined in relation to observations:

(a) at a *point* along the road length, e.g. by means of a passage detector such as a VDAS treadle (switch) detectors (Leschinski and Roper 1993; Akçelik, Besley and Roper 1999), and

(b) at an *instant* in time, e.g. by means of aerial photography.

Headway (h) is the time between passage of the front ends of two successive vehicles, and *spacing* (L_h) is the distance corresponding to the headway time, i.e. the distance between the front ends of two successive vehicles. *Space (gap) length* (L_s) is the following distance between two successive vehicles as measured between the rear end of one vehicle and the front end of the next vehicle (spacing less *vehicle length*, L_v).
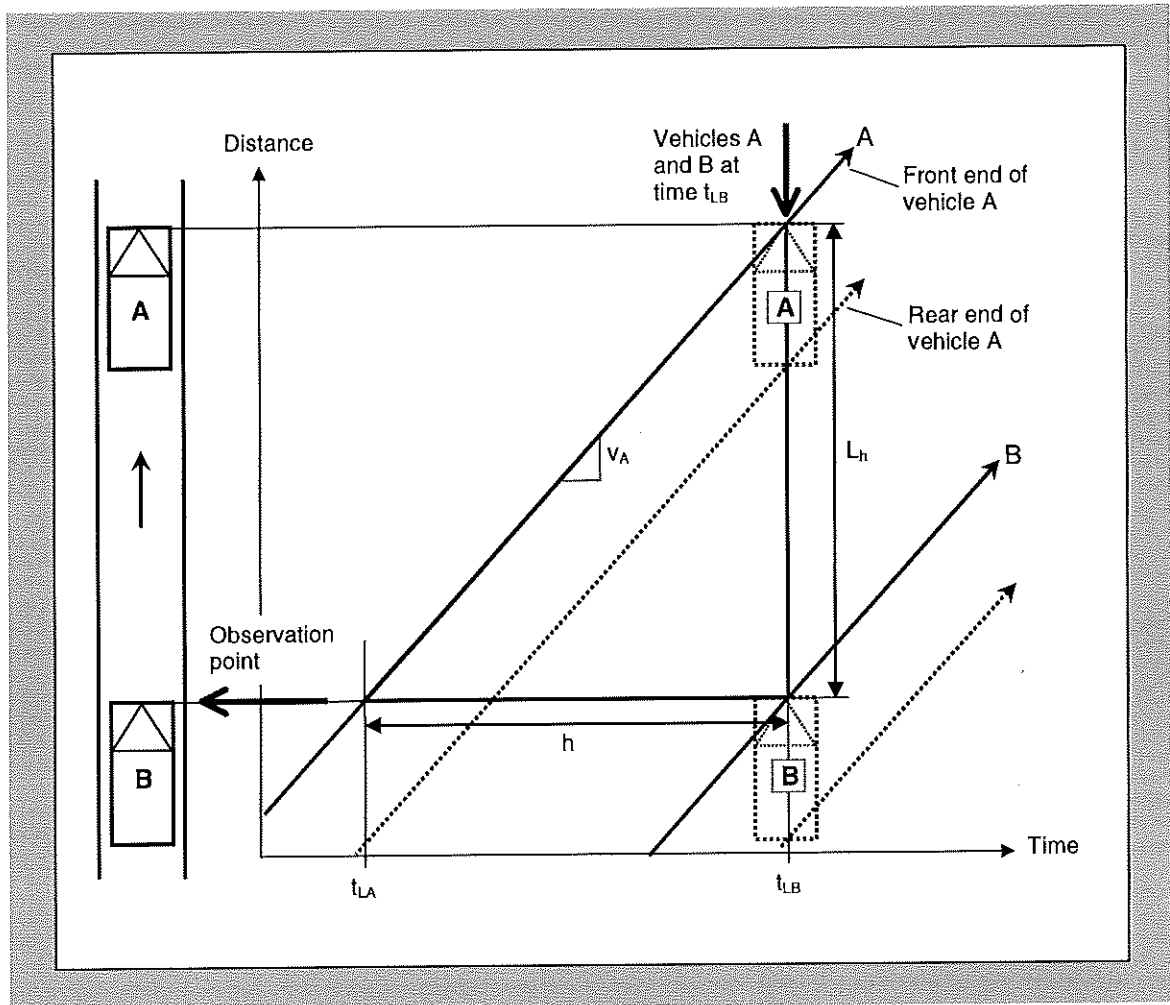
Figure 2.1.1 - Time-distance diagram showing the relationships between basic traffic flow parameters

In *Figure 2.1.1*, the headway time at an observation point along the road is measured as:

$$h = t_{LB} - t_{LA} \qquad\qquad (2.1.1)$$

where t_{LA} and t_{LB} are the passage times of the front (leading) ends of vehicles A and B at the observation point.

The positions of vehicles A and B at time t_{LB} are shown on the left-side of *Figure 2.1.1*. Spacing (L_h) is observed as the distance between the front ends of vehicles A and B at time t_{LB}.

Speed is the distance travelled per unit time. In a time-distance diagram, the slope of the time-distance trace of a vehicle is its speed. In *Figure 2.1.1*, the speed (km/h) is seen to be given by (constant speed shown):

$$v_A \quad = \quad 3.6\,L_h\,/\,h \tag{2.1.2}$$

In practice, speed is measured using an instrumented car, or externally by measuring the time to travel a fixed distance using two detectors placed at a chosen distance between them. Two passage detectors (treadle switches) placed at 3 m distance were used for surveys to determine queue discharge characteristics at traffic signals (Akçelik, Besley and Roper 1999). For freeway surveys, two presence loops of size 2 x 2 m placed at 5 m distance between the trailing edges of the two loops were used. This is discussed in detail in *Section 3*.

Figure 2.1.2 depicts observation of headway, vehicle passage time and gap time with *passage detection* at a point along the road. The term passage detection used here should be understood as a form of presence detector with a very short detection zone length, $L_p \approx 0$ (a detector strip such as the VDAS detector).

Vehicle passage time (t_v) is the time between the passage of the front and back ends of a vehicle from a given point along the road (at the passage detector in *Figure 2.1.2*).

Gap time (t_g) is the time between the passage of the rear end of one vehicle and the front end of the next vehicle, measured at a given point along the road, and is equivalent to headway time less vehicle passage time.

In *Figure 2.1.2*, the vehicle passage times for vehicles A and B, and the gap time between vehicles A and B are measured as:

$$t_{vA} \quad = \quad t_{TA} - t_{LA} \tag{2.1.3}$$

$$t_{vB} \quad = \quad t_{TB} - t_{LB} \tag{2.1.4}$$

$$t_g \quad = \quad t_{LB} - t_{TA} = h - t_{vA} \tag{2.1.5}$$

where t_{LA} and t_{TA} are the passage times of the front (leading) and rear (trailing) ends of vehicle A, and t_{LB} and t_{TB} are the passage times of the front (leading) and rear (trailing) ends of vehicle B.

The *headway* between vehicles A (leading) and B (following) is associated with the following vehicle, i.e. considered to be in front of vehicle B ($h = h_B$). From *Figure 2.1.2*, it is seen that this consists of the gap time in front of vehicle B ($t_g = t_{gB}$) and the passage time of vehicle A ($t_v = t_{vA}$). From *Equation (2.1.5)*:

$$h \quad = \quad t_{vA} + t_{gB} \tag{2.1.6}$$

Similarly, *spacing* is associated with vehicle B (i.e. in front of vehicle B). From *Figure 2.1.2*, it is seen that the spacing measured at time t_{LB} consists of the space (gap) length in front of vehicle B ($L_s = L_{sB}$) and the length of vehicle A ($L_v = L_{vA}$):
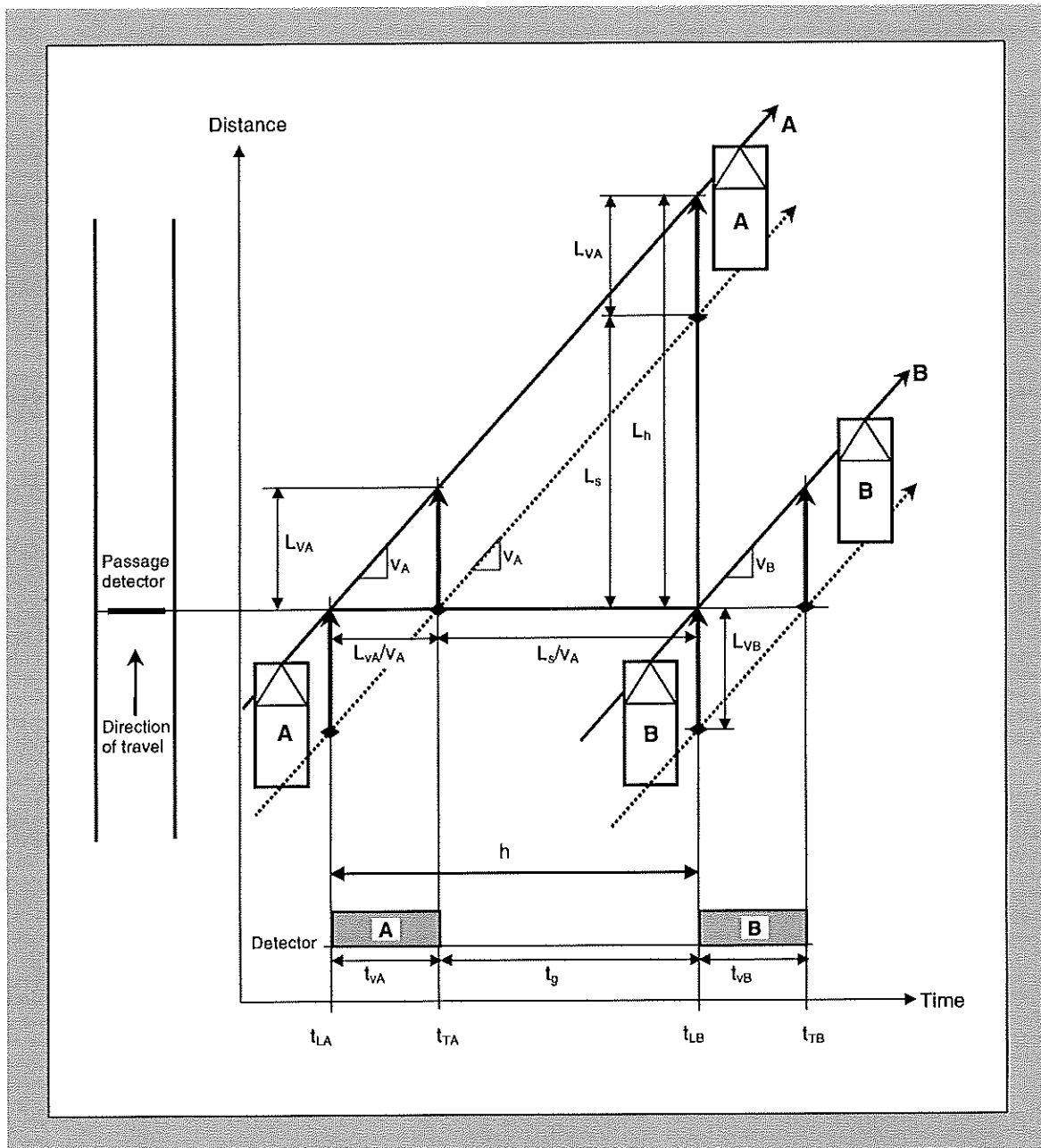
$$L_h \quad = \quad L_{vA} + L_{sB} \tag{2.1.7}$$

*Figure 2.1.2 - Time-distance diagram explaining the observation of
traffic flow parameters with passage detection (constant speeds)*

These relationships are consistent in terms of the use of speed associated with vehicle A (v_A in km/h from *Equation 5.2*) to calculate the passage time of vehicle A (t_{vA}) and the gap time in front of vehicle B (t_{gB}):

$$t_{vA} = 3.6 \, L_{vA} / v_A \qquad (2.1.8)$$

$$t_{gB} = 3.6 \, L_{sB} / v_A \qquad (2.1.9)$$

From *Equations (2.1.2), (2.1.7) and (2.1.8)*:

$$h = 3.6 \, (L_h / v_A) = 3.6 \, (L_{vA} + L_{sB}) / v_A \qquad (2.1.10)$$
$$= t_{vA} + t_{gB}$$

which is consistent with *Equation (2.1.6)*.

While vehicle passage time (t_v) and gap time (t_g) are measured with passage detection, the corresponding parameters measured with presence detection are occupancy time (t_o) and space time (t_s). *Figure 2.1.3* depicts observation of headway, occupancy time and space time with *presence detection* using a detector loop at a point along the roadway.

Occupancy time (t_o) starts when the front of a vehicle enters the detection zone and finishes when the back of the vehicle exits the detection zone. Thus, it is the duration of the period when the detection zone is occupied by a vehicle, and is equivalent to the sum of the vehicle passage time and the time to travel the effective detection zone length.

Space time (t_s) is the duration of the time between the detection of two consecutive vehicles when the presence detection zone is not occupied. It is equivalent to gap time less the time taken to travel the *effective detection zone length*, L_p.

In *Figure 2.1.3*, the vehicle occupancy times for vehicles A and B (t_{oA}, t_{oB}), and the space time between vehicles A and B, considered to be in front of vehicle B (t_s) are:

$$t_{oA} = t_{TA} - t_{LA} \qquad (2.1.11)$$

$$t_{oB} = t_{TB} - t_{LB} \qquad (2.1.12)$$

$$t_s = t_{LB} - t_{TA} = h - t_{oA} \qquad (2.1.13)$$

where t_{LA} is the time when the front (leading) edge of vehicle A enters the detection zone, t_{TA} is the time when the rear (trailing) end of vehicle A exits the detection zone, t_{LB} is the time when the front (leading) edge of vehicle B enters the detection zone, and t_{TB} is the time when the rear (trailing) end of vehicle B exits the detection zone.

As in the case of passage detection, it is considered that *headway* between vehicles A (leading) and B (following) is in front of vehicle B ($h = h_B$). From *Figure 2.1.3*, it is seen that this consists of the space time in front of vehicle B ($t_s = t_{sB}$) and the occupancy time of vehicle A ($t_o = t_{oA}$).

From *Equation (2.1.13)*:

$$h = t_{oA} + t_{sB} \qquad (2.1.14)$$

Equation (2.1.7) for spacing applies in this case as well ($L_s = L_{sB}$, $L_v = L_{vA}$ and $L_h = L_{vA} + L_{sB}$).
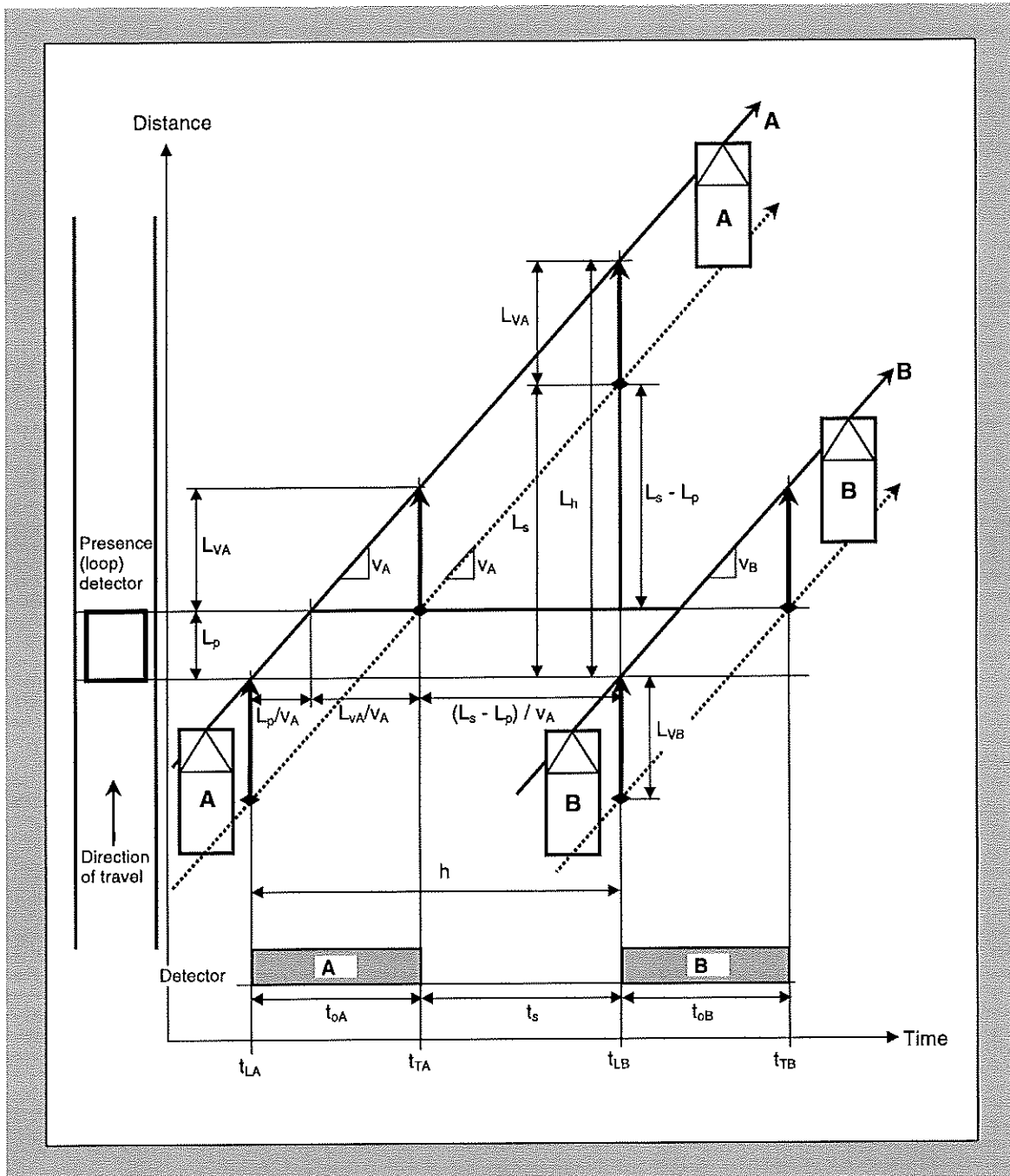
Figure 2.1.3 - Time-distance diagram explaining the observation of traffic flow parameters with presence detection (constant speeds)

The occupancy time of vehicle A (t_{oA}) and the space time in front of vehicle B (t_{sB}) are calculated using the speed of vehicle A (v_A in km/h from *Equation 2.1.2*):

$$t_{oA} \quad = \quad 3.6 \, (L_p + L_{vA}) \, / \, v_A \qquad\qquad (2.1.15)$$

$$t_{sB} \quad = \quad 3.6 \, (L_{sB} - L_p) \, / \, v_A \qquad\qquad (2.1.16)$$

From *Equations (2.1.2), (2.1.7) and (2.1.15)*:

$$
\begin{aligned}
h \quad &= \quad 3.6 \, (L_h \, / \, v_A) = 3.6 \, (L_{vA} + L_{sB}) \, / \, v_A \qquad (2.1.17)\\
&= \quad (L_p + L_{vA}) \, / \, v_A + (L_{sB} - L_p) \, / \, v_A \\
&= \quad t_{oA} + t_{sB}
\end{aligned}
$$

which is consistent with *Equation (2.1.14)*.

Figure 2.1.4 is given as a simple diagram that summarises the relationships among basic traffic flow parameters with passage and presence detection without reference to individual vehicles. In *Figure 2.1.4*, all time-based parameters are in seconds, distance based parameters are in metres, and speed is in m/s. Using speed in km/h, the basic relationships can be summarised as follows:

$$v \quad = \quad 3.6 \, L_h \, / \, h = 3.6 \, L_s \, / \, t_g \qquad\qquad (2.1.18)$$

$$h \quad = \quad 3.6 \, L_h \, / \, v = 3.6 \, (L_v + L_s) \, / \, v = t_v + t_g = t_o + t_s \qquad (2.1.19)$$

$$t_v \quad = \quad 3.6 \, L_v \, / \, v \qquad\qquad (2.1.20)$$

$$t_g \quad = \quad h - t_v = h - 3.6 \, L_v \, / \, v = 3.6 \, L_s \, / \, v \qquad (2.1.21)$$

$$t_s \quad = \quad h - t_o = t_g - 3.6 \, L_p \, / \, v = 3.6 \, (L_s - L_p) \, / \, v \qquad (2.1.22)$$

$$t_o \quad = \quad h - t_s = t_v + 3.6 \, L_p \, / \, v = 3.6 \, (L_p + L_v) \, / \, v \qquad (2.1.23)$$

$$L_h \quad = \quad L_v + L_s = h \, v \, / \, 3.6 \qquad\qquad (2.1.24)$$

$$L_s \quad = \quad L_h - L_v = t_g \, v \, / \, 3.6 = L_p + t_s \, v \, / \, 3.6 \qquad (2.1.25)$$

where

$v \quad = \quad$ vehicle speed (km/h)

$h \quad = \quad$ headway (seconds),

$t_v \quad = \quad$ vehicle passage time (seconds),

$t_g \quad = \quad$ gap time (seconds),

$t_o \quad = \quad$ occupancy time (seconds),

$t_s \quad = \quad$ space time (seconds),

$L_v \quad = \quad$ vehicle length (m),

$L_h \quad = \quad$ spacing (m),

$L_s \quad = \quad$ space (gap) length (m), and

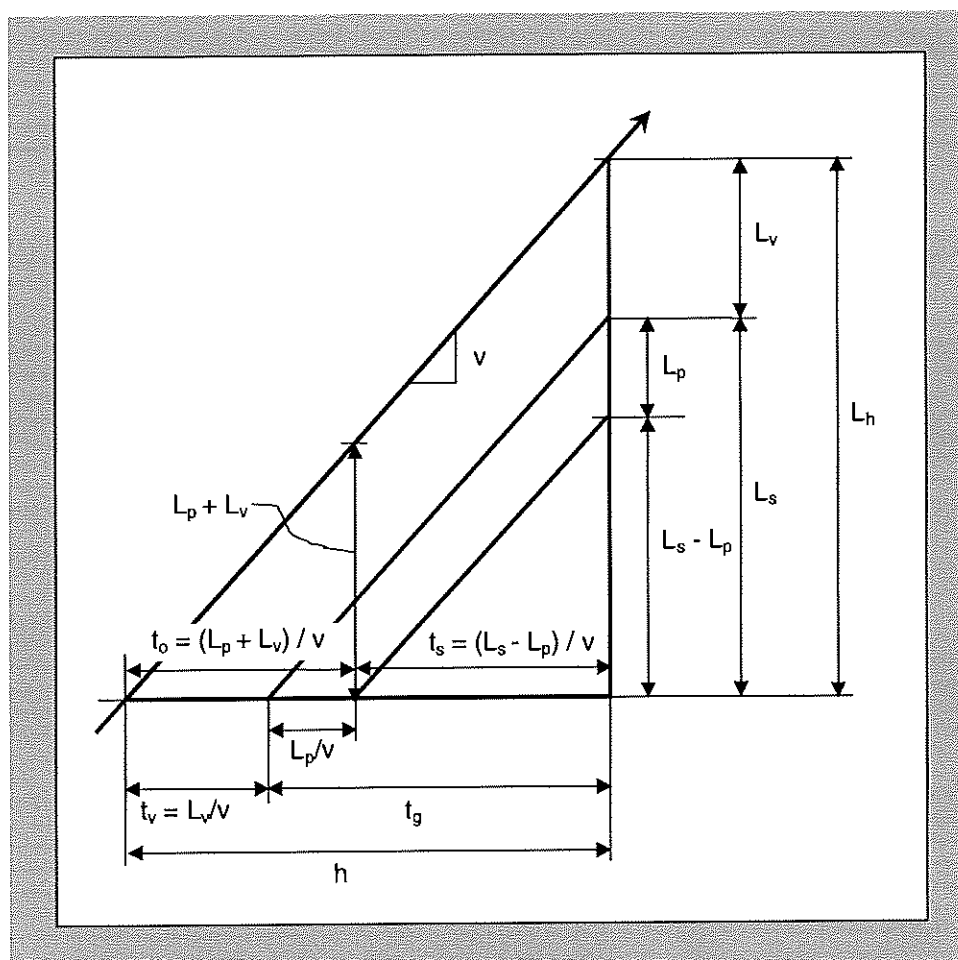$L_p \quad = \quad$ detection zone length (m).

Figure 2.1.4 - Simple diagram showing the relationships among basic traffic flow parameters

Figure 2.1.5 presents another summary of the basic concepts and parameters in presence and passage detection. It was developed from Figure 7.1 of AUSTROADS (1993).

Figure 2.1.5 is based on the definition of headway and spacing from the *front* of the leading vehicle to the *front* of the following vehicle. For completeness, *Figure 2.1.6* is given to show the relationships when headway and spacing are defined and measured from the *back* of the leading vehicle to the *back* of the following vehicle.

In this report headway and spacing parameters are defined and measured in accordance with the method summarised in *Figure 2.1.5.*

A discussion of the aggregation of individual vehicle headway, occupancy time, space time, gap time, vehicle passage time, spacing, space (gap) length, and speed parameters to represent average values for a continuous traffic stream is given in *Section 2.2.*

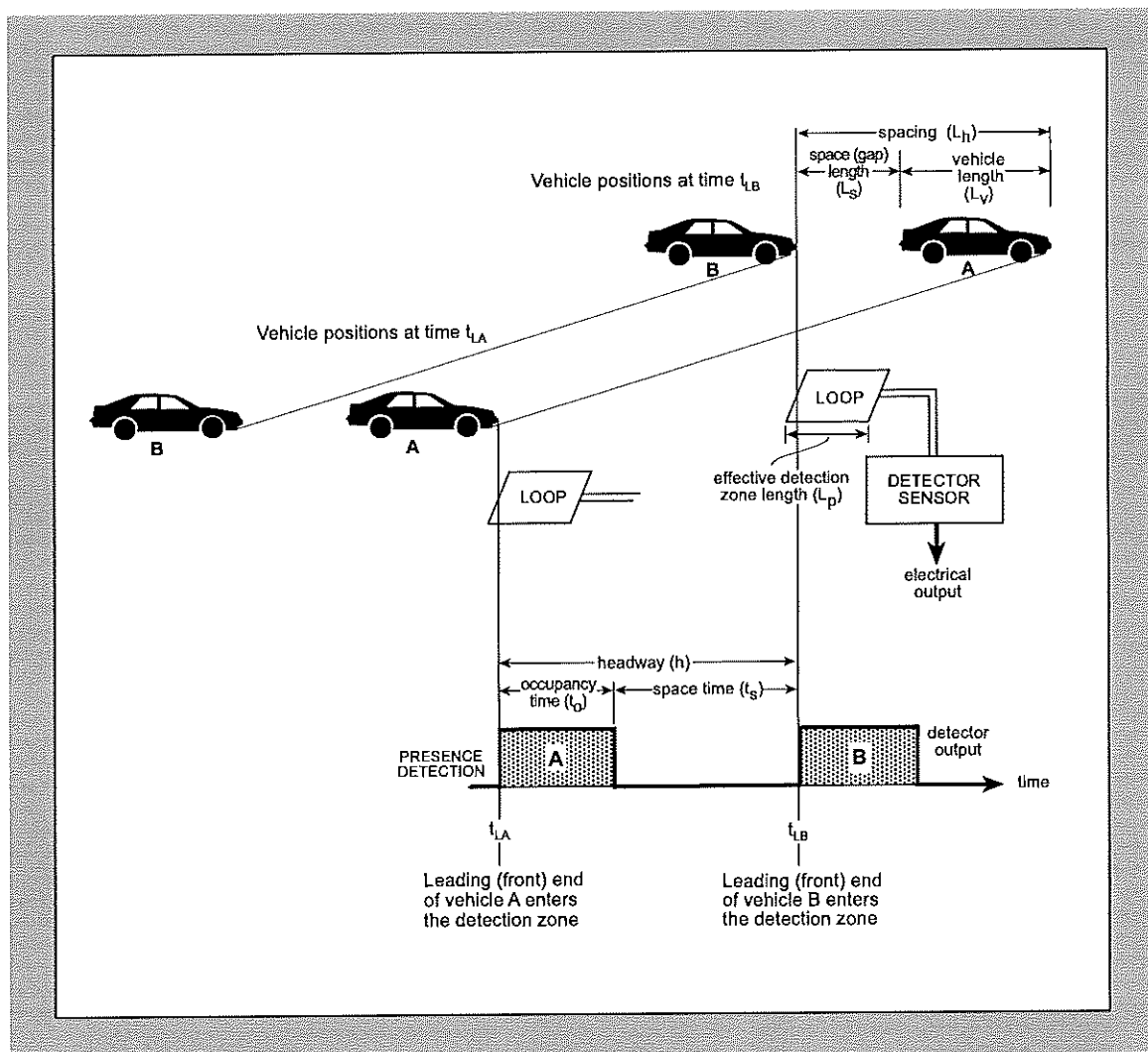*Figure 2.1.5 - Traffic flow parameters and basic relationships in presence and passage detection: headway and spacing defined from the front of the leading vehicle to the front of the following vehicle*
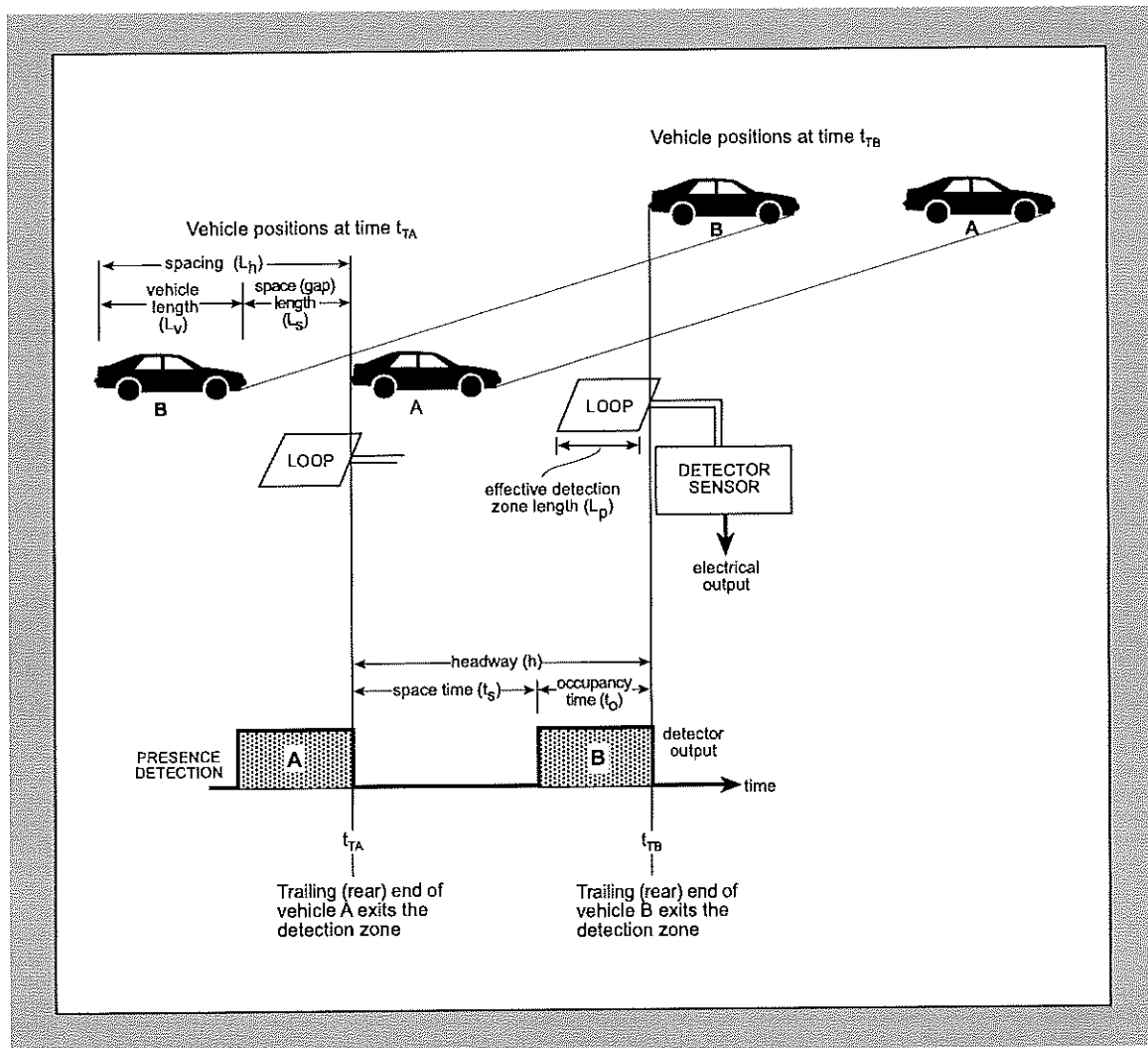
Figure 2.1.6 - Traffic flow parameters and basic relationships in presence and passage detection: headway and spacing defined from the back of the leading vehicle to the back of the following vehicle

2.2 Basic Parameters Describing Traffic Stream Characteristics

Headway (h), occupancy time (t_o), space time (t_s), gap time (t_g), vehicle passage time (t_v), spacing (L_h), gap length (L_s), vehicle length (L_v) and speed (v) as individual vehicle parameters have been discussed in *Section 2.1*. Their use as aggregate (average) traffic stream parameters is discussed in this section. Parameters *flow rate, density* and *occupancy ratios* are introduced as aggregate traffic stream parameters, and their relation to other parameters is discussed.

Parameter values measured for individual vehicles need to be aggregated for use as average values representing a traffic stream during a specified analysis period (duration T_f in hours) or in a specified section of road (length L_a in km). Simple arithmetic averages cannot be used for this purpose in all cases. Furthermore, summation of some parameter values for all vehicles observed during an analysis period or in a road section may not correspond to the duration of the analysis period or the length of the road section. This is more important when the analysis period is short. The aggregation method must produce average values of time-based and distance-based parameters and speed that match. It is also necessary to consider the issue of vehicle mix (different vehicle types).

The following method is recommended for aggregating parameters measured for individual vehicles in order to obtain average values of parameters for traffic stream. The method aims to achieve consistency between all parameters involved so that meaningful relationships are obtained between traffic stream parameters.

Average Headway, Occupancy and Space Times, Vehicle Passage and Gap Times

Calculate the average headway for the traffic stream as a simple arithmetic mean from

$$h = (\Sigma h_j) / (n - 1) \tag{2.2.1}$$

where h_j is the headway measured for *jth* vehicle, n is the number of vehicles observed during the analysis period, and summation is for (n-1) vehicles since a headway is not measured for the first vehicle (in other words, (n-1) is the number of headways).

The sum of individual vehicle headways is less than the duration of the analysis period, $\Sigma h_j < T_f$ because (i) the headway for the first vehicle in the analysis period is not defined and (ii) the last vehicle arrives sometime before the end of the analysis period. Furthermore, the difference may increase if some headway values are eliminated as bad data without reducing the number of vehicles observed (i.e. n should be decreased according to the number of data points eliminated). This is further discussed in relation to the calculation of the flow rate parameter.

Average values of occupancy time, space time, vehicle passage time and gap time should be used for (n-1) vehicles used in the calculation of the average headway from *Equation (2.2.1)*. For presence detection:

$$t_o = (\Sigma t_{oj}) / (n - 1) \tag{2.2.2}$$

$$t_s = (\Sigma t_{sj}) / (n - 1) \tag{2.2.3}$$

For passage detection:

$$t_v = (\Sigma t_{vj}) / (n - 1) \tag{2.2.4}$$

$$t_g = (\Sigma t_{gj}) / (n - 1) \tag{2.2.5}$$

Note that the average values of headway, occupancy time, space time, vehicle passage time and gap time satisfy the basic relationships given in *Section 2.1*, i.e. $h = t_o + t_s = t_v + t_g$.

Average Speed

Average speed for the traffic stream should be calculated as a space-mean speed:

$$v = (n - 1) / \Sigma (1/v_j) \tag{2.2.6}$$

where v_j is the average speed measured for *jth* vehicle, n is the number of vehicles observed during the analysis period, and summation is for (n–1) vehicles used in determining the average headway and related traffic stream parameters using *Equations (2.2.1) to (2.2.5)*.

