DRIVER BEHAVIOUR AT NON-URBAN T-JUNCTIONS

by

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ABSTRACT

The accident situation in Great Britain is reviewed, and the process of gap acceptance is described. Studies of gap acceptance behaviour at intersections are discussed according to the factors which were investigated, and some of the literature on pedestrian gap acceptance is reviewed. The use of the traffic conflicts technique in this country is also outlined.

Two different methods of collecting data at junctions are described and compared, and the advantages of a system which incorporates both video techniques and a microprocessor-based system developed at RHC are detailed. Empirical results on the relationships between gap acceptance parameters and various factors are presented: the effects of major road speeds and flow, manoeuvre time of turning vehicles, class of vehicle in each traffic stream, gender of driver and presence of passengers are investigated. Two models of gap acceptance by queues of vehicles are then discussed.

Preliminary results from a large scale study to test the validity of a conflict simulation model developed at RHC are reported, which indicate that the model may be used to compare accident risk in different situations. Some examples of the ways in which safety at junctions varies with particular parameters are given, using results from the model. Details of the computer programs used to analyse gap acceptance data from T-junctions are appended.

ACKNOWLEDGEMENTS

The work described in this thesis was undertaken while the author was a member of a research group at Royal Holloway College, and most of the results presented here represent the joint effort of the members of that group. In particular, the guidance and advice of Dr. Dale Cooper (now at Southampton University) is gratefully acknowledged. Much of the data collection and analysis was done in collaboration with Mr. Peter Storr, and further results of the work conducted at RHC appear in his thesis (Storr, 1980).

The observational studies described in this thesis were conducted with the assistance (at various times) of students at Royal Holloway College; the Surrey Constabulary and the Sussex Constabulary; the County Engineers Departments of Surrey, Berkshire and Buckinghamshire; and the Civil Systems Research Unit of the Plessey Company.

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CHAPTER ONE

GENERAL BACKGROUND AND DEFINITIONS

Introduction

The Operational Research Group at Royal Holloway College (RHC) has been concerned with studies of driver behaviour at non-urban T-junctions for several years. This work was initiated as part of Project 2001 of the Home Office PSDB, which investigated the allocation of traffic police resources in non-urban situations. One of RHC's main contributions to this project was the development of a conflict simulation model of a T-junction, and the evaluation of the effectiveness of various police tactics using this model (Cooper and McDowell, 1977). The major part of our research, however, has investigated the gap acceptance behaviour of turning drivers at junctions, and the implications of this behaviour for road safety; this aspect of the research is the subject of this thesis.

Road safety is a major concern in modern life. Traffic accidents and their consequences affect many areas of society, from the medical and hospital system, the police and the emergency services, to industry and local government. In Great Britain alone, nearly a quarter of a million injury accidents are reported to the police each year (Road Accidents Great Britain 1976). The cost to individuals and to society has been estimated at over

£800 million, or about a quarter of the capital expenditure on vehicles, roads and associated parts of the road transport system.

The road traffic system contains a number of interacting elements, all of which can affect the occurrence and severity of accidents (Wennell and Cooper, 1977). These elements may be grouped into three main components: the road user, the vehicle and the road environment. The roles of these three elements in accidents are discussed by Sabey and Staughton (1975), using the results of a detailed study of more than 2000 traffic accidents conducted by TRRL.

This study spanned a period of four years, during which time an accident investigation team on call throughout the day and night attended every reported accident which occurred within 20 Km of the Laboratory. The team made detailed assessments of each accident, and identified the various factors which contributed to the cause of the accident. Overall results are shown in Table 1.1; road user factors are the most important single item, with vehicle and road environment factors more likely to appear in combination. The interaction between the road user and his surroundings is particularly large. Sabey and Staughton (1975) conclude that:

> "Human factors contribute to nearly 95 per cent of accidents, and are the sole contributor in 65 per cent. The human errors which play the largest part in accidents are those which can be attributed to failings such as carelessness, lack of concentration, misjudgement and inexperience."

<table>
<tr><th></th><th>ROAD ENVIRONMENT</th><th>ROAD USER</th><th>VEHICLE</th></tr>
<tr><td>Single factors</td><td>2.5</td><td>65</td><td>2.5</td></tr>
<tr><td>Double factors</td><td colspan="2">24</td><td colspan="2">4.5</td></tr>
<tr><td>Treble factors</td><td></td><td>1.25</td><td></td></tr>
<tr><td>Double factors</td><td></td><td>0.25</td><td></td></tr>
<tr><td>Total contribution</td><td>28</td><td>94.75</td><td>8.5</td></tr>
</table>

Table 1.1 Percentage contributions to accidents of road environment, road user and vehicle (reproduced from Sabey and Staughton, 1975).

Accidents are not distributed uniformly within the road network. Over three quarters of all accidents occur on roads subject to 30 or 40 mph speed limits, and more than sixty per cent of these occur at junctions; however, less than thirty per cent of accidents in non built-up areas occur at junctions.

The majority of accidents which occur at junctions involve turning vehicles, and so gap acceptance behaviour is an important aspect of road safety research. When turning drivers make poor gap acceptance decisions, priority vehicles may be forced to take evasive action in order to avoid collision; if such action is not successful, an accident occurs. Clearly some effort is required to investigate ways in which gap acceptance behaviour can be affected, so that the accident situation at junctions may be improved.

Gap acceptance is a very complex process, which may be influenced by many factors. In particular, characteristics of the vehicles and their drivers are important (such factors are considered in later chapters). Environmental factors like sight distance may also affect gap acceptance behaviour, but the influence of such factors is rather difficult to investigate.

The remainder of this chapter is concerned with a description of the gap acceptance process, and the various models of behaviour which have been proposed. The derivation of a population gap acceptance function from empirical data is detailed, and the relationship between gap acceptance and safety is discussed. An outline of the contents of the thesis is then given.

GAP ACCEPTANCE

Consider the driver of vehicle A in Figure 1.1, who wishes to turn right from the major road into the minor road of a standard T-junction (we shall assume that this driver is male). He must give way to oncoming vehicles B and C in the major road, who have priority. We define the time interval between his arrival at the junction and the arrival of the first major road vehicle as a <u>lag</u>, while the interval between arrivals of consecutive vehicles in the oncoming stream of major road traffic is a <u>gap</u>. If the driver turns immediately he reaches the junction, we say he has <u>accepted</u> the lag; otherwise he <u>rejects</u> it. He may then reject several gaps before eventually turning, when he accepts a gap.

The behaviour of this driver may be represented by a <u>gap acceptance function</u> F(t), which gives the probability that a gap of t seconds will be accepted. A simple model of the behaviour of an individual driver was proposed by Tanner (1962), which stated that all gaps longer than some criterion C (the <u>critical gap</u>) would be accepted, and all gaps shorter than C would be rejected ie his gap acceptance behaviour could be represented by the step function shown in Figure 1.2.

Empirical studies (see, for example, Bottom and Ashworth (1978)) have shown that individual drivers are <u>not</u> consistent in behaviour, and so may reject a gap of a particular size, and later accept a gap which is shorter than that previously rejected. However, the probability of an individual driver accepting a gap of a given size is simply related to the size of that gap: for some particular

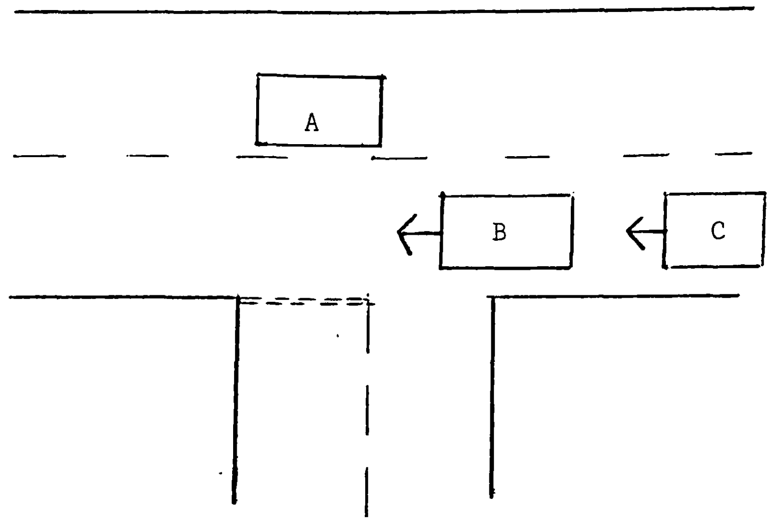

Figure 1.1 A standard T-junction. Driver of vehicle A wishes to turn into the minor road, but must give way to vehicles B and C.

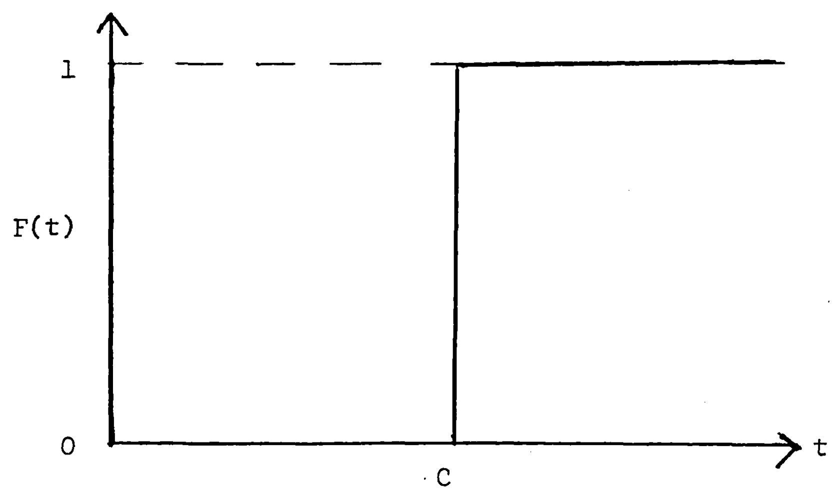

Figure 1.2 Gap acceptance function proposed by Tanner. C is the critical gap.

values T_1 and T_2, the driver always rejects gaps less than T_1 seconds and always accepts gaps greater than T_2 seconds; between T_1 and T_2, the probability of acceptance is linearly related to the logarithm of the time gap (Herman and Weiss, 1961). The values of T_1 and T_2 differ for each driver, and the magnitude of the slope of the linear relationship is a measure of the driver's consistency (Figure 1.3).

Population gap acceptance behaviour

The gap acceptance behaviour of a population of drivers may be represented in various levels of complexity (Blumenfeld and Weiss, 1979). The simplest model, which was proposed by Tanner (1962), assumed that every driver had the same fixed critical gap C, and so a population's behaviour could be represented by the step function shown in Figure 1.2. A more realistic view was discussed by Ashworth (1970), who suggested that each driver had a fixed critical gap, but that this critical gap varied from driver to driver; consequently, gap acceptance behaviour could be represented by a distribution of critical gaps. In practice, individual drivers exhibit the type of behaviour shown in Figure 1.3, but the form of this gap acceptance function varies from driver to driver.

Ebbesen and Haney (1973) discuss the form of the population gap acceptance function appropriate to the case where an individual driver's behaviour may be represented by a linear relationship, and its slope varies from driver to driver. They state that if such slopes are normally distributed among the population, averaging the gap acceptance functions of a number of individual subjects yields a normal ogive ie for a random population of individual drivers, gap acceptance behaviour may be represented by the

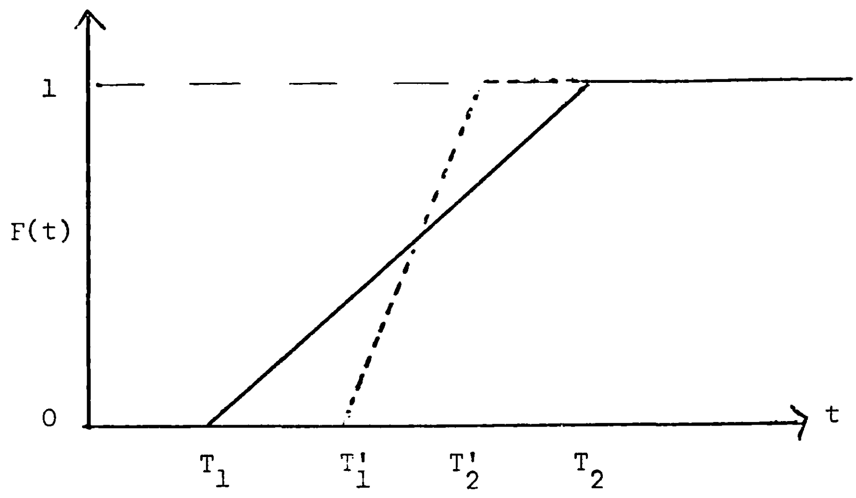

Figure 1.3 Gap acceptance functions for two different individual drivers. Driver represented by the broken line is more consistent in behaviour.

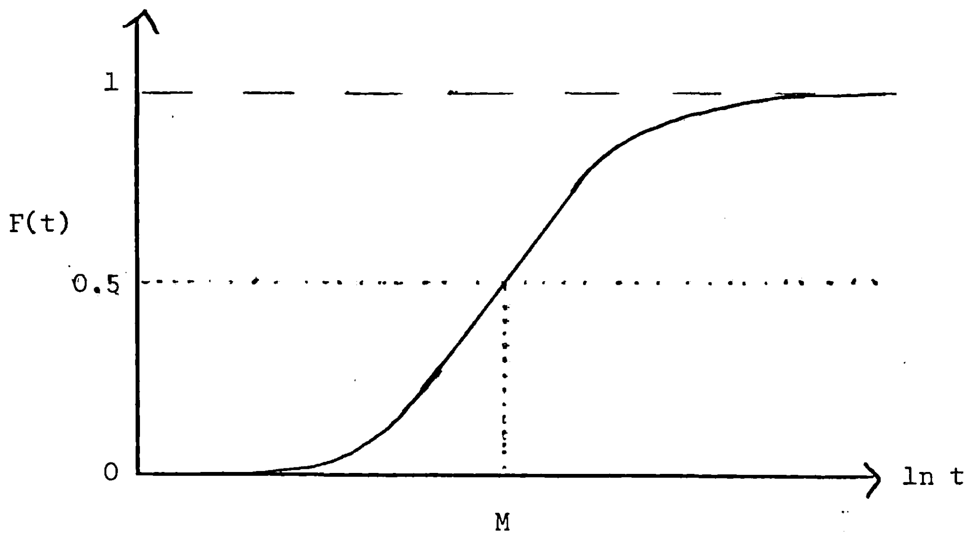

Figure 1.4 Population gap acceptance function. M is the median accepted gap, defined by F(M) = 0.5.

cumulative Normal distribution depicted in Figure 1.4. Justification for the use of a logarithmic time axis is discussed in detail by Miller and Pretty (1968). Such a function seems to fit empirical data well (Solberg and Oppenlander, 1966; Ebbesen and Haney, 1973; Cooper, 1976), and is the form of the gap acceptance function to be used in this thesis.

Derivation of gap acceptance function

This thesis is mainly concerned with the gap acceptance behaviour of drivers performing two simple turning movements at a T-junction (Figure 1.5), in which only the nearside stream of priority traffic need be considered. In order to measure the gap acceptance function which describes the behaviour of these drivers, we must observe the sizes of the lags and gaps that they accept and reject. N.B. Unless otherwise stated, lags and gaps will be referred to jointly as gaps from now on, and all gaps will be measured in seconds.

The results of our observations may be given in the form of the gap acceptance table shown in Table 1.2. This table gives the number of accepted gaps and the number of rejected gaps observed in a particular range of gap sizes (we typically use half-second intervals). The number of presented gaps in each interval is obtained by summing the numbers of acceptances and rejections. The probability of accepting a gap of a given size is then calculated by dividing the number of acceptances by the number of opportunities (presented gaps). Finally, a log-normal gap acceptance function is fitted to this probability distribution using the technique of probit analysis (Appendix 1).

This method has been used before (Bottom and Ashworth, 1978; Cooper, 1976), and it is appropriate to the situation where

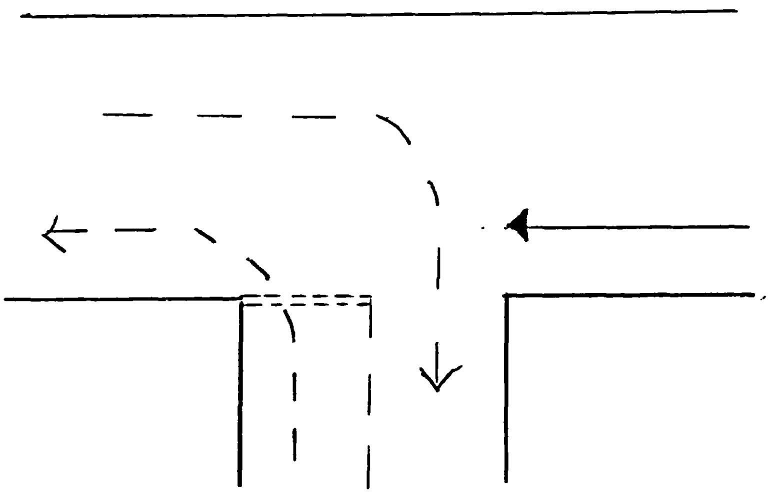

Figure 1.5 Simple gap acceptance manoeuvres at a T-junction, in which turning vehicles yield right-of-way to one stream of priority traffic.

GAP SIZE INTERVAL	NUMBER OF REJECTIONS	NUMBER OF ACCEPTANCES
1.00 - 1.49	100	0
1.50 - 1.99	90	0
2.00 - 2.49	65	5
⋮		
5.00 - 5.49	20	20
⋮		
9.00 - 9.49	0	20
9.50 - 9.99	0	15

Table 1.2 A typical gap acceptance table.

subjects are confronted with a series of stimuli (gaps) to which they make quantal responses (acceptance or rejection). Two parameters of the gap acceptance function are used as our measures of driver behaviour:

i) the median accepted gap, which is the gap corresponding to a 50% probability of acceptance (Figure 1.4);

ii) a variability parameter, which is derived from the slope of the gap acceptance function, and is a measure of the variability in behaviour.

The formulae used to calculate these parameters are given in Appendix 1.

The behaviour of drivers turning right out of the minor road at a T-junction can not be represented by the simple gap acceptance functions derived above. Such drivers must consider two priority streams of traffic, and hence accept or reject a double gap, which consists of a gap in the nearside stream and a gap in the farside stream of traffic. Ways in which the gap acceptance behaviour of these drivers may be represented are discussed by Storr, Cooper and McDowell (1980) and Storr (1980).

Gap acceptance and safety

Each turning manoeuvre at a T-junction takes a certain amount of time to complete. The time needed to carry out a particular manoeuvre depends on the acceleration capability of a vehicle, which in turn depends on the length, weight and engine capacity of the vehicle. In order to complete a turning manoeuvre safely, the size of the gap which the driver accepts should be longer than the time needed to carry out the manoeuvre. If a driver turns and accepts a gap which is shorter than the time required for the turn, a potential accident situation occurs, and the oncoming major road vehicle must take evasive action such as braking or swerving to avoid a collision. Such situations are defined as traffic conflicts, and very serious conflict situations result in accidents.

One of the reasons we collect gap acceptance data is to evaluate the behaviour of turning drivers in terms of conflict situations, using a simulation model. In our modelling work we represent the behaviour of a population of drivers by a gap acceptance function fitted to empirical data, and assume that all members of the population behave in a similar way. This conflict simulation model is described in Chapter 8, and some examples of its results are given in Chapter 9.

OUTLINE OF THESIS

This thesis is mainly concerned with the identification of various factors which may affect gap acceptance behaviour, so that means of improving the behaviour of turning drivers may be suggested. Such improvements may lead to a reduction in the delays incurred by minor road drivers, or to a reduction in the number of accidents which occur at junctions.

In the next chapter, some of the gap acceptance studies reported in the literature are reviewed. Papers are dealt with according to the factors that were investigated, and areas in which further research may be necessary to clarify the results of earlier work are identified. Pedestrian gap acceptance is considered separately, and the development of the traffic conflict technique in this country from an original approach in the USA is outlined.

In order to study gap acceptance behaviour in detail, a vast amount of data must be collected. Two different methods of collecting this data are described and compared in Chapter 3, and the advantages of a system which combines both methods are discussed. The flexibility of each system is demonstrated in the following chapters, where the effects of various factors on gap acceptance behaviour are evaluated. Without the capacity for recording several different details about the vehicles involved in the gap acceptance process, such results would not have been possible.

The factors which are considered in Chapters 4, 5 and 6 are: the speed of approaching major road vehicles; major road flow; manoeuvre time of turning vehicles; class of vehicle turning or approaching; gender of driver of turning vehicle; presence of passengers in turning vehicle. All these factors affect gap acceptance behaviour in some way, which indicates the complexity of the gap acceptance process. Two models of gap acceptance by queues of turning vehicles are presented in Chapter 7, and the advantages of an explanatory model are highlighted.

Chapter 8 is concerned with the validation of a conflict simulation model developed at Royal Holloway College. Although the work reported in this chapter only covers the preliminary results of the validation study (which will probably take several years to complete satisfactorily), it indicates that the model may be used confidently to evaluate our gap acceptance results in terms of accident risk. Some examples of the ways in which safety at a junction varies with particular parameters are given in Chapter 9.

A summary of the contents of this thesis is given in Chapter 10, where our major conclusions are also presented. Several subject areas which would benefit from more detailed research are listed at the end of the chapter.

Details of the computer programs used to analyse our gap acceptance data are given in the appendices. Descriptions of experimental sites and summary tables of the results used for the validation study are also appended. Finally, copies of papers published jointly with other members of the research group are included as Appendix 6.

CHAPTER TWO

REVIEW OF LITERATURE

Introduction

This chapter reviews some of the literature on gap acceptance which is relevant to this thesis. The majority of papers on this subject are concerned with capacity or delay, and an extensive review of these papers appears in Ashworth (1969). Further useful references are contained in the theses of Bottom (1975) and Glen (1976).

We are mainly concerned with results of studies of gap acceptance behaviour at intersections, so the major part of this chapter is devoted to papers which investigate factors which affect gap acceptance behaviour. Studies of pedestrian gap acceptance are also reviewed, and research using the traffic conflicts technique is discussed in the final section.

GAP ACCEPTANCE STUDIES AT INTERSECTIONS

Effect of manoeuvre

Early research into gap acceptance behaviour at intersections only considered the size of the median accepted gap, and investigated whether this varied with manoeuvre. The majority of these studies were conducted at four-way unsignalised intersections in urban areas, where the three manoeuvres of vehicles emerging from the minor road may be observed in a similar environment. We refer to these manoeuvres as straight on, merging and cross-and-merge (Figure 2.1).

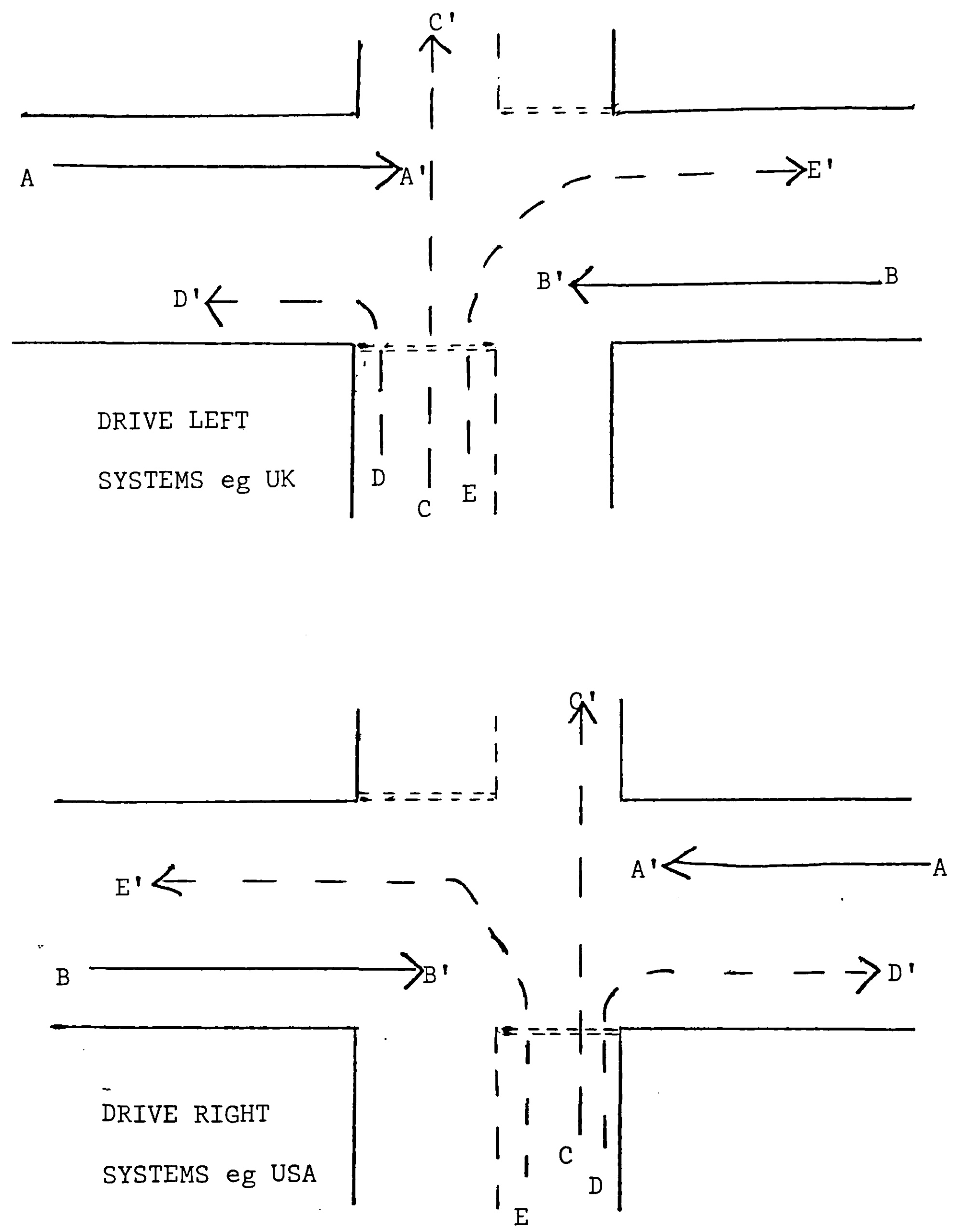

Figure 2.1 Three manoeuvres of vehicles emerging from the minor road at a four-way intersection. Streams AA' and BB' have priority; CC' is "straight on", DD' is "merging" and EE' is "cross-and-merge".

Blunden, Clissold and Fisher (1962) studied behaviour at a crossroads site in Sydney (Australia). They found that the nearside median accepted gap varied according to manoeuvre, but that the farside median accepted gap was the same for straight on and cross-and-merge manoeuvres. Solberg and Oppenlander (1966) observed behaviour at two stop-sign controlled crossroads in Indiana (USA) using time lapse photography. At one of the sites, they found that median accepted gaps were statistically the same for all manoeuvres, while at the other site, there were significant differences between merging and cross-and-merge, and between cross-and-merge and straight on.

Wagner (1966) collected data at a stop-signed intersection in Michigan (USA) using a multiple pen event recorder. He reported no significant differences in the gap acceptance distributions for each manoeuvre. Ashton (1971) studied behaviour at a crossroads where a minor road intersected a one-way major road in Dunedin (New Zealand). She found that the size of accepted gap for merging was less than that for straight on, and that the accepted gap for cross-and-merge was greater than that for straight on.

Ashworth and Green (1966) report observations at the intersection of a minor road with one side of a dual carriageway (UK). Timings were taken with a stopwatch, and only the first vehicle accepting a gap was included in the sample. They found no appreciable difference between the gap size accepted by merging and straight on vehicles.

These results are by no means consistent, and the different methods of observation and analysis used in each study make comparisons difficult. However, one would expect the accepted gaps for these manoeuvres to vary, since the time required to complete each manoeuvre varies. Results of our own studies show that the median accepted gap for merging vehicles is generally longer than that for crossing vehicles at the same T-junction (Table 15.3).

Difference between lag acceptance and gap acceptance

Solberg and Oppenlander (1966) found no significant difference between median accepted lag and median accepted gap whereas Wagner (1966) showed that accepted lags are generally shorter than accepted gaps. In a study of merging behaviour at six freeway entrance ramps in the USA, Drew (1967) found that the critical gap for vehicles which accepted gaps was about 20% higher than that for vehicles accepting lags. The combination of lag and gap acceptance data is discussed by Ashworth (1970).

Bias in gap acceptance functions

Gap acceptance functions derived from observations of drivers are subject to a bias related to the flow in the major road. In general, each driver contributes several rejected gaps to the observational data, but only one accepted gap. Drivers with relatively high acceptance thresholds will probably reject many gaps before finding one that is acceptable, whereas drivers with low acceptance thresholds may accept the first gap offered to them. Consequently, a gap acceptance distribution based on all acceptances and rejections will underestimate the proportion of turning drivers prepared to accept a gap of a given size.

Ashworth (1970) and Miller (1974) discuss the various methods used by different researchers to remove this bias from their data. The method proposed in Ashworth (1968) appears to be the easiest to apply: the gap acceptance curve should be displaced by an amount equal to the product of the variance of the observed gap acceptance distribution and the major road volume (in vehicles per second).

One method of avoiding this problem is to consider only lags; since just one acceptance or rejection decision is recorded for each driver, lag acceptance data is not biased. However, the size of a rejected lag is difficult to determine accurately, since turning vehicles which are going to stop at the junction may delay their arrival at the intersection by decelerating as they approach the junction. In addition, if only lags are considered, observations must be made for much longer time periods in order to collect sufficient data from which to derive a gap acceptance function.

Age and sex of driver

The effect of age on gap acceptance is difficult to evaluate experimentally unless one observes the behaviour of individuals, since it is very difficult to estimate the age of drivers during the observation of a large number of vehicles passing through an intersection. Bottom and Ashworth (1978) studied the behaviour of individual drivers in a simulated experiment. Subjects seated in a car parked at right angles to traffic on a major road indicated by the use of brake lights the time periods when they would not cross the road. Four groups of subjects were studied: undergraduate and postgraduate students, police officers, and members of the Institute of Advanced Motorists. Bottom and Ashworth found that older drivers exhibited greater variability in behaviour, but the effect of experience was not investigated.

In a similar experiment, Hills (1979) studied the behaviour of four groups of subjects: young or old (aged 31 - 40 or 61 - 70) males and females. Subjects seated in a vehicle parked at right angles to one side of a dual carriageway were asked to indicate the "last possible safe moment to cross" in front of an approaching vehicle. He found that females required longer minimum gaps than males, but age had no effect on gap allowed. However, he suggests that young drivers base their acceptance decision on a constant time gap criterion whereas older drivers adopt a constant distance criterion, and that this difference may be due to the poorer speed judgement of the older drivers. Hills' data has also been analysed by Darzentas, McDowell and Cooper (1980), who showed that males are more consistent in their behaviour than females.

Ebbesen and Haney (1973) observed drivers leaving a large car park at a shopping centre in San Diego (USA), and Ebbesen, Parker and Konecni (1978) studied the behaviour of drivers performing left turns at four different T-junctions in San Diego. The results of both studies indicate that males accept shorter gaps than females; the difference in median accepted gaps typically amounted to half a second.

<u>Audience</u>

Ebbesen and Haney (1973) found that the presence of another vehicle beside the turning vehicle had no effect on gap acceptance. They noted a decrease in median accepted gap when there was a car behind the turning vehicle, and a further decrease if there was more than one car behind; however, they suggested this was due to the effects of waiting (either in line or at the junction), and recategorised their data to confirm this. Bottom (1975) found no effects attributable to vehicles queueing behind the turning vehicle.

Although the presence of other vehicles has no effect on gap acceptance behaviour, an audience inside the turning vehicle may have an effect. Ebbesen and Haney (1973) showed that drivers accepted shorter gaps after being forced to wait in line approaching the intersection, but this effect was less marked if the vehicle contained passengers. No difference in behaviour was found for the no-waiting situation.

Speed of oncoming vehicle

Rorbech (1966) evaluated Raff's critical lag* for vehicles emerging from the minor road at five T-junctions in Denmark. Three of the locations were similar except for different speed limits (35, 50 and 70 km/hour); he found that the critical lags at these sites increased as the major road speed limit increased. Since mean speeds are generally higher on roads with higher speed limits, this indicates a positive correlation between gap acceptance and the speed of major road vehicles.

Gibbs (1968) conducted an experiment on a test-track to evaluate the relationship between gap acceptance and speed. Four subjects sat in a parked car while vehicles approached in the nearside stream at constant speeds (16, 32, 48 or 64 mph). At various times during the vehicle's approach, an electrically operated shutter opened for one second, and subjects had to state whether or not they would

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*The critical lag is defined such that the number of accepted lags shorter than the critical lag equals the number of rejected lags longer than the critical lag (Raff, 1950).

accept the available gap, and cross in front of the approaching vehicle (note that subjects did not actually carry out the manoeuvre). Gibbs found that the median accepted gap was directly related to the speed of the approaching vehicle.

Earlier research at Royal Holloway College investigated the effect of the speed of approaching vehicles on the gap acceptance behaviour of merging drivers at a simple T-junction. Cooper, Smith and Broadie (1976) found that the median accepted distance gap is positively correlated with the speed of approaching major road vehicles. The effect of approach speed on the gap acceptance behaviour of crossing drivers is discussed in Chapter 4.

Bottom and Ashworth (1975 and 1978) found a similar result in a study of individual drivers leaving a government establishment's car park. Sixteen drivers were identified, all of whom turned right into a major road. They report a negative correlation between the critical time gap and the speed of approaching vehicles (for gaps measured in distance, the correlation was positive). Uber (1979) studied gap acceptance and speed at a stop-sign controlled junction in Melbourne (Australia). He reported that the median accepted distance gap of merging vehicles is directly proportional to the speed of the approaching major road vehicle.

Acceleration of turning vehicles

Bottom (1975) found that vehicles accepting short gaps covered the first 35 feet after their turn more quickly than those accepting long gaps. He also showed that drivers of cars with larger engine capacities accepted shorter gaps. Both these results suggest that gap acceptance behaviour depends on the acceleration of the turning vehicle; this subject is discussed in more detail in Chapter 5.

Evans and Herman (1976) conducted an experiment on a test-track to evaluate the effect of acceleration capability on gap acceptance, employing similar methods to those used by Herman and Weiss (1961). Seven male subjects were each exposed to several gaps near their median accepted gap in alternate sessions with a test vehicle in normal or modified mode (starting acceleration capability was lower in the modified condition). Subjects familiarised themselves with the characteristics of the car before each run. They found that subjects accepted longer gaps when driving the modified car, but that the increase in gap size was insufficient to compensate fully for the change in vehicle performance.

