

ASPECTS OF DRIVER BEHAVIOUR
IN MAIN ROAD TRAFFIC STREAMS


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ABSTRACT

Studies of composite headway models and platooning are reviewed and further results on time headways distribution are given. It is suggested that a three-population model is more satisfactory than a two-population model when short headways are of primary importance.

Platoons were shown to occur randomly and the distribution of platoon sizes can be closely represented by a modified geometric distribution. It was also found that the distribution of headways in platoons was independent of speed at a given site, and members of platoons were shown to follow too closely in terms of the Highway Code advice; this close following behaviour was shown to be relatively greater at higher speeds.

Observations showed that the deceleration behaviour of driver in a major road approaching a rural T-junction was affected by the T-junction (a) being unoccupied, (b) having vehicles waiting to cross and (c) having vehicles actually crossing. The results in case (a) were used to deconvolute the effects due to background in cases (b) and (c).

Empirical results on the relationship between a measure of deceleration and other factors are presented.

The deconvoluted values together with other observed data were used as input into a conflict simulation model. The output of the model consists of the number of precautionary and severe model conflicts to be expected in various circumstances.

Results on driver gap-acceptance behaviour in adverse weather conditions at T-junctions are presented in appendix. No evidence of more cautious behaviour in wet weather was found.

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INTRODUCTION

At Royal Holloway College, studies on driver behaviour at rural and suburban T-junctions have been undertaken for several years. In particular, various aspects of gap acceptance behaviour of turning drivers have been examined; the emphasis in the work has been on safety rather than on delay or capacity.

The road traffic system contains a number of interacting elements, all of which can affect the occurrence and severity of accidents. These elements may be grouped into three main components: the road user, the vehicle and the road environment. The role of each one of these elements in accidents are discussed by Sabey et al (1975). In the U.K., accidents are not distributed uniformly within the road network. Over 75% of all accidents occur on roads subject to 30 or 40 m.p.h. speed limit, and more than 60% of these occur at or near junctions. The majority of these accidents involve turning drivers. When turning drivers make poor gap acceptance decisions, major road vehicles may be forced to take evasive action in order to avoid a collision. This situation is an example of a 'traffic conflict'. A conflict simulation model has been developed at R.H.C. to allow evaluation of risk at T-junction in terms of conflict situations. The input data in the model are in the forms of distributions and parameters. Thus it is of prime concern to have an analytic representation of both priority and non-priority streams of vehicles.

In this thesis various aspects of main-road stream driving, in rural and suburban environments, are examined; the interest is safety rather than capacity, volume or delay.

In chapter 1, a review of headway models proposed by various researchers is presented. The emphasis is more on composite models where two or more populations of drivers are assumed. The phenomenon of platooning and some platoon size models are also discussed.

In order to study driver behaviour in real life conditions, a number of experiments at and near priority-controlled T-junctions were conducted, using a data collection system based on a microprocessor. A brief description of the system, used with two different versions of software, and the data-collecting are described in chapter 2.

Chapter 3 returns to the arguments of chapter 1. A headway model is presented and further results from suburban sites are discussed. A comparison of some parameters of the model for data from rural and suburban sites is also made.

In chapter 4, a further analysis of headways is considered. In particular, the autocorrelation between successive time headways, the distribution of gaps between platoons and the distribution of platoon sizes are presented for data from rural and suburban sites. Furthermore, the distribution of headways within platoons is considered, in relation to speed, for data from rural sites.

In chapter 5, a technique of measuring the deceleration behaviour of major-road stream drivers approaching a T-junction is presented. The deceleration is measured in terms of a time difference between actual and expected arrival times, when the junction is unoccupied or occupied by a crossing vehicle. The objective is to understand and explore drivers' behaviour in terms of forced or unforced actions within and close to the T-junction, in cases where intersections of vehicle paths may or do occur.

In part II of chapter 5 the measure of deceleration of the approaching vehicle is examined in relation to the vehicles speed as well as in relation to the accepted gap and crossing time of the turning vehicle.

Chapter 6 presents a simulation model of traffic conflicts between crossing vehicles and oncoming major road vehicles; different versions of the model are considered. Real life data in forms of distributions and parameters are used as input to the model. The output consists of the number of precautionary and severe model conflicts to be expected in various circumstances.

Finally, as an appendix, the results of a number of experiments conducted at two suburban and one rural priority-controlled T-junctions, to study the effect of adverse weather on gap acceptance behaviour, are presented. The experiments were conducted either in daylight or darkness and the manoeuvres studied were the right turn into the minor road (crossing) and the left turn out of the minor road (merging).

CHAPTER 1

HEADWAY MODELS AND PLATOONING : A REVIEW.

1.1 Introduction

This chapter reviews some of the literature on headway models and platooning.

We are mainly concerned with those studies where headways distribution is divided into two components, corresponding to two subclasses of driver population : Followers and Non-followers.

Non-followers : are those who are not impeded by the vehicle ahead and drive at their desired speed.

Followers : are those who are impeded by the vehicle ahead and so they are not able to drive at their desired speed.

One simple model, consisting of a single distribution, is also presented, as it has been found by many traffic researchers to be the best simple model of representing headways distribution. The phenomenon of platooning and some platoon size models are also discussed.

1.2 Time headways

Time headways have been a subject of considerable interest to road traffic researchers. The two aspects involved are the probability distribution of headways and the structure of the underlying stochastic process. The distribution of headways based on the concept of a reference point or gaps between vehicles moving along a fixed stretch of a road has been examined by a number of researchers. The approaches used in most studies have varied both in technique and in philosophy. Various models have been proposed but

a comprehensive theory is still lacking. This comment has also been made directly or indirectly by Pahl and Sands (1970), Cowan (1971) and more recently by Branston (1978). Cowan (1971) suggested that an exact mathematical formulation of some of the problems of interest is difficult. Tanner (1961) was most probably the first to express serious doubts that a fully satisfactory theory is possible. Two decades later, these doubts still remain.

1.3 The two main classes

In studying the headways distribution of a stream of traffic, two main classes of model of vehicular behaviour have been considered. One of them allows for overtaking on a given stretch of road; the other does not. Weiss and Herman (1962) have studied the case of low-density traffic for which overtaking is unrestricted. Also in this category are studies by Breiman (1963), Thedeen (1964) and Brown (1972). In all of these, it is shown that headways are distributed according to a negative exponential law.

In the other extreme case in which overtaking is forbidden there are a number of studies among which are those by Hodgson (1968), Buckley (1968) Cowan (1971), Epstein et al (1974) and Branston (1976). Headways in this latter category are claimed to follow a composite distribution involving two subclasses of driver population.

Somewhere in between the two extremes just described, is one in which overtaking is allowed although there is no vehicular interaction involved when sufficiently low flow traffic is considered. Notable in this category are studies by Newell (1955, 1966).

1.4 Composite headway models

The requirement to develop a headway model, which would lead to a considerably closer agreement between theory and practice, generated the following idea: in a traffic stream there might be two populations of headways instead of one, each having its own distribution. Thus the idea of a two component model was first put forward in the middle 50's. Schuhl (1955) suggested that time headway can be represented by a composite probability distribution function (p.d.f.) of the form

$$f(t) = \phi f_1(t) + (1 - \phi) f_2(t),$$

where $0 \leq \phi \leq 1$, $f_1(t) = \lambda_1 e^{-\lambda_1 t}$ and $f_2(t) = \lambda_2 e^{-\lambda_2(t-a)}$. He obtained good results for some available data. Leutzbach (1957) has also suggested a pair of functions similar to these. Buckley (1962) considered two populations of headways. A semi-random model for free flowing single lane traffic was suggested in the following form.

$$f(t) = \phi g(t|\theta, \sigma) + (1 - \phi) \exp[\theta\lambda - y_2 \sigma^2 \lambda] \lambda e^{-\lambda x} \int_{-\infty}^x g(z|\theta, \sigma) dz;$$

where ϕ is the proportion of vehicles following and $g(t)$ their distribution, in this case a Normal. Empirical data were found to be consistent with the model.

In a second paper, Buckley (1968) proposed a generalization of the semi-random model: the generalized semi-Poisson model of traffic flow is given as

$$f(t) = \phi g(t) + (1 - \phi) [g^*(\lambda)]^{-1} \lambda e^{-\lambda t} G(t),$$

where $g^*(\lambda)$ is the Laplace transform of the p.d.f. of following

headways $g(t)$, and $G(t)$ is the distribution function. He applied his model with two different distributions of following headways, Normal and Gamma, to some freeway data from a single lane. He found that the semi-Poisson model gives a good fit when applied with one or the other distribution of following headways.

Dawson (1969) proposed a generalised traffic headway model for single-lane traffic flows on two-lane two-way roads. Two types of vehicles were considered: free and constrained vehicles. The model was described by a Hyperlang function and is of the following form:

$$F(t) = \phi \exp\left[-\frac{t - \delta_1}{\gamma_1 - \delta_1}\right] + (1 - \phi) \exp\left[-\frac{k(t - \delta_2)}{\gamma_2 - \delta_2}\right] \sum_{x=0}^{k-1} \left[\frac{k(t - \delta_2)}{\gamma_2 - \delta_2} \right]^x / x! ,$$

where $F(t)$ is the probability distribution, ϕ is the proportion of free vehicles and k indicates the degree of non-randomness in the constrained headway distribution. The model was applied to some observed data satisfactorily.

Wasielwski (1974) reformulated the semi-Poisson model proposed by Buckley (1968) to derive an integral equation for calculating the following headways distribution directly from the observed headway distribution in order to avoid a parametric representation. The method was applied to observations of freeway traffic at very high flow levels.

Using queuing theory and the concept of platooning, Branston (1976) derived the probability density function (p.d.f.) $f(t)$ of all headways in terms of the p.d.f. $g(t)$ of follower headways. He obtained the following form for $f(t)$:

$$f(t) = \phi g(t) + (1 - \phi) \lambda e^{-\lambda t} \int_0^t g(x) e^{\lambda x} dx ,$$

where ϕ is the proportion of following vehicles and λ^{-1} is the mean interplatoon gap. He then remarks that this is analogous to the queuing model proposed by Tanner (1961). The only major difference is that Tanner (1961) assumes a constant value for follower headways, while Branston permits a probability distribution on follower headways. Branston also claims that the only difference between the semi-Poisson and his queuing model is that, in the first, the non-following headway is obtained by comparing an exponential headway with a following headway, while, in the second, each non-following headway is obtained by adding an exponential headway to a following headway. However, he found that both models provide an acceptable and almost identical fit when a Gamma distribution was used for the followers; but neither were acceptable with a Normal distribution of followers. The best fit to the available data from two different sites was given when the model was used with a log-normal distribution of following headway, namely, the log-normal generalizing queuing model.

Branston (1978) later developed a method for estimating the free distribution using a Cowan (1971) model, even when it was known that the interactions among vehicles were substantial. He considered a single lane section of a road for which the opportunity to overtake was non-existent, and he examined the speed of vehicles passing a given point. He found the procedures suggested and used by some previous workers to yield unsatisfactory results for his data. He developed another method for his own use based on the following observation. Suppose the free headways are distributed according to a common p.d.f. $f(t)$ given by

$$f(t) = \exp[-\lambda(t - T^*)] \quad t \geq T^*,$$

where λ is the parameter of this shifted negative exponential distribution and T^* is the critical value of t beyond which $f(t)$ is valid. Then λ varies with T^* . Consequently, as a criterion for establishing the critical value for any given population of headways, he required that the parameter of the shifted exponential remained relatively constant for all estimates in the region beyond the critical point.

The following four headway models have been studied by Cowan (1975):

$$F(t) = \begin{cases} 0 & t < 0 \\ 1 - \exp[-\lambda t] & t \geq 0, \end{cases}$$

$$F(t) = \begin{cases} 0 & t < \tau \\ 1 - \exp[-\gamma(t-\tau)] & t \geq \tau, \end{cases}$$

$$F(t) = \begin{cases} 0 & t < \tau \\ 1 - (1-\theta) \exp[-\gamma(t-\tau)] & t \geq \tau, \end{cases}$$

$$F(t) = \begin{cases} 0 & t < 0 \\ \theta B(t) + (1-\theta) \int_0^t B(t-u) \gamma \exp[-\gamma u] du & t \geq 0, \end{cases}$$

where $F(t)$ is the distribution function of the total headway represented by the random variable T such that $T = V + U$.

V is the tracking headway component having a distribution function $B(t)$ and U is a free headway component satisfying the negative

exponential law. Cowan's main purpose was to develop systems that were realistic and amenable to stochastic modelling. He also attempted to resolve the crucial issue of the choice of an appropriate headway distribution when the number of lanes is small, for example, a single lane. The models used by Cowan allowed for an increasing generality as we moved from the first to the fourth. Using two example problems, he demonstrated that even the most general of his models was mathematically tractable. Cowan, however, made an unrealistic assumption that the driver's choice of V remained constant for the duration of the journey : he then sampled it as a component of T at the beginning of the journey. This assumption is also inconsistent with the requirement that the fourth of his models be the most general form; unless by "V is constant throughout the duration of the journey", he meant that the distribution function of V did not change in form throughout the journey.

Wasielewski (1979) in a second paper used an integral equation derived from the semi-Poisson model to study the flow dependence characteristics of follower headways. He considered each headway to be either a leader's headway or a follower's headway, depending on whether it exceeded a given threshold value T^* or not. He then solved an integral equation numerically to obtain follower's headway distribution directly from the total observed headways. He assumed that the total headway p.d.f. , $f(t)$, could be represented by

$$f(t) = \phi g(t) + (1 - \phi) h(t), \quad t \geq 0, \quad (1)$$

where $g(t)$ is the p.d.f. of follower headways, $h(t)$ is the p.d.f. of leading headways, and ϕ is the fraction of vehicles in the following mode. In the normalized form, equation (1) becomes

$$f(t) = g_1(t) + h_1(t). \quad (2)$$

He then used the negative exponential distribution to characterise large headways ($t > T^*$) for which it was assumed that there was little interaction among vehicles. Thus

$$f(t) = h_1(t) = A \lambda e^{-\lambda t}, \quad t > T^*, \quad (3)$$

where A and λ are parameters to be evaluated from the observed data. He then noted that some of such vehicles could catch up with slower ones; the fraction of vehicles catching up being

$$P = \int_t^{\infty} g(u) du. \quad (4)$$

Deriving p from the density function of leader's headway gave

$$h_1(t) = A \lambda e^{-\lambda t} \int_0^t g(u) du. \quad (5)$$

Combining equations (1) and (5) he got

$$h_1(t) = A \lambda \phi^{-1} e^{-\lambda t} \int_0^t [f(u) - h_1(u)] du. \quad (6)$$

Equation (6) was then to be solved numerically subject to the constraint

$$\phi = \int_0^{\infty} g_1(u) du$$

to obtain the values of ϕ and $h_1(t)$ for $t < T^*$. He divided up the entire data into subsets with similar flow patterns and emphasised that this was a necessary condition for his model to apply. He got $T^* = 2.5$ seconds for flows under 1,450 veh/hr and $T^* = 3.5$ seconds for flows over 1,450 veh/hr. Wasielewski found that, for his data,

ϕ varied from 900 veh/hr to 2,000 veh/hr. He remarked, however, that individuals were influenced mainly by the vehicle immediately ahead and not necessarily by flow levels. He found that the headway distribution of followers with respect to flow level stayed relatively constant and concluded that it was therefore approximately independent of flow in the range of 900 to 2,000 veh/hr. A mean of 1.3 seconds and a standard deviation of 0.5 seconds was obtained for this distribution.

The headway models based on the assumption of the existence of two population of drivers, followers and non-followers, are usually characterised by a large number of parameters. These parameters have to be estimated from the data before the model can be applied for simulation purposes. That additional effort is justified by the substantial improvement of the goodness-of-fit to the observed data. A better understanding of the underlying factors affecting the arrival process can be achieved as well.

1.5 Simple headway model

In using a headway model, consisting of a single distribution, fewer parameters have to be estimated from the data. The log-normal distribution has been found by a number of traffic researchers as the best simple model to represent headways distribution satisfactorily.

Greenberg (1966) postulated a stochastic process to describe vehicular traffic. The process led to a log-normal distribution to describe the time headways. A comparison between the semi-random model proposed by Buckley (1962) and the log-normal distribution showed that there was no significant difference in the goodness-of-fit for the two models.

Daou (1966) and Heyes and Asworth (1972) also found that the log-normal distribution gave a satisfactory fit to following vehicle headway data obtained from tunnels.

Tolle (1971) also advocated the use of a log-normal distribution for freeway traffic. In particular, by shifting the log-normal distribution by use of three parameters, namely

$$f(t) = \frac{1}{(t-a)\sqrt{2n}\sigma} \exp\left[-\frac{1}{2\sigma^2} (\ln(t-a) - \mu)^2\right], \quad t > a,$$

better results were obtained for some available data at high flows.

Tolle (1976) later fitted a series of headway models to data collected on single lanes of multilane headways in the U.S.A. He obtained good fits using a log-normal distribution alone, which has just two adjustable parameters. However, in most of the studies of headway models, the large headways of the available data each time did not usually show any significant deviation from the exponential function.

1.6 Platooning and Platoon size models

There is a tendency in traffic for vehicles to travel in platoons; this is usually due to speed differentials between consecutive vehicles and to limited opportunities of overtaking. The phenomenon of platooning on rural roads with two opposing traffic streams has been a subject of considerable empirical and theoretical studies. In all these studies the term platoon includes either a single vehicle travelling by itself (defined as a platoon of size-one), or a group of vehicles (defined as a platoon of size $r, r > 1$) moving within close proximity with each other and where speed differences between consecutive vehicles are small.

The more commonly used platoon-size models are the following:

(1) the Geometric distribution, Miller (1961)

$$p_r = (1 - a) a^{r-1}, \quad r \geq 1 ;$$

(2) the Borel-Tanner distribution, Tanner (1953,1961) Miller (1961)

$$p_r = \frac{(rae^{-a})^{r-1} e^{-a}}{r!}, \quad r \geq 1 ;$$

(3) the two-parameter Miller distribution, Miller (1961)

$$p_r = \frac{(m+1)(m+s+1)!(s+r-1)!}{s!(m+s+r+1)!}, \quad r \geq 1 ;$$

(4) the One-parameter Miller distribution, Miller (1961)

$$p_r = \frac{(m+1)(m+1)!(r-1)}{(m+r+1)!}, \quad r \geq 1 .$$

In dealing with empirical platoon-size data, the main difficulty is the different platoon discrimination criteria adopted by various researchers. A number of methods of determining a suitable time headway criterion have been proposed.

Miller (1961) suggested that vehicles are platooning if their time headways are less than 8-seconds and their relative speed in the range -3 to 6 m.p.h. His method was based on the 'random queues' model of traffic flow. He fitted a shifted-negative exponential distribution and tested the goodness-of-fit of the distributions at different points on the time headways axis. He selected the time headway criterion at the point where the fit of the shifted-negative exponential distribution became acceptable at the 5% level of significance. Miller then applied some tests of randomness between consecutive vehicles as well as between consecutive platoons (queues)

and found that the random platoons model was more realistic as a representative description of road traffic than the random vehicles model. The Borel-Tanner and the one-parameter Miller distribution were found to fit the observed frequencies of platoon sizes quite well.

In a second paper, Miller (1982) discussed further the random platoons model with particular regard to the relation between overtaking rates and various factors. Some formulae were derived for the rate of delay to vehicles caused by restricted overtaking for data from rural roads.

Underwood (1963) discussed a three-zones flow concept. He suggested that a zone of normal flow, a zone of force flow based on car-following situations, and a zone of unstable flow, being a transition between the other two zones, may be accepted. We develop this concept in this thesis. He finally discussed the platooning of vehicles and derived that vehicles with time headways less than or equal to 5-seconds can be considered as members of a platoon.

Daou (1966) proposed a model of headways within platoons where distance-headway was considered as a linear function of speed. A log-normal distribution was found to represent the distance headways within platoons satisfactory for each speed class. However, we find a normal more satisfactory. Finally, he arrived at a platooning criterion being a function of platoon speed in which case the time headways range from 2.0 to 4.5 seconds.

Pahl and Sands (1970) arranged the data into time headway ranges; for each range the actual distribution of relative speeds was compared with the distribution of relative speeds, which would arise if consecutive vehicles speeds were independent and normally distributed, by using a modified χ^2 -test. As the time headway increased, the difference

between the two distributions became less marked. The time headway beyond which the difference between the actual and independent distributions of relative speed was no longer significant at the 1%-level was taken as the critical time headway. This critical time headway for platooning was in the range of 2.5 to 4.5 seconds depending on traffic flow rate and lane number.

More recent studies by Keller (1976) and Sumner and Baguley (1978) suggested that a group of vehicles travelling together with headways of 2.0 seconds or less and speed differences less than 10% were formed a platoon. This is in reasonably close accord with our findings.

Taylor et al (1974) compared several platooning models for rural two-lane traffic flow. Vehicles were defined to be members of a platoon if the time headways were less than 8-seconds in one case and less than 5-seconds in the other with speed differential in the range ± 5 m.p.h. The platooning models consisted of a distribution of headways between platoons (a shifted negative exponential) and a distribution of platoon sizes. The platoon-size models were compared in terms of their fit to experimental data collected in Australia, England and Sweden and to some simulated data. For the simulated data the time headways within platoons were considered constant.

The Geometric distribution predicted smaller frequencies of small ($r=1,2$) and large ($r>6$) platoons than the Borel-Tanner and the one-parameter Miller distributions where both appeared to over-predict the proportion of single vehicles. The Borel-Tanner distribution was found to give a reasonable representation of platoon sizes for mean platoon size $\bar{r} < 2$; the two-parameter Miller distribution was found to be more general as it gave satisfactory fits for different ranges of traffic conditions.

Gálin (1980) used Israeli data of two-lane rural roads and a platoon criterion different to that used by Taylor et al (1974) to study some platoon-size models. The platoon criterion applied was a critical time headway of 2.0 seconds and a speed difference between consecutive vehicles of less than 10%, which has been previously suggested by Sumner and Bagulay (1978). The Geometric and One-parameter Miller distributions were found to be less reliable than the Borel-Tanner and the two-parameters Miller distributions. The latter two distributions were found to produce the same results for all the ranges of \bar{r} (mean platoon size), unlike Taylor et al's (1974) results where the Borel-Tanner distribution was found inferior to the two-parameter Miller distribution for high values of \bar{r} .

Finally, it was found that of the parameters m and s of the Miller's distribution each gives a linear relationship with the mean platoon size \bar{r} ; but these relationships can only serve as an approximation since only one parameter can be linear with \bar{r} , according to the following equation:

$$\hat{r} = \frac{m+s+1}{m}$$

where \hat{r} is the maximum likelihood estimator of \bar{r} (mean platoon size).

1.7 Discussion and Conclusions

Irrespective of the reasons for which headway models were developed and used, they showed a number of differences according to the assumption they were based on, and the different road types to which they were applied. All the proposed models differed considerably in philosophy and techniques; this perhaps emphasizes that a fully satisfactory

theory has not been developed yet.

One of the weaknesses of most headway models is that there is no means of generating traffic for road and traffic conditions which are different from those for which the headway data were collected. This is because the parameters defining the distributions are usually not independent and are not related to independent variables which are of interest e.g. flow, road type.

Another weakness is that although a model may be able to generate time headways for the purpose of traffic simulation statistically indistinguishable from the observed distribution, the distribution of platoon-sizes will not be the same. This weakness is not a problem when the interest is the distribution of main road gaps presented to the turning drivers at intersections, but could produce an incorrect waiting time distribution.

It is generally accepted that for an ideal traffic stream in which vehicles do not interact and thus can pass each other freely, the headway distribution will tend toward an exponential function; the series of arrival times will then constitute a Poisson process. These conditions do not prevail very often, with the level of today's traffic!

However, even at flows near the capacity of the road, observed headways can still be represented by an exponential function, but only for headways greater than a critical value T^* (Seconds). These large headway vehicles are regarded as free-movers or Non-follower. Vehicles with headways less than a critical value T^* are regarded as followers. At intermediate headways, a mixture of free-movers and followers may be expected. The definition of this

critical value T^* is always one of the main difficulties in the study of headways.

In chapter three, some results on headways distribution are discussed and a further modification of the semi-Poisson model is presented. Time headways data collected at four-sites in Southern England will be used. Our work differs from earlier work in that we used automatic, electronic, data collection, and thus, for any site, the total volume of data available is large by previous standards.

CHAPTER 2

THE COLLECTION OF DATA

2.1 Introduction

The driver behaviour studied and presented in the following chapters is based on real life observations recorded at sites near priority controlled T-junctions in Southern England.

The experiments were carried out using a microprocessor based data collection system developed by Storr et al (1979). Two different versions of software, each one giving a different time accuracy to the microprocessor, were used.

In this chapter a brief description of the system is given as well as the type of data recorded during the experiments. The two versions of the software used and the processing of the data to their final format are also presented.

2.2 The microprocessor system

The data collection system is based on a microprocessor. The system receives input from either handsets or automatic sensors. Each handset has eight push-button inputs; when a button is pressed, the handset number, the button number and the time at which it was pressed are recorded. Two of the buttons correspond to automatic sensors and they are not pressed by the observer. The sensors are normally coaxial cables. Figure 2.1 shows diagrammatically the structure of the system. The PROM holds the program, written in

assembly language and the RAM chip is the memory. An internal CLOCK gives a time accuracy to the microprocessor, figure 2.1.

The two versions of the software used for the experiments gave a different recorded capability to the system.

The one version of the software was developed at Royal Holloway College; it gives to the microprocessor a time accuracy of 0.01 seconds. There are in all thirty-two input channels, each one representing a specific event which is defined according to the needs of the experiment.

The other version of the software was developed at Sheffield University; it gives to the microprocessor a time accuracy of 0.001 seconds. However, the input channels are now reduced to sixteen by using this version of software.

For both softwares each signal from either a handset or a cable is stored in RAM, holding the time of the event and the channel number. When sixty such events for the first version of software (fifty for the second) are stored in RAM, the program starts the tape recorded and the contents of the memory are transferred to a CASSETTE TAPE.

2.3 Data collected

The first version of the software was used to collect data which will be used in chapters 3,4 and in Appendix-One.

These experiments were carried out during peak-hours in either good or rainy weather conditions in daylight and darkness. Observers

as well as sensors were used during the experiments. The observers were seated inside a car positioned at the site of collection. The experimental sites are T-junctions located in rural or suburban environment.

The second version of the software was used to collect data which will be used in chapter 5. The experiments were conducted in good weather and in daylight. In addition to the use of observers, automatic sensors were laid at specific locations at and near the junction.

Three types of data were usually recorded: The arrival, the departure and the clearance. Observers were positioned on the verge by the junction and recorded the arrival and departure time of turning vehicles (LT and SL) at the points X and Y, figure 2.2. Cable D recorded the clearance time, that is the time at which a vehicle turning right into the minor road cleared the path of the oncoming vehicle. The arrival time of the major road vehicle at the junction was recorded by cable C, figure 2.2.

The speeds of the major road vehicles approaching the junction were measured using a pair of cables A,B at a fixed distance apart, figure 2.2. The speed was calculated from the time interval between the two cables.

2.4 Processing and Analysis of the Data

From the cassette tape the data are transferred to a mainframe computer through a terminal. Every event of the data collected by using the first version of software (accuracy 0.01 secs), originally,

is in the following form: TTTTHBP where the first five letters represent the clock time, H corresponds to a particular handset, B is the button number, and P is a parity digit.

However, every event of the data collected by using the second version of software (accuracy 0.001 secs), originally, is in the following form : XXYYTTTTTT where XX, YY are the current status of handset-one and handset-two and TTTTTT is the time in 0.001 seconds. A program was developed to transform the form of each event produced by the second version of the software to a form similar to that produced by the first version of software.

The next step is to transform the data, collected with one or the other version of software, into a format which contains the actual information about each event. A number of programs is responsible for this. The result is a permanent file where each line of the file corresponds to an event in the form YYMDDCCTTTTSAK. The initial value of YYMDD is specified for each data file and gives the date of the observation; CC gives the clock cycle and TTTTTT the time in that cycle (in 0.01 or 0.001, seconds). The final three digits SAK give the stream, action, and kind of vehicle. Hence, for example, an event could read as 82415 1245812 511 and represents an event on 15th April 1982 at 1245812 time units (in 0.01 or 0.001 secs.) at which time a car arrived in stream RA, figure 2.2.

Finally a number of programs have been written to give the actual information of interest in every case.

2.5 Discussion

The microprocessor based system with the first version of the software (accuracy 0.01 secs) has been successfully used at R.H.C. and a large amount of data have been collected at a lot of sites. Experience showed that it is easy to handle and relatively reliable. More accuracy of measurement is obtained by using automatic sensors than observers.

When the second version of the software was used a greater accuracy of measurement was obtained but in the expense of the collected information since the input channels (thirty-two) were reduced to half (sixteen). It was also found less reliable than the first one, and more time consuming when the tapes were played back to send the data to a mainframe computer. However, because of its accuracy (0.001 secs), it was more appropriate for some experiments (see chapter 5).

Generally, the whole collection system can be considered reliable, and the choice of the software version is a matter of the preferable information needed for the experiment to be conducted.

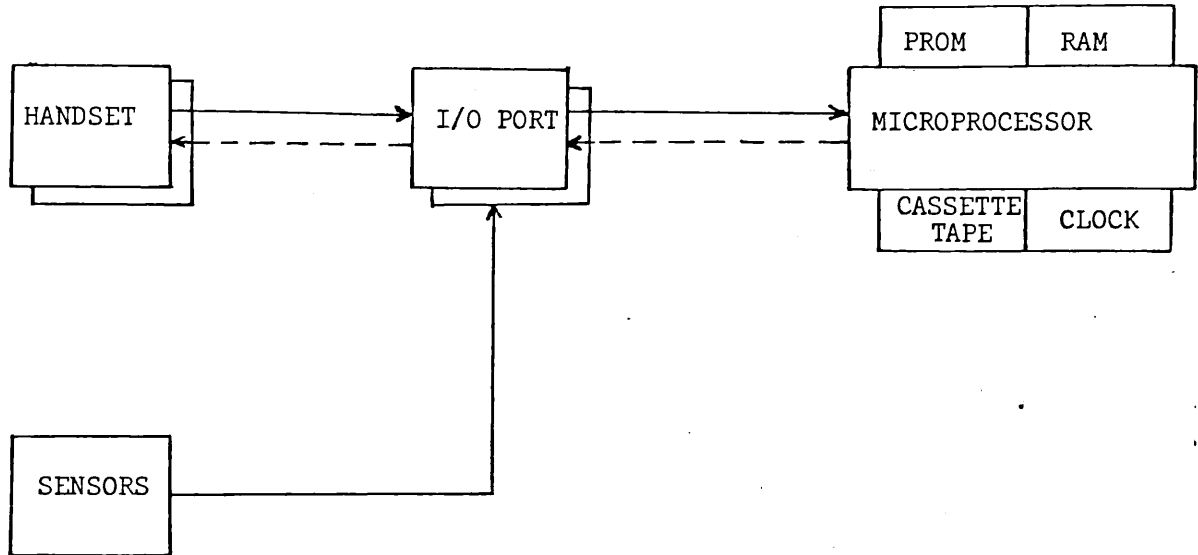


Figure 2.1 The structure of the microprocessor based data collection system.