Equation (2.2.6) is based on measuring the travel time t_j (s) over a fixed travel distance L_a (m) to measure the speed (km/h) for individual vehicles:

$$v_j = 3.6 \, L_a / t_j \tag{2.2.6a}$$

Space-mean speed is calculated by using the total travel time and distance for (n-1) vehicles:

$$v = 3.6 (n - 1) L_a / \Sigma t_j \tag{2.2.6b}$$
$$= 3.6 (n - 1) L_a / \Sigma (3.6 \, L_a / v_j) = (n - 1) / \Sigma (1/v_j)$$

Average Spacing, Gap Length, Vehicle Length

If spacings for individual vehicles have been measured, the average spacing for the traffic stream could be calculated as a simple arithmetic mean as for the average headway from *Equation (2.2.1)*. However, if the spacing is estimated from measured headway and speed values for each vehicle, then a simple arithmetic mean of those values should not be used. In that case, the average spacing (m) for the traffic stream should be estimated from:

$$L_h = h \, v / 3.6 \tag{2.2.7}$$

where h (s) is the *average* headway for the traffic stream calculated from *Equation (2.2.1)* and v (km/h) is the *average* speed for the traffic stream calculated from *Equation (2.2.6)*.

Similar considerations apply to gap length and vehicle length parameters. Using the average values of headway (h), occupancy time (t_o), space time (t_s), vehicle passage time (t_v), gap time (t_g), speed (v) and spacing (L_h) calculated from *Equations (2.2.1) to (2.2.7)*, average gap length (L_s) and average vehicle length (L_v) can be estimated using the following equations.

For presence detection:

$$L_s = L_p + t_s \, v \, / \, 3.6 = L_p + (h - t_o) \, v \, / \, 3.6 \qquad (2.2.8)$$

$$L_v = t_o \, v \, / \, 3.6 - L_p \qquad (2.2.9)$$

For passage detection:

$$L_s = t_g \, v \, / \, 3.6 = (h - t_v) \, v \, / \, 3.6 \qquad (2.2.10)$$

$$L_v = t_v \, v \, / \, 3.6 \qquad (2.2.11)$$

In *Equations (2.2.8) to (2.2.11)*, L_p is the detection zone length (m), the average speed is in km/h, time-based parameters (h, t_s, t_o. t_g, t_v) are in seconds and distance based parameters (L_p, L_s, L_v) are in metres.

Average Flow Rate, Density and Occupancy Ratios

Flow rate (q) is the number of vehicles per unit time passing (arriving or departing) a given reference point along the road. With queuing at a bottleneck on freeways or at interrupted traffic facilities, demand flow rate can be measured as the arrival flow rate at the back of the queue.

It is recommended that flow rate (veh/h) as a traffic stream parameter is calculated using the average headway, h (s) from *Equation (2.2.1)*:

$$q = 3600 \, / \, h \qquad (2.2.12)$$

Flow rate (veh/h) based on the vehicle count during an analysis period is:

$$q = n \, / \, T_f \qquad (2.2.13a)$$

where n is the number of vehicles passing the observation point during the analysis period, and T_f is the duration of the analysis period in hours.

An average headway could be used from the flow rate based on vehicle count when individual vehicle headways are not measured directly, i.e. using:

$$h = 3600 \, / \, q = 3600 \, T_f \, / \, n \qquad (2.2.13b)$$

The headways calculated from *Equations (2.1.1) and (2.2.13b)* differ because the sum of individual vehicle headways is less than the duration of the analysis period ($\Sigma \, h_j < 3600 \, T_f$) as discussed in relation to headway calculation. Similarly, the flow rates from *Equations (2.2.12)* and *(2.2.13a)* differ since $T_f > n \, h \, / \, 3600$.

Using the average headway calculated from *Equation (2.2.13b)* may cause problems when relating the headway to other basic traffic flow parameters (speed and spacing), especially when a very short analysis (aggregation) period is used. For example, consider n = 8 vehicles with a total headway of 14.0 s measured during an aggregation period of T_f = 20 seconds. There are n – 1 = 7 headways, therefore the average headway is h = 2.0 s. The flow rate from *Equation (2.2.12)* is q = 3600 / 2.0 = 1800 veh/h (7 vehicles arriving in 14 seconds). The flow rate from *Equation (2.2.13a)* is q = 8 / (20 / 3600) = 1440 veh/h, and the corresponding headway from *Equation (2.2.13b)* is h = 3600 / 1440 = 20 / 8 = 2.5 s. Differences are reduced when longer periods are used. For example, consider n = 148 vehicles with a total headway of 294.0 s measured during an aggregation period of T_f = 300 seconds. There are n – 1 = 147 headways, therefore

the average headway is h = 2.0 s. The flow rate from *Equation (2.2.12)* is q = 3600 / 2.0 = 1800 veh/h (147 vehicles arriving in 294 seconds). The flow rate from *Equation (2.2.13a)* is q = 148 / (300 / 3600) = 1776 veh/h, and the corresponding headway from *Equation (2.2.13b)* is h = 3600 / 1776 = 300 / 148 = 2.03 s.

Thus, using the flow rate, q (veh/h) from *Equation (2.2.12)* and speed, v (km/h) from *Equation (2.2.6)*, the average spacing, L_h (m) equivalent to the value calculated from *Equation (2.2.7)* is given by:

$$L_h = 1000\,v / q \qquad (2.2.14)$$

Density (k) is the number of vehicles per unit distance along the road as measured at an instant in time. It is recommended that density (veh/km) is calculated using the average spacing, L_h (m) from *Equation (2.2.7)*:

$$k = 1000 / L_h \qquad (2.2.15)$$

Density (veh/km) based on the number of vehicles (n_a) observed in a road section of length L_a (m) at an instant in time is:

$$k = 1000\,n_a / L_a \qquad (2.2.15a)$$

Density values from *Equations (2.2.15) and (2.2.15a)* would be identical if $L_a = n_a\,L_h$ where L_h is the average vehicle spacing (m), and ($n_a\,L_h$) represents the sum of individual vehicle spacings. Because L_h is an estimated value based on summations for (n-1) vehicles observed at a point along the roadside, density values from *Equations (2.2.15) and (2.2.15a)* will differ. Density calculated from *Equation (2.2.15)* is preferred as an *estimated* value that matches other traffic stream parameters.

From *Equations (2.2.1), (2.2.14) and (2.2.15)*, flow rate (veh/h), density (veh/km) and speed (km/h) for a traffic stream are related through:

$$q = v\,k \qquad (2.2.16)$$

Time occupancy ratio (O_t) is the proportion of time in an analysis period when the passage or occupancy detector at a point along the road is occupied by vehicles. Time occupancy ratio as a percentage value can be *estimated* from:

$$O_t = 100\,t_o / h = 100\,(L_v + L_p) / L_h \quad \textit{with presence detection} \qquad (2.2.17)$$
$$\textit{subject to } O_t \leq 100\,\%$$

$$O_t = 100\,t_v / h = 100\,L_v / L_h \quad \textit{with passage detection} \qquad (2.2.18)$$

where t_o, t_v and h are the average occupancy time, vehicle passage time and headway for the analysis period (in seconds) calculated from *Equations (2.2.1), (2.2.2) and (2.2.4)*, and L_h, L_v are the average spacing and average vehicle length (m) estimated from *Equations (2.2.7), (2.2.9) and (2.2.11)*, and L_p is the detection zone length (m).

With presence detection, time occupancy ratio ("occupancy") based on measured occupancy times of individual vehicles during an analysis period, T_r (h) is usually calculated as:

$$O_t = 100\,\Sigma\,t_{oj} / (3600\,T_r) \qquad (2.2.19a)$$

where t_{oj} is the occupancy time for *jth* vehicle and the summation is for n vehicles detected during the analysis period.

The occupancy ratios from *Equations (2.2.17) and (2.2.19a)* differ since the average occupancy time and headway values used in *Equation (2.2.17)* are based on summations for (n-1) vehicles. In other words, *Equation (2.2.17)* is based on:

$$O_t = 100 (n-1) t_o / [(n-1) h] \qquad\qquad (2.2.19b)$$

If the average time occupancy ratio and headway are known, the average occupancy time and vehicle passage time can be estimated from:

$$t_o = O_t h / 100 \qquad \textit{with presence detection} \qquad (2.2.20)$$

$$t_v = O_t h / 100 \qquad \textit{with passage detection} \qquad (2.2.21)$$

If the time occupancy ratio, spacing and effective detection zone length (loop length) are known, the average vehicle length can be estimated from:

$$L_v = O_t L_h / 100 - L_p \qquad \textit{with presence detection} \qquad (2.2.22)$$

$$= O_t L_h / 100 \qquad\qquad \textit{with passage detection} \qquad (2.2.23)$$

Space occupancy ratio (O_s) is the proportion of a road section (distance) occupied by vehicles at an instant in time. The space occupancy ratio as percentage value can be *estimated* from:

$$O_s = 100 L_v / L_h \qquad\qquad (2.2.24)$$

where L_h and L_v are the average spacing and average vehicle length (m) estimated from *Equations (2.2.7), (2.2.9) and (2.2.11)*.

From *Equations (2.2.18) and (2.2.24)*, it is seen that space occupancy and time occupancy ratios are equivalent with passage detection, $O_s = O_t$.

Space occupancy ratio based on the number of vehicles (n_s) observed in a road section of length L_a (m) at an instant in time:

$$O_s = 100 n_s L_v / L_a \qquad\qquad (2.2.25)$$

where L_v is the average vehicle length for the road section (in metres).

Space occupancy ratios from *Equations (2.2.24) and (2.2.25)* would be identical if $L_a = n_s L_h$ where L_h is the average vehicle spacing (m), and $(n_s L_h)$ represents the sum of individual vehicle spacings. Because L_h is an estimated value based on summations for (n-1) vehicles observed at a point along the roadside, occupancy ratios values from *Equations (2.2.24) and (2.2.25)* will differ. Space occupancy ratio calculated from *Equation (2.2.24)* is preferred as an *estimated* value that matches other traffic stream parameters.

From *Equations (2.2.14), (2.2.16) and (2.2.24)*, space occupancy ratio (percentage), flow rate (veh/h), density (veh/km) and speed (km/h) for a traffic stream are related through:

$$O_s = L_v q / (10 v) = L_v k / 10 \qquad\qquad (2.2.26)$$

where L_v is the average vehicle length (m).

Other relationships which may be useful when the time occupancy ratio is known rather than the occupancy time are given below:

$$t_s = (1 - O_t / 100) \, h \tag{2.2.27}$$

$$L_v = (v/3.6) \, (O_t / 100) \, h - L_p \tag{2.2.28}$$

Detection Zone Length

For freeways, two 2-m loops with a distance of 3.0 m between them were used for measuring speeds, headways, occupancy times and space times (see *Section 3*). For traffic signals, the typical stop-line presence loop length used in Australia is 4.5 m. This is a single loop that does not measure the speed.

The *effective detection zone length* for presence detection is not necessarily the same as the detector loop length. Morris, Dean and Hulscher (1984) suggest that due to the *spill-over sensitivity* at the ends of the loop, the effective length of a loop is usually greater than its physical length. Figure 3 of Morris, et al. (1984) shows that excess of effective over physical length is about 0.5 m for the commonly-used symmetripole loops with high sensitivity (Leschinski 1994). Thus, for a loop length of 4.5 m, the effective detection zone length is $L_p = 5.0$ m. On the other hand, RTA NSW (1991) suggests that for a loop length of 4.5 m, the effective detection zone length is $L_p = 4.0$ m. On the basis of this information, the effective detection zone length may be considered to be in the range from (loop length − 0.5 m) to (loop length + 0.5 m).

For general analysis purposes where specific loop sensitivity information is not available, the effective detection zone length may be considered to be equal to the physical loop length as used in this report.

Different Vehicle Types

The effect of different vehicle types on traffic stream parameters can be allowed for by calculating passenger car equivalents (pce). A simple method is to classify vehicles as light vehicles (LV) and heavy vehicles (HV). The use of passenger car equivalents is a well-known method for minimum queue departure headway (or saturation flow rate) purposes. Importantly, pce values determined for a particular traffic parameter should not be used for another parameter. For example, if the average minimum queue departure headways for light and heavy vehicles are 2.0 s and 3.5 s, respectively, then the pce value for headway purposes is 3.5 / 2.0 = 1.75. On the other hand, if the average spacings in queue for light and heavy vehicles are 7.0 m and 14.0 m, respectively, then the pce value for spacing purposes is 14.0 / 7.0 = 2.0.

In the calculations relating to average traffic conditions, the *vehicle length* should represent the actual traffic composition. Where the traffic stream is represented as a mixture of light vehicles (LVs) and heavy vehicles (HVs), the average vehicle length (m/veh) can be calculated as:

$$L_v = (1 - p_{HV}) \, L_{vLV} + p_{HV} \, L_{vHV} \tag{2.2.29}$$

where p_{HV} is the proportion of heavy vehicles in the traffic stream, L_{vLV} is the average vehicle length for light vehicles (m/LV), and L_{vHV} is the average vehicle length for heavy vehicles (m/HV).

2.3 Fundamental Relationships

In the literature, speed – flow rate, speed – density and flow rate – density relationships are often discussed as the *fundamental relationships* of traffic flow. It is useful to consider relationships between other traffic stream parameters discussed in previous sections in order to understand the characteristics of traffic flows for the purpose of traffic management including traffic monitoring, driver information, ramp metering and incident detection systems.

Figures 2.3.1 to 2.3.12 present the following relationships for a freeway example:

– speed - flow rate
– spacing - speed and gap length - speed
– flow rate - density
– speed - density
– headway - speed and speed-headway
– occupancy time - speed and space time - speed
– vehicle passage time - speed and gap time - speed
– time occupancy ratio - flow rate
– time occupancy ratio - speed
– space occupancy ratio - flow rate
– space occupancy ratio - speed

The following parameters are used in the above relationships as limiting values:

(i) *maximum flow rate* q_n obtained at a traffic stream speed of v_n,

(ii) *free-flow speed* v_f obtained at near-zero flow rates under very light traffic conditions, and

(iii) *jam spacing* (or spacing in queue) L_{hj} obtained at the limit of saturated conditions when the average traffic stream speed approaches zero and the corresponding flow rate also approaches zero.

Maximum flow rate corresponds to the capacity concept. Maximum flow rate tends to be larger as the duration of the analysis (aggregation) period is decreased (see *Section 3*). *Capacity* is the maximum flow rate that can be sustained during a reasonably long period of time. The US Highway Capacity Manual (TRB 1998) uses $T_f = 15$ min for defining the capacity. The selection of the aggregation period is therefore very important in relation to design and evaluation of traffic facilities. Traffic flow conditions with demand flow rates below and above capacity are referred to as *undersaturated* and *oversaturated* conditions, respectively. This is further discussed in *Section 2.4*.

The jam spacing, L_{hj} equals the vehicle length, L_v and the average gap length in queue (or jam gap length), L_{sj}:

$$L_{hj} = L_v + L_{sj} \qquad (2.3.1)$$

It is seen from the figures that, as vehicles speed up from a stationary queue, the gap length between vehicles increases gradually, and therefore the spacing increases and the

density decreases. The corresponding flow rate increases to a maximum saturation flow value and then decreases as the speed increases towards the free flow speed.

The density that corresponds to the jam spacing is called the jam density:

$$k_j \quad = \quad 1000 \, / \, L_{hj} = \; 1000 \, / \, (L_v + L_{sj}) \qquad (2.3.2)$$

The time occupancy and space occupancy ratios at jam density are:

$$O_{tj} \quad = \quad 100 \, (L_v + L_p) \, / \, L_{hj} \quad \textit{with presence detection} \qquad (2.3.3)$$
$$\textit{subject to } O_{tj} \leq 100 \, \%$$

$$O_{tj} \quad = \quad 100 \, L_v \, / \, L_{hj} \qquad \textit{with passage detection} \qquad (2.3.4)$$

$$O_{sj} \quad = \quad 100 \, L_v \, / \, L_{hj} \qquad (2.3.5)$$

It is seen that the space occupancy ratio at jam density is always less than one ($O_{sj} < 100 \, \%$) since $L_v < L_{hj}$. On the other hand, the time occupancy ratio at jam density can reach 100 % when the detection zone length is increased: $O_{tj} = 100 \, \%$ when $(L_v + L_p) = L_{hj}$ or $L_p = L_{hj} - L_v = L_{sj}$. In fact, $O_t = 100 \, \%$ can result even when $v > 0$. The condition for this is $L_p > (L_h - L_v)$ or $L_h < (L_p + L_v)$. This also corresponds to zero space time condition as seen from *Equation (2.1.4)*.

The free-flow speed (v_f) and the ratio of the speed at maximum flow to the free-flow speed (v_n / v_f) increase as the facility type improves, e.g. freeways have higher values than signalised arterials. As a rough guide where information is not available, the speed ratio can be estimated from:

$$v_n / v_f \quad = \quad 0.05 + 0.008 \, v_f \qquad (2.3.6)$$

where the free-flow speed v_f is in km/h (e.g. v_f = 100-120 km/h for freeways, 60-90 km/h for arterials, 40-50 km/h for sub-arterials).

The parameter values corresponding to the maximum flow rate can be calculated from:

$$h_n \quad = \quad 3600 \, / \, q_n \qquad (2.3.7)$$
$$= \quad 3.6 \, L_{hn} \, / \, v_n = 3.6 \, (L_v + L_{sn}) \, / \, v_n = t_{vn} + t_{gn} = t_{on} + t_{sn}$$

$$t_{on} \quad = \quad h_n - t_{sn} = t_{vn} + L_p \, / \, v_n = (L_p + L_v) \, / \, v_n \qquad (2.3.8)$$

$$t_{vn} \quad = \quad 3.6 \, L_v \, / \, v_n \qquad (2.3.9)$$

$$t_{gn} \quad = \quad h_n - t_{vn} = \; h_n - 3.6 \, L_v \, / \, v_n = 3.6 \, L_{sn} \, / \, v_n \qquad (2.3.10)$$

$$t_{sn} \quad = \quad h_n - t_{on} = t_{gn} - 3.6 \, L_p \, / \, v_n = \; 3.6 \, (L_{sn} - L_p) \, / \, v_n \qquad (2.3.11)$$

$$L_{hn} \quad = \quad 1000 \, v_n \, / \, q_n = \; h_n \, v_n / 3.6 = \; L_v + L_{sn} \qquad (2.3.12)$$

$$k_n \quad = \quad 1000 \, / \, L_{hn} = \; 1000 \, / \, (L_v + L_{sn}) \qquad (2.3.13)$$

$$O_{tn} \quad = \quad 100 \, t_{on} \, / \, h_n \qquad (2.3.14)$$

$$O_{sn} \quad = \quad 100 \, L_v \, / \, L_{hn} \qquad (2.3.15)$$

where h_n is the minimum headway (seconds), v_n is the speed (km/h), t_{on}, t_{sn}, t_{vn}, t_{gn} are the occupancy time, space time, vehicle passage time and gap time values (seconds), L_{hn} and L_{sn} are the spacing and gap length values (m), k_n is the density (veh/km), O_{tn} and O_{sn} are the time and space occupancy ratios.

Example:

Figures 2.3.1 to 2.3.12 present the graphs depicting fundamental relationships for a freeway traffic stream using Models 4 and 5 described in *Sections 4.4 and 4.5*. The graphs are based on the following example:

v_f = 100 km/h, v_n = 80 km/h (v_n / v_f = 0.80),

q_n = 2300 veh/h

detection zone length for presence detection, L_p = 2.0 m

average vehicle length, L_v = 4.5 m

jam spacing, L_{hj} = 10.0 m/veh (jam density, k_j = 1000 / 10.0 = 100 veh/km).

Other parameters for the maximum flow point calculated from *Equations (2.3.7) to (2.3.15)* are as follows:

headway: h_n = 3600 / q_n = 3600 / 2300 = 1.565 s

occupancy time: t_{on} = 3.6 (L_v + L_p) / v_n = 3.6 x (4.5 + 2.0) / 80 = 0.293 s

space time: t_{sn} = h_n − t_{on} = 1.565 − 0.293 = 1.273 s

vehicle passage time: t_{vn} = 3.6 L_v / v_n = 3.6 x 4.5 / 80 = 0.203 s

gap time: t_{gn} = h_n − t_{vn} = 1.565 − 0.203 = 1.363 s

spacing: L_{hn} = 1000 x 80 / 2300 = 34.8 m/veh (L_{hj} / L_{hn} = 10.0 / 34.8 = 0.29)

density: k_n = 1000 / 34.8 = 28.7 veh/km (k_n / k_j = 28.7 / 100 = 0.29)

gap length: L_{sn} = L_{hn} − L_v = 34.8 − 4.5 = 30.3 m/veh

time occupancy ratio: O_{tn} = 100 x 0.293 / 1.565 = 18.7 %

space occupancy ratio: O_{sn} = 100 x 4.5 / 34.8 = 12.9 %.

At the jam density, the space occupancy ratio is O_{sj} = 100 x 4.5 / 10.0 = 45 %, and the time occupancy ratio is O_{tj} = 100 x (4.5 + 2.0)/ 10.0 = 65 %.

Model parameters (see *Sections 4.4 and 4.5*):

For Model 4, T_f = 0.25 h (15-min analysis period), and from *Equation (4.4.2)*: m_c = 0.92. For Model 5, from *Equation (4.5.4)*: m_v/m_q = 0.29.

Note that, in this example, space time is always greater than zero (t_s > 0) since L_h > (L_p + L_v) at all speeds, and the time occupancy ratio at jam density is less than 100 % since L_p < L_{hj} − L_v.
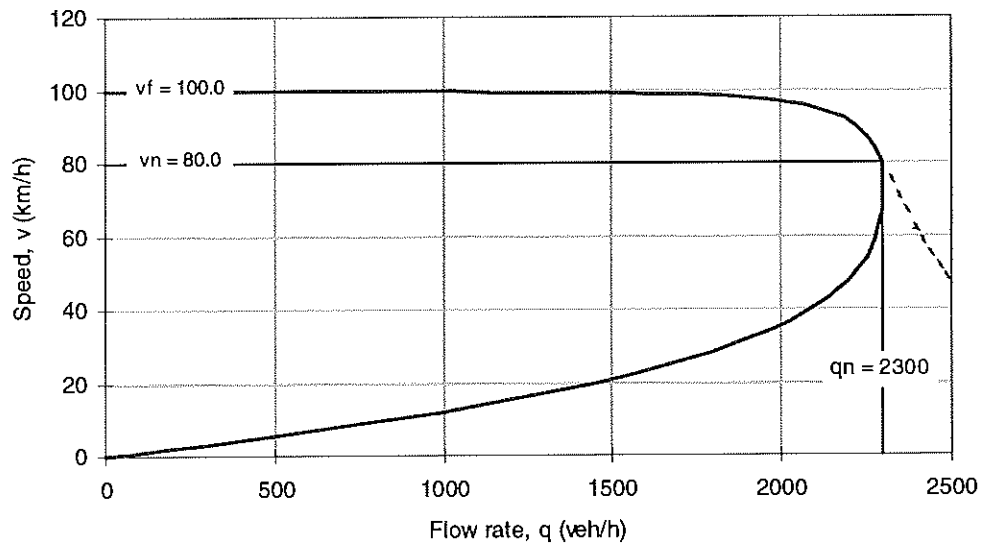
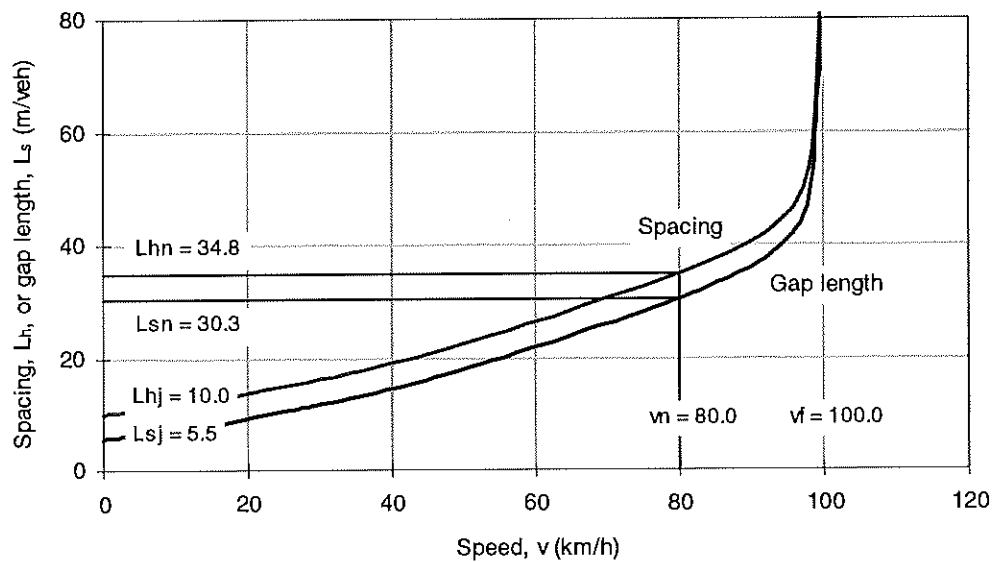*Figure 2.3.1 - Speed as a function of flow rate*

Figure 2.3.2 - Spacing and gap length as a function of speed
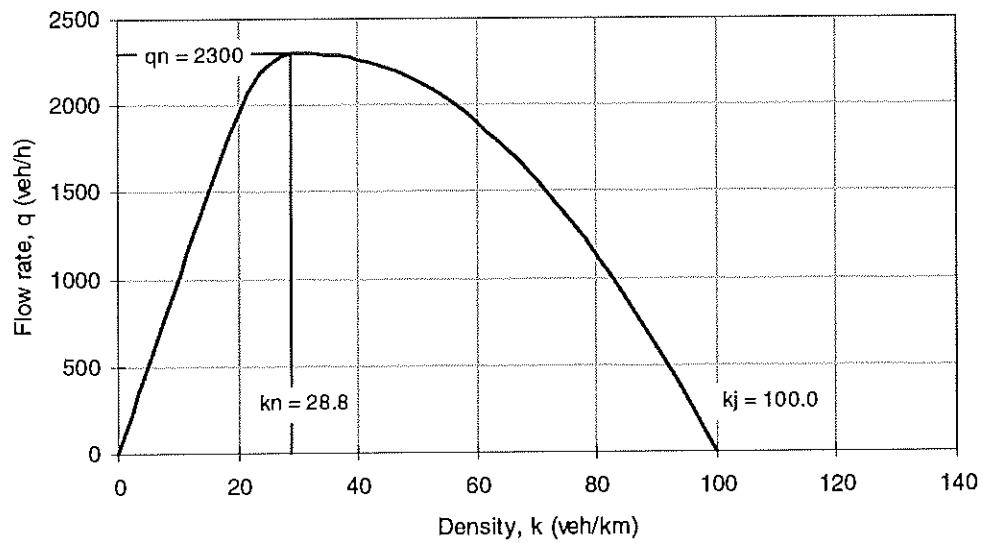
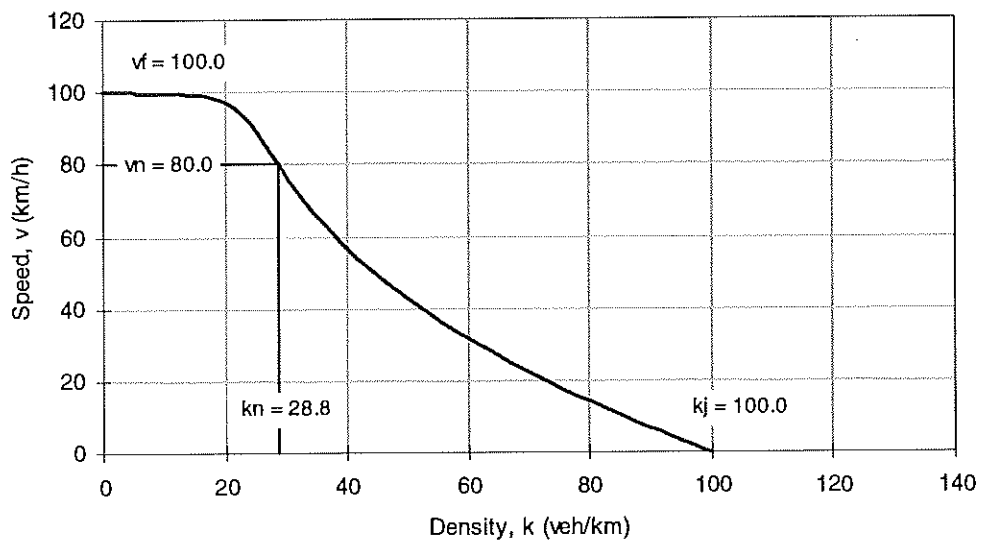*Figure 2.3.3 - Flow rate as a function of density*

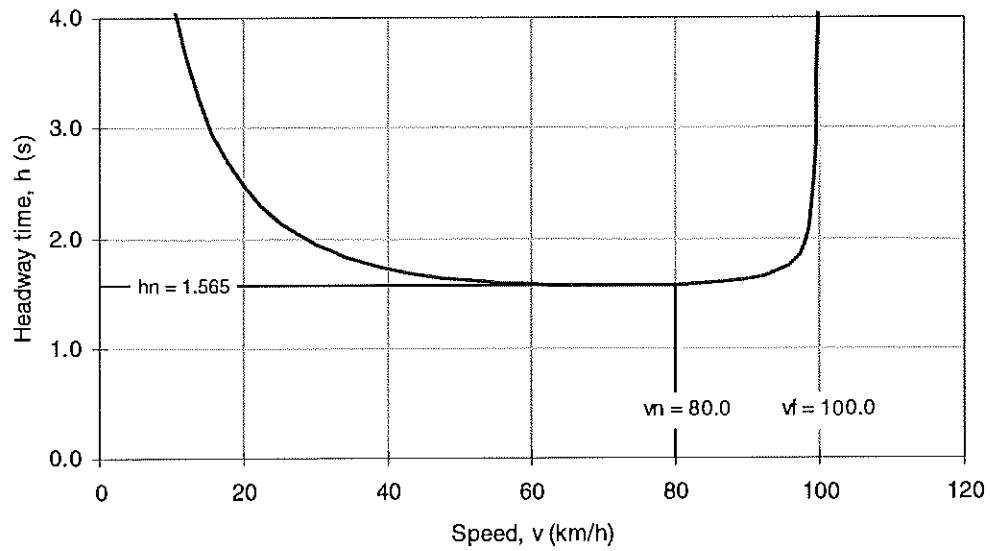
Figure 2.3.4 - Speed as a function of density

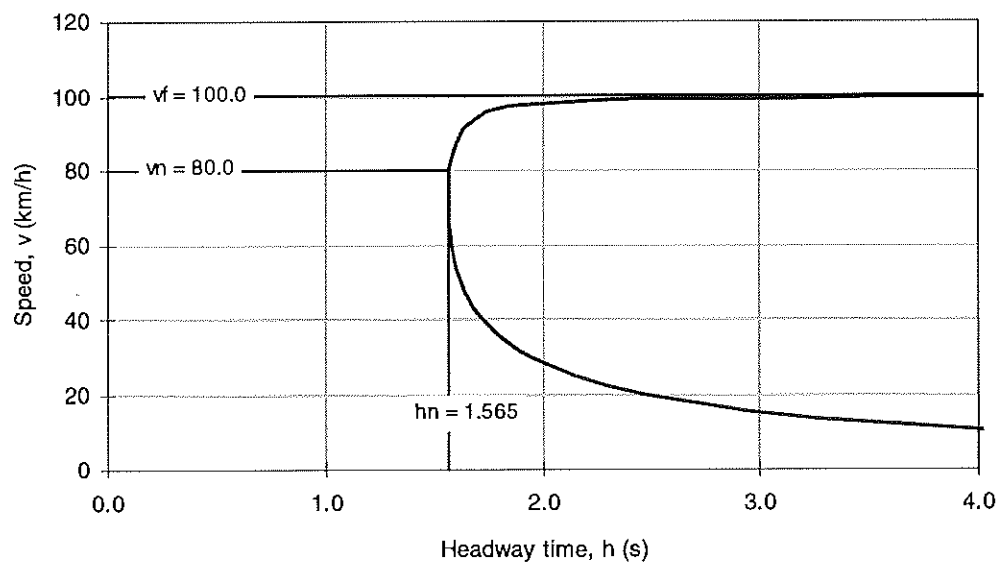*Figure 2.3.5 - Headway as a function of speed*

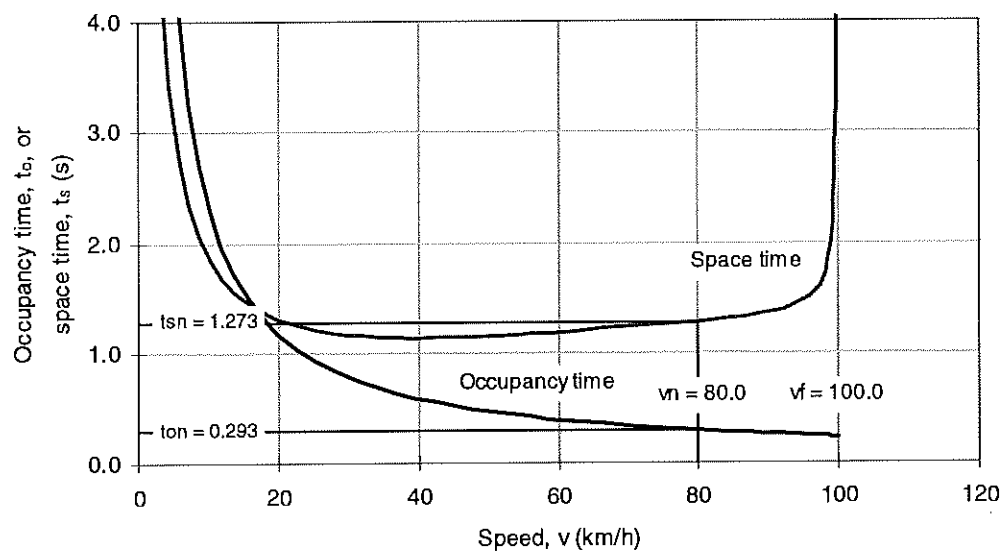*Figure 2.3.6 - Speed as a function of headway*

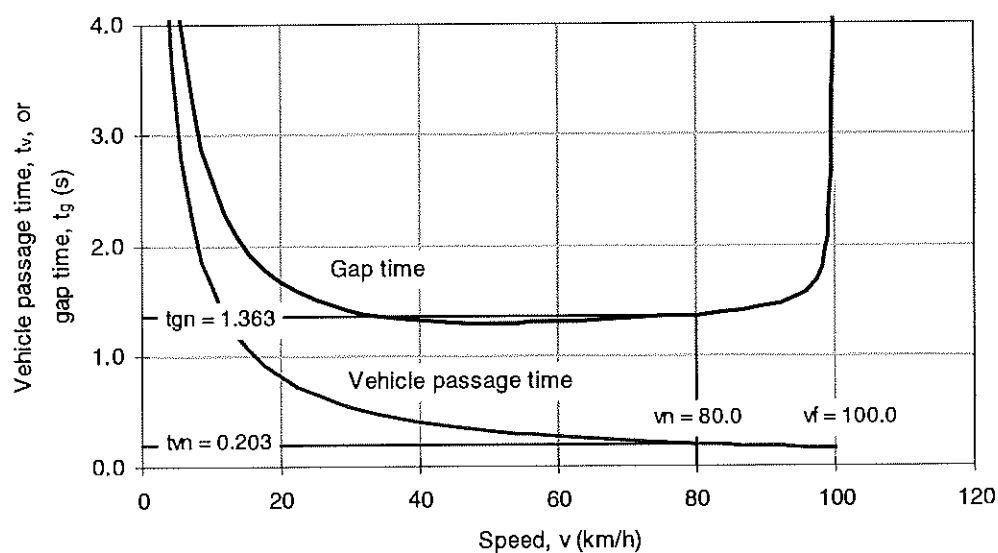*Figure 2.3.7 - Occupancy time and space time as a function of speed*