Type of vehicle turning

Blunden, Clissold and Fisher (1962) conducted studies at three T-junctions in Sydney (Australia), and observed the behaviour of drivers performing a simple merging manoeuvre. They found that cars and commercial vehicles combined accepted slightly longer gaps than cars alone. Ashworth and Green (1966), however, reported no appreciable difference between the gaps accepted by heavy commercial vehicles and all other vehicle classes ie cars, light goods vehicles and two-wheeled vehicles combined. Wagner (1966) claimed he found no significant difference in the gaps accepted by trucks and cars, but one of his figures suggests that trucks accept slightly longer gaps than cars. In a study of merging behaviour at a large urban roundabout, Powell and Glen (1978) showed that the mean critical gap for lorries was about 30% greater than that for cars.

These results are reasonably consistent, despite the different vehicle classifications used in each study. They indicate that turning goods vehicles accept longer gaps than turning cars, in a variety of manoeuvres at different types of intersections. Similar results were found in our studies at T-junctions (Chapter 5).

Queue acceptance

Several researchers have investigated the topic of queue acceptance, which is the acceptance of a large gap by two or more turning vehicles. Pearson and Ferreri (1961) and Uber (1978) both showed that there is a linear relationship between the number of vehicles entering a particular gap and the length of the gap. Queue acceptance is discussed in detail in Chapter 7.

Police presence

The effect of police presence on gap acceptance behaviour has been the subject of recent research at Royal Holloway College (Cooper, 1976; Cooper and McDowell, 1977; Storr, Cooper and McDowell, 1980). The presence of a police motorcyclist at a T-junction had a marked effect on the behaviour of drivers turning right from the minor road; significantly longer gaps were accepted during periods of police presence, and the change was more noticeable in the merging part of this manoeuvre. In addition, similar size effects were produced when warning signs were placed in the minor road on the approach to the junction.

Smith (1974) evaluated the effects of police presence on the behaviour of merging drivers at a T-junction. She showed that the observed gap acceptance distribution was shifted towards longer gaps during periods of police presence; the difference amounted to just over a second, and occurred throughout the whole range of gaps. Similar effects were produced by a manned police car and a police motorcyclist.

PEDESTRIAN GAP ACCEPTANCE

Recent research on pedestrian gap acceptance has mainly been concerned with the behaviour of children crossing roads (see, for example, Grayson (1975)). Since the behaviour of drivers performing simple crossing manoeuvres may be similar to the behaviour exhibited by pedestrians crossing a road, some of the literature on pedestrian gap acceptance is reviewed in this section. A more extensive review appears in Katz, Zaidel and Elgrishi (1975).

Robinson (1951) reported observations of pedestrian behaviour at two urban sites (one of which was one-way). Shorter gaps were accepted at the one-way street site, which is to be expected since the road width was less than at the other site. There was some indication of a difference in behaviour for day and night conditions, but waiting time at the kerb had no effect on the median accepted gap. At the two-way street site, median accepted gaps for the nearside stream of traffic were shorter than those for the farside stream in both day and night conditions.

Moore (1953) summarised early research on pedestrian behaviour by the Road Research Laboratory. One study at a pedestrian crossing with a central refuge found that pedestrians based their decision on a time gap rather than a distance gap, and that pedestrians increased their crossing speed if the time to arrival of the next approaching vehicle was less than 7 seconds.

Cohen, Dearnaley and Hansel (1955) described observations of pedestrians crossing a single lane of traffic. They concluded that females have longer critical gaps than males, and that young males (under 30) have shorter critical gaps than old males. The critical gaps of females increased with age for age groups over 16; very young females (under 16), however, had a high critical gap.

DiPietro and King (1970) report the results of a study of pedestrian behaviour at an unmarked crossing outside a university building. They concluded that females were willing to wait longer at the kerb for an adequate gap; males accepted shorter nearside <u>time</u> gaps and shorter farside <u>distance</u> gaps than females, although there was no significant difference in the size of the total gap (nearside plus farside) accepted by males and females; waiting time at kerb had a significant effect on the size of gap accepted (in general, gap increased as waiting time increased); groups of pedestrians accepted shorter gaps than individuals, but crossed more slowly; and pedestrians crossing the road at high walking speeds accepted shorter gaps than those crossing at normal walking speeds.

These studies report many similar results to those obtained from the observation of driver behaviour. In particular, Cohen, Dearnaley and Hansel (1955) and DiPietro and King (1970) found differences in behaviour between males and females which are similar to those exhibited by drivers. Robinson (1951) suggests that waiting time at the kerb has no effect on gap acceptance whereas DiPietro and King (1970) found a significant effect; results of studies on waiting time and drivers' gap acceptance behaviour are also inconclusive (Ashton (1971); Ebbesen and Haney, (1973); Bottom and Ashworth (1978)), so further research may be needed to clarify the effect of waiting time on gap acceptance. Robinson (1951) has indicated that behaviour may be different during day and night conditions: recent research by Darzentas, Holms and McDowell (1980) showed that merging drivers accepted significantly shorter gaps at night than they did in daylight.

One effect reported by both Moore (1953) and DiPietro and King (1970) has not been investigated for drivers. Both studies found a positive relationship between the size of the gap accepted and the speed at which pedestrians crossed the road; hence pedestrians complete their manoeuvre more quickly when shorter gaps are accepted. Results presented in Chapter 5 show that similar effects are exhibited by drivers performing simple gap acceptance manoeuvres at T-junctions.

TRAFFIC CONFLICTS TECHNIQUE (TCT)

An extensive review of the literature on conflict studies has been produced by Hondel and Kraay (1979). Summary reports of research in progress in various countries were presented at two recent workshops (Proceedings of First and Second International Workshops on Traffic Conflicts). In this section, the development of the traffic conflicts technique at the General Motors Research Laboratories (USA) is outlined, and the use of the technique in the UK by TRRL is described. Work on traffic conflicts at Royal Holloway College is discussed briefly at the end of this section.

Development of TCT

The traffic conflicts technique was developed as a measure of accident potential at intersections by Perkins of the General Motors Research Laboratories in Michigan; this technique is described in a procedures manual available from GMR, and results of initial studies at about thirty intersections are reported in Perkins and Harris (1968). They defined a traffic conflict as any potential accident situation, categorised either by evasive actions such as braking or lane changing, or by traffic violations. Several types of conflict were identified, which depended on the manoeuvres of the vehicles involved eg, a rear-end conflict occurred when a vehicle stopped or slowed down unexpectedly, causing the vehicle behind it to take evasive action. Three 12-hour observation periods were spent at each site, during which the number of traffic conflicts in each category was counted, and the flow in each traffic stream evaluated. Results indicated a high level of association between traffic conflicts and reported accident frequencies.

Conflict studies in the UK

Conflict studies carried out by TRRL (Older and Spicer, 1976; Spicer, 1971, 1972 and 1973) used a slightly different technique. They defined a traffic conflict as "a situation involving one or more vehicles where there is imminent danger of a collision if the vehicle movements remain unchanged"; traffic violations were not classified as conflicts. The basic improvement over the General Motors technique was the introduction of a severity scale; five categories were used, which depended on the severity of evasive action required (Table 2.1). Complementary data to the observers' reports was obtained by filming the intersections.

The relationship between conflicts and accidents at several sites could be investigated in more detail using this refined conflicts technique. Spicer showed that low correlations between conflicts and accidents are found if all conflicts are included

SEVERITY GRADE	DESCRIPTION
SLIGHT	
1	Precautionary braking or lane changing; collision very unlikely.
2	Controlled braking or lane changing to avoid collision, but with ample time for manoeuvre.
SERIOUS	
3	Rapid deceleration or lane change to avoid collision, resulting in a "near miss" situation.
4	Very near miss or minor collision occurred.
5	Serious collision.

Table 2.1 Conflict severity grades used by TRRL (reproduced from Spicer, 1971).

in the analysis; if only serious conflicts are considered however, a significantly high correlation with accidents is found ("serious" means severity grades 3, 4 and 5 only). In addition, serious conflicts occur at the same time of day as accidents, and at similar locations within the intersections. Consequently, serious conflicts are good predictors of accidents at intersections (the factor of proportionality depends on the types of vehicles involved). The conflict technique has also been used in the UK to assess the safety of design elements in "before-and-after" studies (Older and Spicer, 1976).

Recent research at TRRL (Shippey, 1979) investigated the possibility of detecting conflicts automatically. A T-junction site was comprehensively instrumented with permanently installed vehicle sensors, which monitored the passage of vehicles through the junction. Information from these sensors was recorded on magnetic tape for analysis by computer. Details of vehicle lengths, speeds and inter-vehicle gaps were obtained: conflicts occurred when the time separation of two conflicting vehicles was less than a prescribed value, which depended on the manoeuvre. A cine film of the site was also taken and a subjective assessment of conflicts at the site made from this film. Preliminary results indicate some agreement between the conflicts detected by each technique, although many low grade conflicts detected automatically were not recorded in the subjective analysis of the film. A more detailed analysis is in progress.

Use of TCT at Royal Holloway College

A simulation model of a T-junction has been developed at RHC which uses traffic conflicts as a predictor of accident risk.

Details of flows, speeds and gap acceptance at a particular junction are the model's input parameters; the output consists of the numbers of traffic conflicts recorded at various locations within the junction. Model conflicts occur when turning vehicles make poor gap acceptance decisions, and force vehicles in the major road to slow down to avoid collision; the amount of deceleration required gives a measure of the severity of the conflict. The simulation model is described in greater detail in Chapter 8.

The traffic conflicts technique is used in many countries for various purposes. If researchers accept the technique as a useful tool in its own right, it can be used for comparative studies between locations or for assessing improvements in behaviour in before-and-after studies. However, if the technique is used as a direct measure of safety at a particular location, the TCT should be validated against some accepted measure of safety (such as traffic accidents). This whole problem is discussed by Hauer (1979).

SUMMARY

A vast amount of research on gap acceptance behaviour has been published in the literature. The majority of papers are concerned with aspects of capacity and delay at junctions, but several studies have investigated the effects of various factors on gap acceptance behaviour. Areas examined in more detail in this thesis are:

i) major road speeds and flow (Chapter 4)

ii) manoeuvre time and class of vehicle (Chapter 5)

iii) occupants of turning vehicle (Chapter 6)

iv) queue acceptance (Chapter 7).

The need for further studies on the effect of waiting time on gap acceptance behaviour is noted.

The use of the traffic conflicts technique has been described briefly. The problems of validating this technique against an accepted measure of safety are discussed in detail in Chapter 8. Examples of the ways in which gap acceptance results may be evaluated in terms of safety using the TCT are given in Chapter 9.

CHAPTER THREE

EXPERIMENTAL METHODS

Introduction

There are two gap acceptance manoeuvres at a T-junction in which turning drivers consider only one stream of approaching major road traffic: crossing, where a vehicle turns right from the major road into the minor road, across a single stream of oncoming major road vehicles, and merging, where a vehicle turns left out of the minor road and merges with the nearside stream of major road traffic (Figure 3.1. Note that vehicles keep to the left in the UK). The behaviour of drivers performing these manoeuvres has been observed at several non-urban T-junctions in Southern England at various times. The locations of the experimental sites are shown in Table 3.1, and detailed descriptions of the junctions are given in Appendix 2.

Observations at two of these sites (Denton and Tongham) were carried out as part of Project 2001 of the Home Office Police Scientific Development Branch. Video techniques were used to collect data at these two sites, and the way in which the video-tapes were analysed is outlined. The vast amount of time required to analyse these videotapes led us to develop a different system of data collection, based on a microprocessor.

SITE	INTERSECTION (Major/minor road)	NATIONAL GRID REFERENCE
BROADFORD ROAD	A3100/A248	SU 990 465
COMPTON	A3/B3000 (East)	SU 954 474
DENTON	A259/B2109	TQ 453 021
PEASMARSH	A3100/B3000	SU 988 460
PUTTENHAM	A3/B3000 (West)	SU 947 472
SHALFORD	A248/A281	TQ 000 468
TONGHAM	A31/A3014	SU 885 482

Table 3.1 Locations of experimental sites.

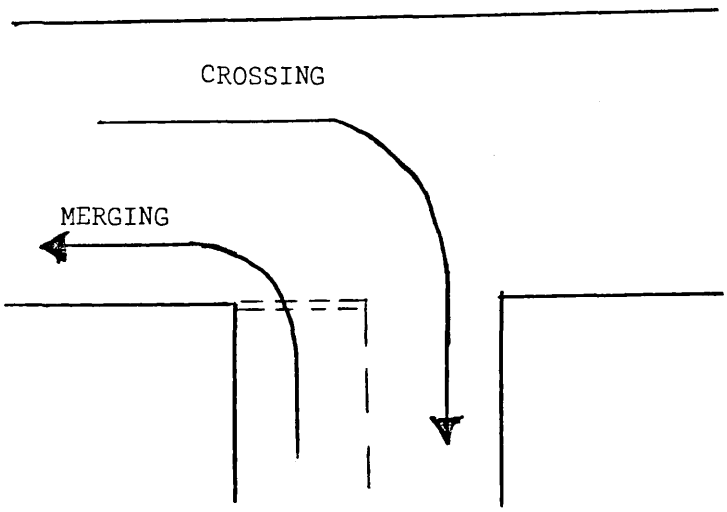


Figure 3.1 Schematic representation of crossing and merging manoeuvres at a standard T-junction.

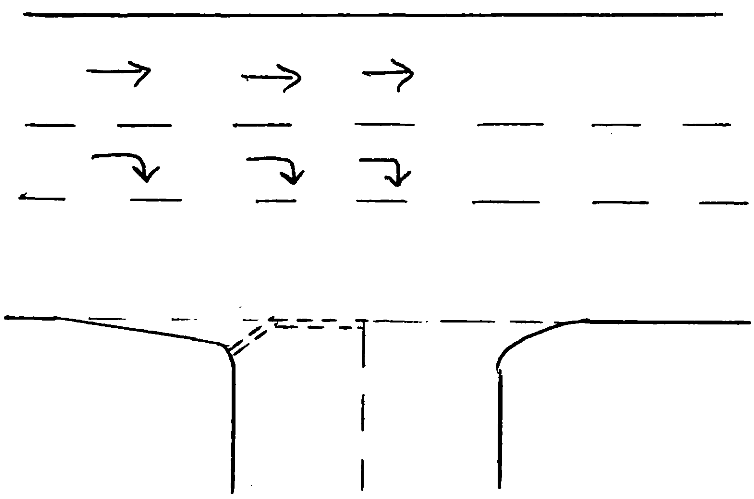


Figure 3.2 Outline of a typical T-junction, showing turning lanes.

This system is briefly described, and its operational use is detailed. The two techniques of data collection are then compared, and the considerable saving in analysis time achieved with the new system of data collection is discussed. The advantages of combining these two techniques are then outlined.

EXPERIMENTAL SITES

All experimental sites are priority controlled T-junctions situated in non-urban areas (Table 3.1). In general, roads at each site are level, and visibility is excellent in all directions. Turning lanes are provided at most of the sites (Figure 3.2). Data were collected during morning and evening peak periods on mid-week days only (ie Tuesday, Wednesday and Thursday). Observations were made only in good weather conditions, usually during the summer months (further details are given in Appendix 2).

VIDEO TECHNIQUES

The filming methods at Denton and Tongham were slightly different. The junction at Denton was filmed from a car parked beside the major road (Figure 3.3), using a portable video camera fitted with a zoom lens. Traffic passing through the junction at Tongham, however, was filmed with a camera mounted in a van parked in a field overlooking the junction (Figure 3.4). This camera was fitted with a wide angle lens, which allowed the behaviour of merging vehicles to be observed during their turn,

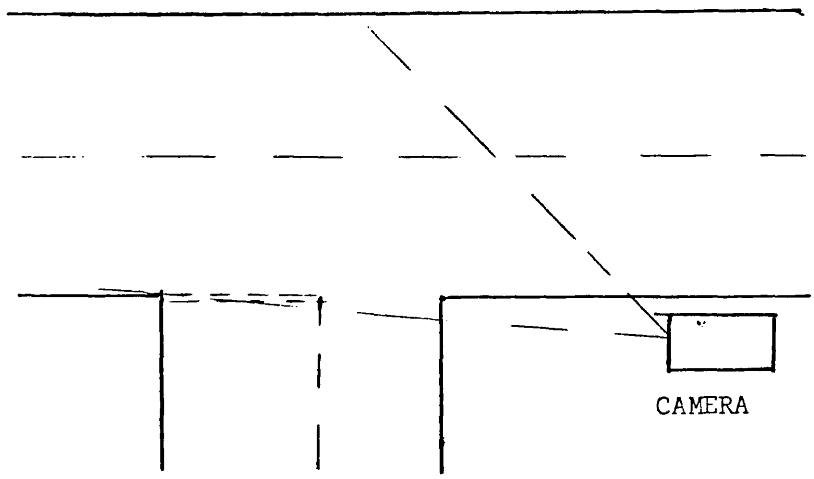


Figure 3.3 Location of camera and field of view at Denton. The camera was fitted with a zoom lens.

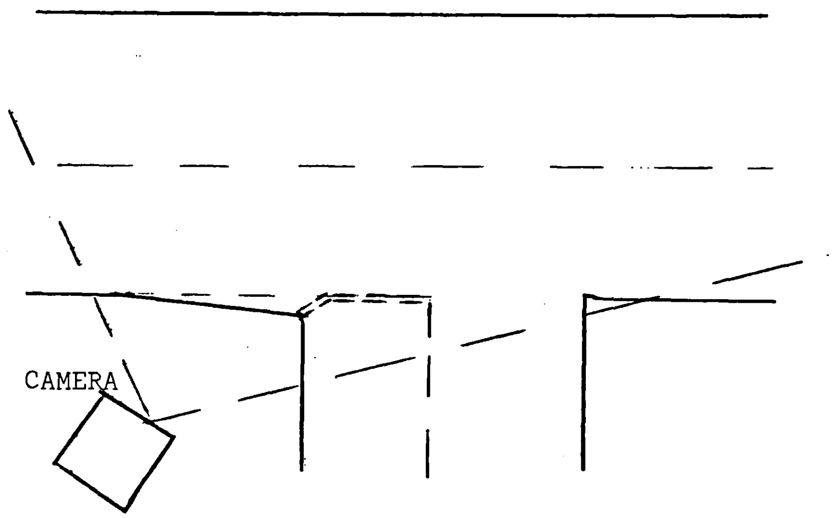


Figure 3.4 Location of camera and field of view at Tongham. The camera was fitted with a wide angle lens.

and for some distance along the major road afterwards (Figure 3.4).

Two different methods were used to incorporate a time base on the picture:

i) The videotapes from the Denton experiment were played through a TV mixing unit, which created a new videotape containing the original picture plus a digital clock image superimposed in one corner.

ii) A digital timer was positioned in front of a large monitor while each Tongham videotape was played back through it. The monitor and timer were then refilmed using the video camera, producing a new picture with the clock image in one corner.

Videotape Analysis

The videotapes of each junction were analysed in the same way. Each videotape was played back through a large monitor in slow motion. The film was stopped when certain events occurred, to enable the clock time to be read (in tenths of seconds). Events of interest were the arrivals of turning and non-turning vehicles at the junction, the commencement of a turn, and the completion of a simple turning manoeuvre: for crossing, this occurred when the rear of the turning vehicle cleared the major road; for merging, it occurred when the front of the turning vehicle reached a fixed (imaginary) line in the major road. The positions within the junction at which these events were recorded are shown in Figure 3.5. Each vehicle was classified by size as one of the following types:

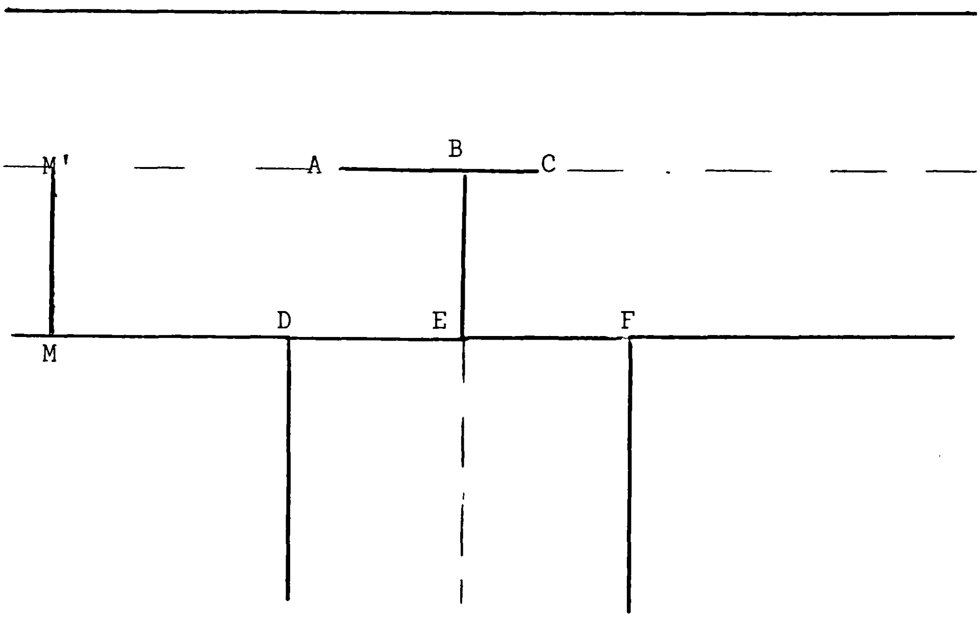


Figure 3.5 Positions at which events were recorded. Arrivals of major road vehicles are recorded at BE. Arrivals and departures of crossing and merging vehicles are recorded at AC and DE respectively. Clearing events occur when the rear of a crossing vehicle clears EF, and marker events occur when the front of a merging vehicle reaches MM'.

car; van or light goods vehicle; truck or heavy goods vehicle; any other kind of vehicle.

The output from this process was a list of events and the times of their occurrence. This information was then recorded on punched cards, ready for analysis by computer. Details of the data processing programs are given in Appendix 3. The data was first cleared of any errors detected by the processing programs. Tables of accepted and rejected lags and gaps for each turning manoeuvre were then obtained, for gap sizes in half second intervals from one to ten seconds. These tables could be classified according to various parameters eg class of vehicle, speed of oncoming vehicle, manoeuvre time of turning vehicle (the manoeuvre time is calculated from the event data as the interval between the start of a turn and its completion). Distributions of the speeds of major road vehicles and the manoeuvre times of turning vehicles were also obtained by computer.

Video filming has been used before to collect data for traffic studies (see, for example, Ashworth (1976) and Seddon, Rowell and Wilson (1974)), but we found the technique had several disadvantages. It provides a complete, permanent record of events, which may be used to derive further results which may not have been considered in the original experiment. However, the extraction of detail from the film is extremely time consuming, and very tiring for the analyst. This means that several months may elapse between the completion of an experiment and the production of results. Such delays are expensive, and they do not allow the experimenter to redesign his research in the light of results, nor do they permit him to judge his success in achieving his aims as the experiment progresses.

MICROPROCESSOR-BASED SYSTEM

The major drawback of the video technique is the vast amount of time required for analysis. We have now developed a new data collection system, based on a microprocessor, which considerably shortens the time needed to produce results (Storr, Wennell, McDowell and Cooper, 1979). Technical details of this data collection system and the programming of the microprocessor are given by Storr (1980).

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 (in hundredths of seconds) are recorded. The triggering of an automatic sensor is treated in the same way as a pressed button. Data is stored on an audio cassette tape.

This system has been used to collect data at T-junctions. In general, an observer uses one handset, and observes one stream of traffic at the junction. He presses a particular button on the handset at the time that a specific event occurs; for example, the arrival of a vehicle, or the start of a turn. He may also record additional information such as the kind of vehicle, the sex of its driver and whether the vehicle contained passengers. Automatic sensors may be used to detect the arrivals of major road vehicles. Additional information about major road vehicles (such as the kind of vehicle) may also be recorded by observers.

Speeds of major road vehicles approaching the junction are measured using pairs of automatic sensors set a fixed distance apart (the speed is calculated from the time interval between the two sensor triggers). The configuration of sensors and equipment at a typical junction is shown in Figure 3.6. Each observer must stay within a certain distance of the equipment due to the length of leads connecting handsets to the system. Experience has shown that the best position for equipment and observers is opposite the mouth of the T-junction, since all streams of traffic can be readily observed from that position; if observers are situated adjacent to the minor road, one minor road stream of traffic may obstruct the view of the other.

This process yields a set of events and the times at which they occurred that is similar to the list of events derived from analysis of the videotapes. The time required to transfer this information from the cassette tape to a mainframe computer depends on the amount of data recorded on the tape. In general, the data collected in one period of observation (up to two hours) can be stored on one side of a standard C90 cassette tape, so the playback time is usually less than 45 minutes. Once transferred to a mainframe computer, the accuracy of the transfer is checked and the data is transformed (as described in Appendix 4) so that it can be analysed with the same processing programs as before.

OBSERVERS

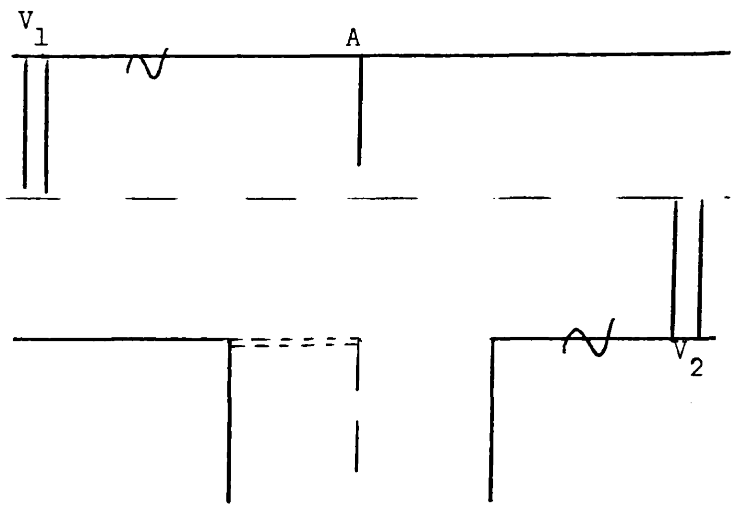


Figure 3.6 Configuration of automatic sensors at a typical junction. Speeds of approaching major road vehicles are measured at V_1 and V_2. Arrivals of LA vehicles are recorded at A. Observers and equipment were generally located opposite the mouth of the junction.

COMPARISON OF TECHNIQUES

An appreciable reduction in analysis time is achieved with this new method of data collection. Average times for each phase of videotape analysis are as follows:

i) Refilming to incorporate a clock image takes about 1.5 times as long as the original filming, since some time is needed to rewind videotapes, reset the timer and set up the equipment.

ii) Extraction of detail from the videotapes is extremely time consuming, especially when all events are recorded. It is also very tiring for the analyst, and should not be done in long sessions if errors are to be minimised. This phase takes of the order of 20 times as long as the original filming.

iii) Punching the data on to cards takes of the order of three hours per hour of film.

This means that at least one week's continuous work is needed to get a raw data file representing one period of data collection on to the computer. However, a data file from the microprocessor-based system can be transferred, checked and transformed in a matter of hours: direct transfer of information from cassette tape takes 30 minutes on average, while parity checks and blocking checks are done by computer program (Appendix 4); transformation of the data takes about 30 minutes, since a separate short program is required for each file, and depends on the events recorded. Thus the data from one observation period can be stored on a computer file in a much shorter time using the new method.

A data collection system based on a combination of these two techniques seems ideal. Observations at the site could be made using video techniques, so a permanent record of events would be obtained. The videotapes could then be analysed _in real time_ using the microprocessor-based system, so the amount of time required for analysis would be much shorter. In particular, the observation and analysis could all be carried out by _one_ researcher, so observer costs would be minimised, and any bias effects introduced by different observers would be removed.

CONCLUSIONS

Two different methods of collecting data at T-junctions have been described. The use of video techniques means that a permanent record of events may be obtained, but the vast amount of time required for analysis of the videotapes leads to a long delay between the experiment and the appearance of any results. The system based on a microprocessor achieves a considerable saving in analysis time, but several observers are needed at the site to collect the relevant information. Given the choice of only one of these methods of data collection, we would recommend the microprocessor-based system. However, if sufficient funds are available, the use of _both_ techniques may provide the best method of data collection.

CHAPTER FOUR

MAJOR ROAD SPEEDS AND FLOW

Introduction

The relationship between gap acceptance and approach speed of priority vehicles has been the subject of several studies. In a test-track situation, Gibbs (1968) investigated the crossing behaviour of four drivers; subjects sat in a parked car while vehicles in the nearside stream approached at constant speeds. At various times during the vehicle's approach, a shutter opened for one second, and subjects were asked to state whether or not they would accept the available gap (subjects did not actually carry out the manoeuvre). Gibbs found that the critical distance gap D (in feet) was related to the approach speed V (in feet/second) by

$$D = 5.4 V. \tag{4.1}$$

Cooper, Smith and Broadie (1976) examined the behaviour of drivers turning out of a minor road and merging with the nearside stream of major road traffic. Accepted and rejected gaps were classified according to the speed of approaching major road vehicles, and they found that the median accepted distance gap D (in feet) could be expressed as a simple function of the approach speed V (in feet/second) by

$$D = 38 + 5 V. \tag{4.2}$$

Bottom and Ashworth (1975 and 1978) found a similar effect in a study of sixteen individual drivers leaving a car park (all subjects turned right on to a major road). They report a negative correlation between the critical time gap and the speed of approaching vehicles (for gaps measured in distance, the correlation was positive). Uber (1979) studied gap acceptance and speed at a stop-sign controlled junction in Melbourne. He showed that the median accepted distance gap of merging vehicles was directly proportional to the speed of approaching major road vehicles.

All these studies indicate that there is a positive correlation between the speed of approaching priority vehicles and gap acceptance measured in terms of distance. In this chapter a similar relationship is derived for crossing vehicles at Denton, and compared with the relationship for merging vehicles given by equation (4.2).

We have already discussed the bias implicit in observed gap acceptance functions (Chapter 2). Ashworth (1968 and 1970) has calculated a correction term for this bias which equals the product of the major road volume (in vehicles per second) and the variance of the driver gap acceptance curve. In this chapter, data for crossing at Denton and merging at Tongham 1976 are used to examine the relationships between gap acceptance and major road flow. Different results are found for the two manoeuvres, but the derived relationships can be explained in terms of the different presented gap distributions for various flows at these sites.

APPROACH SPEED		MEDIAN ACCEPTED GAP	
V(mph)	V(ft/s)	T(s)	D(ft)
27.5	40.3	5.07	204
32.5	47.7	4.09	195
37.5	55.0	3.92	216
42.5	62.3	3.73	233
47.5	69.7	3.66	255

Table 4.1 Median accepted gaps for crossing vehicles at Denton. V is the mid-point of a 5 mph speed band.

DATA ANALYSIS

Gap acceptance tables for crossing vehicles at Denton were classified by the speed of the vehicle closing the gap. A log-normal gap acceptance function was fitted to the data in each 5 mph speed band using the probit technique (Appendix 1). Results are presented in Table 4.1; median accepted gaps for each speed band are expressed in terms of both time T and distance D (=VT).

Gap acceptance tables for five minute blocks of data for Denton and Tongham 1976 were obtained by computer. The number of major road vehicles arriving at the junction during each five minute block of data was also derived, and data from all gap acceptance tables with major road flows in particular ranges were combined. The probit method was used to fit a log-normal gap acceptance function to the data in each flow range; gap acceptance parameters are given in Tables 4.2 and 4.3.

RESULTS

Speed of major road vehicles

There is a significant positive correlation between the median accepted distance gap D (in feet) and the approach speed V (in feet/second) for crossing vehicles at Denton. A least squares fit to the data in Table 4.1 gives the following linear relationship:

$$D = 115 + 1.9V \tag{4.3}$$

$$(p < 0.05; \; 37 < V < 73).$$

MAJOR ROAD FLOW (vph) Range	Mean	MEAN APPROACH SPEED (mph)	MEDIAN ACCEPTED GAP(s) Predicted	Empirical	VARIABILITY PARAMETER
150-199	173	36.55	4.05	3.35	1.62
200-249	232	36.78	4.03	3.90	1.33
250-299	282	36.60	4.04	3.74	1.48
300-324	315	35.59	4.10	3.86	1.42
325-349	340	36.70	4.04	4.10	1.47
350-374	359	36.14	4.07	3.48	1.52
375-399	389	35.52	4.11	4.33	1.39
400-424	410	36.62	4.04	3.82	1.40
425-449	437	35.43	4.11	4.11	1.32
450-499	479	36.02	4.08	4.30	1.24

Table 4.2 Major road flows and speeds and crossing gap acceptance at Denton.

MAJOR ROAD FLOW (vph) Range	Mean	MEDIAN ACCEPTED GAP	VARIABILITY PARAMETER
350-399	369	5.34	1.72
400-449	428	5.24	1.66
450-499	469	5.26	1.56
500-549	532	4.41	1.55
550-599	573	4.47	1.50
600-649	627	4.90	1.49
650-699	674	4.34	1.47
700-749	719	4.45	1.54
750-799	772	4.41	1.53

Table 4.3 Major road flows and merging gap acceptance at Tongham 1976.

Dividing this equation throughout by V gives

$$T = \frac{D}{V} = \frac{115}{V} + 1.9, \qquad (4.4)$$

so the median accepted gap corresponds to a constant time of 1.9 seconds plus a constant distance of 115 feet.

Comparison of crossing and merging

Equations (4.2) and (4.3) are plotted in Figure 4.1; the slopes are significantly different at the 5 per cent level. The approach speed is not critical for crossing, provided that the accepted gap is sufficient for the turning vehicle to clear the lane of oncoming traffic; at each speed, the median accepted time gap was much larger than the 2 to 2.5 seconds typically required to complete the turn (Chapter 5). Crossing is a relatively simple manoeuvre, and drivers appear to base their turning decisions largely on simple distance cues. Merging, on the other hand, requires more detailed information about the speed of the traffic stream to be entered, and more complex cues must be used.

Median accepted gap and flow

Crossing at Denton. There is a significant positive correlation between the median accepted time gap T for each flow range and the mean flow F in that range. A least squares fit to the empirical data in Table 4.2 gives

$$T = 3.1 + (2.3 \times 10^{-3})F \qquad (4.5)$$

($r = 0.68$, d.f. $= 8$, $p < 0.05$), where T is in seconds and F is in vehicles per hour ($150 \leq F \leq 499$).