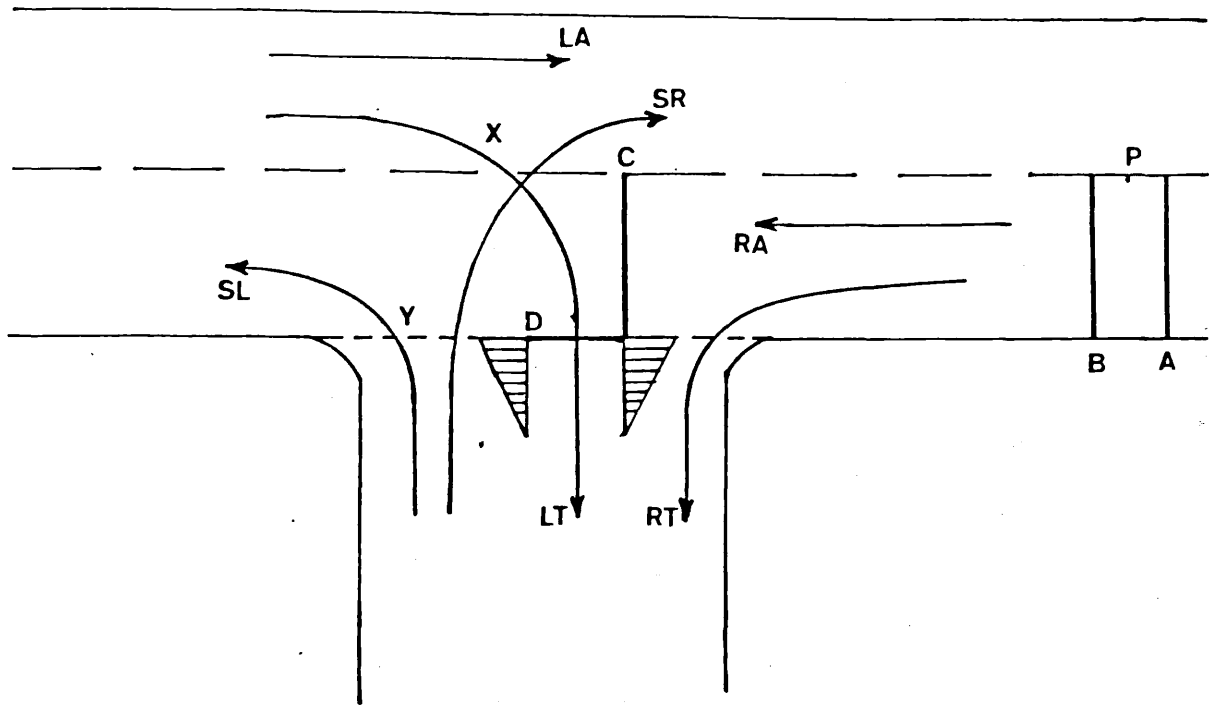


Figure 2.2 Diagram, not to scale, of the priority controlled T-junction and the observed manoeuvres; A, B, C, D indicate the cables. The traffic streams are labelled as follows :
RA = Straight through traffic from the right
RT = Left turn into the minor road
LT = Right turn into the minor road
LA = Straight through traffic from the left
SL = Left turn out of the minor road
SR = Right turn out of the minor road

CHAPTER 3

MAIN ROAD HEADWAYS : A RECONSIDERATION

3.1 Introduction

One of the most important characteristics of a stream of traffic is the distribution of time headways, as it reflects one aspect of the essential nature of the traffic. So it is not surprising that a number of headway models have been developed (Chapter 1) in the last two decades for the purpose of simulation in studies of e.g. capacity, volume, delay and safety. A time headway is defined here to be the time from the front of the vehicle passing a reference point to the front of the following vehicle reaching the same point.

A conflict simulation model was developed at R.H.C. and a headway model was needed that would allow direct sampling from appropriate distributions, for use as input to the model (Chapter 6). A conflict in the model occurs when a minor road driver accepts a gap which forces the mainroad driver to decelerate by any amount, however small. For this reason, short gaps were more important than has been customary; these are the gaps which, if accepted, are more likely to cause a conflict.

Several methods proposed in the literature were used to analyse the available data, but none were found particularly satisfactory. In using Wasielewski's (1974) integral equation method, severe numerical instabilities were found with the data. Branston (1976) introduced a log-normal model for follower headways but he found the fit to his data to be unsatisfactory. Specifically, the data showed large variations about the fit which are unacceptable for

for our purpose.

In this chapter, a brief description of a model, proposed by Ovuworie et al (1980) based on Buckley's (1968) semi-Poisson approach, is given. Further results are reported for data collected at four suburban sites. The study indicates that the proposed three-population model is more satisfactory than the two-population model when short gaps are important. The parameters of the following headways distribution can be held constant for all flow levels without a substantial reduction in goodness-of-fit. Pooled estimates can also be considered for the two critical points distinguishing followers and free movers.

3.2 Sites

The available data are from four sites: Ascot, Chobham, Sandhurst and Shepperton. These sites are located in a suburban environment in Southern England with speed limits of 30 m.p.h. The data were collected during peak-hours in daylight and darkness, and some of them in rainy weather conditions.

All the sites are on single-lane, two-lane roads, at or near intersections where overtaking is prohibited. The sample sizes in a day varied between 300 and 700 vehicles.

3.3 The three-populations headway model

Ovuworie et al (1980) used a further modification of the semi-Poisson model (Buckley (1968)) to obtain appropriate distributions for sixteen sets of data collected at five rural sites. The proposed model was developed as follows.

Let T^* be a value of the time (secs.) headway. Then a truncated

Normal was fitted to the left for an interval $(0, T^*)$ (for the followers) and a truncated exponential to the right for an interval (T^*, ∞) (for the free movers); by incrementing T^* by some amount ΔT (usually equal to 0.2 secs.) the whole fitting process was repeated until a reasonable fit could not be obtained, according to the selected criterion. From the sequence of fits the best for each of the two classes of headways was chosen. The whole process gave rise to a pair of T^* 's (T_1, T_2) in every case and T_1 was always less than T_2 . It was also noticed that relative to the total number of headways observed, the number in the interval (T_1, T_2) was reasonably large. These observations led to the assumption that there might be three populations of headways. Then the fitting process was repeated using a three-population model. A truncated Normal and two negative exponentials with different parameters were now used.

It was found that a headway model including a third intermediate population was far more satisfactory. That additional group of headways was interpreted as composed of vehicles which are either decelerating to join a platoon, or vehicles which have left a platoon and have not yet reached their preferred speed. This is in agreement with the earlier suggestion by Underwood (1963), provided that his "region of unstable flow" is interpreted as the intermediate group. Thus the new headways model suggests the following three subpopulations of drivers. The followers have headways distributed $N(\mu, \sigma)$ truncated on $[T_0, T_1]$; the free-movers, as usual, have an exponential distribution of parameter λ_2 on $(T_2, 10]^*$; and the third group—the

* for all practical purposes $10 = \infty$.

others - has an exponential distribution with parameter λ_1 on $(T_1, T_2]$. Hence the headways were found to be distributed as follows:

$$\begin{aligned} f(t=h) &= \frac{f(h)}{F(T_1)-F(T_0)} \quad , \quad \text{if } T_0 \leq h \leq T_1 \\ &= \frac{g(h)}{G(T_2)-G(T_1)} \quad , \quad \text{if } T_1 < h \leq T_2 \\ &= \frac{l(h)}{L(10)-L(T_2)} \quad , \quad \text{if } T_2 < h \leq 10 . \end{aligned}$$

where f, g, l and F, G, L are the probability distribution functions and the corresponding cumulative distribution functions of the Followers, Others and Free-movers respectively. One shortcoming of the three-population model of headway is that it contains six parameters. However, pooled estimates can be made of four of the six parameters, leaving only λ_1 and λ_2 site-dependent.

3.4 Results and discussion

The model has been used by Ovuworie et al (1980) to describe data from five rural sites with flow levels 450-970 veh/hour and speed limit 60 m.p.h. The minimum observed headway was 0.20 seconds. In what follows, the main analysis is concerned with data from four suburban sites with flow levels 300-700 veh/hour and speed limit 30 m.p.h. The minimum observed headway is 0.40 seconds. When a comparison is made between parameters of suburban and rural sites, the values of the parameters for the latter sites have been taken from the earlier work of Ovuworie et al.

Table 3.1 gives the results from 15 sets of the suburban data.

The following parameters are tabulated.

- (1) μ, σ : the mean and standard deviation of the underlying Normal distribution whose truncated form describes Followers headway.
- (2) $\lambda_{OT}, \lambda_{FM}$: the exponential parameters for Others and Free-movers respectively.
- (3) P_{FO}, P_{OT}, P_{FM} : the calculated upper tail values of the χ^2 -distribution for each interval.
- (4) T_{FO}, T_{FM} : the critical points on the time (secs) axis, for Followers and Free-movers.
- (5) F : main-road flow per hour.

Values of $p_k > 0.05$ indicate a tendency towards a good fit.

From table 3.1 it is noted that the quality of fit is very satisfactory for almost all the sets of data. Figures 3.1 and 3.2 give the visual fits for two sets (ASLD27, CHWL).

The parameters μ and σ , of following headways, varied in the ranges 0.25 to 0.48 (secs.) and 1.17 to 1.49 (secs.) for the rural-sites data. Two pooled values of $\sigma = 0.36$ and $\mu = 1.30$ (secs.) have been given by these sites.

The variability of μ and σ for the suburban-sites data is in the range 0.29 to 0.45 (secs.) and 1.40 to 1.64 (secs.) respectively.

Both parameters μ and σ seem to be independent of site and flow, for the available flow levels for the suburban sites. However, plots of these parameters versus flow (figure 3.3) for all the sets of data from rural and suburban sites show that μ and σ tend to decrease for flow-levels over 700 veh/hour. This tendency is more

consistent for the mean (μ) while the standard deviation (σ) is more scattered.

For the suburban sites data good approximations of $\sigma = 0.36$ and $\mu = 1.50$ (secs.) can be taken. In Figures 3.4 and 3.5 a comparison is made between the prediction of the truncated Normal with constants μ and σ with the best values. From the visual fit the simplification of keeping the two-parameters constant is clearly seen to be acceptable.

The assumed constant value for σ is the same for the data of both sets of sites, rural and suburban; but the value of μ is larger by 0.20 (secs.) in the latter case. This may be due to the fact that for the suburban sites the minimum observed headway was 0.40 (secs.) and subsequently the truncation point for the followers were shifted to the right by 0.20 secs. compared with the rural-site data. This can be explained from the higher flows in the rural sites. If the two exponentials are replaced by a single one the fit is not satisfactory. The comparison made in Figures 3.4 and 3.5 shows that there is a substantial mismatch for headways between 2.0 and 4.5 seconds. Hence the general conclusion is that the assumption of constant μ and σ , leaving the other two-parameters λ_{OT} and λ_{FM} site-dependent, can be an acceptable simplification in fitting the three-populations model.

The values of the critical points T_{FO} , T_{FM} seem to be independent of site and flow as well. The critical point T_{FO} , for followers, varies in the range 1.8 to 2.4 seconds (1.4 to 2.2 - rural sites); and the critical point T_{FM} , for free movers, varies in the range 3.2 to 3.8 seconds (2.8 to 3.8 - rural sites). Two pooled estimates

of $T_{FO} = 1.6$ and $T_{FM} = 3.4$ have been given for the rural-sites.

For the suburban sites, pooled estimates of $T_{FO} = 2.0$ and $T_{FM} = 3.4$ seconds can be considered satisfactory.

Table 3.2 shows the sample sizes as well as the proportions of the three populations of headways. For 9 out of 15 sets the proportion of followers is higher than the proportion of free movers. The proportion of drivers in the intermediate group -others- is always smaller than the proportion of followers and free movers, but can certainly not be ignored.

3.5 Conclusions

In using a headway model as input into a simulation model, three main aspects are considered:

- (a) the primary objectives of the simulation task;
- (b) the simplicity of the model to generate the headways;
- (c) the quality of the goodness-of-fit to the data.

In running the conflict simulation model of Darzentas et al (1980), small and intermediate headways are most important. For this reason, the three-population model seems more appropriate than the two-population model as it gives a very satisfactory goodness-of-fit to the available data in the important ranges. The results are also satisfactory even in the case where pooled estimates for four ($\mu, \sigma, T_{FO}, T_{FM}$) out of six parameters were used.

Generally, the three-population model has been tested against a wide range of data from rural and suburban environments with flow levels up to 1000 veh/hour. It seems more satisfactory than the two-population model when small and intermediate headways are of

primary interest; the two-population model will be quite satisfactory when free-movers predominate.

Data set	Parameter									
	μ	σ	λ_{FM}	λ_{OT}	P_{FO}	P_{OT}	P_{PM}	T_{FO}	T_{FM}	Flow/hr.
ASWN14	1.60	.38	.18	1.01	.64	.85	.88	2.2	3.6	680
ASWN24	1.61	.41	.23	1.37	.13	.61	.23	2.2	3.2	520
ASDN14	1.55	.42	.21	.82	.10	.95	.90	2.2	3.6	660
ASDN35	1.54	.35	.25	.87	.27	.22	.40	2.0	3.6	700
ASLD27	1.62	.45	.16	.92	.35	.84	.66	2.4	3.8	570
ASLD55	1.53	.33	.24	1.18	.21	.61	.92	2.0	3.6	690
CHWN1	1.64	.43	.10	.51	.33	.63	.76	2.4	3.6	390
CHWL13	1.53	.33	.08	1.04	.08	.90	.95	2.0	3.2	410
CHWL2	1.40	.34	.14	1.12	.11	.78	.95	1.8	3.2	300
CHDL24	1.41	.37	.07	.82	.25	.47	.38	2.0	3.8	440
CHDN45	1.50	.40	.08	1.11	.70	.81	.18	2.2	3.4	395
CHDN13	1.40	.30	.12	.88	.25	.95	.19	1.8	3.2	370
SADL1	1.60	.35	.11	1.27	.37	.33	.59	2.2	3.2	370
SADN1	1.63	.37	.15	.95	.65	.74	.98	2.0	3.2	360
SPDL47	1.41	.34	.09	1.30	.25	.55	.84	1.8	3.0	400

Table 3.1 Parameters, in seconds, of the fitted headway model together with the critical points and flow levels for different sets of data from suburban sites. (In the Data set codewords, AS=AScot, CH=CHobham, etc, W=Wet, N=Night, etc.)

Data set	Followers	Others	F.Movers	Total	P _{FO}	P _{OT}	P _{FM}
ASWN14	249	194	255	698	.36	.28	.36
ASWN24	248	95	262	605	.41	.16	.43
ASDN14	263	157	229	649	.40	.25	.35
ASDN35	224	178	223	625	.36	.28	.36
ASDL27	323	122	228	673	.48	.18	.34
ASDL55	247	192	258	697	.36	.27	.37
CHWN1	262	80	200	542	.48	.15	.37
CHWL13	118	72	97	287	.41	.25	.34
CHWL2	153	146	197	496	.31	.29	.40
CHDL24	118	85	115	318	.37	.27	.36
CHDN45	160	66	82	308	.51	.21	.27
CHDN13	96	83	102	281	.34	.29	.37
SADL1	169	74	189	432	.39	.17	.44
SADN1	131	56	143	330	.40	.16	.44
SPDL47	75	70	108	253	.30	.27	.43

Table 3.2 The sample sizes together with the number and proportion assigned by the headway model to each of the three classes.

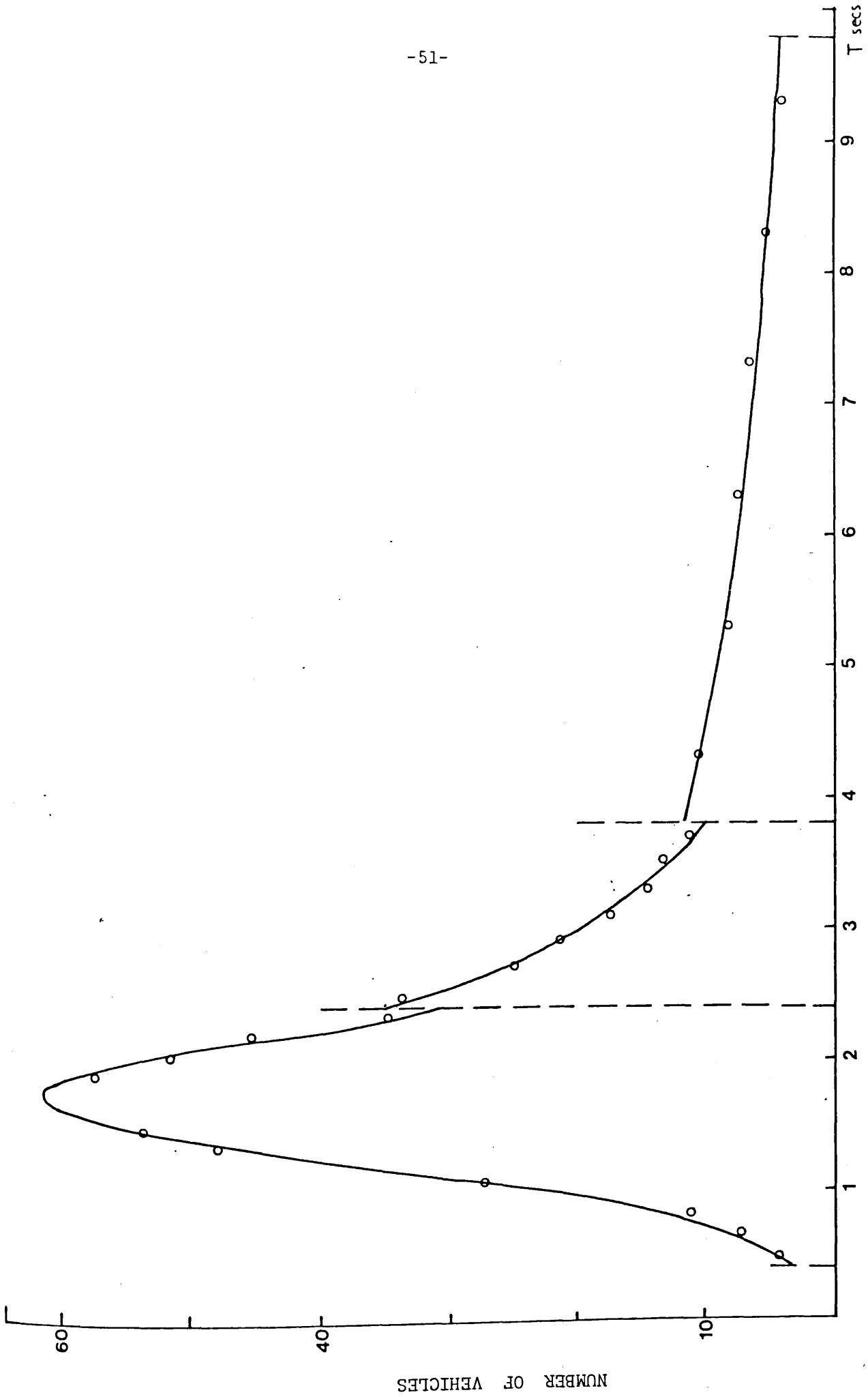


Figure 3.1 Observed headway distribution and model fit for a set of observation (ASLD27) at Ascot.

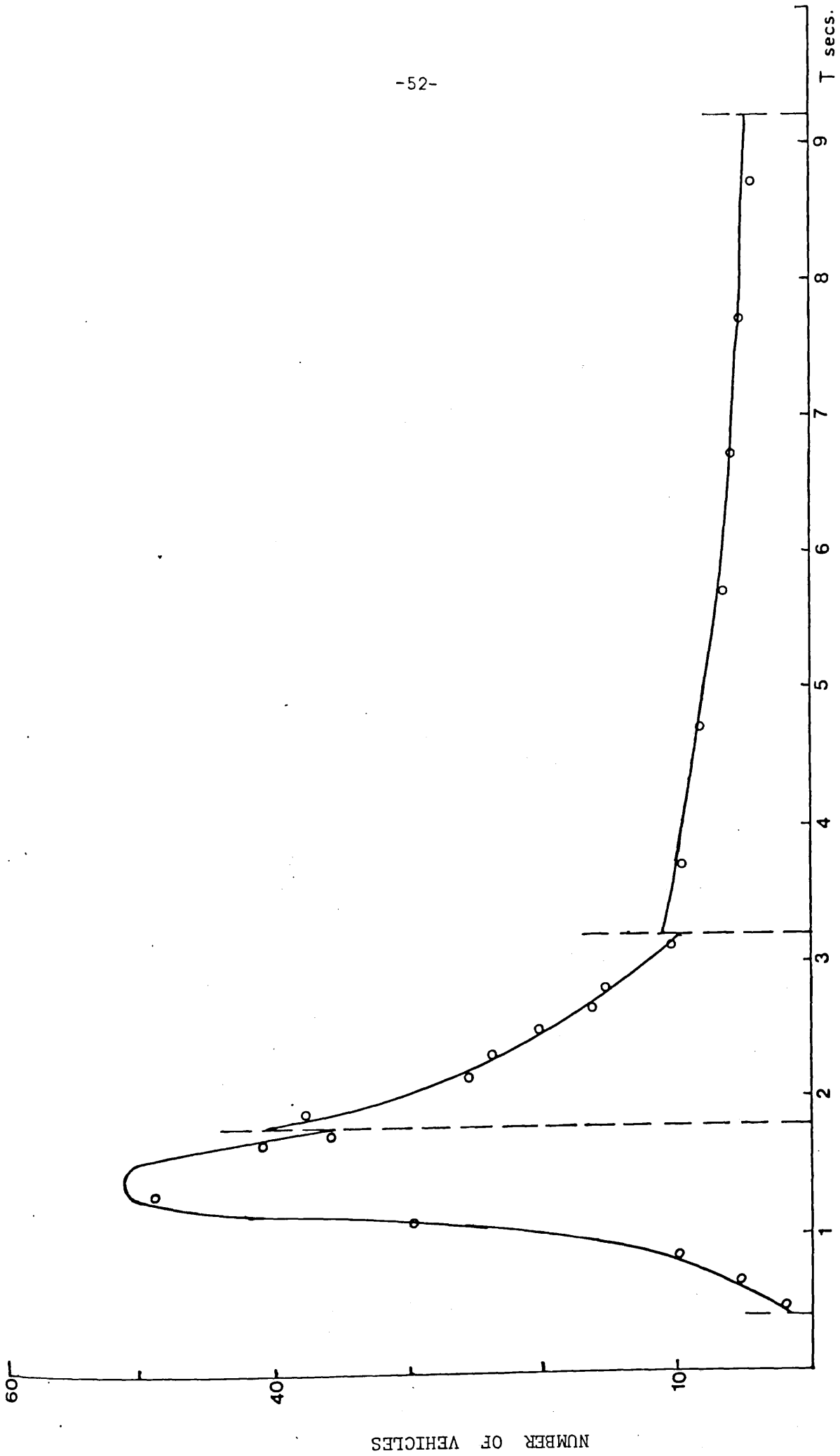


Figure 3.2 As figure 3.1, but for a set of observation (CHWL) at Chobham.

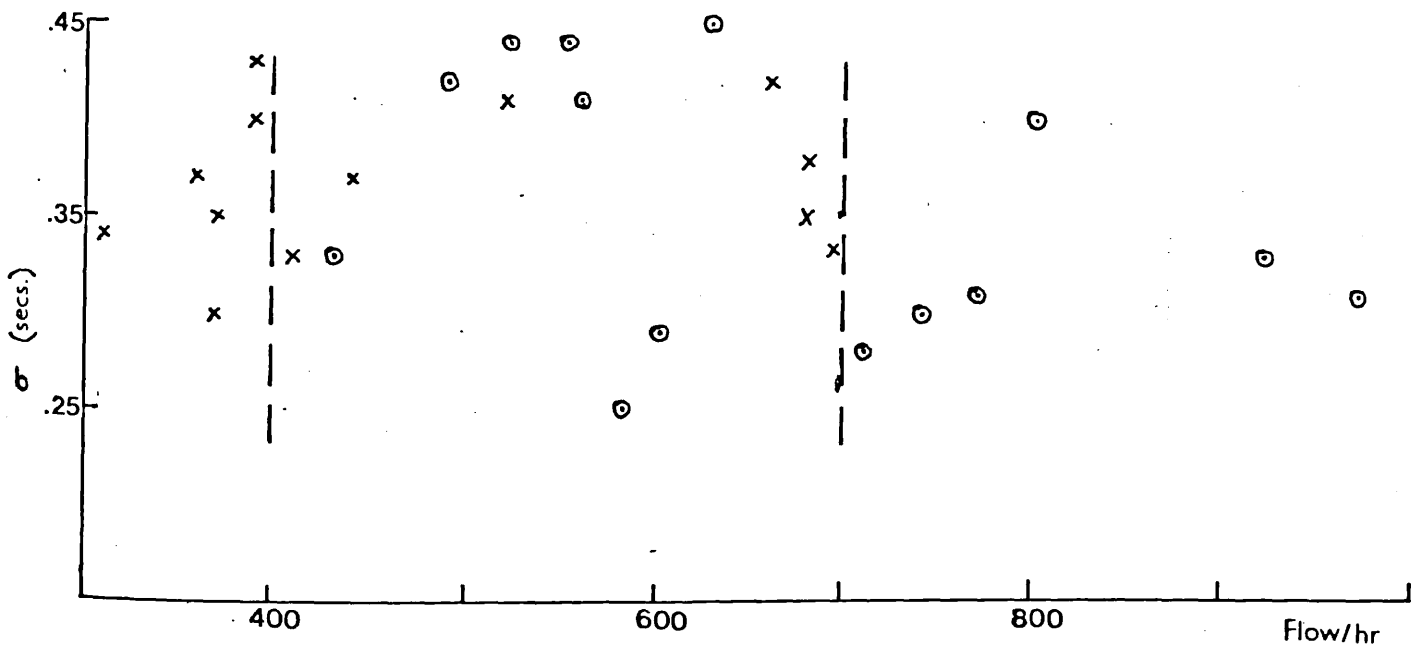
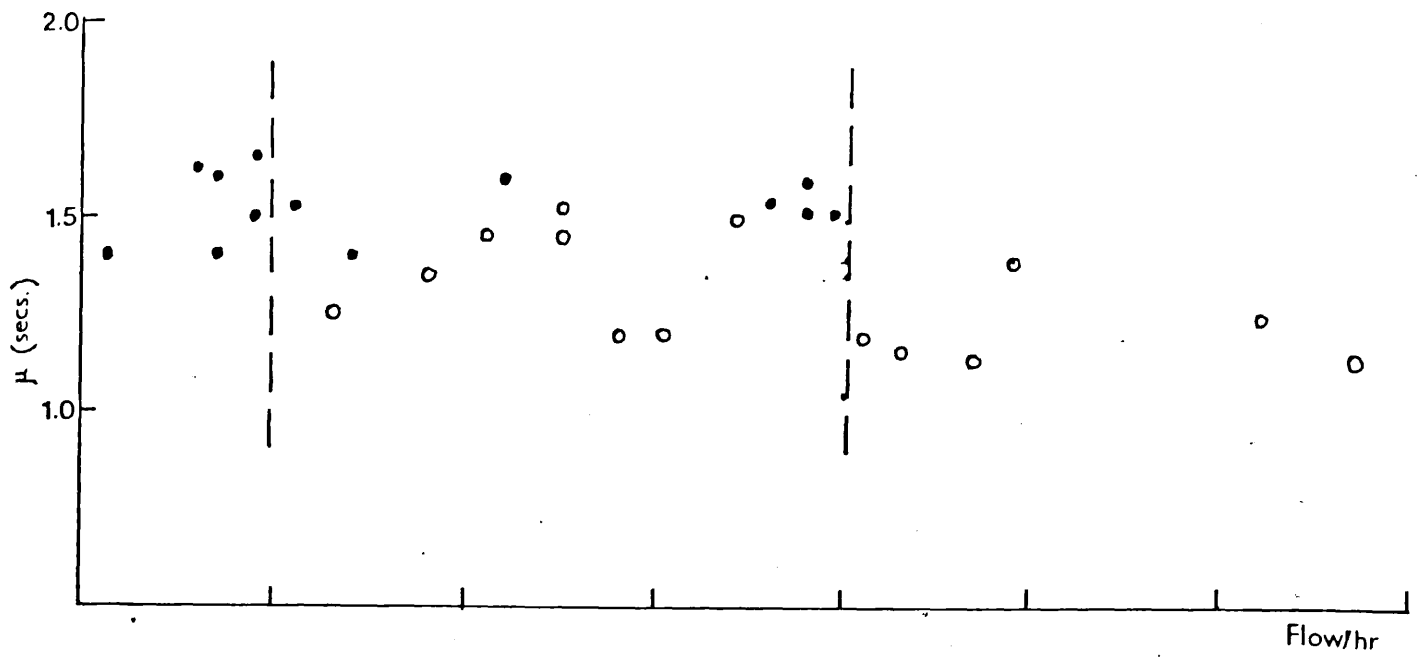


Figure 3.3 Plots of parameters μ and σ of the following headway distribution versus flow for data from the suburban and rural sites. [(•,x) - indicate suburban sites, (○,θ) - indicate rural sites].

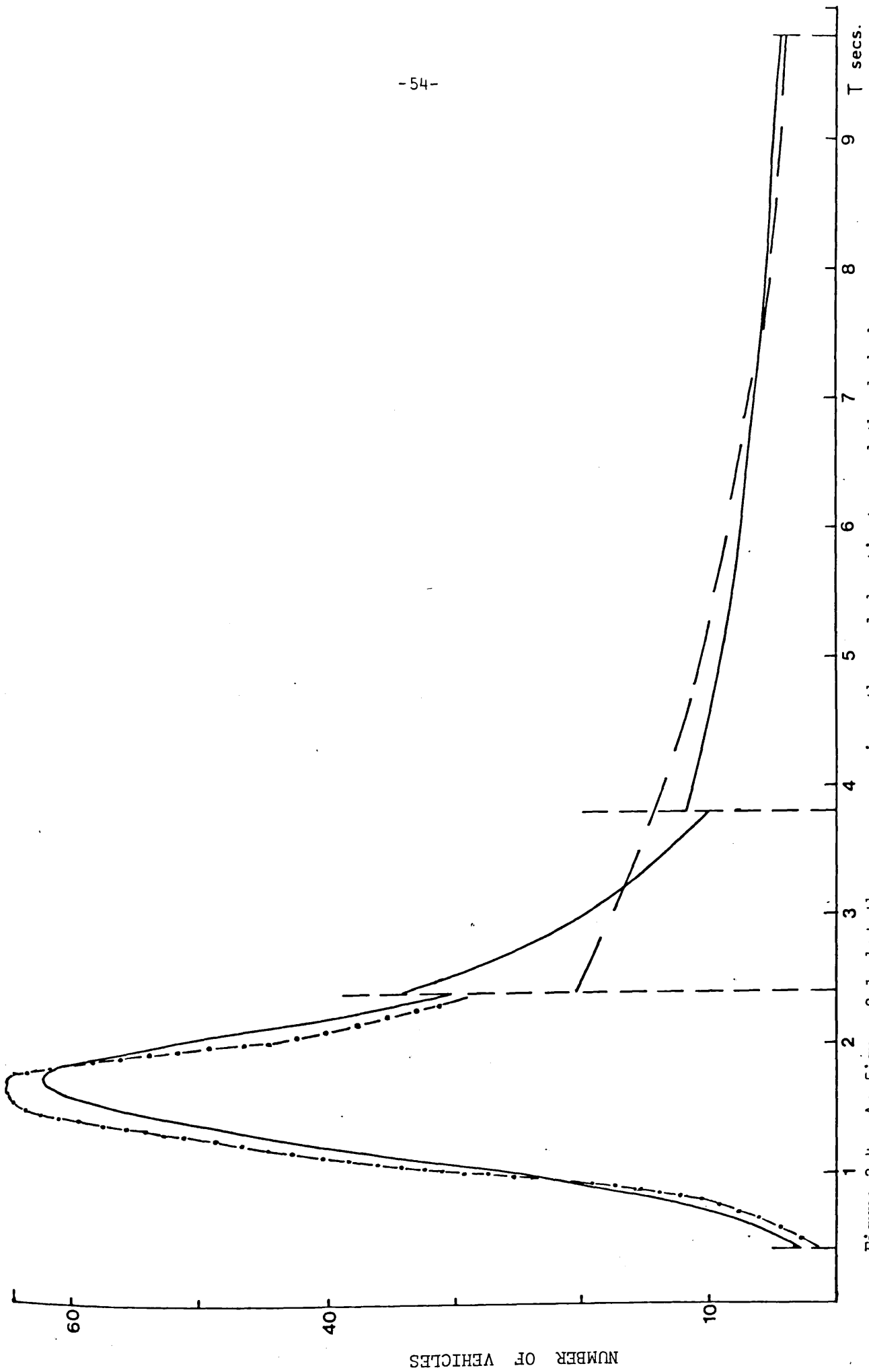


Figure 3.4 As figure 3.1, but the curve \cdots gives the pooled estimate, and the dashed curve $---$ the single exponential fit, at Ascot.