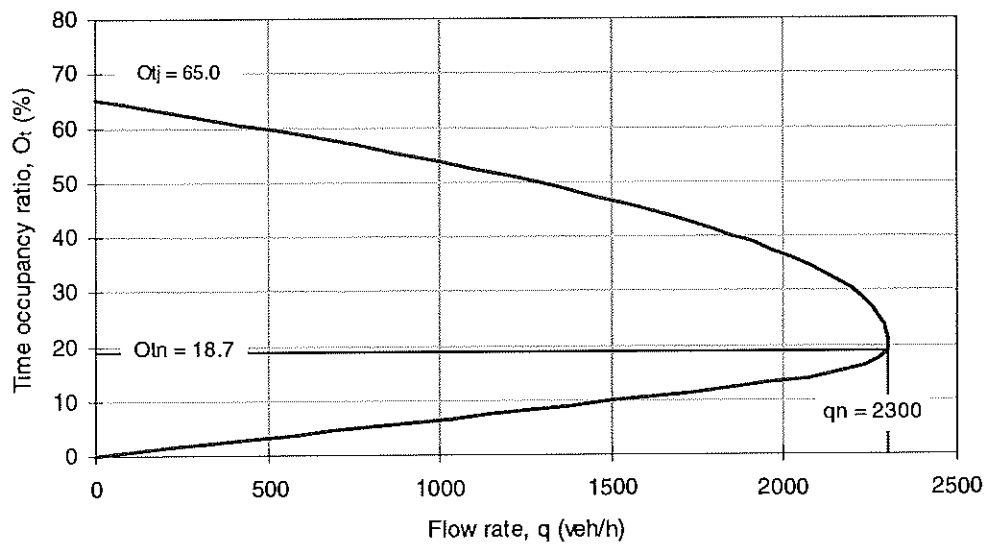*Figure 2.3.8 - Vehicle passage time and gap time as a function of speed*

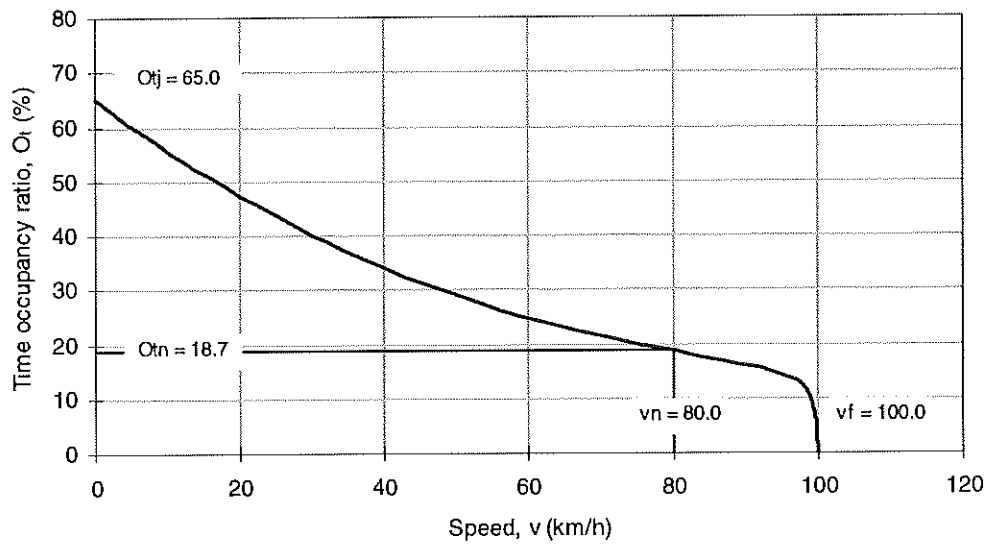*Figure 2.3.9 - Time occupancy ratio as a function of flow rate*

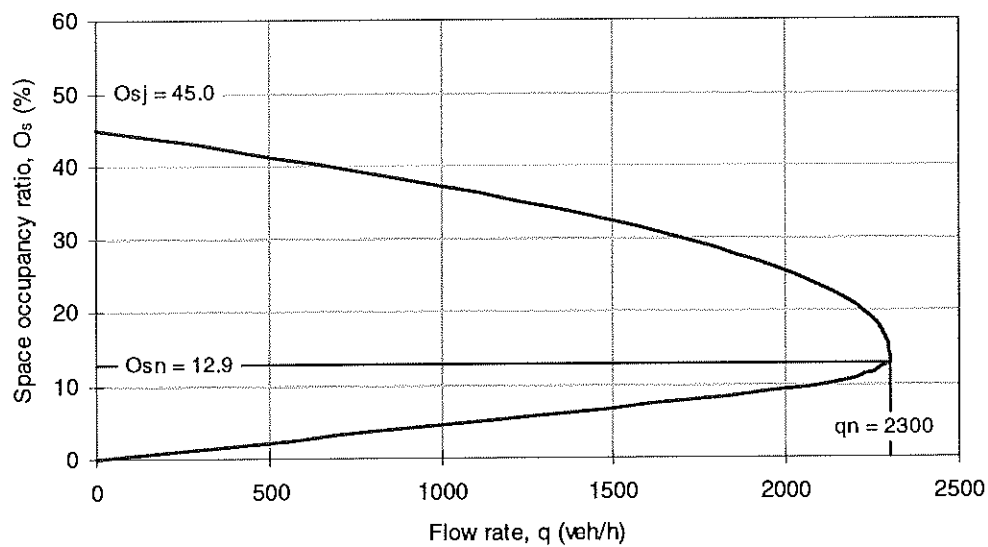*Figure 2.3.10 - Time occupancy ratio as a function of speed*

Figure 2.3.11 - Space occupancy ratio as a function of flow rate
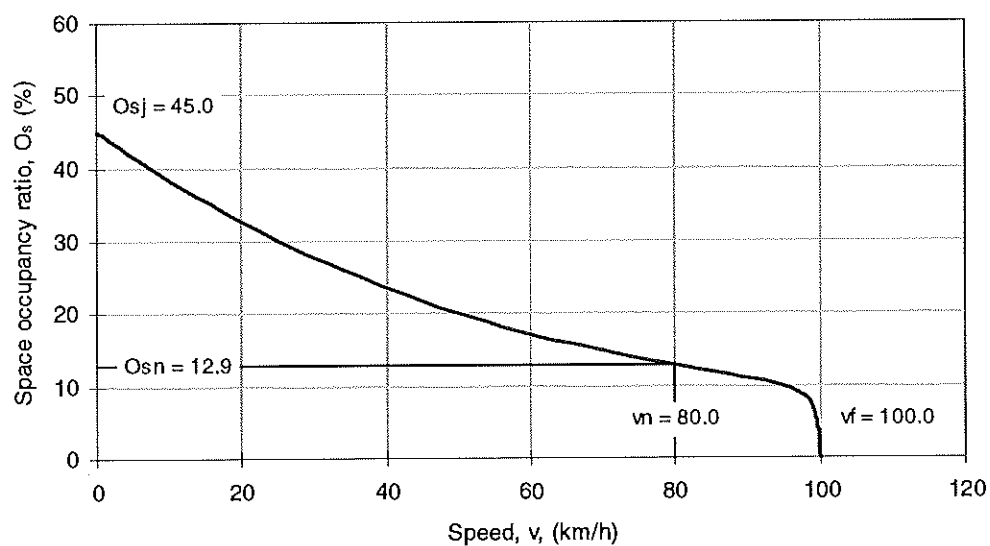

Figure 2.3.12 - Space occupancy ratio as a function of speed

2.4 Speed-Flow Relationships for Uninterrupted and Interrupted Conditions

In *Figure 2.3.1* in *Section 2.3*, the speed-flow curve extension (dotted line) for flow rates above the maximum flow rate (capacity) represents travel speeds for oversaturated (congested) conditions as predicted by Model 4 (see *Section 4.4*). The flow rates above the maximum flow rate ($q > q_n$) are the demand flow rates which are different from the flow rates measured at a point of observation (passage or presence detection). They can only be measured at the back of the queue that develops at a bottleneck point, and the corresponding speeds can only be measured by means of instrumented car surveys, i.e. by travelling through the congested section of road, starting beyond the point where the queues develop.

The relationship between headway and flow rate (*Equation 2.2.12*) is not applicable for flow rates above the maximum flow rate. If applied, such flow rates would imply headways below the minimum headway (h_n) that corresponds to the maximum flow rate. Since headway is related to vehicles passing a point, headways cannot be lower than the minimum value. In fact, when the demand flow rate exceeds the capacity, the flow rate passing a point upstream of the bottleneck point drops below the maximum flow rate. This is associated with the density increasing above the density corresponding to the maximum flow rate ($k > k_n$). This corresponds to the average vehicle spacing dropping below the spacing at the maximum flow rate ($L_h < L_{hn}$). For safe driving reasons, this results in lower speeds, and therefore increased headways. This is shown in *Figure 2.4.1* using parameter values for the example given in *Section 2.3* (*Figures 2.3.1 to 2.3.12*).

For the example in *Figure 2.4.1*, $v_f = 100$ km/h, $v_n = 80$ km/h ($v_n / v_f = 0.80$), $q_n = 2300$ veh/h, $h_n = 1.565$ s, $L_{hj} = 10.0$ m/veh, and $L_{hn} = 34.8$ m/veh. For a flow rate of $q = 1500$ veh/h, $h = 2.40$ s and two speeds are calculated. For unsaturated conditions, $v_u = 99.1$ km/h (close to v_f), and for saturated conditions, $v_s = 20.9$ km/h. Corresponding vehicle spacings are $L_{hu} = 66.0$ m and $L_{hs} = 14.0$ m. These parameters are shown in *Figure 2.4.1* based on basic relationships given in *Figure 2.1.1*. The speed-flow diagram showing these parameter values is given in *Figure 2.4.2*.

From *Equation (2.2.4)*, spacing is represented by the slope of the line that connects a point on the speed-flow rate curve to the origin ($L_h = 1000$ v / q). This is shown in *Figure 2.4.2* for the data points used in *Figure 2.4.1*.

Thus, there is a difference between the speed-flow relationships observed at a point along the road and the speed-flow relationships determined by travel time surveys through congested areas. For undersaturated conditions ($q < q_n$), the two types of speeds coincide. The difference between the two types of speed-flow relationships has not always been understood clearly (see Akçelik 1996, BTCE 1996). This is also necessary for an improved understanding of speed-flow relationships and the capacity concept for uninterrupted and interrupted traffic facilities (e.g. freeways and traffic signals, respectively).

Figures 2.4.3 and 2.4.4 show the speed-flow relationships for uninterrupted and interrupted traffic facilities, respectively.
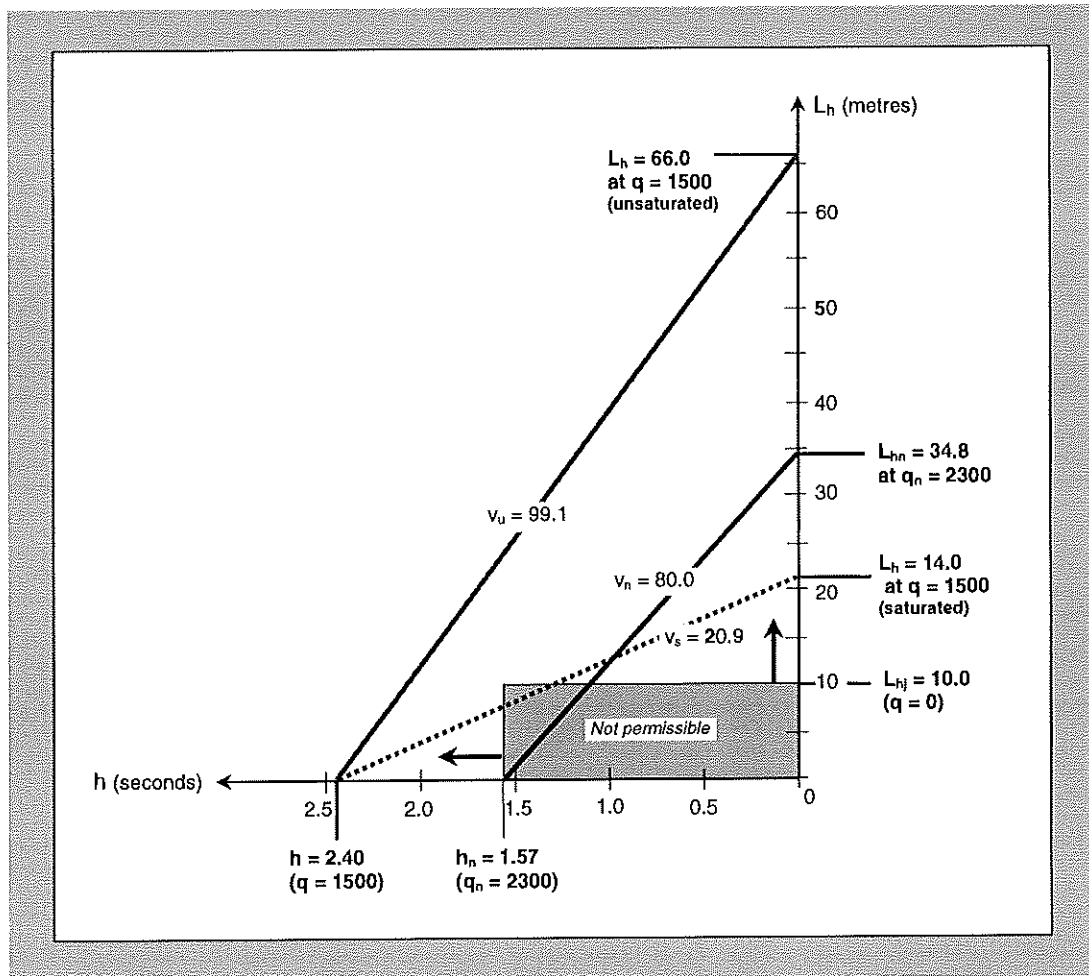
Figure 2.4.1 - Relationships between headway, speed and spacing for saturated and unsaturated conditions
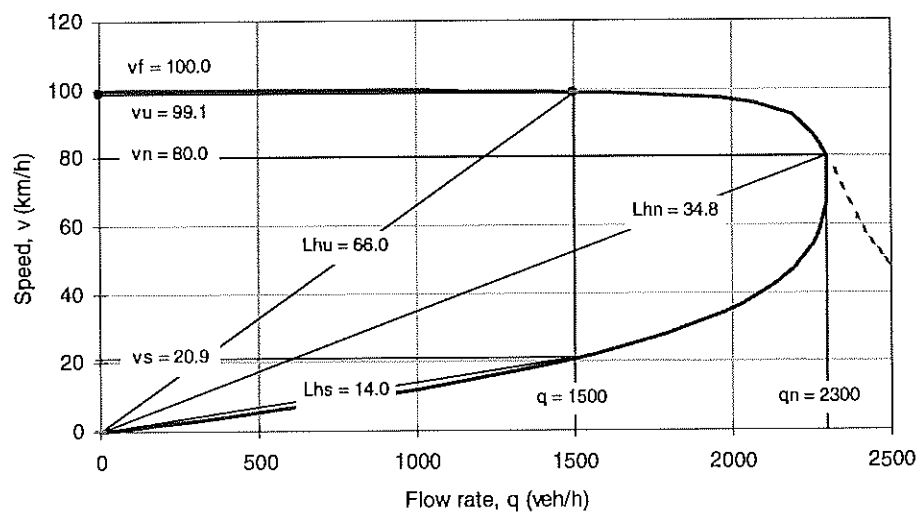

Figure 2.4.2 - Speed-flow relationship showing the parameter values for the example in Figure 2.4.1
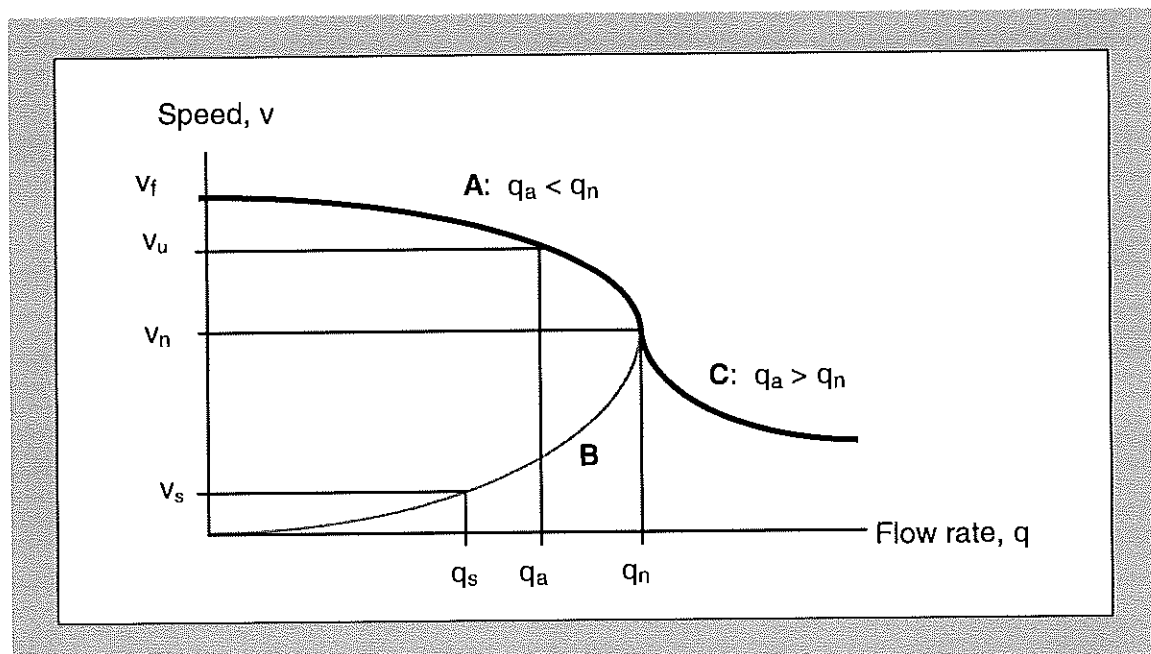
Uninterrupted Traffic Facilities

As seen in *Figure 2.4.3*, capacity for uninterrupted traffic facilities is the maximum flow rate ($Q_e = q_n$). Region A represents *arrival (demand) flow rates* below capacity ($q_a < q_n$) that are associated with uninterrupted flow speeds (v_u) between the speed at maximum flow (v_n) and the free-flow speed (v_f): $v_n < v_u < v_f$. Region B represents the saturated conditions behind a bottleneck point with flow rates below the capacity rate ($q_s < q_n$) and the saturated flow speeds below the speed at capacity flow ($0 < v_s < v_n$). In Region B, the flow rate is not the arrival (demand) flow rate. It is the flow rate based on the number of vehicles passing an observation point along the road.

Changes in conditions from Region A to Region B through the maximum flow point represent queue formation, and changes in condition from region B to Region A represent queue discharge conditions. The point on the speed-flow curve would depend on the position of the observation point relative to the bottleneck point.

While Region B represents *spot speeds* and flow rates under saturated conditions as observed at a point along the road, Region C represents *travel speeds* and demand flow rates under oversaturated conditions. For region C, the flow rate is the demand flow rate that exceeds the capacity value ($q_a > q_n$) as measured upstream of the queues that develop at the capacity point, and the speed is based on the travel time through the road section from a point upstream of the queuing area to a point past the bottleneck point. The speeds for Region C can only be measured by means of instrumented car surveys, i.e. by travelling through the congested section of road, starting beyond the point where the queues develop.

Model 4 given in *Section 4.4* is a time-dependent speed-flow relationship model that applies to both undersaturated and oversaturated conditions (Regions A and C).
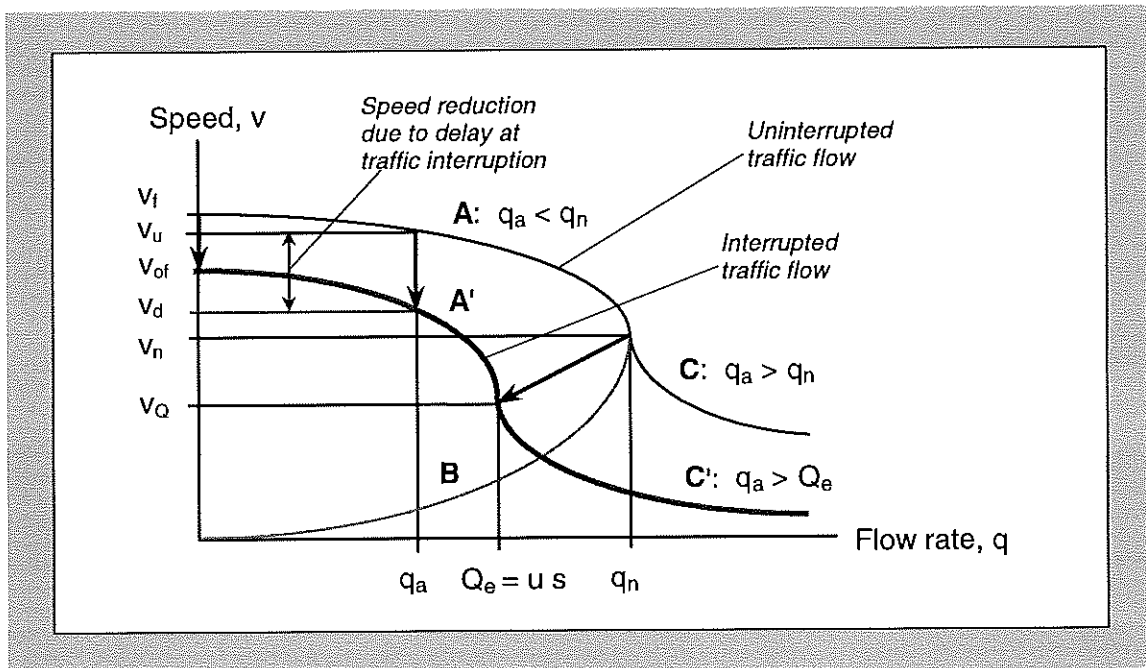

Figure 2.4.3 - Speed as a function of flow rate: uninterrupted flow conditions

Interrupted Traffic Facilities

It is seen in *Figure 2.4.4* for interrupted facilities that the capacity is obtained by reducing the *saturation flow rate*, s by the proportion of time available for departure from the queue, u. The saturation flow is used to approximate the maximum flow rate, $s \approx q_n$. Different definitions of saturation flow to approximate the maximum flow rate are used by different methods as discussed in Research Report ARR 340 (Akçelik, Besley and Roper 1999, Section 12). Thus, the capacity is $Q_e = u\ s$ (at traffic signals, u = g / c is the green time ratio where g is the effective green time and c is the cycle time).

Average travel speeds with interruption (control) delays (v_d) in Regions A' and C are seen to be reduced from the uninterrupted flow speeds (v_u) in Regions A and C due to delays experienced at the interruption point, e.g. traffic signals ($v_d < v_u$). The zero-flow speed for interrupted traffic (v_{of}) includes the free-flow travel time for uninterrupted travel (at v_f) plus the minimum delay time at the traffic interruption. The average interrupted travel speed at capacity is v_Q.

The flow rate for the congested region (B) at a signalised intersection stop line is the queue discharge flow rate (Akçelik, Besley and Roper 1999). This flow rate during the green period increases to a maximum flow rate (q_n). This consideration helps to explain the speed-flow models for uninterrupted flows as well: the flow rate measured for saturated conditions represents a reduced capacity (departure) flow *rate*, not an arrival flow rate. This is a result of vehicles (or moving queues) in the traffic stream interfering in the same way as the downstream queue interferes with the departure rate at the signal stop line, resulting in reduced vehicle spacings, speeds and flow rates. Wardrop's (1965) study of speed-flow relationships based on traffic in a circular track is interesting in this respect.

Figure 2.4.4 - Speed as a function of flow rate:
interrupted flow conditions

3 DATA COLLECTION and ANALYSIS

This section discusses the method used for collecting data on the Eastern Freeway in Melbourne. Issues related to data collection and analysis are discussed in detail. The data collection method used in this study is based on measurement of headways, speeds, occupancy and space times, and speeds *for individual vehicles* at a point on the freeway. Other traffic parameters such as flow rate, density, spacing and occupancy ratios were derived from these parameters using the equations given in *Section 2*.

A permanent VicRoads two-loop presence detection system linked to VDAS traffic counters was used for data collection. The detection system and the characteristics of the freeway survey site are described in *Section 3.1*. Data covered the full range of traffic conditions from free-flowing to congested traffic (individual vehicle speeds were in the range 10 - 132 km/h, and average speeds for 5-min intervals were in the range 36 - 103 km/h).

Travel speeds were also measured during the survey period using an instrumented car. Comparison of speed data from the two survey methods is presented in *Section 3.1*.

A detailed discussion of the data analysis method using the two-loop presence detection system is presented in *Section 3.2*.

3.1 Data Collection

The measurement site was located in the Melbourne suburb of Kew on the Eastern Freeway (F83) between Burke Road and Chandler Highway interchanges. A permanent VicRoads detection site is located approximately half way between Burke Road and Chandler Highway. Two inductive loops are installed in each lane of both carriageways at this site. At this location the freeway consists of four traffic lanes in each direction. During the morning peak survey period, the Westbound lanes have the following traffic allocations:

(i)   lanes one, two and three are general traffic lanes,

(ii) the left emergency lane may be used by buses, taxis and hire cars, and

(iii) lane 4 adjacent to the centre median operates as a transit (high occupancy vehicle) lane for vehicles with two or more occupants.

The traffic at this location regularly becomes congested during the morning peak period. The next intersection downstream from the survey site is Hoddle Street, which is a signalised intersection some 3.2 km beyond Chandler Hwy. *Table 3.1.1* shows the relative distances around the survey site.

The two-loop presence detection system is shown in *Figure 3.1.1*. VDAS traffic counters (Leschinski and Roper 1993; Akçelik, Besley and Roper 1999) were used to record individual loop actuations continuously. Data collection was carried out between 6:30 am and 10:30 am on 3 September 1998 in lane 2 of each carriageway. Data collected at this site were later processed to produce detailed information on an individual vehicle basis and on a time aggregated basis (see *Section 3.2*).

Table 3.1.1

Relative distances around the survey site (Eastern Freeway between Burke Road and Chandler Highway interchanges)

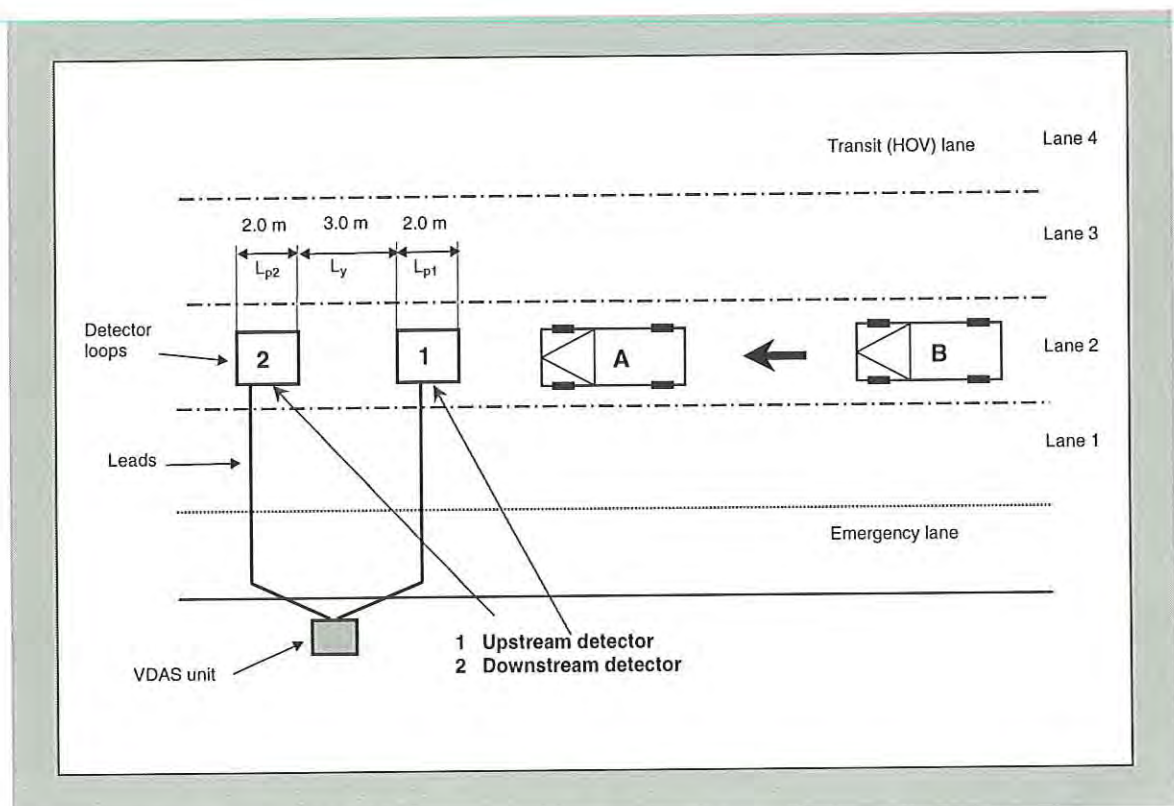| Location	Distance (km)
Burke Road	0.0
On-ramp	0.6
Loops	1.7
Off-ramp	2.7
Chandler Highway	3.2
Hoddle Street	6.4

Figure 3.1.1 - Two-loop detection system used for collecting freeway traffic flow data

Data was also collected using the ARRB TR developed *Gipsitrac* survey vehicle travelling in the traffic stream. This vehicle completed twenty-four circuits of the measurement section during the survey period.

The vehicle was driven in a manner intended to be as close as possible to the average behaviour of the surrounding traffic. The relatively short length of the measurement section did not allow the full floating car technique to be employed. This technique would require the survey vehicle to balance the number of overtaking manoeuvres with the number of vehicles which overtake it. On each circuit, the driver attempted to enter the measured lane as soon as possible after entering the freeway and then adjust speed and location to be approximately half way between the vehicle in front and the vehicle behind. Whenever possible, the speed was adjusted to be as close as possible to the vehicle in front.

The Gipsitrac vehicle records a range of road geometry and vehicle performance data parameters continuously. After processing, the reported data is provided for every ten metres of distance travelled. The primary data items used in this analysis were speed, distance and reference point entries. Reference points were entered into the data file during each circuit at the following locations.

- At the beginning of the section.

- As the vehicle entered the measured lane

- At the inductive loops.

- As the vehicle exited the measured lane.
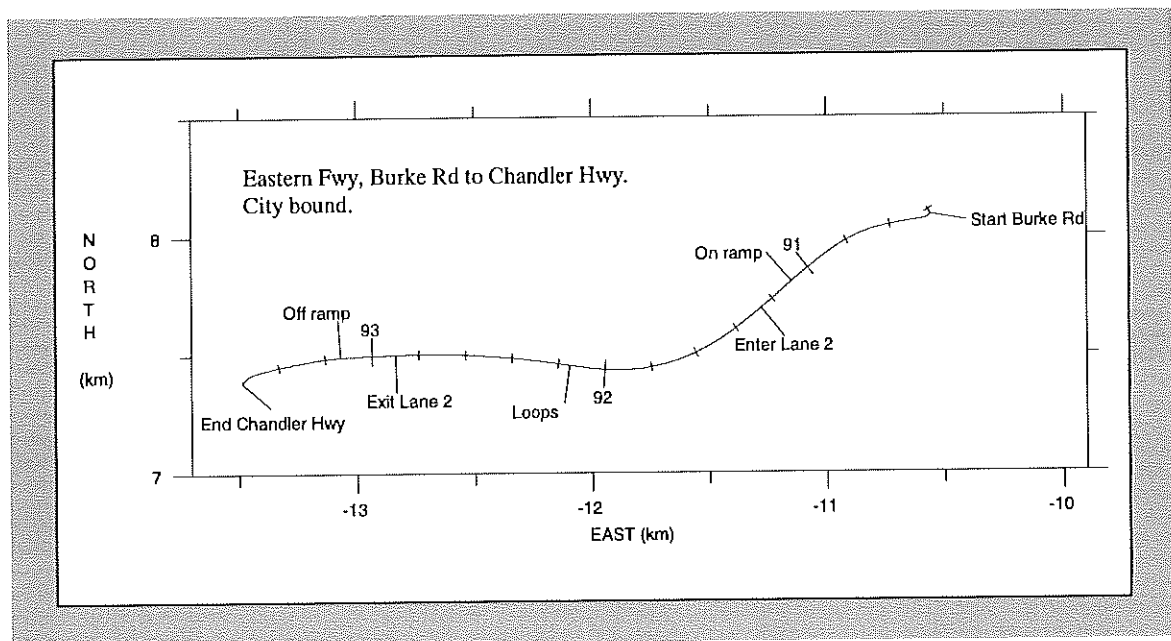
- At the end of the section.

Figure 3.1.2 - Part of map generated by the Gipsitrac vehicle showing measurement site (marked as "Loops")

During most of the circuits, the survey vehicle was able to enter lane 2 approximately 1000 m upstream of the loops and did not leave lane 2 until approximately 700 m downstream of the loops. This allowed ample time for the vehicle to adjust correctly to the behaviour of the surrounding traffic and thus provide at least 500 m of useable data either side of the loops.

Figure 3.1.2 is an extract of part of the mapping information produced by the Gipsitrac vehicle. The point where references were entered are clearly marked. Each scale marker along the vehicle route represents 200 m while the kilometre markers represent total travelled distance for the current file.

Weather conditions during the survey were fine and clear. Traffic conditions during the survey period were considered "normal" (including regular congestion) for this section of freeway. No unusual incidents were observed during the survey that would render the data abnormal. The survey began before the height of the peak period and continued until after the peak had dissipated. The congestion levels increased steadily during the early part of the survey to a point where at times the traffic briefly stopped. The extent of congestion could be observed from the survey vehicle as it returned to the start of the section along the opposite side of the freeway. It could be seen that the points where the traffic was coming to a halt varied considerably along the measured section. In most cases the heavily congested sections were surrounded by sections of more freely moving traffic both upstream and downstream of the observed congestion. The speed - distance profile produced using Gipsitrac travel speed data for the survey section (the first nine runs) shown in *Figure 3.1.3* indicates acceleration and deceleration manoeuvres involved in travelling through the survey section.
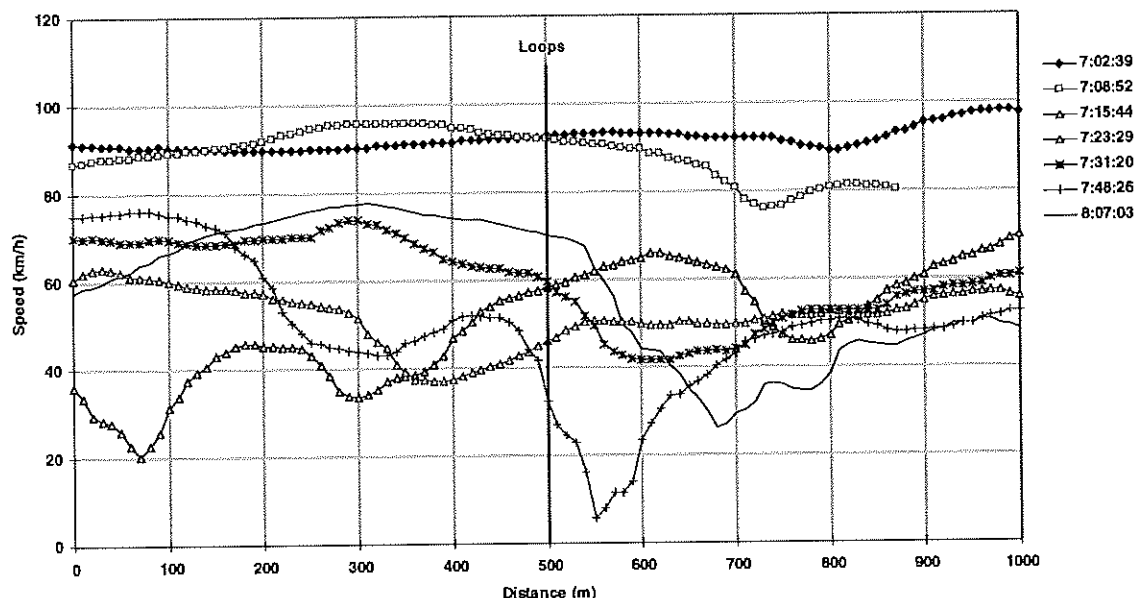

Figure 3.1.3 - Speed - distance profile produced using Gipsitrac travel speed data for the survey section on the Eastern Freeway between Burke Road and Chandler Highway

3.2 Data Analysis Using the Two-Loop Presence Detection System

The two two-loop presence detection system used for data collection on the Eastern Freeway is shown in *Figure 3.1.1* in *Section 3.1*. The method used for measuring individual vehicle parameters can be explained with the help of *Figure 3.2.1*. The reader is referred to *Figure 2.1.4* in *Section 2.1* for a simpler diagram and discussion that explains the basis of the diagram for the two-loop system.