Merging at Tongham. There is a significant negative correlation between the median accepted time gap T for each flow range and the mean flow F in each range. A least squares fit to the data in Table 4.3 gives

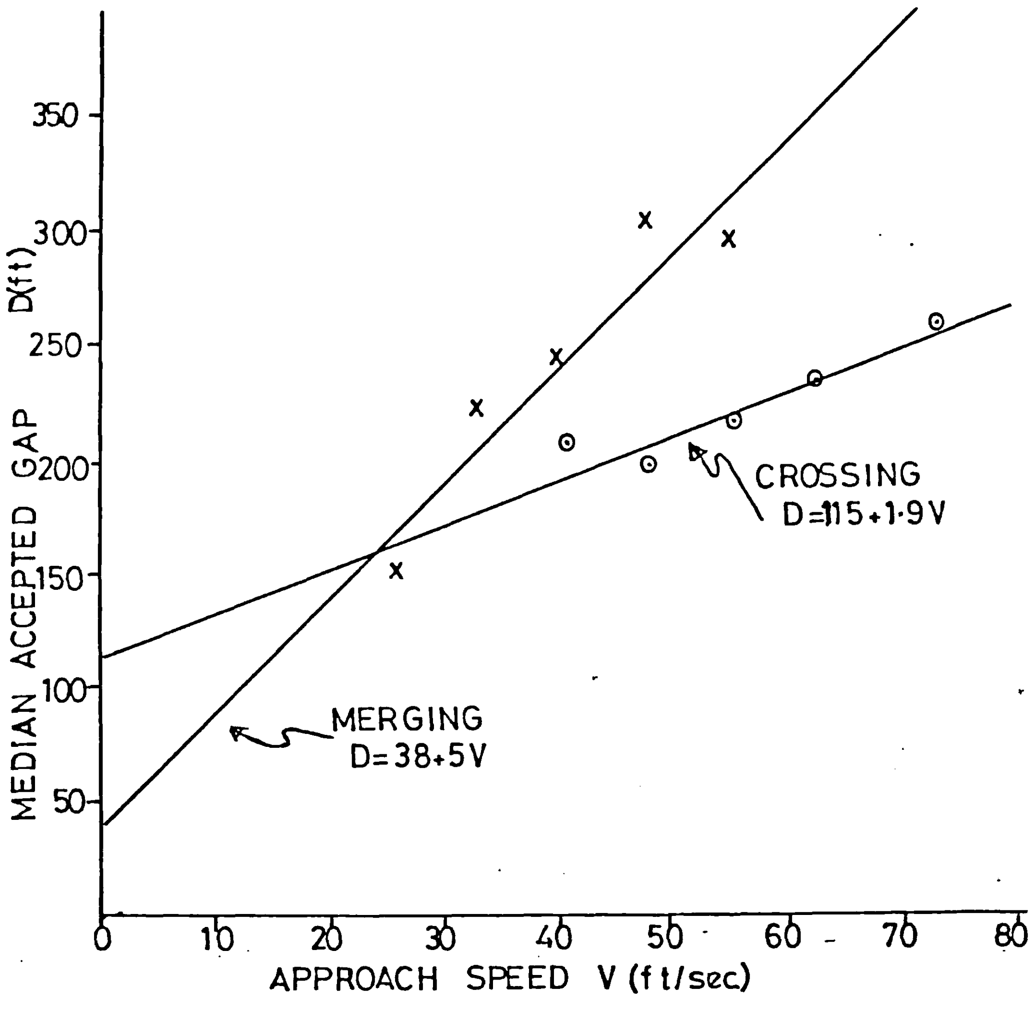


Figure 4.1 Median accepted distance gaps as functions of approach speed V for merging (equation 4.2) and for crossing at Denton (equation 4.3).

$$T = 6.2 - (2.5 \times 10^{-3})F \qquad (4.6)$$

($r = 0.81$, d.f. $= 7$, $p < 0.01$), for T in seconds and F in vehicles per hour ($350 \leq F \leq 799$).

Variability parameter and flow

There are significant negative correlations between the variability parameter S and the mean flow F at both sites. The following linear relationships were derived from the data in Tables 4.2 and 4.3:

a) Denton, crossing, $150 \leq F \leq 499$,

$$S = 2.4 - (2.8 \times 10^{-3})F \qquad (4.7)$$

($r = 0.68$, d.f. $= 8$, $p < 0.05$);

b) Tongham 1976, merging, $350 \leq F \leq 799$,

$$S = 1.8 - (4.6 \times 10^{-4})F \qquad (4.8)$$

($r = 0.79$, d.f. $= 7$, $p < 0.05$).

Major road speeds at Denton

The mean speed of major road vehicles in each five minute data block was calculated, and the overall mean for each flow range was derived. The expected value of the median accepted gap for each flow range was then obtained, using equation (4.4). Results are presented in Table 4.2. Clearly, the mean speed does not vary with flow, and the predicted median accepted gaps do not agree with the empirical values.

Presented gap distributions

The distributions of gaps presented by major road vehicles have been compared for two arbitrary flow ranges at each site. Normalised proportions of presented gaps for half-second intervals from one to ten seconds are shown in Tables 4.4 and 4.5, together with the cumulative proportions in each case.

	MAJOR ROAD FLOW (vph)			
	200-299		400-499	
Start of Time Gap Interval	Normalised Proportion	Cumulative Proportion	Normalised Proportion	Cumulative Proportion
1.0	.1073	.1073	.1021	.1021
1.5	.1073	.2146	.1021	.2042
2.0	.1034	.3180	.0594	.2636
2.5	.0728	.3908	.0641	.3277
3.0	.0651	.4559	.0570	.3847
3.5	.0536	.5095	.0618	.4465
4.0	.0345	.5440	.0475	.4940
4.5	.0306	.5746	.0546	.5486
5.0	.0728	.6474	.0570	.6056
5.5	.0422	.6896	.0570	.6626
6.0	.0498	.7394	.0451	.7077
6.5	.0422	.7816	.0499	.7576
7.0	.0383	.8199	.0523	.8099
7.5	.0422	.8621	.0499	.8598
8.0	.0153	.8774	.0451	.9049
8.5	.0422	.9196	.0333	.9382
9.0	.0536	.9732	.0428	.9810
9.5	.0268	1.0000	.0190	1.0000

Table 4.4 Presented gap distributions at Denton

	MAJOR ROAD FLOW (vph)			
	400-499		600-699	
Start of time gap interval	Normalised Proportion	Cumulative Proportion	Normalised Proportion	Cumulative Proportion
1.0	.2179	.2179	.2647	.2647
1.5	.1955	.4134	.2232	.4879
2.0	.1592	.5726	.1384	.6263
2.5	.0782	.6508	.0657	.6920
3.0	.0531	.7039	.0519	.7439
3.5	.0419	.7458	.0346	.7785
4.0	.0279	.7737	.0277	.8062
4.5	.0335	.8072	.0242	.8304
5.0	.0140	.8212	.0260	.8564
5.5	.0140	.8352	.0242	.8806
6.0	.0279	.8631	.0138	.8944
6.5	.0140	.8771	.0208	.9152
7.0	.0307	.9078	.0173	.9325
7.5	.0279	.9357	.0156	.9481
8.0	.0140	.9497	.0173	.9654
8.5	.0140	.9637	.0087	.9741
9.0	.0168	.9805	.0138	.9879
9.5	.0196	1.0000	.0121	1.0000

Table 4.5 Presented gap distributions at Tongham 1976

Denton (Table 4.4). For all gap size intervals less than eight seconds, the cumulative proportion of presented gaps for the flow range 200-299 vph is greater than that for the range 400-499 vph. This indicates that a higher proportion of shorter gaps is presented at the lower range of major road flow.

Tongham (Table 4.5). The cumulative proportion of presented gaps for the flow range 600-699 vph is greater than that for the range 400-499 vph for all the gap size intervals. This indicates that a higher proportion of short gaps is presented at the higher range of major road flow.

DISCUSSION

Gap acceptance and speed

The crossing behaviour of drivers may be described by a modified time hypothesis, in which the median accepted gap may be considered as a constant time gap plus a constant distance gap. This result is in agreement with those obtained by others in this country for different manoeuvres. It is not clear, however, whether this hypothesis represents a strategy of the turning driver; at least part of the observed variation in gap acceptance can be explained in terms of perceptual errors of judgement associated with vehicles having speeds different from the mean (see, for example, Brain (1962)).

Gibbs' result (equation 4.1) for crossing in a test-track environment is quite different from equation (4.3); his expression corresponds to a constant time gap of 5.4 seconds, with no distance component. There are a number of possible reasons for this difference: Gibbs' experiment was conducted on a test-track with only four subjects, while we observed more than 200 drivers in an actual road situation. In addition, only two of the approach speeds used by Gibbs (16, 32, 48 and 64 mph) were in the range of our experiment.

Speed and risk

Equation (4.4) shows that shorter time gaps are accepted in front of faster oncoming vehicles. This means that vehicles turning in front of fast major road vehicles are more likely to be involved in conflict situations than those turning in front of slower oncoming vehicles, since the difference between the accepted gap and the crossing time is considerably shorter. The gap acceptance and speed results for crossing at Denton are evaluated in terms of accident risk in Chapter 9, using the conflict simulation model; as expected, the risk of conflict involvement increases approximately linearly with speed.

Median accepted gap and flow

Equations (4.5) and (4.6) are plotted in Figure 4.2. Considering each manoeuvre in isolation, the relationship between median accepted gap and flow is quite different. However, the ranges of flow observed at each site were rather different, so the overall relationship between gap acceptance and flow may reflect both those shown (such as an inverted U-shape). Clearly further work is needed to clarify whether the observed differences are due to the different manoeuvres studied, or the different ranges of flow observed. Current research at RHC will investigate this problem in more detail.

The similarity in the mean speed of major road vehicles in each flow range at Denton indicates that the observed difference in gap acceptance behaviour is not due to a difference in the

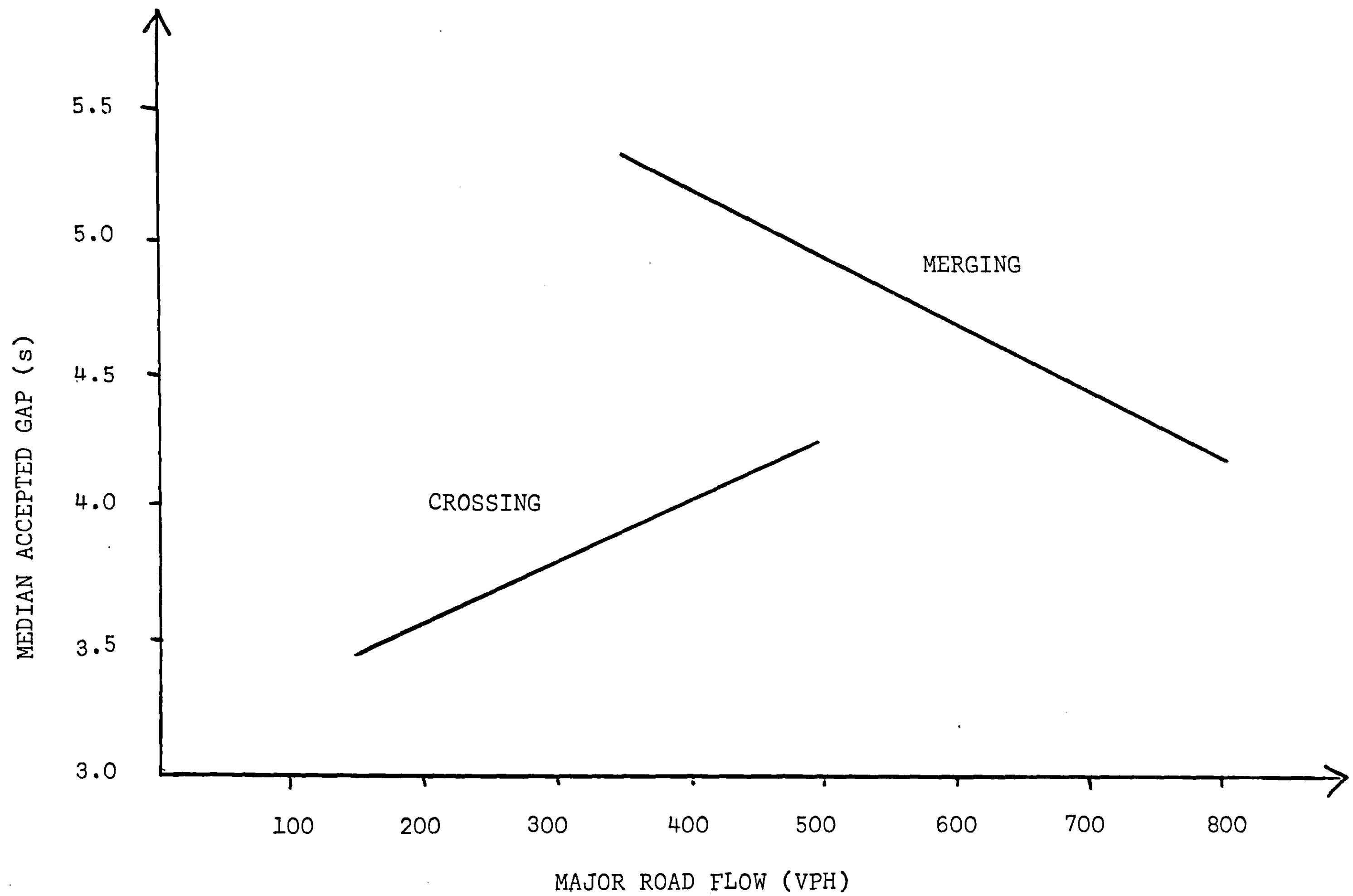


Figure 4.2 Median accepted gaps as functions of major road flow for crossing vehicles at Denton (eq. 4.5) and for merging vehicles at Tongham (eq. 4.6).

speed of approaching vehicles. The difference in behaviour probably reflects a difference in the presented gap distributions at different ranges of major road flow. We have shown above that a higher proportion of short gaps are presented for the major road flow range 200-299 vph than for the range 400-499 vph at Denton; such an effect is surprising, because we would expect less platooning to occur at low flows. However, it does explain why a positive correlation between median accepted gap and flow was obtained for this site. The results from Tongham show the relationship that we expected, since more platooning occurs at high flows. A comparison of the presented gap distributions for major road flows of 400-499 vph and 600-699 vph confirms that more short gaps are presented at the higher range of flow.

Variability parameter and flow

The variability parameter decreases as major road flow increases for both crossing and merging manoeuvres. This indicates that the members of the turning population are more consistent in their behaviour at higher ranges of major road flow at both sites. The reasons for this are not immediately obvious, but it is probably due to the distribution of presented gaps.

A detailed study of the effects of flow on headway distributions in the major road is in progress at RHC (Darzentas, 1980). The results of this research should make the explanation of the relationship between gap acceptance behaviour and flow easier. Only two sites have been considered in this chapter, and different manoeuvres were studied at each junction; clearly, the results we have produced need to be confirmed by studies at more junctions, where both manoeuvres can be investigated for the same flows and headway distributions. Further work is in hand (Wennell, 1980).

Ashworth's correction

Ashworth's correction term for the bias implicit in observed gap acceptance functions equals the product of the major road flow and variance of the driver gap acceptance curve. Our results indicate that the variance depends on the major road flow to some extent, so the actual correction term may be more complicated than the value derived by Ashworth (1968).

Ashworth made two basic assumptions when he calculated his correction term: that the driver gap acceptance curve is normally distributed, and that traffic in the major road is randomly distributed (ie negative exponential distribution of presented gaps). The first assumption is reasonable in the light of results presented in Ashworth and Bottom (1977), where X^2 tests show that a normal distribution gives a good fit to experimental data for individual drivers. The second assumption is probably not true in practice; current research by Darzentas (1980) shows that presented gaps in the major road follow a negative exponential distribution for gaps greater than some value T_o (which varies with site), but gaps less than T_o are normally distributed.

CONCLUSIONS

Gap acceptance and speed

The behaviour of crossing drivers may be represented by a modified time hypothesis in which the median accepted gap may be considered as a constant time gap plus a constant distance gap. At the particular intersection we observed, the median accepted gap D in feet was related to the approach speed V in feet per second by

$$D = 115 + 1.9V.$$

Since shorter time gaps are accepted in front of faster approaching vehicles, we would expect accident risk to increase as approach speed increases.

Gap acceptance and flow

Major road flow affects the gap acceptance behaviour of turning drivers. For the crossing manoeuvre at Denton, there is a significant positive correlation between median accepted gap and flow, while for the merging manoeuvre at Tongham 1976 the correlation is negative. There are significant negative correlations between the variability parameter and flow in both cases. These differences in gap acceptance behaviour may be explained in terms of the different distributions of presented gaps at various major road flows at these sites. Further work is needed to clarify the relationships between gap acceptance and flow.

CHAPTER FIVE

MANOEUVRE TIME AND CLASS OF VEHICLE

Introduction

The gap acceptance behaviour of turning drivers is related to the acceleration capabilities of their vehicles: Bottom (1975) found that vehicles accepting short gaps covered the first 35 feet after their turn more quickly than those accepting long gaps, and he noted that the drivers of cars with larger engine capacities accepted shorter gaps. In a test-track experiment in which a vehicle's acceleration capabilities were reduced, drivers selected larger gaps for a crossing movement after the reduction (Evans and Herman, 1976). Similar behaviour is observed in pedestrians: Moore (1953) noted that a pedestrian increased his speed when an approaching vehicle was less than seven seconds away, while DiPietro and King (1970) reported that the shorter the gaps accepted by pedestrians, the faster their crossing speed.

Blunden, Clissold and Fisher (1962) studied simple merging at three T-junctions, and found that cars and goods vehicles combined accepted slightly longer gaps than cars alone. Ashworth and Green (1966) and Bottom and Ashworth (1978) considered the effect of different kinds of vehicles on gap acceptance at T-junctions, but they did not report any marked difference in behaviour. However, Powell and Glen (1978) found that goods vehicles accepted longer gaps than cars at roundabouts, and

Wagner (1966) indicated that a similar effect occurred at a crossroads site. These results are reasonably consistent, despite the different vehicle classifications used in each study. They suggest that goods vehicles accept longer gaps than cars, in a variety of manoeuvres at different intersections.

In this chapter, empirical relationships between gap acceptance and the time taken to complete a turning movement are derived for crossing cars at Denton and Puttenham, and for merging cars at Tongham 1976. Data collected at Compton, Denton, Puttenham and Tongham is used to examine the effects of different classes of vehicles on gap acceptance behaviour. Median accepted gaps for turning goods vehicles are shown to be significantly longer than those for turning cars at all these junctions, with the difference being greater for merging than for crossing. The differences between the median accepted gaps for different classes of turning vehicles are explained in terms of the manoeuvre times each class requires. The kind of vehicle approaching in the major road seems to affect merging behaviour; this may be due to perceptual errors associated with the size of the approaching vehicle.

RESULTS

Gap acceptance and manoeuvre time for turning cars

Tables of accepted and rejected lags and gaps were classified by the class of vehicle turning and its manoeuvre time. The manoeuvre time is the interval between the start of a turn and its completion; for crossing vehicles, this occurred when the rear of the vehicle cleared the path of oncoming traffic, and for

MANOEUVRE TIME Range	Mean	MEDIAN ACCEPTED GAP
1.36 - 1.65	1.52	2.56
1.66 - 1.95	1.80	2.98
1.96 - 2.25	2.09	3.96
2.26 - 2.55	2.38	4.67
2.56 - 2.85	2.68	4.81

Table 5.1 Crossing gap acceptance for cars at Denton

MANOEUVRE TIME Range	Mean	MEDIAN ACCEPTED GAP
0.91 - 1.20	1.08	3.21
1.21 - 1.50	1.38	3.54
1.51 - 1.80	1.65	4.15
1.81 - 2.10	1.94	3.91
2.11 - 2.40	2.24	4.33

Table 5.2 Crossing gap acceptance for cars at Puttenham

MANOEUVRE TIME Range	Mean	MEDIAN ACCEPTED GAP
2.66 - 2.95	2.80	3.03
2.96 - 3.25	3.10	3.76
3.26 - 3.55	3.40	3.95
3.56 - 3.85	3.69	4.51
3.96 - 4.15	4.00	5.18
4.16 - 4.75	4.41	4.80

Table 5.3 Merging gap acceptance for cars at Tongham

merging vehicles at Tongham, it occurred when the front of the vehicle reached an imaginary line in the major road 22.5 m upstream from the junction. Log-normal gap acceptance functions were fitted to the data for turning cars only (using the probit transformation).

Median accepted gaps for various ranges of manoeuvre time at each site are given in Tables 5.1, 5.2 and 5.3. In all cases, there are significant positive correlations between the median accepted gap for each range of manoeuvre times and the mean manoeuvre time in that range. A least squares fit to the data at each site gives the following linear relationships, where T is the median accepted time gap and M is the manoeuvre time (both measured in seconds):

(a) Denton, crossing, Table 5.1 ($1.36 \leq M \leq 2.85$),

$$T = 2.13\,M - 0.72 \tag{5.1}$$

($r = 0.98$, d.f. = 3, $p < 0.01$);

(b) Puttenham, crossing, Table 5.2 ($0.91 \leq M \leq 2.40$)

$$T = 0.90\,M + 2.33 \tag{5.2}$$

($r = 0.91$, d.f. = 3, $p < 0.05$);

(c) Tongham 1976, merging, Table 5.3 ($2.66 \leq M \leq 4.75$)

$$T = 1.21\,M - 0.11 \tag{5.3}$$

($r = 0.92$, d.f. = 4, $p < 0.01$).

Equations (5.1), (5.2) and (5.3) are plotted in Figure 5.1.

Class of vehicle turning

The gap acceptance behaviour of different classes of turning vehicles has been studied at several sites. For the crossing manoeuvre at Puttenham and for merging at Tongham 1978, there were sufficient data to consider turning vans and trucks separately.

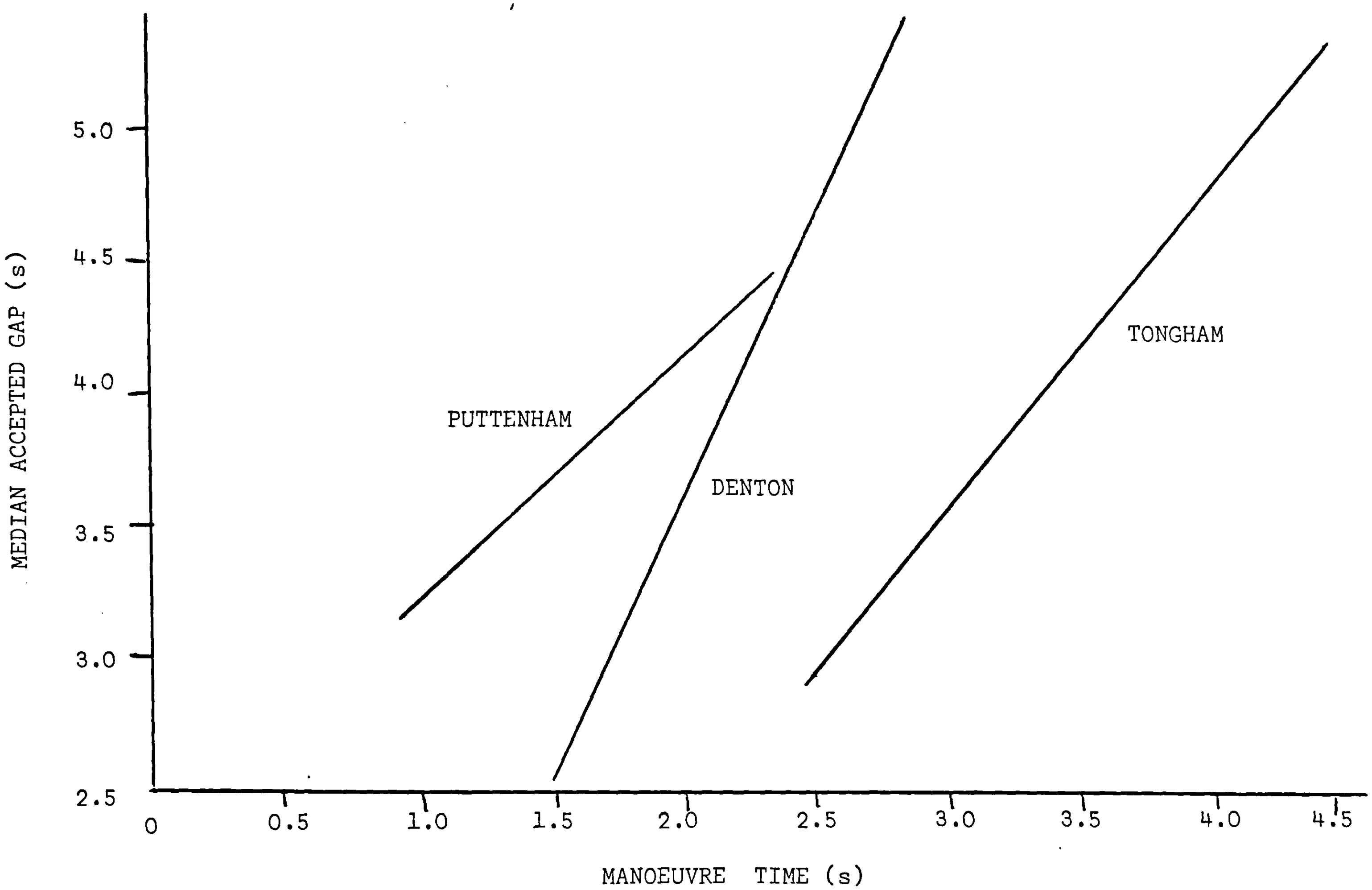


Figure 5.1 Median accepted gaps as functions of manoeuvre time (eqs. 5.1,5.2 and 5.3).

In the other cases data were sparse, and so vans and trucks were combined to give a single class of goods vehicles.

Median accepted gaps for different classes of turning vehicles are given in Table 5.4, for the case where the approaching major road vehicles are cars. For each manoeuvre, at each site, goods vehicles accept longer gaps than cars; the difference between the median accepted gaps for cars and goods vehicles is significant in most cases (using a test based on fiducial limits: Appendix 1). Where the data permits more detailed analysis, cars, vans and trucks accept increasingly longer gaps respectively. At Compton, Puttenham and Tongham 1978, the difference between cars and goods vehicles is more pronounced for merging than for crossing (no comparison between manoeuvres could be made for Denton and Tongham 1976).

Manoeuvre times

The differences in the observed gap-acceptance behaviour of different classes of turning vehicles can be explained in terms of the times required for them to complete their turning manoeuvre. Details of the manoeuvre time distributions of cars and goods vehicles for crossing at Denton and Puttenham and for merging at Tongham 1976 are given in Table 5.5; the difference between the means is significant at all sites ($p < 0.01$).

Median accepted gaps corresponding to the mean manoeuvre times for cars and goods vehicles at these sites, calculated from equations (5.1) to (5.3), are shown in Table 5.6, together with the appropriate empirical values from Table 5.4. The empirical and calculated values of the median accepted gaps for cars are similar at each site, even though the two values were derived from slightly different data

SITE	TURNING VEHICLES	MEDIAN ACCEPTED GAP
CROSSING		
COMPTON	Cars	3.83
	All goods	4.29
DENTON	Cars	3.90
	All goods	4.71
PUTTENHAM	Cars	3.73
	Vans	4.01
	Trucks	5.29
	All goods	4.63
TONGHAM 1978	Cars	4.21
	All goods	4.51
MERGING		
COMPTON	Cars	3.91
	All goods	4.63
PUTTENHAM	Cars	3.66
	All goods	5.33
TONGHAM 1976	Cars	4.31
	All goods	4.99
TONGHAM 1978	Cars	4.41
	Vans	4.56
	Trucks	5.09
	All goods	4.91

Table 5.4 Median accepted gaps for different classes of turning vehicles, when the approaching major road vehicles are cars.

SITE (Manoeuvre)	TURNING VEHICLE	MEAN	VARIANCE	SAMPLE SIZE
DENTON (crossing)	CAR	2.13	0.33	2204
	GOODS	2.61	0.60	184
PUTTENHAM (crossing)	CAR	1.48	0.23	356
	GOODS	2.21	0.80	35
TONGHAM (merging)	CAR	3.63	0.59	1209
	GOODS	4.27	0.79	262

Table 5.5 Manoeuvre time distributions for turning cars and goods vehicles

SITE (Manoeuvre)	TURNING VEHICLE	MEDIAN ACCEPTED GAP Empirical	Calculated
DENTON (crossing)	CAR	3.90	3.82 (Eq.5.1)
	GOODS	4.71	4.84 (Eq.5.1)
PUTTENHAM (crossing)	CAR	3.73	3.66 (Eq.5.2)
	GOODS	4.63	4.32 (Eq.5.2)
TONGHAM (merging)	CAR	4.31	4.27 (Eq.5.3)
	GOODS	4.99	5.06 (Eq.5.3)

Table 5.6 Empirical median accepted gaps when the approaching vehicle is a car (from Table 5.4), and median accepted gaps calculated from the appropriate equations for turning cars

sets: the empirical value was obtained for cars accepting gaps in front of approaching cars, while equations (5.1) to (5.3) were derived from data for all classes of approaching vehicles, and a limited range of manoeuvre times for the turning cars. The similarity between the empirical and calculated values for cars indicates that equations (5.1), (5.2) and (5.3) give an adequate representation of the relationship between gap acceptance and manoeuvre time at each site.

The values of the median accepted gaps for goods vehicles at Denton and Tongham 1976 predicted by equations (5.1) and (5.3) agree well with the empirical values, even though these equations were derived from data for turning cars only. This suggests that the drivers of turning cars and goods vehicles are behaving in similar ways in these manoeuvres, and that the differences in gap acceptance behaviour are due largely to differences in the manoeuvre times required by the different classes of vehicles. The agreement between empirical and calculated values of the median accepted gap for goods vehicles is not as close at Puttenham as at the other sites. However, data on manoeuvre times were only collected during evening peak periods at this site, while the empirical values in Tables 5.4 and 5.6 were derived from observations in both morning and evening peak periods. In addition, the sample size from which the manoeuvre time distribution of goods vehicles was calculated is rather small, and may not give a good estimate of the mean manoeuvre time.

Class of vehicle approaching

Median accepted gaps for turning cars and different classes of approaching vehicle are presented in Table 5.7. For crossing and merging at Tongham 1978 there were sufficient data to consider

SITE	APPROACHING VEHICLES	MEDIAN ACCEPTED GAP
CROSSING		
COMPTON	Cars	3.83
	All goods	3.90
DENTON	Cars	3.90
	All goods	3.78
PUTTENHAM	Cars	3.73
	All goods	3.56
TONGHAM 1978	Cars	4.21
	Vans	3.74
	Trucks	3.60
	All goods	3.65
MERGING		
COMPTON	Cars	3.91
	All goods	4.10
PUTTENHAM	Cars	3.66
	All goods	2.91
TONGHAM 1978	Cars	4.41
	Vans	3.95
	Trucks	3.61
	All goods	3.70

Table 5.7 Median accepted gaps for different classes of approaching major road vehicles, when the turning vehicles are cars.

approaching vans and trucks separately; in the other cases, all goods vehicles were considered in one class.

At Denton, Puttenham and Tongham 1978, the median accepted gap in front of approaching goods vehicles is less than that in front of approaching cars; at Compton, however, the reverse is true (although the difference in median accepted gaps is rather small). At all sites, the observed difference in values is larger for merging than for crossing. The results for Tongham 1978 indicate that shorter gaps are accepted in front of approaching trucks than in front of approaching vans, which suggests that the size of the approaching vehicle may be important.

Vehicle speeds at Denton

Mean speeds of major road cars and goods vehicles at Denton are shown in Table 5.8. In the previous chapter, we derived a relationship between gap acceptance and approach speed at Denton. The values of median accepted gaps in front of approaching cars and goods vehicles, predicted by equation (4.4), are also shown in Table 5.8. These results indicate that the median accepted gap in front of approaching goods vehicles should be longer than that in front of cars, which is not what we observed. A possible reason for the difference is misperception of the speed of oncoming large vehicles.

DISCUSSION AND CONCLUSIONS

Gap acceptance and manoeuvre time

Car drivers who accept short gaps complete their manoeuvres more quickly than those who accept long gaps. This effect has been observed at several T-junctions, for both crossing and

APPROACHING VEHICLES	MEAN SPEED (mph)	CALCULATED MEDIAN ACCEPTED GAP
CAR	36.5	4.05
GOODS	34.8	4.15

Table 5.8 Mean speeds and calculated median accepted gaps for approaching cars and goods vehicles at Denton.

merging turns. The results of Bottom (1975) and Evans and Herman (1976) indicate that this variation in gap acceptance response is related to the acceleration capabilities of cars. This would appear to reflect a deliberate strategy of drivers. The alternative explanation, that drivers accept short gaps in error and then must increase their acceleration to compensate, is less reasonable, particularly for the crossing manoeuvre ; considering the short time intervals involved and the generally limited range of acceleration responses of any particular vehicle, it is unlikely either that errors would be recognised in time for major compensatory action to be initiated, or that vehicles would respond sufficiently quickly to produce large observable changes in manoeuvre times.

Class of vehicle turning

The drivers of other classes of vehicles seem to behave similarly to the drivers of cars in the way they adapt their gap acceptance responses to the capabilities of their vehicles. The drivers of cars, vans and trucks accept increasingly longer gaps, corresponding to the generally increasing size and decreasing acceleration of vehicles in these classes. In addition, the median gaps accepted by goods vehicles are comparable to those that would be accepted by cars with the same manoeuvre times, for both crossing and merging turns. The observed differences between cars and goods vehicles are larger for merging than for crossing turns at those junctions where comparison is possible. This is to be expected since differences in acceleration capability would have a more noticeable effect on the merging manoeuvre, where vehicles must attain the speed of oncoming major road traffic after turning.

Class of vehicle approaching

The median accepted gap for merging cars and approaching goods vehicles is shorter than that for approaching cars. This may be due to misperception of the speed of approaching goods vehicles, but driver strategy could also be important for this manoeuvre. A driver may decide to risk accepting a shorter gap in front of an approaching goods vehicle rather than be caught in a platoon behind it. Alternatively, he might expect professional drivers to be more courteous on the road, and so allow vehicles to join the traffic stream in front of them. Further work may be needed in this area, to clarify the reasons for the observed differences in behaviour.

CHAPTER SIX

OCCUPANTS OF TURNING VEHICLE

Introduction

Characteristics of the turning vehicle's occupants such as the gender of the driver and the number of passengers may affect gap acceptance behaviour. Two studies in San Diego (California) examined the gap acceptance behaviour of men and women drivers. Ebbesen and Haney (1973) observed drivers leaving a large car park at a shopping centre, while Ebbesen, Parker and Konecni (1978) studied the behaviour of drivers turning left at four different T-junctions. The results of both studies show that males accept shorter gaps than females; the difference in median accepted gaps typically amounted to half a second. Similar differences are observed between male and female pedestrians (Cohen, Dearnaley and Hansel, 1955; DiPietro and King, 1970).

Recent research by Hills at TRRL investigated the behaviour of individual men and women drivers. Subjects seated in a vehicle parked at right angles to one side of a dual carriageway were asked to indicate the "last possible safe moment to cross" in front of an approaching vehicle. The results indicated that females required longer minimum gaps than males. Hills' data has also been analysed by Darzentas, McDowell and Cooper (1980), who showed that females are less consistent in their behaviour than males.

Ebbesen and Haney also investigated the effects of the presence of passengers in the turning vehicle. They showed that drivers generally accepted shorter gaps after being forced to wait in line approaching the intersection, but this effect was less marked if the vehicle contained passengers. However, no significant difference in gap acceptance behaviour was detected in the no-waiting situation.

The gap acceptance behaviour of men and women drivers is compared in this chapter, using data collected at Compton, Puttenham and Tongham (for both crossing and merging manoeuvres). In all cases, women drivers have longer median accepted gaps than men. The effect of the presence of passengers in the turning vehicle is also investigated at Compton. Only small samples were considered, but the results indicate that the presence of passengers significantly affects the behaviour of crossing drivers. In addition, different effects may occur for male and female drivers.