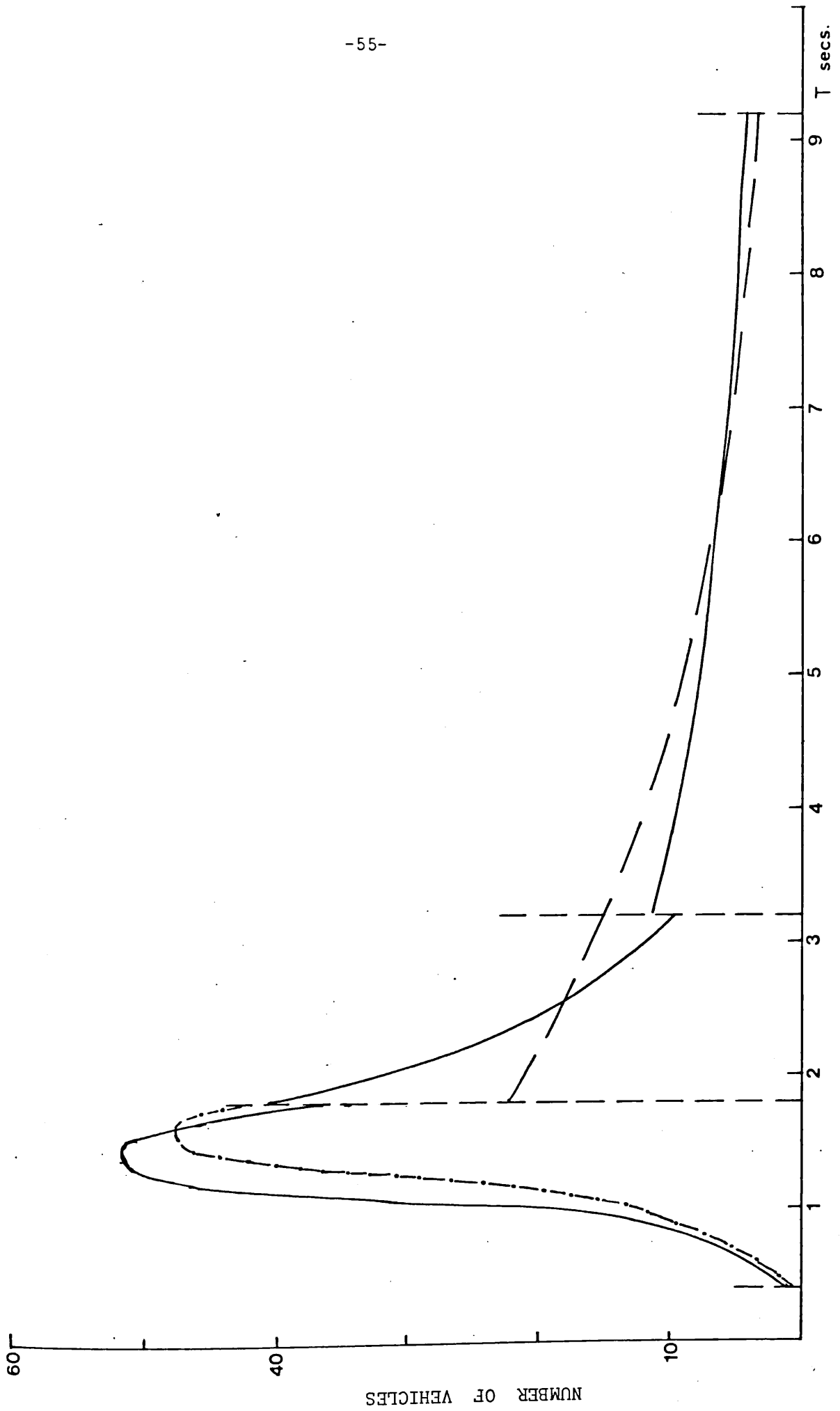


Figure 3.5 As figure 3.4, but for Chobham.

CHAPTER 4

ASPECTS OF HEADWAY DISTRIBUTIONS AND PLATOONING ON MAJOR ROADS

4.1 Introduction

Time headway distributions were presented in chapter three following an earlier paper by Ovuworie et al (1980). It was shown that headways could be described as if the drivers belonged to one of three classes:

- (a) free movers;
- (b) joining or leaving a platoon; and
- (c) followers or members of a platoon.

This is in agreement with the earlier suggestion in Underwood's work (1966), provided that his "region of unstable flow" is interpreted as the category (b).

In this chapter, first the autocorrelation between successive time headways in the main road streams are considered; then the distribution of gaps between platoons (the gap between the last vehicle of a platoon and the leader of the subsequent one) and thirdly the distribution of platoon sizes (number of vehicles in a platoon) is presented. The data used are from rural and suburban sites with different speed limits and flows, although the distribution of headways within platoons is only considered for data from three rural sites - Puttenham, Compton and Peasmarsch - where speed data was available.

The primary purpose of the investigation was to identify any autocorrelation in the time headways in order to use it in the

sampling process based on the distributions of headways. However, information of more general interest was obtained, especially for the rural sites, and is the main subject of this chapter.

4.2 Data

For work not involving a knowledge of the speed data from rural and suburban sites were used; that is, the same data as in chapter 3 and in the paper by Ovuworie et al (1980).

For the three rural sites - Puttenham, Compton and Peasmarsch - speed data for each main-road vehicle were collected. The speeds were measured in terms of the time to cross two detectors separated by 88 inches, with a clock accurate to 0.01 seconds, and the speeds are therefore accurate to better than 5 m.p.h. at 40 m.p.h. Sufficient data at each site were available for separate analysis and the sample sizes are given in Table 4.1.

4.3 Autocorrelation in main-road streams

The previous work by Breiman et al (1968,1972) refers to highways (motorways) in Detroit; each lane was treated as a separate stream. In the first paper, Breiman et al (1968) studied the underlying stochastic process of the sequence of headways (time) for freely-flowing traffic. An attempt was made to determine the validity of the hypothesis that successive headways are independent. They determined periods of time for which the rate of flow of vehicles was without trend. They tested the hypothesis of independence within these periods for eight sets of data. Using a variety of tests, including correlogram analysis, they found that this hypothesis could

be accepted at the 5% level in all, except possibly one, of their eight sets of data. They deduced that the headways followed a negative exponential distribution. This differs from some other results (chapter 3, Ovuorie et al (1980)) that, for headways less than about two seconds, a more complicated model is required. They give no information on whether a significant number of such headways occurred in their sample

In a second paper, Breiman et al (1977) reinvestigated the headway process in time and space. The hypothesis of independence in time and space was rejected only six times out of 48 sets of data at the 5% level. There was evidence of some correlation at the heaviest flow (> 1000 veh/hr). Headways (both time and space) were described by a negative exponential distribution except for times less than 3.0 seconds and distances less than 300 ft. Vehicle speeds were normally distributed, and successive vehicles' speeds were highly correlated. In addition a small correlation (0.29) between speed and headway was found if the flow was greater than 1,000 veh/hr. We have applied a similar analysis to data collected on U.K. suburban and rural roads, on which the opposing streams are not physically separated. In particular, the data was collected near intersections with major roads, at rush hours, under which conditions overtaking is very rare.

The following question was asked; is the j -th headway in a stream correlated with the $(j+k)$ -th for $k=1,2,3\dots$? The answer is contained in the autocorrelation coefficients r_k . These are defined by

$$r_k = c_k/c_0, \quad k=1, \dots, m < n \quad (4.1)$$

$$c_k = \frac{1}{n-k} \sum_{j=1}^{n-k} (x_j - \bar{x})(x_{j+k} - \bar{x}) \quad (4.2)$$

$$c_0 = \frac{1}{n} \sum_{j=1}^n (x_j - \bar{x})^2 = \text{Var}(x) \quad (4.3)$$

where n is the sample size, \bar{x} the mean headway and x_j the headway to the following vehicle. The results for a number of streams from the suburban and rural sites, for r_1, \dots, r_{10} , are given in tables 4.2 and 4.3 respectively. All the values are small, and consistent with there being no significant autocorrelation. If a time series is in fact random, then $\bar{r}_k \approx 0$, $k \neq 0$ and, when n is large, the coefficients r_k are approximately normally distributed, $N(0, 1/n)$ (Cox et al (1966)). Therefore, we can plot 95% confidence limits on r_k as $\pm 2n^{-1/2}$: if the calculated values lie outside these limits, the series is likely to be non-random. Two typical such correlograms are shown as Figures 4.1 and 4.2, and indicate that the headways are randomly distributed. Two other tests, the turning points test and the rank test - Kendal's τ [Kendal (1973)], were applied to the same sets of data; both tests confirmed the same results. Note that one cannot ask the same question about platoons only, unless their size is large. For the available data, a mean platoon size of 3 is found which is too small to allow an investigation of correlation within platoons. Generally the results are in agreement with that of Breiman et al (1977) and establish that for single streams on suburban and rural

roads and motorways the time gaps may be considered as independently distributed for flow levels less than 1000 veh/hr. However, Breiman's work leaves a question about small gaps which is addressed later, but only for the rural sites, as speed data are available only for these sites.

4.4 Time headways between platoons

We have earlier defined (chapter 3), somewhat arbitrarily, what we shall mean by a platoon. A platoon occurs if the leading vehicle, B_0 , is at least T_i seconds behind its predecessor, and the platoon consists of a stream $B_0, B_1, B_2, \dots, B_{n-1}, B_n$ in which no headway $z_k = t(B_k) - t(B_{k-1})$, $k = 1, 2, \dots, n$ is greater than T_i , where T_i is a location-dependent measured parameter for the i -th site. The time headway between successive platoons P and P' is $t(B'_0) - t(B_n)$. It was found that for the suburban sites - Ascot, Chobham and Sandhurst - the time gaps between successive platoons follow a negative exponential distribution; and for the three rural sites - Tongham, Peasmarsh and Compton - the time gaps between successive platoons follow approximately a negative exponential distribution, though the Puttenham data suggests a relatively high frequency of small gaps.

4.5 Distribution of platoon sizes

Each platoon comprises a leader and followers. Vehicles in classes (a) and (b) (see section 4.1) are not considered to be a platoon.

The mean platoon size for all the sets of data from the rural

sites was greater than the mean platoon size of the data from the suburban sites. Generally, larger platoons were observed at the rural sites.

Two distributions were fitted to the platoon sizes (that is, the number of vehicles in a platoon) and a comparison between them was made. Two of the most widely used distributions (see chapter 1) for the platoon sizes are

(i) the Geometric distribution,

$$p_r = (1-\alpha)\alpha^{r-1}, \quad r \geq 1, \text{ and}$$

(ii) the Borel-Tanner distribution

$$p_r = \frac{(r\alpha e^{-\alpha})^{r-1} e^{-\alpha}}{r!}, \quad r \geq 1,$$

where r is the platoon size, and $\hat{\alpha} = 1 - 1/\bar{r}$ is the maximum likelihood estimator of α . Two modified distributions, which do not include $r=1$ terms are

(iii) modified Geometric distribution

$$p_r = (1-\alpha)\alpha^{r-2}, \quad r \geq 2,$$

where the maximum likelihood estimator of α is $\hat{\alpha} = (\bar{r}-2)/(\bar{r}-1)$; and

(iv) modified Borel-Tanner distribution

$$p_r = \frac{(r\alpha e^{-\alpha})^{r-1} e^{-\alpha}}{r!(1-e^{-\alpha})}, \quad r \geq 2,$$

where in this case the maximum likelihood estimator of α is obtained from the equation

$$\frac{(\bar{r}-1)}{\alpha} - \bar{r} + \frac{1}{e^{\alpha}-1} = 0.$$

The observed and the expected frequencies (f_r) of platoon sizes r , the latter being derived from (iii), together with the total number of platoons and the mean platoon size $\bar{r} = \sum r f_r / \sum f_r$ are given in tables 4.4 and 4.5 for six sets of data from the rural and suburban sites. The modified Geometric distribution gives an excellent description of the data in each case. The χ^2 -test shows (table 4.6) that there is no significant difference between the observed and expected frequencies of platoon sizes. However, the modified Borel-Tanner distribution fails to describe the data.

Other authors have used the unmodified Geometric and Borel-Tanner distributions to describe data sets which include our classes (a) and (b) as 'platoons' of size one. These are inappropriate in our case. It is noted that in any case they give much poorer fits. Miller's one-parameter distribution of platoon sizes, Miller (1961), has not been fitted since the mean platoon size for the available data is close to 3 and therefore too large for this approach, while his two-parameter distribution was not fitted as it is unnecessarily complicated compared with the Geometric.

4.6 Distribution of time headways with platoons

In chapter 3 it was shown that the distribution of time headways of followers at suburban sites may be described by a truncated normal distribution; the same results also hold for data from rural sites as was shown by Ovuworie et al (1980) in a previous study. This suggests the hypothesis that, for a given flow, drivers in platoons appear to maintain a constant headway, modified in practice by other factors such as skill. Since speed data for each vehicle is available for the rural sites, a further analysis of these time headways within

platoons was made to attempt to elucidate this behaviour.

The mean speed of vehicles in a platoon was calculated and the platoons were classified into 5 m.p.h. (mean) speed bands, for each site separately. For each speed class, with sufficient sample size, the time headways for all the platoons of the class were classified into 0.2 second intervals. The mean and standard deviation for each speed class are shown in table 4.7. A truncated normal distribution was fitted to each class using a standard minimizing routine. A χ^2 -test was used to test the quality of the fit. Table 4.7 gives the results for three sets of data. In all cases, the hypothesis that the time headways follow a truncated normal distribution for each speed class could not be rejected at the 5% level of significance except for the third set (Compton) for the 35-40 m.p.h. class.

Table 4.7 also shows that the mean time headways and the standard deviations of each speed class were almost equal for the same site. Bartlett's test showed no significant differences between the variances of the speed classes for each site. The values of χ^2 were not significant at the 5% level.

A one-way analysis of variance was applied for each site. For all sets of data the hypothesis that the mean time headways were equal, i.e.

$$H_0 : \bar{x}_1(i) = \bar{x}_2(i) = \bar{x}_3(i) = \bar{x}_4(i)$$

where $i=1,2,3$ represent the site, and \bar{x}_j , $j=1,2,3,4$ are the mean time headways for speed class j , cannot be rejected at the 5% level.

So it is concluded that, at a given site, a truncated normal distribution

can describe the time headways within each speed class, and that the mean and standard deviation are independent of speed. That is, provided that the definition of platoon given above holds, the distribution of headways within platoons, at a specific site, is independent of speed.

To understand further the above results, the distance headways were considered and an investigation of the relationship between distance headway and speed was made, both for vehicles within platoons and for all vehicles (see figures 4.3, 4.4 and 4.5).

Distance headways were estimated from time headway and speed for each vehicle (in ft/sec). These headways are plotted according to speed and are shown in figures 4.3, 4.4 and 4.5 for these sets of data, where dots indicate vehicles within platoons and crosses those not in a platoon. The curve D_{HC} is the Highway Code curve representing the minimum stopping distances according to the speed. These figures show that almost all drivers with $T \leq 2.0$ ($T \leq 1.8$ for Compton, where T is time headway) are following too closely for, say, $30 \leq v \leq 65$ ft/sec. In effect, the definition of platoon given above, closely corresponds to the set of vehicles which are following too closely.

The proportion of followers (i.e. second and subsequent members of platoons) as a function of speed, is given in Table 4.8 for each site, and in general, decreases with increasing speed - the Compton results are uncertain because the low speed data was sparse.

It may be supposed that the mean distance headway for each speed band would be greater than or equal to the Highway Code recommendation $D_{HC}(v)$. However, since the proportions of vehicles with distance-headways below the D_{HC} curve were relatively high, for each 5 ft/sec.

band the ratio of the mean distance-headway of those vehicles below the D_{HC} curve to the value of D_{HC} corresponding to the centre of that band was calculated. Table 4.9 shows that this ratio tends to decrease as the speed increases. That is, as the speed increases, the amount by which drivers in platoons follow too closely increases. This indicates that faster drivers are more in danger according to the Highway Code standards.

Mackie and Russam (1975) carried out an experiment where different instructions about spacing between following vehicles were given to the drivers. They found that up to 17% of drivers at 30 m.p.h. and 40 m.p.h. followed at less than 1-second and as many as 33% felt that the Highway Code recommendations of 2.0 seconds was too great.

However, real-life observations show (Table 4.10) that, with the exception of Peasmarsh, more than 20% of all drivers follow at half or less of the recommended separation, and, of those in platoons, more than 30% follow at similarly short separations. Further, [Table 4.10] for all sites more than 20% of all drivers follow at less than three-quarters of the recommended separation, and of those in platoons at least 50%, this figure being as high as 86% at Compton. From a previous study of these sites, McDowell et al (1983) of the sites investigated Compton has the highest risk of conflict.

The most important result of our investigation is the demonstration that almost all followers follow too closely, in terms of Highway Code advice, and the amount by which they do so increases with speed, while the proportion following at less than three-quarters of the recommended distance can be as high as 45%.

Tongham	1	578
Tongham	2	752
Puttenham		548
Compton	1	570
Compton	2	976
Peasmarsh		570
Ascot	1	775
Ascot	2	800
Chobham	1	440
Chobham	2	495
Sandhurst		370
Shepperton		450

Table 4.1 Typical sample sizes for each site for a single run.

In investigating the distribution of gaps in platoons (rural sites) larger samples were used by combining several runs from the same site.

Set	r_1	r_2	r_3	r_4	r_5	r_6	r_7	r_8	r_9	r_{10}
TONGHAM 1	.029	-.053	-.059	.050	.026	-.062	.015	.001	.014	.012
TONGHAM 2	.143	.069	-.035	.007	-.021	.007	.057	.030	.043	-.020
PUTTENHAM	.037	.010	-.050	-.012	.005	-.097	.008	-.027	-.051	-.014
COMPTON 1	-.026	-.038	.041	-.044	-.051	-.020	.004	.065	-.019	-.066
COMPTON 2	.012	-.045	.041	-.034	.009	.008	.040	.010	-.013	-.055
PEASMARSH	.031	.004	.122	.034	.014	-.001	-.035	.000	.021	.004

Table 4.2 Autocorrelation coefficients r_k ($k = 1, 2, \dots, 10$)
of time gaps for various data sets at rural sites.

Set	r_1	r_2	r_3	r_4	r_5	r_6	r_7	r_8	r_9	r_{10}
ASCOT 1	-.005	.005	-.003	.014	-.004	-.005	.011	.004	-.008	-.008
ASCOT 2	.046	.059	-.023	.007	.003	-.062	.047	-.011	.012	.100
CHOBHAM 1	-.045	-.011	-.005	.050	.025	-.041	.032	-.077	-.019	-.071
CHOBHAM 2	.022	-.051	.051	-.010	-.016	.048	.045	.023	.029	-.032
SANDHURST	-.063	.003	-.023	-.028	-.046	-.023	.015	.022	.054	.029
SHEPPERTON	.025	-.035	.057	.013	-.018	-.066	.037	.022	.064	.047

Table 4.3 As table 4.2, but for the suburban sites.

	r	F(Obs)	F(calc)	
Flow 545 veh/hr N = 124 $\bar{r} = 3.2$	2	58	56.4	
	3	29	30.7	
	4	13	16.7	PUTTENHAM
	5	11	9.1	($T \leq 2.0$)
	6	6	5.0	
	> 6	7	6.1	
Flow 425 veh/hr N = 171 $\bar{r} = 2.7$	2	100	100.7	
	3	47	41.4	
	4	11	16.9	PEASMARSH
	5	7	7.0	($T \leq 2.0$)
	> 5	6	4.5	
Flow 630 veh/hr N = 236 $\bar{r} = 3.0$	2	120	119.2	
	3	61	59.0	
	4	23	29.2	TONGHAM
	5	17	14.4	($T \leq 2.0$)
	6	7	7.1	
	7	4	3.5	
	> 7	2	3.4	
Flow 925 veh/hr N = 189 $\bar{r} = 2.8$	2	107	105.1	
	3	38	46.7	
	4	23	20.6	COMPTON
	5	12	9.1	($T \leq 1.8$)
	> 5	9	7.6	

Table 4.4 Observed and expected frequencies of platoon sizes $r(r > 1)$.
calculated in the modified Geometric model at various rural sites.

	r	F (Obs)	F(calc)	
Flow 660 veh/hr. N = 158 $\bar{r} = 2.6$	2	105	101.4	
	3	30	36.0	ASCOT
	4	14	13.1	(T ≤ 2.0)
	5	6	4.7	
	> 5	3	2.3	
Flow 370 veh/hr N = 79 $\bar{r} = 2.5$	2	53	52.6	
	3	16	17.6	CHOBHAM
	4	7	5.9	(T ≤ 2.0)
	> 4	3	3.0	

Table 4.5 As table 4.4., but for the suburban sites.

Site	Geom.	d.f.
Puttenham	1.71	4
Peasmarsh	3.03	3
Tongham	2.60	5
Compton	3.13	3
Ascot	1.55	3
Chobham	0.50	2

Table 4.6 χ^2 - table.

Mean (m.p.h.) Platoon speed	Sample sizes	Mean	s.d.	χ^2	d.f.	
20 - 25	-	-	-	-	-	PUTTENHAM ($T \leq 2.0$)
25 - 30	199	1.21	.39	6.34	6	
30 - 35	303	1.20	.38	10.43	6	
35 - 40	167	1.32	.40	4.70	6	
40 - 45	45	1.24	.36	-	-	
20 - 25	68	1.32	.35	1.55	6	PEASMARSH ($T \leq 2.0$)
25 - 30	133	1.34	.41	3.09	6	
30 - 35	194	1.31	.40	8.54	6	
35 - 40	122	1.35	.40	9.20	6	
40 - 45	-	-	-	-	-	
25 - 30	-	-	-	-	-	COMPTON ($T \leq 1.8$)
30 - 35	124	1.06	.36	4.56	5	
35 - 40	237	1.07	.36	13.89	5	
40 - 45	208	1.08	.38	6.09	5	
45 - 50	53	1.17	.34	3.05	5	

Table 4.7 The sample sizes, and the observed means and standard deviations in seconds of the headways in speed ranges along with the χ^2 values of the fitted truncated Normal distribution. The data are from three rural sites.

Speed ft/sec.	Compton	Peasmarsh	Puttenham
25-30	.208	.512	.833
30-35	.285	.446	.807
35-40	.555	.489	.651
40-45	.333	.397	.656
45-50	.651	.437	.747
50-55	.523	.380	.671
55-60	.423	.250	.590
60-65	.469	.260	.393
65-70	.355	.208	.274
70-75	.440	.185	-

Table 4.8 Proportion of followers as a function of speed
for three rural sites.

Speed ft/sec	Compton	Peasmarsh	Puttenham
25-30	.871	.957	.942
30-35	.863	.900	.840
35-40	.722	.703	.738
40-45	.643	.696	.684
45-50	.568	.724	.718
50-55	.607	.696	.655
55-60	.607	.692	.623
60-65	.537	.663	.592
65-70	.560	.671	.595
70-75	.585	.670	-

Table 4.9 Ratio of the mean distance-headway (D_b) of vehicles with headways less than the Highway Code curve (D_{HC}) to that value, $R = D_b/D_{HC}$, as a function of speed. The value of D_{HC} is taken at the centre of the speed band. Results are given for three sites.

Site	Proportion of followers with Gap \leq H.C. Gap	Proportion of followers with Gap \leq $\frac{3}{4}$ H.C. Gap	Proportion of followers with Gap \leq $\frac{1}{2}$ H.C. Gap
Compton	.454	.390 (.859)*	.208 (.459)
Puttenham	.628	.456 (.724)	.203 (.323)
Peasmarsh	.376	.204 (.542)	.066 (.175)

Table 4.10 Proportion of followers which follow too closely in terms of Highway Code advice, for data from three road sites. H.C. is the Highway Code recommendation. ()* indicates the proportion of drivers out of the total number of drivers in platoons.

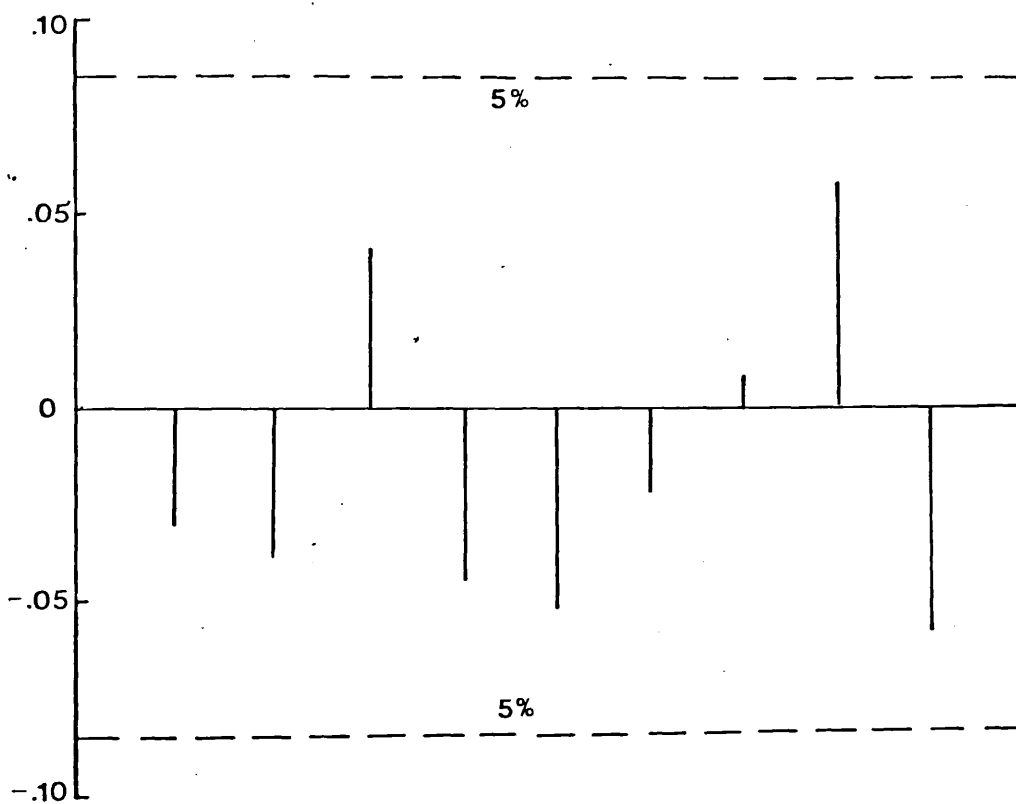


Figure 4.1 Autocorrelogram for time headways at Compton
(data-set 1).

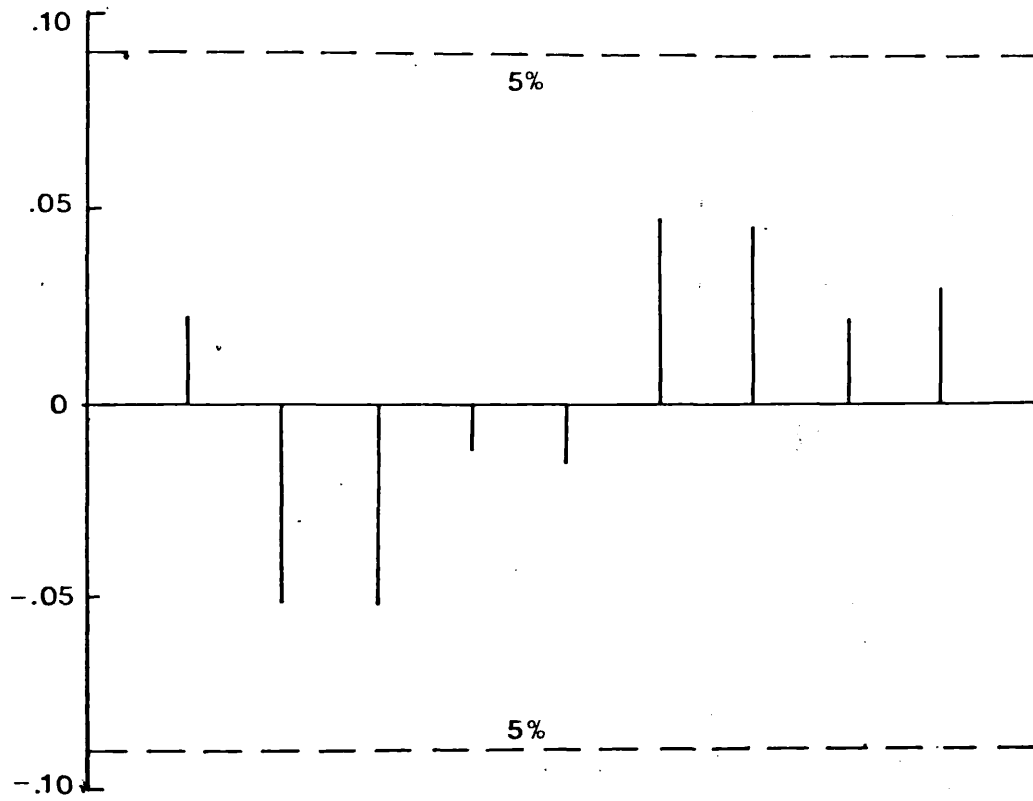


Figure 4.2 Autocorrelogram for time headways at Chobham.
(data-set 1).

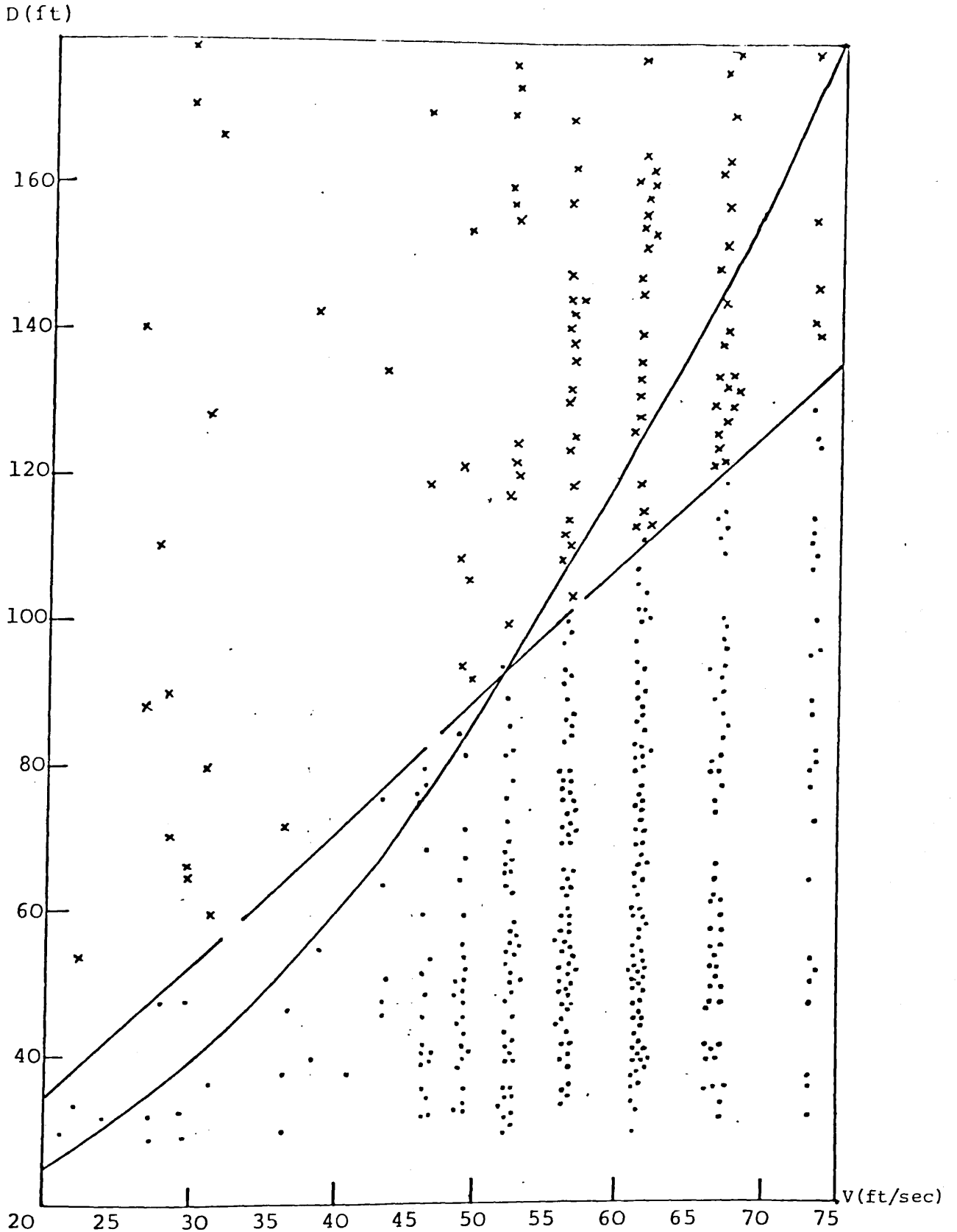


Figure 4.3 The observed distance gaps at Compton classified by speed bands (ft/sec) for all vehicles. Those vehicles below the dashed line are members of platoons ($T \leq 1.8$ secs). The curve is the Highway Code minimum stopping distance (D_{HC}). The data appear banded as the timing is uncertain to ± 0.02 seconds, thus, for example, any transit time between 0.07 and 0.11 seconds is recorded as 0.09 seconds. (x Free movers and Others . Followers) .