Data for Individual Vehicles

In *Figure 3.2.1*, vehicles A and B (leading and following vehicles, respectively) are shown during acceleration rather than constant speeds over the section with loops. Acceleration and deceleration manoeuvres under saturated conditions, as seen in *Figure 3.1.3* in *Section 3.1*, result in complications in data collection. This needs to be understood in relation to the accuracy expected from data under these conditions.

The parameters related to the loop system are as follows:

L_{p1} : detection zone length for *detection zone 1*

L_{p2} : detection zone length for *detection zone 2*

L_y : distance between the two detection zones (between the trailing edge of *detection zone 1* and the leading edge of *detection zone 2*)

$L_{p1} + L_y$: travel distance between the *leading* edges of the two detection zones

$L_y + L_{p2}$: travel distance between the *trailing* edges of the two detection zones

For the two-loop system used on the Eastern freeway, detector loop sizes are the same (2 x 2 m inductive loops) with the distance between them $L_y = 3.0$ m. Assuming that effective detection zone length is the same as the loop length as discussed in *Section 2.2*, detection zone lengths are equal: $L_p = L_{p1} = L_{p2} = 2.0$ m. Thus, travel distances for the leading and trailing edges are also equal: $L_p + L_y = 5.0$ m. These values have been used in data analysis.

The raw data recorded by the VDAS unit are the base actuation times shown on the time axis at the bottom of *Figure 3.2.1*. These are:

t_{1LA} : the time when the *leading* (front) end of vehicle A *enters detection zone 1*

t_{1TA} : the time when the *trailing* (rear) end of vehicle B *exits detection zone 1*

t_{2LA} : the time when the *leading* end of vehicle A *enters detection zone 2*

t_{2TA} : the time when the *trailing* end of vehicle A *exits detection zone 2*

t_{1LB} : the time when the *leading* end of vehicle B *enters detection zone 1*

t_{1TB} : the time when the *trailing* (end of vehicle B *exits detection zone 1*

t_{2LB} : the time when the *leading* end of vehicle B *enters detection zone 2*

t_{2TB} : the time when the *trailing* end of vehicle B *exits detection zone 2*

Using the raw data, the following quantities are calculated (see *Figure 3.2.1*):

T_{LA} : travel time between the *leading* edges of *detection zones 1 and 2 for vehicle A*

$$T_{LA} = t_{2LA} - t_{1LA} \qquad\qquad (3.2.1)$$

T_{TA} : travel time between the *trailing* edges of *detection zones 1 and 2 for vehicle A*

$$T_{TA} = t_{2TA} - t_{1TA} \qquad (3.2.2)$$

T_{LB} : travel time between the *leading* edges of *detection zones 1 and 2 for vehicle B*

$$T_{LB} = t_{2LB} - t_{1LB} \qquad (3.2.3)$$

T_{TB} : travel time between the *trailing* edges of *detection zones 1 and 2 for vehicle B*

$$T_{TB} = t_{2TB} - t_{1TB} \qquad (3.2.4)$$

The speeds based on the leading edge and trailing edge measurements are calculated from (given for vehicle A):

$$v_L = 3.6\,(L_{p1} + L_y)\,/\,T_{LA} = 18\,/\,T_{LA} \qquad (3.2.5)$$

$$v_T = 3.6\,(L_y + L_{p2})\,/\,T_{TA} = 18\,/\,T_{TA} \qquad (3.2.6)$$

Headways, occupancy times and space times for individual vehicles as measured at the leading edges of detection zones 1 and 2 are calculated as follows:

$$h_{1L} = t_{1LB} - t_{1LA} \qquad (3.2.7)$$

$$h_{2L} = t_{2LB} - t_{2LA} \qquad (3.2.8)$$

$$t_{o1A} = t_{1TA} - t_{1LA} \qquad (3.2.9)$$

$$t_{o2A} = t_{2TA} - t_{2LA} \qquad (3.2.10)$$

$$t_{s1} = t_{1LB} - t_{1TA} \qquad (3.2.11)$$

$$t_{s2} = t_{2LB} - t_{2TA} \qquad (3.2.12)$$

For increased data accuracy considering possible sampling errors involved in the recording of loop data by the VDAS system, average headway, occupancy time, space time and speed values are calculated as follows:

$$h = 0.5\,(h_{1L} + h_{2L}) \qquad (3.2.13)$$

$$t_o = 0.5\,(t_{o1A} + t_{o2A}) \qquad (3.2.14)$$

$$t_s = 0.5\,(t_{s1} + t_{s2}) \qquad (3.2.15)$$

$$v_a = 0.5\,(v_L + v_T) \qquad (3.2.16)$$

As seen in *Figure 3.2.1*, $h_{1L} = t_{o1A} + t_{s1}$, $h_{2L} = t_{o2A} + t_{s2}$, and $h = t_o + t_s$.

As discussed in *Section 2.1*, the headway and space time parameters given by *Equations (3.2.13) and (3.2.15)* belong to the following vehicle (i.e. in front of vehicle B) since these parameters are observed when vehicle B arrives at the detection system. However, the occupancy time from *Equation (3.2.14)* is associated with the leading vehicle (vehicle A) as it depends on the speed of vehicle A (v_L, v_T, v_A as shown in *Figure 3.2.1*).

Note that an arithmetic mean of the leading and trailing speeds is used. Assuming a constant acceleration rate during that part of the acceleration manoeuvre of the vehicle over the two loop system, the average speed v_a represents the speed at mid-point in time between where speed v_L occurs (at time $t_{1LA} + 0.5\,T_{LA}$) and speed v_T occurs (at time $t_{1TA} + 0.5\,T_{TA}$). In aggregating data, space-mean speed values of leading and trailing speeds are obtained first, and the average speed is than found using *Equation (3.2.16)*.
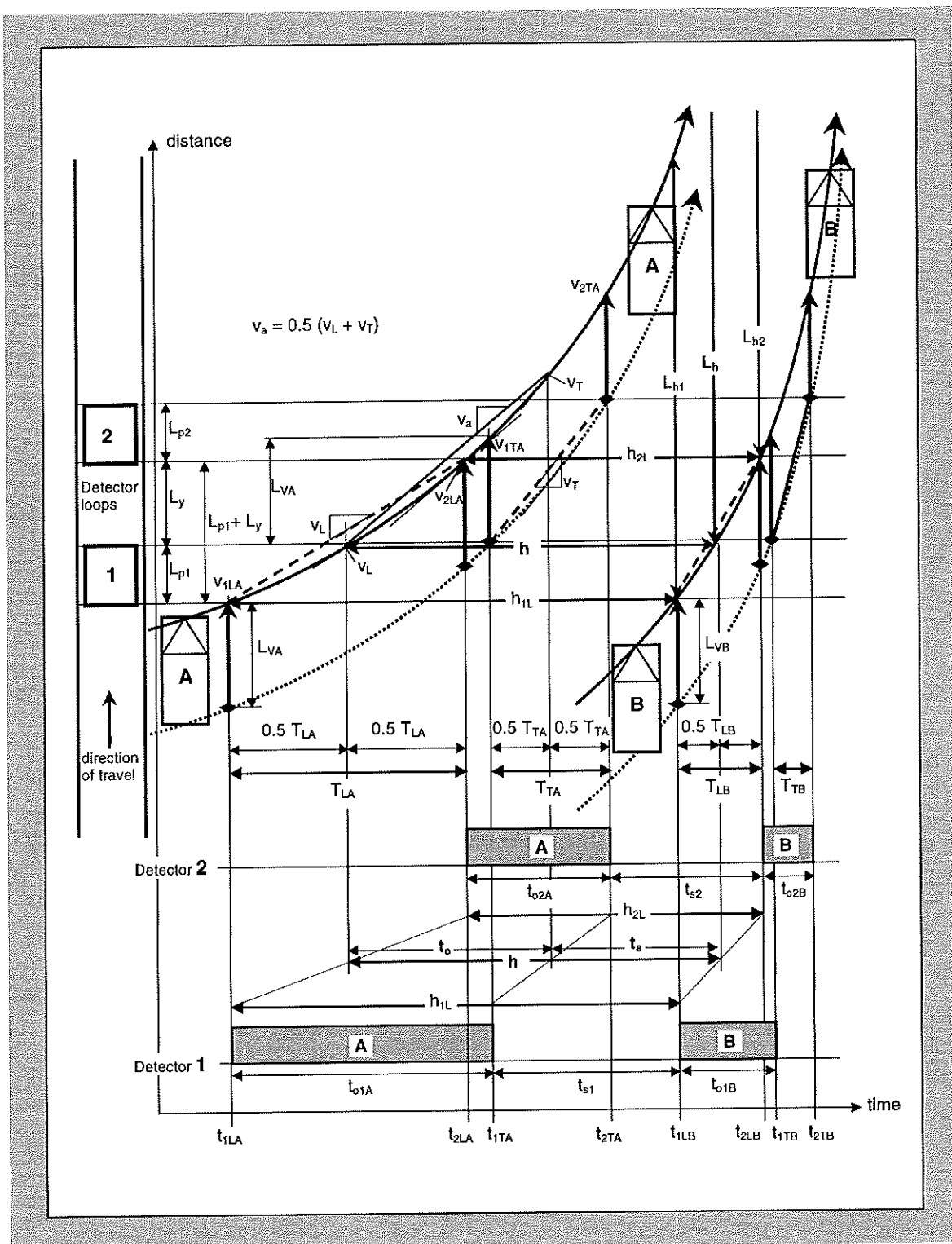
Figure 3.2.1 - Time-distance diagram explaining the measurement of traffic flow parameters with the two-loop presence detection system

An acceleration rate based on the assumption of constant acceleration can be estimated using the basic data as follows:

$$a \quad = \quad (v_T - v_L) / t_o \tag{3.2.17}$$

where t_o is the average occupancy time from *Equation (3.2.14)*, and v_L and v_T are the leading and trailing speeds from *Equations (3.2.5) and (3.2.6)*.

Similarly, the acceleration rate estimate with passage detection is:

$$a \quad = \quad (v_T - v_L) / t_v \tag{3.2.17a}$$

This is relevant to the passage detection system used for collecting queue discharge headways and speeds at signal stop line (Akçelik, Besley and Roper 1999).

For freeway data, the constant acceleration rate estimate from *Equation (3.2.17)* could be used to obtain speeds v_{1LA}, v_{2LA}, v_{1TA}, v_{2TA} for the front of the vehicle at base times t_{1LA}, t_{2LA}, t_{1TA}, t_{2TA} as shown in *Figure 3.2.1*:

$$v_{1LA} = \quad v_L - 0.5 \, a \, T_{LA} \tag{3.2.18a}$$

$$v_{2LA} = \quad v_L + 0.5 \, a \, T_{LA} \tag{3.2.18b}$$

$$v_{1TA} = \quad v_T - 0.5 \, a \, T_{TA} \tag{3.2.18c}$$

$$v_{2TA} = \quad v_L + 0.5 \, a \, T_{TA} \tag{3.2.18d}$$

Therefore, $v_L = 0.5 \, (v_{1L} + v_{2L})$ and $v_T = 0.5 \, (v_{1T} + v_{2T})$, and v_L and v_T represent average speeds at mid-points of T_{LA} and T_{TA} times (however, the corresponding distances are not at the mid-point of the travel distance).

However, preliminary analysis of data indicated that the acceleration rate estimates for freeway data were not reliable due to the insufficient accuracy level of the speed and occupancy time data obtained from the two-loop system. As a result, the headway, occupancy time and space time values measured at the two loops are averaged for improved accuracy levels. Similarly, the leading and trailing speeds based on the two loops are averaged. The method presented here matches the average values of relevant parameters. Importantly:

(i)   Average speed from *Equation (3.2.16)* matches the average occupancy from *Equation (3.2.14)* so that the basic relationship between speed and occupancy time holds:

$$t_o \quad = \quad h - t_s = 3.6 \, (L_p + L_v) / v \tag{3.2.19}$$

(ii)   The occupancy time for the leading vehicle is a component of the leading headway. Therefore the average speed for the leading vehicle is used in association with the leading headway and occupancy.

The average speed from *Equation (3.2.16)* differs from the following space-mean speed used by VicRoads:

$$\begin{aligned} v'_a \quad &= \quad 3.6 \, (L_{p1} + L_{p2} + 2 \, L_y) / (T_{LA} + T_{TA}) = \tag{3.2.20} \\ &= \quad 2 / (1 / v_L + 1 / v_T) \\ &= \quad 36 / (T_{LA} + T_{TA}) \; \textit{for the two-loop system used for surveys} \end{aligned}$$
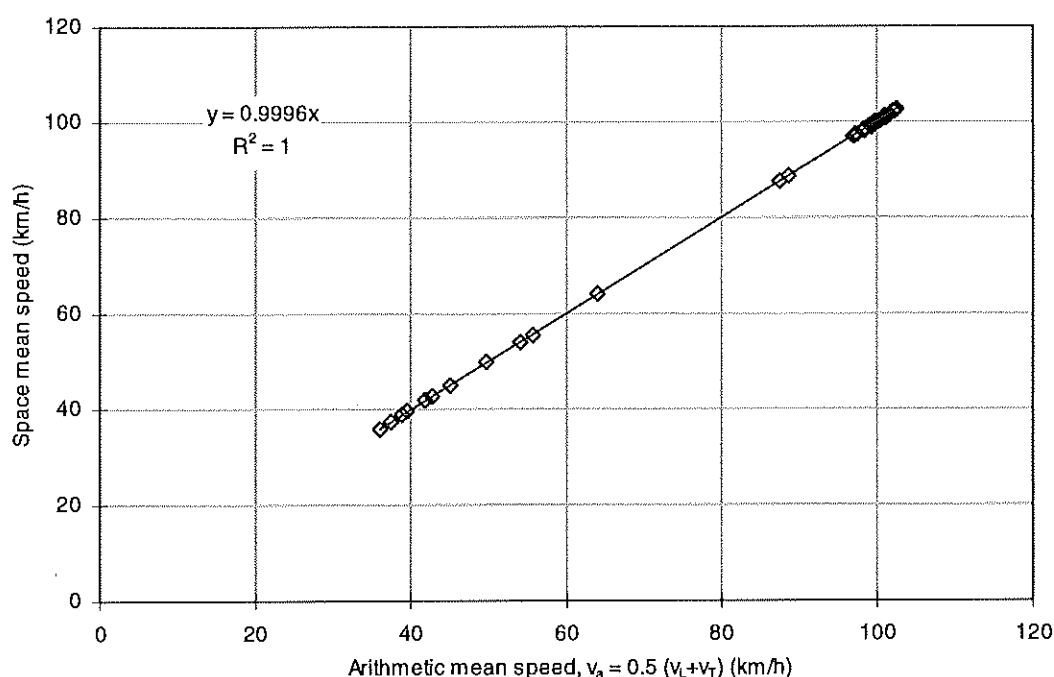
Figure 3.2.2 - Space-mean speed vs arithmetic mean using the space-mean values of the leading and trailing speeds for the Eastern Freeway data (average values for 5-min intervals)

The average speeds from *Equation (3.2.20)* and *Equation (3.2.16)* are very close as seen from the comparison of the two speeds for the Eastern Freeway 5-min aggregate data in *Figure 3.2.2*.

Data Aggregation to Determine Traffic Stream Parameters

Headway, occupancy time, space time and speed data for individual vehicles are aggregated using the general principles presented in *Section 2.2* in order to obtain average values of vehicle parameters describing the characteristics of freeway traffic flows. Other vehicle parameters such as spacing, vehicle length, etc. are not measured directly as individual vehicle parameters. These and other traffic stream parameters such as flow rate, density and occupancy ratios are calculated as average values using average headway, occupancy time, space time and speed data.

Data for aggregation periods of 20 s, 1 min, 2 min, 5 min and 15 min were calculated. Importantly, aggregate data (average parameter values) were always obtained from the original individual vehicle data set, not from an aggregate data set for a shorter period (e.g. 1-min averages are not obtained from 20-s averages). The method used for data aggregation is described below.

Using individual vehicle data, calculate the average headway, occupancy time and space time values for the analysis (aggregation) period as a simple arithmetic mean from:

$$h = (\Sigma h_j) / (n - 1) \qquad\qquad (3.2.21)$$

$$t_o = (\Sigma t_{oj}) / (n - 1) \qquad (3.2.22)$$

$$t_s = (\Sigma t_{sj}) / (n - 1) \qquad (3.2.23)$$

where h_j, t_{oj}, t_{sj} are the headway, occupancy time and space time values measured for *jth* vehicle (from *Equations (3.2.13) to (3.2.15)*), n is the number of vehicles observed during the analysis period, and summation is for (n − 1) vehicles.

The average values from *Equations (3.2.21) to (3.2.23)* should satisfy the general relationship $h = t_o + t_s$.

To calculate the average speed for the analysis period, use average values of travel times between the *leading* edges of *detection zones 1 and 2* (T_{LA}) and between the *trailing* edges of *detection zones 1 and 2* (T_{TA}):

$$T_{LA} = (\Sigma T_{LAj}) / (n - 1) \qquad (3.2.24)$$

$$T_{TA} = (\Sigma T_{TAj}) / (n - 1) \qquad (3.2.25)$$

where T_{LAj}, T_{TAj} are the values for the *jth* vehicle (from *Equations (3.2.1) and (3.2.2)*), n is the number of vehicles observed during the analysis period, and summation is for (n − 1) vehicles.

Calculate the average leading and trailing speeds from:

$$v_L = 3.6 (L_{p1} + L_y) / T_{LA} \qquad (3.2.26)$$

$$v_T = 3.6 (L_y + L_{p2}) / T_{TA} \qquad (3.2.27)$$

where T_{LA} and T_{TA} are the average values for the analysis period, and $L_{p1} + L_y = L_y + L_{p2} = 5.0$ m.

Note that v_L and v_T from *Equations (3.2.26) and (3.2.27)* are space-mean speeds equivalent to:

$$v_L = (n - 1) / \Sigma (1/v_{Lj}) \qquad (3.2.28)$$

$$v_T = (n - 1) / \Sigma (1/v_{Tj}) \qquad (3.2.29)$$

where v_{Lj}, v_{Tj} are the speed values measured for *jth* vehicle, n is the number of vehicles observed during the analysis period, and summation is for (n − 1) vehicles.

Using the same formula as for the individual vehicles, calculate the average speed for the traffic stream from:

$$v_a = 0.5 (v_L + v_T) \qquad (3.2.30)$$

where v_L and v_T are the average leading and trailing speed values (space-mean speeds) for the analysis period.

Calculate average values of other traffic stream parameters for the analysis period from the following equations using the average values of headway, occupancy time, space time and speed calculated using equations given above. These equations are based on presence detection. For the corresponding equations for passage detection, see *Section 2.2*.

Average spacing, L_h (m), gap length, L_s (m), vehicle length, L_v (m):

$$L_h = h\, v_a / 3.6 \qquad\qquad (3.2.31)$$

$$L_s = L_p + t_s\, v_a/3.6 = L_p + (h - t_o)\, v_a / 3.6 \qquad (3.2.32)$$

$$L_v = t_o\, v_a / 3.6 - L_p \qquad\qquad (3.2.33)$$

Average flow rate, q (veh/h), density, k (veh/km) and time and space occupancy ratios, O_t and O_s (percentage):

$$q = 3600 / h \qquad\qquad (3.2.34)$$

$$L_h = 1000\, v / q \qquad\qquad (3.2.35)$$

$$k = 1000 / L_h \qquad\qquad (3.2.36)$$

$$O_t = 100\, t_o / h = 100\, (L_v + L_p) / L_h \qquad (3.2.37)$$
$$subject\ to\ O_t \le 100\ \%$$

$$O_s = 100\, L_v / L_h \qquad\qquad (3.2.38)$$

where $L_p = 2.0$ m.

Figure 3.2.3 shows comparison of flow rates calculated from headway data (*Equation 3.2.34*) and using the number of detector actuations (averages for 5-min intervals). It is seen that flow rates from the number of detector actuations gave about 3 per cent higher values.
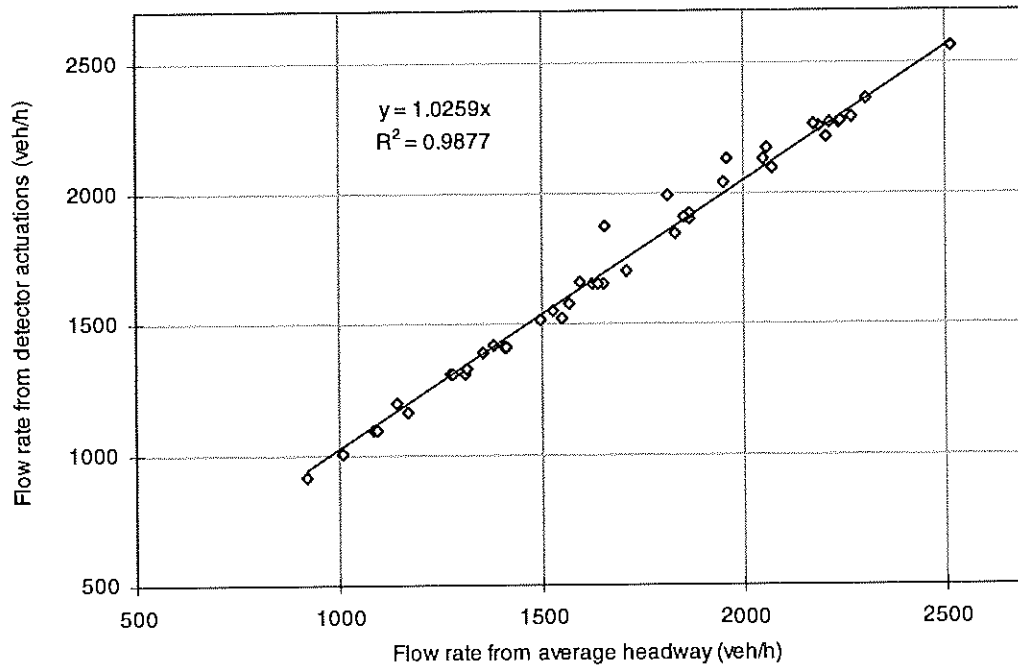

Figure 3.2.3 - Comparison of flow rates calculated from headway data and detector actuations (averages for 5-min intervals)

Speed-Flow Data for Various Aggregation Periods

Speed-flow data for individual vehicles and for aggregation periods of 20 s, 1 min, 2 min, 5 min and 15 min are given in *Figures 3.2.4 to 3.2.10*. Individual vehicle data are given in two parts due to the maximum number of data points that can be handled by the Excel spreadsheet application. The low speed points observed with individual vehicle data are seen to be lost in data aggregation.

Comparison of In-Vehicle vs Loop Data

Speed - flow data using travel speeds obtained from the Gipsitrac vehicle are given in *Figures 3.2.11 to 3.2.16*. These include results with travel speeds at the loop, over 100 m around the loop and over 400 m around the loop, each with flow rates measured at the two-loop detection system and aggregated for 1-minute and 5-minute periods. It is seen that the speed - flow relationships are consistent independent of the flow rate aggregation period and speed measurement distance. They are also comparable to the speed-flow relationships obtained using the two-loop system.

Figures 3.2.17 to 3.2.22 show travel speed data from the Gipsitrac vehicle measured at the loop, over 100 m around the loop and over 400 m around the loop, each against the speed measured by the loop detectors (1-minute and 5-minute aggregation). Generally, the two methods appear to compare reasonably well.

Figure 3.2.23 shows travel speed data from the Gipsitrac vehicle measured at the loop vs speed measured by the loop detectors for Eastbound traffic (individual vehicles). *Figure 3.2.24* shows speed data measured using VDAS treadle switches vs speed data from loop detection (from an earlier study).

These comparisons indicate that data collected by the two-loop system is generally acceptable.

Frequency of Individual Vehicle Headways

Figures 3.2.4 and 3.2.5 indicate that a high frequency of small headways are observed for individual vehicles at the survey site. *Figures 3.2.25 and 3.2.26* show the distribution of individual vehicle headways for speeds above 80 km/h (in 0.2 s intervals). It is seen that, for example, the frequency of headways equal to or less than 1.60 s (instantaneous flow rates equal to or higher than 2250 veh/h) is 52 per cent, and the frequency of headways equal to or less than 1.00 s (instantaneous flow rates equal to or higher than 3600 veh/h) is 28 per cent. The minimum individual vehicle headways observed are around 0.4 s, and individual vehicle spacings at this minimum headway are around 11 m (see *Figures 3.2.27 and 3.2.28*). Also see the discussion on jam spacing in *Section 5.7*.