DATA ANALYSIS

Arrivals and departures of turning vehicles at Compton were coded by the gender of the driver (as assessed by individual observers) and the presence of passengers, whereas those for Puttenham and Tongham were coded by the kind of vehicle and the gender of its driver. Details of the proportions of men and women drivers in the turning streams at each site are given in

SITE	TURNING POPULATION	MEN Number	%	WOMEN Number	%
CROSSING					
COMPTON	No passengers	281	89	33	11
	Passengers	107	83	22	17
	All vehicles	388	88	55	12
PUTTENHAM	Cars	521	88	71	12
	Non-cars	137	97	4	3
	All vehicles	658	90	75	10
TONGHAM	Cars	537	82	118	18
	Non-cars	106	100	0	0
	All vehicles	643	84	118	16
MERGING					
COMPTON	No passengers	288	88	41	12
	Passengers	111	78	32	22
	All vehicles	399	85	73	15
TONGHAM	Cars	671	83	137	17
	Non-cars	126	98	3	2
	All vehicles	797	85	140	15

Table 6.1 Proportion of men and women drivers in turning populations at each site. "Non-cars" includes all vehicles not classified as cars.

Table 6.1; in general, about one sixth of the turning drivers are women, and the majority of vehicles other than cars are driven by men. In addition, the proportion of women drivers who carry passengers is higher than that for men.

Tables of accepted and rejected lags and gaps were obtained for various classifications of the turning vehicle and its occupants (which depended on the data available at each site). Log-normal gap acceptance functions were fitted to the data in each classification using the probit technique (Appendix 1). Results are presented in Tables 6.2 and 6.3.

RESULTS

Gender of driver

Median accepted gaps for men and women drivers at Compton could only be derived for all turning vehicles, while turning cars could be considered separately at Puttenham and Tongham (Table 6.2). In all cases, the median accepted gap for women drivers is longer than that for men; the size of the difference varies with site and manoeuvre, and with the class of turning vehicles considered. A valid comparison between the gap acceptance behaviour of men and women drivers may only be made for turning cars, since the majority of drivers of vehicles other than cars are men. The differences between the median accepted gaps for men and women drivers of cars at Puttenham and Tongham are significant for both manoeuvres (using the test based on fiducial limits outlined in Appendix 1). In addition, the populations of men and women drivers exhibit similar variability in behaviour.

SITE (Turning Vehicles)	MEN		WOMEN	
	Median Accepted Gap	Variability Parameter	Median Accepted Gap	Variability Parameter
CROSSING				
COMPTON (All)	3.59	1.54	3.60	1.48
PUTTENHAM (Cars)	3.41	1.47	3.68	1.53
(All)	3.56	1.51	3.66	1.52
TONGHAM (Cars)	3.78	1.44	4.20	1.38
(All)	3.87	1.45	4.20	1.38
MERGING				
COMPTON (All)	4.01	1.57	4.33	1.54
TONGHAM (Cars)	3.78	1.61	4.41	1.57
(All)	3.90	1.64	4.11	1.89

Table 6.2 Gap acceptance parameters for men and women drivers and different classes of turning vehicles.

DRIVERS	NO PASSENGERS		PASSENGERS	
	Median Accepted Gap	Variability Parameter	Median Accepted Gap	Variability Parameter
CROSSING				
MEN	3.38	1.53	4.03	1.58
ALL	3.40	1.51	3.96	1.60
MERGING				
MEN	4.10	1.65	3.84	1.40
WOMEN	4.44	1.54	4.17	1.54
ALL	4.15	1.63	3.92	1.43

Table 6.3 Gap acceptance parameters for turning vehicles at Compton, classified by the vehicles' occupants.

Presence of passengers

Gap acceptance parameters for turning vehicles at Compton which did or did not contain passengers are shown in Table 6.3. Men and women drivers could be considered separately for the merging manoeuvre, but there was insufficient data to look at crossing females as a separate class. The difference in median accepted gaps for the crossing manoeuvre is significant, and single drivers accept shorter gaps than drivers who carry passengers. The merging behaviour of men drivers with and without passengers can not be compared, because the variability parameters are rather different: the male members of the merging population are more consistent in behaviour when passengers are in the vehicle. Women drivers exhibit similar variability in merging behaviour whether or not they carry passengers; however, the difference in median accepted gaps is not significant.

DISCUSSION

Gender of driver

Women drivers accept longer gaps than males for all situations studied. These results confirm those obtained by Ebbesen et al (1973 and 1978), although the size of the difference we observed was generally less than half a second; this probably reflects the different manoeuvres studied, and the different driving conditions in the two countries. Men and women drivers exhibit similar variability in gap acceptance behaviour, so women drivers are consistently more cautious than males. Darzentas, McDowell and Cooper (1980) showed that females were less consistent in behaviour, which does not agree with our results. However, the data they used was collected in a rather artificial situation, since subjects did not actually carry out the manoeuvre, and the sample sizes they considered were rather small.

Kind of vehicle turning

We have already shown that gap acceptance behaviour varies with the kind of vehicle turning (Chapter 5). The difference in the values in Table 6.2 for cars and all vehicles at Puttenham and Tongham shows that the inclusion of data for vehicles other than cars has biassed the results considerably (particularly for women drivers). This emphasises the importance of removing any extraneous factors which may affect the results when comparing two samples.

Presence of passengers

Because of the nature of the data collected at Compton, all turning vehicles had to be considered in this analysis. As we have noted above, this probably affects the validity of our results. However, the presence of passengers in the turning vehicle seems to have an effect on gap acceptance behaviour, which may be different for men and women drivers. This result was obtained from relatively small samples observed at a single site; clearly, further observations are needed to clarify the effect of passengers on drivers' turning behaviour.

CONCLUSIONS

Median accepted gaps for women drivers are significantly longer than those for men in both crossing and merging manoeuvres. Men and women drivers exhibit similar variability in behaviour, so women are consistently more cautious than men. The presence of passengers in the turning vehicle may also affect gap acceptance behaviour. The need for further work investigating the effects of passengers on the turning behaviour of men and women drivers is noted.

CHAPTER SEVEN

QUEUE ACCEPTANCE

Introduction

The acceptance of gaps by queues of vehicles is important in assessing the capacity of junctions and freeway entrance ramps, particularly in peak periods. We define queue acceptance as the acceptance of a large gap in a major road traffic stream by two or more waiting minor road vehicles, where the minor road queue is not exhausted.

Pearson and Ferreri (1961) examined queue acceptance in terms of the percentage of gaps of a given size accepted by streams of vehicles entering a freeway. From their gap acceptance distributions, they derived a linear relationship between N, the number of vehicles entering, and T, the gap length in seconds:

$$N = 0.28T - 1.07. \qquad (7.1)$$

They claim a high correlation coefficient for this relationship, but the method of derivation is not clear.

Bendtsen (1972) studied queues of turning vehicles at the intersection of a freeway exit ramp with a primary road. His main concern was with measurement techniques, and the intervals between successive vehicles. Uber (1978) considered the behaviour of queues of turning vehicles moving into large gaps at a T-junction controlled by a STOP sign. He derived an expression relating T and N based on the median start-up times of the first and subsequent vehicles making the turn, and the "median remainder rejected lag":

$$N = 0.29T - 0.74. \qquad (7.2)$$

Two methods for deriving queue acceptance relationships are compared in this chapter. Empirical data for the simple crossing manoeuvre at Denton and the merging manoeuvre at Tongham (1976) are used to illustrate the two methods. A direct linear relationship and an explanatory model are presented, and then compared and discussed in detail. We conclude that the explanatory model is the better of the two methods we have described. A version of this chapter has been published in Cooper and Wennell (1978).

A DIRECT LINEAR RELATIONSHIP

The simplest approach to queue acceptance would appear to be a direct examination of the length of the time gap T accepted by N vehicles from a queue. The results of previous workers (Pearson and Ferreri (1961); Uber (1978)) suggest that there is a linear relationship between T and N. The simple linear form, when fitted to the data, gives:

(a) merging at Tongham ($2 \leqslant N \leqslant 6$)

$$T = 2.8N + 4.9 \qquad (7.3)$$

($r = 0.76$, d.f. $= 34$, $p < 0.01$);

(b) crossing at Denton ($2 \leqslant N \leqslant 5$)

$$T = 3.8N - 0.1 \qquad (7.4)$$

($r = 0.95$, d.f. $= 14$, $p < 0.01$).

Note that we have treated N as an independent variable and T as a dependent variable; to regard T as independent is not appropriate for this data, which is sampled from continuous distributions of gap sizes for fixed, integer, values of N.

A simple examination of the correlation coefficients would imply that a linear relationship is reasonable. However, it must be noted that, like many distributions that occur in the study of traffic, the distribution of the lengths of gaps accepted by a given number of vehicles is markedly skew (see, for example, Figure 7.1). Thus the normality assumptions of any linear regression model are likely to be violated; indeed, we prefer not to use the term "regression". Because the direct linear relationship may still be suspect we look for a different kind of model to describe this data.

THE EXPLANATORY MODEL

An alternative to the simple linear relationship may be constructed from the components of the queue acceptance process. The sequence of events we are considering is initiated by the arrival at the junction of a vehicle in the major road, m_1, when there is a queue of vehicles waiting to turn. The first N queueing vehicles, a_1, a_2, ..., a_N, then turn, while the next one, r, stops. The gap is closed by the next vehicle, m_2, in the major road. The times at which these events occur are $t(m_1)$, $t(a_1)$, ..., $t(a_N)$, $t(r)$ and $t(m_2)$.

The intervals between the events in this sequence can be classified in three distributions: the start-up time of the first turning vehicle, $t(a_1) - t(m_1)$; the move-up time of subsequent vehicles, $t(a_i) - t(a_{i-1})$, $i = 2, \ldots, N$, $t(r) - t(a_N)$; and the residual lag which is rejected, $t(m_2) - t(r)$. All these distributions are skew, and, in line with most simple gap

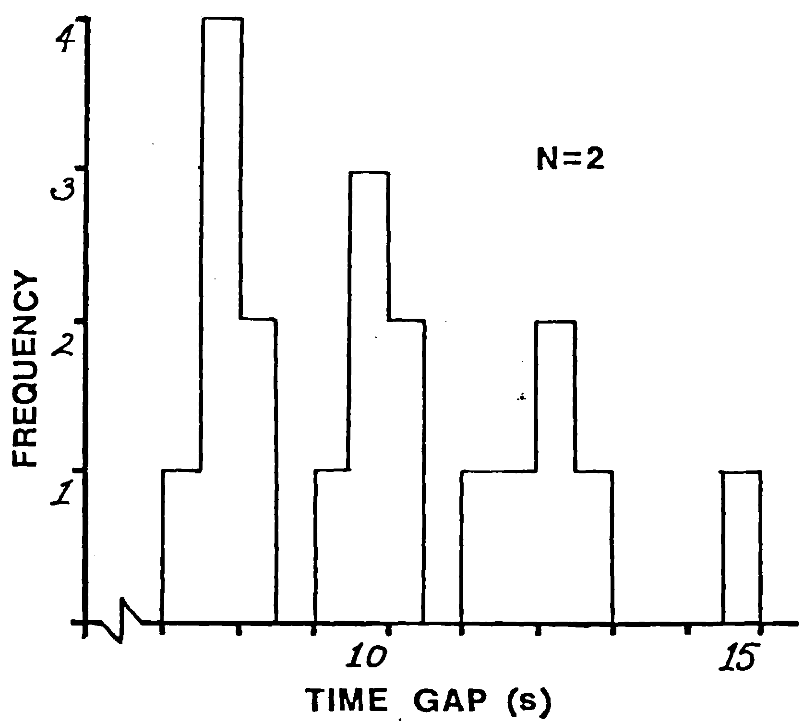


Figure 7.1 The distribution of time gaps accepted by two merging vehicles at Tongham.

acceptance measurements (Ashworth (1970); Miller (1972)), and good statistical practice, we choose the median value for our calculations. We can now construct the time gap T accepted by the N vehicles as the median start up time S, plus the median move-up time M for each of the next N vehicles, plus the median residual lag R:

$$T = S + N.M + R \qquad (7.5)$$

We can use more data to form these distributions than is available from analysis of the queue acceptance process alone. For example, there is evidence that the gap acceptance behaviour of a turning vehicle does not depend on the presence of vehicles waiting behind it (Ebbesen and Haney, 1973), and it is unlikely that the starting behaviour will be affected either. Thus the start-up times for <u>all</u> vehicles which accept gaps may be included in the distribution. Similarly, all rejected lags may be included in the residual lag distribution.

We have assumed above that the move-up times of queueing vehicles are independent of whether or not they turn immediately. Previous results of Pearson and Ferreri (1961), Bendtsen (1972), Greenshields (quoted by Bendtsen) and Uber (1978) are by no means consistent. Our own results also show some variation. Under these circumstances, our assumption does not seem unreasonable. It enables us to include in the move-up time distribution data derived from queues of two or more vehicles of which only the first one turns. Move-up times are considered again in more detail below.

Empirical results are shown in Table 7.1. From these we derive the relationships

$$T = 3.0\,N + 3.0 \qquad (7.6)$$

for the merging manoeuvre at Tongham, and

$$T = 2.4\,N + 2.9 \tag{7.7}$$

for the crossing turn at Denton.

DISCUSSION

The linear form appeared to be the simplest method for estimating the queue acceptance relationship directly. Although it seemed to produce good results, a closer inspection of the data revealed that the normality assumptions required for a regression model were violated, and so any attempt to regard this as a linear regression model would be quite wrong. This illustrates the danger of using linear regression as a convenient tool (in terms of statistical arithmetic) without ensuring that the assumptions of this particular statistical model are satisfied. In addition, the results given by this method are merely descriptive of the queue acceptance relationship, and cannot be used for detailed analysis.

The explanatory model is more appropriate to the data and produces a more useful result. It enables the effect of changes in the individual components of queue acceptance on the overall relationship to be evaluated, and so provides a useful tool for traffic engineers. For example, S and R may be affected by structural changes to a junction or by improvements in visibility, while M may be affected by changes in the performance of the vehicle or its driver.

Flow effects

In many gap acceptance studies, the proportion of gaps of a given size that are accepted is of interest. The proportions are obtained from observations of drivers, each driver

	TONGHAM		DENTON	
Distribution	Median	Sample Size	Median	Sample Size
Start-up time for first in gap (S)	1.7	417	1.1	572
Move-up time for second in gap	2.9	236	2.3	268
Move-up time for third in gap	3.2	133	2.6	88
Move-up time for fourth and following vehicles in gap	2.9	156	2.9	71
Move-up time of all following vehicles in gap (M)	3.0	525	2.4	427
Residual lag (R)	1.3	432	1.8	546

Table 7.1 Medians of the distributions contributing to the queue acceptance process.

	TONGHAM (merging)	DENTON (crossing)
Median (M)	3.0	2.4
Mean	3.1	2.7
Variance	1.06	1.28
Sample size	525	427

Table 7.2 Move-up time distributions.

contributing a sequence of rejected gaps before he turns, and a single accepted gap when he makes his turn. Under conditions of high flow, there are more small gaps presented to the turning driver than at low flows, and, in general, he will reject more of them before one that is long enough is presented to him and accepted. Thus the derived gap acceptance distribution depends on the distribution of presented gaps, which in turn depends on the traffic volume in the major road. Ashworth (1968) has shown how to calculate the flow bias in the observed gap acceptance functions which results from this effect.

It is fortunate that neither of the two models presented in this chapter leads to bias effects of this kind. The direct linear relationship is based on acceptances, which do not depend on the presented gap distribution, while the explanatory model uses the residual lag distribution, again independent of flow. However, there are some flow effects, not related directly to the presented gap distribution, which should be considered.

Wagner (1966) examined the mean start-up times for vehicles accepting both lags and gaps in peak and off-peak periods. In peak periods, when the flow was presumably high, he observed significantly shorter start-up times in each case. Uber (1978) investigated flow effects explicitly: he found no relationship between the start-up time of the first turning vehicle and flow, but the move-up times of subsequent vehicles decreased with increasing major road flow. Thus both the S and M terms in equation (7.5) may be flow dependent. This is not a bias effect in Ashworth's sense, arising in the methods employed for observation and analysis, but a behavioural effect.

There are two possible explanations for these observations: there may be a true change in behaviour in a single population; or the observations may be of different populations. Wagner's results, derived in peak and off-peak periods, are likely to reflect different driving populations. The peak period population is composed largely of males travelling to and from work, while the off-peak population might contain more housewives on shopping and school trips, and more professional and commercial drivers (Storie, 1977); these groups are known to differ in their driving characteristics (Ebbesen and Haney (1973); James and Goldman (1971)). It is harder to apply this argument to Uber's results, which are all derived from observations made at or near the morning peak period. The change in move-up times appears to be a true behavioural change, possibly caused by a desire to turn quickly rather than risk an extended wait for a longer acceptable gap. Why, then, was no similar effect observed in the start-up times of the first queueing vehicle to turn? Further empirical evidence is needed in this area.

Traffic volume in the major road will influence the results obtained by both models. However, with the explanatory model we can examine flow effects in greater detail, and explain more satisfactorily the phenomena we observe.

<u>Data use</u>

The two models we have proposed use different amounts of the large quantity of data available from intersection observations. The direct linear relationship makes use of the information about queues only: the number of vehicles from each queue who accept a particular gap and the size of that gap. Much of this data, at least in the observations we conducted, relates to small queues of two or three turning vehicles. The explanatory model, as we have noted, uses far more information about traffic behaviour, drawn from a wider data base. Each of the components of the model, the start-up time distribution, the move-up time distribution and the residual lag distribution, may contain data from manoeuvres other than queue acceptance. In its use of data, the explanatory model appears preferable, enabling reliable and more representative results to be obtained from shorter periods of observation.

The Pearson and Ferreri (1961) method makes quite different use of the available data. Like the simple linear relationship, it examines queue acceptance directly, but it uses both accepted gaps <u>and</u> rejected gaps to derive gap acceptance relationships for queues of different lengths. As the method they use to derive the linear relationship between T and N (equation 7.1) is not specified in detail, the Pearson and Ferreri technique cannot be compared directly with the two methods outlined in this chapter. In addition, their result is not corrected for the flow bias which we have noted above is present in all empirical gap acceptance distributions, and so a comparison of the numerical values is not possible either.

<u>Merging and crossing turns</u>

The explanatory model enables the results from the merging queue acceptance at Tongham to be compared with the crossing acceptance at Denton. Table 7.1 indicates that S, M and R all have different values at the two sites. The differences between the start-up times, and between the residual lags, may be due entirely to the method of data collection. As we have noted previously (Cooper, 1976), the measurement of lags depends critically

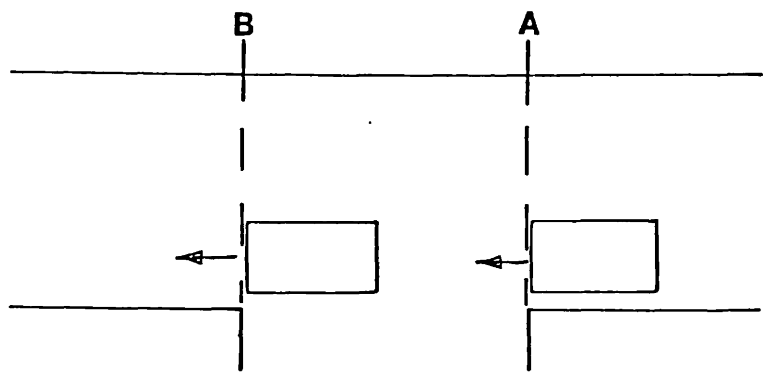


Figure 7.2 Possible recording positions for major road arrivals.

on the point within the junction at which the arrival of a major road vehicle is recorded: for example, observations at A and B in Figure 7.2, say 22 feet apart, would lead to differences in measurement of 0.5 seconds at speeds of 30 mph in the major road. Consistency between recordings of major road vehicle arrivals at a specific junction is relatively easy to achieve; here, errors of this kind are negligible. However, variations in experimental techniques and observational positions at different junctions mean that inconsistencies of this kind may arise when different junctions are compared. The situation is further complicated by the fact that the turning vehicle is at different locations within the junction for merging and crossing: even if the recording of major road vehicles is consistent, a merging vehicle must wait longer for the major road vehicle to clear its path than a crossing vehicle (Figure 7.3), affecting the measured start-up times. Thus neither S nor R should be compared in Table 7.1.

It is reasonable, however, to compare the move-up times for the two manoeuvres, as these do not depend on the position of the major road vehicle. The distributions of move-up times are not markedly skew (at least for Tongham, Figures 7.4 and 7.5) and it is possible to conduct the analysis in terms of the normal distribution. The relevant data are shown in Table 7.2; the means are significantly different at the two sites ($z = 5.65$; $p < 0.001$).

There are a number of explanations for the difference in the move-up times for merging and crossing. First, the merging manoeuvre is a more difficult task than the crossing manoeuvre, since it depends to a greater extent on the speed of approaching major road vehicles (Chapter 4). As a result, the driver's

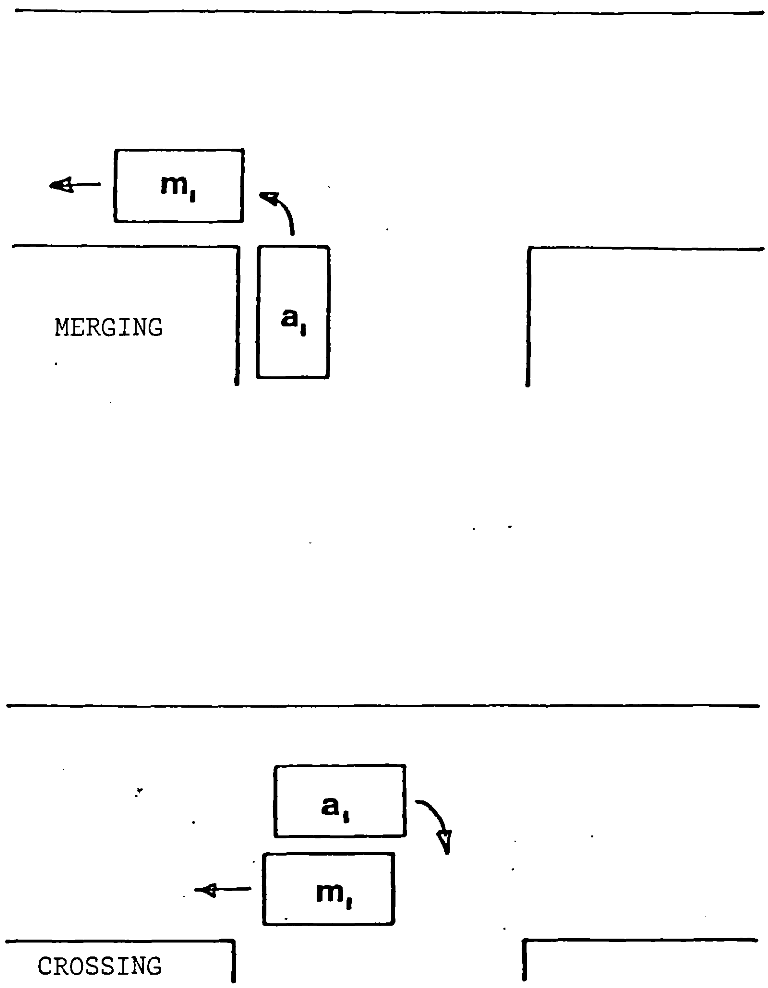


Figure 7.3 The earliest possible start of a turn for merging and crossing vehicles.

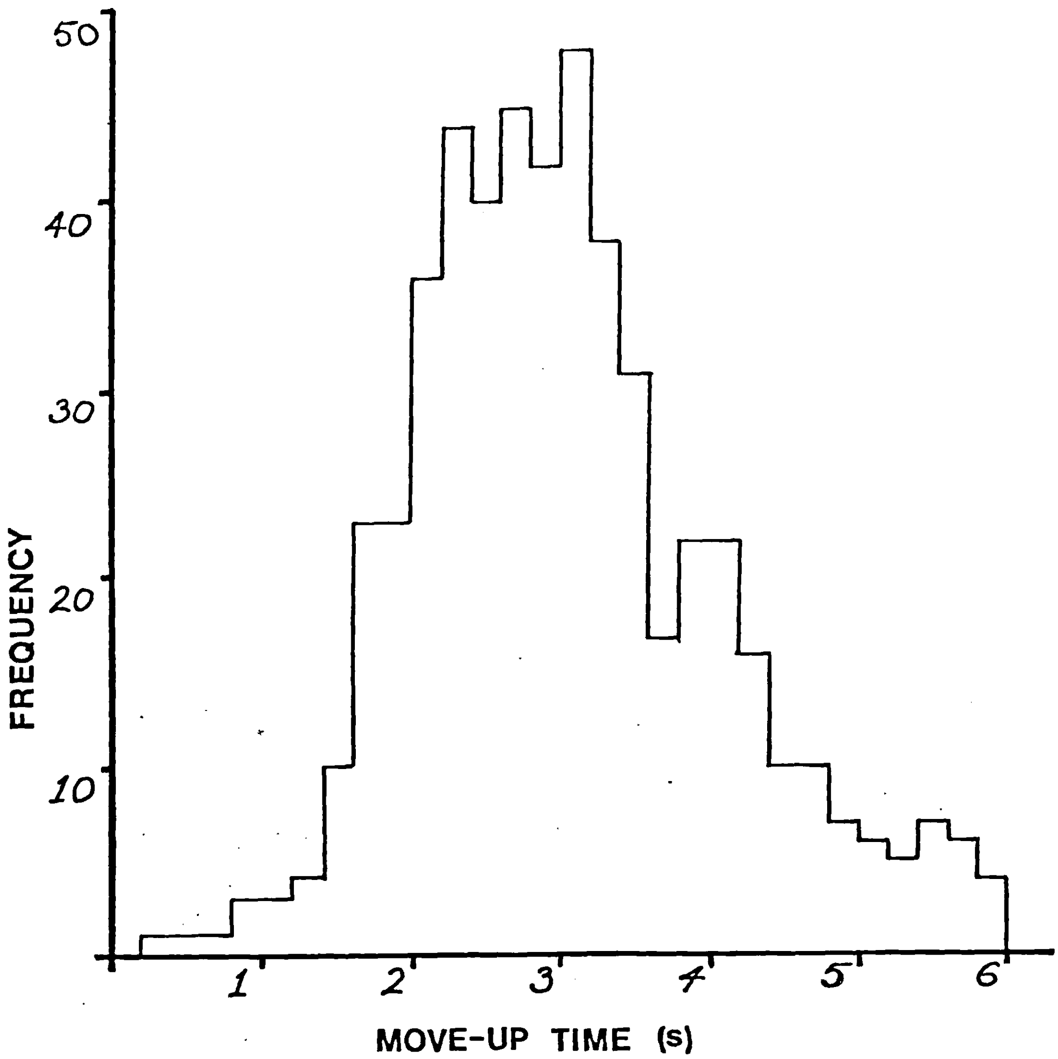


Figure 7.4 Distribution of move-up times for merging vehicles at Tongham.

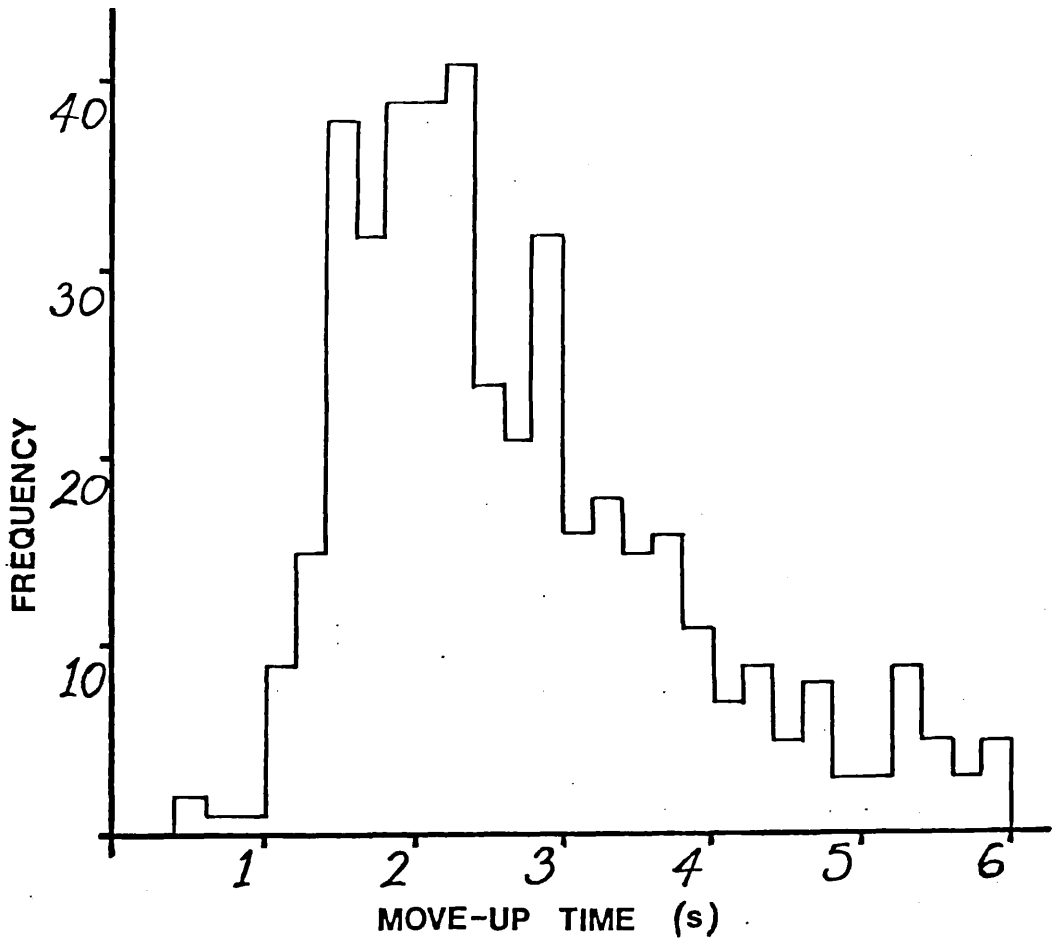


Figure 7.5 Distribution of move-up time for crossing vehicles at Denton.

decision time may be longer for merging, so increasing the interval between consecutive turns. The geometry of the intersection provides a second explanation; a driver making the simple crossing turn we are considering is located in the centre of the major road, generally with a better view of approaching vehicles than a merging driver (Figure 7.3). Thus the crossing driver may be able to make his decision to turn or not before he reaches the location at which the turn physically commences. On the other hand, the merging driver may have problems seeing the oncoming major road traffic before he reaches the "Give Way" line, particularly if there are minor road vehicles beside him waiting to turn right, and he may be unable to participate.

A third, less obvious, explanation is related to the detailed movements of crossing vehicles as they turn. Often, the leading vehicle in a queue a_1, has moved well forward into the junction before it is able to turn, and the second vehicle, a_2, is also within the junction. When a_1 is able to turn, he turns sharply; a_2 is able to start his turn almost at once, as he generally follows a different line through the junction, "cutting the corner" as in Figure 7.6. Subsequent vehicles may also cut the corner. As a result, the move-up time of the second vehicle to turn, $t(a_2) - t(a_1)$, may be artificially short. Since the majority of move-up times are derived from the second vehicle in the queue (Table 7.1), the median move-up time will tend to be shorter. This effect cannot occur in the case of merging.

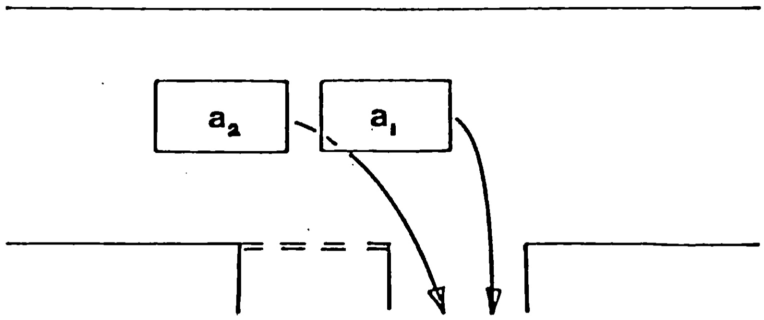


Table 7.6 Possible paths of crossing vehicles.

CONCLUSIONS

The explanatory model is better than the direct linear relationship for the analysis of queue acceptance; it is more appropriate to the data, as the distributions of accepted gaps are skew, violating any regression assumption; It is more useful for diagnosis and the prediction of the effect of changes in the intersection and its environment, since the individual components of the process are included explicitly; and it makes better use of the available data.

The explanatory model may also be more useful than that of Pearson and Ferreri (1961), since it does not contain bias effects due to the flow level of traffic in the major road. Traffic volume appears to have other behavioural effects apart from causing an observational bias; with the explanatory model, these effects can be identified more clearly. The need for further observations on the effects of flow is noted.

Using the explanatory model, the time gap T seconds required by N vehicles turning from a queue was found to be:

$$T = 3.0N + 3.0$$

for merging at Tongham, and

$$T = 2.4N + 2.9$$

for crossing at Denton. Although it is unwise to compare these relationships numerically with regard to the start-up and residual lag components, a closer examination of the move-up component for the two cases provides interesting insights into the detailed functioning of the intersections, and indicates some of the reasons for the different results for the merging and crossing turns.

CHAPTER EIGHT

VALIDATION OF THE CONFLICT SIMULATION MODEL

Introduction

A simulation model of traffic conflicts at T-junctions has been developed at Royal Holloway College. This model has not been validated against an accepted measure of safety, so a study was undertaken to attempt to relate model conflicts to the number of recorded injury accidents at several sites. Details of flows, speeds and gap acceptance were obtained for a number of non-urban T-junctions, and this data was used as input to the simulation model. The conflicts predicted by the model were then compared with the injury accident records for each junction.

The simulation model is described below, and the various parameters required to run the model are detailed. The problems associated with validating such a model are then outlined, and the method of validation chosen is discussed. Complete validation of the model requires a long term effort, so only preliminary results can be presented in this chapter. However, model conflicts involving crossing and merging vehicles are shown to be good predictors of accident risk at the six T-junctions at which data was collected with the microprocessor-based system (Appendix 2). Denton was not included in the validation study because data was collected there in 1975, when driver behaviour may have been affected by the fuel crisis.

THE SIMULATION MODEL

A simple type of priority junction is modelled: a non-urban T-junction with the structure shown in Figure 8.1. This model was developed from a preliminary approach by Ferguson (1973) and subsequent improvements by Cooper and Ferguson (1976) and Darzentas, Cooper, Storr and McDowell (1980). A detailed description of the model is given by Darzentas et al (1980).

Model environment

The T-junction is assumed to be located in a non-urban environment; drivers have an essentially infinite line of sight, and no pedestrians are permitted. All vehicles have similar characteristics, and accelerate and decelerate uniformly. Lane-changing and overtaking are prohibited in the junction. All drivers must maintain at least a minimum headway between their vehicle and the previous one, which may increase as speed increases.