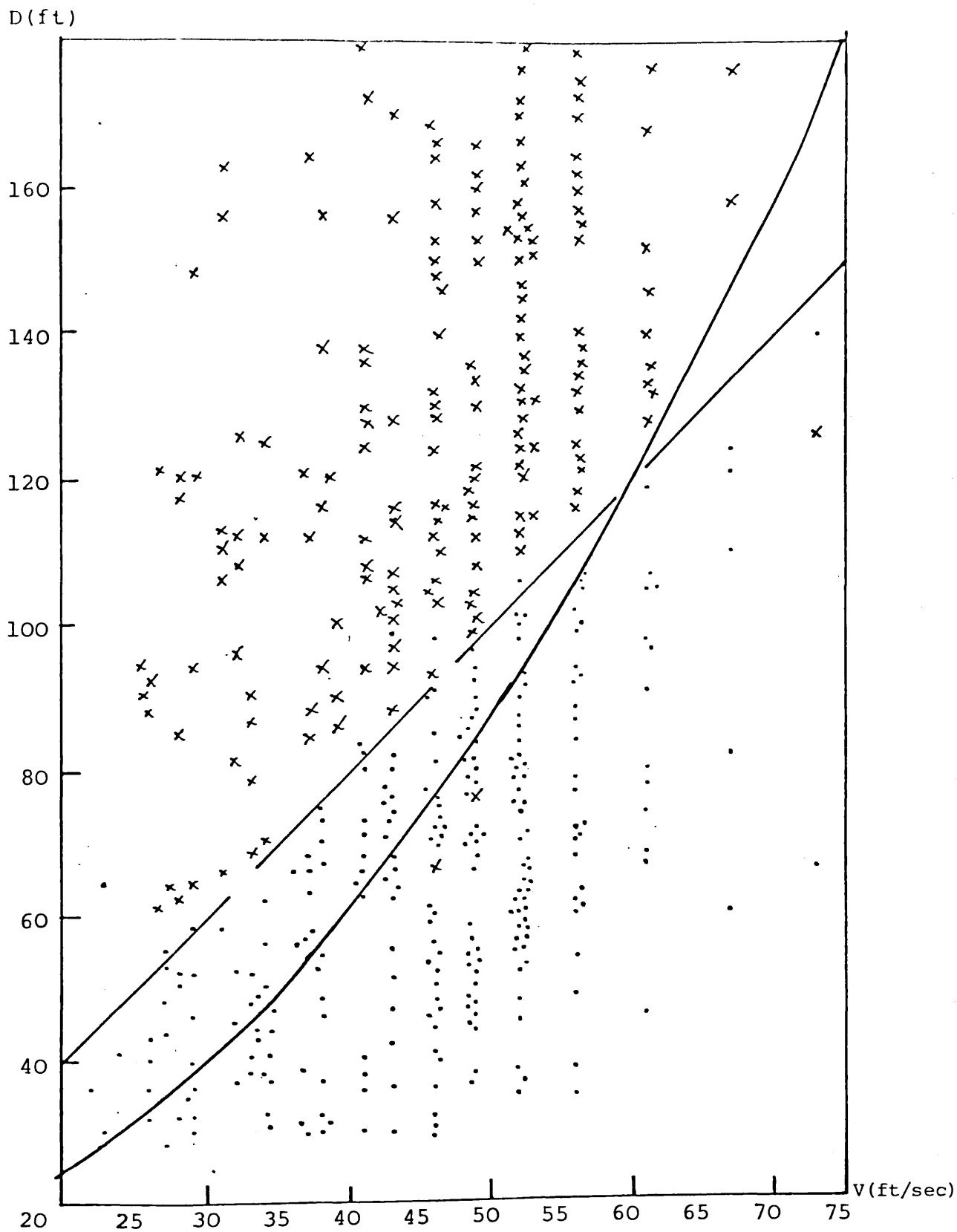


Figure 4.4 As figure 4.3, but for Peasmarsh with $T \leq 2.0$.

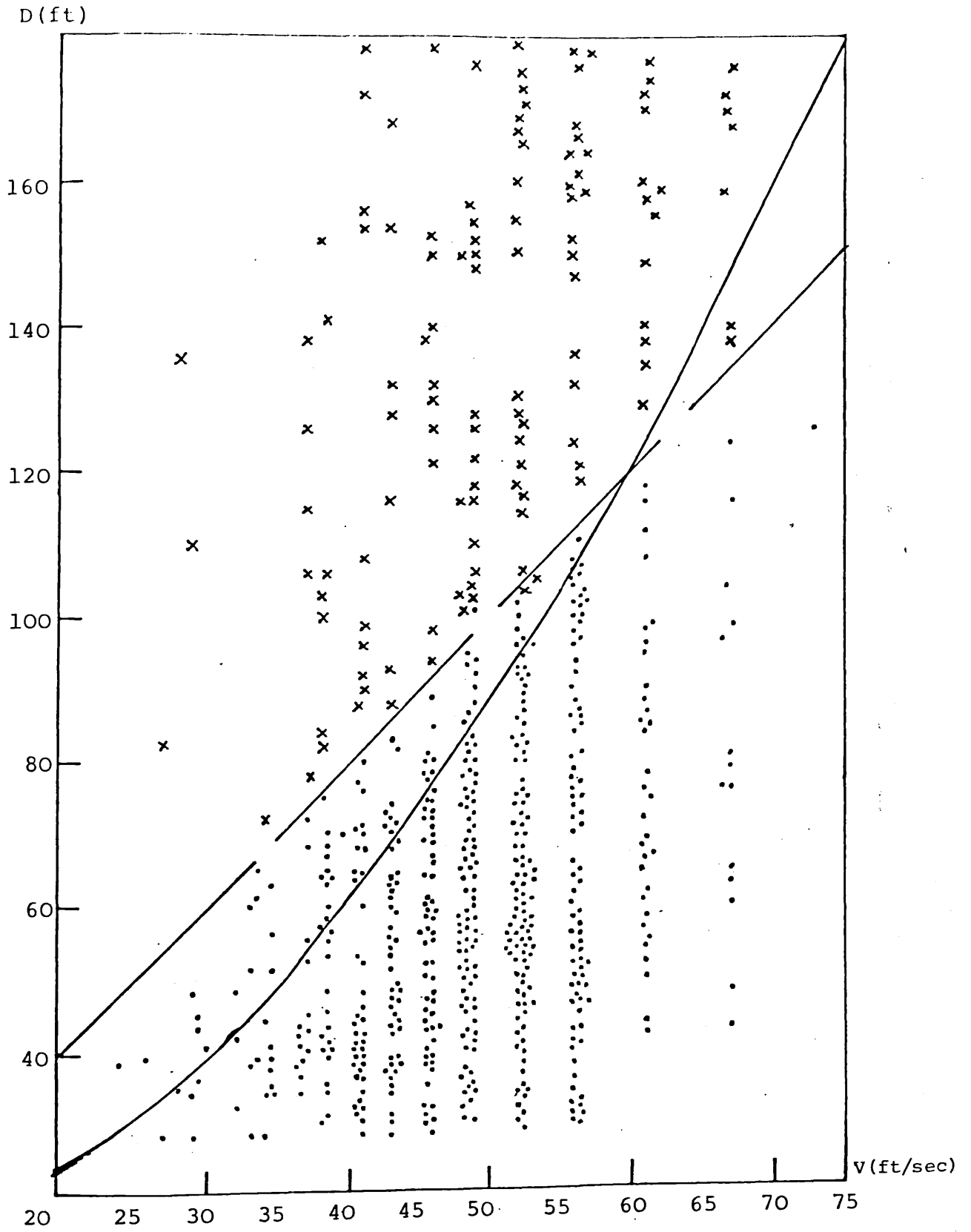


Figure 4.5 As figure 4.3, but for Puttenham with $T \leq 2.0$.

CHAPTER 5

I. DECELERATION BEHAVIOUR OF MAIN-ROAD STREAMS DRIVERS

APPROACHING A T-JUNCTION

5.1 Introduction

In the United Kingdom, more than half of the total number of injury accidents occur at or near intersections. Accident studies are usually based on accumulated accident statistics collected on a national scale. Furthermore, only injury accidents are routinely reported; they are rare events and the number occurring at a specific location is generally too small to be considered for analysis. Therefore, accidents themselves do not always offer reliable data for the measurement of intersection performance.

Recently, an alternative approach to the study of accident risk at intersections, based on traffic conflicts, has been introduced (Spicer (1972); Russam and Sabey (1972); Older and Spicer (1976); Malaterre and Muhlrad (1980); Spicer et al (1980)). A traffic conflict is defined as a situation in which two or more road users approach each other in time and space in such a way that a collision is imminent if their movements remain unchanged. Serious conflicts have been shown, [Spicer (1972)] to occur within intersections in the same manner as accidents, but conflicts may be as much as 2,000 times more frequent than accidents.

However, although the concept of traffic conflict was initially perceived as a possible measure of accident risk, some questions about its value as a risk measure still remain, especially when attempts are made to relate the frequencies of observed conflicts

to the number of injury accidents reported. Furthermore, it is not known if an action taken by a driver within an intersection is due to precaution rather than collision avoidance, to lack of precise control of his vehicle; or to some other behaviour. Any avoidance of conflict, or reduction in its severity by either or both the drivers involved necessarily involves a change in at least one velocity vector, that is to say an acceleration or deceleration; all such changes will be referred to as decelerations (negative or positive). It has been shown [Balasha et al (1980)] that one may neglect swerving to a good approximation. Thus all deceleration may be considered to be in the line of motion.

This chapter describes a technique of measuring the deceleration behaviour of drivers in major road streams approaching a T-junction. The objective is to understand and explore driver behaviour in terms of forced or unforced actions within and close to the T-junction, in cases where intersections of the vehicle paths may or do occur. Furthermore, the detected 'measures of deceleration' of the approaching vehicles, which are due to the action of turning vehicles, are intended to be used as input into a conflict simulation model (chapter 6).

5.2 Assumptions and Method

It is initially assumed that a vehicle in the major road stream passing through the T-junction will continue at constant speed, unless it is forced to change. This assumption will be tested below. Then its position can be calculated, at any time t , from its speed at a reference point P (fig, 5.1a), provided that P is not too far from the junction.

The typical manoeuvres at a T-junction are merging with or crossing a major road stream of traffic, or a combination of the two (figure 5.1b). In this chapter, only the crossing manoeuvre is considered; the merging manoeuvre appears to be more complicated and requires further assumptions and experimental effort.

Consider a crossing vehicle which has accepted a presented time gap in the major road stream; then if the time gap available appears to the oncoming major road driver to be less than the time the turning vehicle requires to clear his path, he must take some action to avoid collision. The actual action taken to avoid collision is not our main concern. Instead, from the speed at the point P, the time t_e at which the oncoming vehicle was expected to arrive at the collision point is estimated; the time t_a at which it actually arrived is also measured. The time difference $\tau = t_a - t_e$ does not indicate the actual deceleration (acceleration) of the approaching vehicle as this may occur over any time $\tau' \leq \tau$. However, the quantity τ is taken as a measure which describes a change in the speed vehicle over the potential collision distance. A negative value of the difference τ is defined as a 'measure of acceleration' and a positive value as a 'measure of deceleration'. As shorthand this difference τ will be referred to as deceleration. Naturally, not every measured deceleration will correspond to a traffic conflict since drivers may decelerate as a precaution, randomly, or for some other reason. Our measurements give a mean value of the 'measure of deceleration' over the distance from P to cable C (figure 5.1a): the actual deceleration might be larger but confined to part of the path, so a lower bound on

the maximum deceleration is actually measured in each case. The observed values of the 'measure of deceleration' are studied by classifying them into three groups according to whether

- (a) the junction was unoccupied, or
- (b) vehicles were waiting to cross, or
- (c) vehicles were actually crossing.

In cases (b) and (c) data were taken only if no vehicles were involved in other manoeuvres.

The experimental arrangements are described in the next section. This is followed by a section on the analysis and discussion of the data. Section 5.5 gives the conclusions.

5.3 Data Collection

The experiments were conducted at three priority controlled T-junctions, in daylight and under fine weather conditions, in Surrey. These were at Virginia Water A30/A329 and Wentworth A30/B389, both with speed limit 40 m.p.h., and at Sunningdale A30/A330, with speed limit 50 m.p.h.

The data collection system was based on the microprocessor unit described by Storr et al (1979). Observers, as well as automatic sensors (coaxial cables) laid at specific locations at or near the junction, were used (figure 5.1a).

For this series of experiments, the improved software used in the microprocessor was developed at Sheffield University; it gives the instrument a clock time accuracy of .001 seconds. This degree of accuracy was necessary to detect small values of the defined 'measure of deceleration' (see section 5.2). The speed at P was

calculated from the times of arrival at cables A and B. Cable C recorded the actual time of arrival of the major-road vehicles at the T-junction and cable D recorded the time at which a vehicle turning right into the minor road cleared the path of the oncoming vehicle. The observers were positioned on the verge by the junction and recorded the arrival and departure times of turning vehicles at the points X and Y (figure 5.1a). The inescapable observer error (typically ± 0.1 sec.) is not significant provided it is small compared with the travel time from P to cable C.

Every signal from observers' handsets or from cables was stored onto a cassette tape, which was played back into a mainframe computer. Several programs have been developed to process the data. After processing the following information was obtained:

- (i) the speed of the oncoming nearside vehicles at point P, calculated from the times at A and B (figure 5.1a) determined to ± 0.001 second;
- (ii) the actual time of arrival of the oncoming nearside vehicle at C, ignoring those vehicles which were turning left into the side road (to ± 0.001 second);
- (iii) the arrival and departure times of the turning vehicle at X (to about ± 0.1 second);
- (iv) the clearance of the rear of the turning vehicle at D (to ± 0.001 second); and
- (v) the arrival and departure times of the merging vehicles at Y (to about ± 0.1 second).

5.4 Analysis of Data and Discussion

In this study, the objective was to examine the behaviour of major road stream drivers approaching a T-junction in terms of a time difference, $\tau = t_a - t_e$, by assuming τ to indicate a change in the speed of the approaching vehicle. The decelerations of those oncoming vehicles which turned left into the side road together with following vehicles which may have been affected were not considered in the study.

The mean and standard deviations of the speeds (m.p.h.) of major road vehicles and the flows veh/hr. are given in tables 5.1 and 5.2 respectively, for the three sites.

The highest mean speed was observed at Sunningdale and the lowest at Virginia Water (table 5.1). The flows were significantly different amongst the sites: the major road flow at Virginia Water was almost twice that at Wentworth, whilst the crossing flow/hour at Wentworth was twice that at Virginia Water. The major road flow at Sunningdale was between the flows of the other two sites, whilst the crossing flow was much smaller than at the other two sites (table 5.2).

The decelerations were classified into three groups (a), (b) and (c), as defined previously (section 5.2). The frequency distributions of decelerations were conveniently classified into .08 second intervals and are shown in tables 5.3.

The substantial changes in behaviour between the three cases are obvious at a glance, and confirmed by statistical analysis. Table 5.4 gives the medians and table 5.5 the mean and standard deviations for

cases (a), (b) and (c) at the three sites. Application of the median test showed a highly significant difference ($p < .005$) between the medians of the distributions for each case (a), (b) and (c) at each site.

The Kolmogorov-Smirnov two sample nonparametric test was also applied to test the difference between the cumulative distribution of decelerations corresponding to (a), (b) and (c). It was found that the distribution of decelerations for cases (b) and (c) were stochastically larger* than the distribution of decelerations for case (a) ($p < .001$) at each of the three sites. The same difference was found between cases (b) and (c) ($p < .01$). Figures 5.2, 5.3 and 5.4 present graphically the cumulative standardised frequency distribution for each site.

Figures 5.5, 5.6 and 5.7 show plots of the measure of deceleration versus speed when the junction was unoccupied, case (a). Although the majority of the positive decelerations are due to the drivers with higher speeds, the scatter of these decelerations are such that no firm conclusion can be drawn. In general, the distribution of deceleration cannot be explained as a function of speed only. Therefore, a necessary assumption is that this random appearance may be also due to the inability of drivers to maintain a constant speed and the assumption should also be made in the other two cases (b) and (c).

Generally the results indicate that drivers in the major road stream tended to take precautionary action when another vehicle is occupying the junction, and reacted most strongly when a vehicle was

* A variable x is stochastically larger than a variable y if the c.d.f.s. $F(x)$ and $G(y)$ satisfy $F(a) < G(a)$ for every a .

actually turning and the possibility of a conflict was obvious. In about 1% of the events of class (c) the deceleration was very severe (greater than 0.5 seconds change in arrival time).

Despite the difference amongst the flows and the mean speeds in the major road at the three sites (see above), it appears from the results that the behaviour of major road drivers approaching a T-junction was very similar at all three sites. It seems that, judging from the difference between the frequency distributions, the behaviour of approaching major road drivers depended on whether or not the junction was occupied by vehicles waiting to cross, or actually crossing. The ratios of the number of drivers who showed a positive change in the time difference, $\tau = t_a - t_e$, over those who showed a negative one are given in table 5.6 for each case separately. In the case where the junction was unoccupied, the distribution of decelerations was located around zero. In case (b), where a vehicle was present waiting to turn, more than two-thirds of the drivers decelerated; only 22% to 27% of the drivers did not decelerate. The observed accelerations in this case may be due to the major road drivers who were very close to the junction at the moment a turning vehicle arrived and one might assume that they preferred to accelerate and clear the junction, although the results are possibly due, in some cases, to the random effect noted for case (a). In the case where a vehicle was turning, 94% of the major road drivers decelerated and appeared to have a precautionary attitude.

The results for case (a) show that the observed distributions of decelerations in the other two cases (b) and (c), may not be due to the

turning vehicles actions alone, since the inconsistency observed in case (a) may also be present. However, if that background is eliminated, the remaining deceleration can be assigned to precautionary actions. Such actions will be referred to as "precautionary conflicts".

Let $f(t)$ be the probability density (p.d.f.) of the vehicle deceleration when the junction is unoccupied, case (a), and $g_k(t)$ ($k = 1,2$) the p.d.f. when the junction is occupied, cases (b) and (c) respectively. It is assumed that $g_k(t)$ is a distribution of the sum of deceleration "noise" represented by $f(t)$ and deceleration which was due to either vehicles waiting to turn (case (b)), or to the ones actually turning (case (c)). If the unknown p.d.f. of this latter deceleration is represented by $u_k(t)$ ($k = 1,2$), then the distribution $g_k(t)$ is the convolution of the distributions $f(t)$ and $u_k(t)$. viz.

$$g_k(t) = \int_{-\infty}^{\infty} f(r) u_k(t-r) dr, \quad K = 1,2. \quad (5.1)$$

For case (a), a Normal distribution has been fitted to the available data using a standard minimisation routine, Powell (1964). A χ^2 -test was used to test the quality of the fit. Table 5.7 (case (a)) gives the results for the three sites. It is seen that the observed data can be reasonably well described by a Normal distribution, although some outliers were identified at all three sites.

It is also assume that* :

- (i) the unknown p.d.f. $u_1(t)$ for case (b) is a Normal distribution with mean m_1 , and standard deviation σ_1 ;

* other possible distributions have been examined and rejected.

- (ii) the unknown p.d.f. $u_2(t)$ for case (c) is a truncated Normal distribution with mean m_2 , and standard deviation σ_2 .

After the calculations, which are given in the Appendix (5.1) becomes

$$g_k(t) = v_k(t) \int_{T_1}^{T_2} h_k(r) dr \quad (5.2)$$

or
$$g_k(t) = v_k(t)[\Phi(\beta) - \Phi(\alpha)]$$

where the $g_k(t)$ is a function of the parameters $m, \sigma; m_k, \sigma_k; k=1,2$. m and σ being the mean and standard deviation of $f(t)$, which were estimated by the minimisation routine for the case (a).

A computer program was written, incorporating a standard minimisation routine [Powell (1964)] for obtaining the parameters m_i, σ_i of the distribution $u_k(t)$, the optimisation procedure being as follows. From initial estimates based on maximum likelihood values, the minimisation routine alters the values of the parameters of the distribution in order to converge to an optimum. The value of the χ^2 statistic, given by

$$\chi_k^2 = \frac{n}{\sum_{i=1}^n} \frac{(g_{ok}(t_i) - g_{tk}(t_i))^2}{g_{tk}(t_i)},$$

where g_{ok} is the observed frequency and g_{tk} is the theoretical frequency, was used in the procedure. The upper tail (p) value of the χ^2 -distribution was then calculated. The estimated parameters, together with the values of χ^2 and p , are given in table 5.7.

Values of $p < 0.5$ would be indicators of a significant

difference; by implication, in the current situation, values larger than 0.5 indicate a tendency towards a 'good fit'.

However, as it can be seen from table 5.7, the highest values of χ^2 and consequently the smallest values of p , are associated with the larger samples (case (a)). This is due to the fact that large samples usually produce large χ^2 values, even though a curve may give a good visual fit. In such cases, differences would not be regarded as significant unless $p < .01$.

The parameters of the fitted Normal distribution (case (a)) are very close to the observed ones. The deconvoluted values of the mean values (cases (b) and (c)) are in all cases smaller than the observed ones, as is expected; the standard deviations are comparable. The parameters define the distribution function of decelerations due to the turning drivers, namely the precautionary effect.

5.5 Conclusions

The results have shown that the behaviour of drivers in major road streams approaching a T-junction is affected, in terms of a time difference between actual and expected times of arrival, by the junction

- (1) having vehicles waiting to cross (case (b)), and
- (2) having vehicles actually crossing (case (c)).

This time difference does not directly measure the deceleration but, as will be shown (Part II of this chapter) it is independent of speed, and can be considered as a surrogate measure of deceleration.

It was also found that the major road drivers decelerated more in case (c) than in case (b). However, small decelerations were observed even when the T-junction was unoccupied; the majority of drivers with significant decelerations in these cases were found to have a high speed.

A normal distribution describes the observed data in case (a), although some outliers were observed. In the other two cases, (b) and (c), the real effect due to the crossing vehicles, that is the observed decelerations after eliminating the background (noise), were described by normal and truncated normal distributions respectively.

When the data from the three experimental sites were compared, it was found that there were no significant differences except in case (c), where drivers seemed to decelerate more at Virginia Water than at either of the other two sites, when vehicles were actually crossing. Since Virginia Water is the site with the lowest speed amongst the three, the difference in case (c) might be due to traffic volume, to layout design, or to other factors related to the turning drivers decisions. In about 1% of the events in case (c) the decelerations was severe, greater than 0.5 seconds change in arrival time.

In the next chapter, the distributions of the observed measures of deceleration (cases (b) and (c) after the background of case (a) has been eliminated) will be used as input in a conflict simulation model, to obtain a measure of risk of conflict.

APPENDIX

Derivation of formulae (5.2)

It is assumed that the observed measure of deceleration for cases (b) and (c) is the convolution of two independent variables. One represents the random effect (cases (a)) with p.d.f. $f(t)$ and the other the effect due to the turning vehicle with p.d.f. $u_i(t)$ ($i = 1,2$) for cases (b) and (c) respectively.

Then the assumed model representing the observed measure of deceleration is defined by

$$g_i(t) = \int_{-\infty}^{\infty} f(r) u_i(t-r) dr.$$

It was shown that $f(t)$ is a normal distribution, $N(m, \sigma^2)$. Let the unknown distribution for case (b) $u_1(t)$ be also normal, $N(m_1, \sigma_1^2)$ (a truncated normal for case (c)).

Then

$$g_1(t) = \frac{1}{2\pi\sigma\sigma_1} \int_{-\infty}^{\infty} \exp[-\frac{1}{2\sigma^2}(r-m)^2] \exp[-\frac{1}{2\sigma_1^2}((t-r)-m_1)^2] dr$$

or

$$g_1(t) = \frac{1}{2\pi\sigma\sigma_1} \int_{-\infty}^{\infty} \exp[-\frac{1}{2} (\frac{1}{\sigma^2}(r-m)^2 + \frac{1}{\sigma_1^2}(r-(t-m_1))^2)] dr.$$

Let $\frac{1}{\sigma^2} = k$, $\frac{1}{\sigma_1^2} = \lambda$, $k, \lambda > 0$, and $t - m_1 = \ell$.

Then

$$g_1(t) = \frac{1}{2\pi\sigma\sigma_1} \int_{-\infty}^{\infty} \exp(-\frac{1}{2} A) dr, \tag{A.1}$$

where $A = k(r-m)^2 + \lambda(r-\ell)^2$.

Then $A = k(r^2 + m^2 - 2rm) + \lambda(r^2 + \ell^2 - 2r\ell)$

$$\begin{aligned}
 &= (k + \lambda)r^2 + km^2 + \lambda\ell^2 - 2r(km + \lambda\ell) \\
 &= (k + \lambda)r^2 + \frac{(k + \lambda)(km^2 + \lambda\ell^2)}{k + \lambda} - 2r(km + \lambda\ell) \\
 &= (k + \lambda)r^2 + \frac{k^2m^2 + \lambda^2\ell^2}{k + \lambda} - 2r(km + \lambda\ell) + \frac{\lambda k(m^2 + \ell^2)}{k + \lambda} \\
 &= (k + \lambda)r^2 + \frac{k^2m^2 + \lambda^2\ell^2 + 2k\lambda\ell m}{k + \lambda} - 2r(km + \lambda\ell) + \frac{\lambda k(m^2 + \ell^2 - 2\ell m)}{k + \lambda} \\
 &= (k + \lambda)r^2 + \frac{(km + \lambda\ell)^2}{k + \lambda} - 2r(km + \lambda\ell) + \frac{\lambda k}{k + \lambda} (\ell - m)^2,
 \end{aligned}$$

$$\text{or } A = (k + \lambda)(r - \mu)^2 + \frac{\lambda k}{k + \lambda} (\ell - m)^2, \quad (\text{A.2})$$

$$\text{where } \mu = \frac{km + \lambda\ell}{k + \lambda}.$$

By substituting (A.2) into (A.1), $g_1(t)$ can be written as follows

$$\begin{aligned}
 g_1(t) &= \frac{(\lambda k)^{\frac{1}{2}}}{2\pi} \int \exp\left[-\frac{1}{2}(k + \lambda)(r - \mu)^2 + \frac{\lambda k}{k + \lambda} (\ell - m)^2\right] dr \\
 &= \frac{(\lambda k)^{\frac{1}{2}}}{2\pi} \exp\left(-\frac{1}{2} \left(\frac{\lambda k}{k + \lambda}\right) (\ell - m)^2\right) \int \exp\left(-\frac{1}{2}(k + \lambda)(r - \mu)^2\right) dr \\
 &= \frac{(\lambda k)^{\frac{1}{2}}}{\sqrt{2\pi}(k + \lambda)^{\frac{1}{2}}} \exp\left(-\frac{1}{2} \left(\frac{\lambda k}{k + \lambda}\right) (\ell - m)^2\right) \frac{(k + \lambda)^{\frac{1}{2}}}{\sqrt{2\pi}} \int \exp\left(-\frac{1}{2}(k + \lambda)(r - \mu)^2\right) dr.
 \end{aligned}$$

However,

$$v(t) = \frac{1}{\sqrt{2\pi}} \left(\frac{\lambda k}{k + \lambda}\right)^{\frac{1}{2}} \exp\left(-\frac{1}{2} \left(\frac{\lambda k}{k + \lambda}\right) (\ell - m)^2\right) \sim N(m + m_1, \frac{\lambda + k}{\lambda k}),$$

$$h(r) = \frac{(k + \lambda)^{\frac{1}{2}}}{\sqrt{2\pi}} \exp\left(-\frac{1}{2}(k + \lambda)(r - \mu)^2\right) \sim N\left(\mu, \frac{1}{\lambda + k}\right),$$

and so (A.1) can be written

$$g_1(t) = v(t) \int h(r) dr ,$$

where $v(t) \sim N(m + m_1, \sigma^2 + \sigma_1^2)$

$$h(r) \sim N \left(\frac{m\sigma_1^2 + (t-m_1)\sigma^2}{\sigma^2 + \sigma_1^2} , \frac{\sigma^2\sigma_1^2}{\sigma^2 + \sigma_1^2} \right) .$$

Since $T_1 \leq t \leq T_2$, the final form of $g_1(t)$ becomes

$$g_1(t) = v(t) \int_{T_1}^{T_2} h(r) dr$$

or

$$g_1(t) = v(t) (\Phi(\beta) - \Phi(\alpha)) ,$$

where $\Phi(\cdot)$ represents the normal distribution $N(0,1)$,

and

$$\beta = \frac{T_2(\sigma^2 + \sigma_1^2) - ((t-m_1)\sigma^2 + m\sigma_1^2)}{\sigma\sigma_1(\sigma^2 + \sigma_1^2)^{\frac{1}{2}}} ,$$

$$\alpha = \frac{T_1(\sigma^2 + \sigma_1^2) - ((t-m_1)\sigma^2 + m\sigma_1^2)}{\sigma\sigma_1(\sigma^2 + \sigma_1^2)^{\frac{1}{2}}} ,$$

$$0 < \Phi(\beta) - \Phi(\alpha) < 1 .$$

<u>Site</u>	Speed (m.p.h.)	
	<u>Mean</u>	<u>s.d.</u>
Sunningdale	46.0	9.0
Wentworth	43.7	9.3
Virginia Water	39.7	8.7

Table 5.1 The parameters (mean, standard deviation) of speed distribution in m.p.h.

<u>Site</u>	Flow (veh/hr)	
	<u>Main Road</u>	<u>Crossing</u>
Sunningdale	400	85
Wentworth	350	300
Virginia Water	650	150

Table 5.2 Flows at the experimental sites in veh/hour.

T	Sunningdale			Wentworth			Virginia Water		
	(a)	(b)	(c)	(a)	(b)	(c)	(a)	(b)	(c)
-.44							7		
-.36	3			8			10		
-.28	11			14			15		
-.20	37	2		32	6		26	8	
-.12	62	15		64	27		61	20	
-.04	113	23	4	158	86	7	110	40	5
.04	132	39	12	208	143	37	159	53	10
.12	106	29	15	160	152	50	111	50	23
.20	46	21	9	83	72	21	53	40	17
.28	9	12	6	28	37	12	23	19	15
.36	2	8	5	6	16	3	13	13	6
.44		3	3		5	2		6	5
.52			1			1		3	3
.60									2
.68									1

Table 5.3 Frequency distributions of decelerations classified in .08 second intervals, for all three sites. The numbers represent the midpoint of the interval.

<u>Site</u>	<u>Medians</u>		
	(a)	(b)	(c)
Sunningdale	.026	.075	.140
Wentworth	.037	.085	.113
Virginia Water	.038	.088	.186

Table 5.4 The medians, in seconds, of the observed distributions of decelerations, at all three sites.

<u>Site</u>	<u>Mean</u>			<u>s.d.</u>		
	(a)	(b)	(c)	(a)	(b)	(c)
Sunningdale	.025	.090	.130	.125	.135	.130
Wentworth	.032	.095	.125	.120	.120	.110
Virginia Water	.035	.120	.210	.140	.142	.145

Table 5.5 The means and standard deviations, in seconds, of the observed distributions of decelerations at all three sites.

Site	Sample Size			Ratio +/-		
	(a)	(b)	(c)	(a)	(b)	(c)
Sunningdale	521	152	55	1.3	2.8	12.2
Wentworth	766	544	133	1.7	3.6	18.0
Virginia Water	588	252	88	1.5	2.7	16.6

Table 5.6 The sample sizes and the ratios of the number of observed decelerations over the number of observed accelerations regardless of magnitude.