As seen from *Figures 3.2.6 to 3.2.10*, the maximum flow rate is decreased with increased aggregation periods due to the process of averaging. This issue is important in defining the *capacity* of freeways. The US Highway Capacity Manual (TRB 1998) defines capacity as the maximum flow rate that can be sustained during a reasonably long period of time, and uses $T_f = 15$ min for this purpose. For this period, the maximum flow rate (capacity) is around 2300 veh/h (as seen from *Figure 3.2.10*), and the corresponding average headway is 1.565 s. *Figures 3.2.25 and 3.2.26* indicate that about 50 per cent of individual vehicle headways are below this value.
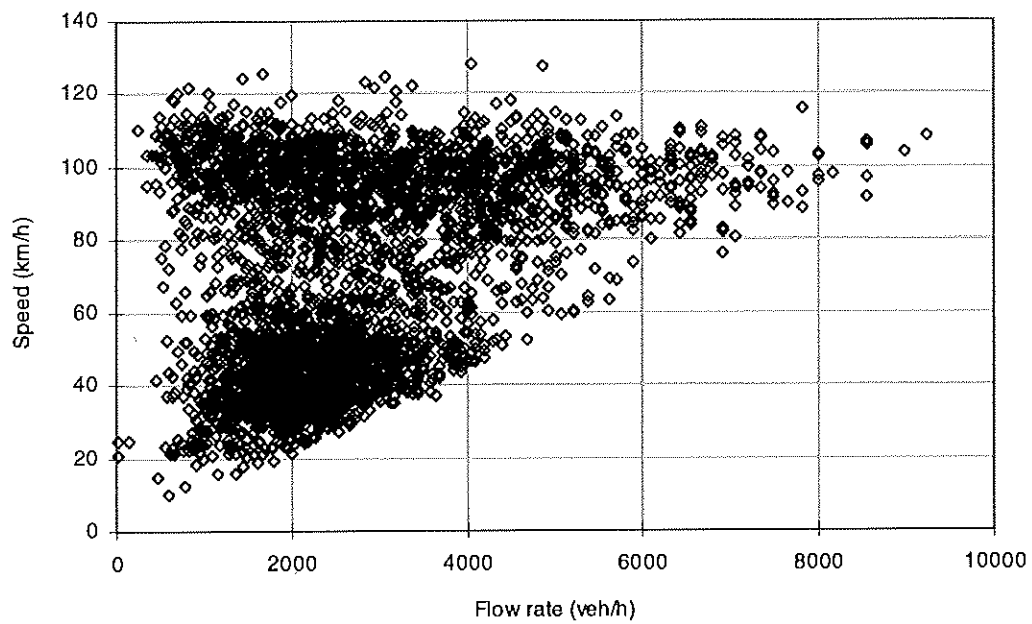
Figure 3.2.4 - Speed - flow data using individual vehicle data (6.30 - 8.20 am)
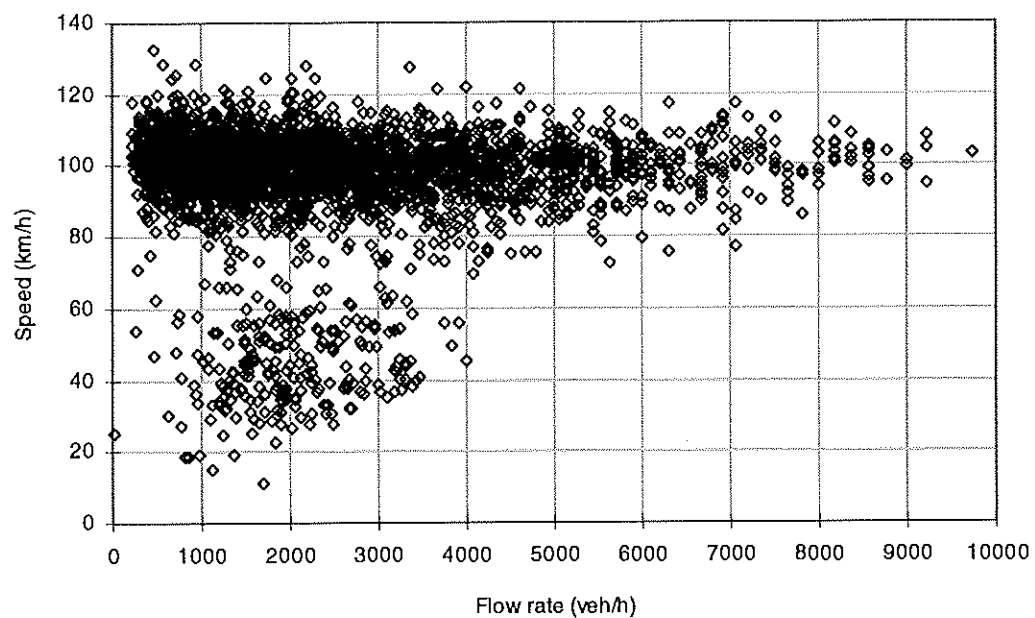
Figure 3.2.5 - Speed - flow data using individual vehicle data (8.20 – 10.40 am)
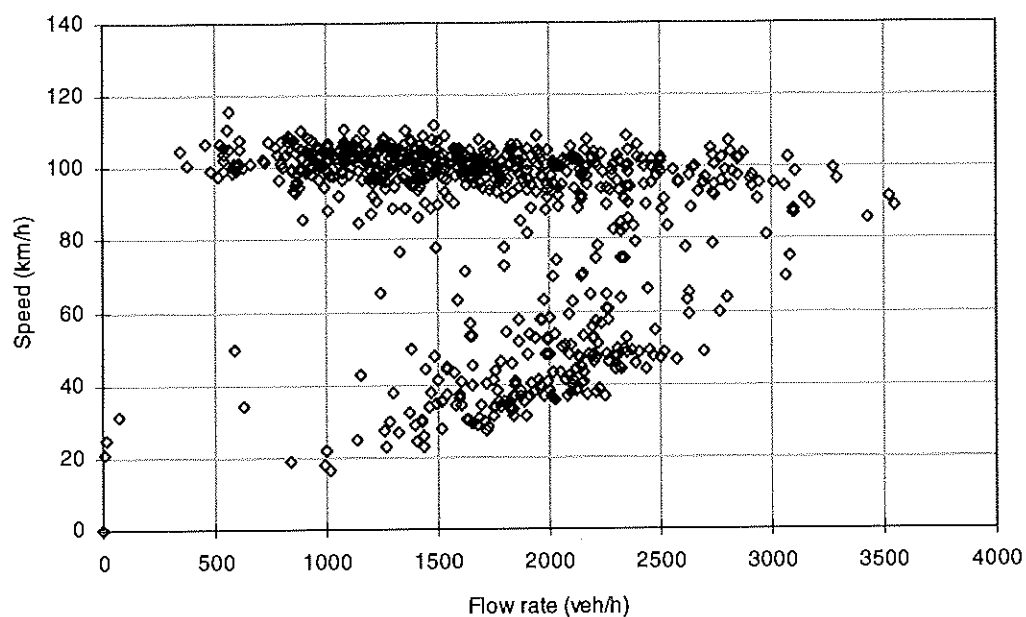
Figure 3.2.6 - Speed - flow data using 20-second aggregation period
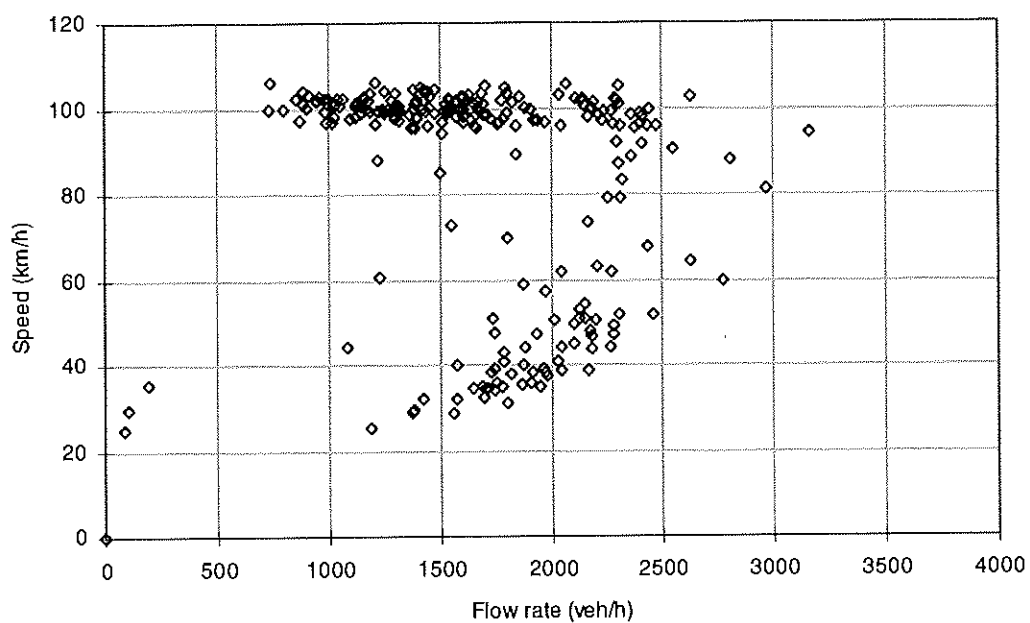
Figure 3.2.7 - Speed - flow data using 1-minute aggregation period
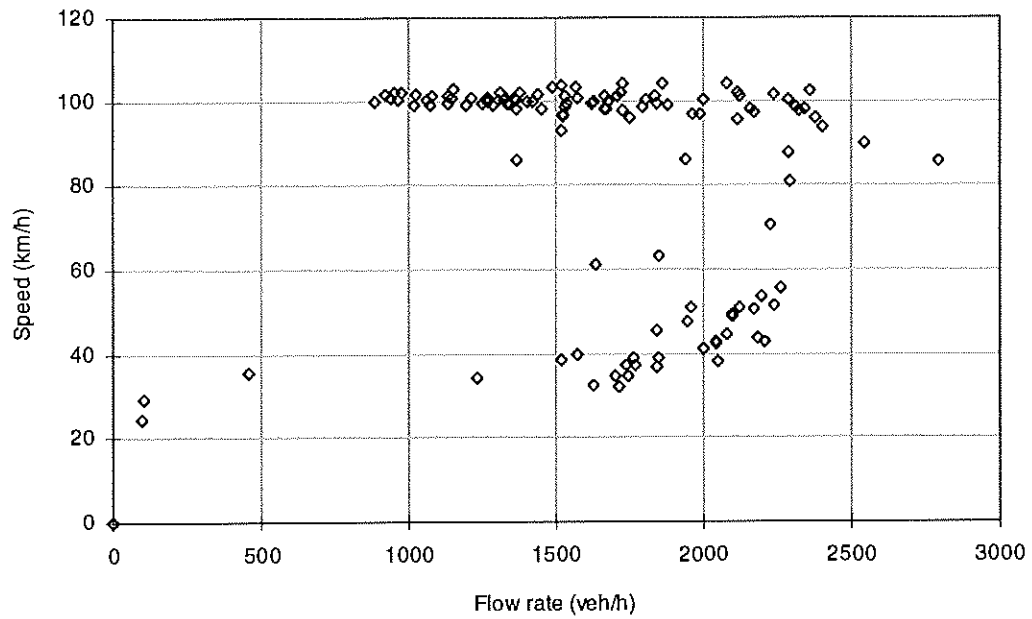
Figure 3.2.8 - Speed - flow data using 2-minute aggregation period
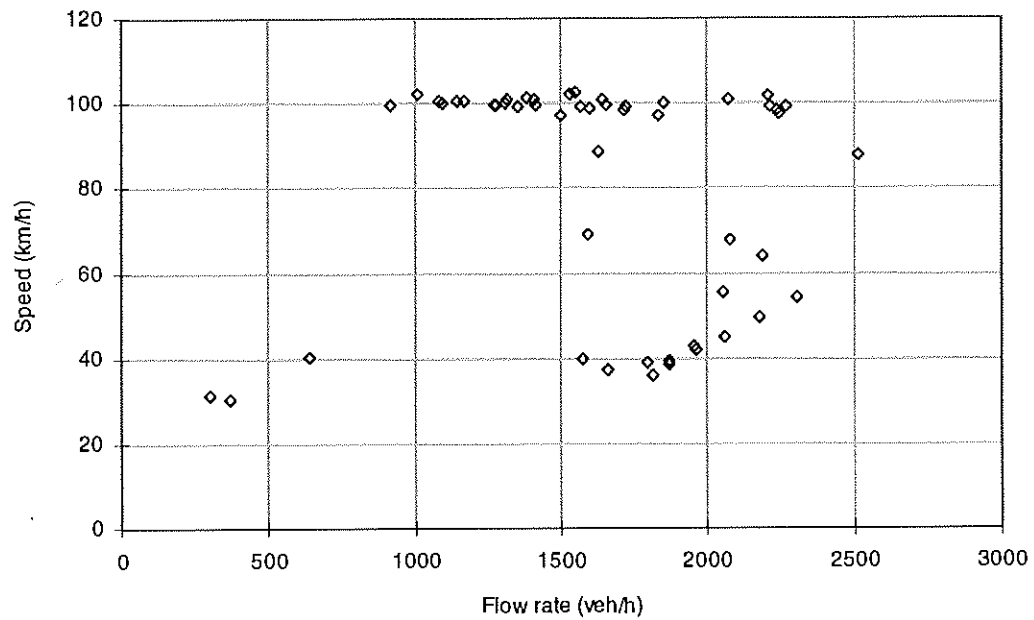
Figure 3.2.9 - Speed - flow data using 5-minute aggregation period
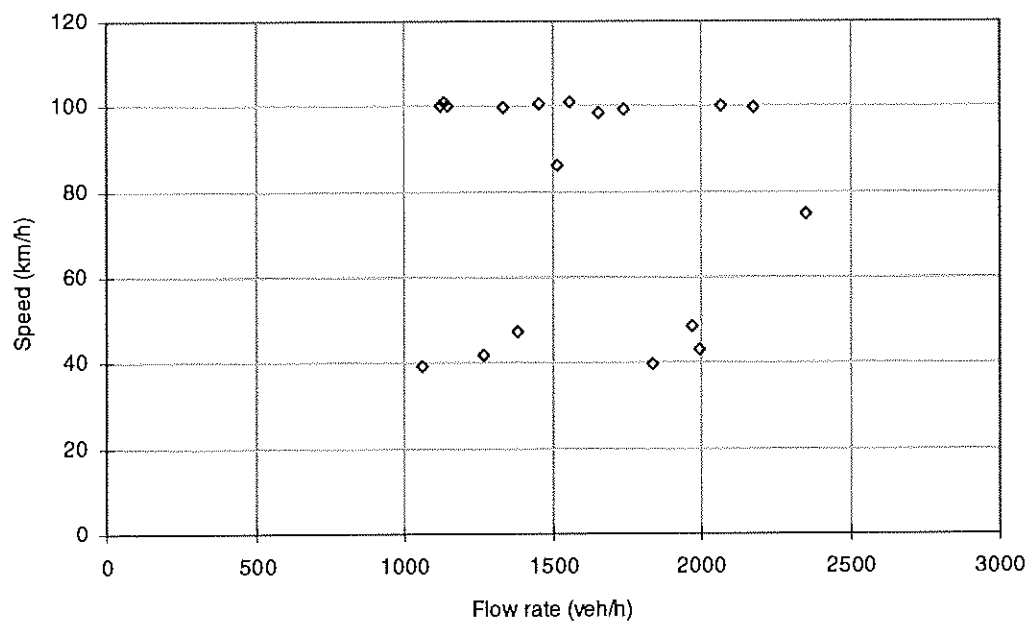
Figure 3.2.10 - Speed - flow data using 15-minute aggregation period
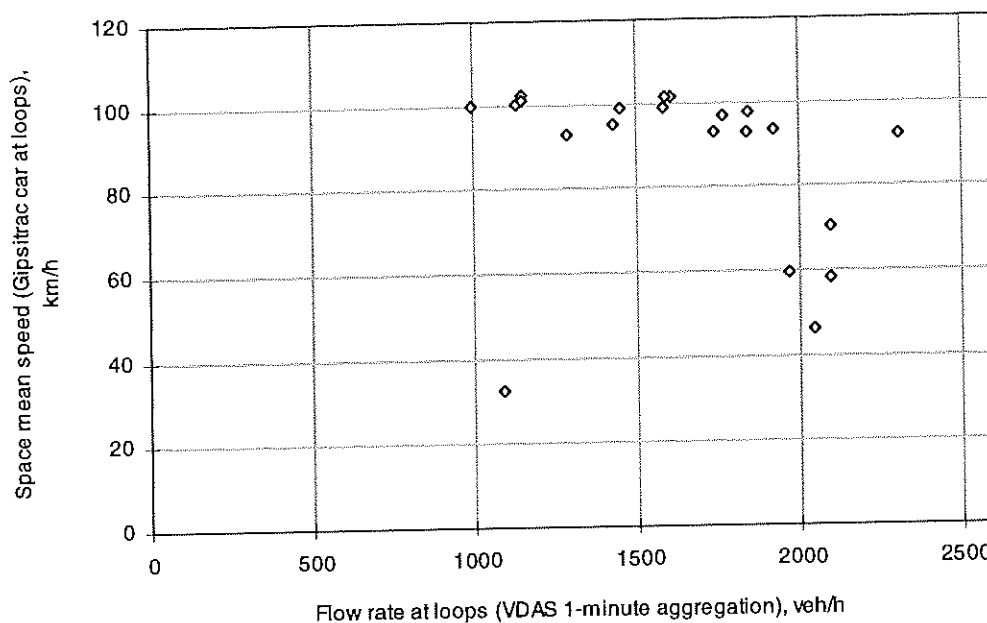
Figure 3.2.11 - Speed - flow data using travel speeds from the Gipsitrac vehicle measured at the loop and flow rates from the loop detectors (1-minute aggregation)
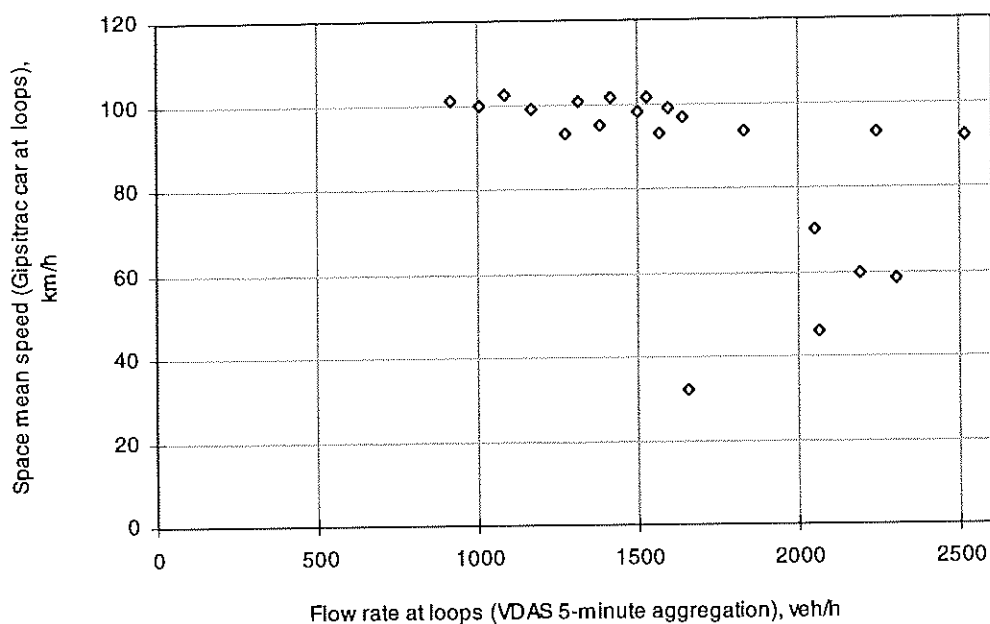
Figure 3.2.12 - Speed - flow data using travel speeds from the Gipsitrac vehicle measured at the loop and flow rates from the loop detectors (5-minute aggregation)
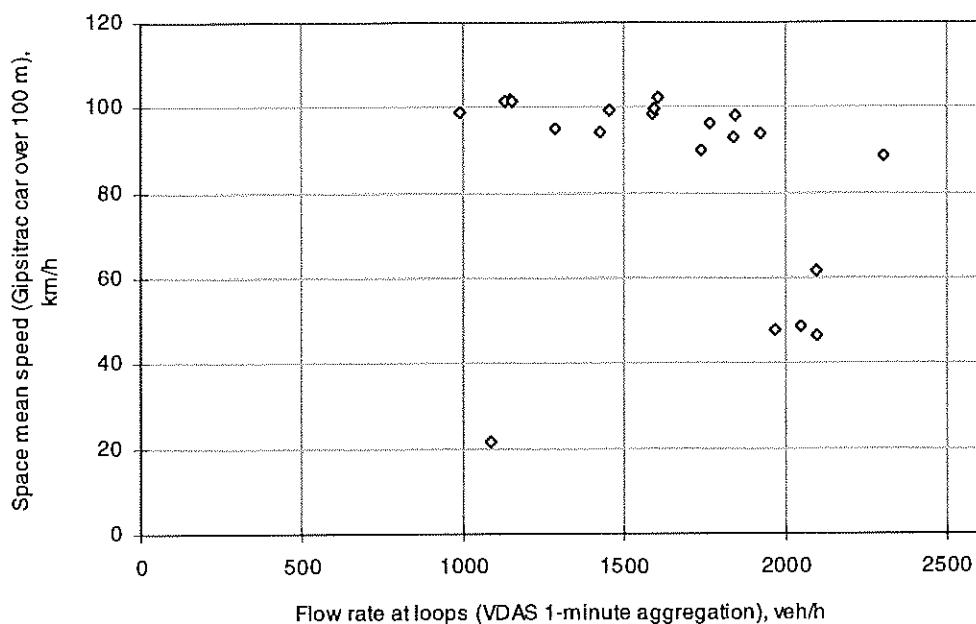
Figure 3.2.13 - Speed - flow data using travel speeds from the Gipsitrac vehicle measured over 100 m around the loop and flow rates from the loop detectors (1-minute aggregation)
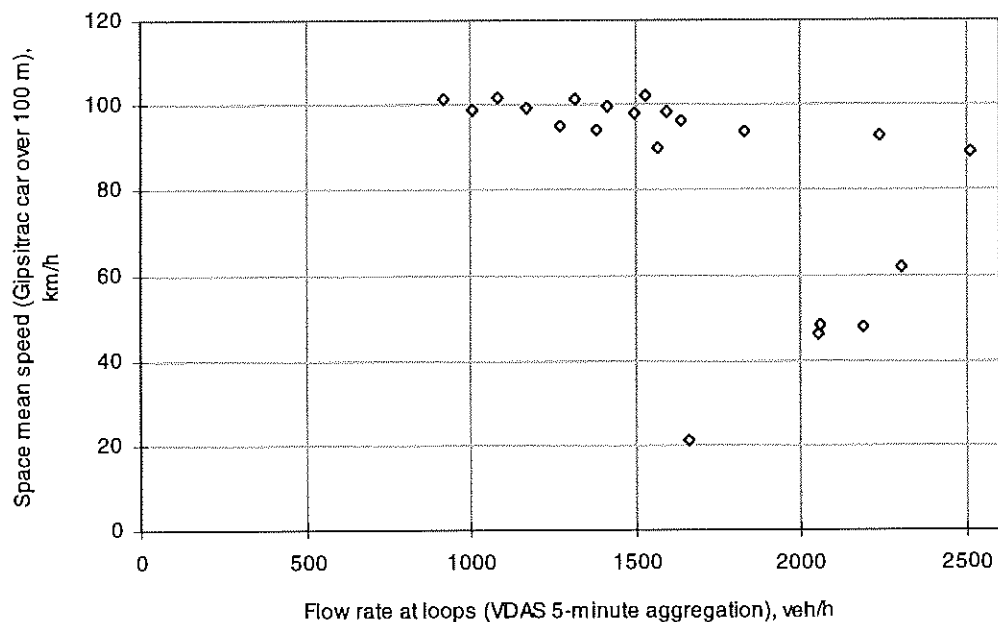
Figure 3.2.14 - Speed - flow data using travel speeds from the Gipsitrac vehicle measured over 100 m around the loop and flow rates from the loop detectors (5-minute aggregation)
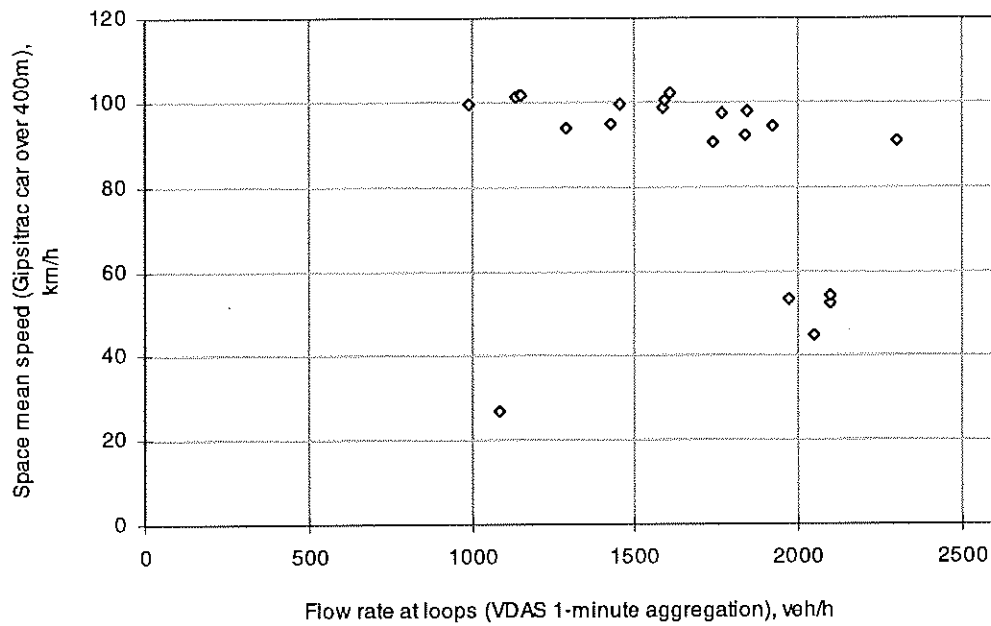
Figure 3.2.15 - Speed - flow data using travel speeds from the Gipsitrac vehicle measured over 400 m around the loop and flow rates from the loop detectors (1-minute aggregation)
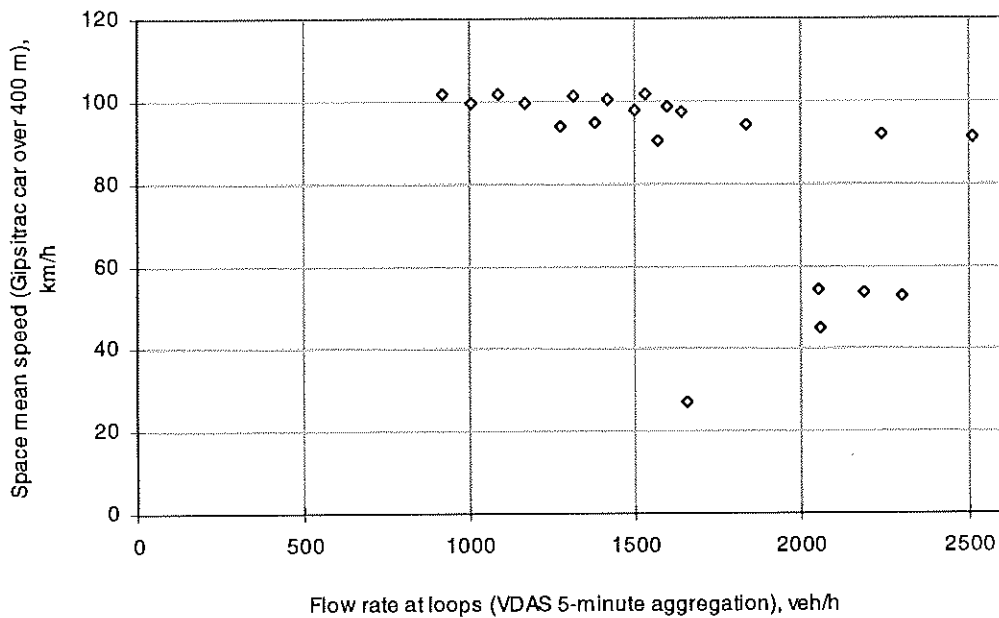
Figure 3.2.16 - Speed - flow data using travel speeds from the Gipsitrac vehicle measured over 400 m around the loop and flow rates from the loop detectors (5-minute aggregation)
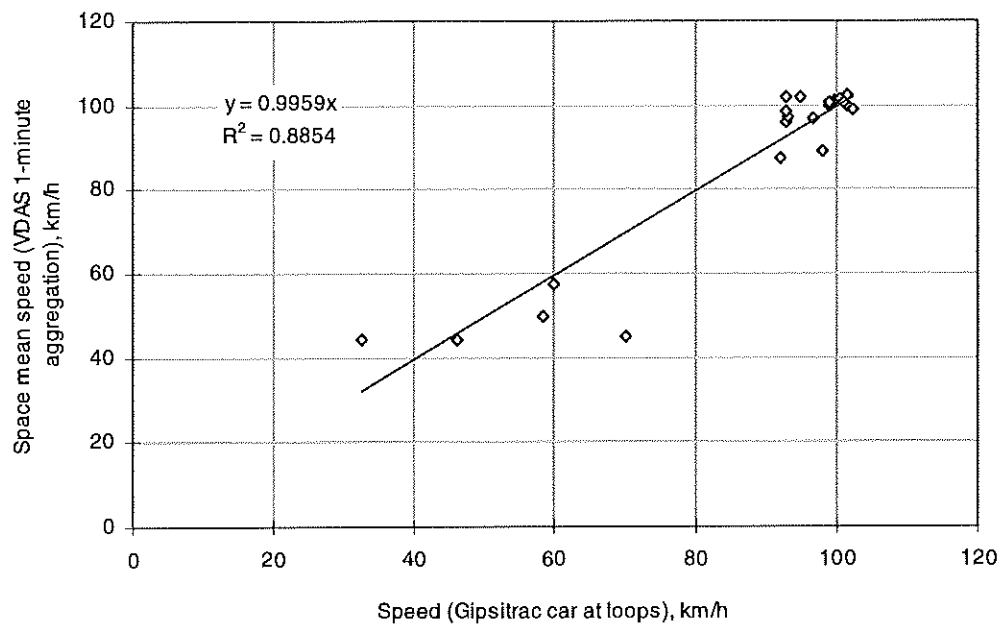
Figure 3.2.17 - Speed data from the Gipsitrac vehicle at the loop vs speed measured by the loop detectors (1-minute aggregation)
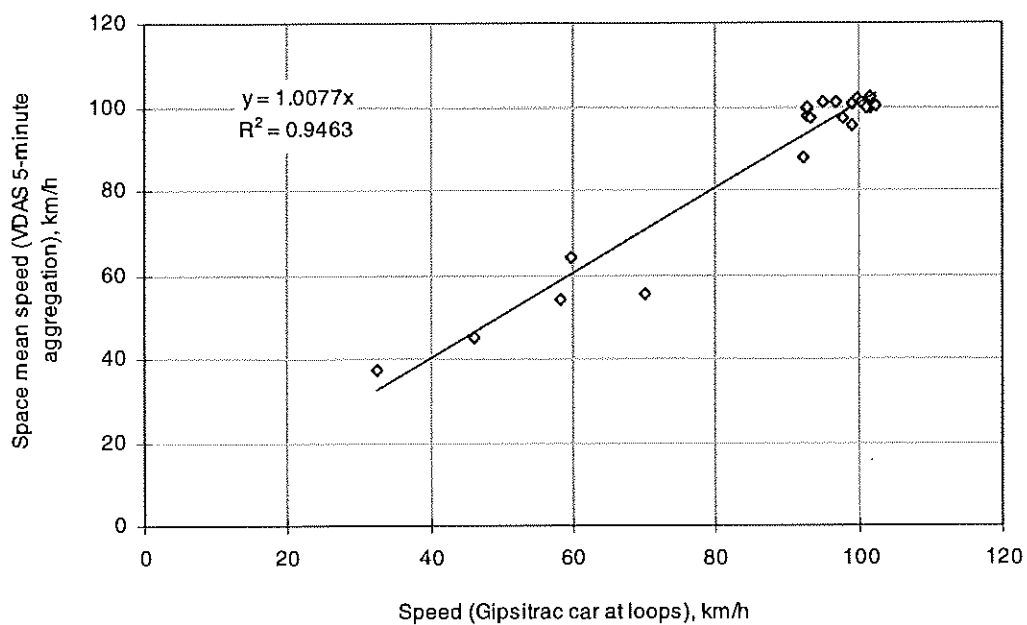
Figure 3.2.18 - Speed data from the Gipsitrac vehicle at the loop vs speed measured by the loop detectors (5-minute aggregation)
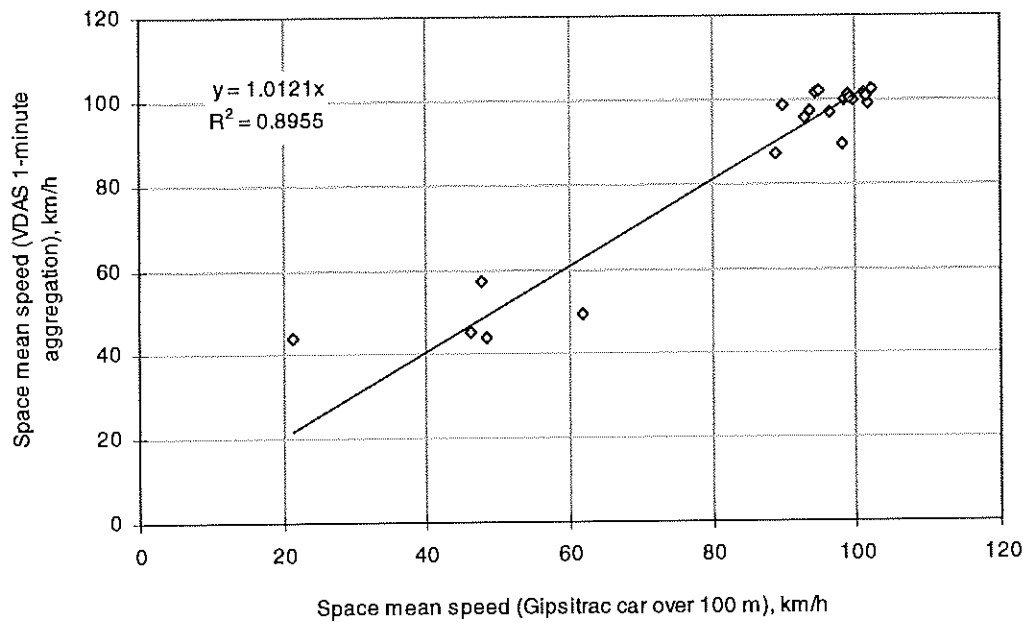
Figure 3.2.19 - Speed data from the Gipsitrac vehicle over 100 m around the loop vs speed measured by the loop detectors (1-minute aggregation)
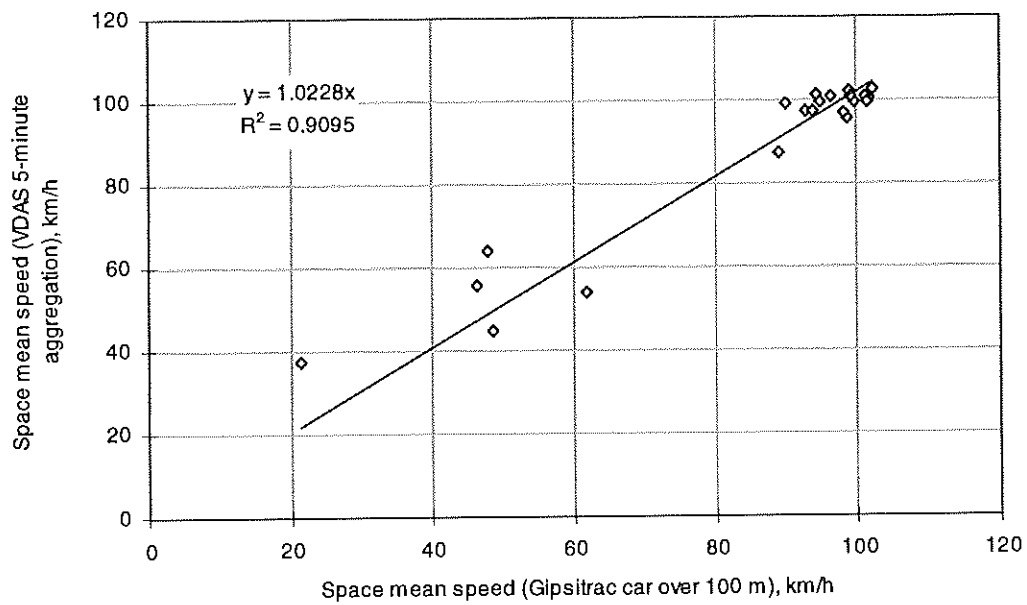
Figure 3.2.20 - Speed data from the Gipsitrac vehicle over 100 m around the loop vs speed measured by the loop detectors (5-minute aggregation)
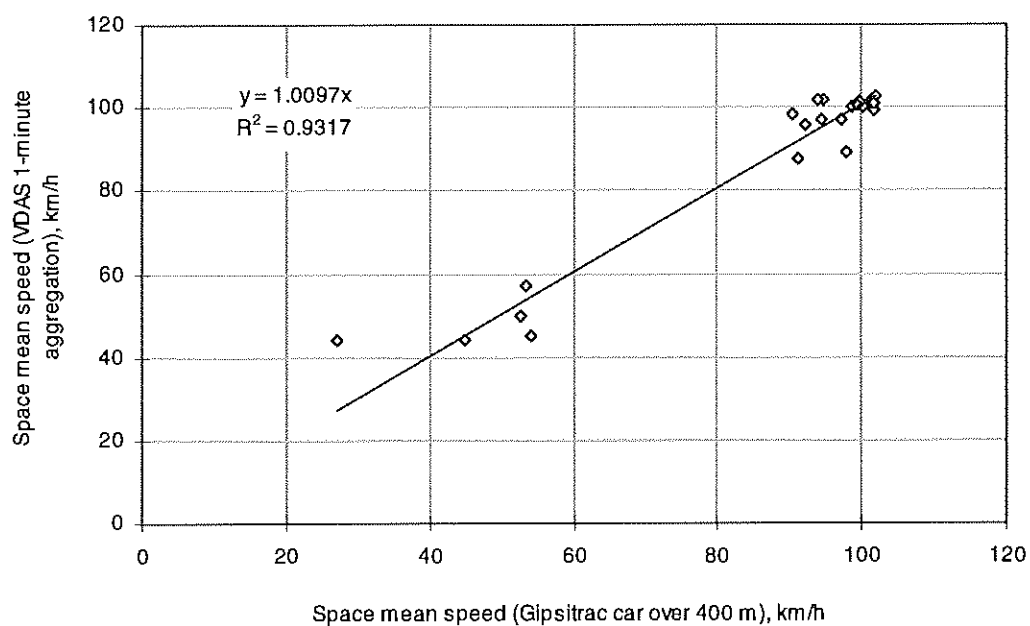
Figure 3.2.21 - Speed data from the Gipsitrac vehicle over 400 m around the loop vs speed measured by the loop detectors (1-minute aggregation)
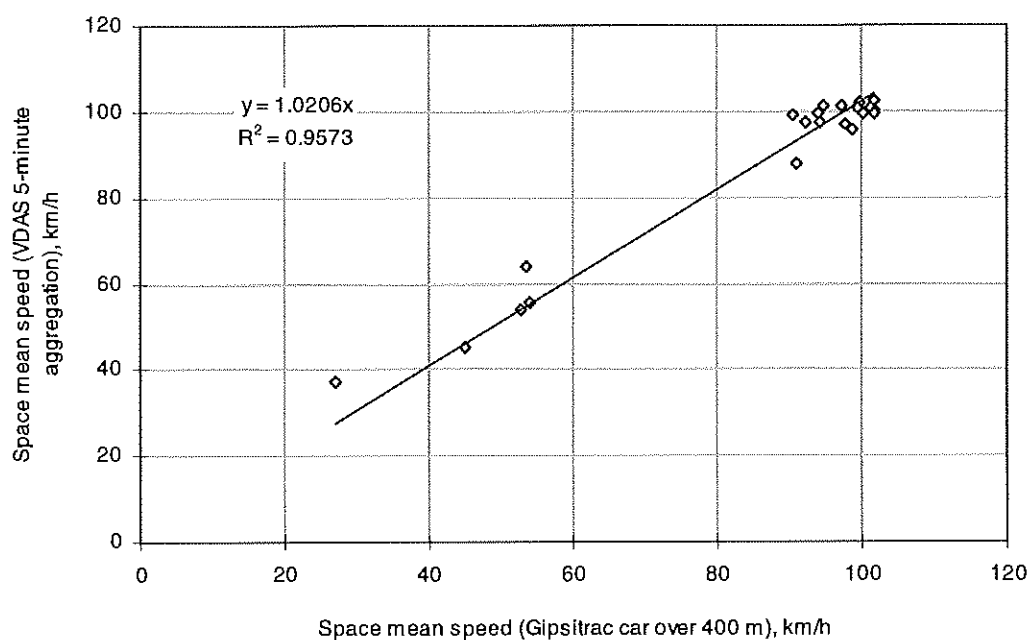
Figure 3.2.22 - Speed data from the Gipsitrac vehicle over 400 m around the loop vs speed measured by the loop detectors (5-minute aggregation)
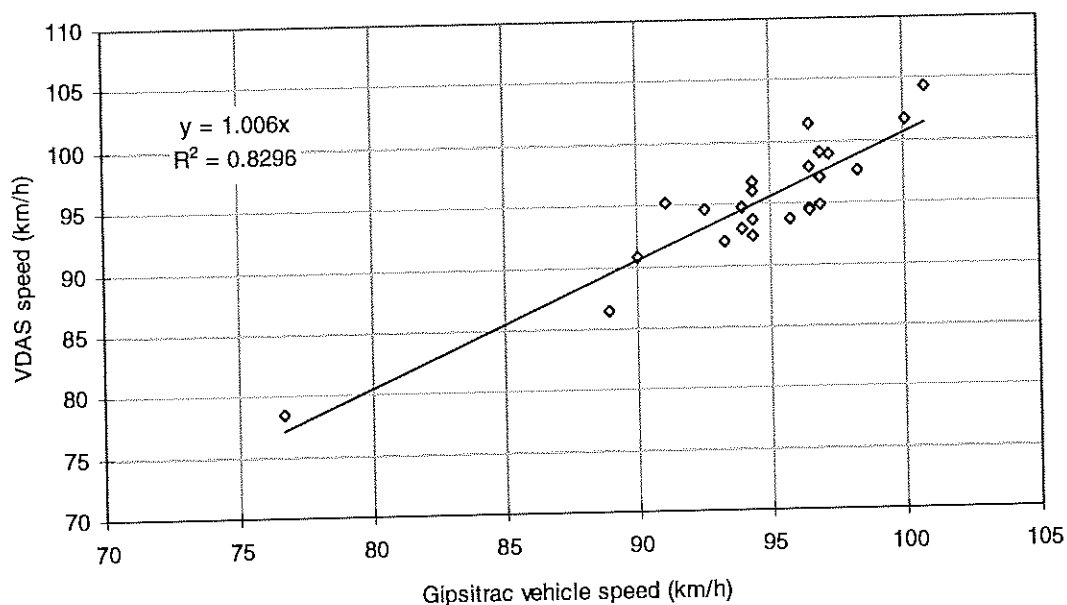
Figure 3.2.23 - Travel speed data from the Gipsitrac vehicle measured at the loop vs speed measured by the loop detectors for Eastbound traffic (individual vehicles)
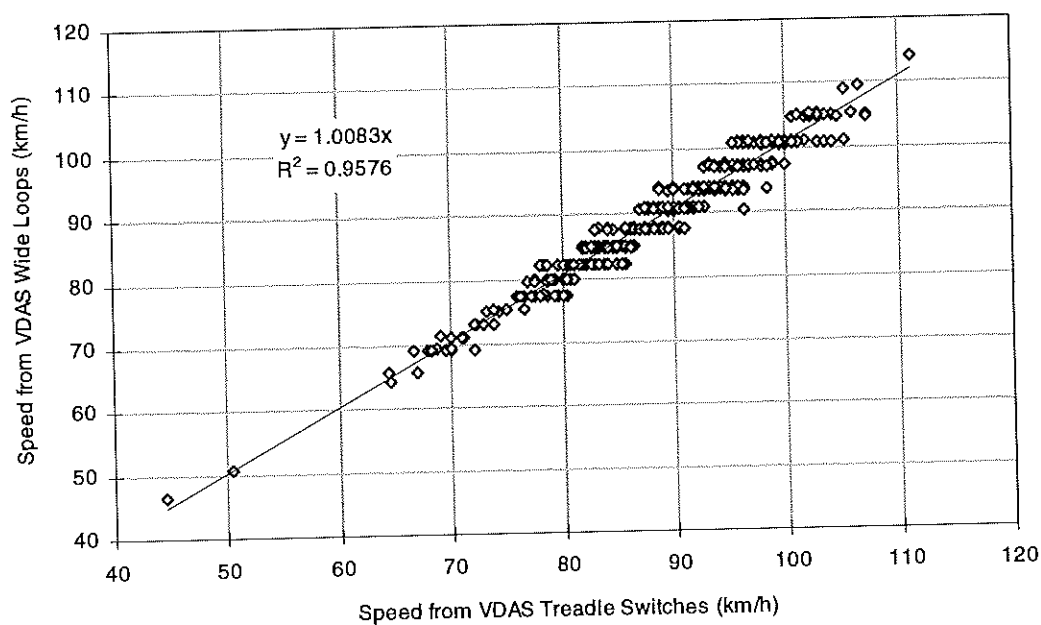
Figure 3.2.24 - Speed data measured using VDAS treadle switches vs speed data from loop detection
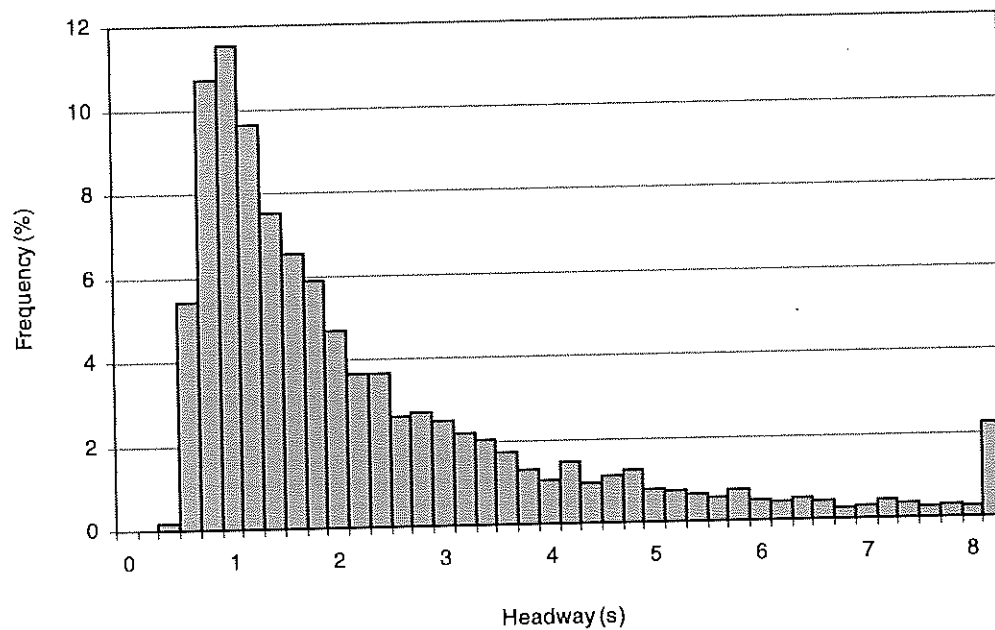
*Figure 3.2.25 - Frequency distribution of individual vehicle headways
for speeds above 80 km/h*
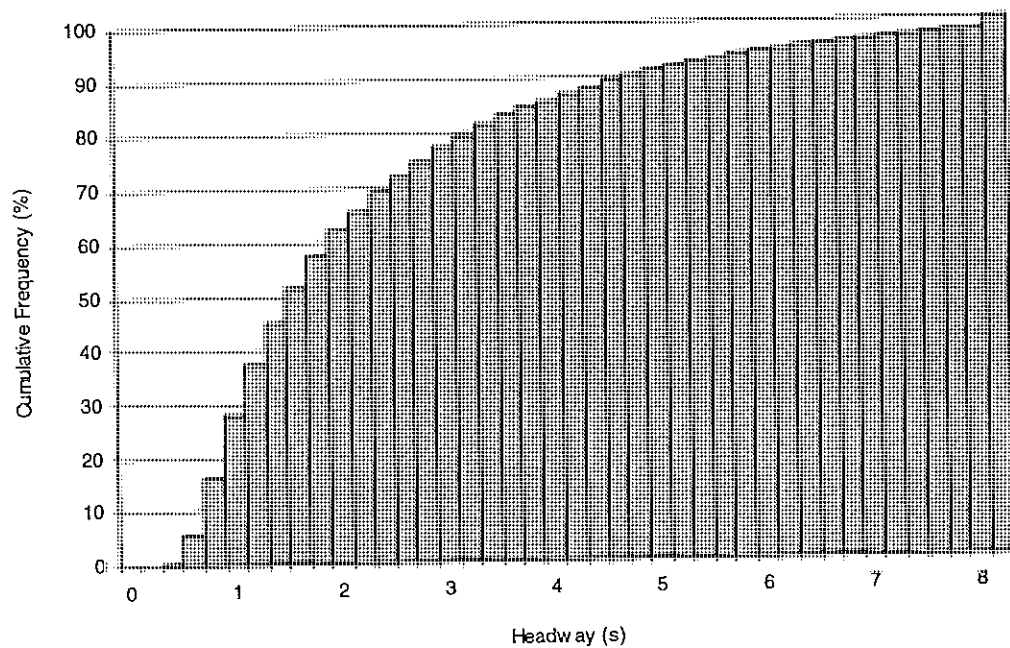
*Figure 3.2.26 - Cumulative frequency distribution of individual vehicle headways
for speeds above 80 km/h*
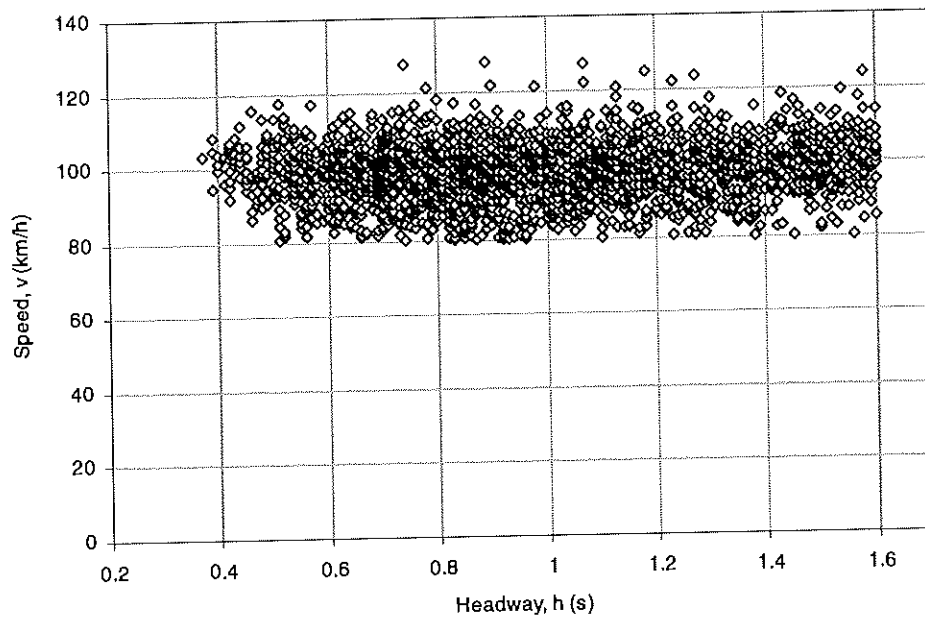
Figure 3.2.27 - Individual vehicle speeds and headways for speeds above 80 km/h and headways below 1.6 s
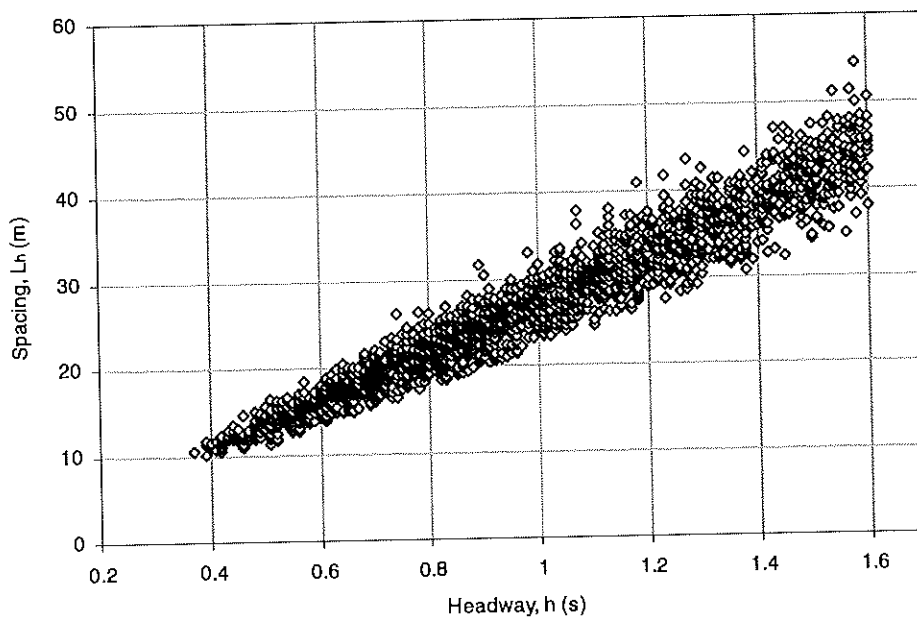
Figure 3.2.28 - Individual vehicle headways and spacings for speeds above 80 km/h and headways below 1.6 s