Six possible vehicle movements are permitted at the junction (Figure 8.1), which is controlled by a "Give Way" sign. All turning drivers yield right-of-way to priority vehicles on the major road, so queues are allowed to form in the major or minor road as vehicles wait for an opportunity to turn. A single queue of crossing vehicles may form in the major road, and two parallel queues may form in the minor road: one for vehicles waiting to turn left, the other for vehicles waiting to turn right. The maximum length of these queues

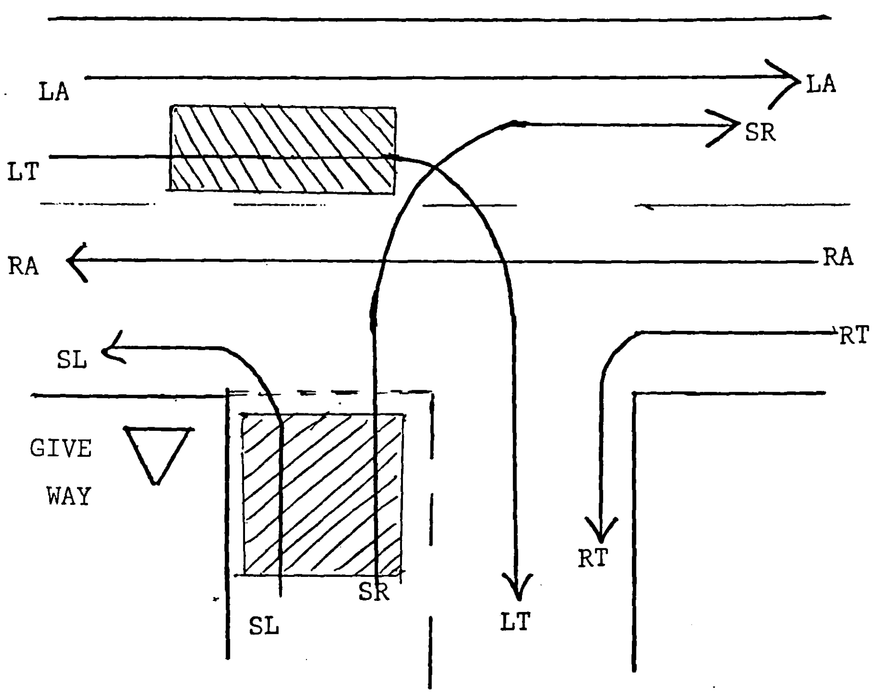


Figure 8.1 A standard T-junction, showing the six streams of traffic considered by the model. Shaded areas indicate locations where queues may form.

may be varied, and both queues in the minor road are fed from a single stream of traffic.

Gap acceptance process

Only the leader of a queue may attempt to turn: if it has just arrived at the junction, it may turn while "rolling", otherwise it starts the turn from rest. Drivers performing simple crossing or merging manoeuvres are presented with a series of gaps between vehicles in the nearside stream of major road traffic. 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. The gap acceptance decision of drivers waiting to turn right out of the minor road depends on a pair of presented gaps (nearside and farside). The decision process for this manoeuvre is currently being modified as a result of recent research by Storr, Cooper and McDowell (1980).

Conflict occurrence

Each driver is assigned a manoeuvre time, which is the time needed to complete the turning movement. For crossing vehicles, the manoeuvre time is sampled from an empirical distribution of exposure times; for merging vehicles, the manoeuvre time corresponds to the time taken to attain the vehicle's preferred speed. 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 collision, and a conflict occurs. The severity of the resulting conflict depends on the rate of deceleration required. Only decelerations in the original line of motion are considered, since lane-changing is not permitted in the model, but Balasha, Hakkert and Lirneh (1979) have indicated that transverse decelerations may be neglected.

Input parameters

Several input parameters are currently held constant for all runs, although the facility exists to vary them. They are the standard vehicle acceleration and deceleration; the turning speed of vehicles which are rolling at the start of a turn; the move-up time of vehicles in a queue; maximum queue lengths; the rates of deceleration which determine the severity grade of a conflict (see below); the initialisation time and the simulated time (10 hours); and the number of runs (usually 10) for each set of parameters (random number seeds are different for each run).

The following parameters are generally varied for each set of runs:

i) flow in each traffic stream (in vehicles per hour);

ii) mean and standard deviation of speed distribution (in feet per second);

iii) gap acceptance parameters;

iv) initial random number generators.

Conflict severity

Five grades of severity are currently used in the model. The ranges of deceleration which determine the severity grade of a model conflict are as follows:

Severity Grade	1	2	3	4	5
Deceleration (feet/second^2)	< 5	5-10	10-15	15-20	> 20

VALIDATION

Any model is by definition a simplified representation of the real system of interest. The validity of a model may be tested by comparing its results with those derived from observations of the actual system it represents. Initial stages of validation are concerned with checking that distributions of parameters used in the simulation correspond to those observed in the system that is modelled, and that the results produced are similar for different random number generators. In addition, any results produced by the model should agree with accepted hypotheses about the system it represents. These aspects of validation are discussed in detail by Darzentas, Cooper, Storr and McDowell (1980).

This chapter is concerned with the external validation of the model ie does risk predicted by the model correspond to some measure of safety at junctions? The main problem in validating this model is that no direct measure of accident risk actually exists, so there is no simple criterion against which the model's output can be tested. However, if the results predicted by the model are to be accepted by others, we must show that they correspond in some way to a measure of safety already in use.

Perhaps the most obvious method of validation would be to compare conflicts predicted by the model with conflicts observed on the road. However, the assessment of conflicts by observers is a very subjective technique, and the relationship between observed conflicts and accidents is still a matter of some controversy (Proceedings of Second International Workshop on Traffic Conflicts, 1979). Direct validation of simulated decelerations against actual decelerations of vehicles involved in conflict requires extensive instrumentation of a site, but it is feasible. This would only be the first stage in a validation study, however, since observed decelerations would then have to be related to accidents. Such a process is indirect and difficult, so an alternative method of validation was sought.

The most common measure of safety currently in use is recorded injury accidents. There are many problems associated with using such statistics, but they provide the only direct measure against which the model can be validated. Accidents are rare events, and so data must be collected for a long time period at a given location to provide reasonable statistics. Only accidents involving injury need be reported in the UK, and so most damage-only accidents do not appear in the records. In addition, the information contained in accident records is often very sketchy, and reporting methods vary from one county to another, so it is almost impossible to locate accurate details of the accident history for a particular location. Despite all these drawbacks, recorded injury accidents are the criterion against which we have chosen to validate the simulation model.

Validation process

The process of validation detailed in this chapter compares risk predicted by the model with recorded accidents at a number of sites. The criteria by which sites were selected for inclusion in the study are outlined in the next section. Data collected at these sites has been analysed to give the relevant parameters needed to run the model. Since accidents do not provide ideal statistics, no attempt is made to derive a mathematical relationship between model conflicts and accidents. The sites are ranked in terms of conflicts predicted by the model, and also by the number of accidents; the agreement between these two rankings demonstrates the validity of the model as a predictor of accident risk.

SITE SELECTION

The purpose of the study was to compare model conflicts with recorded injury accidents, so the first condition imposed was that a positive number of accidents should have occurred at the site in recent years. This condition may bias our results, since a random sample of sites would include junctions which had no accidents in recent years. However, the time period chosen is critical: since accidents are essentially random events, the accident record for a site may list two or three accidents in one year, no accidents during the next few years, and then two or three accidents the following year. This point is discussed further later.

The second condition imposed was that fairly high flows should pass through the junction during peak periods: previous experience indicated that acceptable flows would be in the region of 1000 vph in the major road and 300 vph in the minor road. This condition ensured that sufficient data to derive gap acceptance functions could be collected in a few days, and so data could be collected at an appreciable number of sites in a relatively short time period. However, since data would only be collected in good weather conditions, the time required to complete observations at several sites might in practice span several months.

Two further conditions set a geographical limit to the study area. The site should be located in a non-urban area, since the model simulates a non-urban environment. In addition, the site

SITE	INTERSECTION (Major/minor road)	NATIONAL GRID REFERENCE
Amersham*	A413/A404	SU 972 968
Broadford Road	A3100/A248	SU 990 465
Burchett's Green*	A4/A404	SU 832 803
Compton	A3/B3000(East)	SU 954 474
Frith Hill*	A413/B485	SP 897 014
Great Missenden*	A413/A4128	SP 896 016
Peasmarsh	A3100/B3000	SU 988 460
Puttenham	A3/B3000(West)	SU 947 472
Shalford	A248/A281	TQ 000 468
Tongham	A31/A3014	SU 885 482
Windlesham*	A30/B3020	SU 925 649

Table 8.1 Locations of sites selected for validation study.

should be located within a 50 km radius of Royal Holloway College (RHC). This meant that extra costs would not be incurred due to overnight stays at a site, and enabled observers to return to RHC during the day.

Accident records for the years 1973 to 1977 for T-junctions located in non-urban areas were obtained from the County Engineer's Departments of Surrey, Berkshire and Buckinghamshire. Many of these junctions were considered unsuitable because of narrow major roads or bad visibility. The remaining sites were visited during peak periods, when flow counts for all streams were sampled. Low flows eliminated several sites, leaving a total of eleven T-junctions considered suitable for our purposes; these sites are listed in Table 8.1.

RESULTS

Conflicts

Model conflicts may occur in four positions (Figure 8.2), depending on the manoeuvres involved. Results for conflicts involving crossing (Position 1) or merging (Position 2) vehicles are presented in this chapter, using data for Broadford Road, Compton, Peasmarsh, Puttenham, Shalford and Tongham. Model conflicts in Positions 3 and 4 involve vehicles turning right out of the minor road. Data for the other five sites listed in Table 8.1 is being used to investigate the relationship between these conflicts and recorded injury accidents; results will appear elsewhere (Final Report to TRRL, 1980).

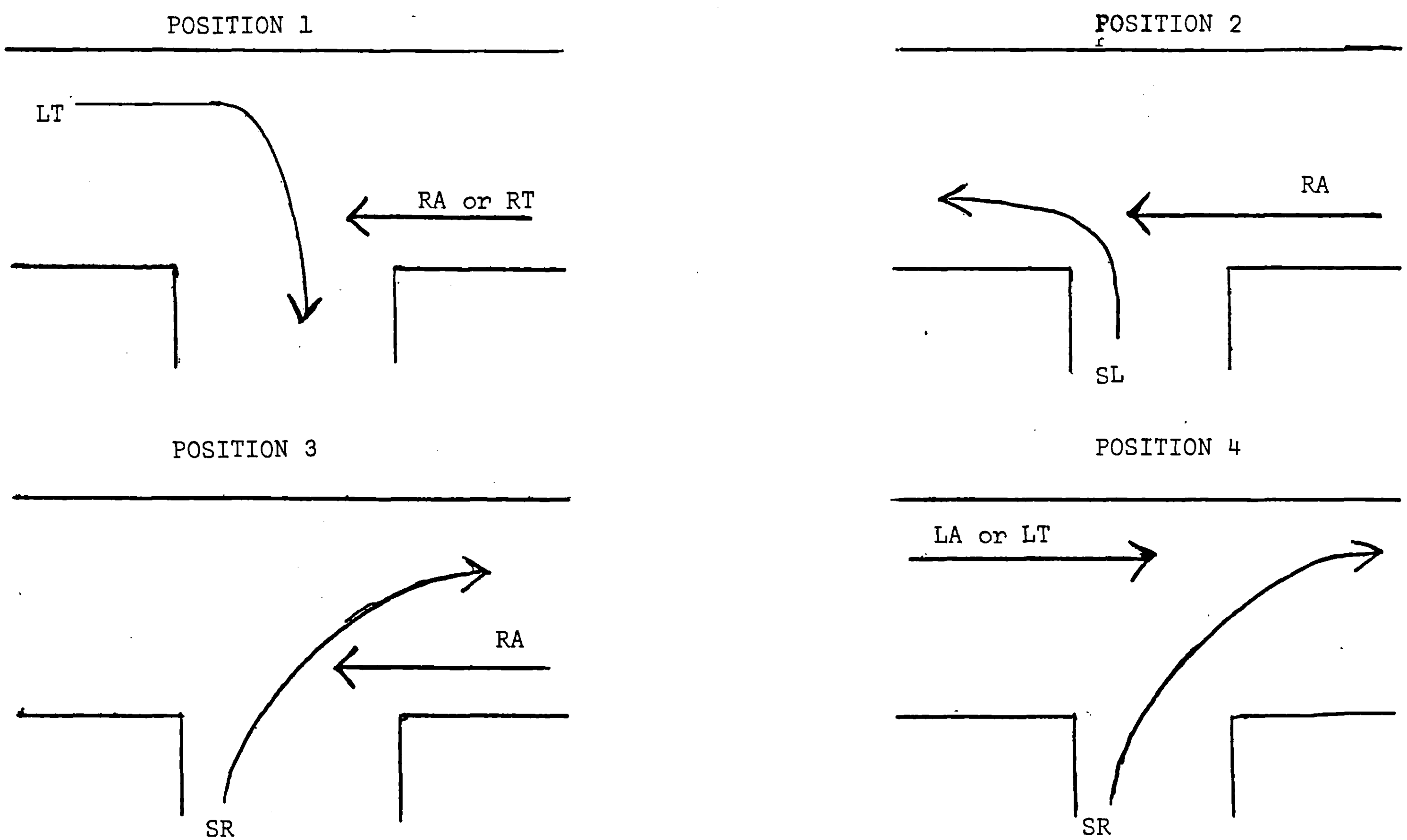


Figure 8.2 Conflict positions.

Data collected at all six junctions has been analysed to give the following details:

i) average flows in each direction;

ii) distribution of speeds of vehicles in nearside stream of major road traffic;

iii) gap acceptance parameters for crossing and merging manoeuvres.

In general, flows in the major road were rather different during morning and evening peak periods, so the above parameters were obtained for morning and evening data separately; results are presented in Tables 15.1 to 15.3 (Appendix 5).

Exposure time data was not available for most of these sites, so two alternative assumptions were made in order to run the model:

<u>Assumption 1</u> All crossing vehicles have the same exposure time, which is set to 1.5s for all sites.

<u>Assumption 2</u> Exposure times are sampled from a normal distribution with mean 1.5s and standard deviation 0.5s (this is similar to the distribution obtained for Puttenham).

Neither of these assumptions reflects the true situation, since gap acceptance and exposure time are directly related (Chapter 5). However, we are only concerned with the comparison of results from different sites, so the absolute number of conflicts predicted by the model is not important. Provided similar overall results are obtained with both assumptions, the outcome of the study should not be affected.

SITE	ACCIDENTS	RANK
Broadford Road	4	2
Compton	8	1
Peasmarsh	3	$3\frac{1}{2}$
Puttenham	3	$3\frac{1}{2}$
Shalford	2	5
Tongham	1	6

Table 8.2 Numbers of reported injury accidents involving turning vehicles (January 1973 to December 1977 inclusive). Sites are ranked in decreasing order.

The model has been run with the relevant parameters from each site for Assumptions 1 and 2 separately. Numbers of conflicts predicted in each case are shown in Tables 15.4 to 15.6 (Appendix 5), for various grades of severity. The difference in conflict numbers for Assumptions 1 and 2 is quite noticeable, and many more merging conflicts occur than crossing conflicts; these differences are discussed below.

Accidents

The number of injury accidents which involved turning vehicles was derived from the accident record for each site (Table 8.2). All accidents involving priority vehicles in the major road and either vehicles emerging from the minor road or turning right from the major road were included (in many cases, it was not possible to ascertain from the records exactly which turning manoeuvre was being carried out). Single vehicle accidents and those involving vehicles travelling in the same traffic stream were ignored, since these generally occurred on the approaches to the junction, not at the junction.

Ranking of sites

The sites have been ranked by the total number of conflicts involving merging vehicles and by the total number of crossing conflicts for each of Assumptions 1 and 2; results are presented in Table 8.3 for morning and evening data separately and for all data combined (both morning and evening data were not available for Shalford or merging at Puttenham). Rankings by the number of conflicts in severity grades 3, 4 and 5 only are given in Table 8.4.

The rankings for morning and evening data separately show considerable variation. However, we need an estimate of overall risk at a site, so the significant rankings are those for morning

SITE	AM	PM	AM+PM
CROSSING (ASS.1)			
Broadford Road	1	2	2
Compton	4	1	1
Peasmarsh	5	4	5
Puttenham	2	3	3
Shalford	-	5	-
Tongham	3	6	4
CROSSING (ASS.2)			
Broadford Road	2	2	2
Compton	3	1	1
Peasmarsh	5	3	5
Puttenham	1	5	3
Shalford	-	6	-
Tongham	4	4	4
MERGING			
Broadford Road	5	1	1
Compton	3	2	2
Peasmarsh	1	5	4
Puttenham	4	-	-
Shalford	-	3	-
Tongham	2	4	3

Table 8.3 Rankings of sites by total number of conflicts (in decreasing order).

SITE	AM	PM	AM+PM
CROSSING (ASS.1)			
Broadford Road	1	2	2
Compton	2	1	1
Peasmarsh	5	$5\frac{1}{2}$	5
Puttenham	3	3	3
Shalford	-	$5\frac{1}{2}$	-
Tongham	4	4	4
CROSSING (ASS.2)			
Broadford Road	3	2	2
Compton	$1\frac{1}{2}$	1	1
Peasmarsh	5	$4\frac{1}{2}$	5
Puttenham	$1\frac{1}{2}$	$4\frac{1}{2}$	3
Shalford	-	6	-
Tongham	4	3	4
MERGING			
Broadford Road	5	1	2
Compton	3	2	1
Peasmarsh	2	5	3
Puttenham	1	-	-
Shalford	-	3	-
Tongham	4	4	4

Table 8.4 Rankings of sites by number of conflicts in severity grades 3 - 5 only.

and evening data combined. Unfortunately, this means that Shalford must be excluded from the validation exercise, since data collected during the mornings could not be recovered.

All rankings by number of conflicts involving crossing vehicles give the same result, irrespective of the assumption about exposure times or the severity grades used ie the order of risk predicted by the model is as follows (highest risk first):

Compton/Broadford Road/ Puttenham/Tongham/Peasmarsh

The two overall rankings by merging conflicts do not agree exactly, but they indicate that Broadford Road and Compton together should be ranked above Peasmarsh and Tongham together. Consequently, the overall order of risk for the five sites is the same as that shown above.

Using the information given in Table 8.2, the order of risk for these five sites in terms of recorded injury accidents is as follows:

Compton/Broadford Road/Peasmarsh and Puttenham/Tongham

This ranking is very similar to that obtained from the model results; indeed, Peasmarsh is the only site in the wrong position. Spearman's rank correlation coefficient (R) has been calculated for these two sets of ranks; the relevant data appears in Table 8.5. Although five sites is not a very good statistical sample, the value of R is significant. This means that the two sets of ranks are not independent, and so the order of risk predicted by the model is consistent with that derived from accident statistics.

SITE	RANK BY ACCIDENTS	RANK BY CONFLICTS	DIFFERENCE IN RANKS
Broadford Road	2	2	0
Compton	1	1	0
Peasmarsh	$3\frac{1}{2}$	5	$1\frac{1}{2}$
Puttenham	$3\frac{1}{2}$	3	$\frac{1}{2}$
Tongham	5	4	1

Spearman's rank correlation coefficient R is given by

$$R = 1 - 6D/n(n^2 - 1),$$

where D is the sum of the squares of the differences in ranks, and n is the number of ranks.

$$D = 0+0+2\tfrac{1}{4}+\tfrac{1}{4}+1 = 3\tfrac{1}{2}, \text{ and } n = 5.$$

Hence $R = 1 - 21/120$

$$= 0.825 \ (0.05 < p < 0.1).$$

Table 8.5 Calculation of Spearman's rank correlation coefficient.

DISCUSSION

Validation process

Testing the validity of a model as complex as the conflict simulation model is a lengthy process. The main problem associated with external validation of the model is that accident risk can not be measured directly, and so the measure against which the model should be validated is not immediately obvious. We have chosen to take recorded injury accidents as our measure of safety at junctions, which is a widely used criterion. However, accident records do not provide very good statistics, so the number of accidents reported at a particular location may not give an accurate estimate of the accident risk associated with that location.

We have used all accidents involving turning vehicles in all conditions, whereas the data used to run the model was derived from observations of crossing and merging vehicles during peak periods in good weather conditions. However, the number of accidents which occurred at each site was rather small, and further stratification of the accident data would make it very difficult to distinguish between the sites; in any case, it is not always possible to determine from the records exactly which manoeuvres were involved. Consequently, it would not really be feasible to obtain accident data for exactly the same conditions that prevailed when observations were made.

Although the accident statistics we have used are not ideal, they provide the only reasonable criterion against which the model can be validated. If the model is to be used with any confidence, then we have to show that its results correspond to some alternative measure of safety; despite the disadvantages outlined above, the number of recorded injury accidents involving turning vehicles is the only practical measure that can be used in this situation.

Site selection

The criteria by which sites were selected for inclusion in this study meant that only "high risk" sites were chosen ie sites with high flows and a bad accident record. A random sample of sites would have been preferable from a statistical point of view, but it would not have suited our purposes. We wanted to observe behaviour at several sites in a relatively short time period, so high flows were essential. Since accidents and flow are related (Chapter 9), high flow sites probably also have an accident record. Consequently it would not really be possible to find a "safe" site (ie one with no accidents) which satisfied all the other conditions imposed.

Number of predicted conflicts

The model predicts many more merging conflicts than crossing conflicts, which reflects the different criteria under which conflicts occur for these two manoeuvres. Crossing conflicts occur when the sampled exposure time of crossing vehicles is greater than the time available before the arrival of the next major road vehicle at the centre of the junction. Merging conflicts occur when oncoming major road vehicles are forced to decelerate to avoid collision with the vehicle which has merged with the traffic stream in front of them; such conflicts may take place upstream of the junction, since a merging vehicle is "exposed" to oncoming traffic until the driver has attained his preferred speed. In particular, a conflict will always take place if the sampled speed of the merging vehicle is less than that of the oncoming major road vehicle.

Accident records for the two manoeuvres show the reverse pattern ie crossing vehicles are involved in more accidents than merging vehicles. This is to be expected, however, since only accidents which involve injury are generally reported. Collisions involving merging vehicles are more likely to be damage-only accidents, since the relative velocities involved are fairly low, and impact usually occurs at the rear of the vehicle. Collisions involving crossing vehicles are generally more hazardous, since the relative velocities are higher, and impact usually occurs near the front of the vehicle.

More crossing conflicts are predicted when the model is run under Assumption 2. This is reasonable, since crossing conflicts occur when exposure times are relatively long, and sampling from the distribution will generate such exposure times.

Ranking of sites

The order given when sites are ranked in terms of risk predicted by the model agrees with the ranking by recorded injury accidents. Although only five sites were used in this study, the validity of the model as a predictor of accident risk has been demonstrated. These results are very encouraging, and indicate that the model may now be used with confidence to compare accident risk for different situations.

We have already noted that accident statistics are not very useful for research purposes. Data must be collected for long time periods or over large areas to provide sufficient information for analysis, and so only limited problems can be investigated. The advantages of the modelling technique are obvious; assessments of risk can be made from data collected in short time periods, and the effects of individual parameters on accident risk can easily be studied. For instance, it is not really feasible to examine the relationship between accidents and flow at a given location, but the way in which accident risk varies as flow levels change at a particular site may be investigated using the model.

CONCLUSIONS

The conflict simulation model developed at RHC is a good predictor of accident risk at T-junctions. This model may now be used confidently to compare assessments of risk for different situations. (Some examples of this are given in the next chapter.)

CHAPTER NINE

GAP ACCEPTANCE AND SAFETY

Introduction

To illustrate the use of the conflict simulation model, some of the gap acceptance results given in Chapters 4, 5 and 6 are evaluated in terms of accident risk in this chapter. In the majority of these examples, no direct comparison can be made with accident statistics, since the relevant data are not available. However, for those cases where comparisons can be made, the results predicted by the model agree with those derived from studies of traffic accidents. This further validates the model as a predictor of accident risk, and indicates that this approach gives a meaningful interpretation of the ways in which safety may be affected by various factors.

CONFLICTS AND FLOW

The relationship between model conflicts and flow has been investigated by keeping all input parameters constant except for the flows in each traffic stream. Major road flows were varied up to 1000 vehicles per hour while turning flows were held constant at various levels. In all conflict positions, conflict frequency is proportional to the product of the flows in the interacting traffic streams ie $C = kF_MF_T$, where C is conflict rate, F_M is the major road flow and F_T is the turning flow. For a fixed turning flow, this reduces to $C = k'F_M$, and conflict rate increases approximately linearly with major road flow. The rate of increase in conflict rate is greater for higher turning flows (Figure 9.1).

These results are to be expected, since there are more opportunities for interactions between conflicting traffic streams as more vehicles pass through the junction. The maximum flow values used mean that we only consider uncongested junctions; at very high levels of flow, the gap acceptance process is meaningless since turning drivers must either force their way out of the junction or wait for a major road vehicle to stop and let them turn. Since the number of conflicts predicted by the model is sensitive to flow, the results given in the following sections are derived for fixed major road flows of 1000 vph and constant turning flows of 300 vph; consequently the number of opportunities for conflict is equal in all cases.

We have already shown that gap acceptance parameters at Denton and Tongham vary with major road flow (Chapter 4), and that this variation is probably due to differences in the presented gap distribution at different flows. These results can not be evaluated directly using the current version of the model, but they suggest that the relationship between conflicts and flow may be more complex than outlined above, and will vary for different locations. Current research at RHC will consider this problem in more detail (Final Report to TRRL, 1980).

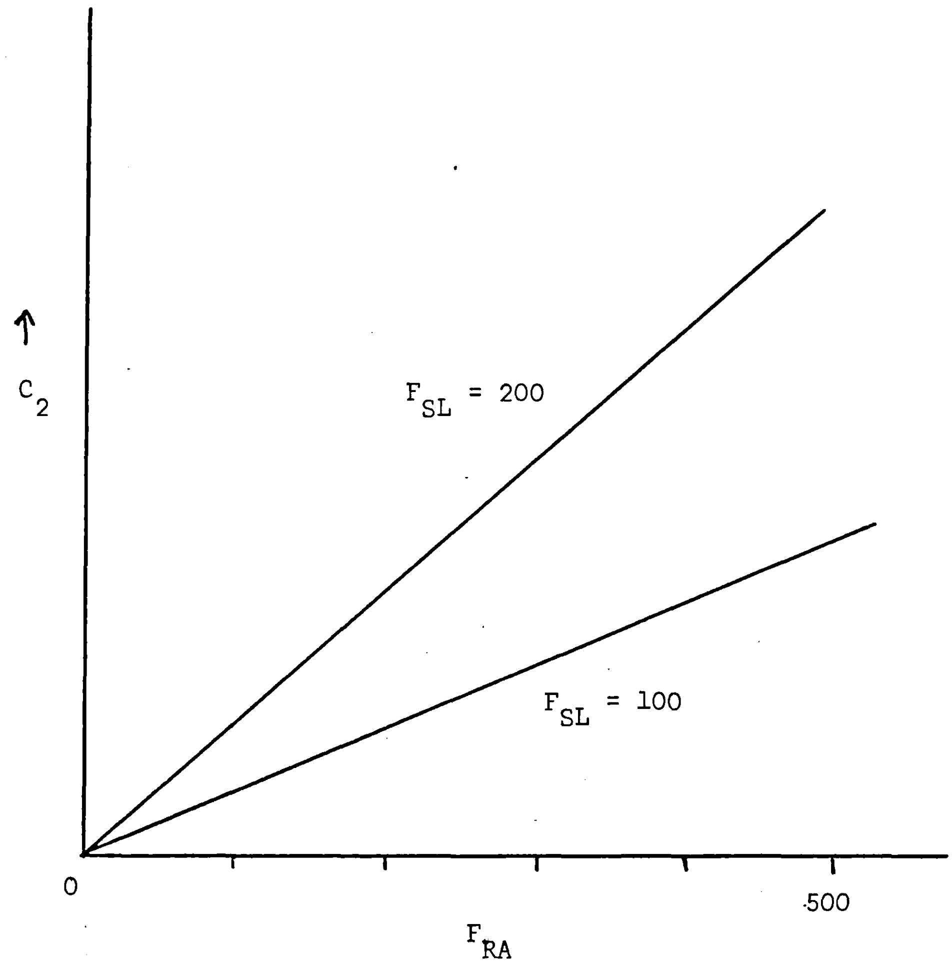


Figure 9.1 Conflict rate C_2 in position 2 against flow from the right F_{RA} (vph), for fixed merging flows F_{SL}.

MAJOR ROAD SPEED DISTRIBUTION

The effects of approach speed on conflict rate are discussed in detail by Cooper, Storr and Wennell (1977) and Storr (1980); an example of these results is given in this section, using data for the crossing manoeuvre at Denton. Model conflicts involving crossing vehicles were classified by the approach speed of the major road vehicle in five feet/second ranges; normalised proportions of major road vehicles involved in conflict are shown in Figure 9.2.

The drivers of high speed major road vehicles are more likely to be involved in crossing conflicts, and risk varies approximately linearly with speed. The effect of the distribution of major road speeds (assumed normal) was investigated by varying the standard deviation while keeping the mean constant; the overall conflict rate involving crossing vehicles was not affected, but the average severity of the conflicts increased with increased dispersion of speeds. This increase in risk with increasing approach speed is probably due to the errors of turning drivers in judging gaps in front of fast vehicles; we have already noted that perceptual errors are greater for speeds different from the mean (Chapter 4). Faster major road vehicles are involved in more severe conflicts than the slower vehicles; this is reasonable since the amount of deceleration enforced on the approaching vehicle is greater when relative speeds are higher.

Cooper, Storr and Wennell (1977) suggest that the risk associated with fast major road vehicles may be lowered when the acceleration behaviour of turning drivers is considered. However,

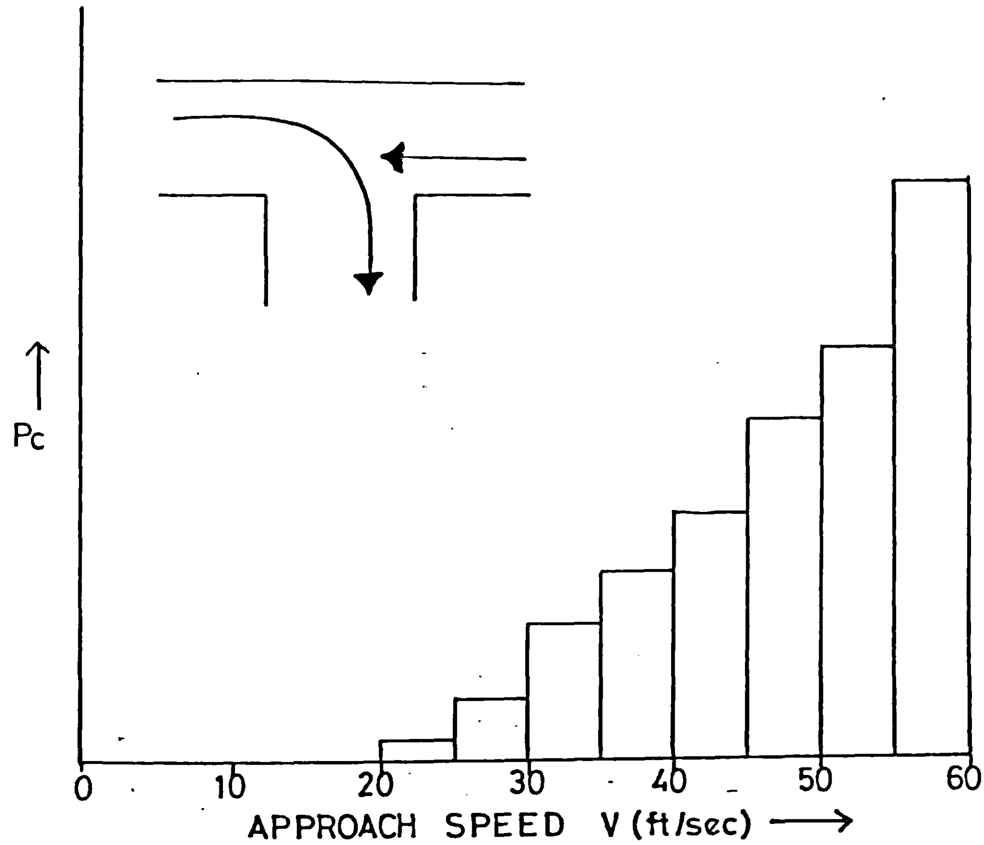


Figure 9.2 Normalised proportion P_c of major road vehicles involved in conflicts with crossing vehicles as a function of approach speed V.

Cooper, Wennell, Storr and McDowell (1978) show that similar results are obtained even when the effects of acceleration are allowed for, so the relationship between speed and risk is as detailed above.

MANOEUVRE TIME AND CLASS OF VEHICLE

Empirical results for crossing cars at Denton (Table 5.1) have been used to examine the relationship between the manoeuvre time of turning vehicles and risk. The model was run with the relevant gap acceptance parameters for each range of manoeuvre times, and a constant exposure time set equal to the mean manoeuvre time in each range. The numbers of model conflicts predicted in each case are shown in Table 9.1, for various grades of severity.

Drivers of crossing vehicles with short manoeuvre times are involved in many more conflicts than those with longer manoeuvre times, and the average severity of the conflicts is much greater for short manoeuvre times. These results show that crossing drivers who accept short gaps are taking greater risks, even though they complete their manoeuvre more quickly. There is some indication that drivers with long manoeuvre times may also be at risk, although they are not likely to be involved in severe conflicts.

The gap acceptance behaviour of different classes of vehicles is related to their manoeuvre times (Chapter 5): we would therefore expect turning goods vehicles to be involved in slightly more conflicts than turning cars, since their manoeuvre times are relatively long. Model results for different classes of crossing

MANOEUVRE TIME		NUMBER OF CONFLICTS		
Range	Mean	Grades 1 & 2	Grades 3 - 5	Total
1.36 - 1.65	1.52	24.7	21.6	46.3
1.66 - 1.95	1.80	18.6	4.2	22.8
1.96 - 2.25	2.09	0.9	0.0	0.9
2.26 - 2.55	2.38	0.3	0.0	0.3
2.56 - 2.85	2.68	3.1	0.0	3.1

Table 9.1 Numbers of crossing conflicts predicted for each manoeuvre time class (average of 10 runs of 10 hours each).

TURNING	NUMBER OF CONFLICTS		
VEHICLES	Grades 1 & 2	Grades 3 - 5	Total
CARS	6.0	0.6	6.6
GOODS	7.6	0.0	7.6

Table 9.2 Model conflicts for crossing cars and goods vehicles at Denton (average of 10 runs of 10 hours each).

vehicles at Denton are shown in Table 9.2; although turning goods vehicles are involved in slightly more conflicts, the average severity of the conflicts is greater for turning cars.