	Sunningdale			Wentworth			Virginia Water		
	(a)	(b)	(c)	(a)	(b)	(c)	(a)	(b)	(c)
Mean	.020	.060	.100	.033	.065	.095	.034	.070	.170
s.d.	.125	.110	.120	.120	.105	.100	.120	.115	.130
χ^2	8.9	4.9	4.7	13.6	14.8	6.8	15.4	9.8	7.3
p	.180	.580	.330	.090	.075	.450	.050	.195	.390

Table 5.7 Parameters of the fitted normal ((a) and (b)) and truncated normal (c) deceleration distributions in seconds, after eliminating the background effect in cases (b) and (c).

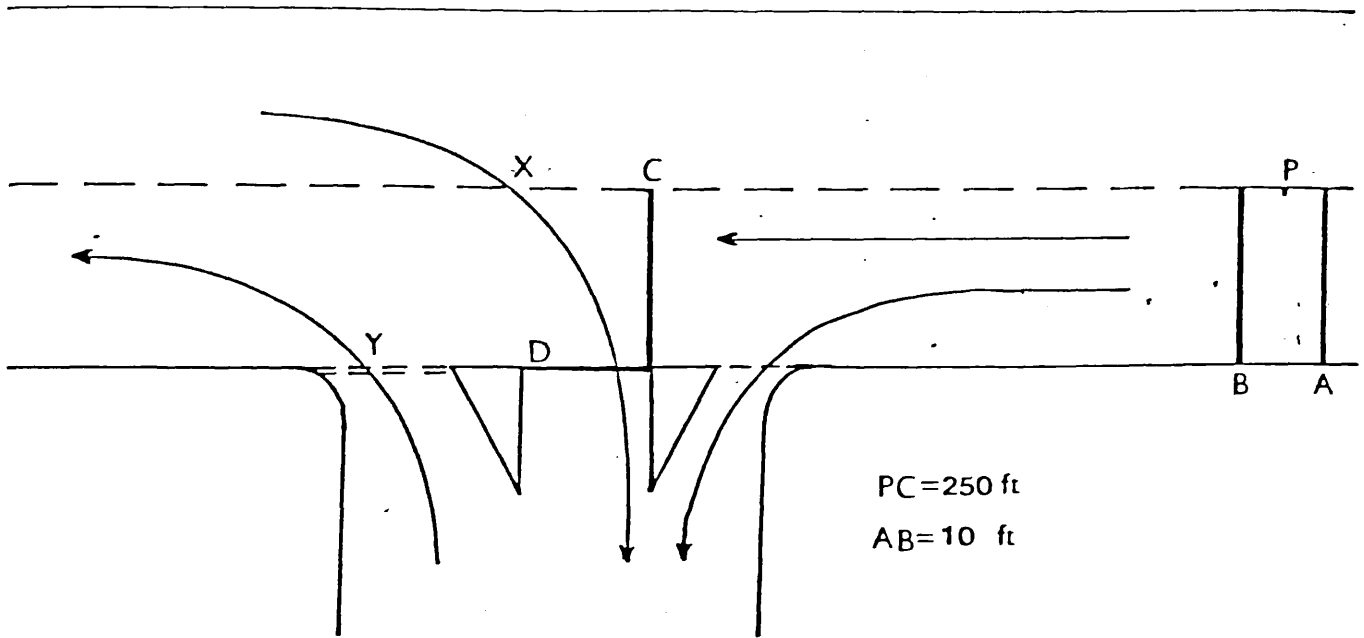


Figure 5.1a Diagram, not to scale, of the priority controlled T-junction and the observed manoeuvres; A,B,C,D indicate the cables.

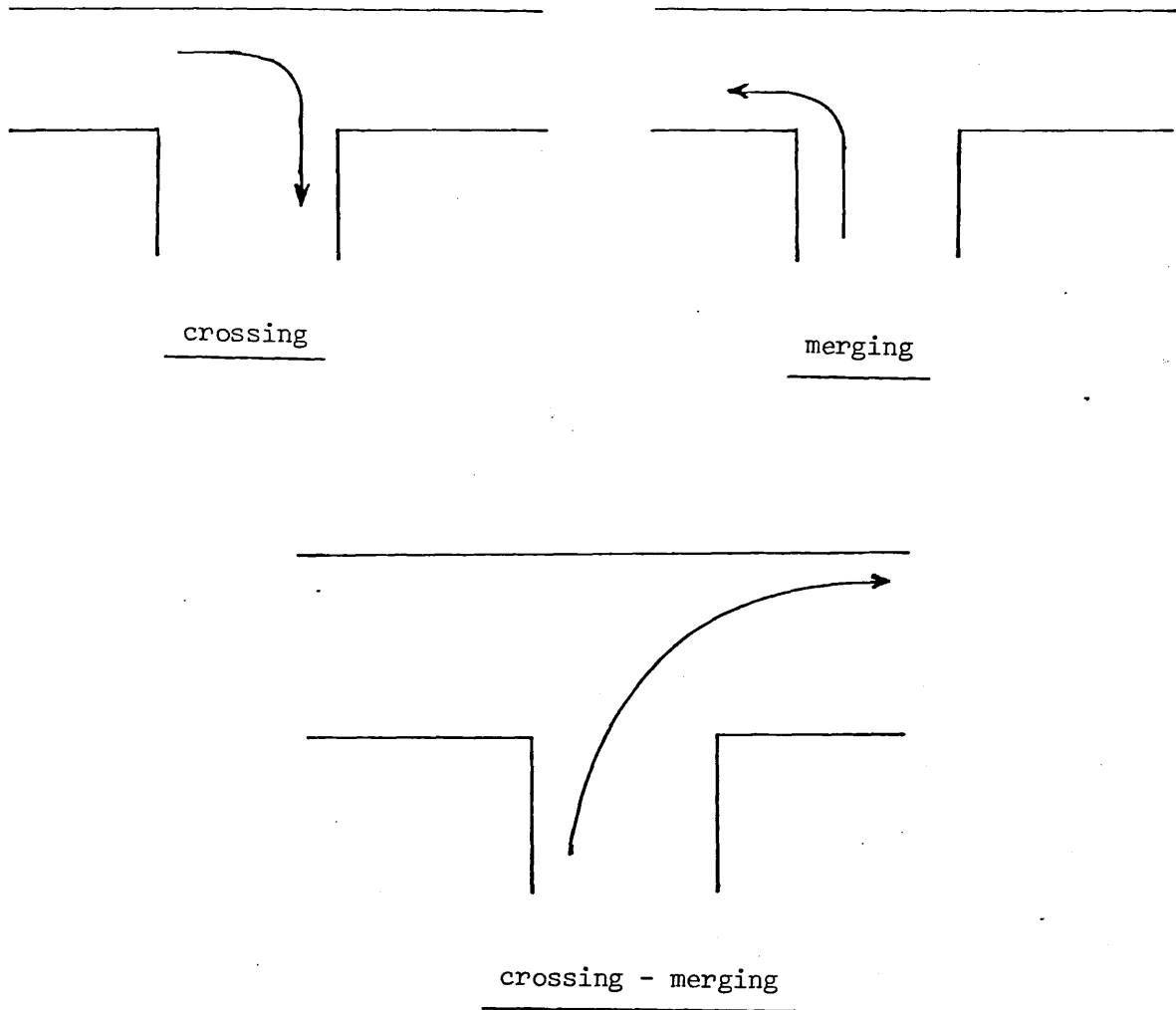


Figure 5.1b Diagram of each manoeuvre at a priority controlled T-junction .

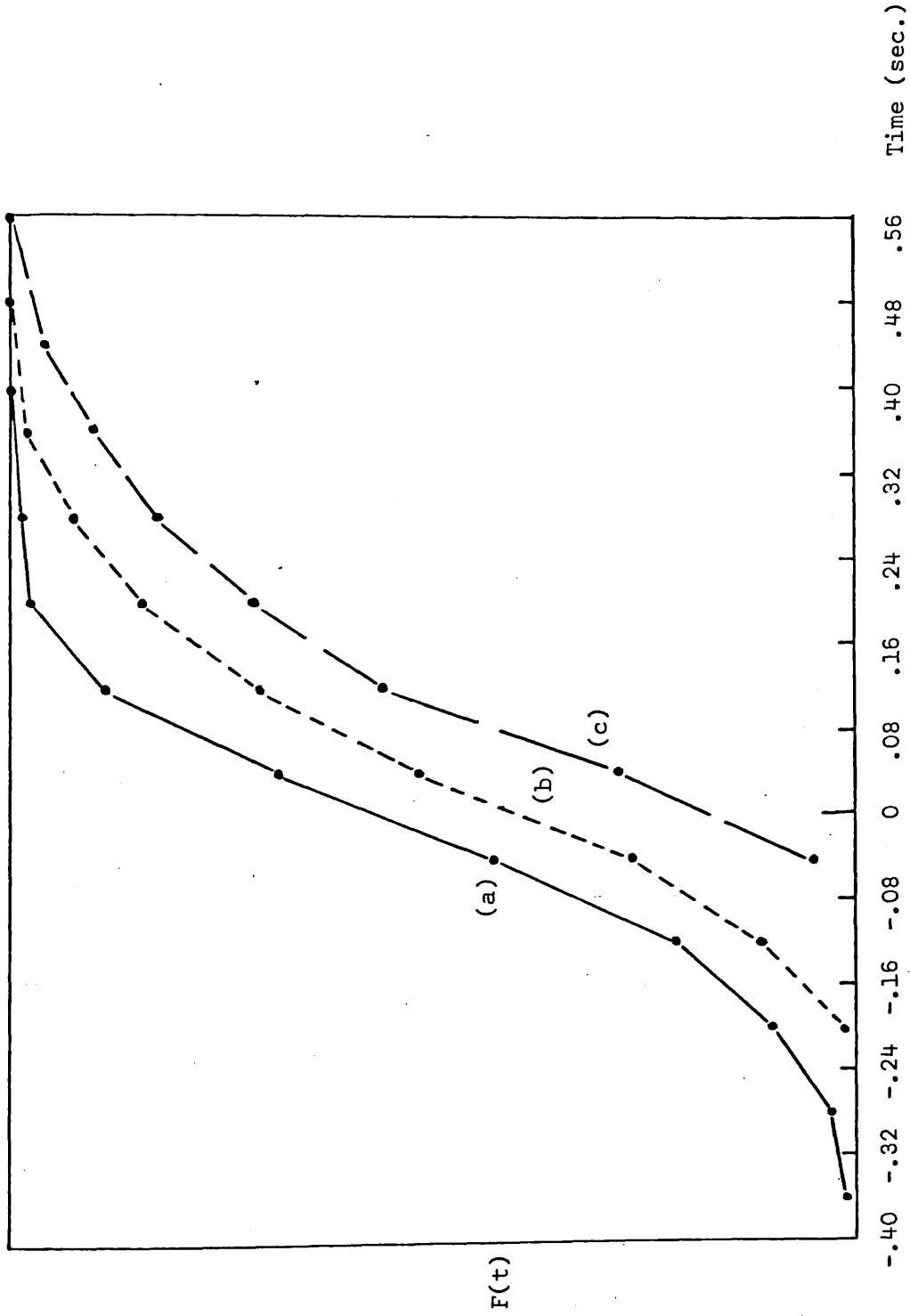


Figure 5.2 Cumulative standardised frequency distributions of decelerations. as a function of time $F(t)$, for each case separately, at Sunningdale.

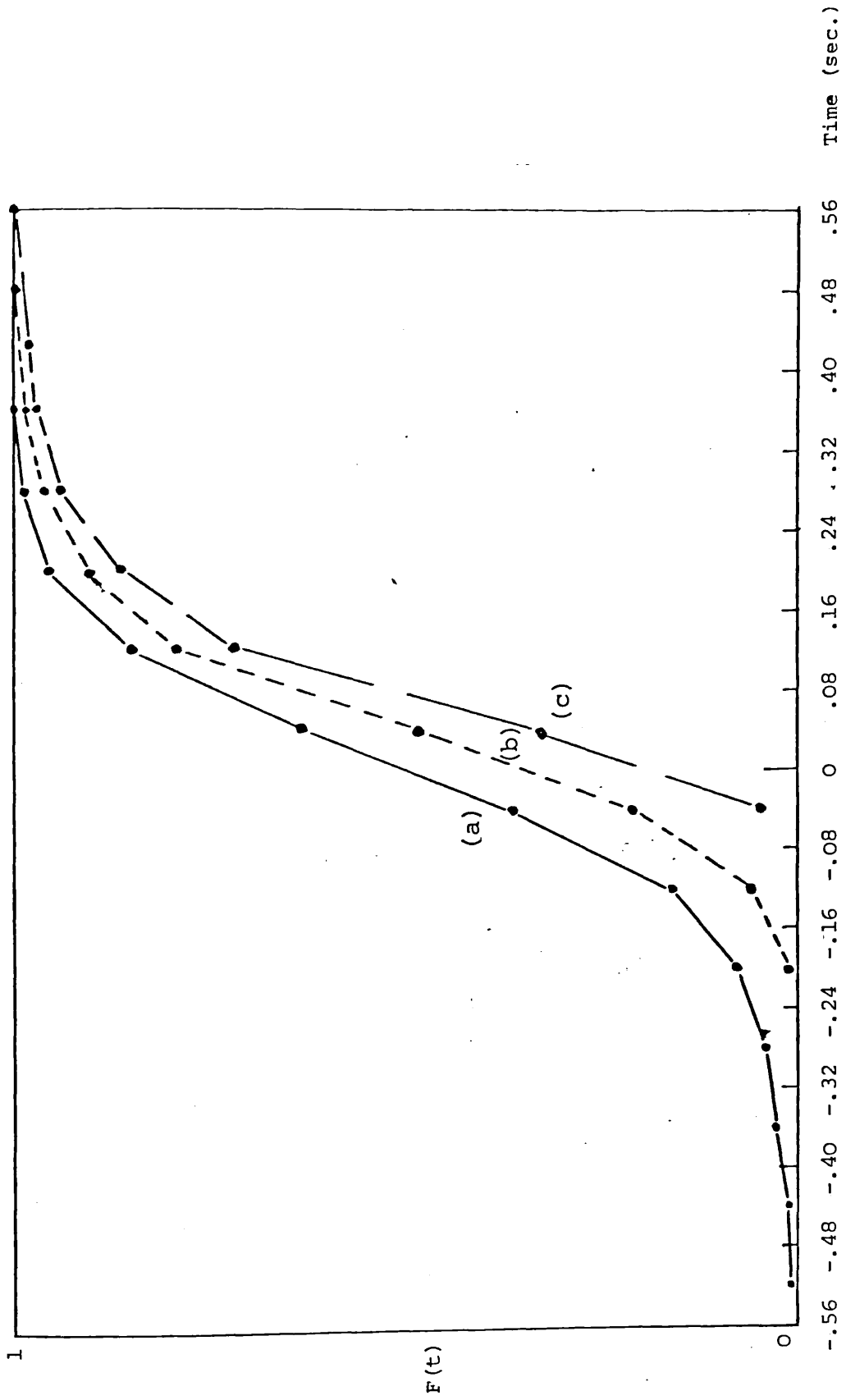


Figure 5.3 As Fig. 5.2, but for Wentworth

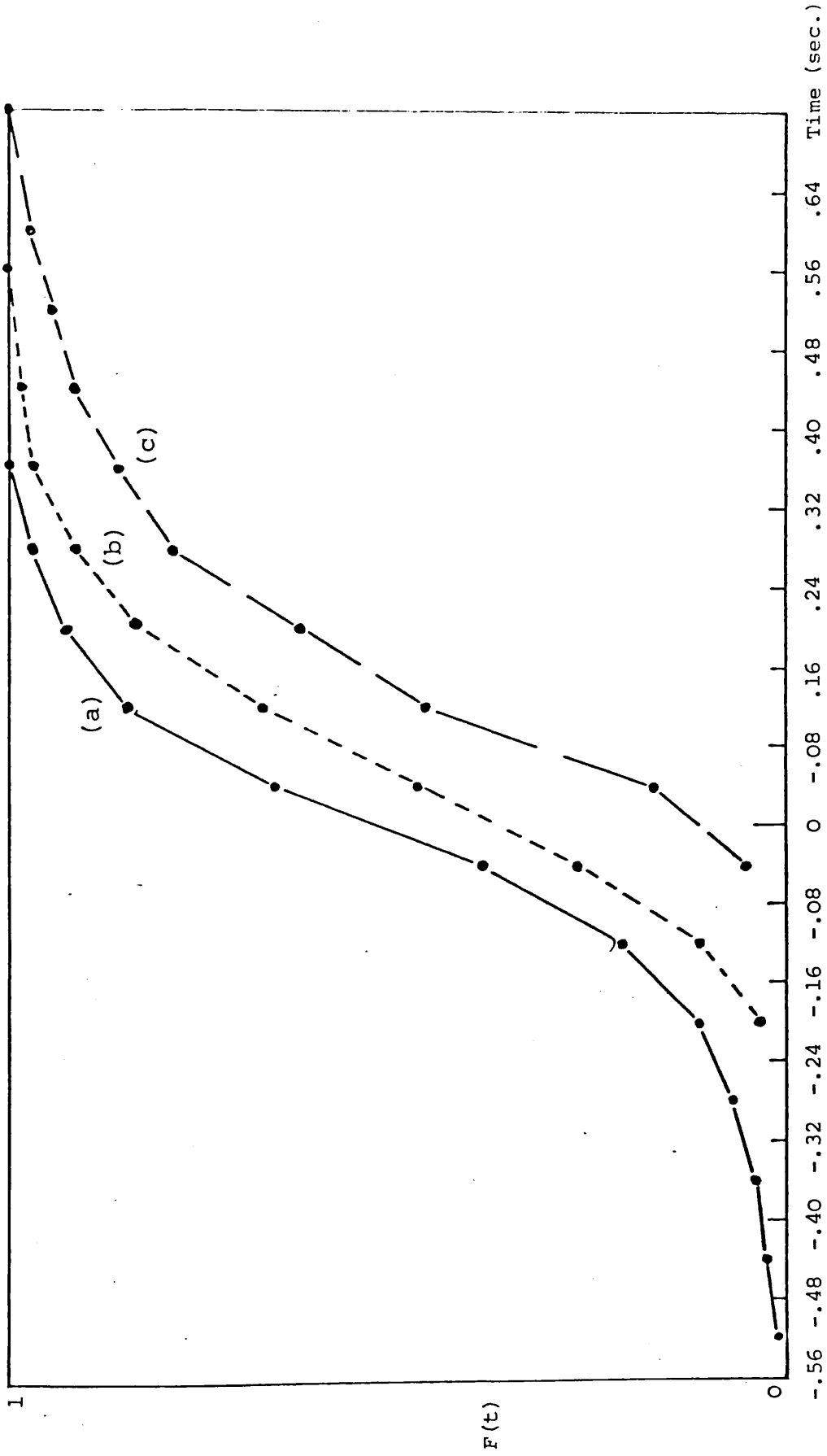


Figure 5.4 As Fig. 5.2, but for Virginia Water.

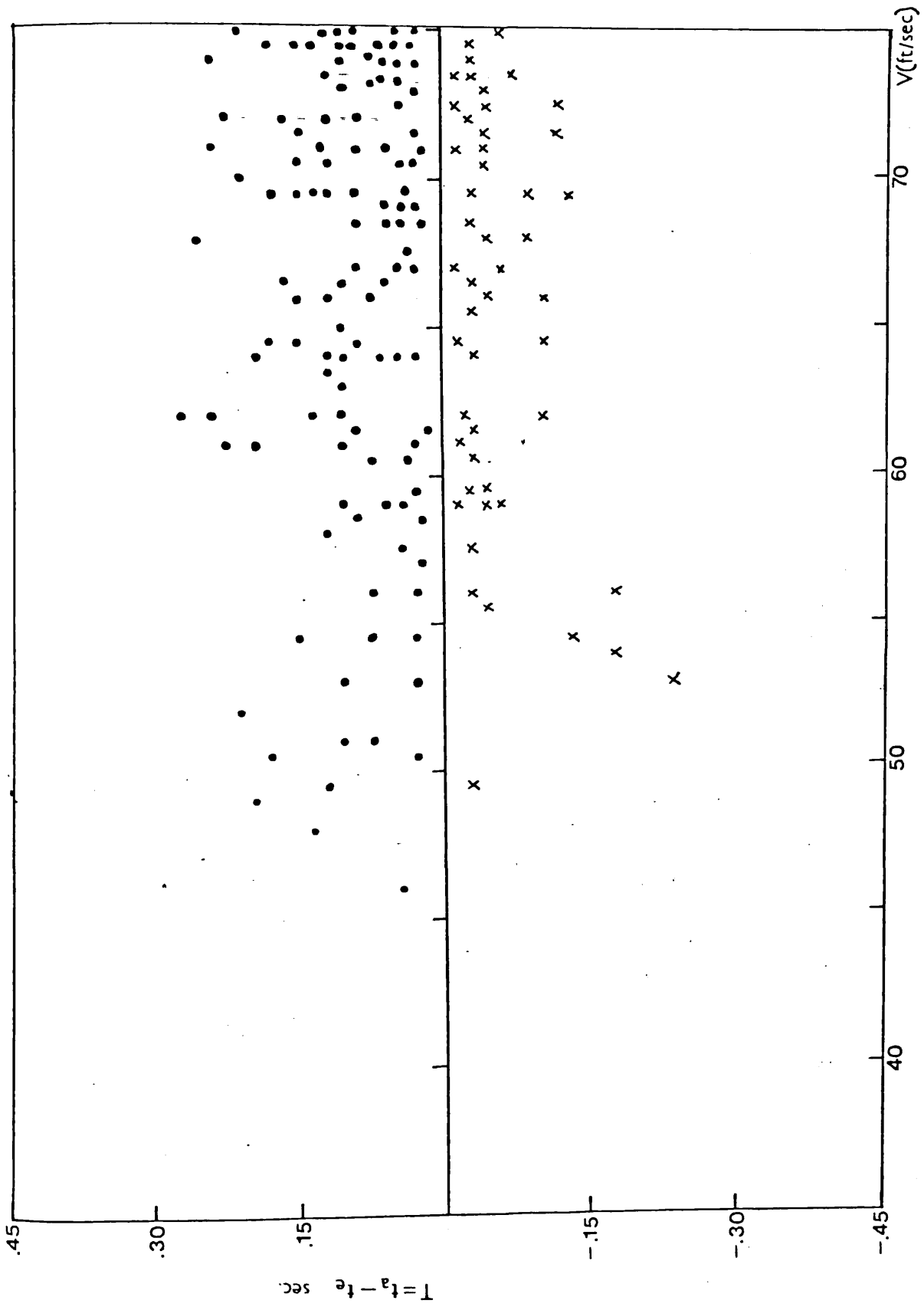


Figure 5.5 The observed decelerations in seconds versus speed in ft/sec., for case (a), at Sunningdale.

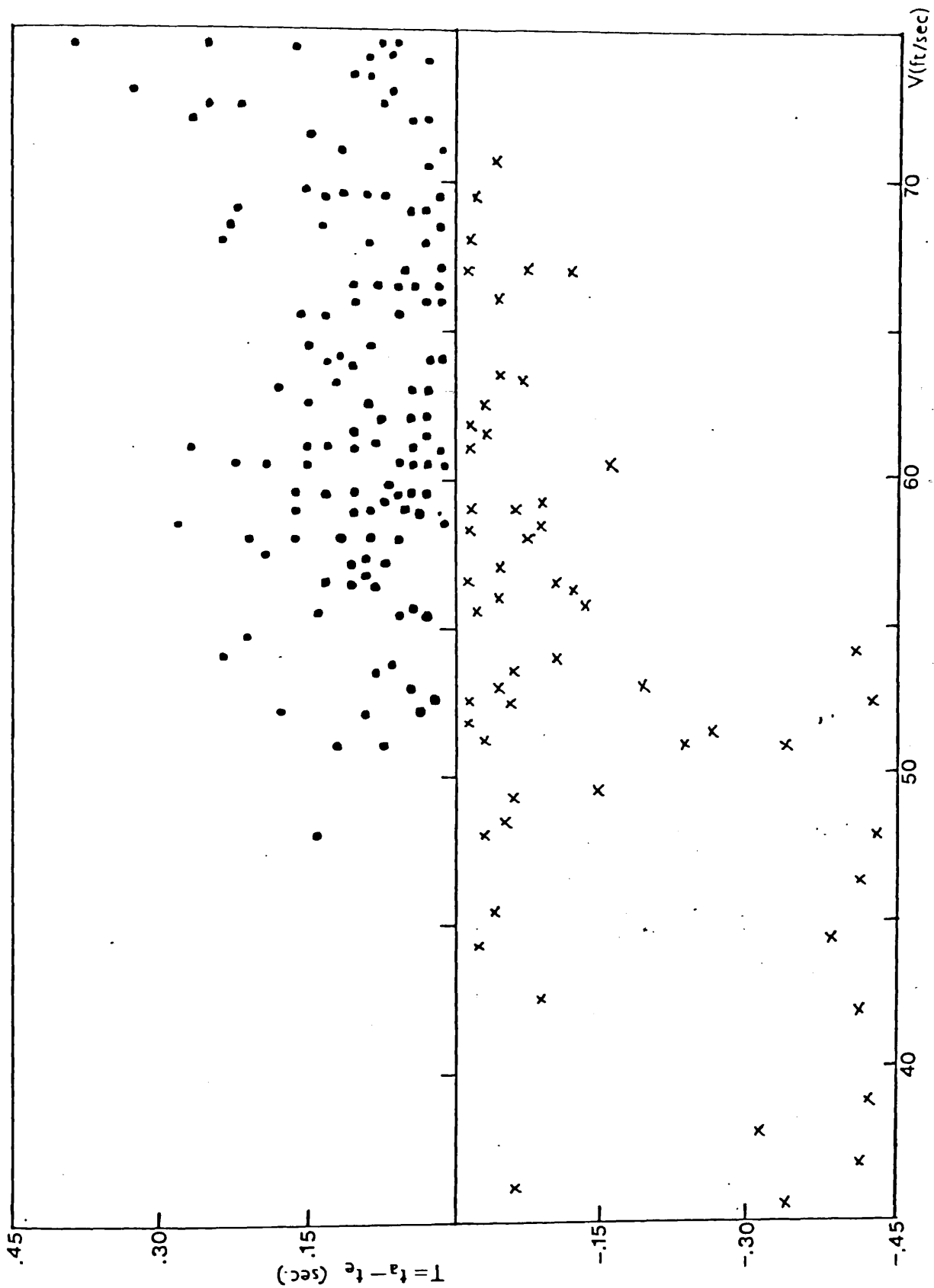


Figure 5.6 As figure 5.5, but for Wentworth.

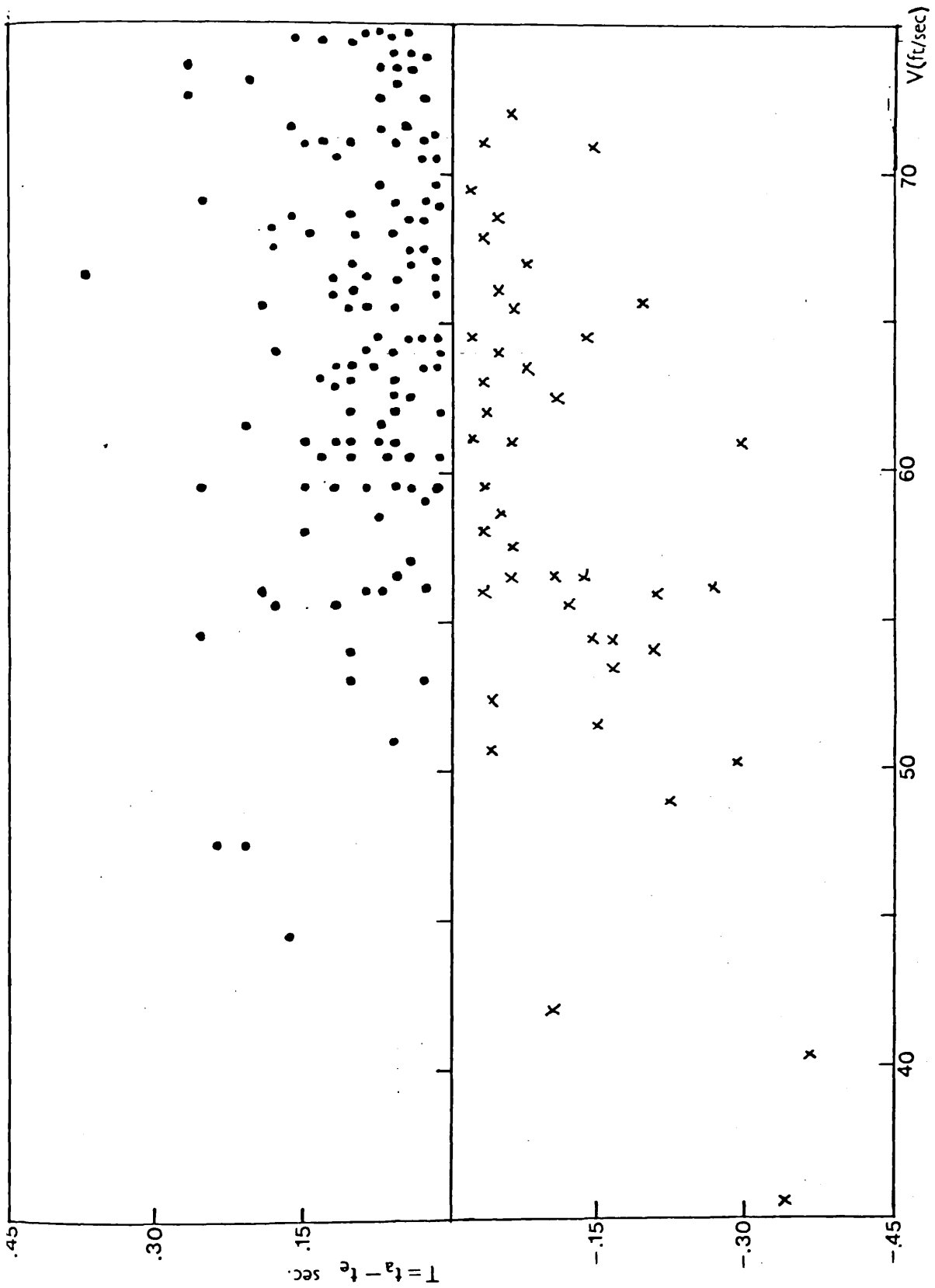


Figure 5.7 As figure 5.5, but for Virginia Water.

II. A MEASURE OF DECELERATION OF MAIN ROAD STREAMS

DRIVERS AT T-JUNCTIONS: A REGRESSION ANALYSIS

5.6 Introduction

In part I of this chapter, the deceleration behaviour of nearside oncoming vehicles at priority control rural T-junction was studied. The time difference $\tau = t_a - t_e$, where t_a and t_e denote the actual and expected times of arrival respectively, was taken as a measure of this behaviour.

It was shown that the behaviour of an approaching driver depends on whether or not (a) the junction is unoccupied, (b) a crossing vehicle is present and (c) a crossing vehicle is actually turning. A positive value of the difference τ was considered as a measure of deceleration and a negative value of τ as a measure of acceleration. In the case (c) it was found that 94% of the observed approaching major road vehicles showed a positive change in τ , that is a real deceleration.

In addition, data were collected on the accepted gaps of the turning drivers, and their manoeuvre time (the time to complete the turning manoeuvre). Here this measure of deceleration, for each approaching vehicle for case (c), is examined in relation to the speed of the vehicle as well as in relation to the accepted gap and crossing time of the turning vehicle. The intention is to see whether these observed measures of deceleration were random, or whether they were due to such factors as, speed of approaching vehicle, accepted gap, and crossing time.

5.7 Speed of approaching vehicles

The time difference τ of the two arrival times is considered as a measure representing a change in the speed of the approaching vehicle. Furthermore, since τ is taken as a measure of deceleration, its relation to the speed is of primary importance as deceleration (acceleration) may be a function of speed. As a first step, a regression analysis was used, with the speed as the independent variable. No correlation between the two variables was found and the regression was not significant at the 5% level of significance, table 5.8.

Figures 5.8, 5.9 and 5.10 show plots of these observed measures of deceleration versus speed for the three sites. The scatters are such that no relationship can be observed, and it is deduced that τ is independent of speed. Thus, τ can be considered not just as a behavioural variable but as a valid measure of the actual deceleration (acceleration) of the approaching vehicle.