4 SIX ANALYTICAL TRAFFIC FLOW MODELS

There is a vast amount of literature on fundamental traffic relationships for uninterrupted traffic flow offering empirical as well as theoretical models based on car-following, hydrodynamic and kinetic theories of traffic flow (e.g. Drew 1968, May 1990, TRB 1975, 1998). However, these models fail to provide a complete description of the speed, flow and density characteristics observed in practice for a full range of flow conditions.

In this section, six analytical traffic flow models are presented including single-regime and two-regime models. A single-regime model is a single function that applies to both saturated and unsaturated flow conditions. A two-regime model consists of two functions that apply to saturated and unsaturated flow conditions separately.

Models 3 and 6 are single-regime models. All other models are two-regime models. Other combinations of two-regime models could be obtained by using Models 1, 2, 4 and 5 together, e.g. Models 4 and 2B. The two functions used for a two-regime model must have the same values of maximum flow rate and the speed at maximum flow rate.

The models are presented as speed - flow rate relationships from which other relationships such as speed - density, space time - speed and occupancy ratio - flow rate can be derived using the fundamental relationships given in *Section 2*.

Calibration options and results obtained for these models are presented in *Section 5*.

4.1 Model 1 Description

This is a two-piece model with two separate functions for unsaturated and saturated regions of flow (Models 1A and 1B, respectively). The two functions are used together as one model with the same values of maximum flow rate and speed at maximum flow (q_n, v_n) used for both functions.

Model 1A: Unsaturated Conditions

$$v = v_f + b_1 (q / 1000)^{p_2} \qquad (4.1.1)$$
$$\textit{subject to } p_2 > 0 \textit{ and } b_1 < 0$$

where v (km/h) is the speed, q (veh/h) is the flow rate, v_f (km/h) is the free-flow speed, and b_1, p_2 are constants related through:

$$b_1 = -(v_f - v_n) / (q_n / 1000)^{p_2} \qquad (4.1.2)$$

where v_n (km/h) is the speed at maximum flow rate, q_n (veh/h) is the maximum flow rate.

The model is expressed in the following form for calibration purposes using b_1 from *Equation (4.1.2)* in *Equation (4.1.1)*:

$$v = v_f - (v_f - v_n) (q / q_n)^{p_2} \qquad (4.1.3)$$
$$\textit{subject to } p_2 > 0$$

This model is identical to Model 1 used in ARR 340 (Akçelik, Besley and Roper 1999, Section 10) for uninterrupted flows at signalised intersections.

Model 1B: Saturated Conditions

$$v = L_{hj} (q / 1000) + a_1 (q / 1000)^{p_1} \qquad (4.1.4)$$
$$\textit{subject to } p_1 > 0 \textit{ and } a_1 > 0$$

where v (km/h) is the speed, q (veh/h) is the flow rate, L_{hj} (m/veh) is jam spacing, and a_1, p_1 are constants related through:

$$a_1 = (L_{hn} - L_{hj}) / (q_n / 1000)^{p_1 - 1} \qquad (4.1.5)$$

where q_n (veh/h) is the maximum flow rate, and L_{hn} (m/veh) is the spacing at maximum flow rate calculated from:

$$L_{hn} = 1000 \, v_n / q_n \qquad (4.1.6)$$

where v_n (km/h) is the speed at maximum flow rate.

From *Equation (4.1.5)*, jam spacing is:

$$L_{hj} = L_{hn} - a_1 (q_n / 1000)^{p_1 - 1} \qquad (4.1.7)$$

The model is expressed in the following form for calibration purposes using a_1 from *Equation (4.1.5)* with L_{hn} from *Equation (4.1.6)* in *Equation (4.1.4)*:

$$v = L_{hj} (q / 1000) + (v_n - L_{hj} \, q_n / 1000) (q / q_n)^{p_1} \qquad (4.1.8)$$
$$\textit{subject to } p_1 > 0$$

4.2 Model 2 Description

This is a two-piece model with two separate functions for unsaturated and saturated regions of flow (Models 2A and 2B, respectively). The two functions are used together as one model with the same values of maximum flow rate and speed at maximum flow (q_n, v_n) used for both functions.

Model 2A: Unsaturated Conditions

$$q = q_n [1 - ((v - v_n) / (v_f - v_n))^{p_2}]$$
$$\text{subject to } p_2 > 0$$

(4.2.1)

where q (veh/h) is the flow rate, q_n (veh/h) is the maximum flow rate, v (km/h) is the speed, v_n (km/h) is the speed at maximum flow rate, v_f (km/h) is the free-flow speed, and p_2 is a constant:

The speed-flow function corresponding to *Equation (4.2.1)* is:

$$v = v_n [1 + (v_f / v_n - 1) (1 - q / q_n)^{1/p_2}]$$
$$\text{subject to } p_2 > 0$$

(4.2.2)

This model is identical to Model 2 used in ARR 340 (Akçelik, Besley and Roper 1999, Section 10) for uninterrupted flows at signalised intersections.

Model 2B: Saturated Conditions

$$q = q_n [1 - (1 - v / v_n)^{p_1}]$$
$$\text{subject to } p_1 > 0$$

(4.2.3)

where q (veh/h) is the flow rate, q_n (veh/h) is the maximum flow rate, v (km/h) is the speed, v_n (km/h) is the speed at maximum flow rate, and p_1 is a constant.

The speed-flow function corresponding to *Equation (4.2.3)* is:

$$v = v_n [1 - (1 - q / q_n)^{1/p_1}]$$
$$\text{subject to } p_1 > 0$$

(4.2.4)

Parameter p_1 can be determined from the constraint $dv/dq = L_{hj}$ at $v = 0$. This gives:

$$p_1 = L_{hn} / L_{hj}$$

(4.2.5)

This indicates that Model 2B is equivalent to Model 5 (*Section 4.5*) through $p_1 = m_q / m_v$.

From *Equation (4.2.5)*, jam spacing is:

$$L_{hj} = L_{hn} / p_1$$

(4.2.6)

4.3 Model 3 Description

This model covers both unsaturated and saturated conditions as a single model. It is expressed as:

$$q = 1000 \, v \, / \, [L_{hj} + p_1 \, v \, / \, (1 - v \, / \, v_f)^{p2}] \tag{4.3.1}$$
$$\text{subject to } p_1 > 0 \text{ and } p_2 > 0$$

where q (veh/h) is the flow rate, v (km/h) is the speed, v_f (km/h) is the free-flow speed, L_{hj} (m/veh) is the jam density, and p_1, p_2 are constants related through:

$$p_1 = [(L_{hn} - L_{hj}) \, (1 - v_n \, / \, v_f)^{p2}] \, / \, v_n \tag{4.3.2}$$

where v_n (km/h) is the speed at maximum flow rate, L_{hn} (m/veh) is the spacing at maximum flow rate calculated from:

$$L_{hn} = 1000 \, v_n \, / \, q_n \tag{4.3.3}$$

where q_n (veh/h) is the maximum flow rate.

From *Equation (4.3.2)*, jam spacing is:

$$L_{hj} = L_{hn} - [p_1 \, v_n \, / \, (1 - v_n \, / \, v_f)^{p2}] \tag{4.3.4}$$

The model is expressed in the following form for calibration purposes using p_1 from *Equation (4.3.2)* with L_{hn} from *Equation (4.3.3)* in *Equation (4.3.1)*:

$$q = 1000 \, / \, [\, (L_{hj} \, / \, v) + (1000 \, / \, q_n - L_{hj} \, / \, v_n) \, ((1 - v_n \, / \, v_f) \, / \, (1 - v \, / \, v_f))^{p2} \,] \tag{4.3.5}$$
$$\text{subject to } p_2 > 0$$

This model is identical to Model 3 used in ARR 340 (Akçelik, Besley and Roper 1999, Section 10) for uninterrupted flows at signalised intersections.

A useful method to derive the fundamental speed-flow-density relationships is to start by modelling vehicle spacing as a function of speed as discussed in detail by Lay (1979, 1985) who showed that this approach is well related to driver behaviour (reaction times, stopping distances, out-of-lane factors, and so on). Lay (1979) proposed a polynomial spacing-speed model which produced all desirable characteristics of a speed-flow relationship except one condition: speeds exceeded the free-flow speed as the flows approached zero in Region A ($v > v_f$ when $q = 0$). Model 3 given here is based on the following spacing-speed relationship that satisfies this condition:

$$L_h = L_{hj} + p_1 \, v \, / \, (1 - v \, / \, v_f)^{p2} \tag{4.3.6}$$
$$\text{subject to } p_1 > 0 \text{ and } p_2 > 0$$

In Model 3, parameter p_1 is related to driver reaction times (including out-of-lane effects) and determines the extra spacing required under low to medium flow conditions in Region A. Parameter p_2 is important in determining the characteristics of the speed-flow model for conditions around the maximum flow rate and in Region B.

4.4 Model 4 Description

This model is based on a function described by Akçelik (1991), and is used in the SIDRA software package for uninterrupted traffic flows (Akçelik and Besley 1999). This model is identical to Model 4 used in ARR 340 (Akçelik, Besley and Roper 1999, Section 10) for uninterrupted flows at signalised intersections.

This model can be used as a general travel time (cost) function for all types of interrupted and uninterrupted traffic facilities for traffic planning purposes. A study in the USA (Dowling, Singh and Cheng 1998) showed the validity of this model, and emphasised its advantage in giving short computing times for iterative traffic assignment purposes.

Model 4 is a general time-dependent model that applies to both unsaturated and oversaturated conditions (see *Section 2.4* for a general discussion on this subject). The basis of the model is a queuing model that predicts delays to vehicles as additional travel time above the zero-flow speed. For undersaturated conditions at *uninterrupted* traffic facilities such as freeways ($q_a < q_n$, $v_f > v_u > v_n$), this delay is related to speed reductions due to interactions between vehicles. Such delays are negligible for high-type freeway facilities until flow rates approach capacity (high x values).

For oversaturated conditions when the demand flow rate exceeds the capacity ($q_a > q_n$), significant queuing delays occur upstream of the bottleneck point. For such conditions, the model requires demand flow rates rather than flow rates measured by detectors. Therefore, for calibration purposes under this project, this model was applied to unsaturated conditions only.

The model is expressed as:

$$v = v_f / \{1 + 0.25\, v_f\, T_f\, [z + sqrt\, (z^2 + m_c\, x\, /\, (q_n\, T_f))]\} \qquad (4.4.1)$$

where v (km/h) is the speed, v_f (km/h) is the free-flow speed, T_f (h) is the duration of the analysis period, m_c is a constant and

$$z = x - 1 \qquad (4.4.1a)$$
$$x = q_a / q_n \qquad (4.4.1b)$$

where q_a (veh/h) is the arrival (demand) flow rate, and q_n (veh/h) is the maximum flow rate.

Parameter m_c is related to other parameters through:

$$m_c = 16\, q_n\, (v_f / v_n - 1)^2 / (v_f^2\, T_f) \qquad (4.4.2)$$

where v_n (km/h) is the speed at maximum flow rate.

The spacing at maximum flow rate, L_{hn} (m/veh) can be calculated from:

$$L_{hn} = 1000\, v_n / q_n \qquad (4.4.3)$$

The model is expressed in the following form for calibration purposes using m_c from *Equation (4.4.2)* in *Equation (4.4.1)*:

$$v = v_f / \{1 + 0.25\, v_f\, T_f\, [z + sqrt\, (z^2 + (16\, (q\, /\, q_n)\, (v_f / v_n - 1)^2 / (v_f T_f)^2))]\} \qquad (4.4.4)$$

where $z = (q\, /\, q_n) - 1$.

Model 4 can also be used to represent Regions A' and C for *interrupted* traffic facilities (see *Section 2.4*). For this purpose, in the equations given above, free-flow speed (v_f) is replaced by zero-flow speed (v_{of}), and maximum flow rate (q_n) is replaced by interrupted traffic capacity (Q_e). Values of parameter m_c for interrupted traffic calculated from *Equation (4.4.2)* differ significantly from the values of m_c for uninterrupted traffic.

Model 4 with Initial Queued Demand

The following extended version of Model 4 has recently been described in Akçelik (1999). *Equation (4.4.1)* assumes that the previous flow period is undersaturated, and therefore there is no *initial queued demand* at the start of the current analysis period. Thus, the initial queued demand is the residual queued demand at the end of the previous oversaturated flow period as shown in *Figure 4.4.1*.

The following equation can be used to allow for the effect of initial queued demand for uninterrupted traffic facilities:

$$v = v_f / \{1 + 0.25 \ v_f \ T_f \ [z + sqrt \ (z^2 + m_c \ x \ / \ (q_n \ T_f) + 2 \ m_c \ N_i \ / \ (q_n \ T_f)^2)]\}$$

(4.4.5)

where v_f, T_f, m_c, x, q_n are as in *Equation (4.4.1)*, and N_i is the initial queued demand observed at the start of the analysis period (vehicles), and parameter z is given by:

$$z = x - 1 + 2 \ N_i / (q_n \ T_f)$$

(4.4.5a)

The model given by *Equation (4.4.1)* is obtained by putting $N_i = 0$ in *Equation (4.4.5)*.
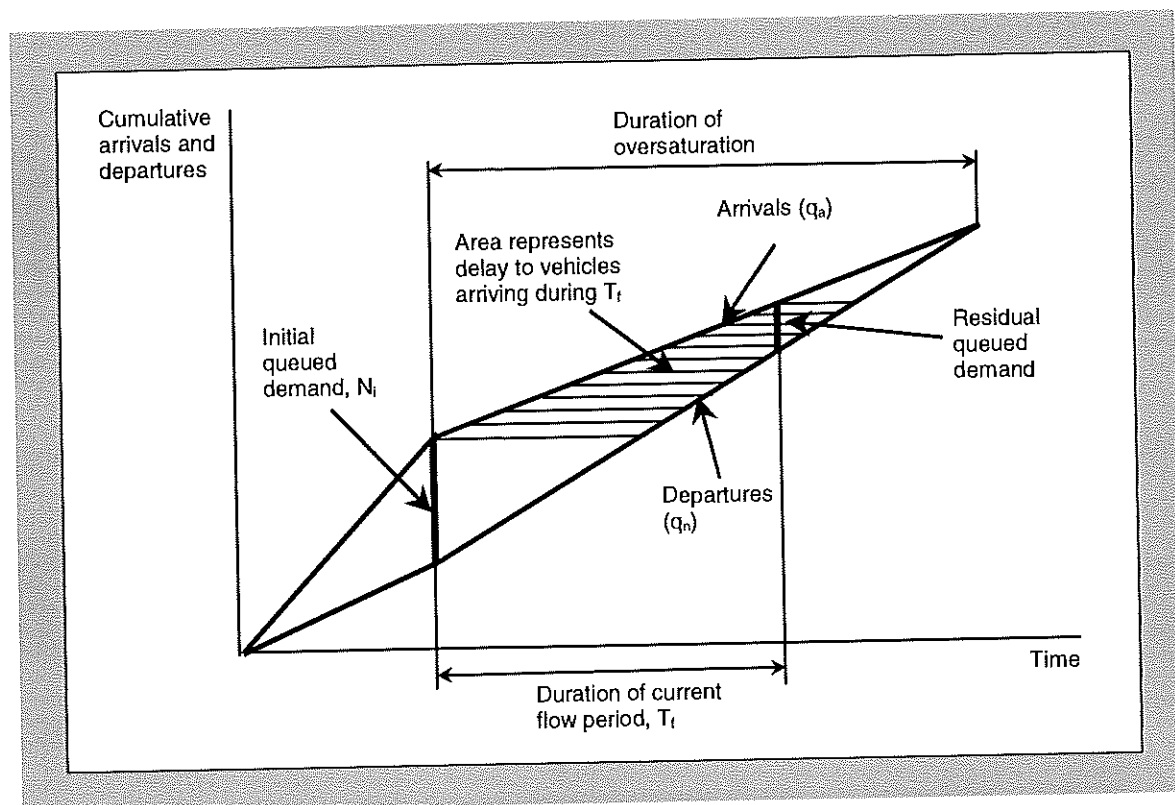
Figure 4.4.1 - Deterministic queuing analysis with initial queued demand for the extended form of Model 4

4.5 Model 5 Description

This is based on the exponential queue discharge speed and flow models for signalised intersections given in ARR 340 (Akçelik, Besley and Roper 1999). For its application to freeway traffic, it is considered as a model for saturated conditions only. The model is expressed as:

$$v = v_n [1 - (1 - q / q_n)^{m_v / m_q}] \qquad (4.5.1)$$

where v (km/h) is the speed, v_n (km/h) is the speed at maximum flow rate, q (veh/h) is the flow rate, q_n (veh/h) is the maximum flow rate, and m_v, m_q are constants related through:

$$m_v = m_q (L_{hj} / L_{hn}) \qquad (4.5.2)$$

where L_{hj} (m/veh) is the jam spacing, and L_{hn} (m/veh) is the spacing at maximum flow rate:

$$L_{hn} = 1000 \, v_n / q_n \qquad (4.5.3)$$

Therefore:

$$m_v / m_q = q_n L_{hj} / (1000 \, v_n) \qquad (4.5.4)$$

The flow rate - speed function corresponding to *Equation (4.5.1)* is:

$$q = q_n [1 - (1 - v / v_n)^{m_q / m_v}] \qquad (4.5.5)$$

From *Equation (4.5.4)*, jam spacing is:

$$L_{hj} = 1000 \, v_n (m_v / m_q) / q_n \qquad (4.5.6)$$

The model is expressed in the following form for calibration purposes using m_v / m_q from *Equation (4.5.4)* in *Equation (4.5.1)*:

$$v = v_n [1 - (1 - q / q_n)^{q_n L_{hj} / (1000 \, v_n)}] \qquad (4.5.7)$$

Model 5 is equivalent to Model 2A (*Section 4.2*) through $m_v / m_q = 1 / p_1$ when p_1 in Model 2A is set to satisfy the constraint $dv/dq = L_{hj}$ at $v = 0$.

4.6 Model 6 Description

This is based on a model given in Akçelik (1974). As in the case of Model 3, this model covers both unsaturated and saturated conditions as a single model. The model is expressed as:

$$q = 1000 \, v \, / \, [\, L_{hj} (1 - (v / v_f)^{p_1})^{p_2}] \tag{4.6.1}$$

$$subject \ to \ 0 < p_1 < 1.0 \ and \ -1.0 < p_2 < 0$$

where q (veh/h) is the flow rate, v (km/h) is the speed, v_f (km/h) is the free-flow speed, L_{hj} (m/veh) is the jam spacing, and p_1, p_2 are constants related through:

$$p_1 = 1 \, / \, [\, p_2 (1 - (L_{hj} / L_{hn})^{1/p_2})] \tag{4.6.2}$$

$$p_2 = [1 - (v_f / v_n)^{p_1}] \, / \, p_1 \tag{4.6.2a}$$

where v_n (km/h) is the speed at maximum flow rate, L_{hn} (m/veh) is the spacing at maximum flow rate calculated from:

$$L_{hn} = 1000 \, v_n \, / \, q_n \tag{4.6.3}$$

where q_n (veh/h) is maximum flow rate.

Thus, model parameters p_1 and p_2 together determine the following values of the ratio of speed at maximum flow to free-flow speed, and the ratio of the spacing at maximum flow to jam spacing:

$$L_{hn} / L_{hj} = [1 - 1 / (p_1 p_2)]^{-p_2} \tag{4.6.3a}$$

$$v_n / v_f = 1 / (1 - p_1 p_2)^{1/p_1} \tag{4.6.3b}$$

This model is based on the following vehicle spacing-speed relationship:

$$L_h = L_{hj} [1 - (v / v_f)^{p_1}]^{p_2} \tag{4.6.4}$$

Therefore, spacing at maximum flow is given by:

$$L_{hn} = L_{hj} [1 - (v_n / v_f)^{p_1}]^{p_2} \tag{4.6.5}$$

From *Equation (4.6.5)*, jam spacing is:

$$L_{hj} = L_{hn} [1 - (v_n / v_f)^{p_1}]^{-p_2} \tag{4.6.6}$$

The model is expressed in the following form for calibration purposes using L_{hj} from *Equation (4.6.6) and* L_{hn} from *Equation (4.6.3)* in *Equation (4.6.1)*:

$$q = q_n (v / v_n) [(1 - (v_n / v_f)^{p_1}) / (1 - (v / v_f)^{p_1})]^{p_2} \tag{4.6.7}$$

$$subject \ to \ 0 < p_1 < 1.0 \ and \ -1.0 < p_2 < 0$$

Derivation of Model 6

Model 6 is derived from the following differential equation based on car-following theory:

$$dv/dk = -c\, v^m\, k^{l-2}$$ (4.6.8)

where v is speed, k is density, l and m are model parameters (constants), and c is a parameter that is determined according to the values of l and m. The speed-density model is determined by integrating *Equation (4.6.8)* and applying various boundary conditions.

1. $v = v_f$ for $k = 0$ (also $q = 0$)

2. $v = 0$ for $k = k_j$ (also $q = 0$, $L_h = L_{hj}$)

3. $dq/dk = 0$ for $k = k_n$ (where $v = v_n$, $q = q_n$, $L_h = L_{hn}$)

4. $dq/dk = v_f$ for $k = 0$

5. $dq/dk = 0$ for $k = k_j$ (also $q = 0$, $L_h = L_{hj}$)

6. $dv/dk = 0$ for $k = 0$

7. $dv/dk = 0$ for $k = k_j$ (also $q = 0$, $L_h = L_{hj}$)

8. $dv/dq = L_{hj}$ for $v = 0$ (also $q = 0$, $k = k_j$)

Different models can be derived by selecting different values of parameters l and m. For conditions 1 to 4, the range of parameters are limited to $1.0 > m \geq 0$ and $l > 1.0$. To satisfy all conditions, the range of parameters would be limited to $1.0 > m > 0$ and $l > 2.0$.

It is best to test the boundary conditions using models with speed, v in m/s, flow rate, q in veh/s and density, k in veh/m so that $q = v\, k$ and $L_h = 1/k$ relationships apply with correct units.

Model 6 given by *Equation (4.6.1)* is obtained by using parameters:

$$p_1 = 1 - m$$ (4.6.9)

$$p_2 = -1 / (l - 1)$$

Therefore the following constraints apply to these parameters:

$$1.0 > p_1 > 0$$ (4.6.10)

$$0 > p_2 > -1.0$$

Table 4.6.1 summarises parameter values and implied v_n / v_f, L_{hn} / L_{hj} and $k_n / k_j = L_{hj} / L_{hn}$ ratios for several traditional models that can be derived from the general form of Model 6.

Table 4.6.1

Deriving various models given in the literature from the general form of Model 6

Model	l	m	$p_1 = 1 - m$	$p_2 = -1/(l-1)$	v_n/v_f	L_{hn}/L_{hj}	k_n/k_j
Parabolic (Drew 1965, 1965-66)	1.5	0	1.00	-2.000	0.333	2.250	0.444
Linear (Greenshields 1934)	2.0	0	1.00	-1.000	0.500	2.000	0.500
Parabolic (Pipes 1967)	3.0	0	1.00	-0.500	0.667	1.732	0.577
Parabolic (Pipes 1967)	2.0	0.5	0.50	-1.000	0.444	3.000	0.333
Polynomial (Akcelik 1974)	2.5	0.5	0.50	-0.667	0.563	2.520	0.397
Polynomial for freeway traffic (May and Keller 1967, 1968)	2.8	0.8	0.20	-0.556	0.590	3.594	0.278
Polynomial for tunnel traffic (May and Keller 1967, 1968)	2.1	0.6	0.40	-0.909	0.461	3.325	0.301

5 MODEL CALIBRATION RESULTS

Six analytical flow models for freeways described in *Section 4* include various single-regime and two-regime models. A single-regime model is a single function that applies to both saturated and unsaturated flow conditions. A two-regime model consists of two functions that apply to saturated and unsaturated flow conditions separately.

This section presents the model calibration results for the Eastern Freeway site. A constrained non-linear regression analysis method was used for model calibration. Regression analyses were carried out using data aggregated for 5-min intervals. This interval was chosen as the most appropriate interval for the purpose of evaluating alternative models.

Several regression options are tried for each model with different model parameters specified. Results for Models 1 to 6 are presented in *Sections 5.1 to 5.6*. A discussion of the calibration results is given in *Section 5.7*.

In *Section 5.8*, graphs showing fundamental relationships with measured data and model predictions are presented for Model 4+5 (combination of Model 4 for unsaturated conditions and Model 5 for saturated conditions).

Free-Flow Speed

Before regression analysis, the free flow speed (v_f) was estimated by means of a simple trendline analysis using Excel. For this purpose, high speed data with average vehicle spacing over 60 m were selected. This coincided with flow rates below about 1600 veh/h in the unsaturated flow region. Trendline equation is $v_f = 101.08 - 0.0005\ q$, predicting v_f values in the range 101.0 to 101.4 km/h. Thus, speeds for low to medium flow rates in unsaturated conditions are fairly constant. This is in line with the method recommended in the US Highway Capacity Manual which defines the free-flow speed as the "mean speed of passenger cars measured under low to moderate flows up to 1300 passenger car unit per hour per lane" (Schoen, et al. 1995; TRB 1998). The results obtained here included some heavy vehicle data but the effect is expected be minimal due to the low percentage of heavy vehicles in the stream (3 %).

Average vehicle length for the Eastern Freeway data was estimated to be $L_v = 4.4$ m. This also indicates the dominance of light vehicles.

In most cases, $v_f = 101$ km/h was specified as input in the regression analyses. When not specified, v_f in the range 96 to 102 km/h were found by regression analyses for different models ($v_f = 100.5$ km/h by Model 2A and $v_f = 100.3$ km/h by Model 4 provided the best predictions). Thus, the free-flow speed is found to be very close to the speed limit of 100 km/h for this freeway.

Regression Method

Speed-flow regressions were performed using the SPSS for Windows package (Norusis and SPSS Inc. 1993). SPSS Version 8.0 (1997) was used for the analyses. The Constrained Non-Linear Regression (CNLR) procedure was used for all options of all models. This procedure allows constraints for each of the estimated parameters to be defined to ensure that only estimates within a reasonable range of values are returned. No data weighting was used in the analyses.

Regression results indicated that, in most cases, confidence intervals span very large ranges. Norusis (1993, p. 221) explains that it is not possible to obtain exact confidence intervals with non-linear regression. More importantly, small number of data points in the aggregated data set is likely to have caused this problem.

With the non-linear regression method employed, it was found necessary to provide realistic starting values for the estimated parameters prior to the regression analysis, otherwise the regression tended to return an invalid result.

Parameter Constraints

The following constraints were applied in regression analyses based on the preliminary analysis of this data set:

(i)   Estimate of the free-flow speed (v_f) was restricted to the range $90 < v_f < 110$ km/h.

(ii)   Estimate of the speed at maximum flow (v_n) was restricted to the range $60 < v_n < 90$ km/h.

(iii)   Estimate of the maximum flow (q_n) was restricted to the range $2200 < q_n < 2800$ veh/h.

(iv)   Estimate of the jam spacing (L_{hj}) was restricted to the range $L_{hj} > 6$ m/veh (equivalent to a jam density of 167 veh/km).

(v)   Model 1 parameters were restricted to $p_1 > 0$ and $p_2 > 0$.

(vi)   Model 2 parameters were restricted to $p_1 > 0$, and $p_2 > 0$.

(vii)   Model 3 parameters were restricted to $p_1 > 0$, and $p_2 > 0$.

(viii) Model 5 parameter was restricted to $m_v/m_q > 0$ (results from other parameters estimated).

(ix)   Model 6 parameters were restricted to $0 < p_1 < 1.0$, and $-1.0 < p_2 < 0$.

In Model 4, parameter $m_c > 0$ results from the other parameters estimated.

In the case of the two-piece models 1 and 2, simultaneous regressions were performed on both sections of the model. This is possible by combining the model equation with a logical expression that defines which cases should be used for each section of the model. In this case, the actual speed variable (v_u) was used to select which cases to use with each section based on the starting value of v_n, usually 85.

When v_f was specified, data points with $v > v_f$ were eliminated from regression.

Model Details

The speed - flow equation used for regression purposes is given for each model. Refer to *Section 4* for detailed description of each model. The spacing at maximum flow was always calculated from the general relationship $L_{hn} = 1000\ v_n / q_n$. The symbols used for traffic parameters in *Sections 5.1 to 5.6* are:

v_f : free-flow speed (km/h)

v_n : speed at maximum flow rate (km/h)

q_n : maximum flow rate (veh/h)

L_{hj} : jam spacing (m/veh)

L_{hn} : spacing at maximum flow rate (m/veh)

T_f : duration of the analysis (data aggregation) period (h) used in Model 4 only.