GENDER OF DRIVER

We have shown that women drivers are more cautious in their gap acceptance behaviour than men (Chapter 6), so we would expect them to be involved in fewer conflicts. The results for men and women drivers of turning cars at Puttenham and Tongham have been evaluated in terms of risk using the simulation model: numbers of predicted conflicts involving merging and crossing vehicles are given in Table 9.3. As we expected, turning women drivers are involved in fewer conflicts than men; however, the difference is not very large at Puttenham. The average severity of crossing conflicts is slightly higher for women drivers, but no such difference occurs for merging conflicts.

ACCIDENT STATISTICS

Flow

The effect of flow on accidents is a well-researched topic, but with many conflicting results (Chatfield, 1970 and 1973; Erlander, Gustavsson and Larusson, 1969; Gwynn, 1967; Pfundt, 1968; Silyanov, 1973). The number of opportunities for collision increases as flow increases, so we would expect accident frequency to increase with flow. However, consideration must be given to the types of accident which may occur: the number of multi-vehicle accidents per million vehicle miles

SITE (manoeuvre)	DRIVERS	NUMBER OF CONFLICTS		
		Grades 1 & 2	Grades 3 - 5	Total
PUTTENHAM	MEN	10.6	3.0	13.6
(crossing)	WOMEN	9.1	3.1	12.2
TONGHAM	MEN	4.7	0.5	5.2
(crossing)	WOMEN	1.0	0.2	1.2
TONGHAM	MEN	843.9	26.1	870.0
(merging)	WOMEN	689.2	11.3	700.5

Table 9.3 Numbers of model conflicts for men and women drivers of turning cars at Puttenham and Tongham.

does increase with increasing average daily traffic (ADT), but that for single vehicle accidents decreases (Chapman, 1969; Kihlberg and Tharp, 1968; Wright and Mak, 1976).

In studies of accident frequencies for different junctions with different values of ADT, Colgate and Tanner (1967) and Leong (1973) showed that the number of accidents was a function of the square root of the product of the intersecting flows at a junction. This relationship may not apply to changes in accident frequency with variations in flow at any particular junction, but it would be very difficult to obtain sufficient accident data to investigate such a relationship. Theoretical (Chapman, 1971) and observational (Spicer, 1972) studies of traffic conflicts (which are proportional to accidents) suggest that the relationship at a given junction depends on the product of the intersecting flows, which agrees with our simulation results.

Speed

Some of the studies relating speed to accidents have been concerned with the effect of imposing or reducing speed limits, which result in a significant reduction in accident rate and severity (Labrum, 1976; Scott and Barton, 1976; Smeed, 1960a and 1960b); such studies generally consider national statistics. The majority of investigations into the effect of individual vehicle speeds on accident occurrence obtained a U-shaped relationship between accident rate and deviation from the mean speed ie slow and fast drivers have a higher accident involvement rate than those who drive at speeds near the mean (Cirillo, 1968; Lefeve, 1955; Munden, 1967; Research Triangle Institute, 1970; Solomon, 1964; West and Dunn, 1971). Most of these studies used data for highways in the USA, and the speeds of vehicles involved were derived from witnesses' estimates. Beatty (1972) and Joksch (1975) both show that accident severity increases as speed increases.

Class of vehicle

National statistics show that light goods vehicles (under $1\frac{1}{2}$ tons unladen weight) and cars have comparable accident involvement rates, but the rate for heavy goods vehicles (over $1\frac{1}{2}$ tons unladen weight) is about three-quarters that for cars (Table 12 in Road Accidents Great Britain 1976). Results of a study in Queensland (Australia) found that private vehicles have a significantly higher accident involvement rate than commercial vehicles (Foldvary, 1977). The engine capacity of a vehicle, its weight and its power-to-weight ratio may also be important (Foldvary, 1977; Jones, 1975).

Gender of driver

The national casualty rate for male car drivers is about three times that of female drivers (Table 29 in Road Accidents Great Britain 1976). However, males tend to do more driving at times when accident risk is high eg at night, so the difference is probably much lower when exposure is taken into account (Peck, McBride and Coppin, 1971). Males and females generally exhibit different driving patterns, which lead to different errors: female drivers are more likely to make perceptual errors or errors relating to skill, since they tend to be less experienced than male drivers; however, male drivers are more likely to be impaired through alcohol, or drive too fast for the conditions, and they take risks more readily (Harrington and McBride, 1970; Storie, 1977).

DISCUSSION

It is rather difficult to compare directly the results for risk predicted by the model with those obtained from accident studies. In most cases, the relationship between a particular factor and accident rates has been obtained from a study of accidents occurring at all times of day at all locations (junctions and links). Our results are generally confined to either the turning population or the vehicles travelling along the major road at priority junctions during weekday peak periods. Given these basic differences in the situations studied, the amount of agreement between the two sets of results is remarkable.

Flow

Studies at particular junctions indicate that conflict frequency depends on a function of the product of the intersecting flows whereas accident studies over a range of junctions suggest that a square root relationship is appropriate. However, for the ranges of flow we examined, there is a significant correlation between the product of the flows and the square root of the product of the flows, so either relationship could provide equally satisfactory results in terms of statistics. In any case, the actual effects of flow on safety at junctions may be more complex than either of these relationships suggest.

Speed

Risk predicted by the model increases as the speed of spproaching vehicles increases, and so fast major road vehicles are at greatest risk. Accident studies, however, indicate that both fast and slow vehicles are at risk. If we include the speed of the turning vehicle in our results, the relationship depicted in

Figure 9.2 may become more U-shaped, so both approaches probably give equivalent results. Similar relationships between speed and severity are obtained in both cases.

Manoeuvre time

It would not be possible to examine the relationship between manoeuvre time and accidents directly, since turning vehicles involved in accidents do not complete their manoeuvre. However, we have already indicated in Chapter 5 that manoeuvre times are related to vehicle characteristics such as length, weight and acceleration capability, and two studies have shown that such factors may be important in accidents. Our model results suggest that drivers who accept short gaps are taking greater risks than those who accept longer gaps, even though they complete their manoeuvre more quickly. This seems reasonable, since the margin for safety is much smaller when short time gaps are involved.

Class of vehicle

The two sets of results for different classes of vehicle do not appear to agree very well, but risk predicted by the model is for turning vehicles only and the accident results are for all involved vehicles at all locations; consequently, we can not make a valid comparison between these results.

Gender of driver

Accident rates for male and female drivers are usually calculated with respect to total population, and so they are not really representative of the different involvement rates of the two sexes. Storie's results from a detailed investigation of accidents in a specific area indicated that males and females are about equally at risk, but she noted that essentially different populations were driving at different times of day. The results predicted by the model may not reflect the true situation, since we assumed that drivers of both sexes behaved in similar ways with regard to manoeuvre time. As females generally accept longer gaps, they may also take longer to complete their manoeuvre than males, and use less acceleration; this would certainly affect the conflict numbers given in Table 9.3. Clearly a more detailed investigation of the differences in driving behaviour of males and females is necessary before we can make confident predictions about their relative risk of accident involvement.

CONCLUSIONS

Various gap acceptance results derived in earlier chapters have been evaluated in terms of risk using the simulation model. Where appropriate, these results have been compared with the findings of studies of accident statistics, and shown to be in agreement; this further validates the model as a predictor of accident risk. It also highlights the advantages of this technique over accident studies, since predictions about accident risk can be made from data collected in relatively short time periods. The need for further empirical studies of the ways in which driving behaviour is affected by the occupants of the vehicle is noted.

CHAPTER TEN

SUMMARY AND CONCLUSIONS

Introduction

This thesis has investigated the behaviour of drivers at non-urban T-junctions, with particular emphasis on the factors which influence gap acceptance responses and their implications for road safety. A review of the literature on gap acceptance studies showed that gap acceptance is a very complex process, which may be affected by various factors relating to the driver, his vehicle and the road environment.

Observations of driver behaviour at several non-urban T-junctions have been reported. Two different methods of data collection have been described: one using video techniques, the other using a microprocessor-based system. Neither of these techniques is perfect, but given the choice of only one method of data collection we recommend the microprocessor-based system, because the time required for analysis of the data is relatively short. However, we note that a system which incorporates both techniques is preferable to either technique in isolation.

The main conclusions reached in earlier chapters are presented together in this chapter, under headings which relate to the particular factors investigated. Subjects which would benefit from further research are listed in the final section.

MAJOR ROAD SPEEDS

The behaviour of crossing drivers may be represented by a modified time hypothesis in which the median accepted gap may be considered as a constant time gap plus a constant distance gap. At the particular intersection we observed, the median accepted gap D in feet was related to the approach speed V in feet per second by

$$D = 115 + 1.9V .$$

GAP ACCEPTANCE AND FLOW

Variations in major road flow affect the gap acceptance behaviour of turning drivers. Different relationships between median accepted gaps and flow were obtained at the two intersections studied, but both relationships could be explained in terms of the presented gap distributions at different flows. At both junctions, variability in population gap acceptance behaviour decreased as major road flow increased.

MANOEUVRE TIME

Car drivers who accept short gaps complete their manoeuvres more quickly than those who accept long gaps. This effect has been observed at several T-junctions, for both crossing and merging turns. This variation in gap acceptance response is probably related to the acceleration capabilities of cars, and seems to reflect a deliberate strategy of drivers.

CLASS OF VEHICLE

Turning vehicles

The drivers of cars, vans and trucks accept increasingly longer gaps, corresponding to the generally increasing size and decreasing acceleration capability of vehicles in these classes. In addition, the median gaps accepted by goods vehicles are comparable to those that would be accepted by cars with the same manoeuvre times, for both crossing and merging turns. The observed differences in median accepted gaps for turning cars and goods vehicles are larger for the merging manoeuvre than for the crossing manoeuvre at each junction.

Approaching vehicle

The median accepted gap for merging cars is significantly shorter when the approaching vehicle is a goods vehicle rather than a car. This effect may be explained in terms of misperception of the speed of oncoming large vehicles. Further work is needed to investigate the reasons for this difference in gap acceptance response in front of approaching cars and goods vehicles.

OCCUPANTS OF TURNING VEHICLE

Gender of driver

Median accepted gaps for women drivers are longer than those for men, for both crossing and merging turns. This difference in gap acceptance behaviour has been observed at several sites, for various classes of turning vehicles. At all the intersections studied, the populations of men and women drivers exhibited similar variability in behaviour, and so women drivers are consistently more cautious than men drivers.

Presence of passengers

The presence of passengers in the turning vehicle seems to have an effect on gap acceptance behaviour, which may be different for men and women drivers. However, this phenomenon was only investigated at one site, where fairly small samples were observed, so these results should be treated with caution. Further research is needed to clarify the effects of passengers on drivers' gap acceptance responses.

QUEUE ACCEPTANCE

The explanatory model described in Chapter 7 is better than the direct linear relationship for the analysis of queue acceptance. The explanatory model is more appropriate to the data, and it is more useful for diagnostic purposes because the individual components of the queue acceptance process are included explicitly. At the particular intersections observed, the time gap T required by N merging vehicles was found to be

$$T = 3.0N + 3.0 ,$$

while the gap required by N crossing vehicles is given by

$$T = 2.4N + 2.9 .$$

VALIDATION STUDY

Results derived from observations of several T-junctions in the Guildford area have been used as input parameters for the conflict simulation model developed at RHC. These sites have been ranked in terms of the number of conflicts predicted by the model, and by the number of injury accidents involving turning vehicles which were reported in a five year period. The two sets of rankings are consistent, which indicates that the model is a good predictor of accident risk at these intersections. We note that these results are only preliminary, and that more detailed work is required before we can claim complete validation of the model.

GAP ACCEPTANCE AND SAFETY

Some of the gap acceptance results reported in this thesis have been evaluated in terms of risk using the conflict simulation model. Where appropriate, these results have been compared with those derived from studies of accident statistics, and shown to be in agreement; this further validates the modelling technique as a means of assessing safety at junctions.

AREAS FOR FURTHER RESEARCH

Throughout this thesis, several subjects have been identified which require further study, either because conflicting results were found at different locations or because there was insufficient data available for a comprehensive investigation of the problem. The effects of flow on gap acceptance behaviour appear to be rather complex, and further investigations of this phenomenon will be reported in due course (Wennell, 1980). Other factors which require more detailed studies are listed below:

i) class of approaching vehicle;

ii) presence of passengers in turning vehicle;

iii) waiting time on approach to intersection;

iv) interactions between various factors.

The effects of adverse environmental factors such as darkness and bad weather (rain in particular) are being investigated by the Operational Research Group at RHC.

R E F E R E N C E S

ASHTON, W.D. Gap acceptance problems at a traffic intersection. Applied Statistics, Vol. 20, pp. 130-138, 1971.

ASHWORTH, R. A note on the selection of gap acceptance criteria for traffic simulation studies. Transportation Research, Vol. 2, pp. 171-175, 1968.

ASHWORTH, R. Gap acceptance, capacity and delays at priority-type intersections. Ph.D. Thesis, University of Liverpool, 1969.

ASHWORTH, R. The analysis and interpretation of gap acceptance data. Transportation Science, Vol. 4, pp. 270-280, 1970.

ASHWORTH, R. A videotape-recording system for traffic data collection and analysis. Traffic Engineering and Control, Vol. 17, No. 11, pp. 468-470, 1976.

ASHWORTH, R. and C.G. BOTTOM. Some observations of driver gap-acceptance behaviour at a priority intersection. Traffic Engineering and Control, Vol. 18, No. 12, pp. 569-571, 1977.

ASHWORTH, R. and B.D. GREEN. Gap acceptance at an uncontrolled intersection. Traffic Engineering and Control, Vol. 7, pp. 676-678, 1966.

BALASHA, D., A.S. HAKKERT and M. LIRNEH. A quantitative definition of the near accident concept. In Proceedings of the Second International Workshop on the Traffic Conflict Technique.

BEATTY, R.L. Speed analysis of accidents on Interstate highways. Public Roads, Vol. 37, No. 3, pp. 89-102, 1972.

BENDTSEN, P.H. Starting performance of queues at unsignalised T-junctions. Traffic Engineering and Control, Vol. 14, No. 5, pp. 214-217 and p. 220, 1972.

BLUMENFELD, D.E. and G.H. WEISS. The effects of gap acceptance criteria on merging delay and capacity at an uncontrolled junction. Traffic Engineering and Control, Vol. 20, No.1, pp. 16-20, 1979.

BLUNDEN, W.R., C.M. CLISSOLD and R.B. FISHER. Distribution of acceptance gaps for crossing and turning manoeuvres. Proceedings of the Australian Road Research Board, Vol. 1, No. 1, pp. 188-205, 1962.

BOTTOM, C.G. Investigations into the variability of driver gap acceptance behaviour. Ph.D. Thesis, University of Sheffield, 1975.

BOTTOM, C.G. and R. ASHWORTH. Driver behaviour at priority type intersections. Presented at First International Conference on Driver Behaviour, Zurich, October 1973.

BOTTOM, C.G. and R. ASHWORTH. Factors affecting the variability of driver gap acceptance behaviour. Ergonomics, Vol. 21, No. 9, pp. 721-734, 1978.

BRAIN, R. Vehicle speed estimation at a right-angled at-grade junction. Surveyor, Vol. 121, pp. 789-791, 1962.

CHAPMAN, R.A. Five studies of accident rates. Traffic Engineering and Control, Vol. 11, No. 5, pp. 224-227, 1969.

CHAPMAN, R. Some relationships between road accident frequencies and measures of exposure to risk. Ph.D. Thesis, University of London, 1971.

CHATFIELD, B.V. Relationship of fatality rates and fatal accident rates to travel densities on the Interstate system. Public Roads, Vol. 36, No. 2, pp. 48-49, 1970.

CHATFIELD, B.V. Fatal accidents and travel density. Highway Research Board, Highway Research Record No. 469, pp. 40-51, 1973.

CIRILLO, J.A. Interstate system accident research study II, Interim Report II. Public Roads, Vol. 35, No. 3, pp. 71-75, 1968.

COHEN, J., E.J. DEARNALEY and C.E.M. HANSEL. The risk taken in crossing a road. Operational Research Quarterly, Vol. 6, pp. 120-128, 1955.

COLGATE, M.G. and J.C. TANNER. Accidents at rural three-way junctions. TRRL Report No. LR 87, Transport and Road Research Laboratory, Crowthorne, 1967.

COOPER, D.F. Traffic studies at T-junctions: 3. Observations of gap acceptance using television. Presented at the Second European Congress on Operations Research EURO II, Stockholm, 29 Nov. - 1 Dec. 1976.

COOPER, D.F. and N. FERGUSON. Traffic studies at T-junctions: 2 A conflict simulation model. Traffic Engineering and Control, Vol. 17, No. 7, pp. 306-309, 1976.

COOPER, D.F. and M.R.C. McDOWELL. Police effects on accident risk at T-junctions. Traffic Engineering and Control, Vol. 18, No. 10, pp. 486-487, 1977.

COOPER, D.F. and J. WENNELL. Models of gap acceptance by queues at intersections. Traffic Engineering and Control, Vol. 19, No. 4, pp. 178-180 and p. 185, 1978.

COOPER, D.F., W. SMITH and V. BROADIE. Traffic studies at T-junctions: 1. The effect of approach speed on merging gap acceptance. Traffic Engineering and Control, Vol. 17, No. 6, pp. 256-257, 1976.

COOPER, D.F., P.A. STORR and J. WENNELL. Traffic studies at T-junctions: 4. The effect of speed on gap acceptance and conflict rate. Traffic Engineering and Control, Vol. 18, No. 3, pp. 110-112, 1977.

COOPER, D.F., J. WENNELL, P.A. STORR and M.R.C. McDOWELL. Effects of acceleration and kind of vehicle on drivers gap acceptance behaviour at T-junctions. Presented at the Tenth Annual Conference of the Universities Transport Study Group, London, 4-6 Jan. 1978.

DARZENTAS, J. Ph.D. Thesis, University of London, 1980 (in preparation).

DARZENTAS, J., V. HOLMS and M.R.C. McDOWELL. Driver behaviour at a T-junction in daylight and darkness. Traffic Engineering and Control, Vol. 21, No. 4, pp. 186-189, 1980.

DARZENTAS, J., M.R.C. McDOWELL and D.F. COOPER. Minimum acceptable gaps and conflict involvement in a simple crossing manoeuvre. Traffic Engineering and Control, Vol. 21, No. 2, pp. 58-61, 1980.

DARZENTAS, J., D.F. COOPER, P.A. STORR and M.R.C. McDOWELL. Simulation of road traffic conflicts at T-junctions. Simulation, May 1980 (to appear).

DI PIETRO, C.M. and L.E. KING. Pedestrian gap acceptance. Highway Research Record, No. 308, pp. 80-91, 1970.

DREW, D.R. Gap acceptance characteristics for ramp-freeway surveillance and control. Highway Research Record, No. 157, pp. 108-135, 1967.

EBBESEN, E.B. and M. HANEY. Flirting with death: variables affecting risk-taking at intersections. Journal of Applied Social Psychology, Vol. 3, pp. 303-324, 1973.

EBBESEN, E.B., S. PARKER and V.J. KONECNI. Laboratory and field analyses of decisions involving risk. Journal of Experimental Psychology, Vol. 3, No. 4, pp. 576-589, 1977.

ERLANDER, S., J. GUSTAVSSON and E. LARUSSON. Some investigations on the relationship between road accidents and estimated traffic. Accident Analysis and Prevention, Vol. 1, pp. 17-64, 1969.

EVANS, L. and R. HERMAN. Note on driver adaptation to modified vehicle starting acceleration. Human Factors, Vol. 18, No. 3, pp. 234-240, 1976.

FERGUSON, N. The T-junction conflict simulation program and results. Project 2001, Working Paper CP9, Plessey Company, 1973.

FINAL REPORT TO THE TRANSPORT AND ROAD RESEARCH LABORATORY, Mathematics Department, Royal Holloway College, Egham, Surrey, July 1980 (in preparation).

FINNEY, D.J. Probit Analysis, Cambridge University Press, Third Edition, 1971.

FOLDVARY, L.A. Road accident involvement per miles travelled III. Accident Analysis and Prevention, Vol. 8, pp. 97-127, 1977.

GIBBS, W.L. Driver gap acceptance at intersections. Journal of Applied Psychology, Vol. 52, pp. 200-204, 1968.

GLEN, M.G.M. Ph.D. Thesis, University of London, 1976.

GRAYSON, G.B. Observations of pedestrian behaviour at four sites. TRRL Report No. LR 670, Transport and Road Research Laboratory, Crowthorne, 1975.

GWYNN, D.W. Relationship of accident rates and accident involvements with hourly volumes. Traffic Quarterly, pp. 407-418, 1967.

HARRINGTON, D.M. and R.S. McBRIDE. Traffic violations by type, age, sex and marital status. Accident Analysis and Prevention, Vol. 2, pp. 67-79, 1970.

HAUER, E. Opening remarks to session on Methodological Assessment of the Techniques. In Proceedings of the Second International Workshop on Traffic Conflicts.

HERMAN, R. and G.H. WEISS. Comments on the highway crossing problem. Operations Research, Vol. 9, pp. 828-840, 1961.

HILLS, B. Unpublished data. Transport and Road Research Laboratory, Crowthorne, 1979.

HONDEL, M. and J.H. KRAAY. Review of traffic conflicts technique studies. Report No. R-79-9, Institute for Road Safety Research SWOV, The Netherlands, 1979.

JAMES, J.E. and M. GOLDMAN. Driving patterns for different sized vehicles. Ergonomics, Vol. 14, No. 3, pp. 363-369, 1971.

JOKSCH, H.C. An empirical relation between fatal accident involvement per accident involvement and speed. Accident Analysis and Prevention, Vol. 7, pp. 129-132, 1975.

JONES, I.S. The role of vehicle handling in accident causation. Society of Automotive Engineers, Paper No. 750115, 1975.

KATZ, A., D. ZAIDEL and A. ELGRISHI. An experimental study of driver and pedestrian interaction during the crossing conflict. Human Factors, Vol. 17, No. 5, pp. 514-527, 1975.

KIHLBERG, J.K. and K.J. THARP. Accident rates as related to design elements of rural highways. National Cooperative Highway Research Program Report No. 47, 1968.

LABRUM, W.D. The 55 mph speed limit and fatality reduction in Utah. Traffic Engineering, Vol. 46, No. 9, pp. 13-16, 1976.

LEFEVE, B.A. Relation of accidents to speed habits and other driver characteristics. Highway Research Board Bulletin No. 120, pp. 6-30, 1955.

LEONG, H.J.W. Relationship between accidents and traffic volumes at urban intersections. Australian Road Research, Vol. 5, No. 3, pp. 72-90, 1973.

MILLER, A.J. Nine estimators of gap acceptance parameters. In Traffic Flow and Transportation, G.F. Newell Ed., Elsevier, New York; pp. 215-236, 1972.

MILLER, A.J. A note on the analysis of gap-acceptance in traffic. Applied Statistics, Vol. 23, No. 1, pp. 66-73, 1974.

MILLER, A.J. and R.L. PRETTY. Overtaking on two-lane rural roads. Proceedings of the Australian Road Research Board, Vol. 4, pp. 582-594, 1968.

MOORE, R.L. Pedestrian choice and judgement. Operational Research Quarterly, Vol. 4, pp. 3-10, 1953.

MUNDEN, J.M. The relation between a driver's speed and his accident rate. TRRL Report No. LR 88, Transport and Road Research Laboratory, Crowthorne, 1967.

OLDER, S.J. and B.R. SPICER. Traffic conflicts - a development in accident research. Human Factors, Vol. 18, No. 4, pp. 335-350, 1976.

PEARSON, R.H. and M.G. FERRERI. Operational study - Schuylkill Expressway. Highway Research Board Bulletin No. 291, pp. 104-123, 1961.

PECK, R.C., R.S. McBRIDE and R.S. COPPIN. The distribution and prediction of driver accident frequencies. Accident Analysis and Prevention, Vol. 2, pp. 243-299, 1971.

PERKINS, S.R. and J.I. HARRIS. Traffic conflict characteristics - accident potential at intersections. Highway Research Record, No. 225, pp. 35-43, 1968.

PFUNDT, K. Three difficulties in the comparison of accident rates. Accident Analysis and Prevention, Vol. 1, pp. 253-259, 1969.

POWELL, D.G. and M.G.M. GLEN. Operating characteristics of roundabouts. Presented at the Tenth Annual Conference of the Universities Transport Study Group, London, 4-6 Jan. 1978.

PROCEEDINGS OF THE FIRST INTERNATIONAL WORKSHOP ON TRAFFIC CONFLICTS, Oslo, September 1977. Published by the Royal Norwegian Council for Scientific Research.

PROCEEDINGS OF THE SECOND INTERNATIONAL WORKSHOP ON TRAFFIC CONFLICTS, Paris, May 1979. To be published by TRRL.

RAFF, M.S. A volume warrant for urban stop signs. Eno Foundation, Saugatuck, USA, 1950.

RESEARCH TRIANGLE INSTITUTE. Speed and accidents. Volume 1. Final Report. Research Triangle Institute, Research Triangle Park, North Carolina 27709, USA. RT 1 Project SU-409, 1970.

ROAD ACCIDENTS GREAT BRITAIN 1976. HMSO, London, 1977.

ROBINSON, C.C. Pedestrian interval acceptance. Proceedings of the Institute of Traffic Engineers, Vol. 22, pp. 144-150, 1951.

RORBECH, J. Critical lag in at-grade intersections. Traffic Engineering and Control, Vol. 8, No. 8, pp. 500-501, 1966.

SABEY, B.E. and G.C. STAUGHTON. Interacting roles of road environment, vehicle and road user in accidents. Presented at the Fifth International Conference of the Association for Accident and Traffic Medicine, London, September 1975.

SCOTT, P.P. and A.J. BARTON. The effects on road accident rates of the fuel shortage of November 1973, and consequent legislation. TRRL Report No. SR 236, Transport and Road Research Laboratory, Crowthorne, 1976.

SEDDON, P.A., W. ROWELL and P WILSON.Videotape recording for traffic studies. Audio Visual, Vol. 28, pp. 12-16, 1974.

SHIPPEY, J. An assessment of the automatic detection of vehicle conflicts. In Proceedings of the Second International Workshop on Traffic Conflicts.

SILYANOV, V.V. Comparison of the pattern of accident rates on roads of different countries. Traffic Engineering and Control, Vol. 14, No. 9, pp. 432-435, 1973.

SMEED, R.J. The influence of speed and speed regulations on traffic flow and accidents. Roads and Road Construction, No. 38, pp. 393-397 and No. 39, pp. 15-24, 1960a.

SMEED, R.J. Speed, flow and accidents. Traffic Engineering and Control, Vol. 2, No. 8, pp. 474-478 and p. 482, 1960b.

SMITH, W.E. The effect of a police tactic on gap acceptance - an experimental study. M.Sc. Thesis, University of Sussex, 1974.

SOLBERG, P. and J.C. OPPENLANDER. Lag and gap acceptances at stop-controlled intersections. Highway Research Record, No. 118, pp. 48-67, 1966.

SOLOMON, D. Accidents on main rural highways related to speed, driver and vehicle. U.S. Department of Commerce, Bureau of Public Roads, Washington D.C., 1964.

SPICER, B.R. A pilot study of traffic conflicts at a rural dual carriageway intersection. TRRL Report No. LR 410, Transport and Road Research Laboratory, Crowthorne, 1971.

SPICER, B.R. A traffic conflict study at an intersection on the Andoversford by-pass. TRRL Report No. LR 520, Transport and Road Research Laboratory, Crowthorne, 1972.

SPICER, B.R. A study of traffic conflicts at six intersections. TRRL Report No. LR 551, Transport and Road Research Laboratory, Crowthorne, 1973.

STORIE, V.J. Male and female car drivers: differences observed in accidents. TRRL Report No. LR 761, Transport and Road Research Laboratory, Crowthorne, 1977.

STORR, P.A. Ph.D. Thesis, University of London, 1980 (in preparation).

STORR, P.A., D.F. COOPER and M.R.C. McDOWELL. Analysis of gap acceptance in a complex traffic manoeuvre. European Journal of Operational Research, 1980 (to appear).

STORR, P.A., J. WENNELL, D.F. COOPER and M.R.C. McDOWELL. A microprocessor based system for traffic data collection. Traffic Engineering and Control, Vol. 20, No. 4, pp. 156-158, 1979.

TANNER, J.C. A theoretical analysis of delays at an uncontrolled intersection. Biometrika, No. 49, pp. 163-170, 1962.

UBER, C.B. Start up times and queue acceptance at T-junctions. Traffic Engineering and Control, Vol. 19, No. 4, 1978.

UBER, C.B. Ph.D. Thesis, University of Melbourne (Australia), 1979.

WAGNER, F.A. An evaluation of fundamental driver decisions and reactions at an intersection. Highway Research Record, No. 118, pp. 68-84, 1966.

WENNELL, J. The effects of flow on gap acceptance behaviour at T-junctions. Report No. RHC/MOR/TRRL/30, Mathematics Department, Royal Holloway College, Egham, Surrey, April 1980.

WENNELL, J. and D.F. COOPER. A survey of factors in road traffic accidents. Report No. RHC/MOR/HOR/6, Mathematics Department, Royal Holloway College, Egham, Surrey, July 1977.

WEST, L.B. and J.W. DUNN. Accidents, speed deviation and speed limits. _Traffic Engineering_, Vol. 41, No. 10, pp. 52-55, 1971.

WRIGHT, P.H. and K.K. MAK. Single vehicle accident relationships. _Traffic Engineering_, Vol. 46, No. 1, pp. 16-21, 1976.

APPENDIX ONE

PROBIT ANALYSIS

Probit analysis (Finney, 1971) is appropriate to situations where subjects make all-or-nothing (ie quantal) responses to a stimulus. In general, the type of response depends on the intensity of the stimulus, and there is a certain level of intensity (called tolerance) below which the response does not occur and above which it occurs. This tolerance value varies from one subject to another in the population observed, and may also vary from one occasion to another.

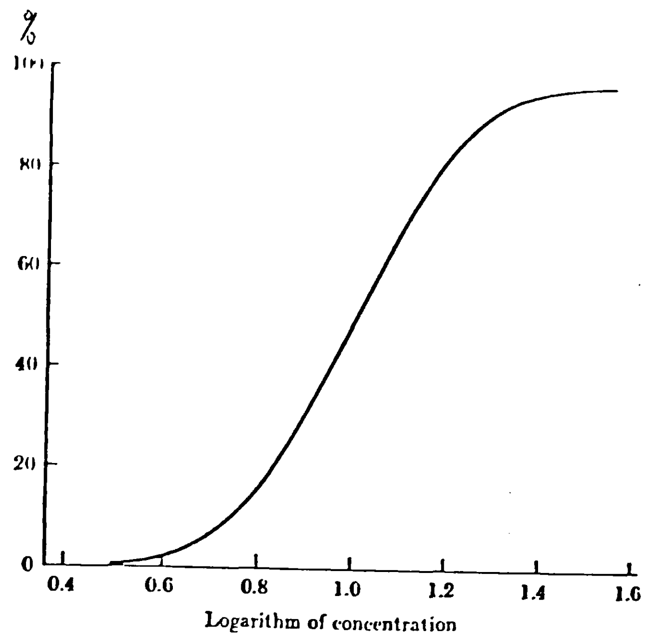
The gap acceptance process is a good example of the situation outlined above: the quantal response is either acceptance or rejection, and the stimulus is the presented gap; the size of the presented gap is a measure of the intensity of the stimulus (called dose). Tolerance is analagous to the concept of a critical gap for individual drivers. It is clearly appropriate to use this technique to analyse gap acceptance data, as several researchers have done before us (see, for example, Ashton, 1971; Bottom and Ashworth, 1977 and 1978).

Finney discusses quantal response data in terms of the frequency distribution of tolerances over the population studied, and shows that behaviour can be represented by the normal sigmoid curve depicted in Figure 11.1. This curve plots the proportion of subjects who respond to a dose against the logarithm of the dose, and a similar shape is obtained when we plot observed probability of acceptance against the size of the presented gap on a logarithmic scale. Finney characterises this curve by the median effective dose, which is the dose that produces a response in half the population. For any distribution of tolerances, the value of this parameter is estimated by the dose corresponding to the 50% level of response. For a log-normal curve in particular, the antilog of this parameter corresponds to the mean of the underlying normal distribution. Calculation of this value is discussed in the next section.

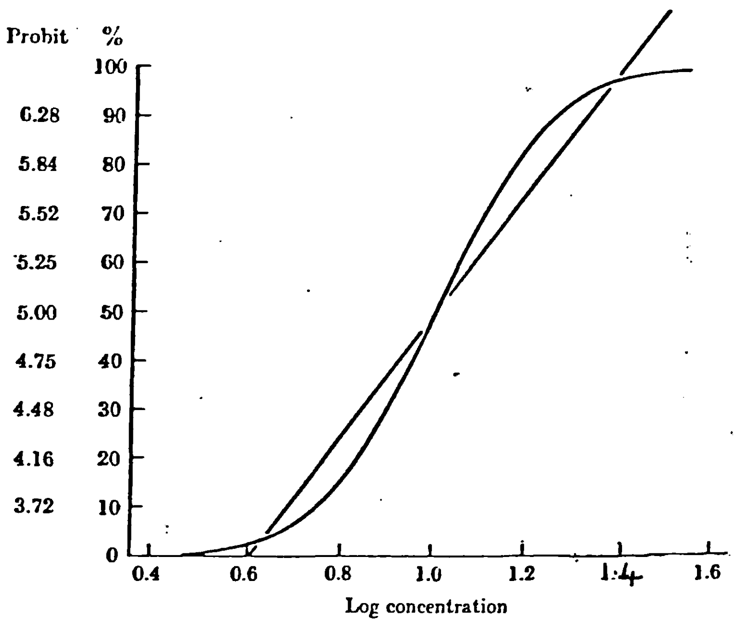
The N.E.D. transformation

The normal equivalent deviate (N.E.D) transformation measures the probability of response P on a different scale. The N.E.D. of any value of P between 0 and 1 is defined as the abscissa corresponding to a probability P in a normal distribution with mean 0 and variance 1. The probit of P is simply the N.E.D. plus 5, which is positive for nearly all values of P (values of the N.E.D. are negative if P is less than 50%). The effect of this transformation on a normal sigmoid curve is shown in Figure 11.2; the relationship between the probit of the expected proportion of response and the dose is linear.

Analysis of experimental data is now reduced to estimating the parameters of a straight line. This is done by a computer program at RHC, which is based on the calculations outlined by Finney. Input to the program consists of the number of presented gaps and the number of acceptances in particular gap size intervals. The proportion of acceptance is obtained for each interval, and the mid-point of the interval is taken as the dose. The probit regression line which gives the best fit to this data is then calculated.



<u>Figure 11.1</u> Normal sigmoid curve, showing percentage of subjects with log tolerances less than a specified value.



<u>Figure 11.2</u> Effect of the probit transformation. The normal sigmoid curve is transformed to a straight line when the ordinates are measured on a scale linear in probits instead of in percentages.

We represent a gap acceptance function by two parameters: the median accepted gap M and the variability parameter S. Estimates of these parameters are derived from the intercept A and slope B of the calculated probit line as follows:

$$M = e^{-A/B} \quad \text{and} \quad S = e^{1/B}.$$

These parameters are themselves related to the mean and standard deviation of the underlying normal distribution (Finney, 1971).