5.8 Accepted gap and manoeuvre time of turning vehicles

The observed measure of deceleration was examined in relation to the accepted gap of the turning vehicle; although an indication of negative correlation between the two variables was found, the overall correlation was not significant at the 5% level. Then a multiple regression was also applied with the measure of deceleration of the approaching major road vehicle as the dependent variable and the accepted gap and the manoeuvre time of the turning vehicle as

the independent variables. Again a negative correlation was indicated, but the overall multiple regression did not give a significant result at the 5% level.

However, the results of previous work [Botton (1975), Evans and Herman (1976)], show that drivers accepting short gaps complete their manoeuvre more quickly than those accepting long gaps. Wennell and Cooper (1981) also found that there was a positive correlation between the median accepted gap for each range of crossing time and the mean crossing time at that range. Accordingly, it was decided to use as independent variable the difference between the accepted gap and the manoeuvre time in order to examine the observed measure of deceleration in relation to the two parameters of the turning vehicle's manoeuvre. This difference shows the time remaining, before the approaching vehicle arrives at the junction, from the moment the turning vehicle has completed its manoeuvre.

A further regression analysis was carried out. The following model was fitted on the data from the three sites.

$$\tau = a + b(G - T_c) + e, \text{ where}$$

τ = a measure of deceleration in secs.

G = accepted gap in secs., and

T_c = manoeuvre time in secs.

The linear regression gives the following results, for each site separately: $X = G - T_c$ and $\tau = Y$ (see also table 5.9)

(1) Sunningdale; Crossing.

$$Y = .45 - 0.094 X \quad 1.0 \leq X \leq 4.5$$

(2) Wentworth; Crossing,

$$Y = .41 - 0.067 X \quad 1.5 \leq X \leq 4.8$$

(3) Virginia Water; Crossing,

$$Y = .47 - 0.047 X \quad 1.7 \leq X \leq 4.9$$

For each site the measure of deceleration, τ , of the approaching major road vehicle is negatively correlated with the time difference, $G - T_c$ (table 5.9). The results are significant at all the sites, with the level of significance being higher at the first two sites than at the third. The regression lines are shown in figure 5.11. Although the slopes of the regressions lines are different, the difference between these slopes are not statistically significant.

The proportions of variation explained by the regression are around 35% for all the sites. This is rather low, but it should be noted that the regression equations for the measure of deceleration τ are based on data for individual drivers and not on grouped data, and therefore, there a higher residual variance is expected.

5.9 Discussion

The results of the present study seem to give an explanation of the observed measures of deceleration of the approaching major road vehicles, when a crossing vehicle was actually turning.

The finding that there is no significant correlation between the measure of deceleration and the speed is not surprising, since it was observed that the site with the highest mean value of the

measure of deceleration was the Virginia Water junction, which had the lowest mean speed as well.

The slopes of the regressions, although they are different - with the Sunningdale junction having the highest and Virginia Water the lowest - were not statistically significantly different at the 5% level.

However, the analysis of the data showed that the smaller the difference (G-T) - that is the smaller the accepted gap and the larger the crossing time - the higher the observed measure of deceleration of the approaching major road vehicle. Hence the hypothesis that the observed measure of deceleration depend on the decision of the driver of the turning vehicle and the performance of the turning vehicle may be accepted. Thus our measure of deceleration indicates a precautionary attitude of the approaching major road driver when a crossing vehicle is performing its manoeuvre.

The significance of the regression can be accepted as an explanation of this precautionary attitude in general. However, the outliers in the data are important as well, since they are mainly caused by severe changes in the speed. Therefore, apart from the characteristics of the turning vehicle and a random effect of the approaching driver and the junction layout, there may be some other explanatory factors of the observed measure of deceleration.

site	correlation coefficient	D.F.	significance of r	slope of the regression line	significance of b
Sunningdale	.256	32	No sign	.004	No sign
Wentworth	.154	78	No sign	.003	No sign
Virginia Water	.104	61	No sign	.006	No sign.

Table 5.8 Regression analysis of the measure of deceleration data τ , with speed V , for all three sites.

site	Correlation coefficient	D.F	significance of r	slope of the regression line	significance of b
Sunningdale	-.593	32	p < .01	-.094	p < .01
Wentworth	-.376	78	p < .05	-.067	p < .01
Virginia Water	-.281	61	p < .05	-.047	p < .05

Table 5.9 Regression analysis of the measure of deceleration data T_c , with the difference $G-T_c$, for all three sites.

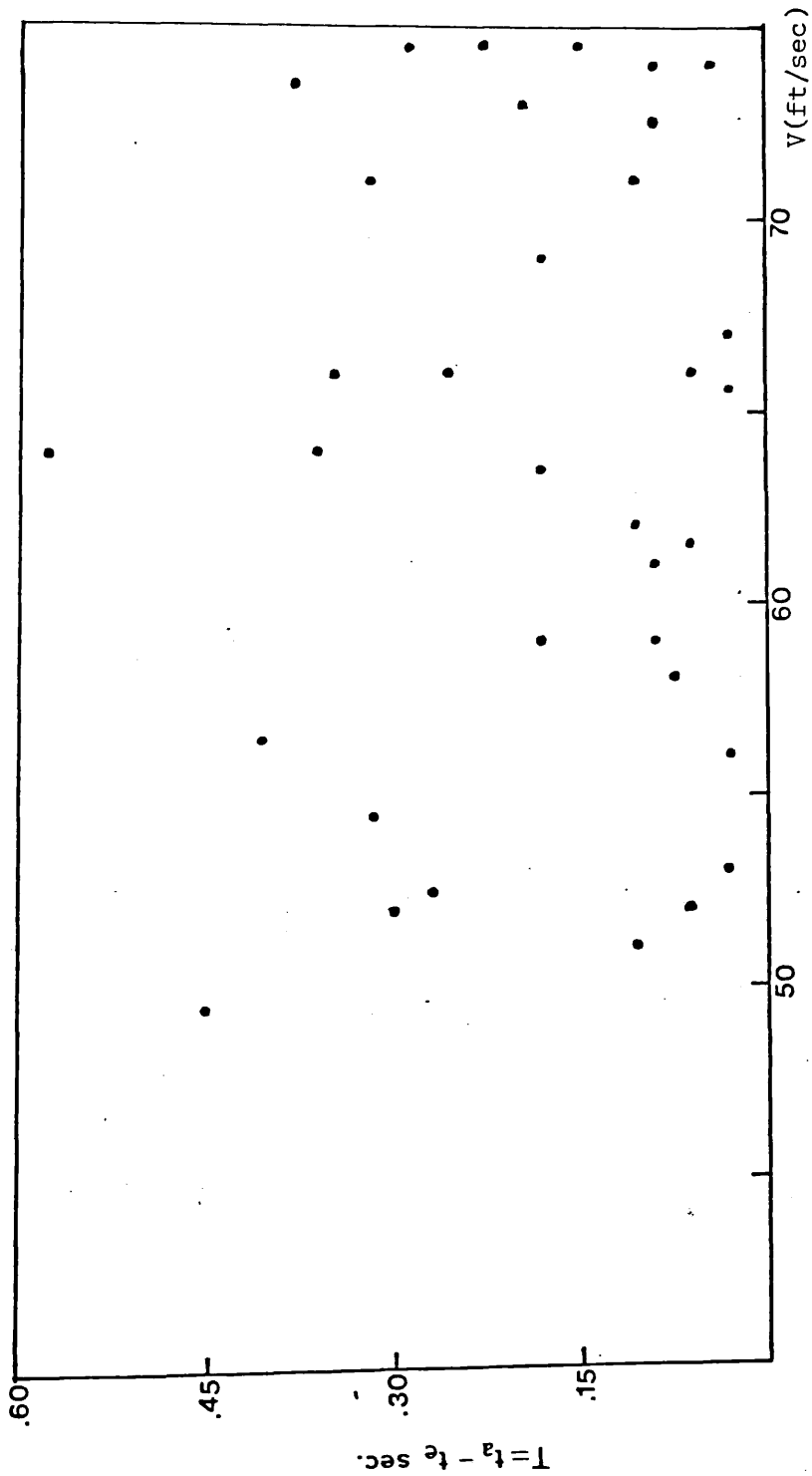


Figure 5.8 The observed decelerations in seconds versus speed in ft/sec, for case (c), at Sunningdale.

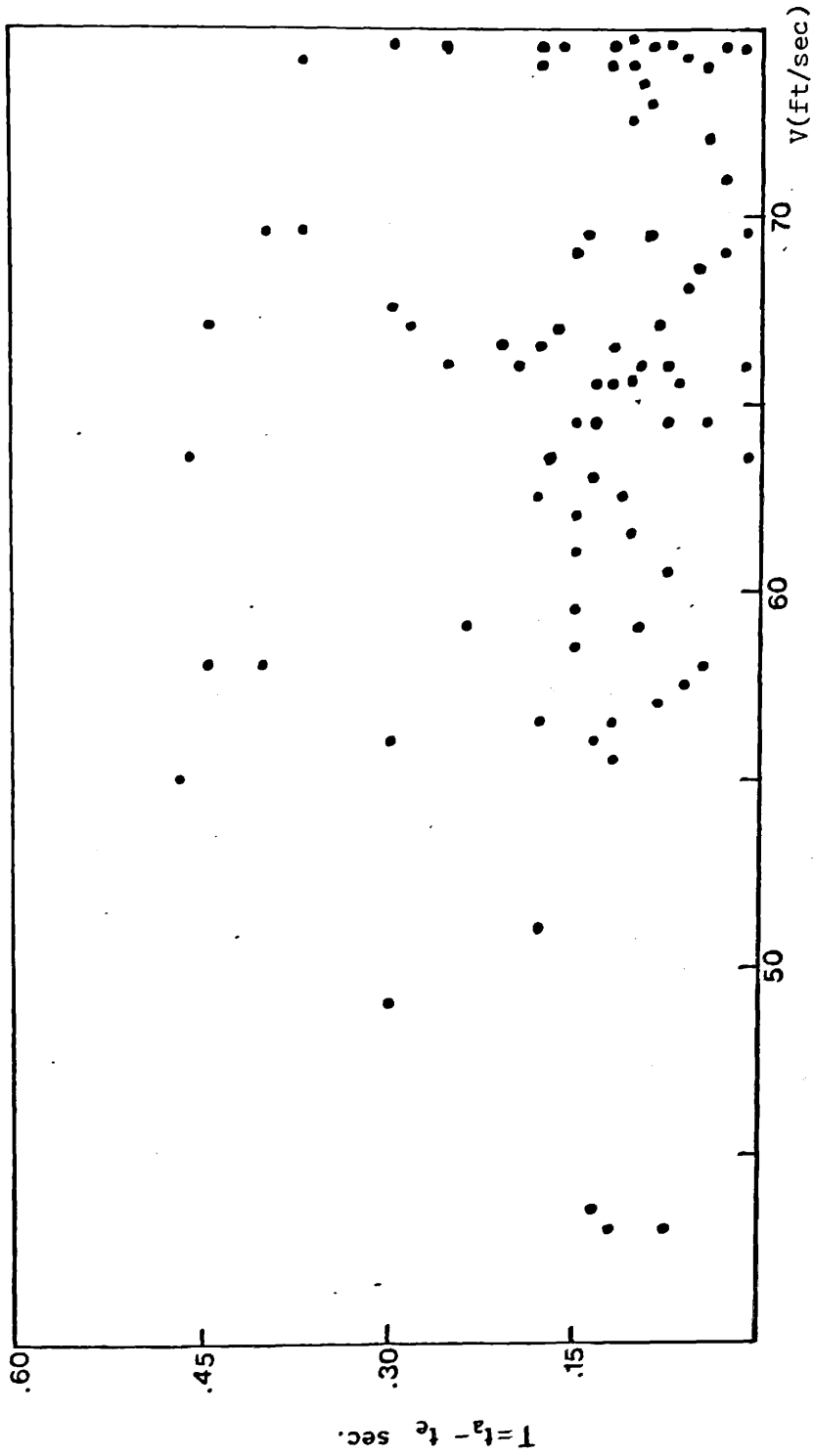


Figure 5.9 As figure 5.8, but for Wentworth.

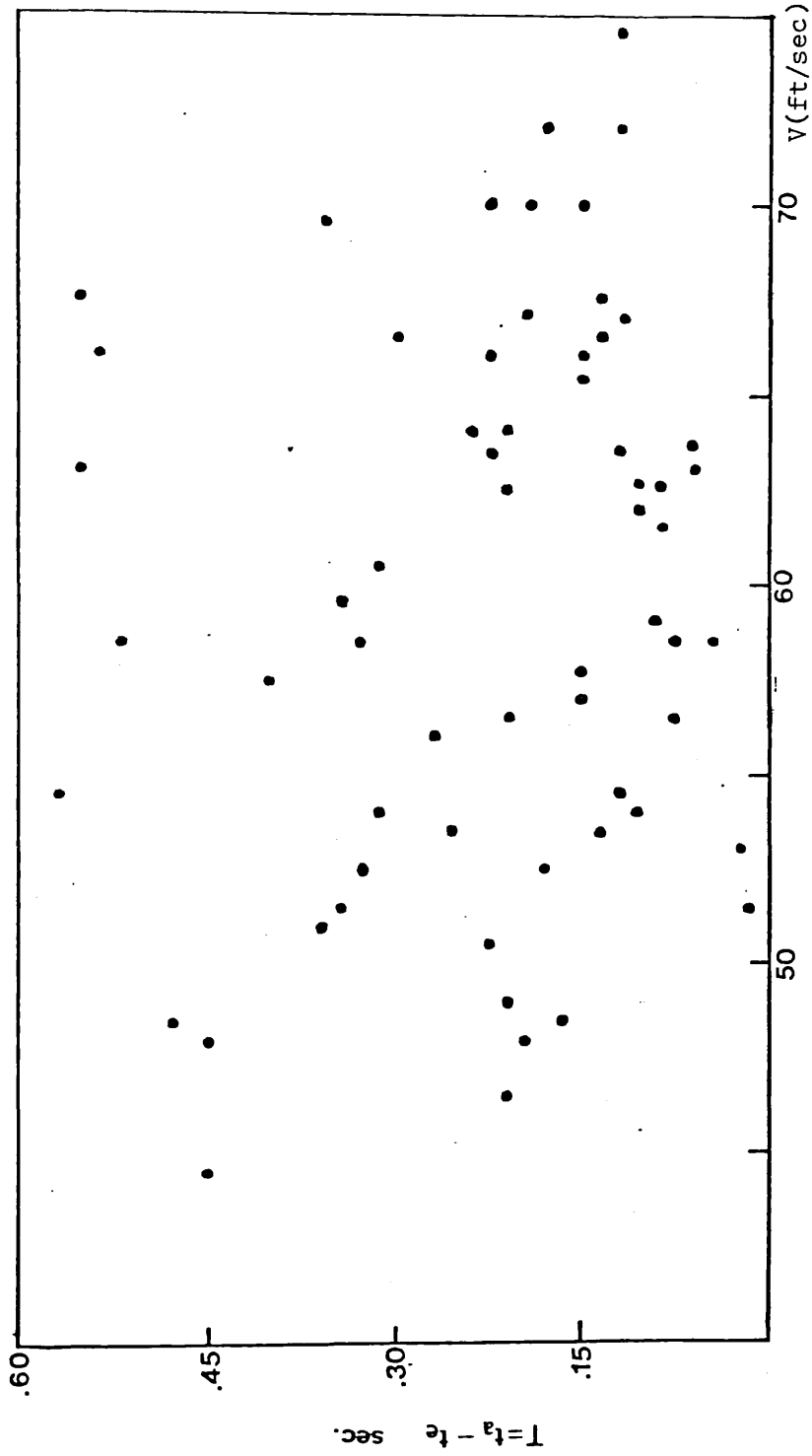


Figure 5.10 As figure 5.8, but for Virginia Water.

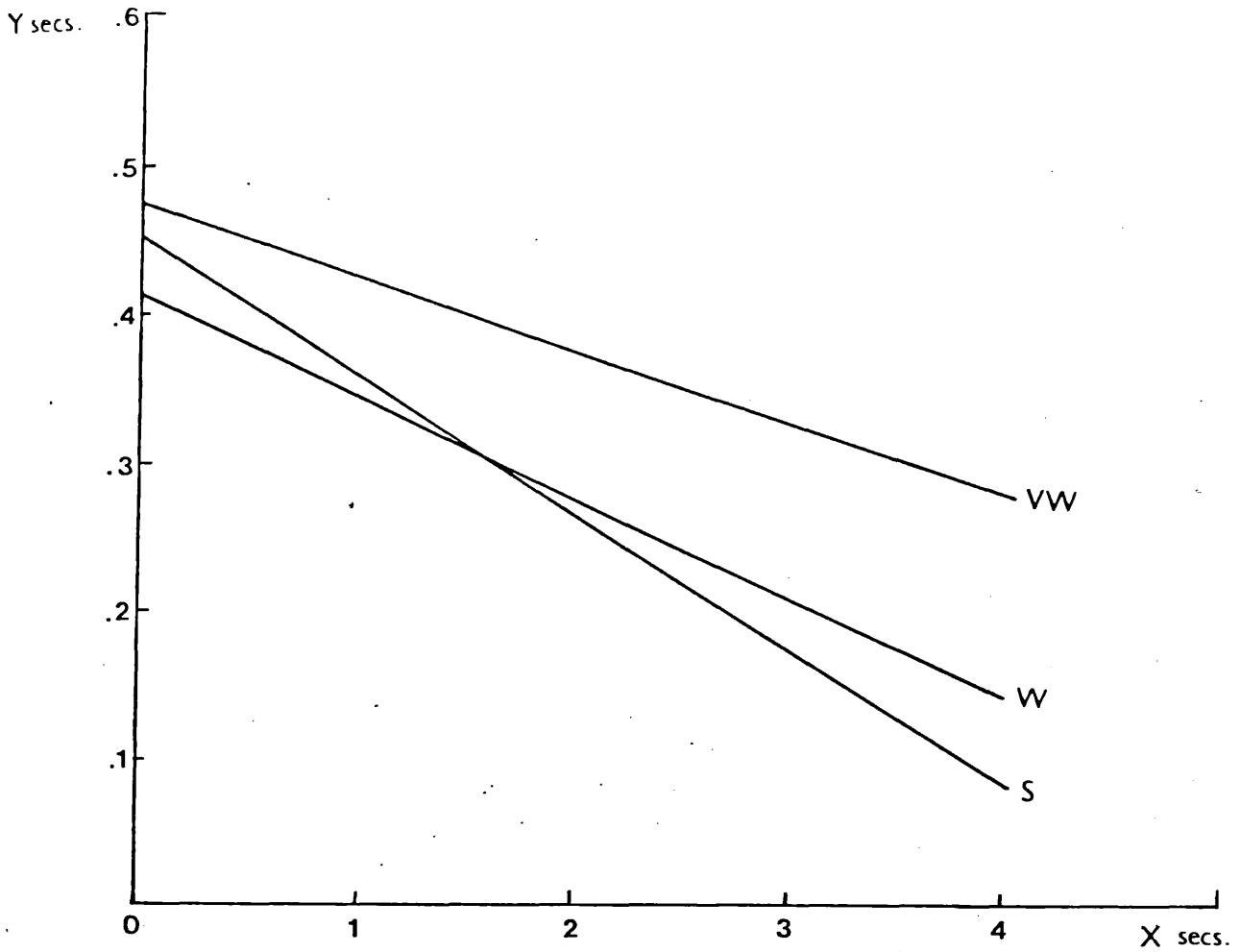


Figure 5.11 Regression lines of deceleration (secs) against the difference $(G - T_c)$, between accepted gap (G) and crossing time (T_c) in seconds, for the three sites. $(X = G - T_c, Y = \tau)$ (S = Sunningdale, W = Wentworth, and VW = Virginia Water).

CHAPTER 6

A CONFLICT SIMULATION MODEL AT T-JUNCTIONS

6.1 Introduction

Simulation models have been extensively used in studying various aspects of traffic. Intersection performance has been one major area of these studies as far as reduction of delay or improvement of maximum capacity are concerned, for example, Salter (1971), Rumsey et al (1972), Ashworth et al (1975), Beilby (1975), Varsarhelyi (1976) and Varol et al (1977).

Although it is of obvious interest to be able to estimate risks for different road-users at various locations, none of the above traffic simulation studies have been concerned with a detailed study of measure of risk. Traditionally, the only method of estimating risks has been by analysing accidents. Recently, the traffic conflicts technique (TCT) was developed by Perkins and Harris (1968), as an operational tool in road safety research. They defined a traffic conflict as any potential accident situation, leading to the occurrence of evasive actions such as braking, lane change or traffic violations. Campbell and King (1970) used this (TCT) to measure the accident potential of two rural intersections. They found an association between observed conflicts and reported accidents only when rear-end conflicts and rear-end accidents were omitted; the reason for the lack of association was that a disproportionately large number of minor rear-end conflicts was observed. Spicer (1972), Older et al (1976) improved the TCT introduced by Perkins et al (1968) by introducing

a severity scale for the evasive action. They developed a severity grading in five categories, ranging from precautionary braking or lane change to an emergency action followed by a collision.

Hyden (1975, 1980), in order to describe the danger of a conflict situation, used the time-to-collision concept. The following definition was used: a serious conflict occurs when two road-users are involved in a conflict situation in which a collision would have occurred within 1.5 seconds if both road-users involved had continued with unchanged speed and direction; the time is calculated from the moment one of the road-users started braking or changed lane to avoid collision. Later, Hyden et al (1982) introduced a threshold-level depending on the actual speed instead of the fixed 1.5 seconds.

Since more than 50% of the road accidents in the U.K. (Road Accidents, HMSO (1975)) occur at or near intersections, a conflict simulation model has been developed at Royal Holloway College (under contract to T.R.R.L.) to allow evaluation of risk at T-junctions. This model was developed from a preliminary approach by Ferguson (1973) and subsequent improvements by Cooper and Ferguson (1976) and Darzentas et al (1980).

In the next section a brief description of this model is given, while in section 6.3 early results of its validation are presented. A modification of this model and the event generation process are described in sections 6.4 and 6.5 respectively. Some results and discussion are presented in section 6.6.

6.2 The Simulation Model

A simple type of priority junction is modelled; the junction is assumed to be located in a rural environment. Six possible vehicles movements are permitted at the junction, which is controlled by a

'Give Way' sign (figure 6.1). However, the following description of the model is restricted to the simple crossing manoeuvre (right turn into the minor road, LT Figure 6.1).

All vehicles in this model have similar characteristics and accelerate and decelerate uniformly. The major road vehicles are divided into two classes, followers and free movers. The followers are assigned a minimum headway which is a function of speed, while the free movers are assigned a headway equal to that minimum value plus a value sampled from a negative exponential distribution. All turning drivers yield right-of-way to priority vehicles on the major road, so a queue is allowed to be formed in the major road. Only the leader of the queue may attempt to turn. Each driver is assigned a critical gap which is sampled from an empirical distribution; a different gap is sampled at each turning attempt. If the presented gap is longer than this critical gap the driver accepts the gap and turns; otherwise the gap is rejected and the whole process is repeated for the next presented gap. Each turning driver is assigned a manoeuvre time which is the time needed to complete the turning movement. If the manoeuvre time is less than the time available before the arrival of the next major road vehicle, the approaching vehicle is forced to slow down to avoid a collision and a conflict occurs. The severity of the resulting conflict depends on the rate of deceleration required. Five grades of severity are used in the model. Only decelerations in the original lane of motion are considered, since lane change is not permitted in the model. Balasha et al (1980) showed that transverse decelerations may be neglected.

Several input parameters are held constant for all runs, although

the facility exists to vary them. They are the maximum queue length; the move up time of vehicles in the queue; the rate of deceleration which determines the severity grade of a conflict; the initialisation time and the simulated time; and the number of runs for each set of parameters (random numbers are different for each run). The following parameters are generally varied for each set of runs:

- (i) flow in each traffic stream (veh/hr);
- (ii) mean and standard deviation of speed distribution (ft/sec); and
- (iii) gap acceptance parameters.

6.3 Validation of the model

The conflict simulation model, described above, was initially based on the premise that there was a relationship between serious conflicts observed at a junction and the number of reported accidents at that junction. McDowell et al (1982) compared the model's predicted conflict rates for eleven junctions with the numbers of reported injury accidents. The junctions were ranked in decreasing order by the number of accidents, as well as model conflicts. A significant agreement between the two rankings was found, using Spearman's rank correlation coefficient, when the number of model conflicts for the simple crossing manoeuvre (LT) was combined with the number of conflicts corresponding to the crossing part of the complex cross-merge manoeuvre (SR). However, some questions still remain; such as, whether the number of sites was sufficient to justify the strong conclusions from the rank correlation, and the selection of accidents by turning involvement but not by other factors observed during the conflict studies.

In the validation process, an attempt was also made to relate conflicts in the model to conflicts on the road by comparing simulated and observed decelerations. This was also carried out only for the crossing manoeuvre (LT). Experiments at three rural T-junctions (see chapter 5) showed that, firstly, there was an overestimation of the simulated decelerations compared with the observed ones; and, secondly, it was found that, in the case where a turning vehicle was present at the junction waiting to turn, the oncoming drivers were decelerating in most cases.

As a consequence of this attempted two-stage validation of the model, as well as of the observed mismatch of small and intermediate headways when the two population headway model is used as input, a modification of the simulation model has been made.

6.4 A modification of the model

The two major changes in the simulation model are the generation of major road vehicles arrival times and the input of the two deceleration distributions (chapter 5).

The three population headway model is now used as it was shown (chapter 3) to give a more satisfactory representation of small and intermediate headways. Hyden (1980) suggests that most serious conflicts occur when the accepted gap is less or equal to 1.5 seconds. In our model, the followers are described by a truncated normal distribution, while the free movers and others are described by two truncated negative exponential distributions. The truncation points, by which the headways are divided into three classes, as well as the proportion of headways in each class are calculated from the observed data. As a consequence of the new headways model, the minimum headway

allowed between consecutive vehicles will be equal to the observed minimum headway.

The parameters of the deceleration distributions are calculated as in chapter 5, and will be used in the simulation model in the following way;

- (1) when a gap is rejected by the waiting turning driver, the expected arrival time of the major road vehicle will be adjusted by adding the amount of deceleration sampled from the appropriate distribution (case (b), chapter 5); and
- (11) when a gap is accepted and the oncoming major road vehicle is going to arrive at the junction before the turning vehicle has cleared it, the expected time of arrival of the oncoming vehicle will be adjusted by an amount sampled from the appropriate distribution (case (c), chapter 5) - this sampled deceleration is referred to as 'Precautionary Conflict' and it is measured in seconds. Then it is checked whether the sampled time is enough for the collision to be avoided; if it is not, a further deceleration is imposed on the oncoming vehicle equal to the amount needed for a collision not to occur. This is called a 'Model Conflict' and it is measured in ft/sec^2 .

6.5 Generation of events in the model.

The revised version of the simulation model deals only with the simple crossing manoeuvre (right turn into the minor road, LT figure 6.1). Vehicles approaching the junction from the right (streams RA and RT, figure 6.1) are generated in a single process which does not allocate them to streams. A known proportion of these vehicles will turn into

the minor road (RT); these are chosen by sampling from an uniform distribution. Vehicles approaching the junction from the left are generated independently in the two major road streams (LA and LT, figure 6.1).

The arrival of vehicles in the priority streams are generated by the following process: the class in which the vehicle belongs is decided by sampling from a uniform distribution. Then the time headway (time between the previous vehicle and the present one arriving) is sampled from the appropriate distribution; the time of arrival of the present vehicle, assuming its motion remains unchanged, is the time of arrival of the previous vehicle plus the sampled time headway. The vehicle speed in the major road is sampled from a Normal distribution whose parameters are estimated from observations. The vehicles' position may then be inferred from its speed and arrival time. If the present vehicle is travelling faster than the previous one, it may have to decelerate in order to maintain the minimum headway; in this case, the speed and arrival time are adjusted.

A crossing driver who is presented with a gap either accepts or rejects it. The decision to accept or reject is made by comparing a gap t_c (critical gap) sampled from the appropriate distribution for the population of drivers with the presented gap t_p , which is accepted if $t_c \leq t_p$. The gap acceptance function has the form of a cumulative Normal distribution, with the logarithm of the time as the variable.

The manoeuvre time of the crossing vehicle is sampled from a truncated Normal distribution, with mean and standard deviations estimated from observations.

6.6 Results and Discussion

Simulation runs have been restricted to less than 1000 veh/hr in major road stream and turning flows up to 300 veh/hr, i.e. an uncongested junction has been considered.

Data from three rural sites have been used for input in the simulation model. The results from the three versions of the model are given in table 6.1. Version MOD(1) is the one described in section 6.2; version MOD(2) is slightly modified in that the three-population headway model is included; while MOD(3) is a further modification using the deceleration distributions (see section 6.4) as input in the model as well. All the results (table 6.1) are the average from 10 runs of a 10-hour simulated period each.

When the three - population headway model [MOD(2)] is used as input instead of the two - population model [MOD(1)] an increase in the number of model conflicts by severity and in total is observed, table 6.1. This is expected, however, since by using the two - population model a mismatch of short and intermediate headways has occurred; these headways, if accepted, are more likely to cause a conflict. Generally, from the results, it seems that the simulation model depends on the form of the headway model used. The site with the highest number, in total, of model conflicts is Virginia Water, while Sunningdale is the site with the lowest number.

In the case where the distribution of decelerations is introduced in the simulation model, version MOD(3), the output consists of two types of conflicts: precautionary and model conflicts. The results from table 6.1 show that the increase in the total number of model conflicts, compared with those from version MOD(1), still remains.

However, there is a reduction in the total number of conflicts compared with version MOD(2); the reduction is a consequence of precautionary conflicts in the current version of the simulation model. It is also noted that there is a reduction in the number of strong conflicts, where by strong conflicts we meant model conflicts in the fourth and fifth classes of severity, table 6.1. This reduction in the number of strong conflicts is observed because model conflicts occur if the initial sampled deceleration (precautionary conflict) imposed on the oncoming vehicle was not big enough.

It is believed that the latest version [MOD (3)] of the conflict simulation model is the most realistic. It is considered so, because the observed precautionary decelerations were a result of a distribution obtained by some hours of observations at each site. However, in a longer period the number as well as the size of deceleration might be increased. Hence, the simulated model conflicts might be explained by the higher decelerations which might have been observed if the experiment has lasted for a longer period of time.

Severity	VERSION			
	MOD(1)	MOD(2)	MOD(3)	
1	.1	3.3	2.0	
2	.2	.7	.9	
3	.1	.7	.9	VIRGINIA
4	.2	.5	.2	WATER
5	.0	.2	.0	
Total	.6	5.4	4.0 (27.0)	
1	.3	2.3	.9	
2	.1	.7	.6	
3	.1	.2	.6	WENTWORTH
4	.1	.2	.1	
5	.0	.2	.1	
Total	.6	3.6	2.3 (14.0)	
1	.3	1.2	.8	
2	.3	.8	.6	
3	.2	.2	.3	SUNNINGDALE
4	.1	.2	.1	
5	.1	.1	.0	
Total	1.0	2.5	1.8 (10.0)	

Table 6.1 Numbers of conflicts involving crossing vehicles by using different versions of the simulation model (average of 10 runs of 10 hours each).