In *Tables 5.1 to 5.6*, 95% Confidence Intervals (C.I.) are given for all estimated parameters, except when they are too large to display.

5.1   Model 1 Calibration Results

Model 1 consists of Model 1A for unsaturated conditions and Model 1B for saturated conditions which are calibrated together as one model (the same value of v_n and q_n apply).

Model 1A:

$$v = v_f - (v_f - v_n) (q / q_n)^{p2} \tag{5.1.1}$$
$$subject\ to\ p_2 > 0$$

Model 1B:

$$v = L_{hj} (q / 1000) + (v_n - L_{hj}\ q_n / 1000) (q / q_n)^{p1} \tag{5.1.2}$$
$$subject\ to\ p_1 > 0$$

The following calibration options were tested:

a. Specify: none. Estimate v_f, v_n, q_n, L_{hj}, p_1, p_2. Calculate a_1, b_1, L_{hn}.

b. Specify $v_f = 101$ km/h. Estimate v_n, q_n, L_{hj}, p_1, p_2. Calculate a_1, b_1, L_{hn}.

c. Specify $v_n = 85$ km/h. Estimate v_f, q_n, L_{hj}, p_1, p_2. Calculate a_1, b_1, L_{hn}.

d. Specify $q_n = 2500$ veh/h. v_f, v_n, L_{hj}, p_1, p_2. Calculate a_1, b_1, L_{hn}.

e. Specify $v_f = 101$ km/h, $v_n = 85$ km/h. Estimate L_{hj}, q_n, p_1, p_2. Calculate a_1, b_1, L_{hn}.

f. Specify $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate v_f, L_{hj}, p_1, p_2. Calculate a_1, b_1, L_{hn}.

g. Specify $v_f = 101$ km/h, $q_n = 2500$ veh/h. Estimate v_n, L_{hj}, p_1, p_2. Calculate a_1, b_1, L_{hn}.

h. Specify $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate L_{hj}, p_1, p_2. Calculate a_1, b_1, L_{hn}.

i. Specify $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m), $L_{hj} = 7.5$ m/veh. Estimate v_f, p_1, p_2. Calculate a_1, b_1, L_{hn}.

j. Specify $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m), $L_{hj} = 7.5$ m/veh. Estimate p_1, p_2. Calculate a_1, b_1, L_{hn}.

The results are presented in *Table 5.1*.

Table 5.1

Parameters for Model 1 (combined Models 1A and 1B)
with various calibration options

Option	V_f	V_n	q_n	L_{hj}	p_1	p_2	a_1	b_1	L_{hn}
a	99.7	73.0	2588	12.3	2.55	26.9	3.65	-2.1E-10	28.2
95% C.I.	98.3 - 101.0	50.7 - 95.2	2429 - 2478	-87 - 112	-11.8 - 16.9	-27.8 - 81.6	-	-	-
	Estimated	Estimated	Estimated	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
b	101	74.7	2620	12.2	2.55	16.3	3.67	-4E-06	28.5
95% C.I.	-	51.5 - 97.8	2441 - 2799	-92.7 - 117.2	-12.5 - 17.6	0.576 - 32.0	-	-	-
	Specified	Estimated	Estimated	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
c	99.7	85	2550	20.5	8.37	21.4	0.0130	-3E-08	33.3
95% C.I.	98.3 - 101.1	-	2342 - 2758	14.3 - 26.6	-8.4 - 25.1	-12.0 - 54.7	-	-	-
	Estimated	Specified	Estimated	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
d	99.7	88.3	2500	21.3	12.3	25.3	0.0005	-1E-09	35.3
95% C.I.	98.3 - 101.1	65.4 - 111.1	-	17.9 - 24.8	-8.28 - 32.8	-18.9 - 69.4	-	-	-
	Estimated	Estimated	Specified	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
e	101	85	2575	20.1	7.4	13.4	0.0294	-5E-05	33.0
95% C.I.	-	-	2297 - 2853	12.1 - 28.1	-9.56 - 24.4	-0.187 - 27.0	-	-	-
	Specified	Specified	Estimated	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
f	99.7	85	2500	21.1	10.8	22.8	0.0016	-1.2E-08	34.0
95% C.I.	98.3 - 101.1	-	-	16.9 - 25.2	-8.29 - 29.9	-3.7 - 49.3	-	-	-
	Estimated	Specified	Specified	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
g	101	87.9	2500	21.3	12.1	16.7	0.0005	-2.9E-06	35.1
95% C.I.	-	67.8 - 107.9	-	17.7 - 25.0	-9.2 - 33.4	0.758 - 32.7	-	-	-
	Specified	Estimated	Specified	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
h	101	85	2500	21.1	10.8	17.6	0.0016	-1.6E-06	34.0
95% C.I.	-	-	-	16.7 - 25.4	-9.1 - 30.7	3.1 - 32.1	-	-	-
	Specified	Specified	Specified	Estimated	Estimated	Estimated	Calculated	Calculated	Calculated
i	99.8	85	2500	7.5	3.42	22.8	2.88	-1.2E-08	34.0
95% C.I.	98.1 - 101.4	-	-	-	-4.38 - 11.2	-7.1 - 52.7	-	-	-
	Estimated	Specified	Specified	Specified	Estimated	Estimated	Calculated	Calculated	Calculated
j	101	85	2500	7.5	3.42	17.6	2.88	-1.6E-06	34.0
95% C.I.	-	-	-	-	-4.6 - 11.5	1.35 - 33.9	-	-	-
	Specified	Specified	Specified	Specified	Estimated	Estimated	Calculated	Calculated	Calculated

5.2 Model 2 Calibration Results

Model 2 consists of Model 2A for unsaturated conditions and Model 2B for saturated conditions which are calibrated together as one model (the same value of v_n and q_n apply).

Model 2A:

$$q = q_n [1 - ((v - v_n) / (v_f - v_n))^{p_2}]$$
$$\text{subject to } p_2 > 0$$

(5.2.1)

Model 2B:

$$q = q_n [1 - (1 - v / v_n)^{p_1}]$$
$$\text{subject to } p_1 > 0$$

(5.2.2)

The following calibration options were tested:

a. Specify: none. Estimate v_f, v_n, q_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

b. Specify $v_f = 101$ km/h. Estimate v_n, q_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

c. Specify $v_n = 85$ km/h. Estimate v_f, q_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

d. Specify $q_n = 2500$ veh/h. Estimate v_f, v_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

e. Specify $v_f = 101$ km/h, $v_n = 85$ km/h. Estimate q_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

f. Specify $v_f = 101$ km/h, $q_n = 2500$ veh/h. Estimate v_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

g. Specify $v_n = 85$ km/h, $q_n = 2500$ veh/h. Estimate v_f, p_1, p_2. Calculate L_{hn}, L_{hj}.

h. Specify $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate p_1, p_2. Calculate L_{hn}, L_{hj}.

The results are presented in *Table 5.2*.

Table 5.2

*Parameters for Model 2 (combined Models 2A and 2B)
with various calibration options*

Option	V_f	V_n	q_n	p_1	p_2	L_{hj}	L_{hn}
a 95% C.I.	100.5 96.6 - 104.3 Estimated	83.4 -269 - 436 Estimated	2513 -118 - 5144 Estimated	2.01 -5.7 - 9.7 Estimated	21.5 -557 - 600 Estimated	16.5 - Calculated	33.2 - Calculated
b 95% C.I.	101 - Specified	84.0 -218 - 386 Estimated	2513 314 - 4711 Estimated	2.03 -4.7 - 8.7 Estimated	12.8 -290 - 316 Estimated	16.5 - Calculated	33.4 - Calculated
c 95% C.I.	100.2 96.9 - 103.4 Estimated	85 - Specified	2513 -150 - 5175 Estimated	2.07 -6.5 - 10.6 Estimated	29.1 -886 - 944 Estimated	16.4 - Calculated	33.8 - Calculated
d 95% C.I.	100.3 96.8 - 103.4 Estimated	81.8 -245 - 409 Estimated	2500 - Specified	1.97 -5.3 - 9.2 Estimated	26.3 -590 - 643 Estimated	16.6 - Calculated	32.7 - Calculated
e 95% C.I.	101 - Specified	85 - Specified	2513 320 - 4705 Estimated	2.06 -4.9 - 9.0 Estimated	12.7 -314 - 339 Estimated	16.5 - Calculated	33.8 - Calculated
f 95% C.I.	101 - Specified	82.1 -207 - 371 Estimated	2500 - Specified	1.99 -4.4 - 8.4 Estimated	15.4 -305 - 336 Estimated	16.6 - Calculated	32.8 - Calculated
g 95% C.I.	100.4 96.7 - 104.0 Estimated	85 - Specified	2500 - Specified	2.1 -5.8 - 10.0 Estimated	21.3 -583 - 626 Estimated	16.2 - Calculated	34.0 - Calculated
h 95% C.I.	101 - Specified	85 - Specified	2500 - Specified	2.1 -4.8 - 9.0 Estimated	12.9 -301 - 327 Estimated	16.2 - Calculated	34.0 - Calculated

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5.3 Model 3 Calibration Results

Model 3 is a single regime model that covers both unsaturated and saturated conditions:

$$q = 1000 / [(L_{hj} / v) + (1000 / q_n - L_{hj} / v_n) ((1 - v_n / v_f) / (1 - v / v_f))^{p_2}] \quad (5.3.1)$$

subject to $p_2 > 0$

The following calibration options were tested with Model 3:

a. Specify: none. Estimate v_f, v_n, q_n, L_{hj}, p_2. Calculate p_1, L_{hn}.

b. Specify $v_f = 101$ km/h. Estimate v_n, q_n, L_{hj}, p_2. Calculate p_1, L_{hn}.

c. Specify $v_f = 101$ km/h, $v_n = 85$ km/h. Estimate q_n, L_{hj}, p_2. Calculate p_1, L_{hn}.

d. Specify $v_f = 101$ km/h, $q_n = 2500$ veh/h. Estimate v_n, L_{hj}, p_2. Calculate p_1, L_{hn}.

e. Specify $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate L_{hj}, p_2. Calculate p_1.

f. Specify $v_n = 85$ km/h $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m) and $L_{hj} = 7.5$ m. Estimate p_2 and v_f. Calculate p_1.

g. Specify: $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m) and $L_{hj} = 7.5$ m. No estimation. Calculate p_1 and p_2.

The results are presented in *Table 5.3*. With Option g, $p_1 = 0.279$ and $p_2 = 0.060$ were found. These parameters did not produce good predictions for saturated conditions due to the low value of jam spacing specified ($L_{hj} = 7.5$ m). With $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n. = 2500$ veh/h (therefore $L_{hn} = 34.0$ m) and $L_{hj} = 12.0$ m specified, $p_1 = 0.215$ and $p_2 = 0.100$ were found to give improved estimates.

Table 5.3

Parameters for Model 3 (saturated and unsaturated conditions)
with various calibration options

Option	V_f	V_n	q_n	L_{hj}	p_2	p_1	L_{hn}
a 95% C.I.	97.1 -83 - 277 Estimated	65.8 * Estimated	2226 * Estimated	10.3 -6.4 - 27.0 Estimated	0.155 -1.8 - 2.1 Estimated	0.246 - Calculated	29.6 - Calculated
b 95% C.I.	101 - Specified	80.1 * Estimated	2212 * Estimated	10.5 2.7 - 18.3 Estimated	0.186 0.044 - 0.329 Estimated	0.239 - Calculated	36.2 - Calculated
c 95% C.I.	101 - Specified	85 - Specified	2200 * Estimated	11.1 3.5 - 18.6 Estimated	0.197 0.05 - 0.344 Estimated	0.226 - Calculated	39.6 - Calculated
d 95% C.I.	101 - Specified	76.1 * Estimated	2500 - Specified	14.2 8.0 - 20.4 Estimated	0.287 0.106 - 0.468 Estimated	0.143 - Calculated	30.4 - Calculated
e 95% C.I.	101 - Specified	85 - Specified	2500 - Specified	15.4 9.3 - 21.5 Estimated	0.314 0.110 - 0.517 Estimated	0.123 - Calculated	34.0 - Calculated
f 95% C.I.	101 - Specified	66.5 * Estimated	2500 - Specified	7.5 - Specified	0.204 0.057 - 0.351 Estimated	0.231 - Calculated	26.6 - Calculated
g 95% C.I.	101 - Specified	85 - Specified	2500 - Specified	7.5 - Specified	0.279 - Calculated	0.060 - Calculated	34.0 - Calculated

Note: 95% C.I. values marked with an asterisk are too large to display.

5.4 Model 4 Calibration Results

Although Model 4 is a general time-dependent model that applies to both unsaturated and oversaturated conditions (demand flow rate larger than the maximum flow rate), calibration was carried out for unsaturated conditions only (speed less than the speed at maximum flow rate) due to the nature of data available. The simple form of the model with no initial queued demand was used as relevant to unsaturated conditions.

$$v = v_f / \{1 + 0.25\, v_f\, T_f\, [z + sqrt\, (z^2 + (16\, (q/q_n)\, (v_f/v_n - 1)^2 / (v_f T_f)^2))]\}$$
(5.4.1)

where $z = (q/q_n) - 1$.

The following calibration options were tested with Model 4:

a. Specify: $T_f = 0.0833$ h (5-min analysis period). Estimate v_f, v_n, q_n. Calculate m_c, L_{hn}.

b. Specify $T_f = 0.0833$ h, $v_f = 101$ km/h. Estimate v_n, q_n. Calculate m_c, L_{hn}.

c. Specify $T_f = 0.0833$ h, $v_f = 101$ km/h, $v_n = 85$ km/h. Estimate q_n. Calculate m_c, L_{hn}.

d. Specify $T_f = 0.0833$ h, $v_f = 101$ km/h, $q_n = 2500$ veh/h. Estimate v_n. Calculate m_c, L_{hn}.

e. Specify $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate v_f. Calculate m_c.

f. Specify $T_f = 0.0833$ h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate v_f. Calculate m_c.

g. Specify $T_f = 0.0833$ h, $v_f = 101$ km/h, $v_n = 90$ km/h, $q_n = 2500$ veh/h. No estimation. Calculate m_c, L_{hn}.

The results are presented in *Table 5.4*.

Calibration of the Following Steady-State Form of Model 4

Calibration of the following *steady-state* form of Model 4 was also undertaken in order to estimate parameter m_c independently of the flow period (T_f):

$$v = v_f / [1 + v_f\, m_c / (8\, q_n\, (q_n/q - 1))]$$
$$subject\ to\ q/q_n < 0.95$$
(5.4.2)

This method requires elimination of data points with flow rates greater than 95 per cent of the maximum flow rate. When q_n is not specified (i.e. to be estimated from regression), an iterative process is performed where an initial estimate of q_n is provided.

The steady-state form of Model 4 assumes that the demand flow period lasts for an infinite period of time. As a result, the function is defined only for conditions below the maximum flow rate. This requires the calculation of the speed at maximum flow rate, v_n (km/h) from the time-dependent form:

$$v_n = v_f / (1 + 0.25\, v_f\, sqrt\, (m_c\, T_f / q_n))$$
(5.4.3)

The following calibration results were obtained for the steady-state form of Model 4 ($T_f = 0.0833$ h was used for calculating v_n for all calibration options listed).

(i)   With no parameters specified, $v_f = 100.2$ km/h, $q_n = 2500$ veh/h, $m_c = 0.70$ were estimated, and $v_n = 89.1$ km/h and $L_{hn} = 35.7$ m were calculated. This supported calibration results using the time-dependent form of the model.

(ii)   With $v_f = 101$ km/h specified, $q_n = 2800$ veh/h, $m_c = 2.50$ were estimated, and $v_n = 82.9$ km/h and $L_{hn} = 29.6$ m were calculated. The capacity estimate was too high in this case.

(iii) With $q_n = 2500$ veh/h specified, $v_f = 99.9$ km/h, $m_c = 0.21$ were estimated, and $v_n = 93.6$ km/h and $L_{hn} = 37.5$ m were calculated.

(iv)   With $v_f = 101$ km/h, $q_n = 2500$ veh/h specified, $m_c = 0.64$ was estimated, and $v_n = 90.4$ km/h and $L_{hn} = 36.2$ m were calculated.

Generally, these results support the calibration results obtained using the time-dependent form of Model 4 (given in *Table 5.4*) although the use of the steady-state form of the model suffered from the lack of reduced speed values as the flow rate approached capacity (resulting from the constraint $q / q_n < 0.95$).

Table 5.4

*Parameters for Model 4 (unsaturated conditions)
with various calibration options*

Option	V_f	V_n	q_n	T_f	m_c	L_{hn}
a 95% C.I.	100.3 97.3 - 103.3 Estimated	90.0 -65 - 245 Estimated	2491 48 - 4933 Estimated	**0.0833** - **Specified**	0.625 - Calculated	36.1 - Calculated
b 95% C.I.	**101** - **Specified**	90.0 -51 - 231 Estimated	2487 254 - 4721 Estimated	**0.0833** - **Specified**	0.70 - Calculated	36.2 - Calculated
c 95% C.I.	**101** - **Specified**	**85** - **Specified**	2629 -1000 - 6259 Estimated	**0.0833** - **Specified**	1.754 - Calculated	32.3 - Calculated
d 95% C.I.	**101** - **Specified**	89.5 -63 - 242 Estimated	**2500** - **Specified**	**0.0833** - **Specified**	0.777 - Calculated	35.8 - Calculated
e 95% C.I.	**101** - **Specified**	**85** - **Specified**	**2500** - **Specified**	0.1763 -4.8 - 5.24 Estimated	0.788 - Calculated	34.0 - Calculated
f 95% C.I.	101.3 97.1 - 105.5 Estimated	**85** - **Specified**	**2500** - **Specified**	**0.0833** - **Specified**	1.717 - Calculated	34.0 - Calculated
g 95% C.I.	**101** - **Specified**	**90** - **Specified**	**2500** - **Specified**	**0.0833** - **Specified**	0.703 - Calculated	36.0 - Calculated

5.5 Model 5 Calibration Results

This model applies for saturated conditions only. The model is expressed as:

$$v = v_n [1 - (1 - q / q_n)^{q_n L_{hj} / (1000\, v_n)}]$$
(5.5.1)

The following calibration options were tested with Model 5:

a. Specify: none. Estimate: v_n, q_n, L_{hj}. Calculate L_{hn}, m_v/m_q.

b. Specify $v_n = 85$ km/h. Estimate q_n, L_{hj}. Calculate L_{hn}, m_v/m_q.

c. Specify $v_n = 85$ km/h, $q_n = 2500$ veh/h. Estimate L_{hj}. Calculate m_v/m_q.

d. Specify: $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate L_{hj}. Calculate m_v/m_q.

e. Specify: $v_n = 85$ km/h, $L_{hj} = 7.5$ m. Estimate q_n. Calculate L_{hn}, m_v/m_q.

f. Specify $v_n = 90$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 36.0$ m) and $L_{hj} = 15.0$ m. No estimation. Calculate L_{hn}, m_v/m_q.

An additional option was also tested:

d1. Specify: $v_n = 90$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate L_{hj}. Calculate m_v/m_q.

The results are presented in *Table 5.5*.

Table 5.5

Parameters for Model 5 (saturated conditions)
with various calibration options

Option	V_n	q_n	L_{hj}	m_v/m_q	L_{hn}
a 95% C.I.	90.0 -993 - 1174 Estimated	2570 -4858 - 9999 Estimated	16.5 -9.9 - 42.8 Estimated	0.470 - Calculated	35.0 - Calculated
b 95% C.I.	85 - Specified	2538 -3982 - 9057 Estimated	16.6 -8.5 - 41.7 Estimated	0.496 - Calculated	33.5 - Calculated
c 95% C.I.	81.7 -58.9 - 75.2 Estimated	2500 - Specified	16.6 -6.4 - 39.5 Estimated	0.507 - Calculated	32.7 - Calculated
d 95% C.I.	85 - Specified	2500 - Specified	16.2 -5.9 - 39.5 Estimated	0.477 - Calculated	34.0 - Calculated
d1 95% C.I.	90 - Specified	2500 - Specified	18.0 -40.0 - 26.3 Estimated	0.499 - Calculated	34.0 - Calculated
e 95% C.I.	85 - Specified	2200 1891 - 2508 Estimated	7.5 - Specified	0.194 - Calculated	38.6 - Calculated
f 95% C.I.	90 - Specified	2500 - Specified	15.0 - Specified	0.417 - Calculated	36.0 - Calculated

5.6 Model 6 Calibration Results

Like Model 3, this is a single regime model that covers both unsaturated and saturated conditions:

$$q = q_n (v/v_n) [(1-(v_n/v_f)^{p_1})/(1-(v/v_f)^{p_1})]^{p_2} \qquad (5.6.1)$$

subject to $0 < p_1 < 1.0$ and $-1.0 < p_2 < 0$

using

$$p_1 = 1/[p_2(1-(L_{hj}/L_{hn})^{1/p_2})] \qquad (5.6.2)$$

The following calibration options were tested with Model 6:

a. Specify: none. Estimate: v_f, v_n, q_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

b. Specify $v_f = 101$ km/h. Estimate: v_n, q_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

c. Specify $v_f = 101$ km/h, $v_n = 85$ km/h. Estimate q_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

d. Specify $v_f = 101$ km/h, $q_n = 2500$ veh/h. Estimate v_n, p_1, p_2. Calculate L_{hn}, L_{hj}.

e. Specify $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m). Estimate p_1, p_2. Calculate L_{hj}.

f. Specify $v_f = 101$ km/h, $q_n = 2500$ veh/h and $L_{hj} = 7.5$ m. Estimate p_2 and v_f. Calculate p_1.

g. Specify: $v_f = 101$ km/h, $v_n = 85$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 34.0$ m) and $L_{hj} = 7.5$ m. No estimation. Calculate p_1 and p_2.

The results are presented in *Table 5.6*. With Option g, $p_1 = 0.0009$ and $p_2 = -0.172$ were found. These parameters did not produce good predictions for saturated conditions. This could be improved using a lower value of v_n. With $v_f = 101$ km/h, $v_n = 80$ km/h, $q_n = 2500$ veh/h (therefore $L_{hn} = 32.0$ m) and $L_{hj} = 7.5$ m specified, $p_1 = 0.0087$ and $p_2 = -0.234$ were found to give improved estimates.

Table 5.6

Parameters for Model 6 (saturated and unsaturated conditions) with various calibration options

Option	V_f	V_n	q_n	p_1	p_2	L_{hj}	L_{hn}
a 95% C.I.	102.0 99.1 - 104.9	75.1 *	2461 *	0.014 -4.9 - 4.9	-0.298 -0.55 - -0.042	6.0 -	30.5 -
	Estimated	Estimated	Estimated	Estimated	Estimated	Calculated	Calculated
b 95% C.I.	101 -	83.6 *	2485 *	0.005 -4.25 - 4.26	-0.248 -0.372 - -0.124	6.0 -	33.7 -
	Specified	Estimated	Estimated	Estimated	Estimated	Calculated	Calculated
c 95% C.I.	101 -	85 -	2470 *	0.005 -4.26 - 4.27	-0.248 -0.368 - -0.127	6.0 -	34.4 -
	Specified	Specified	Estimated	Estimated	Estimated	Calculated	Calculated
d 95% C.I.	101 -	81.4 *	2500 -	0.005 -4.15 - 4.16	-0.248 -0.365 - -0.131	6.0 -	32.6 -
	Specified	Estimated	Specified	Estimated	Estimated	Calculated	Calculated
e 95% C.I.	101 -	85 -	2500 -	0.0056 -4.33 - 4.34	-0.250 -0.374 - -0.126	6.0 -	34.0 -
	Specified	Specified	Specified	Estimated	Estimated	Calculated	Calculated
f 95% C.I.	101 -	76.1 *	2500 -	0.0125 -4.19 - 4.22	-0.248 -0.366 - -0.130	7.5 -	-
	Specified	Estimated	Specified	Estimated	Estimated	Specified	Calculated
g 95% C.I.	101 -	85 -	2500 -	0.0009 -	-0.172 -	7.5 -	34.0 -
	Specified	Specified	Specified	Calculated	Calculated	Specified	Calculated

Note: 95% C.I. values marked with an asterisk are too large to display.

5.7 Discussion of Calibration Results

Calibration results for six analytical models presented in *Sections 5.1 to 5.6* for the survey site on the Eastern Freeway in Melbourne on the basis of average traffic stream data using 5-min aggregation period are summarised in *Table 5.7*. The results are given for the case when the free-flow speed of $v_f = 101$ km/h is specified and other parameters are estimated using the constrained non-linear regression method (Calibration Option b for each model). The free-flow specified is based on direct estimation of this parameter using high-speed data with average vehicle spacing over 60 m.

Although all models gave generally satisfactory results, two-regime models provided more flexibility in calibration against saturated and unsaturated conditions, especially when some parameters were specified. It was noted that Models 3 and 6 were "lazy" in terms of flow rates calculated when speed approaches the free-flow speed. For example, Model 6 gives $q = 1600$ veh/h for $v / v_f = 0.999$. Because of this feature, the use of Models 3 and 6 could be limited to the saturated region only as a component of two-regime models.

By inspecting the graphs of predicted traffic flow parameters against measured values (see *Section 5.8*), it was found that Models 4 and 5 together as a two-regime model ("Model 4+5") provided the best estimates of traffic characteristics for this site. The parameter values selected as representing traffic at the survey site, based on the use of Model 4+5, are given in *Table 5.7*.

Table 5.7

Summary of parameters for Models 1 to 6 for Calibration Option b with the free-flow speed specified as $v_f = 101$ km/h ($T_f = 0.0833$ h for Model 4)

Option	v_f	v_n	v_n / v_f	q_n	L_{hj}	L_{hn}
Model 1	101 Specified	75 Estimated	0.74 Calculated	2620 Estimated	12.2 Estimated	28.2 Calculated
Model 2	101 Specified	84 Estimated	0.83 Calculated	2513 Estimated	16.5 Calculated	33.4 Calculated
Model 3	101 Specified	80 Estimated	0.79 Calculated	2212 Estimated	10.5 Calculated	36.2 Calculated
Model 4	101 Specified	90 Estimated	0.89 Calculated	2487 Estimated	-	36.2 Calculated
Model 5	-	90 Estimated	-	2570 Estimated	16.5 Estimated	35.0 Calculated
Model 6	101 Specified	84 Estimated	0.83 Calculated	2485 Estimated	6.0 Calculated	Calculated
Selected (Model 4+5)	101 Specified	90 Specified	0.89 Calculated	2500 Specified	15.0 Specified	36.0 Calculated

In addition to giving best predictions overall, Model 4+5 combination was preferred because:

(i)   Model 4 is applicable for both undersaturated and oversaturated conditions, including allowance for initial queued demand at the start of the analysis period (see *Sections 2.4 and 4.4* for detailed discussions on this aspect of Model 4),

(ii)   Model 4 has more general applicability as it has been used to model uninterrupted movements in the SIDRA software package (Akçelik and Besley 1999), and a US study has shown that it has desirable characteristics for traffic assignment purposes (Dowling, Singh and Cheng 1998),

(iii)   in Research Report ARR 340 (Akçelik, Besley and Roper 1999), Model 4 is shown to be applicable to uninterrupted flows at traffic signals and Model 5 is shown to be applicable to queue discharge flows at traffic signals, thus both Models 4 and 5 apply to both uninterrupted and interrupted facilities.

A summary of traffic flow parameters for the survey site on the Eastern Freeway are given below (based on 5-min aggregation period):

Free-flow speed, v_f (km/h)	101
Speed at maximum flow rate, v_n (km/h)	90
The ratio of speed at max. flow rate to free-flow speed, v_n / v_f	0.89
Maximum flow rate, q_n (veh/h)	2500
Average headway corresponding to maximum flow rate, h_n (s)	1.440
Spacing at maximum flow rate, L_{hn} (m/veh)	36.0
Jam spacing, L_{hj} (m/veh)	15.0
The ratio of jam spacing to spacing at max. flow rate, L_{hj} / L_{hn}	0.42
The ratio of spacing at max. flow rate to jam spacing, L_{hn} / L_{hj}	2.4
Density at maximum flow rate, k_n (veh/km)	27.8
Jam density, k_j (veh/km)	66.7
The ratio of densities, k_n / k_j	0.42
Vehicle length, L_v (m/veh)	4.4
The ratio of spacing at max. flow rate to vehicle length, L_{hn} / L_v	8.2
Occupancy time at max. flow rate, t_{on} (s)	0.254
Space time at maximum flow rate, t_{sn} (s)	1.186
Time occupancy ratio at max. flow rate, O_{tn}	18 %
Time occupancy ratio at jam density, O_{tj}	42 %

The ratio of vehicle spacing at maximum flow rate to vehicle length, L_{hn} / L_v indicates that the average vehicle spacing at capacity is about 8 car lengths. The ratio L_{hn} / L_{hj} indicates that the average jam spacing is 2.4 car lengths.

The US Highway Capacity Manual (TRB 1998, Chapter 3) specifies a capacity of 2300 veh/h (passenger cars) for freeways with a free-flow speed of 60 mph (about 100 km/h). The HCM capacities are based on 15-min aggregation period. *Figures 3.2.9 and 3.2.10* in *Section 3* indicate maximum flow rates of 2500 veh/h based on 5-min aggregation period and around 2300 veh/h based on 15-min aggregation period. Thus, the lane capacity of the Eastern Freeway site is in line with the HCM value.

Frequency distributions for individual vehicle headways given in *Section 3* indicate that there is a high proportion of small headways at this site, for example, the frequency of headways equal to or less than 1.00 s is 28 per cent. It was also observed that about 50 per cent of individual vehicle headways were below the headway corresponding to the 15-min capacity value of 2300 veh/h.

Hughes (1998) reported that, based on 5-min aggregation period, "high flow rates of the order of 2500 veh/h" were observed at three sites on a congested section of the Southern Motorway in Auckland, New Zealand. This is also in line with the results for the Eastern Freeway site.

The time occupancy ratio at maximum flow rate (18 %) is consistent with the value indicated by Lowrie (1996, Figure 1) for US freeways.

Jam Spacing

Table 5.7 indicates that the jam spacing (L_{hj}) estimated (or calculated) for different models were in the range 6 m to 17 m. Excepting Model 6, the jam spacing was high compared with a jam spacing of 7.0 m per light vehicle observed at signalised intersections (Akçelik, Besley and Roper 1999). Model 6 tended to give the minimum value allowed in regression (constraint of minimum $L_{hj} = 6.0$ m).

An inspection of individual vehicle data for speeds up to 60 km/h given in *Figure 5.7.1* indicates that a lower-bound value of jam spacing for freeway traffic flows is probably 10 m/veh. In *Figure 5.7.1*, a linear trendline equation implies a jam spacing of 9.3 m. An exponential trendline indicated a jam spacing of 12.2 m. The minimum spacings around 11 m corresponding to minimum headways around 0.4 s are also observed for individual vehicles under high speed conditions as shown in *Figure 5.7.2* (for speeds above 80 km/h and headways below 1.6 s).

Using jam spacing of 10 m/veh instead of 15 m/veh stated above, jam density is 100 veh/km, the ratio $L_{hj} / L_{hn} = k_n / k_j = 0.28$, and the time occupancy ratio at jam density for a 2.0 m detection zone is 64 %.

Estimation of jam spacing is important for finding a single valued space time - speed function which is of interest in relation to adaptive control (Akçelik, Besley and Roper 1999). A single valued space time - speed function requires an effective detection zone of $L_p > L_s$, where $L_s = L_h - L_v$ is the gap length (distance between vehicles). Using a spacing value of 10 m at very low speed conditions, and an average vehicle length of 4.4 m, $L_s = 5.6$ m is found. Therefore, a single valued space time - speed function would require $L_p > 5.6$ m (with a higher value of 15 m, $L_p > 10.6$ m would be required). However, loop lengths much longer than the jam gap length need to be avoided as they result in zero space time values for a large range of speed values. Refer to Research Report ARR 340 (Akçelik, Besley and Roper 1999, Section 14) for detailed discussion on this topic.

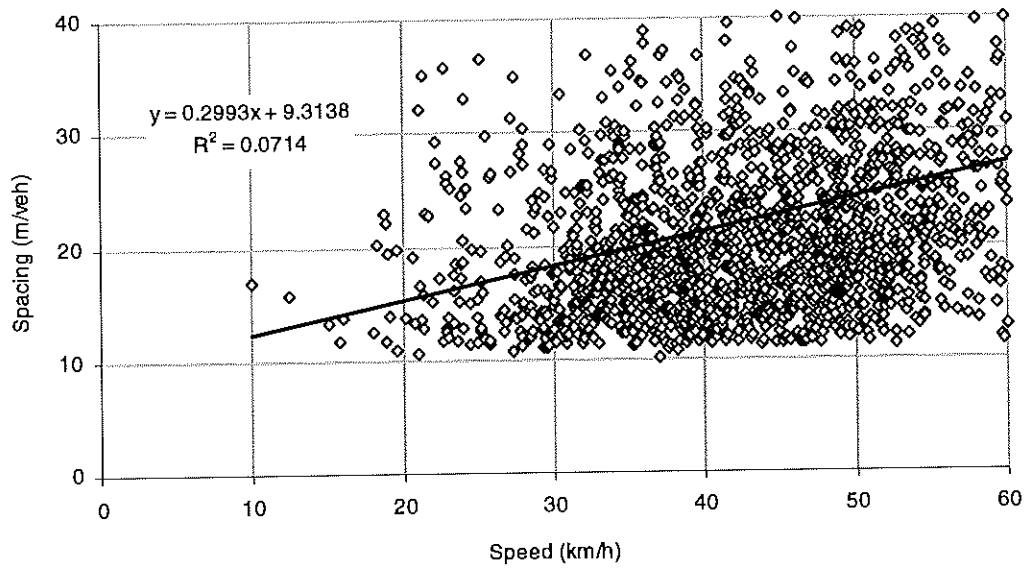Further investigation of the prediction of jam spacing is recommended.

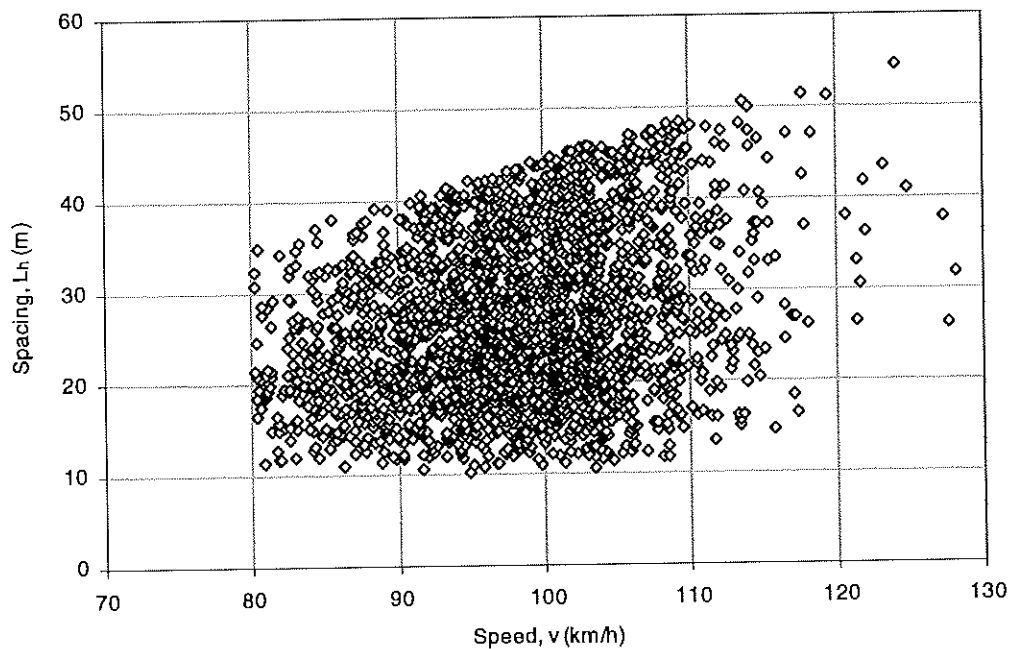**Figure 5.7.1 - Spacing - speed relationship for speeds below 60 km/h (individual vehicle data)**

Figure 5.7.2 - Spacing - speed relationship for speeds above 80 km/h and headways below 1.6 s (individual vehicle data)

5.8 Comparison of Data and Predicted Relationships for Model 4+5

Figures 5.8.1 to 5.8.10 present fundamental relationships with measured data and model predictions for Model 4+5. This is a combination of Model 4 for unsaturated conditions and Model 5 for saturated conditions. The predictions were obtained with the parameters considered to be representative of the Eastern Freeway site (*Table 5.7*). In summary, the model parameters are as follows:

$$v_f = 101 \text{ km/h}$$
$$v_n = 90 \text{ km/h } (v_n / v_f = 0.89)$$
$$q_n = 2500 \text{ veh/h}$$
$$L_{hn} = 36.0 \text{ m/veh}$$
$$k_n = 27.8 \text{ veh/km}$$
$$L_{hj} = 15.0 \text{ m/veh}$$
$$k_j = 66.7 \text{ veh/km}$$
$$T_f = 0.0833 \text{ h (5 minutes)}$$
$$m_c = 0.703$$
$$m_v / m_q = 0.417 (= L_{hj} / L_{hn} = k_n / k_j)$$

As discussed in detail in *Sections 2.4 and 4.4*, Model 4 is applicable for both unsaturated and oversaturated conditions. In this section, it is applied to unsaturated conditions only.