Differences between the values of M for two groups of data can only be tested if S is similar for both groups ie if the probit lines are parallel. This test is based on the fiducial limits for the difference in median values given in Chapter 6 of Finney's book. Provided zero does not fall within the range of these limits, we can say that the median accepted gaps are significantly different.

APPENDIX TWO

EXPERIMENTAL SITES

The experimental sites listed in Table 3.1 are described in this appendix, and details of the data recorded at each site are given. Video techniques were used only at Denton and Tongham; data was collected with the microprocessor-based system at all sites except Denton.

Denton

The Denton site is at the intersection of the A259 Newhaven to Eastbourne road and the B2109 in East Sussex (Figure 12.1). A special feature of this site is the one way by-pass which diverts traffic turning left into the B2109 before it reaches the main part of the intersection; this simplifies the task of drivers turning at the junction. The site was filmed during morning peak periods on several weekdays in the summer of 1975. Speeds of individual eastbound major road vehicles were measured using pneumatic tubes and a Venner timer, and the time interval between cable triggers for each vehicle was recorded on the audio track of the videotape.

The following details were later extracted from the videotapes:

i Arrivals of eastbound major road vehicles at the junction.

ii Arrivals and departures of vehicles turning right from the major road into the minor road.

iii Clearing events for these turning vehicles, which occur when the rear of the vehicle crosses into the minor road.

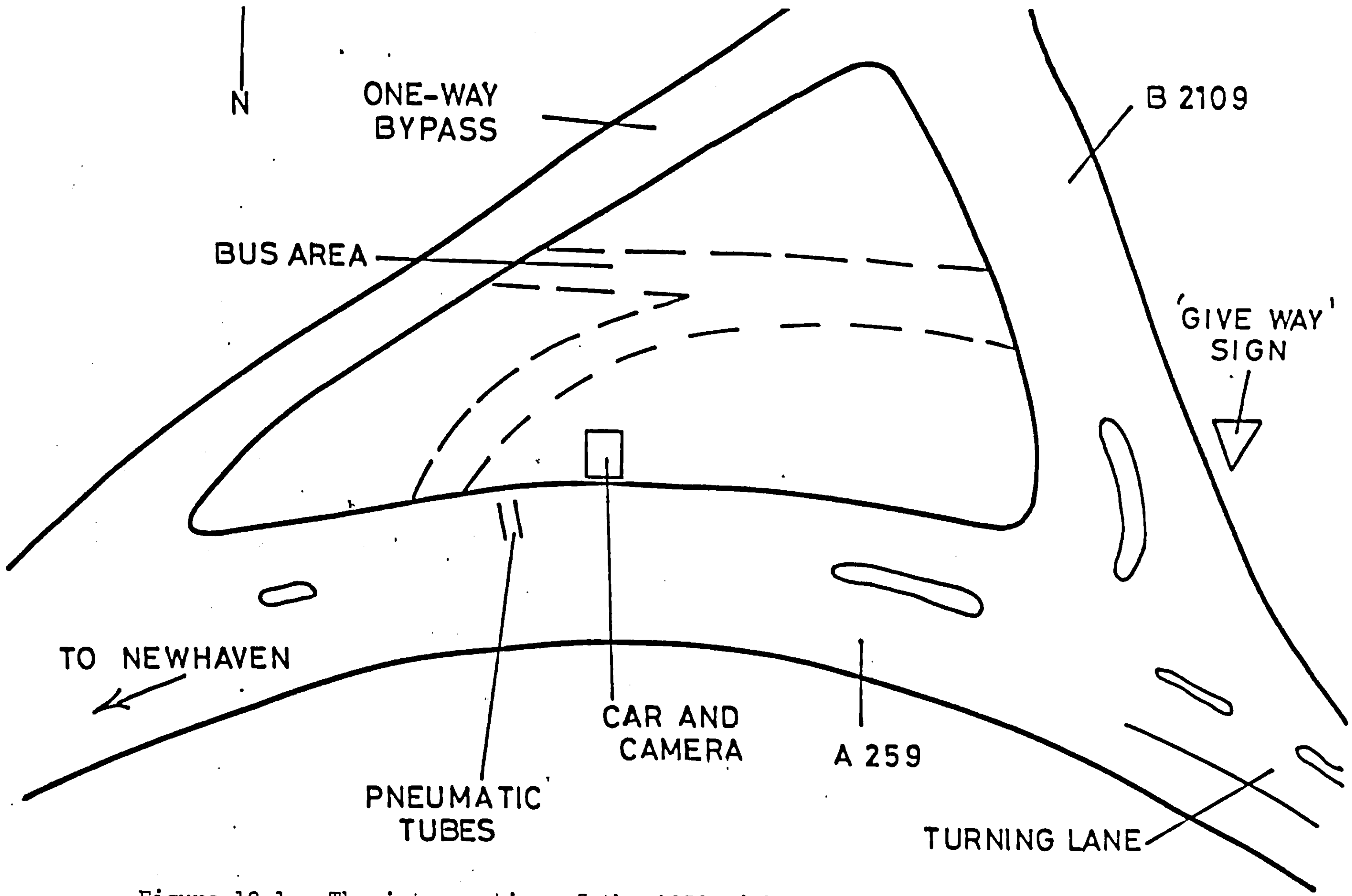


Figure 12.1 The intersection of the A259 with the B2109 near Denton in Sussex.

All these events were coded by the kind of vehicle concerned. Further experiments conducted at this site are described by Cooper (1976).

Tongham

The Tongham intersection is formed where the A3014 meets the A31 in Surrey (Figure 12.2). There is a small side road nearby, but it carries little traffic and has a negligible effect on turning manoeuvres at the main junction. The A31 is a major trunk road, and carries heavy commuter traffic to Guildford and Farnham. Observations were made at this site using both techniques of data collection.

The site was filmed with a video camera during morning and evening peak periods on three weekdays in April 1976. Details of merging gap acceptance behaviour were later recorded from these videotapes *ie* arrivals of nearside major road vehicles; arrivals and departures of vehicles turning left from the minor road. The times at which merging vehicles reached a fixed point in the major road after turning were also noted. All these events were classified by the kind of vehicle in each stream.

A further experiment was conducted at this site in May 1978. Data was collected using the microprocessor-based system during morning and evening peak periods on four weekdays: details of the events recorded at this site are given in Table 12.1. Due to equipment problems, speeds of eastbound major road vehicles were sampled at a later date (during one evening peak period in July).

A3/B3000 intersection

Two T-junctions are located at the intersection of the B3000 with the A3 a few miles south of Guildford in Surrey (Figure 12.3).

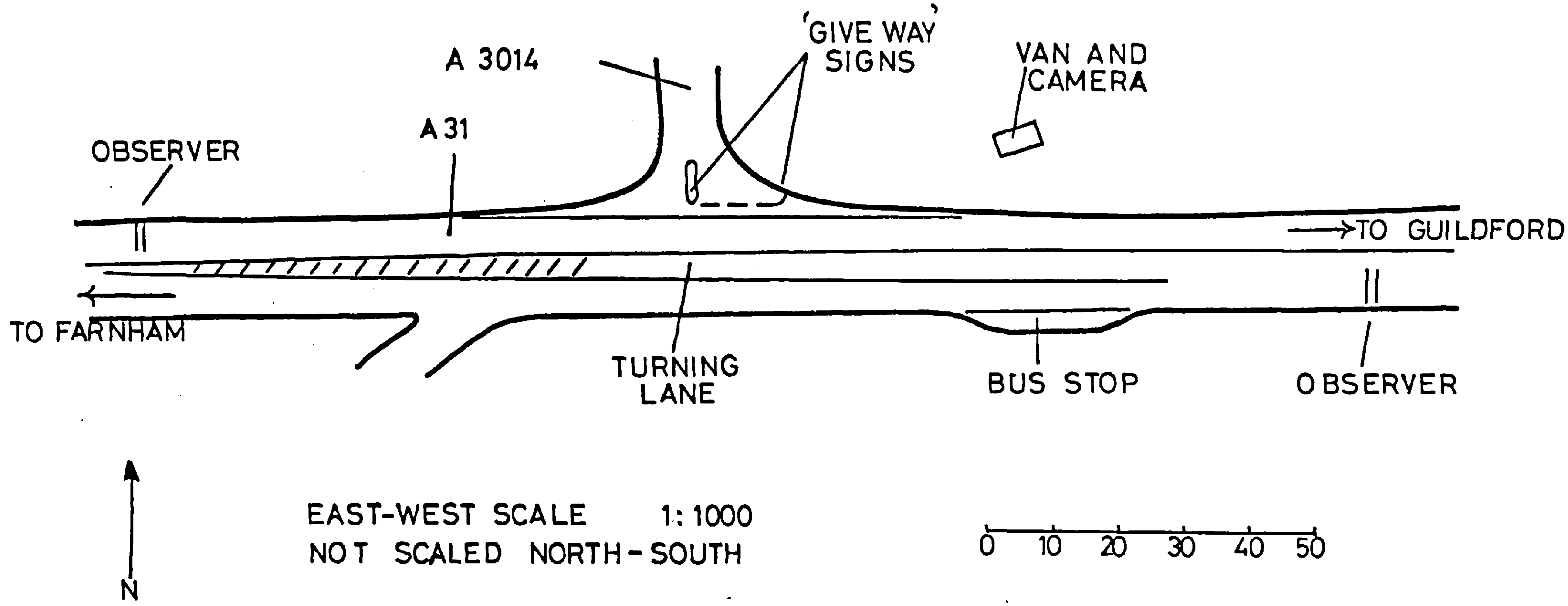


Figure 12.2 The junction of the A31 and the A3014 near Tongham in Surrey.

BUTTON NUMBER		HANDSET NUMBER ONE	TWO	THREE
	1	RA arrival (car)	LT turn (car)	SL turn (car)
	2	RA arrival (van)	LT turn (van)	SL turn (van)
	3	RA arrival (truck)	LT turn (truck)	SL turn (truck)
	4	RA arrival (other)	LT turn (other)	SL turn (other)
	5	RT turn	LT arrival	SL arrival
	6	-	Female driver	Female driver
	7	-	Male driver	Male driver

Table 12.1 Events recorded at Tongham (1978) using the microprocessor-based system.

We refer to the eastern arm of this intersection as Compton, and the western arm as Puttenham; these junctions are about half a mile apart. The A3 is a major trunk road between Guildford and Portsmouth, and carries heavy commuter traffic in peak periods. Data was collected at both junctions with the microprocessor-based system.

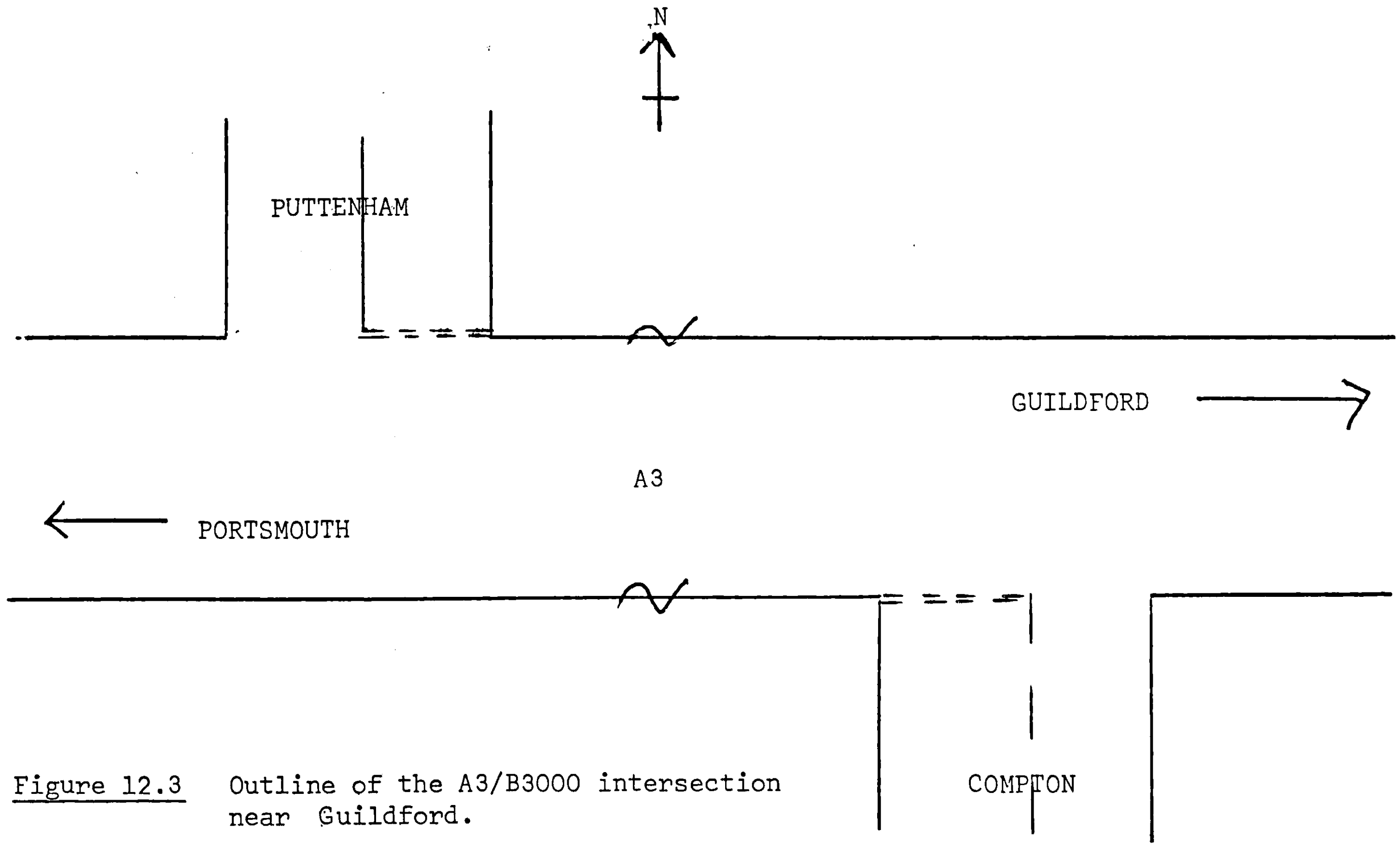
Compton

Although roads are level in the immediate vicinity of this junction, there is a steep uphill gradient in the minor road as it approaches the junction, and a slight downhill gradient in the major road on the approach from the Guildford direction. Observations were made during six morning or evening peak periods on weekdays in July 1978. Arrivals of nearside major road vehicles were coded by the kind of vehicle. Data for merging and crossing vehicles was classified by either the kind of vehicle turning or the gender of the driver and the presence of passengers. Speeds of major road vehicles were measured as they approached the junction, using pairs of automatic sensors.

Puttenham

A particular feature of this site is the rather long merging lane which allows drivers turning left from the minor road to merge with major road traffic several yards downstream from the junction; for the purposes of our data collection, such vehicles were ignored. There is a small side road near the junction, but traffic using it does not interfere with the operation of the B3000 intersection.

Observations were made during six morning or evening peak periods on weekdays in August 1978, but speeds of approaching major road vehicles were only sampled on one occasion.



<u>Figure 12.3</u> Outline of the A3/B3000 intersection near Guildford.

During morning peak periods, gap acceptance data was collected for all turning manoeuvres, classified by the kind of vehicle turning and the gender of its driver. Only the crossing manoeuvre was observed during evening peak periods, however, since flows out of the minor road were rather low then; similar events to those collected at Denton were recorded.

Peasmarsh

The Peasmarsh intersection is situated at the junction of the A3100 with the B3000 near Guildford in Surrey. The major road did not contain a turning lane at the time of our experiment, but the junction has since been altered to provide a separate lane for crossing drivers. Observations were made during four peak periods in June 1978; bad weather prevented any further data collection during June, so the amount of data available for this site is minimal. All events were coded by the kind of vehicle only.

Broadford Road

Observations were made at the junction of Broadford Road (A248) with the A3100 during five peak periods in June 1979. Gap acceptance data for crossing and merging traffic was coded by the kind of vehicle in each stream. No turning lanes are provided at this site.

Shalford

The Shalford site is located at the southern arm of the intersection of the A248 with the A281; the other arm of this intersection has recently been converted to a mini-roundabout. Observations were made at this junction during the week following that of the experiment at Broadford Road, and similar events to those detailed above were recorded. Due to problems with tapes, data collected during morning peak periods could not be recovered.

APPENDIX THREE

PROGRAMS USED TO PROCESS DATA
COLLECTED AT T-JUNCTIONS

The initial versions of the programs described in this appendix were written by Dr Dale Cooper. All data to be analysed must be in a particular format, which is described in the first section. The processing programs fall into four distinct groups, which are treated separately below: error checking routines, simple gap acceptance tables, double gap acceptance tables and miscellaneous programs. Modifications made to these programs so that the additional data collected using the microprocessor-based system could be processed are detailed in the final section.

Data format

Each event may contain several details, the first two of which are compulsory:

i) DATE/TIME (T) This consists of twelve digits, YYMMDDCCTTTT, where

YY	represents the year in which the data was collected eg 78 for 1978;
MM	represents the month eg 01 for January;
DD	represents the day of the month;
CC	represents the cycle number on the clock, which is incremented every one thousand seconds when the digital timer recycles. Cycle numbers start at 01 for data collected in morning peak periods, and at 11 for evening data;
TTTT	is the timer reading in tenths of seconds.

ii) STREAM (S) This consists of a single digit from 1 to 9. Values of 1 to 6 represent the different streams of vehicles which pass through the T-junction (Figure 13.1):

1 LA vehicle; non-turning vehicle from the left

2 LT vehicle; turning vehicle from the left

3 SL vehicle; turning left from the minor road

4 SR vehicle; turning right from the minor road

5 RA vehicle; non-turning vehicle from the right

6 RT vehicle; turning vehicle from the right

Stream 7 indicates the end of a film and hence a discontinuity in the data; Stream 8 denotes the end of data on the file and is always the final event, while Stream 9 represents a comment event.

iii) ACTION (A) This consists of a single digit from 1 to 5 where:

1 (or blank) represents an arrival ie a vehicle reaches the junction;

2 represents the start of a turn eg a vehicle crosses the give way lines in the minor road;

3 represents a clearing event (LT or SR vehicles only) which occurs when the vehicle has cleared the path of oncoming RA vehicles;

4 represents a marker event (SL vehicles only);

5 represents the turn of an SL vehicle which edges along a merging lane before accepting a gap.

iv) KIND (K) This consists of a single digit from 1 to 6 where:

1 (or blank) represents a car

2 represents a van or light goods vehicle

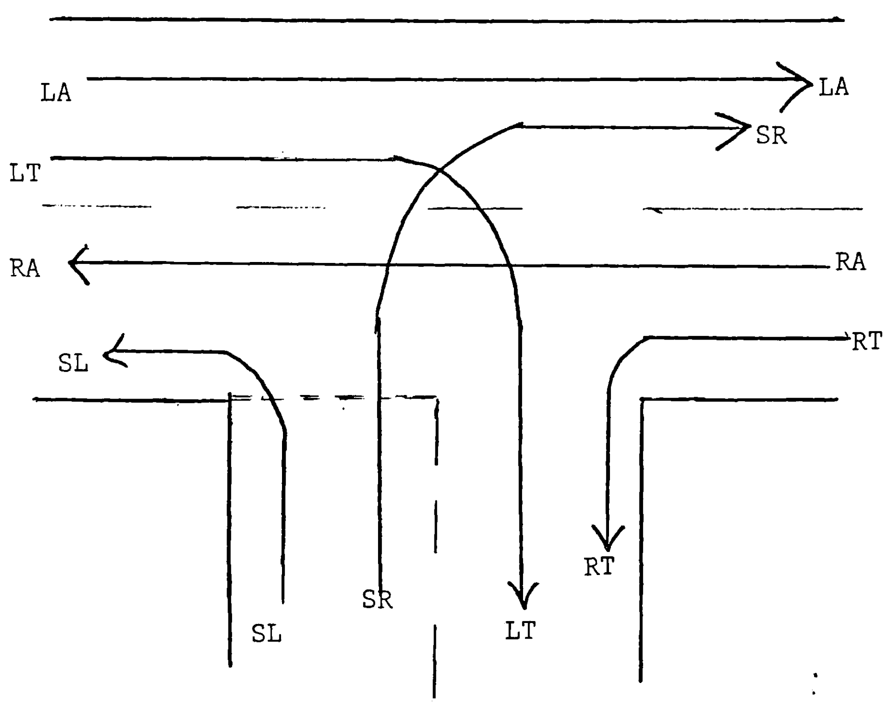


Figure 13.1 Six traffic streams at a T-junction.

3 represents a truck or heavy goods vehicle

4 represents a bus or coach

5 represents a two-wheeled vehicle

6 represents any other kind of vehicle.

v) SPEED (V) This consists of three digits, and corresponds to the Venner timer reading in milliseconds for the interval between the crossing of each of two pneumatic tubes in the major road.

vi) COMMENT (C) This information can be up to 60 characters long.

The processing programs assume that each such event occurs on a separate line of the data file, as a computer card image. T, S, A, K and V occupy the first eighteen digits, while C occupies the final sixty digits on the card. The complete data file may be printed out (250 events to a page) if required.

Error checking routines

CHEKPUN. This routine checks the data for valid codes viz. the value of S may be any number between 1 and 9, A may be 1 to 5 (or blank) and K may be 1 to 6 (or blank). It also checks for valid combinations of codes; for example, only arrival events are recorded in major road streams, hence if S = 1 or 5, A must be 1 or blank. If no coding errors are detected, the routine prints a table of the total numbers of events recorded for each possible value of S, A and K. For example, the total number of cars turning left out of the minor road is given when S = 3, A = 2 and K = 1.

CHEKSEQ. This routine checks that the events are sequenced correctly in time, so each event should occur at a later time than the previous one. If no errors are detected, the duration of each film (in seconds) is printed.

CHECKLT and CHECKSL. These routines have a similar structure. They check the logic for vehicles in streams LT and SL, to ensure that arrivals are followed by turns. An error message is generated if two arrival events occur consecutively in the same stream.

CHEKCLR and CHEKMKR. These routines also have a similar structure. CHEKCLR checks the sequencing of turning and clearing events for streams LT and SR, while CHEKMKR does the same for turning and marker events in stream SL. Error messages are printed if turn or clear/marker events occur singly, or if more than two vehicles turn before a clear/marker event occurs.

Simple gap acceptance tables

These routines produce tables of accepted and rejected lags and gaps for LT or SL vehicles (Table 13.1); the procedure for incrementing an entry in these tables is similar for all routines. The data file is searched for events in streams LT (or SL) and RA/RT only. The process is initiated by the arrival or immediate turn of an LT (or SL) vehicle. If the vehicle turns at once, the time of the turn is stored, and subtracted from the time of arrival of the next major road vehicle, to give the accepted lag. If the vehicle stops, its arrival time is subtracted from the arrival time of the next major road vehicle, to give the rejected lag; inter-arrival times of subsequent major road vehicles are stored as rejected gaps, and when the vehicle eventually turns,

GAP SIZE INTERVAL	APPROACHING VEHICLE Car	Van	Truck	Total
1	0/90	0/7	0/3	0/100
2	1/69	0/6	0/4	1/79
3	5/54	1/5	0/5	6/64
⋮	⋮	⋮	⋮	⋮
18	15/0	8/0	2/0	25/0
19	20/0	6/0	4/0	30/0
20	40/0	10/0	10/0	60/0

Table 13.1 Example of a simple gap acceptance table classified by the kind of vehicle approaching; separate tables are produced by computer for each kind of turning vehicle. Gap size intervals 2 to 19 are usually half second ranges between one and ten seconds; interval 1 corresponds to gaps less than one second, and interval 20 corresponds to gaps greater than ten seconds. Each entry in the table is in the form A/R, where A gives the number of acceptances and R gives the number of rejections; lags and gaps are tabulated separately.

the time between arrivals of the previous major road vehicle and the next major road vehicle is stored as the accepted gap.

Each routine classifies these gap acceptance tables differently: LTGAPS and SLGAPS produce gap acceptance tables classified according to the kind of vehicle turning (car, van, truck and total) and either the kind of vehicle approaching, or the speed of the next major road vehicle (calculated from the Venner timer reading). For LT only, gap acceptance tables classified by vehicle type are produced for lags and gaps measured in distance as well as time. SLMARK and LTCLEAR produce gap acceptance tables classified according to the kind of vehicle turning (car or total) and its manoeuvre time. The manoeuvre time of LT vehicles is calculated as the interval between consecutive turning and clearing events, while that for SL vehicles is the difference between appropriate turning and marker events.

Double gap acceptance tables

Gaps in both major road streams of traffic must be considered for vehicles turning right out of the minor road (stream SR). In order to analyse this complex manoeuvre, a temporary file is produced by SRGAPS which contains the following information for each pair of gaps in major road traffic (a gap to the left and a gap to the right):

a) time of initiating event - time of arrival of SR vehicle for lags, and time of arrival of first major road vehicle for gaps;
b) time of arrival of next vehicle from left (LA or LT) and right (RA or RT);
c) kind of vehicle in each stream ie turning vehicle and next vehicle from left and right;

d) flags denoting whether lag or gap, and acceptance or rejection;

e) stream of next vehicle from the left (LA or LT);

f) speed of next vehicle from the right;

g) stream of initiating vehicle (for gaps only).

This information is used by SRA to produce tables of accepted and rejected lags and gaps indexed by gap sizes to left and right in one second intervals (Table 13.2). These tables are classified by the stream of the next vehicle from the left (LA or LT) and by the kind of vehicle turning. Further gap acceptance tables classified according to the streams opening and closing the gaps are produced by SRB.

Miscellaneous programs

These programs produce distributions of various traffic parameters. VENNERD calculates the speed distribution of RA vehicles from Venner timer readings. Parameters of the distribution (assumed normal) are given in 5 mph ranges, for each kind of vehicle separately. EXPOT produces a distribution of manoeuvre times for LT or SL vehicles. Parameters of the distribution are calculated in ten 0.3 s ranges, which are generally different for the two manoeuvres; these ranges can be easily altered. Distributions for different kinds of turning vehicles are obtained by controlling the value of the parameter K selected.

QACC1 produces a distribution of rejected lags for LT or SL vehicles. QACC2 calculates the distribution of start-up times for LT or SL vehicles; the start-up time is defined as the interval between the turn of the first vehicle accepting a gap and the arrival of the major road vehicle which opened the gap. QACC3

GAP TO LEFT	GAP TO RIGHT 1	2	3	......	9	10+
1	0/100	0/10	0/5	......	0/5	0/30
2	0/50	0/6	0/4	......	1/4	3/27
3	0/20	1/8	2/6	......	4/6	5/8
⋮	⋮	⋮	⋮		⋮	⋮
9	0/25	1/9	3/7	......	25/0	30/0
10+	0/30	2/8	4/6	......	20/0	100/0

Table 13.2 Example of a double gap acceptance table, indexed by gap to the left and gap to the right (ie farside and nearside streams of priority traffic respectively). Intervals correspond to one second ranges from one to ten seconds; interval 10+ corresponds to gaps greater than ten seconds. Each entry is in the form A/R, where A and R are the numbers of acceptances and rejections respectively.

produces distributions of move-up times for LT or SL vehicles, classified according to position within a queue. The move-up time is the interval between consecutive turns of vehicles accepting the same gap, or between the time of the last turn and the next LT/SL arrival before the arrival of the major road vehicle which closes the gap.

Modifications to processing programs

Some changes had to be made to the processing programs before data collected using the microprocessor based system could be analysed. Times were recorded in hundredths of seconds rather than tenths of seconds, so the format of the event was changed slightly. Information for DATE/TIME is now in the form YYMDDCCTTTTT, so only one digit is allowed for the month, leaving five digits for the clock time.

Three additional action codes have been introduced (small changes were made to subroutine CHEKPUN to allow for these new codes):

6 and 7 denote the triggering of each of the pair of automatic sensors used to collect speed information; the programs assume that 6 occurs first.

8 denotes that the event contains information about the occupants of the vehicle. The value of K determines the type of information; if K = 1, the driver is male, and if K = 2, the driver is female; if K = 3, the vehicle contained passengers.

Vehicle type codes have been modified to take account of the less detailed information recorded at each site. The new values are as follows:

K = 1 represents a car

K = 2 represents a van or light goods vehicle

K = 3 represents a truck or heavy goods vehicle

K = 4 represents any other kind of vehicle

K = 5 indicates that the vehicle type was not recorded eg for automatic sensor triggers.

Two additional error checking routines have also been written:

CHKCOAX checks the sequencing of triggers of the automatic sensors used to collect speed information (for each major road stream separately). It assumes that action 6 occurs first, and checks to see whether the next event in that stream is an action 7. Error messages are printed if either sensor is triggered alone, or if a major road arrival occurs between two triggers (this helps to detect cases in which one vehicle triggers cable 6 only, while another vehicle triggers cable 7 only).

CHKSAPS checks whether information about the gender of the driver is recorded after every turn event in streams LT, SL and SR (if such data was recorded). Error messages are generated if a turning event is not followed by an action 8 event (with K = 1 or 2), or if more than one such event is recorded for a particular turning vehicle.

Speed distributions for streams LA and RA are calculated in subroutine COAXSPD. This routine also prints error messages if sensors are triggered singly or simultaneously. Warning messages are printed if the time between triggers is very short or very long (generally for speeds outside the range 10 to 70 mph).

In order to analyse the new data with as few modifications as possible, one simple change was made to all the gap acceptance programs: a statement was inserted each time an event was read from the data file which caused the event to be ignored if the value of A was greater than 5 (and so the next event was read immediately). Calculations of all time intervals were altered because of the change of time base.

The modifications described above were fairly straightforward, and enabled the same suite of programs to be used to analyse data collected with the microprocessor-based system. Details of the programs used to transfer the data from cassette tape to a data file suitable for input to the processing programs are given in Appendix 4.

APPENDIX FOUR

TRANSFORMATION OF DATA FROM CASSETTE TAPE

Data format

Data collected using the microprocessor-based system is stored on cassette tape in blocks. Each block of data consists of four lines:

1) String of 8's

2/3) Ten data events

4) String of 9's

The first and last lines act as delimiters. Each line of data consists of ten events, and every event is in the form TTTTTHBP, where

TTTTT	represents the clock time in hundredths of seconds;
H	represents the input port through which the data was received, and usually corresponds to a particular handset;
B	represents the individual channel on the input port, which generally corresponds to a particular button on a handset;
P	is a parity digit.

Processing programs

Three programs are used to process the data from the cassette tape after its transfer to a mainframe computer. The first of these gives a complete listing of the data on the file; the second checks the parity digit, corrects a software error which occurs when two events are recorded at the same time, and produces an interim data file without the delimiters; the third transforms the data ready for input to the suite of programs described in Appendix 3.

EVENTS This program produces a complete listing of the events on the file. Data is printed ten events to a line, without the parity digit: times, handset numbers and button numbers are printed in separate columns for clarity. Any errors in blocking of the data show up clearly in this listing, since an almost complete line of zeroes is printed when a set of 8's or 9's is treated as a line of ten events. If such errors do occur, the incomplete block is deleted, and an end of film event added later.

CHKDATA This program was written by Mr P Storr. It checks the accuracy of the data transfer by means of the parity digit contained in each event, and prints an error message if mistakes occur. Such errors are usually corrected interactively, after the data listing has been compared with the original file on the cassette tape. Due to a fault in the programming of the micro-processor, incorrect events are recorded whenever two input channels on different ports are triggered during the same cycle ie if buttons on different handsets are pressed at the same time. Such errors do not occur very often however, and they are automatically corrected by this program. Once all errors have been removed from the file, CHKDATA creates an interim data file with each event written on a separate line; the parity digit and delimiters do not appear in this new data file.

TRANS This program creates a new data file which is suitable for input to the processing programs described in Appendix 3. Each event of the form TTTTTHB from the interim file is rewritten in the form YYMDDCCTTTTTSAK. The initial value of YYMDDCC is specified for each data file; CC starts at 1 for data collected in the morning and at 11 for evening data. The values of S, A and K are determined from the table of events recorded for each data collection period.

Separate sections of the program assign particular values of S, A and K eg for LA arrivals detected by an automatic sensor, S = 1, A = 1 and K = 5. The program branches to these sections according to the values of H and B, so the branching statements must be altered for each data file; in practice, similar events were recorded at most junctions, so only a few different sets of branching statements exist. Error messages are generated if an invalid handset or button number occurs in the data file.

The zero button on each handset was reserved for an error code. Observers pressed this button once if their previous entry was a mistake, and pressed it three times if they completely missed recording a vehicle (two consecutive error codes mean that the zero button was pressed by mistake). TRANS prints a warning message when zero buttons are detected, and the event is not transferred to the new file.

APPENDIX FIVE

RESULTS FOR EXPERIMENTAL SITES USED IN VALIDATION STUDY

This appendix contains several tables of results from the six experimental sites used for the validation study. In all these tables, morning and evening data have been treated separately. Data was not available for merging at Puttenham during the evenings, nor for Shalford in the mornings. The contents of each table are summarised below:

1) Average crossing, merging and major road flows (in vehicles per hour).
2) Speed distribution of vehicles in nearside stream of major road traffic (in feet per second). Data for Puttenham and Tongham were only collected during one evening peak period.
3) Crossing and merging gap acceptance parameters (in seconds).
4) Model conflicts involving crossing vehicles for Assumption 1 (constant exposure time).
5) Model conflicts involving crossing vehicles for Assumption 2 (normal distribution of exposure times).
6) Model conflicts involving merging vehicles.

In Tables 4, 5 and 6, the sites have been ranked by the number of conflicts in decreasing order; hence rank 1 corresponds to highest risk.

SITE	MAJOR ROAD (Straight on/Turn)	CROSSING	MERGING
MORNING			
Broadford Road	335/40	285	275
Compton	495/85	245	205
Peasmarsh	885/110	175	300
Puttenham	920/135	145	140
Tongham	850/85	200	245
EVENING			
Broadford Road	605/100	215	395
Compton	935/145	185	210
Peasmarsh	420/100	325	145
Puttenham	590/140	180	-
Shalford	370/70	355	280
Tongham	570/80	250	160

Table 15.1 Average flows in each traffic stream (in vehicles/hour).

SITE	MORNING		EVENING	
	Mean	Standard Deviation	Mean	Standard Deviation
Broadford Road	49.2	12.4	52.0	9.5
Compton	58.4	14.0	61.1	11.7
Peasmarsh	46.5	10.5	49.8	12.1
Puttenham	-	-	51.0	13.9
Shalford	-	-	46.7	10.0
Tongham	-	-	51.5	10.9

Table 15.2 Speed distribution of nearside major road vehicles (in feet/second).