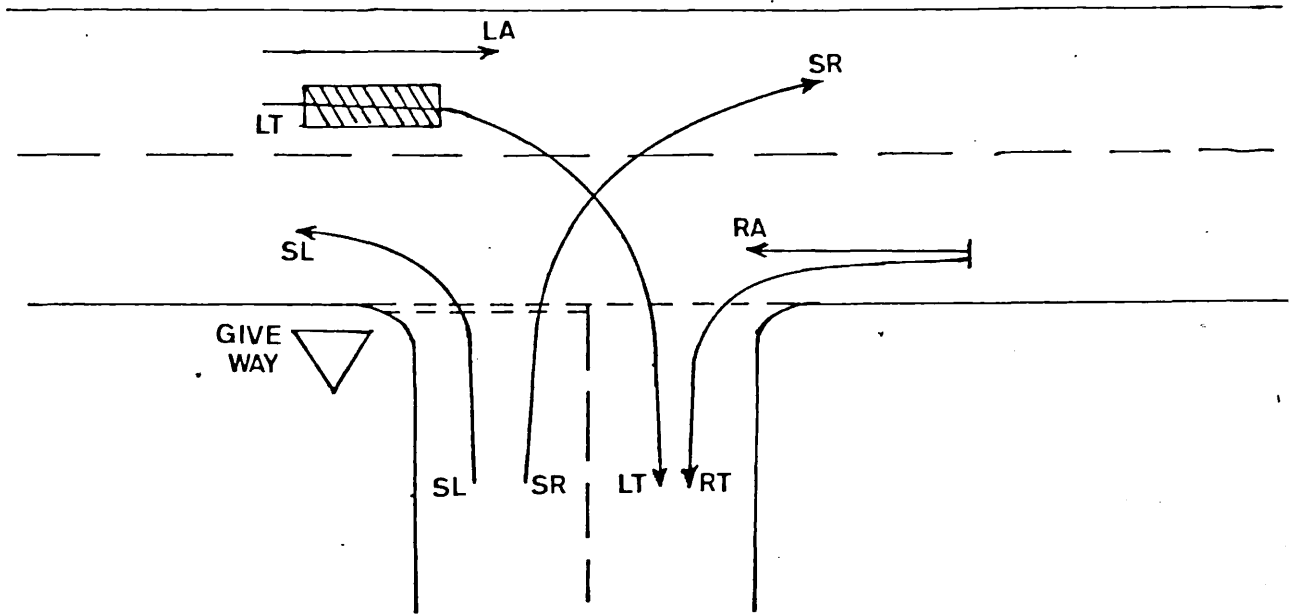


Figure 6.1 The priority controlled T-junction showing the traffic streams . The traffic streams are labelled as in figure 2.2 .

SUMMARY AND CONCLUSIONS

Headways are among the most important of all traffic variables and thus it is of prime importance to have adequate models to describe them appropriately. A great number of headway models has been developed and used by road traffic researchers; these models have shown a number of differences according to the assumptions they were based on, and the different road types to which they were applied.

In an ideal stream of traffic, in which vehicles do not interact and thus can pass each other freely, the headways can be described by a negative exponential distribution; however, in a real situation, this only prevails for headways greater than a critical value. Headways less than this critical value are considered to be in the following-the-leader mode, and so platoons are formed in a stream of traffic. The choice of this critical value is one of the main difficulties in dealing with observed data and thus a mixture of the two groups of headways may be expected at the intermediate headways.

A three population headway model was found to be more satisfactory than a two headway model when small and intermediate headways were predominant. The model was tested against a wide range of data from rural and suburban sites with different flow levels. The three groups of drivers are: the followers described by a truncated normal distribution, the free-movers described by a truncated exponential and an intermediate group (those leaving or joining a platoon) described by a further truncated exponential distribution.

A modified geometric distribution was found to give an excellent description of the platoon sizes. Generally, longer platoons were observed at the rural than at the suburban sites. This may be due to the higher flows at the rural sites. The time gaps between successive platoons, that is the time gap between the last vehicle of a platoon and the leader of the next one, were found to follow a negative exponential at most of the sites.

The autocorrelation coefficients of the time series of headways were calculated and it was found that for single traffic streams, on suburban and rural roads, the time headways may be considered as independently distributed for flow levels than 1000 veh/hr. A further analysis of headways within platoons showed that a truncated normal distribution can describe time headways within each speed range; generally, it was found that the distribution of headways within platoons, at a given rural site, was independent of speed. Plots of distance headways versus speed showed that the definition of platoon given in this thesis, closely corresponds to the set of vehicles which are following too closely. Generally, it was found that all followers (second and subsequent members of platoons) follow too closely, in terms of Highway Code advice, and the amount by which they do so increases with speed, while the proportion following at less than three quarters of the recommended distance can be as high as 45%.

A technique of measuring the deceleration behaviour of major road stream drivers approaching a priority-controlled T-junction was developed. It was found that this behaviour was affected, in terms of

a time difference between actual and expected times of arrival, by the junction

- (1) having vehicles waiting to cross [case (b) of chapter 5] and
- (2) having vehicles actually crossing [case (c) of chapter 5].

The time difference does not directly measure the deceleration but as it was shown to be independent of speed, it can be considered as a surrogate measure of deceleration. In the case where the junction is unoccupied [case (a)] the distribution of decelerations, located around zero, is described by a normal distribution. In the other two cases, (b) and (c), the real effect due to the crossing vehicles, that is the observed decelerations after eliminating the background, are described by normal and truncated normal distributions respectively. For all sites, it was found that the major road drivers decelerated more in case (c) than in case (b). In about 1% of the events in case (c) the deceleration was severe, greater than 0.5 seconds change in arrival time.

Furthermore, a regression analysis of the data for case (c) showed that the smaller the difference between accepted gap and crossing time, that is the smaller the accepted gap and the longer the crossing time, the higher the observed deceleration of the approaching major road vehicle. Thus our observed measure of deceleration indicates a precautionary attitude of the approaching major road stream drivers when a crossing vehicle is performing its manoeuvre.

Observed data in the forms of parameters and distributions are used as the input of a conflict simulation model. The output of the model consists of precautionary and model conflicts which are

defined as the decelerations imposed upon the approaching major road vehicles. Other studies suggest that these conflicts may be used to assess the relative risk of accidents at a site, but accident data on the sites and manoeuvre considered are sparse.

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APPENDIX 1

EFFECT OF WET AND DRY WEATHER CONDITIONS ON
GAP ACCEPTANCE BEHAVIOUR

1. Introduction

Previous studies, Tsongos and Weiner (1969), Darzentas and McDowell (1981) have shown no significant difference in gap acceptance behaviour of drivers merging with or crossing a major road stream of traffic in daylight and darkness at rural T-junctions without road lighting. For the complex traffic manoeuvre (crossing and merging) under the same conditions, it was found [Darzentas and McDowell (1981)] that drivers, given a large farside gap, accepted larger nearside gaps in darkness than in daylight. A preliminary experiment at a street-lit suburban T-junction (Darzentas et al (1980)) showed that merging drivers accepted a higher proportion of short gaps in darkness than in daylight. However, further experiments at three street-lit suburban T-junctions McDowell et al (1982) did not show any significant difference in gap acceptance behaviour for both simple manoeuvres (crossing, merging) between daylight and darkness, while observations at one lit suburban T-junction for the complex manoeuvre showed the same result as in unlit rural T-junctions.

The above studies and those reported in the main text have

been carried out in dry weather conditions. It is obvious that rainy weather creates road and environmental conditions less favourable for driving. Malo and Mika (1960) analysed the relationship between weather and traffic accidents for one-year in two urban expressway systems in Detroit. They found that rain increases the number of accidents. They compared the proportion of accidents and the estimated vehicle miles travelled in different conditions. Foldvary and Ashton (1962) examined daily numbers of accidents and weather conditions in Melbourne. They defined a 'rainy day' as a day with over 30-minutes of rain. On these days they found an average of 30% more accidents than on dry days. The analysis also showed that wet weather effect was less at night than during the day. The effect of duration of rainfall was found not to increase the accidents rates. Thus, they pointed out that the beginning of a rainy period was the worst time, with slippery roads and the drivers not yet adjusted to the altered conditions. Orne and Yang (1973), proceeding from the idea that weather affects drivers behaviour, stated that the most important weather factors affecting traffic accidents were, in order, rain, temperature, surface conditions, lighting. They also found that sudden changes of weather do not have a direct effect on the accident rates. Sabey (1973) found that in the U.K., under dark and wet conditions, 20% more accidents occurred on wet roads than would be expected if roads were dry. Colding (1974) found an

increase, by comparison with dry periods, in injury accident frequency of 52% in rain and 50% on wet roads without rain in daylight. This increase was greater in darkness than in daylight. Satterthwaite's (1978) results agree broadly with these. They both classified a day as 'wet' or 'dry' according to the proportion of accidents occurring on wet or dry roads on that day. Talab's (1973) studies appear to show that rainfall had a smaller effect on injury accidents in Huddersfield than in London. The increase was found to be up to 50% in London, which compares with Colding's figure. The most severe effects of rainfall in increasing accident rates were during the night periods. There was no evidence that moderate and heavy rainfall had different effects on accident frequency. It is also suggested [OECD (1976)] that the increasing accident risk in rain is due to the reduction of visibility as well as to the decreasing skid resistance. The skid resistance during heavy rainfall is similar to that when there is no rain but the road surface is wet.

Since it is generally accepted that driving in rainy weather, or on wet roads without rain, is more dangerous than driving in dry weather conditions, a more cautious driver behaviour would be expected. A study of driver behaviour at two suburban and at one rural priority controlled T-junctions in daylight and darkness under dry and wet weather conditions was undertaken.

In the next section the experimental arrangements are described; section 3 presents the results and section 4 the discussion.

2. Experiment - Data Collection

Data were collected in Southern England: at two suburban T-junctions with road lighting and at one ^{rural} T-junction with road lighting in Ascot (A329/A330), Chobham (A319/A3046), and Wentworth (A30/A329) respectively.

At Ascot, the experiment was conducted during winter evening rush hours between 4.50 and 6.00 p.m. and therefore in darkness, under dry and wet road conditions. In the latter case, it was either drizzling or moderately raining for a period of half an hour, during which all drivers were using wipers; the road surface was wet during the whole period of these observations. At Chobham, the experiment was conducted during Spring and Winter rush hours under dry and wet road conditions in both daylight and darkness. In darkness, the weather conditions were very similar to those at Ascot, while in daylight, it was either drizzling or moderately raining during the whole period of observations; the road surface was wet and drivers were using wipers. At Wentworth, the experiment was conducted during off-peak hours in Spring, in daylight under dry and wet road conditions. In the latter case, the weather conditions were similar to those at Chobham in daylight.

Data were collected at all three junctions on the left turn out of the minor road (merging manoeuvre - SL) and on right turn into the minor road (crossing manoeuvre - LT), figure 7.1, using the microprocessor-based system Storr et al. (1979).

From inside a stationary car positioned on the verge by the junctions, observers recorded the arrival and departure times of

the turning vehicles and the arrival time at the junction of the vehicles in the priority stream. The recordings were made using the handset input devices described by Storr et al. (1979); the signals from the handsets were combined with a clock time (0.01 seconds) in the microprocessor, and stored on a cassette tape. These tapes were played back into a mainframe computer and then analysed. The speed data were collected using a hand radar; only the speeds of the oncoming vehicles passing through the junction were measured, ignoring those which were turning left into the minor road. At Wentworth, the speed data in daylight were collected by using cable detectors.

3. Results

The main purpose of the experiments was to investigate the effect of rain and wet road-surface on driver gap acceptance behaviour. Hence an attempt was made to isolate, as much as possible, other factors which were able to be measured.

The major and minor road flows (veh/hour), table 7.1, were similar in darkness at Ascot in dry and wet road conditions. There were significant differences at Chobham in both major and minor road flows (veh/hour) between wet and dry conditions in darkness (15%-22% less in dry conditions), while no significant differences were observed between dry and wet conditions in both major and minor road flows (veh/hour) in daylight, table 7.1. At Wentworth, some differences between dry and wet road conditions were observed in daylight (15% less flow in wet conditions).

The means and standard deviations of the speed distributions were

not significantly different at Ascot between the two conditions in darkness, table 7.2. However, at Chobham there was a significant difference in the standard deviations of speed between wet and dry conditions in daylight, table 7.2; in addition the mean speed in wet conditions was found to be smaller. In darkness at Chobham, no significant differences between the mean and standard deviations of the speed were observed.

A non-parametric χ^2 test showed no significant differences between the frequency distributions of headways at all three junctions (the time headways between major road vehicles in the same stream) table 7.3, despite some differences between the flows (veh/hour).

The median accepted gaps (gaps and lags combined) for both manoeuvres at each site, under both weather conditions in daylight and darkness, are given in table 7.4. No significant differences were found in the median accepted gaps between wet and dry conditions in both manoeuvres and at all three junctions, except for the crossing manoeuvre at Chobham where the median accepted gap in wet conditions was found to be significantly smaller than that in dry conditions in darkness (median test $p < .025$).

To illustrate the above results, the cumulative standardised frequency distributions of accepted gaps, classified into one-second intervals, for both manoeuvres at Chobham and Ascot in darkness and at Chobham and Wentworth in daylight, are given in tables 7.5, 7.6, 7.7. Figures 7.2 to 7.5 show the proportion of accepted gaps as well as the standardised cumulative frequencies for both manoeuvres and for all sites, under both weather conditions in daylight and darkness.

Marshall's (1951) non-parametric test showed that the accepted gaps under dry weather conditions are stochastically larger than those observed under wet road conditions at Chobham; that is, drivers accept smaller gaps in wet conditions than in dry in the crossing manoeuvre at that junction in darkness. Quite similar results (on a smaller scale) were found at the Ascot junction for the same manoeuvre under the same weather conditions in darkness, figure 7.3, although Marshall's test did not indicate any significant differences. In all the other cases, Marshall's test again did not show any significant differences. Furthermore, when lags and gaps were examined separately, the same results were obtained as when lags and gaps were examined together.

4. Discussion

Accident studies suggest that rainy weather and wet road conditions increase the accident rate, especially in darkness, and it is generally accepted that these conditions were less favourable for driving.

However, the experiments at Ascot, Chobham and Wentworth, where a number of parameters related to driver behaviour were measured, no significant differences were found in any of these parameters at any site, between wet and dry conditions, except in one case: in the crossing manoeuvre at Chobham, in dry conditions in darkness, drivers appeared to be more cautious than they were in any other conditions for the same manoeuvre at this junction, table 7.4. It is noted that there was also a smaller variation in speed of the major road vehicles at this junction under wet conditions than in dry in daylight. These observed differences at Chobham may be due to a

site effect, as compared with the Ascot junction. The Chobham junction is narrower, and the mean major road speed is greater, and it is normally without parked vehicles in the major road. The illuminance (light intensity) in darkness at three different points (1,2,3 Fig. 7.1) within each of the junctions was also measured, using a minilux portable photometer, and it was found that there was a 60% lower illuminance at Chobham.

In daylight, despite the fact that the Chobham and Wentworth junctions were located in different environments - one suburban (Chobham) and one rural (Wentworth), with different speed limits - no significant differences in the gap acceptance behaviour were observed between the two weather conditions at either junction.

The results of the experiments are not conclusive, mainly because of the small number of sites examined, and the restricted range of wet weather conditions. Nevertheless, a large amount of data was collected in the experiments during which it was either drizzling or raining moderately.

Accident studies have shown no evidence that the accident rate in moderately wet conditions is significantly lower than in heavy rainfall. Furthermore, they give some evidence of site effects - Talab (1973).

These findings, together with the results of the experiments, give some evidence on one factor involved in the observed increase in injury accident rates in wet weather conditions, especially in darkness, although they suggest a strong site effect. It would

appear difficult to draw general conclusions about driver behaviour at suburban or rural T-junctions between wet and dry conditions: the most one can say is that some appear more dangerous than others, but that the design of the site, and the special features of the user population, may be the overriding factors. Given similar situations, except for the site factors, drivers behave more dangerously at some junctions than at others, and, where they do, more accidents occur [McDowell et al. (1981)]. The most surprising finding from the conducted experiments was that drivers not only were not more cautious in rainy weather than in dry, but in some cases they appeared to be less cautious.

	DARKNESS			
	Chobham		Ascot	
	Dry	Wet	Dry	Wet
Major Road	332	392	687	615
Crossing	167	200	258	252
Merging	277	356	261	256

	DAYLIGHT			
	Chobham		Wentworth	
	Dry	Wet	Dry	Wet
Major Road	432	393	350	290
Crossing	226	213	300	270
Merging	337	336	200	170

Table 7.1 Flows at the experimental sites in darkness and daylight (veh/hour).

DARKNESS

	Chobham		Ascot	
	Dry	Wet	Dry	Wet
Mean	30.4	31.0	25.5	25.5
s.d.	3.5	4.2	3.8	3.6

DAYLIGHT

	Chobham		Wentworth	
	Dry	Wet	Dry	Wet
Mean	32.0	30.4	43.6	-
s.d.	4.3	3.2	9.3	-

Table 7.2 Mean and standard deviation of speed distributions
in the major road m.p.h.

	Chobham		
	ND-NW	DW-NW	DD-DW
χ^2	16.5	10.8	13.0
d.f.	17	17	17

	Ascot	Wentworth
	ND-NW	DD-DW
χ^2	19.3	17.5
d.f.	17	17

ND = Night Dry

NW = Night Wet

DD = Day Dry

DW = Day Wet

Table 7.3 χ^2 -test for differences in headway distributions
in major road.

DARKNESS				
	Chobham		Ascot	
	Dry	Wet	Dry	Wet
Crossing	6.50 [*] **	5.88 [*]	5.65	5.62
Merging	6.26	6.23	4.77	4.89

DAYLIGHT				
	Chobham		Wentworth	
	Dry	Wet	Dry	Wet
Crossing	5.87 **	5.96	6.75	6.74
Merging	6.21	6.14	5.67	5.94

Table 7.4 Median accepted gaps (lags and gaps) in seconds for both manoeuvres. The only statistically significant difference is between the starred and double starred pairs.

CHOBHAM

T(secs)	Crossing			Merging			
	Daylight		Darkness	Daylight		Darkness	
	Dry	Wet	Dry	Dry	Wet	Dry	
1.5	.019	.039	.017	.030	.041	.041	.057
2.5	.094	.087	.040	.096	.102	.110	.113
3.5	.214	.177	.127	.211	.195	.186	.207
4.5	.382	.326	.352	.342	.318	.309	.327
5.5	.518	.507	.578	.471	.478	.470	.463
6.5	.669	.645	.763	.607	.632	.602	.606
7.5	.781	.782	.832	.751	.788	.782	.751
8.5	.892	.909	.948	.880	.915	.896	.866
9.5	1.000	1.000	1.000	1.000	1.000	1.000	1.000

Table 7.5 Cumulative standardised frequency distributions of accepted gaps in daylight and

darkness in wet and dry weather conditions at Chobham.

<u>ASCOT</u>				
T(secs)	Crossing		Merging	
	Dry	Wet	Dry	Wet
1.5	.033	.026	.106	.066
2.5	.099	.109	.253	.219
3.5	.225	.243	.401	.348
4.5	.403	.414	.542	.489
5.5	.563	.600	.669	.634
6.5	.716	.742	.786	.771
7.5	.834	.855	.876	.870
8.5	.927	.930	.948	.952
9.5	1.000	1.000	1.000	1.000

Table 7.6 Cumulative standardised frequency distributions of accepted gaps in darkness in wet and dry weather conditions at Ascot

WENTWORTH

T(secs)	Crossing		Merging	
	Dry	Wet	Dry	Wet
1.5	.002	.019	.060	.072
2.5	.030	.032	.158	.121
3.5	.095	.124	.300	.276
4.5	.201	.261	.425	.364
5.5	.348	.398	.536	.508
6.5	.549	.536	.678	.613
7.5	.709	.679	.759	.751
8.5	.887	.823	.862	.878
9.5	1.000	1.000	1.000	1.000

Table 7.7 Cumulative standardised frequency distributions of accepted gaps in daylight in wet and dry weather conditions at Wentworth.

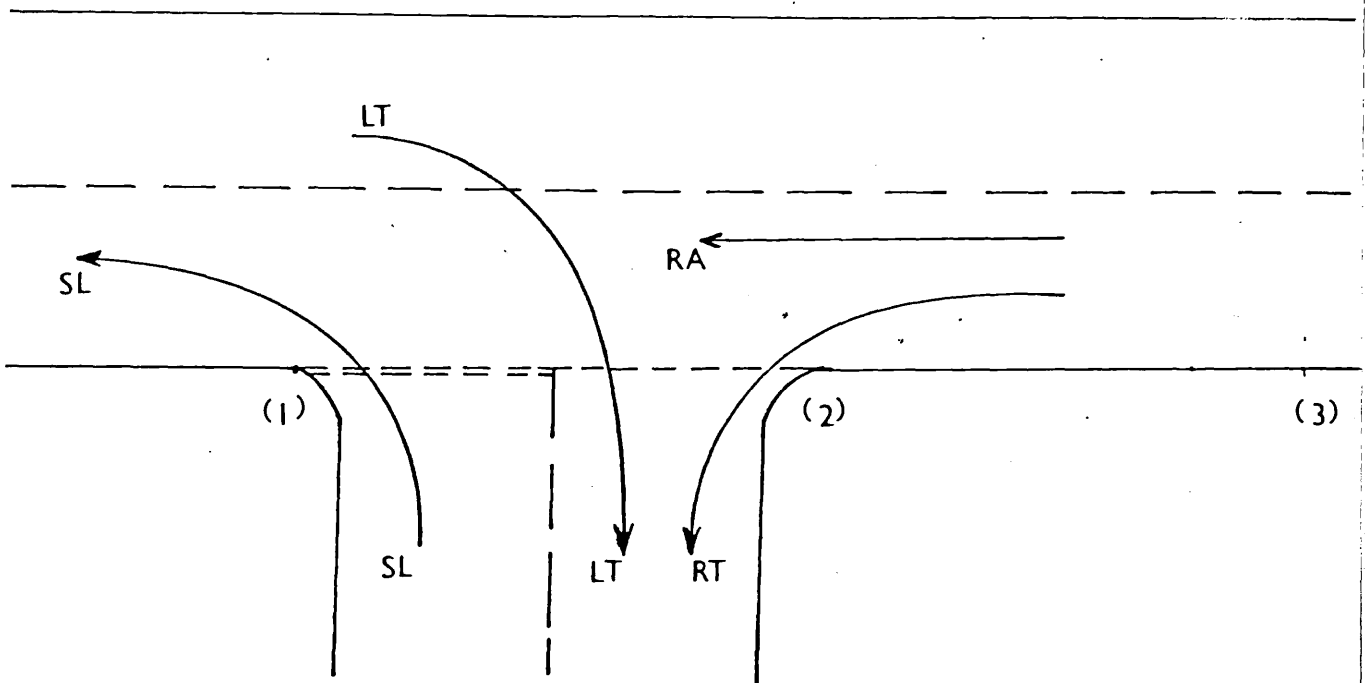
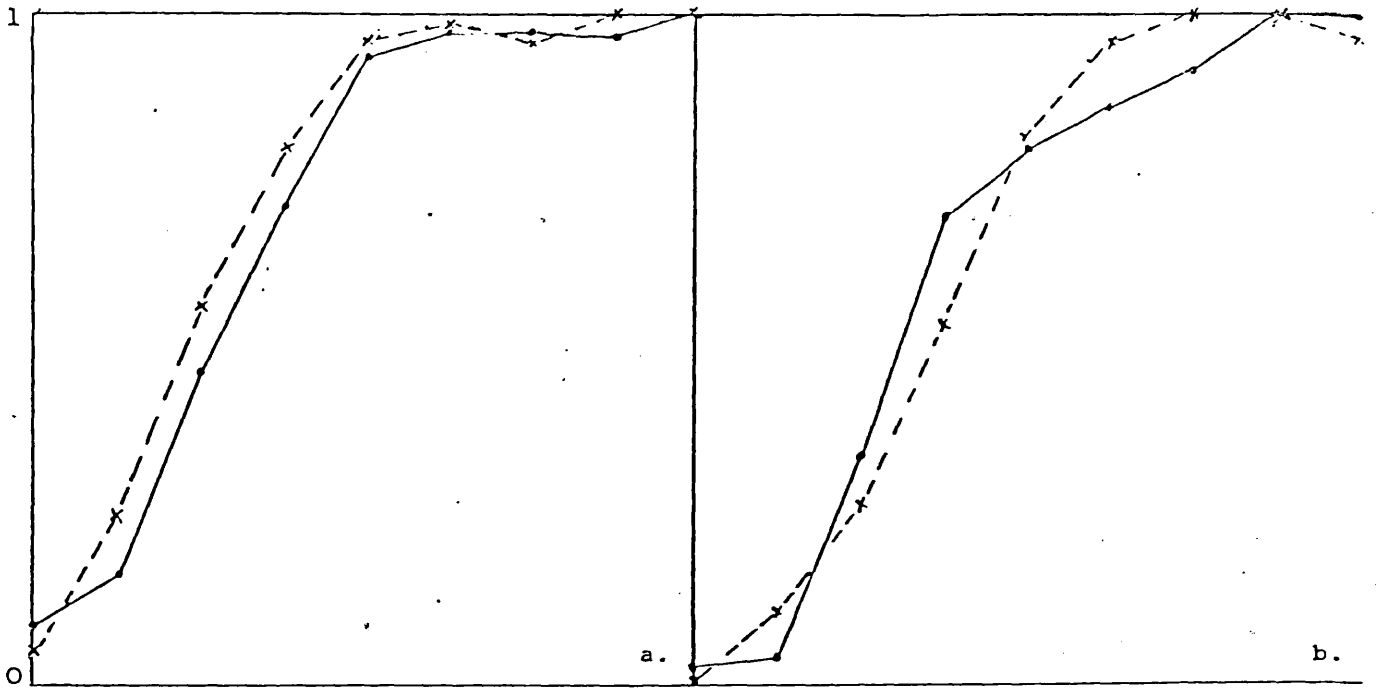
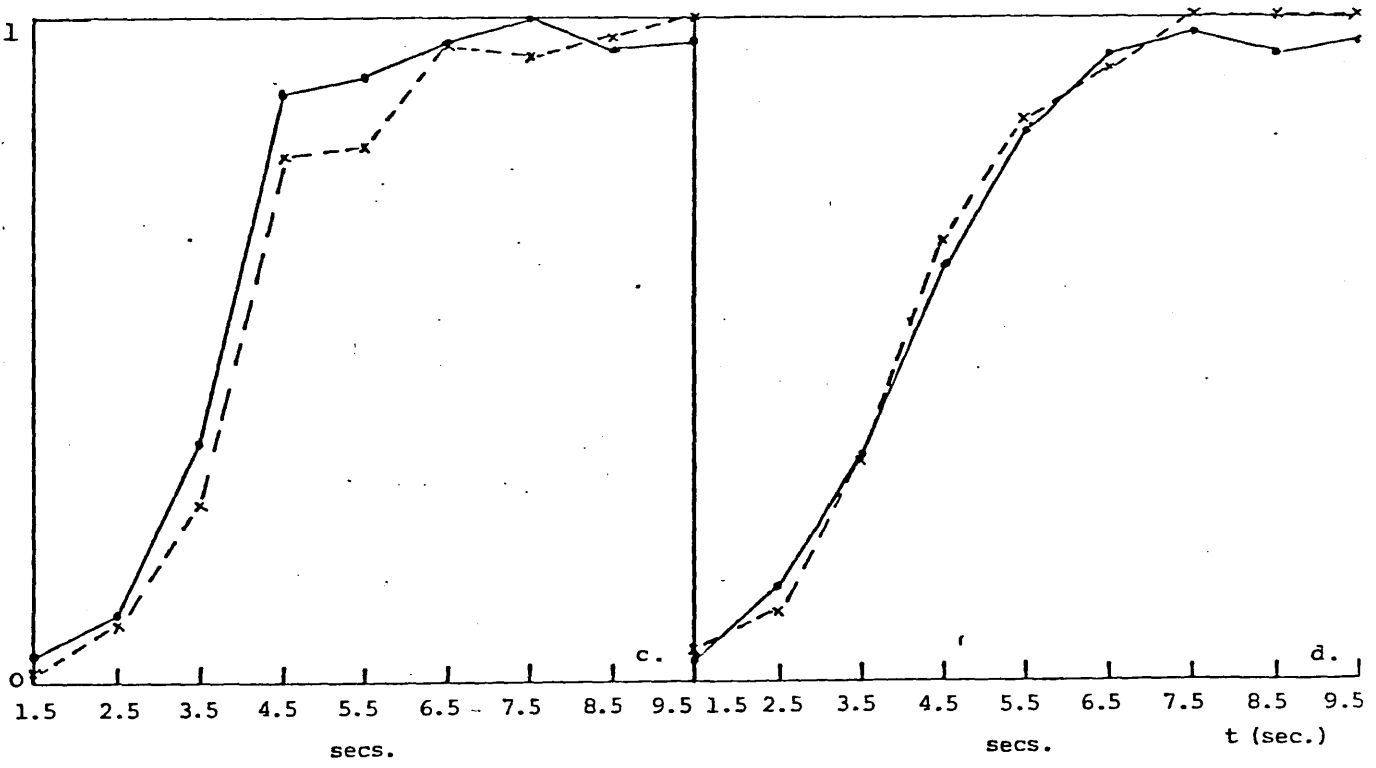


Figure 7.1 A priority controlled T-junction . The arrows indicate the manoeuvres ; 1 , 2 , 3 indicate the points at which illuminance measurements were taken .



a = Daylight Chobham
 b = " Wentworth



c = Darkness Chobham
 d = " Ascot

Figure 7.2 The proportion of accepted gaps observed in wet (.—) and dry (x---) weather conditions for the crossing manoeuvre in daylight and darkness. The vertical scale gives the proportions of accepted gaps (number of accepted/number of accepted + number of rejected) and the horizontal scale gives the size of gaps in seconds. The numbers present the midpoint of the intervals.

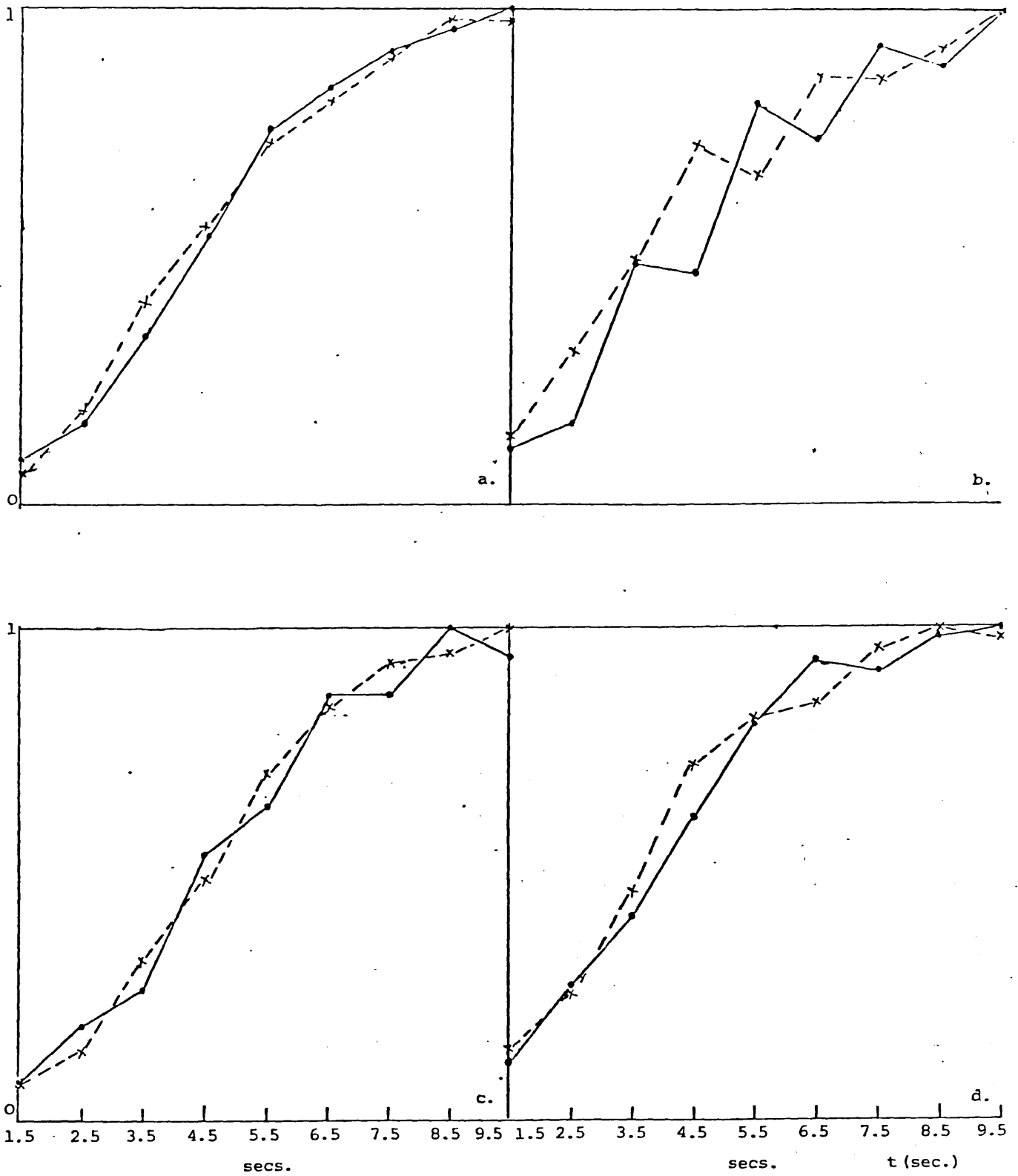


Figure 7.3 As figure 7.2, but for the merging manoeuvre.