In *Figures 5.8.1 to 5.8.4*, broken lines are for saturated conditions (Model 5) obtained using the lower-bound jam spacing value of $L_{hj} = 10$ m. Related parameters are jam density, $k_j = 100$ veh/km, and the ratio $L_{hj} / L_{hn} = k_n / k_j = m_v / m_q = 0.278$.
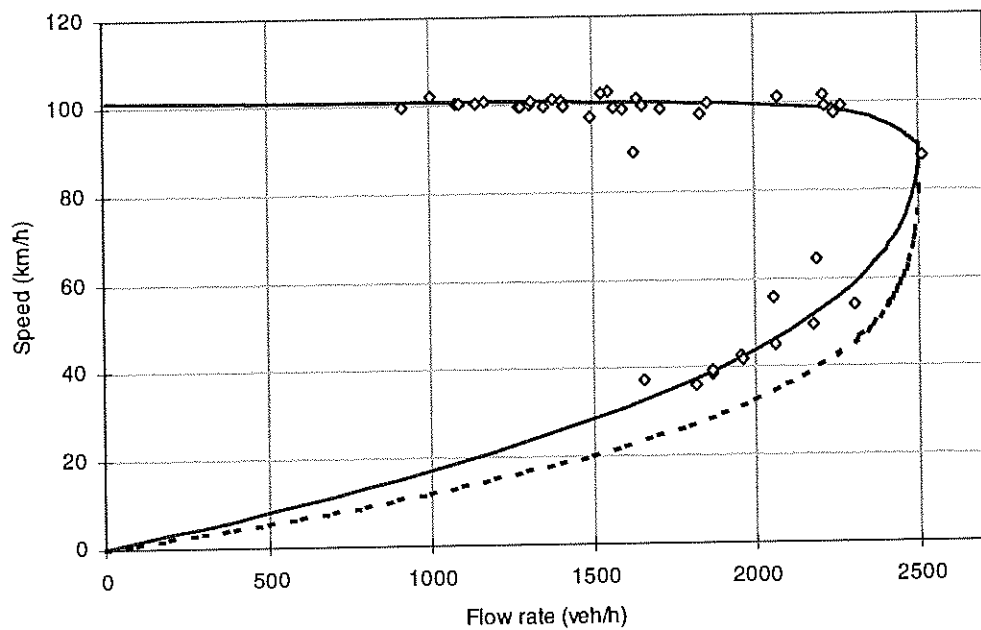
*Figure 5.8.1 - Measured and predicted speed
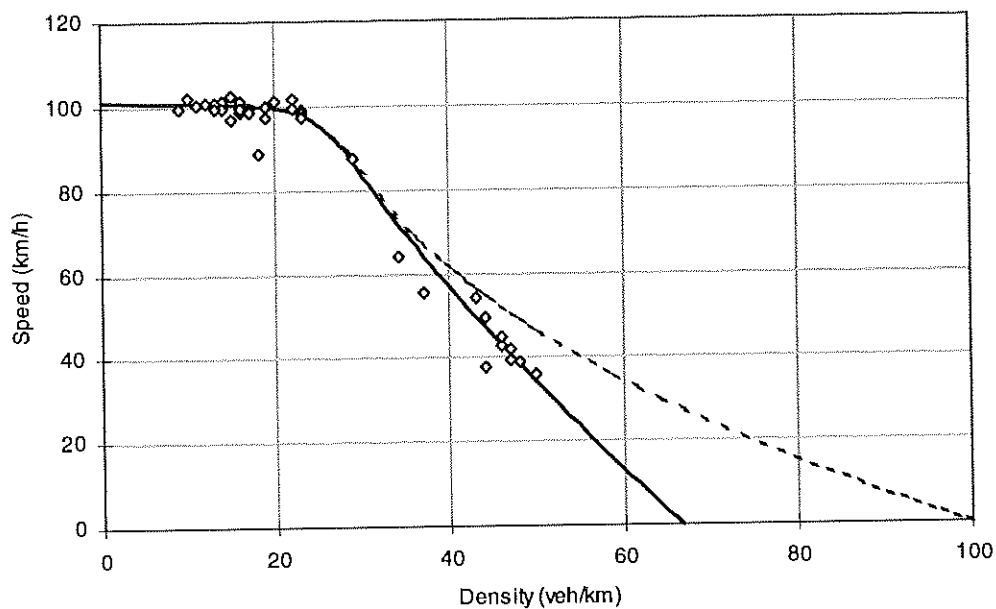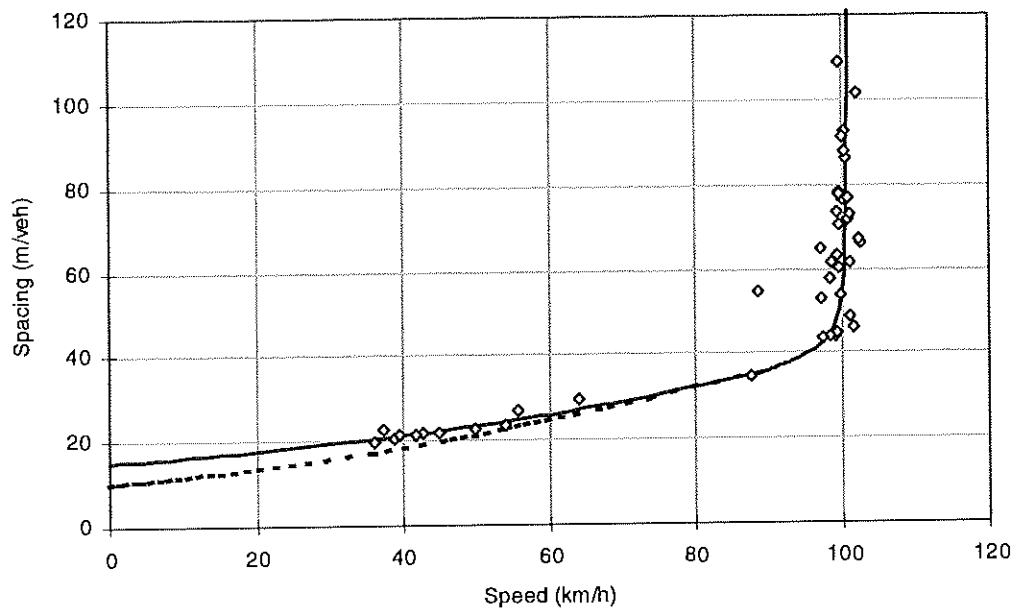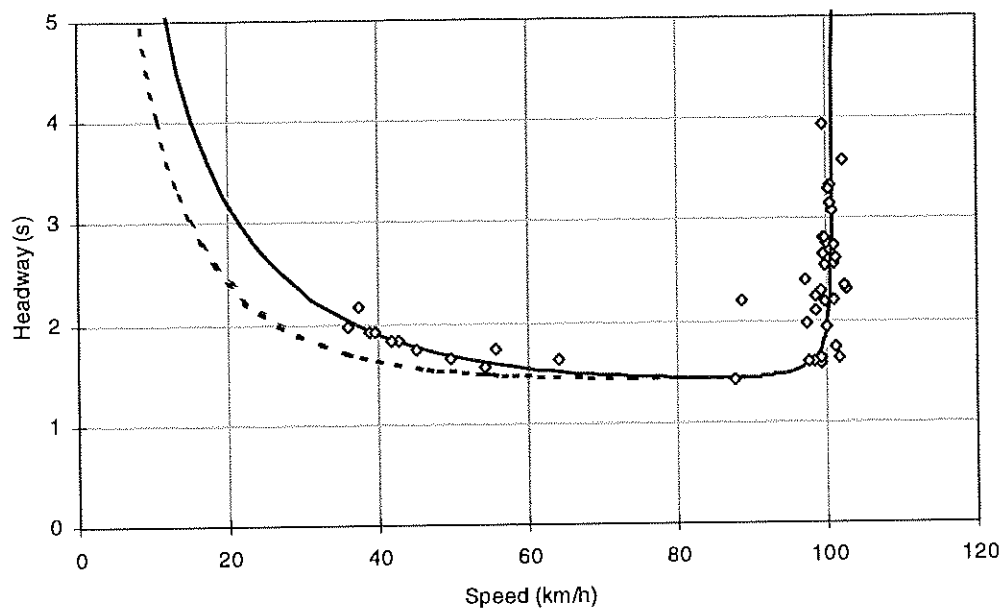as a function of flow rate (Model 4+5)*

*Figure 5.8.2 - Measured and predicted speed
as a function of density (Model 4+5)*

Figure 5.8.3 - Measured and predicted spacing
as a function of speed (Model 4+5)

Figure 5.8.4 - Measured and predicted headway
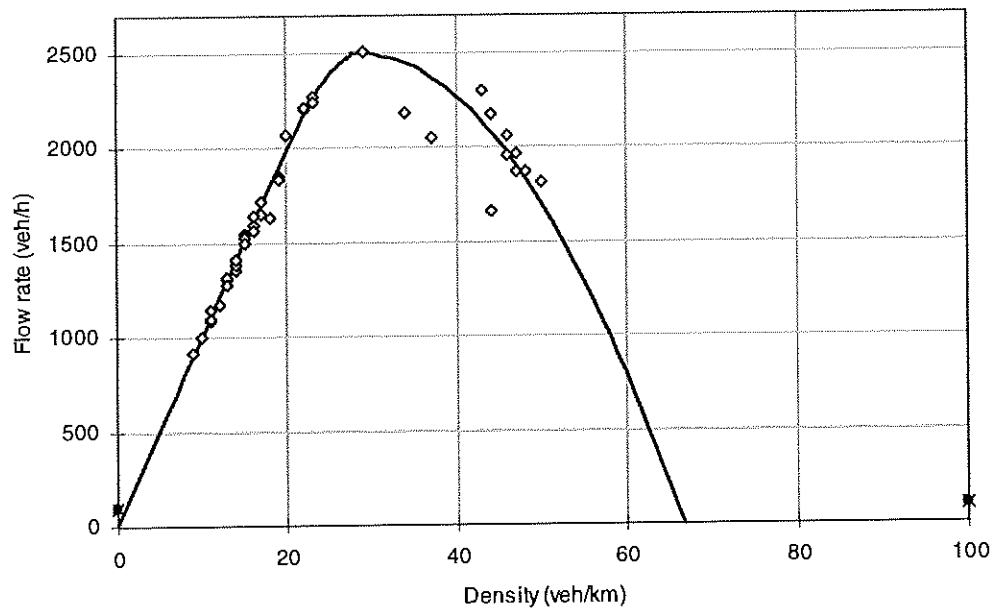*as a function of speed (Model 4+5)*

Figure 5.8.5 - Measured and predicted flow rate
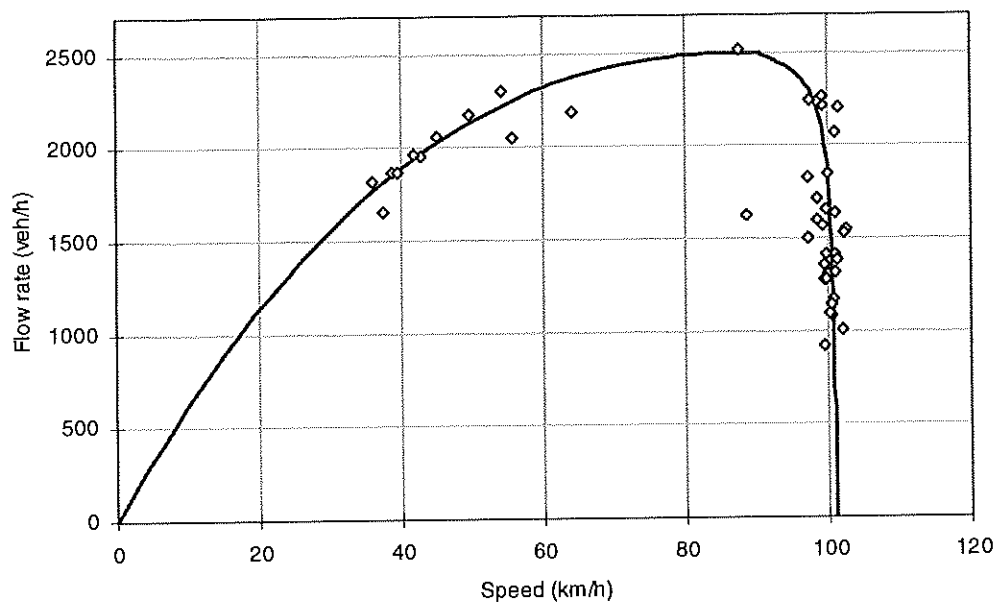*as a function of density (Model 4+5)*

Figure 5.8.6 - Measured and predicted flow rate
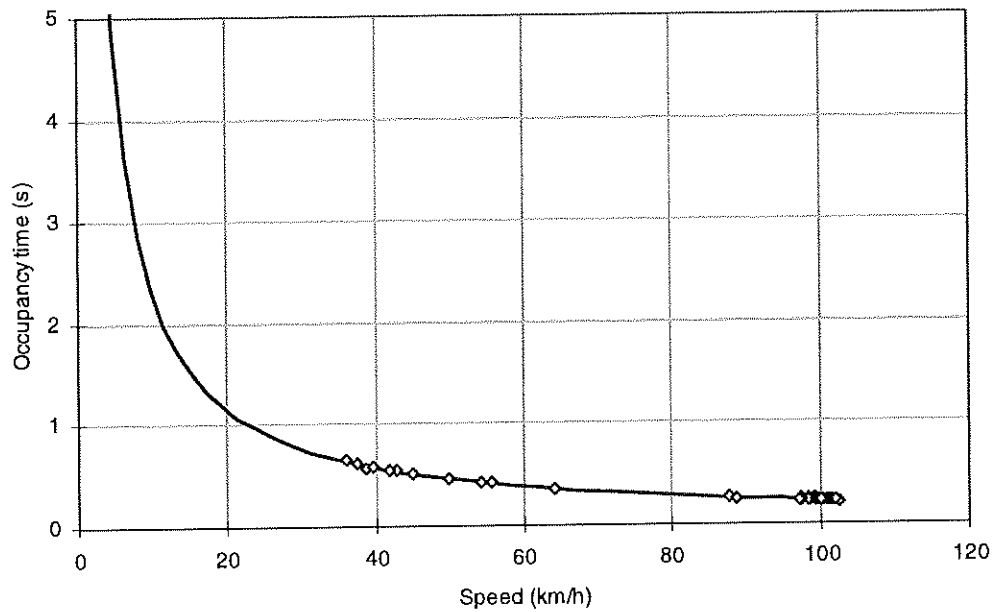*as a function of speed (Model 4+5)*

*Figure 5.8.7 - Measured and predicted occupancy time
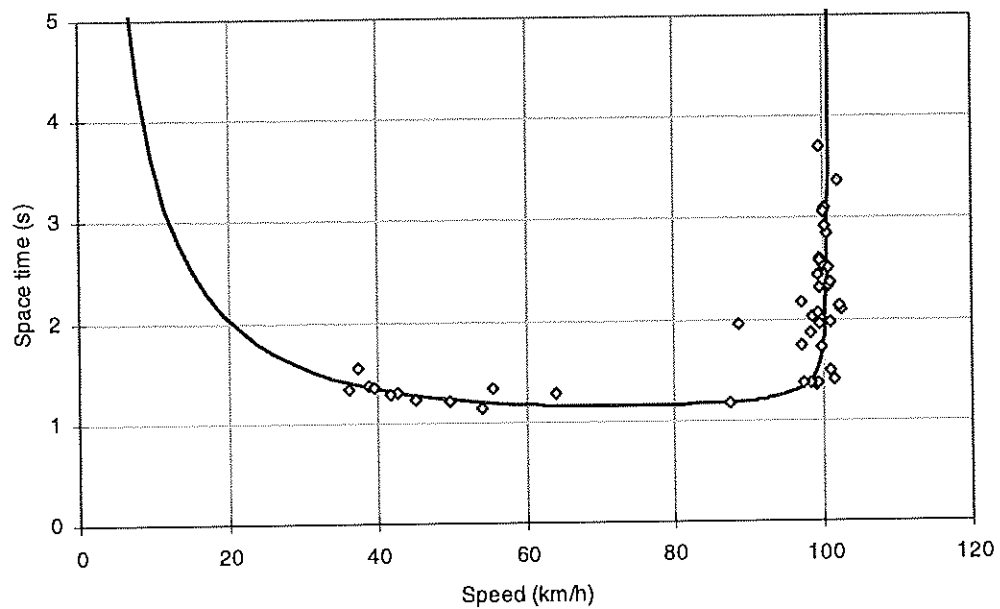as a function of speed (Model 4+5)*

*Figure 5.8.8 - Measured and predicted space time
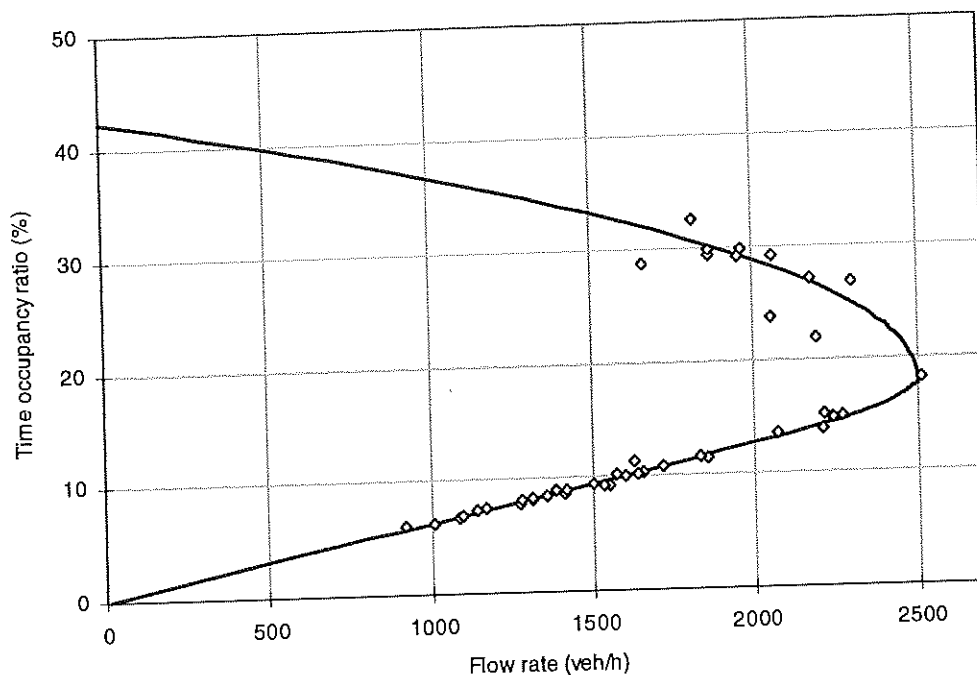as a function of speed (Model 4+5)*

*Figure 5.8.9 - Measured and predicted time occupancy ratio
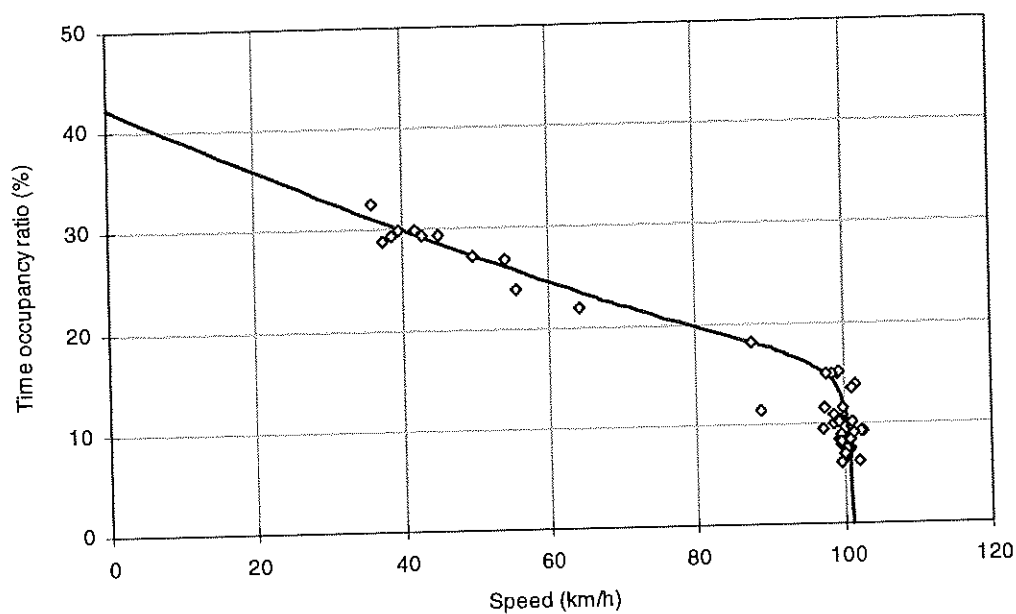as a function of flow rate (Model 4+5)*

*Figure 5.8.10 - Measured and predicted time occupancy ratio
as a function of speed (Model 4+5)*

6 CONCLUSION

Most studies to date have concentrated on traffic stream parameters speed, flow rate, density and occupancy. This study used traffic parameters measured for individual vehicles including speed, headway, occupancy time, space time, gap time, vehicle passage time, spacing, gap length and vehicle length. Individual vehicle parameters were aggregated to determine traffic stream characteristics including average speed, flow rate, density as well as time and space occupancy ratios. Analytical models describing the relationships among these parameters are useful for practical applications including the design of freeway facilities with adequate capacity and level of service, and development of improved methods for the purposes of incident detection, ramp metering, traffic monitoring, and driver information.

Section 2 presented an introduction to the fundamental traffic flow parameters and their relationships. *Section 3* discussed the method used for data collection on the Eastern Freeway in Melbourne, and described the analysis method used to establish fundamental relationships for freeway traffic flows.

A permanent VicRoads two-loop presence detection system was linked to VDAS traffic counters for the purpose of this research. Data covered the full range of traffic conditions from free-flowing to congested traffic (individual vehicle speeds were in the range 10 - 132 km/h, and average speeds for 5-min intervals were in the range 36 - 103 km/h). Travel speeds were also measured during the survey period using an instrumented car. Comparison of speed data from the two survey methods indicated that data collected by the two-loop detection system was generally acceptable.

To obtain parameters describing the characteristics of freeway traffic streams, data for individual vehicles were aggregated using the method described in *Section 2* for periods of 20 s, 1 min, 2 min, 5 min and 15 min. Aggregate data were always derived from individual vehicle data, not from the aggregate data for a shorter period. The data aggregation method used in this study aimed to achieve consistency between speed and all time-based and distance-based parameters. The method recognises that summation of some parameter values for all vehicles observed during an analysis period or in a road section may not correspond to the duration of the analysis period or the length of the road section. This is more important when the analysis period or the road length is short.

It is recommended that the data collection and analysis method described in this report is employed for future analysis of freeway traffic flows. For research purposes, basic headway, occupancy and speed data should be stored in the database for individual vehicles, and any aggregation should be conducted according to the method described in this report. In summary:

(i) calculate the average headway for the traffic stream as a simple arithmetic mean of (n − 1) headways observed between n vehicles, excluding headway values eliminated for any reason,

(ii) calculate average values of occupancy time, space time, vehicle passage time and gap time for (n − 1) vehicles consistent with the calculation of the average headway,

(iii)   calculate the average speed as the space mean speed for (n − 1) vehicles used in determining the average headway and related traffic stream parameters in (i) and (ii),

(iv)   if spacings for individual vehicles have been measured, calculate the average spacing for the traffic stream as a simple arithmetic mean for (n − 1) vehicles as in (i) and (ii), otherwise calculate it using the average headway and speed values for the traffic stream from (i) and (iii),

(v)   similarly, if the gap length and vehicle length are not measured for individual vehicles, calculate them using the average headway, average occupancy or space time, average gap time or vehicle passage time and average speed values for the traffic stream from (i) to (iii),

(vi)   calculate the average flow rate using the average headway for the traffic stream from (i),

(vii)   calculate the average density using the average spacing for the traffic stream from (iv),

(viii) calculate the time occupancy ratio using the average headway and average occupancy or vehicle passage time values for the traffic stream from (i) and (ii),

(ix)   calculate the space occupancy ratio using the average spacing and vehicle length values for the traffic stream from (iii) and (v).

Equations for the aggregation method summarised in (i) to (ix) are given in *Section 2.2.*

Six analytical traffic flow models including single-regime and two-regime models are described in *Section 4.* The models are presented as speed - flow rate relationships from which other relationships such as speed - density, space time - speed and occupancy ratio - flow rate can be derived using the fundamental relationships given in *Section 2.*

Section 5 presents the model calibration results for the Eastern Freeway site. A constrained non-linear regression analysis method was used for model calibration. Regression analyses were carried out using data aggregated for 5-min intervals. This interval was chosen as the most appropriate interval for the purpose of evaluating alternative models. Several regression options were tried for each model with different model parameters specified. Results for all models and all regression options are presented. The calibration results indicate that strong relationships can be identified among fundamental traffic parameters for freeway flows.

By inspecting the graphs of predicted traffic flow parameters against measured values for all models, it was found that Models 4 and 5 together as a two-regime model ("Model 4+5") provided the best estimates of traffic characteristics for the survey site. In addition to giving best predictions overall, Model 4+5 combination was preferred because:

(i)   Model 4 is applicable for both undersaturated and oversaturated conditions, including allowance for initial queued demand at the start of the analysis period,

(ii)   Model 4 has more general applicability as it has been used to model uninterrupted movements in the SIDRA software package (Akçelik and Besley 1999), and a US study has shown that it has desirable characteristics for traffic assignment purposes (Dowling, Singh and Cheng 1998), and

(iii) in Research Report ARR 340 (Akçelik, Besley and Roper 1999), Model 4 is shown to be applicable to uninterrupted flows at traffic signals and Model 5 is shown to be applicable to queue discharge flows at traffic signals, thus both Models 4 and 5 apply to both uninterrupted and interrupted facilities.

Graphs showing fundamental relationships for all traffic flow parameters with measured data and model predictions are presented for Model 4+5.

The data used for the results given in this report include some heavy vehicle data (about 3 per cent). The effect of this is expected be minimal due to the low percentage of heavy vehicles. Further work is recommended on the effect of heavy vehicles on fundamental relationships. Sites with a high percentage of heavy vehicles are needed for this purpose.

This study used data from a single survey site with the purpose of in-depth assessment of data collection and analysis methods, and evaluation of alternative models. It is recommended that the method established in this study should be applied to collect and analyse data at a large number of sites to establish the range of traffic parameters possible on Australian freeways. The VicRoads driver information system routinely collects large amounts of data that could be used for this purpose. An example is given in *Figures 6.1 and 6.2* that show speed-flow rate and time occupancy rate - flow rate data for two lanes at the same site on the Westgate Freeway in Melbourne. The maximum flow rates for the two lanes are lower than the Eastern Freeway site, and the characteristics of the two lanes are seen to be rather different.

Using a database that includes a large number of freeway sites, methods could be developed to predict traffic flow parameters from freeway geometric and traffic characteristics, e.g. taking into account factors such as number of lanes, lane width, lateral clearance, grade, driver behaviour, and proportion of heavy vehicles in the stream, similar to the method used in the US Highway Capacity Manual (TRB 1998, Chapter 3).

Research into lane utilisation on different freeway segments (basic segment, ramp junctions and weaving areas) is recommended to take into account different flow rates in different lanes on a given section of freeway.

Further research is recommended on saturated traffic flows on freeways covering a wide range of low speed conditions. A data collection method with higher accuracy levels than the two-loop presence system is needed for this purpose. Accuracy of loop data is not sufficient for refining data under saturated conditions to allow for accelerations and decelerations. VDAS treadle detectors were not considered to be strong enough to endure the high speed, high volume traffic conditions on freeways. It may be possible to overcome such problems using new detector technologies.

A summary of traffic flow parameters for the survey site on the Eastern Freeway are given below (based on 5-min aggregation period):

Free-flow speed, v_f (km/h)	101
Speed at maximum flow rate, v_n (km/h)	90
The ratio of speed at max. flow rate to free-flow speed, v_n / v_f	0.89
Maximum flow rate, q_n (veh/h)	2500

Average headway corresponding to maximum flow rate, h_n (s)	1.440
Spacing at maximum flow rate, L_{hn} (m/veh)	36.0
Jam spacing, L_{hj} (m/veh)	15.0
The ratio of jam spacing to spacing at max. flow rate, L_{hj} / L_{hn}	0.42
The ratio of spacing at max. flow rate to jam spacing, L_{hn} / L_{hj}	2.4
Density at maximum flow rate, k_n (veh/km)	27.8
Jam density, k_j (veh/km)	66.7
The ratio of densities, k_n / k_j	0.42
Vehicle length, L_v (m/veh)	4.4
The ratio of spacing at max. flow rate to vehicle length, L_{hn} / L_v	8.2
Occupancy time at max. flow rate, t_{on} (s)	0.254
Space time at maximum flow rate, t_{sn} (s)	1.186
Time occupancy ratio at max. flow rate, O_{tn}	18 %
Time occupancy ratio at jam density, O_{tj}	42 %

The ratio of vehicle spacing at maximum flow rate to vehicle length, L_{hn} / L_v indicates that the average vehicle spacing at capacity is about 8 car lengths. The ratio L_{hn} / L_{hj} indicates that the average jam spacing (average spacing between vehicles in a stationary queue) is 2.4 car lengths.

Frequency distributions for individual vehicle headways at this site indicated that there was a high proportion of small headways, for example, the frequency of headways equal to or less than 1.0 s was 28 per cent.

The capacity characteristics observed at the Eastern Freeway site appear to be in line with those specified in the US Highway Capacity Manual (TRB 1998, Chapter 3) and found in recent studies in New Zealand (Hughes 1998).

The time occupancy ratio at maximum flow rate (18 %) is consistent with the value indicated by Lowrie (1996, Figure 1) for US freeways.

The maximum flow rate for the Eastern Freeway site is 2500 veh/h based on 5-min aggregation period and approximately 2300 veh/h based on 15-min aggregation period as seen from *Figures 3.2.9 and 3.2.10* in *Section 3*. While the 15-min aggregation period is appropriate for capacity and level of service analysis, investigation of the most appropriate aggregation period for incident detection and ramp metering purposes is recommended.

Jam spacing and jam gap length are important parameters that determine the nature of fundamental traffic relationships. Generally, the predicted jam spacing was high compared with a jam spacing of 7.0 m per light vehicle observed at signalised intersections as discussed in the accompanying Research Report ARR 340 (Akçelik, Besley and Roper 1999). An inspection of individual vehicle data indicated that a lower-bound value of jam spacing for freeway traffic flows is probably 10 m/veh. Further investigation of the prediction of jam spacing is recommended.
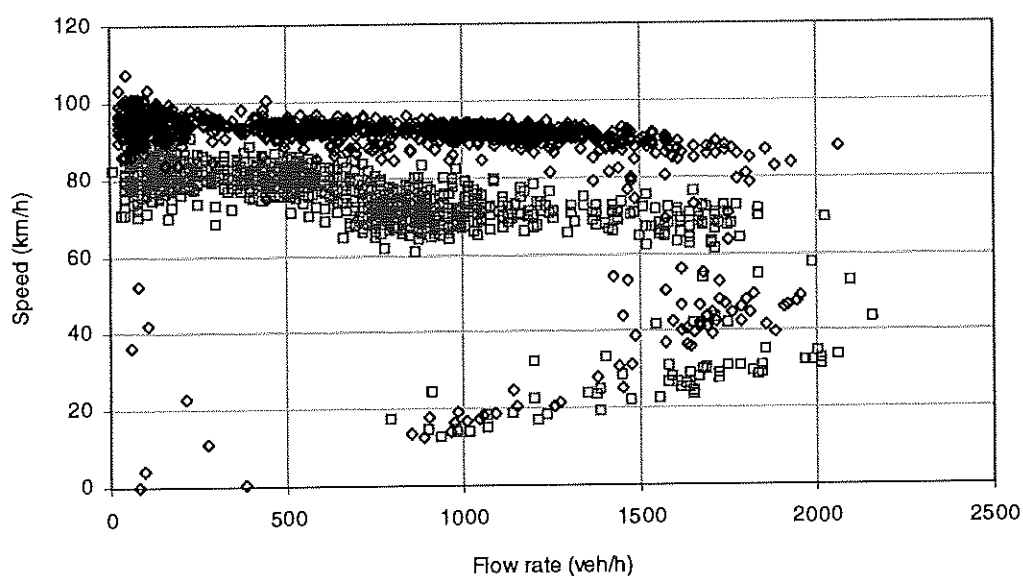
Figure 6.1 - Speed - flow rate data for two lanes at the same site on Westgate freeway (VicRoads data with 5-min averages)
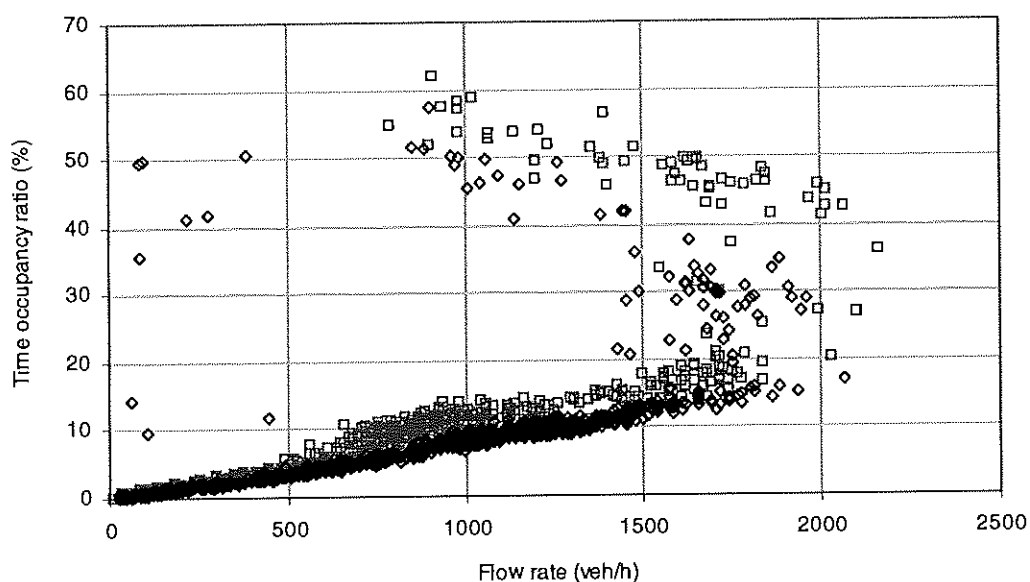

Figure 6.2 - Time occupancy ratio - flow rate data for two lanes at the same site on Westgate freeway (VicRoads data with 5-min averages)

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