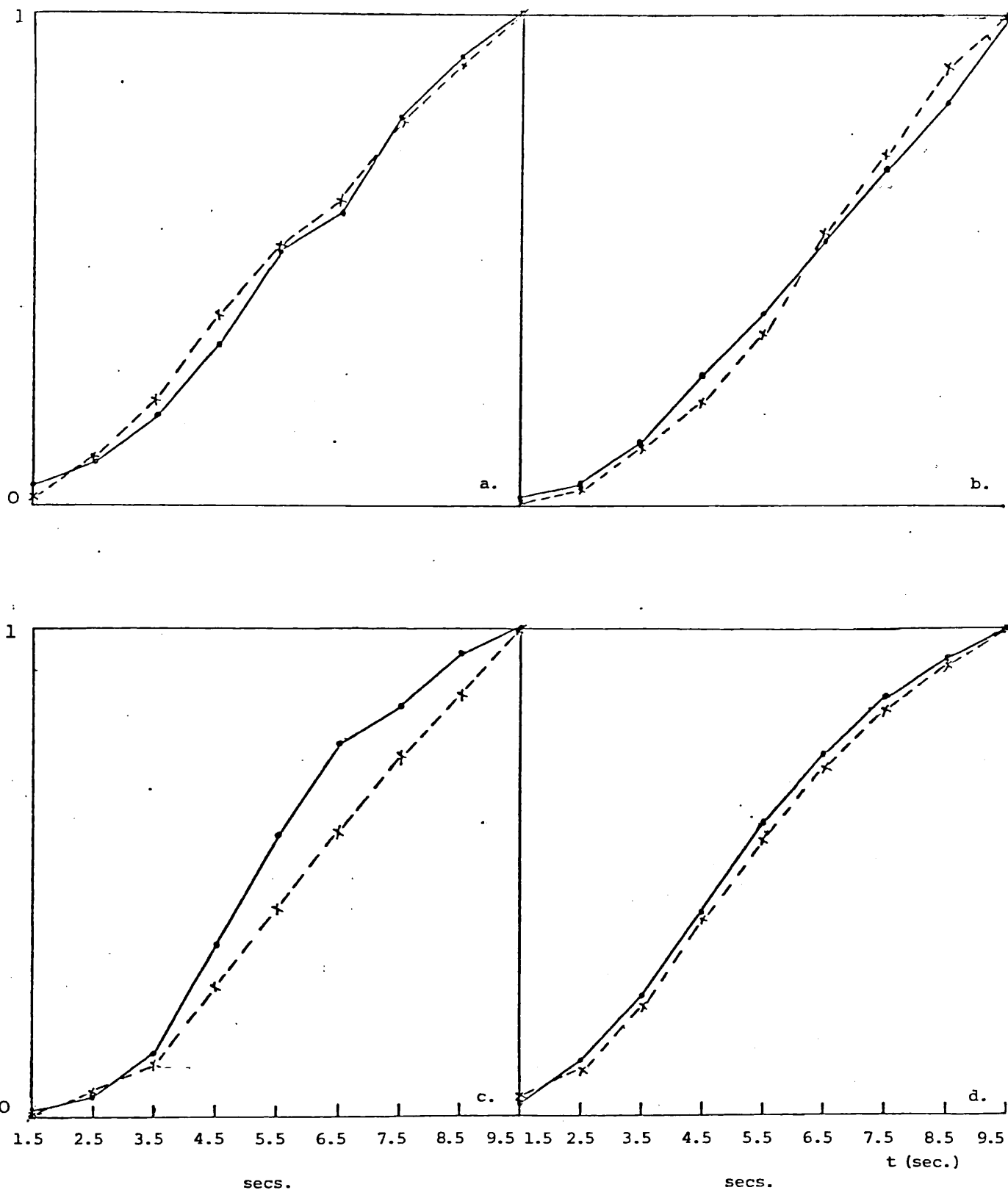


Figure 7.4 The standardised cumulative distributions of accepted gaps in wet (.—) and dry (x---) weather conditions for the crossing manoeuvre in daylight and darkness. The scale and units are as figure 7.2.

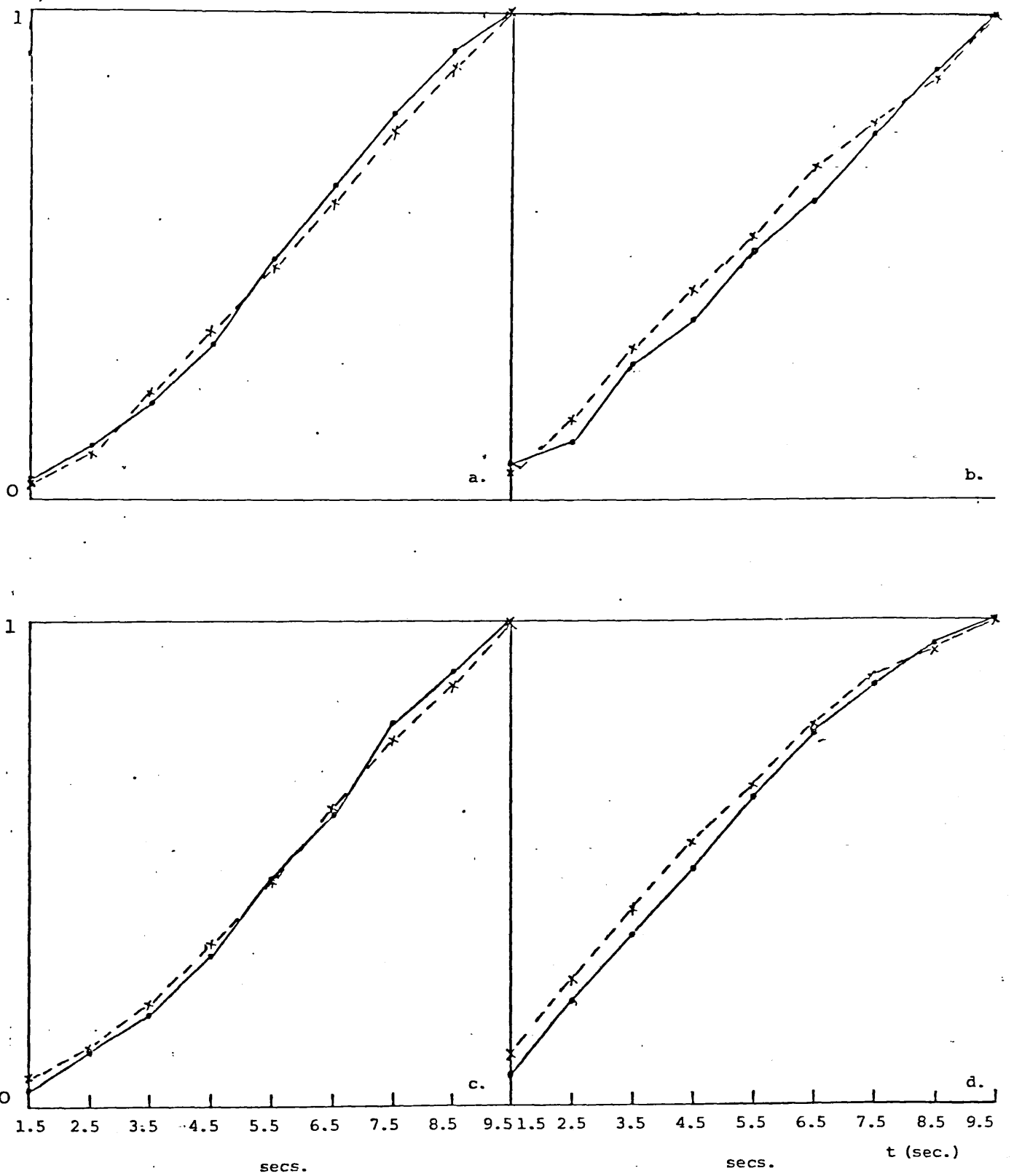


Figure 7.5 As figure 7.4, but for the merging manoeuvre.

APPENDIX 2

PUBLISHED MATERIAL

Aspects of headway distributions and platooning on major roads

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We follow our earlier work on the distribution of headways on trunk roads by a study of platooning. We find that platoons occur randomly, and that the distribution of platoon sizes can be closely represented by a modified geometric distribution. A study of autocorrelations between successive headways shows that they are uncorrelated. The distribution of headways in platoons is found to be independent of speed at a given site. The most important result is that almost all followers follow too closely in terms of the *Highway Code* advice, and this close following is relatively greater, and consequently more hazardous, at greater speeds.

1. Introduction. We have earlier¹ discussed time-headway distributions observed on a number of rural trunk roads in the U.K. In that paper we showed that the headways could be modelled as if the drivers belonged to one of three classes:

- (a) free movers;
- (b) joining or leaving a platoon; and
- (c) followers or members of a platoon.

This is in agreement with the earlier suggestion in Underwood's work², provided that his 'region of unstable flow' is interpreted as our category (b). The definition of platoon used in this paper is that given in Ovuworie *et al.*¹, and is restated in Section 4 below.

We consider first the autocorrelation between successive time headways in the main road streams; then the distribution of gaps between platoons (the gap between the last vehicle of a platoon and the leader of the subsequent one); the distribution of platoon sizes (number of vehicles in a platoon); and finally, the distribution of headways within platoons. The primary purpose of the investigation was to identify any autocorrelation in the time series of headways in order to use it in the sampling process based on the distributions derived earlier¹. However, information of more general interest was obtained, and is the main subject of this paper.

2. Data

We have used the same data as in Reference (1). For the Puttenham, Peasmarsh and Compton sites (see Reference (1)) we had recorded the speed of each main-road vehicle. We had sufficient data at each of four sites for separate analysis: the sample sizes are given in Table I. The speeds were

measured in terms of the time to cross two detectors separated by 88 inches, with a clock accurate to 0.01 seconds, and the speeds are therefore accurate to better than 5 mile/h at 40 mile/h. For work not involving a knowledge of the speeds we also used two data-sets taken at Tongham (see Reference (1)).

3. Autocorrelation in main-road streams

The previous work (Breiman *et al.*^{3,4}) refers to highways (motorways) in Detroit. Each lane was treated as a separate stream. In the first paper³, Breiman and his colleagues studied the underlying stochastic process of the sequence of headways (time) for freely-flowing traffic. An attempt was made to determine the validity of the hypothesis that successive headways are independent. They determined periods of time for which the rate of flow of vehicles was without trend. They tested the hypothesis of independence within these periods for eight-sets of data. Using a variety of tests, including correlogram analysis, they found that this hypothesis could be accepted at the 5 per cent level in all, except possibly one, of their eight sets of data.

Table I. Sample sizes for each site

	Sample size
Puttenham	548
Tongham A	578
Tongham B	752
Compton A	570
Compton B	976
Peasmarsh	570

In investigating the distribution of gaps in platoons larger samples were used and different runs from the same site combined.

They deduced that the headways followed a negative exponential distribution. This differs from our earlier result¹ that, for headways less than about two seconds, a more complicated model is required. They give no information on whether a significant number of such headways occurred in their sample.

In the second paper⁴, they reinvestigated the headway process in time and space. The hypothesis of independence in time and space was rejected only six times out of 48 sets of data at the 5 per cent level.

There was evidence of some correlation at the heaviest flow (> 1000 veh/h). Headways (both time and space) were described by a negative exponential distribution except for times less than 3.0 seconds and distances less than 300 ft. Vehicle speeds were normally distributed, and successive vehicles' speeds were highly correlated. In addition a small correlation (0.29) between speed and headway was found if the flow was greater than 1000 veh/h.

We applied a similar analysis to our data, which were taken on U.K. trunk roads, on which the opposing streams are not physically separated. In particular, the data were taken near intersections with minor roads, at rush hours, under which conditions overtaking is very rare.

We asked the question, is the j -th headway in a stream correlated with the $(j+k)$ -th, for $k = 1, 2, 3 \dots$? The answer is contained in the autocorrelation coefficients r_k . These are defined by

$$r_k = C_k/C_0, \dots k = 1, \dots, m < n \dots (1)$$

where

$$C_k = \frac{1}{n-k} \sum_{j=1}^{n-k} (x_j - \bar{x})(x_{j+k} - \bar{x}) \dots (2)$$

$$C_0 = \frac{1}{n} \sum_{j=1}^n (x_j - \bar{x})^2 = \text{Var}(x) \dots (3)$$

in which n is the sample size, \bar{x} the mean headway and x_j the headway to the following vehicle. The results for a number of streams for r_1, \dots, r_{10} are given in Table II. (The descriptors are PEA = PEASMARSH, TON = TONGHAM, PT = PUTTENHAM, COMP = COMPTON.) All the values are small, and consistent with there being no significant autocorrelation. If a time series is in fact random, then $\hat{r}_k \approx 0$, $k \neq 0$ and when n is large the coefficients r_k are approximately normally distributed⁵

$N\left(0, \frac{1}{n}\right)$. Therefore we can plot 95 per cent

Table II. Autocorrelation coefficients r_k ($k = 1, \dots, 10$) of time gaps for various data sets (see text)

Set	r_1	r_2	r_3	r_4	r_5	r_6	r_7	r_8	r_9	r_{10}
TON A	-0.29	-0.53	-0.58	-0.50	-0.26	-0.62	-0.15	-0.00	-0.14	-0.12
TON B	-0.143	-0.69	-0.35	-0.07	-0.21	-0.07	-0.57	-0.30	-0.43	-0.20
PT	-0.37	-0.10	-0.50	-0.12	-0.05	-0.97	-0.08	-0.27	-0.51	-0.14
COMP A	-0.26	-0.38	-0.41	-0.44	-0.51	-0.20	-0.04	-0.65	-0.19	-0.66
COMP B	-0.12	-0.45	-0.41	-0.34	-0.09	-0.08	-0.40	-0.10	-0.13	-0.55
PEAS	-0.31	-0.04	-0.22	-0.34	-0.14	-0.01	-0.35	-0.06	-0.21	-0.04

confidence limits on r_k as $\pm 2n^{-1/2}$; if the calculated values lie outside these limits, the series is likely to be non-random. A typical such correlogram is shown as Fig 1, and indicates that the headways are randomly distributed. Various other tests were applied which confirmed this result. Note that one cannot ask the same question about platoons, unless their mean size is large. We find, typically, a mean size of 3, which is much too small to allow an investigation of correlation within platoons.

Our result is in agreement with that of Breiman *et al.*⁴ and establishes that for single streams on both trunk roads and motorways the time gaps are independently distributed. However, Breiman's work leaves a question about small gaps, which we address later.

4. Time headways between platoons

We have earlier defined, somewhat arbitrarily, what we mean by a platoon¹. A platoon occurs if the leading vehicle, B_0 , is at least T_i seconds behind its predecessor, and the platoon consists of a stream $B_0 B_1 B_2 \dots B_{n-1} B_n$ in which no headway $Z_k = t(B_k) - t(B_{k-1})$, $k = 1, \dots, n$ is greater than T_i , where T_i is a location dependent measured parameter for the i -th site. The time headway between successive platoons P and P' is $t(B'_0) - t(B_n)$; we find that at two of the three sites (Compton and Peasmarsh) these time headways follow approximately a negative exponential distribution, though the Puttenham data suggest a relatively high frequency of small headways.

5. Distribution of platoon sizes

Each platoon comprises a leader and followers. Vehicles in classes (a) and (b) (see Section 1) are not considered to be a platoon. Platoons are referred to as intermediate or large if of sizes 2-4 or ≥ 5 respectively.

The mean platoon size was close to 3 for all the data sets examined. Two distributions were fitted to the platoon sizes (that is, the number of vehicles in a platoon) and a comparison between them made. The two distributions, which are widely used (see, e.g.^{6,9}), for the platoon sizes are:

(i) the Geometric distribution,

$$p_r = (1 - \alpha)\alpha^{r-1}$$

(ii) the Borel-Tanner distribution,

$$p_r = \frac{(r\alpha e^{-\alpha})^{r-1} e^{-\alpha}}{r!}$$

where r is the platoon size, and $\alpha = 1 - \bar{r}^{-1}$ is the maximum likelihood estimator.

We define modified distributions, which do not include $r = 1$ terms as follows:

(iii) modified Geometric distribution

$$p_r = (1 - \alpha)\alpha^{r-2}$$

where the maximum likelihood estimator of α is $(\bar{r} - 2)/(\bar{r} - 1)$.

(iv) modified Borel-Tanner,

$$p_r = \frac{(r\alpha e^{-\alpha})^{r-1} e^{-\alpha}}{r!(1 - e^{-\alpha})}$$

where in this case the maximum likelihood estimator of α is obtained from the equation

$$(\bar{r} - 1)/\alpha - \bar{r} + 1/(e^\alpha - 1) = 0.$$

The observed and the expected frequencies (f_r) of platoon sizes r , the latter being derived by (iii), together with the total number of platoons and the mean platoon size $\bar{r} = \frac{\sum r f_r}{\sum f_r}$ are given in Table III for three sets of data.

The modified Geometric distribution gives an excellent description of the data in each case. The χ^2 test shows (Table IV) that there is no significant difference between the observed and expected frequencies of platoon sizes.

Other authors have used the unmodified Geometric and Borel-Tanner distributions to describe data sets which include our classes (a) and (b) as 'platoons' of size one. These are inappropriate in our case. We note that in any case they give much poorer fits. The modified Borel-Tanner distribution also fails to describe our data. Miller's

one-parameter⁸ distribution of platoon sizes has not been fitted since the mean platoon size for our data is close to 3, and therefore too large for this approach, while his two-parameter distribution was not fitted as it is unnecessarily complicated compared with the Geometric.

6. Distribution of time headways within platoons

In the previous study¹, we showed that the distribution of time headways of followers may be described by a truncated normal distribution. This suggests the hypothesis that at a given flow drivers in platoons appear to maintain a constant headway, modified in practice by other factors such as skill. A further analysis of these time headways within platoons was therefore made to attempt to elucidate this behaviour.

The mean speed of vehicles in a platoon was calculated and the platoons were classified into 5 mile/h speed (mean) bands, for each site separately. For each speed class, with sufficient sample size, the time headways for all the platoons of the class were classified into 0.2-second intervals. The mean and standard deviations for each speed class are shown in Table V. A truncated Normal distribution was fitted to each class using a standard minimization routine. A χ^2 -test was used to test the quality of the fit. Table V gives the results for three sets of data. In all cases the hypothesis that the time headways follow a truncated Normal distribution for each speed class could not be rejected at the 5 per cent level of significance except for the third set (Compton) for 35-40 mile/h class. Table V also shows that the mean time headways and the standard deviations of each speed class were almost equal for the same site. Bartlett's test showed no significant differences between the

Table IV. χ^2 -table

Site	Geom	d.f
Puttenham	1.71	4
Compton	3.13	3
Peasmarsh	3.03	3

Table III. Observed and expected frequencies of platoon sizes r ($r > 1$), calculated in the Geometric model at various sites

	r	Obs.	Fcalc (Geom)	
Flow 545 veh/h $N = 124$ $\bar{r} = 3.2$	2	58	56.42	PUTT. ($T \leq 2.0$)
	3	29	30.74	
	4	13	16.75	
	5	11	9.13	
	6	6	4.97	
	>6	7	6.07	
Flow 925 veh/h $N = 189$ $\bar{r} = 2.8$	2	107	105.08	COMP. ($T \leq 1.8$)
	3	38	46.68	
	4	23	20.60	
	5	12	9.07	
	>5	9	7.56	
Flow 425 veh/h $N = 171$ $\bar{r} = 2.7$	2	100	100.71	PEAS. ($T \leq 2.0$)
	3	47	41.39	
	4	11	16.90	
	5	7	6.99	
	>5	6	4.55	

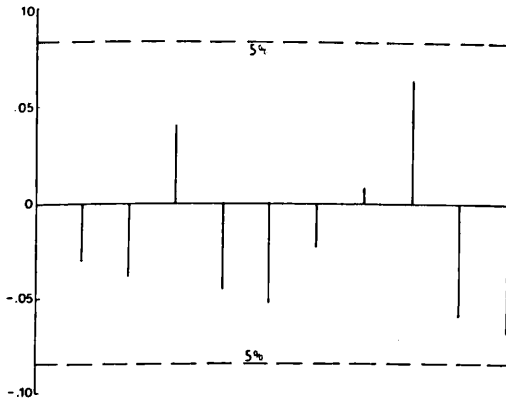


Fig 1. Autocorrelogram for time headways at Compton (data-set A).

variances of the speed classes for each site. The values of χ^2 were not significant at the 5 per cent level (not shown).

A one-way analysis of variance was applied for each site. For all sets of data the hypothesis that the mean time headways were equal, i.e.

$$H_0: \bar{x}_1^{(i)} = \bar{x}_2^{(i)} = \bar{x}_3^{(i)} = \bar{x}_4^{(i)}$$

where $i = 1, 2, 3$ represent the site, and \bar{x}_j , $j = 1, 2, 3, 4$ are the mean time headways for speed class j , cannot be rejected at the 5 per cent level.

Table V.

Mean (mile/h) platoon speed	Sample sizes	Mean	S.d	χ^2	d.f	
20-25	—	—	—	—	—	—
25-30	199	1.21	.39	6.34	6	PUTT. ($T \leq 2.0$)
30-35	303	1.20	.38	10.47	6	12.59
35-40	167	1.22	.40	4.70	6	5 per cent
40-45	45	1.24	.36	—	—	significance level
20-25	68	1.32	.35	1.55	6	—
25-30	133	1.34	.41	3.09	6	—
30-35	194	1.31	.40	8.54	6	PEAS. ($T \leq 2.0$)
35-40	122	1.35	.40	9.20	6	—
40-45	—	—	—	—	—	—
20-25	—	—	—	—	—	—
25-30	—	—	—	—	—	COMP. ($T \leq 1.8$)
30-35	124	1.06	.36	4.56	5	—
35-40	237	1.07	.36	13.89	5	11.07
40-45	208	1.08	.38	6.09	5	5 per cent significance level
45-50	53	1.17	.34	3.05	5	level

Table VI. Proportion of followers as a function of speed

Speed ft/sec	Compton	Peasmarsh	Puttenham
25-30	.208	.512	.833
30-35	.285	.446	.807
35-40	.555	.489	.651
40-45	.333	.397	.656
45-50	.651	.437	.747
50-55	.523	.380	.671
55-60	.473	.250	.590
60-65	.469	.260	.393
65-70	.355	.208	.274
70-75	.440	.185	—

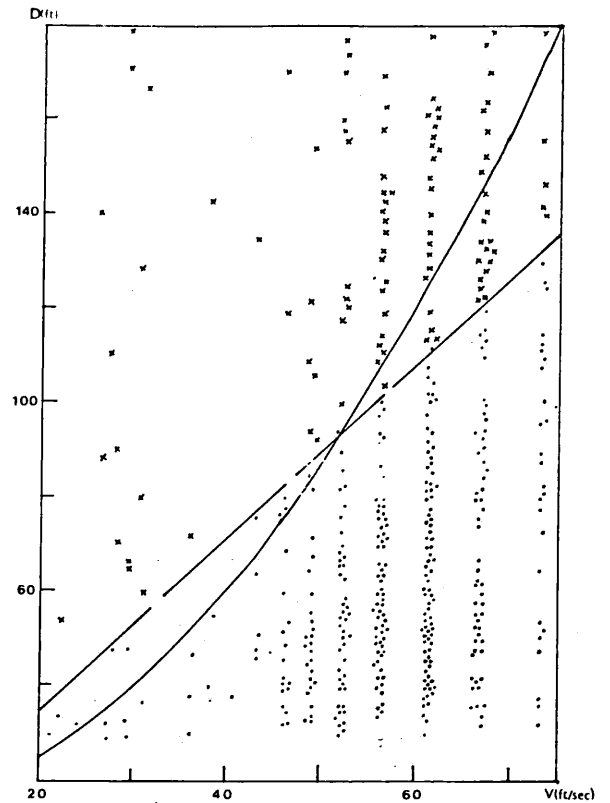


Fig 2. The observed distance gaps at Compton classified by speed bands (ft/sec.) for all vehicles. Those vehicles below the dashed line are members of platoons ($T \leq 1.8$ sec.). The curve is the 'Highway Code' minimum stopping distance (D_{HC}). The data appear banded as the timing is uncertain to ± 0.02 sec., thus, for example, any transit time between 0.07 and 0.11 sec. is recorded as 0.09 sec. (\times Free movers and others \cdot Followers).

We conclude that, at a given site, a truncated Normal distribution can describe the time headways within each speed class, and that the mean and standard deviation are independent of speed. That is, provided our definition of a platoon holds, the distribution of headways within platoons, at a specific site, is independent of speed.

To understand further the above results the distance headways are considered and an investigation of the relationship between distance headway and speed is made, both for vehicles within platoons and for all vehicles (see Figs 2, 3 and 4).

Distance headways were estimated from time headway and speed, for each vehicle, in ft/sec. These headways are plotted according to speed and are shown in Figs 2, 3 and 4 for these sets of data, where dots indicate vehicles within platoons and crosses those not in a platoon. The curve D_{HC} is the highway code curve representing the minimum stopping distances according to the speed. These figures show that almost all drivers with $T \leq 2$ ($T \leq 1.8$ for Compton, where T time headway) are following too closely for, say, $30 \leq V \leq 65$ ft/sec. In effect, our definition of platoon closely corresponds to the set of vehicles which are following too closely.

The proportion of followers (i.e. second and subsequent members of platoons) (see Section 1) as a function of speed at each site, is given in Table VI, and in general decreases with increasing speed (PEA, PUT), except COMP, where the low speed data is sparse. We might suppose that the mean distance headway for each speed band would be very close to the Highway Code recommendation $D_{HC}(v)$. However, since the proportions of vehicles with distance-headways below the

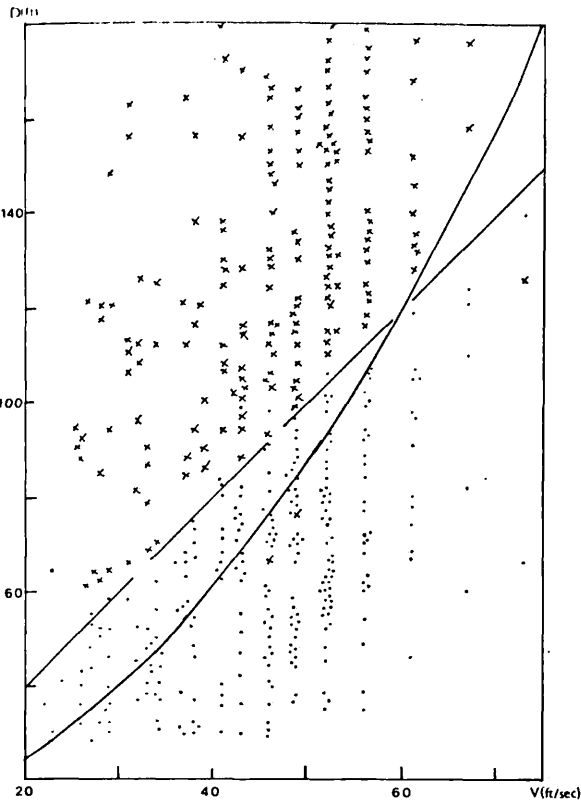


Fig 3. As Fig 2, but for Peasmarsh with $T \leq 2.0$.

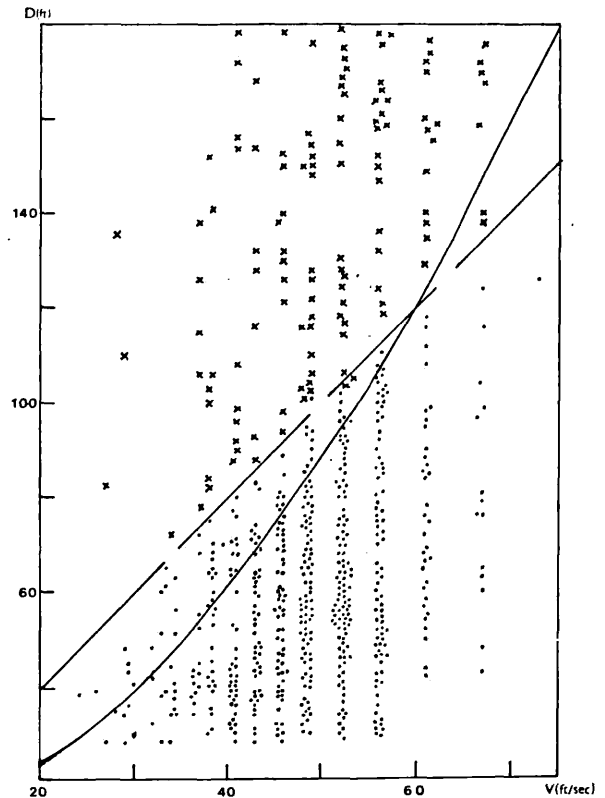


Fig 4. As Fig 2, but for Puttenham with $T \leq 2.0$.

D_{HC} curve are relatively high, we calculated for each 5 ft/sec. band the ratio of the mean distance-headway of those vehicles below the D_{HC} curve to the value of D_{HC} corresponding to the centre of that band.

Table VII shows that this ratio tends to decrease as the speed increases. That is, as the speed increases, the amount by which drivers in platoons follow too closely increases. This indicates that faster drivers are more in danger according to the *Highway Code* standards.

We show in Table VIII that, with the exception of Peasmarsh, more than 20 per cent of all drivers follow at half or less of the recommended separation, and of those in platoons, more than 30 per cent. Further (Table VIII) at all sites more than 20 per cent of all drivers follow at less than three-quarters of the recommended separation, and of those in platoons at least 50 per cent, this figure being as high as 86 per cent at Compton. Further, from our previous analysis, Compton has the highest risk of conflict¹⁰.

The most important result of the investigation is to demonstrate that almost all followers follow too closely, in terms of *Highway Code* advice, and the amount by which they do so increases with speed, while the proportion following at less than three-quarters of the recommended distance can be as high as 45 per cent.

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Table VII. Ratio of the mean distance-headway (D_b) of vehicles with headways less than the *Highway Code* curve (D_{HC}) to that value, $r = D_b/D_{HC}$, as a function of speed. The value of D_{HC} is taken at the centre of the speed band. Results are given for three sites

Speed ft/sec.	Compton	Peasmarsh	Puttenham
25-30	.871	.957	.942
30-35	.863	.900	.840
35-40	.722	.703	.738
40-45	.643	.696	.684
45-50	.568	.724	.718
50-55	.607	.696	.655
55-60	.607	.692	.623
60-65	.537	.663	.592
65-70	.560	.671	.595
70-75	.585	.670	—

Table VIII.

Site	Proportion of followers with Gap \leq H.C. Gap	Proportion of followers with Gap $\leq \frac{3}{4}$ H.C. Gap	Proportion of followers with Gap $\leq \frac{1}{2}$ H.C. Gap
Compton	.454	.390 (.859)*	.208 (.459)
Puttenham	.628	.455 (.724)	.203 (.323)
Peasmarsh	.376	.204 (.542)	.066 (.175)

() * indicates the proportion of drivers out of the total number of drivers in platoons
H.C. is the *Highway Code* recommendation

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Deceleration of major-road vehicles approaching a T-junction

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Abstract

The decelerations of oncoming nearside major-road vehicles have been observed at three rural T-junctions.

Our results showed that the behaviour of drivers in a major road approaching an intersection was affected in terms of deceleration by: the junction being unoccupied (case (a)); vehicles waiting to cross (case (b)); and vehicles actually crossing (case (c)). A Normal distribution described our data in case (a), although some outliers were observed. The data in case (a) were used to deconvolute the true decelerations due to the presence of other crossing vehicles from the observations. The deconvoluted results in cases (b) and (c) were well described by a Normal and a Truncated Normal distribution, respectively. We also found that major-road drivers decelerated much more in case (c) than in case (b). In case (b) between 20 and 27 per cent of drivers showed negative decelerations, but in case (c) 94 per cent of drivers decelerated positively.