SITE	MORNING Median Accepted Gap	MORNING Variability Parameter	EVENING Median Accepted Gap	EVENING Variability Parameter
CROSSING				
Broadford Road	3.36	1.55	3.83	1.52
Compton	3.74	1.51	3.56	1.52
Peasmarsh	4.09	1.40	3.62	1.46
Puttenham	3.79	1.51	3.94	1.49
Shalford	-	-	3.82	1.44
Tongham	3.74	1.47	3.96	1.45
MERGING				
Broadford Road	3.21	1.73	3.65	1.75
Compton	3.65	1.63	4.09	1.57
Peasmarsh	4.08	1.64	4.13	1.76
Puttenham	3.90	1.72	-	-
Shalford	-	-	3.36	1.88
Tongham	4.01	1.56	3.93	1.69

Table 15.3 Crossing and merging gap acceptance parameters for each site.

SITE	NUMBER OF CONFLICTS (RANK) Grades 1 & 2	Grades 3 - 5	Total
MORNING			
Broadford Road	3.4 ($1\frac{1}{2}$)	0.9 (1)	4.3 (1)
Compton	1.2 (4)	0.6 (2)	1.8 (4)
Peasmarsh	0.3 (5)	0.1 (5)	0.4 (5)
Puttenham	3.4 ($1\frac{1}{2}$)	0.5 (3)	3.9 (2)
Tongham	2.2 (3)	0.3 (4)	2.5 (3)
EVENING			
Broadford Road	2.1 (2)	0.3 (2)	2.4 (2)
Compton	6.3 (1)	1.9 (1)	8.2 (1)
Peasmarsh	1.3 (4)	0.0 ($5\frac{1}{2}$)	1.3 (4)
Puttenham	1.4 (3)	0.2 (3)	1.6 (3)
Shalford	0.5 (5)	0.0 ($5\frac{1}{2}$)	0.5 (5)
Tongham	0.2 (6)	0.1 (4)	0.3 (6)

Table 15.4 Numbers of conflicts involving crossing vehicles when exposure time is held constant (average of 10 runs of 10 hours each). Sites are ranked in decreasing order.

SITE	NUMBER OF CONFLICTS (RANK)		
	Grades 1 & 2	Grades 3 - 5	Total
MORNING			
Broadford Road	4.2 ($1\frac{1}{2}$)	2.2 (3)	6.4 (2)
Compton	3.2 ($3\frac{1}{2}$)	2.4 ($1\frac{1}{2}$)	5.6 (3)
Peasmarsh	0.4 (5)	0.2 (5)	0.6 (5)
Puttenham	4.2 ($1\frac{1}{2}$)	2.4 ($1\frac{1}{2}$)	6.6 (1)
Tongham	3.2 ($3\frac{1}{2}$)	1.3 (4)	4.5 (4)
EVENING			
Broadford Road	3.3 (2)	1.5 (2)	4.8 (2)
Compton	6.6 (1)	6.5 (1)	13.1 (1)
Peasmarsh	3.1 (3)	0.6 ($4\frac{1}{2}$)	3.7 (3)
Puttenham	1.8 (5)	0.6 ($4\frac{1}{2}$)	2.4 (5)
Shalford	1.5 (6)	0.2 (6)	1.7 (6)
Tongham	2.3 (4)	0.7 (3)	3.0 (4)

Table 15.5 Numbers of conflicts involving crossing vehicles when exposure times are sampled from a normal distribution (average of 10 runs of 10 hours each). Sites are ranked in decreasing order.

SITE	NUMBER OF CONFLICTS (RANK)		
	Grades 1 & 2	Grades 3 - 5	Total
MORNING			
Broadford Road	337.7 (5)	23.9 (5)	361.6 (5)
Compton	416.1 (3)	32.5 (3)	448.6 (3)
Peasmarsh	561.5 (2)	32.9 (2)	594.4 (1)
Puttenham	349.0 (4)	43.5 (1)	392.5 (4)
Tongham	561.6 (1)	27.3 (4)	588.9 (2)
EVENING			
Broadford Road	763.2 (1)	62.5 (1)	825.7 (1)
Compton	670.4 (2)	59.0 (2)	729.4 (2)
Peasmarsh	179.6 (5)	13.1 (5)	192.7 (5)
Shalford	315.5 (3)	28.2 (3)	343.7 (3)
Tongham	282.8 (4)	18.0 (4)	300.8 (4)

Table 15.6 Numbers of model conflicts involving merging vehicles at each site (average of 10 runs of 10 hours each). Sites are ranked in decreasing order.

APPENDIX SIX

PUBLISHED MATERIAL

Copies of the following papers published jointly with other members of Royal Holloway College are contained in this appendix:

1) COOPER, D.F., P.A. STORR and J. WENNELL. Traffic studies at T-junctions : 4. The effect of speed on gap acceptance and conflict rate. Traffic Engineering and Control, Vol. 18, No.3, pp. 110-112, March 1977.

2) WENNELL, J., D.F.COOPER, P.A. STORR and M.R.C. McDOWELL. Risk factors in accidents. In Proceedings, Traffex '77, Printerhall Ltd, London.

3) COOPER, D.F. and J. WENNELL. Models of gap acceptance by queues at intersections. Traffic Engineering and Control, Vol.19, No. 4, pp. 178-180 and p. 185, April 1978.

4) STORR, P.A., J. WENNELL, D.F. COOPER and M.R.C. McDOWELL. A microprocessor-based system for traffic data collection. Traffic Engineering and Control, Vol.20, No.4, pp. 156-158, April 1979.

Traffic studies at T-junctions

4. The effect of speed on gap acceptance and conflict rate

by Dale F. Cooper, P. A. Storr and Jenny Wennell
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Introduction. In a previous paper[1], we examined the behaviour of drivers turning out of a minor road and merging with the nearside stream of major road traffic. Accepted and rejected gaps were classified according to the speeds of the approaching major road vehicles, and it was found that the median accepted distance gap D in feet could be expressed as a simple function of the approach speed V in ft/sec. by

$$D = 38+5V \qquad ......(1)$$

i.e. the median accepted gap could be regarded as a constant time of five seconds plus a constant distance of 38 ft (11·5 m). In this paper, a similar equation is obtained for a simple crossing movement, and the results for both crossing and merging are evaluated in a conflict model[2].

Crossing gap acceptance

As part of Project 2001 of the Home Office Police Scientific Development Branch (PSDB), an experiment was conducted at a semi-rural T-junction in East Sussex in the late summer of 1975. The experimental team involved members from the PSDB, the Civil Systems Research Unit of the Plessey Company, the Sussex Police and the Department of Mathematics of Royal Holloway College. The intersection of the A259 (the major road) and the B2109, near Denton, was filmed using portable television equipment; the videotapes were processed and analysed to obtain a record of events and turning movements during morning peak periods over several months. Speeds of vehicles in one lane of the main road were measured with pneumatic tubes and a Venner timer, and incorporated in the record of events. A more complete description of the methods of observation and analysis may be found elsewhere[3].

A simple right-turn movement was studied, in which a vehicle turns from the A259 across a stream of east-bound traffic into the B2109 (Fig 1). Accepted and rejected gaps were classified according to the speeds of the approaching vehicles, and a log-normal gap acceptance function was fitted to the data in each 5 mile/h speed-band using probit analysis[4]. Median accepted gaps for each speed V were expressed in terms of both time T and distance D ($=VT$) (Table I). There is a significant correlation between the median accepted distance gap D and the approach speed V, at the 5 per cent level; for D in ft and V in ft/sec.,

$$D = 115+1{\cdot}9V \qquad ......(2)$$

or

$$T = 1{\cdot}9+115/V \qquad ......(3)$$

(For D in metres, V in m/sec., $D = 35+1{\cdot}9V$, $T = 1{\cdot}9+35/V$.) The median accepted gap corresponds to a constant time of 1·9 sec. plus a constant distance of 115 ft (35 m).

Bias

Ashworth[5] showed that observed gap acceptance functions are subject to a bias related to the traffic flow, due to the different distributions of gaps presented to the turning driver at different flow rates. With the method of classification by speed used here, it was not possible to remove the flow bias and derive 'absolute' gap acceptance functions. However, the distributions of presented gaps in each speed range were obtained and compared with the distribution of all presented gaps. Proportions of gaps in half-second intervals from 1 to 10 seconds were compared using Wilcoxon's Signed Ranks Test[6]; no speed range had a gap distribution significantly different from the overall gap distribution, at the 10 per cent level. We conclude that the median accepted gaps in Table I are biased equivalently, and that Equation (2) represents a behaviour effect rather than a bias effect introduced by the method of analysis.

at each speed, the median accepted time gap was much larger than the 2 to 2·5 seconds typically required to turn and move out of the junction. Crossing is a relatively simple manoeuvre, and drivers appear to base their turning decisions largely on simple distance cues. Merging, on the other hand, requires more detailed information about the speed of the traffic stream to be entered, and more complex cues must be used.

Speed and risk

The empirical gap acceptance functions were tested in a conflict simulation model of a T-junction[2]. Conflicts occur when the drivers of turning vehicles make poor gap acceptance decisions, and vehicles approaching in the main road must slow down; the model assesses the severity of each such conflict by calculating the deceleration required to avoid a collision.

As a preliminary measure of the risk to which main road drivers are subjected due to errors by turning drivers, the proportion of main road vehicles involved in conflicts was calculated in 5 ft/sec. speed ranges. Normalised proportions are shown in Fig 3, for crossing, and Fig 4, for merging. In the case of crossing, the 'risk' varies approximately linearly with speed; as approach speed increases, the median accepted time gap decreases according to Equation (3), while the crossing time remains constant, so conflict involvement increases. For merging, the 'risk' increases faster than linearly; with increasing approach speed, the median accepted time gap decreases, but the time required to complete the merge and accelerate to the speed of the main road vehicle increases. In addition, the merging driver is likely to prefer to travel more slowly than main road vehicles with speeds above the mean traffic speed, increasing the chances of a conflict with such vehicles.

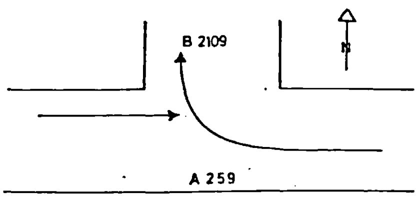
Table I. Median accepted gaps for crossing; V is the mid-point of a 5 mile/h speed range

Approach speed		Median accepted gap		
V (ft/sec.)	V (m/sec.)	T (sec.)	D (ft)	D (m)
40·3	12·3	5·07	204	62·3
47·7	14·5	4·09	195	59·4
55·0	16·8	3·92	216	65·7
62·3	19·0	3·73	233	70·9
69·7	21·2	3·66	255	77·7

Comparison of crossing and merging

The regression equations for merging (1) and crossing (2) are plotted in Fig 2. The slopes are significantly different, at the 5 per cent level. For crossing, the approach speed is not critical, provided that the accepted gap is sufficient for the turning vehicle to clear the lane of oncoming traffic;

Fig 1. The simple crossing movement (schematic).



Effects of speed distribution

These results indicate that the rate of conflict involvement may be related to the distribution of speeds in the major road.

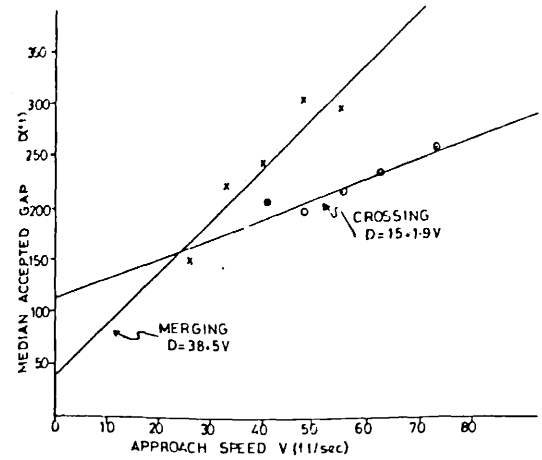


Fig 2 (left). Median accepted gaps as functions of approach speed V for crossing (Denton data) and merging (Nutbourne data[1]).

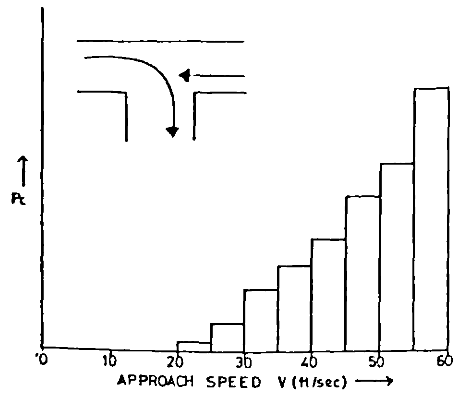


Fig 3 (right). Normalised proportion P_c of major road vehicles involved in conflicts with crossing vehicles as a function of approach speed V.

This hypothesis was tested in the simulation model, with major road speeds normally distributed with the same mean but different standard deviations. It is certainly true for merging, when, for fixed flow conditions, the conflict rate increases with increasing speed standard deviation (Fig 5). This is a direct consequence of the non-linear relationship between proportional conflict involvement and speed seen in Fig 4; as the standard deviation increases, the numbers of faster and slower vehicles increase, but the increased conflict involvement of the fast vehicles exceeds the decreased involvement of the slow vehicles, and so the overall conflict rate rises.

For crossing, where proportional conflict involvement varies more or less linearly with speed, overall conflict rate does not change with changing speed standard deviation. However, the average severity of the conflicts increases with increased dispersion of speeds, since fast vehicles must decelerate more quickly to avoid collision than slow vehicles. Spicer[7] has found that the occurrence of serious conflicts, defined subjectively as sudden rapid braking or lane-changing to avoid collision and counted by observers beside the road, is closely related to the occurrence of accidents at junctions, and that serious conflicts are about 2 000 times more frequent than accidents. If there is a correspondence between conflicts defined objectively in the simulation model and Spicer's subjective conflicts, then the increased conflict severity for the crossing manoeuvre at high standard deviations of speed implies an increase in accident risk under these conditions.

Discussion

As was the case for the merging movement[1], the crossing behaviour of drivers may be described by a modified time hypothesis, in which the median accepted gap may be considered as a constant time gap plus a constant distance gap. It is not clear whether this represents a strategy of the turning driver; at least part of the observed variation in gap acceptance can be explained in terms of perceptual errors of judgment associated with vehicles having speeds different from the mean speed (see, for example, Brain[8]).

Equation (2) is quite different from the result of Gibbs[9] for crossing in a test-track environment. Gibbs derived an expression $D = 5{\cdot}4V$ for the critical distance gap D (in feet) in terms of the approach speed V (in ft/sec.), corresponding to a constant time of 5·4 sec., with no distance component. Possible reasons for the differences are that Gibbs' experiment was performed on a test-track and involved only four subjects, while we observed 468 accepted and rejected gaps, involving 221 separate turns, in an actual road situation. Further discussion is given in Reference 1 and in Bottom[10], section 2.1.3.

Results from the conflict model indicate that accident risk may increase as the dispersion of main road speeds increases, and that the faster vehicles may be more at risk than the slower ones. The increase in risk is a consequence of poor decisions by turning drivers when they must judge gaps in front of fast vehicles. Faster vehicles are involved in more severe conflicts than slower vehicles; this is in agreement with general accident statistics (see, for example, Tables I and II of Reference 11) and empirical studies[12], where the severity of accidents increases with increasing speed (although these data are not confined to junctions). Munden[13] derived a U-shaped relationship between accident rate and relative speed, which was not observed in our results for vehicles in the major road; however, if turning vehicles are included, then there will be a higher proportion of slow vehicles involved in conflicts, and Figs 3 and 4 may become more U-shaped. Spicer[7] found no evidence that vehicles travelling at high speeds were more involved in (subjective) conflicts than those travelling at about the average speed, and he notes that 'the effect of vehicle flow and speed patterns on the conflict and accident rate appears to be complex'.

One possible reason for any differences between our results and those of others is that gap acceptance alone is not a sufficient measure of risk. In the simulation model, all vehicles have the same acceleration properties. However, Bottom[10] found that drivers who accepted short gaps accelerated more quickly than those who accepted long gaps. In the model, a higher rate of acceleration for turning vehicles which accept short gaps would lower the risk to the faster main road vehicles. We have conducted an experiment to examine acceleration and gap acceptance behaviour of merging drivers at a T-junction in more detail; results will be available shortly.

Conclusions

(1) The turning behaviour of drivers in a simple crossing situation may be explained by a modified time hypothesis.

(2) At the particular intersection we observed, the median accepted gap D in feet was related to the approach speed of major road vehicles V in ft/sec. by

$$D = 115 + 1{\cdot}9V$$

(With D in metres and V in m/sec., $D = 35 + 1{\cdot}9V$.) The median accepted gap consists of a constant time of 1·9 sec. plus a constant distance of 115 ft (35 m).

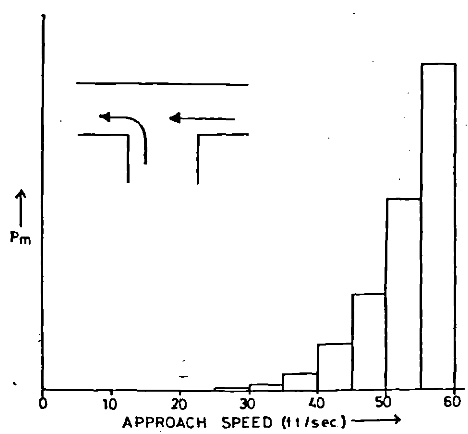


Fig 4 (left). Normalised proportion P_m of major road vehicles involved in conflicts with merging vehicles as a function of approach speed V.

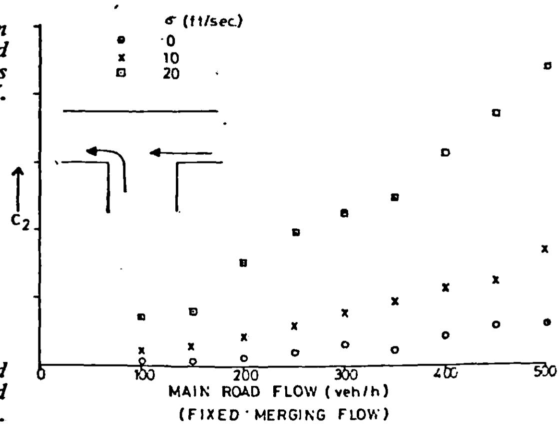


Fig 5 (right). Effect of speed standard deviation on normalised conflict rate for merging conflicts.

(3) Results from a conflict simulation model indicate that fast drivers in the major road are more at risk than slow drivers, due to the poor decisions made by turning drivers when they judge gaps in front of the fast vehicles.

(4) As the dispersion of speeds in the major road increases, the overall risk increases.

(5) The acceleration behaviour of turning drivers may modify the simulation results by lowering the risk associated with fast major road vehicles. Gap acceptance by itself may not be a sufficient indicator of risk.

ACKNOWLEDGMENTS

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REFERENCES

[1]COOPER, D. F., W. SMITH and V. BROADIE. Traffic studies at T-junctions: 1. The effect of approach speed on merging gap acceptance. *Traff. Engng Control*, **17** (6), June 1976, 256–257

[2]COOPER, D. F. and N. FERGUSON. Traffic studies at T-junctions: 2. A conflict simulation model. *Traff. Engng Control*, **17** (7), July 1976, 306–309.

[3]COOPER, D. F. Traffic studies at T-junctions: 3. Obervations of gap acceptance using television. Paper presented at the Second European Congress on Operations Research, EURO II, Stockholm, Sweden, Nov. 29–Dec. 1, 1976.

[4]FINNEY, D. J. *Probit Analysis*. (Third Edition). Cambridge University Press, 1971.

[5]ASHWORTH, R. A note on the selection of gap acceptance criteria for traffic simulation studies. *Trans. Res.*, **2**, 1968, 171–175.

[6]LANGLEY, R. *Practical Statistics*. Pan, London, 1970.

[7]SPICER, B. R. A study of traffic conflicts at six intersections. TRRL Report LR551, Transport and Road Research Laboratory, Crowthorne, 1973.

[8]BRAIN, R. Vehicle speed estimation at a right-angled 'at-grade' junction. *Surveyor*, **121**, 1962, 789–791.

[9]GIBBS, W. L. Driver gap acceptance at intersections. *J. App. Psych.*, **52**, 1968, 200–204.

[10]BOTTOM, C. G. Investigations into the variability of driver gap acceptance behaviour. PhD Thesis, University of Sheffield.

[11]JOHNSON, H. D. and F. GARWOOD. Notes on accident statistics. RRL Report LR394, Road Research Laboratory, Crowthorne, 1971.

[12]JOKSCH, H. C. An empirical relation between fatal accident involvement per accident involvement and speed. *Acc. Anal. and Prevention*, **7** (2), June 1975, 129–132.

[13]MUNDEN, J. M. The relation between a driver's speed and his accident rate. RRL Report LR88, Road Research Laboratory, Crowthorne, 1967.

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Risk Factors in Accidents

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INTRODUCTION

This paper describes a preliminary investigation into methods for obtaining a parameter for assessing the risk of road traffic accidents in a given non-urban situation. The work was undertaken by the Mathematics Department Operational Research Group, Royal Holloway College, on behalf of the Advisory Panel Sub-Committee on Evaluation Techniques of the Home Office Police Scientific Development Branch Project 2000. This paper is concerned with the development of the risk parameter. Details of the literature survey conducted at the Transport and Road Research Laboratory will be given elsewhere.

Terms of reference

The study was to develop, at least in preliminary form, a risk parameter based on results reported in the literature. The purposes of the parameter were to enable instantaneous accident risk to be assessed, both for identifying "trouble spots" as they developed and for evaluating police tactics, and to provide a predictive facility for use in scheduling traffic police resources. The parameter was to be based on factors which were already contained in the current experimental system in Sussex, or could be incorporated easily into that system. It was recognised that new empirical data were unlikely to be available within the six person-weeks to be devoted to the study, and that no definite improvement to the existing parameter might be obtained in this period. However, it was anticipated that the investigation would suggest definite areas of research likely to lead to improvements over the existing methods.

Structure

Part 1 of this report considers the general form of a risk parameter. The results of a literature survey are summarised, and factors which have important effects on the risk of accidents are identified. Functional relationships between these factors and accident risk are derived. A method for combining the factors into a risk parameter is presented, and methods for obtaining quantitative estimates of unknown or ill-defined factors are suggested. The concept of accident severity and the use of the risk parameter as a predictive tool are discussed.

In Part 2, the implementation of the risk parameter in Sussex is considered. Sample values of risk based on traffic data from one recording site on the A27 are calculated, and the problems of seasonal variations in traffic characteristics peculiar to Sussex are noted. A summary is presented in Part 3.

1. DEVELOPMENT OF A RISK FACTOR FROM THE LITERATURE

The search of the literature identified many factors which may influence accident risk. They can be classified into the following six groups:

(1) Road Factors

(2) Environmental Factors

(3) Driver Factors

(4) Vehicle Factors

(5) Traffic Parameters

(6) Temporary Changes.

(Pedestrians are not explicitly considered).

The qualitative effects of each group of factors are crudely summarised below. The statements made in the remainder of this section should be regarded as aide-memoires on the major identified effects. Nearly all of them require considerable qualification, and a much more detailed account is being prepared for publication elsewhere.

(1) Road Factors

The overall accident rate* in built-up areas is nearly three times that in non built-up areas. Accident rates decrease as the class of road improves, but accident severity is higher on the better roads (excluding motorways). In built-up areas, the majority of accidents occur at unctions, whereas in non built-up areas

*Accident rate refers to the number of accidents per vehicle km (or mile).

the majority of accidents occur away from junctions.

Accident rate decreases with increasing carriageway width and with increasing shoulder width. At bends, accident rate increases as the radius of curvature decreases. However, roads with higher average curvature have lower accident rates. An increase in gradient increases accident rate. Sight distance is also important: the accident rate increases as sight distance decreases.

The layout of a site has some effect: before-and-after studies have shown the beneficial effects of altering layout e.g. staggering of crossroads. At junctions, accident rate varies as the intersection angle varies. Road signs are a contributory factor in some accidents.

Street lighting conditions affect the night accident rate: in general, the better the lighting, the lower the accident rate.

Road surface characteristics significantly affect the accident rate, especially during wet weather. Harsh, rough surfaces are "safer" than smooth, polished surfaces.

(2) Environmental Factors

Weather has an important effect on the road accident rate. Inclement weather (rain, snow or fog) increases the accident rate on average by about 50 per cent. Darkness also affects accident rate: the dark accident rate is about one third higher than the daylight rate. Accident rates vary by the time of day, the day of the week and the month. Accident rates are highest at night (10 pm to 4 am) and in the early evening (4 pm to 6 pm). The overall accident rate is lowest on Sunday and highest on Friday and Saturday. There is a monthly variation in accident rate; the higher rates occur in the winter months (October to March).

(3) Driver Factors

The accident involvement rate is highest among young drivers (17–24 years). As drivers gain more experience, their accident rate is reduced. The accident rate is higher for males than it is for females. These summary results do not explicitly take account of the different exposures to risk of different classes of road user.

The risk of accident involvement is significantly higher for drivers with blood alcohol levels exceeding 80mg/100 ml. The effect of drugs on accident risk varies. Illness is usually only contributory to an accident rather than directly causal.

(4) Vehicle Factors

Two-wheeled vehicles have a much higher accident rate than four-wheeled vehicles. Vehicle colour and conspicuity may have important effects on risk. Vehicle defects play a part in about 8 per cent of all accidents, but the proportion of vehicles with defects in an accident goup is not significantly different from the proportion with defects in a control group.

(5) Traffic Parameters

Speed distribution has a significant effect on accident rate. Several investigations report a U-shaped relationship between accident rate and deviation from mean speed. Mean speed itself appears to have little effect on accident rate, but it has a large effect on accident severity.

The effect of flow on accident risk is not clear. All studies agree that the accident rate increases as the total flow increases, but the form of the relationship is not well defined. A linear relationship between accident risk and total flow may be assumed i.e. accident risk increases as flow increases, up to a limit when congestion occurs.

(6) Temporary Changes

Obstructions in the road create a hazard to the driver, and so increase accident risk. In particular, roadworks and parked vehicles present problems.

Certain police activities have a beneficial effect on accident risk.

The parameters to be used in the risk factor were selected from those given above. Since we are concerned with the variation of the risk factor over a limited area, we may assume that driver and vehicle populations will be constant at any given time. Hence the parameters to be used are:

(1) Road Parameter
(2) Environmental Parameter
(3) Traffic Parameter.

The derivation of the functional form adopted for each parameter is described below.

(1) Road Parameter

Denote this parameter by r_g. The road parameter depends on several variables (e.g. carriageway width, surface), but is constant for a given site. This parameter may be initially calculated from accident statistics for the site (see below). However, in order to allow for the effect of structural changes to the site, a road parameter which involves all the independent variables may need to be adopted in later work.

(2) Environmental Parameter

This parameter depends on weather, darkness and time. If the time factor is evaluated from monthly, daily and hourly variations in accident risk, the effect of darkness will be included in the time factor. Thus we need to derive two factors: a weather factor (say r_w) and a time factor (say r_t).

Weather Factor r_w: Define each weather condition by a value c. Codling (1974) has calculated the percentage increase in accident rates due to inclement weather. If we denote the rate for clear, dry weather by 1·0, the accident rate for a set of weather conditions relative to clear weather can be derived. For example, the increase in accident rate in wet weather is 53 per cent, so the accident rate for the wet weather condition is 1·53 (relative to clear dry weather).

Let w_c be the relative accident rate for condition c. Values of c and w_c, derived from Codling's results, are given in Table I in Appendix 1.

Suppose a change in accident rate reflects a change in risk. In this case the value of each element of the environmental parameter may be equated to the accident rate relative to an average value. Then the value of the weather factor r_w is given by w_c in Table I; e.g. in good weather conditions, $r_w = 1$.

Time Factor r_t: Accident rates vary by month, day and hour, so this factor has three elements.

(i) time of day i.e. hour. Denote this by t_d.
(ii) time of week i.e. day. Denote this by t_w.
(iii) time of year i.e. month. Denote this by t_y.

As before, these elements can be evaluated from the accident rates.

Let t_d be the accident rate for each hour of the day relative to an average hour, t_w that for each day of the week relative to an average day, and t_y that for each month relative to an average month. Then in an average hour, $t_d = 1, t_w = 1$ and $t_y = 1$.

For simplicity we assume a multiplicative relationship between these three elements

$$r_t = t_d \times t_w \times t_y.$$

Then for an average hour, $r_t = 1$.

Values of these three elements, derived from national data, are given in Tables II—IV in Appendix 1. (Satterthwaite 1976 a,b,c).

(3) Traffic Parameter

This parameter has two elements; a speed factor (say r_v) and a flow factor (say r_f).

Speed Factor r_v: Several investigations have derived a U-shaped form of the relationship between accident rate and deviation from mean speed (e.g. Munden (1967), Lefeve (1955), Solomon (1964). The most useful study seems to be Solomon's since his data are given in more detail (not as histograms, as in the report by the Research Triangle Institute (1970) used in Project 2001 (1976).

A quadratic function can be fitted to his data (see Appendix 2 and Fig. 1).

$$y(v) = a(v - \mu)^2 + b(v - \mu) + c \quad (1)$$

where $y = \log_{10}$ (accident involvement rate), $(v - \mu)$ = deviation from mean speed and a,b and c are constants. A good fit was obtained, and is reported in Appendix 2.

A normal distribution $N(\mu,\sigma^2)$ of speeds may be assumed (see, for example, Smeed (1958).

If the risk associated with a vehicle of speed v is y(v), then the speed factor $r_v^{(1)}$ is given by

$$r_v^{(1)} = \int y(v)\, N(\mu,\sigma^2)\, dv.$$

i.e. $r_v^{(1)} = a\sigma^2 + c$, where σ^2 is the variance of the speed distribution, and a and c are the constants in equation (1) (Appendix 2).

This speed factor depends only on the variance. The effect of mean speed on accident rate is not well understood. Solomon's data show no significant difference between the "risk" curves for 2-lane highways with different average speeds (using a simple Sign Test). There may be some relationship between the mean and standard deviation of a speed distribution, so the use of one may include some effect of the other.

Alternatively, we can use accident involvement directly by defining

$$r_v^{(2)} = \int e^{2.3y(v)} N(\mu,\sigma^2)\, dv$$

which (Appendix 2) yields

$$r_v^{(2)} = Ae^k$$

where A and k are functions of σ^2 but not of μ.

Mean speed does have an effect on accident severity. Solomon (1964) shows that the number of persons injured per accident involved vehicle increases with speed and that property damage per accident-involved vehicle also increases with speed. Joksch (1975) shows that fatal involvements/all involvements increases with speed. "Severity" cannot be measured objectively but most investigators agree that accident severity increases with increased speed .

A severity factor could be introduced into the risk parameter using any or all of the above measurements. This could be done by modifying the speed factor to take severity into account. If a normal distribution of speeds is assumed, a "severity factor" can be calculated using the mean and standard deviation of the speeds (see Appendix 2 for details).

Such a severity factor could be included in the speed parameter, but this implies a judgment of the relative costs or disbenefits of accidents of different classes. The Department of the Environment quotes costs of accidents by severity but these are admittedly somewhat subjective (Dawson 1971). Since we are mainly concerned with the risk of an accident occurring, such a term has not been included in this risk factor.

Flow Factor r_f: A simple linear relationship between accidents and total flow may be assumed; Project 2001 results on accident/flow relations in Sussex showed that the accident rate was proportional to flow. The flow factor to be used in our traffic parameter is chosen to be

$$r_f = \frac{F}{100}, \text{ where F is total flow.}$$

NB. F must be less than congestion level.

The risk factor R

Suppose a site exists where the risk is currently 1. As conditions at the site change, risk changes by a multiplicative factor R. This factor depends on the parameters above i.e. r_g, r_w, r_t, r_v and r_f. If these factors act independently on risk, then R has the form

$$R = r_g r_w r_t r_v r_f.$$

(The assumption of independence is discussed below).

At a particular site, we require the integral of R over a time period to be a measure of the accident risk in that period. Thus for a sufficiently long period (T, T′) in which there are A_g accidents at the site, we require

$$A_g = \int_T^{T'} R\, dt$$

$$= \int_T^{T'} r_g r_w r_t r_v r_f\, dt$$

$$= r_g \int_T^{T'} r_w r_t r_v r_f\, dt, \qquad (3)$$

since r_g is constant with respect to time.

Functional forms of r_w, r_t, r_v and r_f have been derived above, so the variation in these parameters over time at a site can be evaluated from empirical measurement. A_g is known from accident records. The integral can be evaluated over the time period (T, T′), and r_g, the risk due to the geometry of the site, may be obtained from (3) by

$$r_g = A_g \Big/ \int_T^{T'} r_w r_t r_v r_f\, dt.$$

For a site g, the term r_g provides a base level of risk to which the other terms can be applied multiplicatively. Thus the risk of accidents in the time period (T, T′) at a particular site with flow f_g, speed distribution v_g and weather conditions w_g is given by:

$$R = \int_T^{T'} r_g r_w(w_g) r_t(t) r_v(v_g) r_f(f_g)\, dt$$

Discussion

This risk factor is based on parameters derived from the literature. The number of independent reports relating to any one of these parameters is small, but in most cases the results are qualitatively similar, even if the numerical values differ. For example, the U-shaped relationship between risk and speed difference from the mean speed has been observed both in the UK and in the USA; we use a set of USA figures because they are presented in a more useful form and demonstrate the expected relationship.

The very limited data available in the literature must be used for want of any better, and we see no way to make a more accurate estimate of risk at present. To derive a risk factor specific to a particular geographical region, using only data obtained from investigations in that region, would take an appreciable time, during which changes might well take place in the region. Thus we are initially forced to assume that the results we use also hold in the region of interest, and we must refine the parameters as new results become available.

The time parameter is based on averaged national data, and so may not reflect seasonal changes peculiar to a particular region. It appears feasible to carry out analysis of local accident statistics to obtain a refined version of the time parameter specific to the region of interest.

Independence of the parameters which constitute the risk factor has been assumed in this report. This is an over-simplification, and is known to be not true in general e.g. weather and time interact (Codling, 1974) and speed and flow interact (Duncan, 1974). Our assumption is that the main effects of each of the parameters are much larger than any interaction effects. Since there is as yet little empirical evidence to quantify the interaction terms, this assumption is permissible for the initial investigation.

At present, the road parameter r_g cannot be obtained *a priori.* A method for evaluating it, which depends on accident statistics, has been proposed. For a given site, however, the accident numbers involved are in general small. This means that sites must be suitably grouped before the base level of risk can be calculated.

Analysis of the r_g factor to make explicit its dependence on the relevant variables (e.g. carriageway width, road surface) is desirable. This would be of value in distinguishing between sites, if a relatively large number of sites had to be grouped to obtain the initial estimate of r_g. It would also enable changes in accident risk due to structural changes at the site to be predicted. However, this is infeasible at present, since only a few empirical results on the form of the relationships between these variables and risk are known.

The risk parameter we have proposed does not depend on the mean speed of the traffic, but only on the dispersion of speeds. We have already noted that the inclusion of a severity parameter would introduce a mean speed term, and we have shown how such a parameter might be calculated. As we have stated, we have not included a severity parameter because of its subjective elements. This has the important implication that the present risk factor is applicable only to non-junction sites. There is some evidence that mean speed has a significant effect on accident risk at intersections (Cooper, Storr, and Wennell 1977), and also that

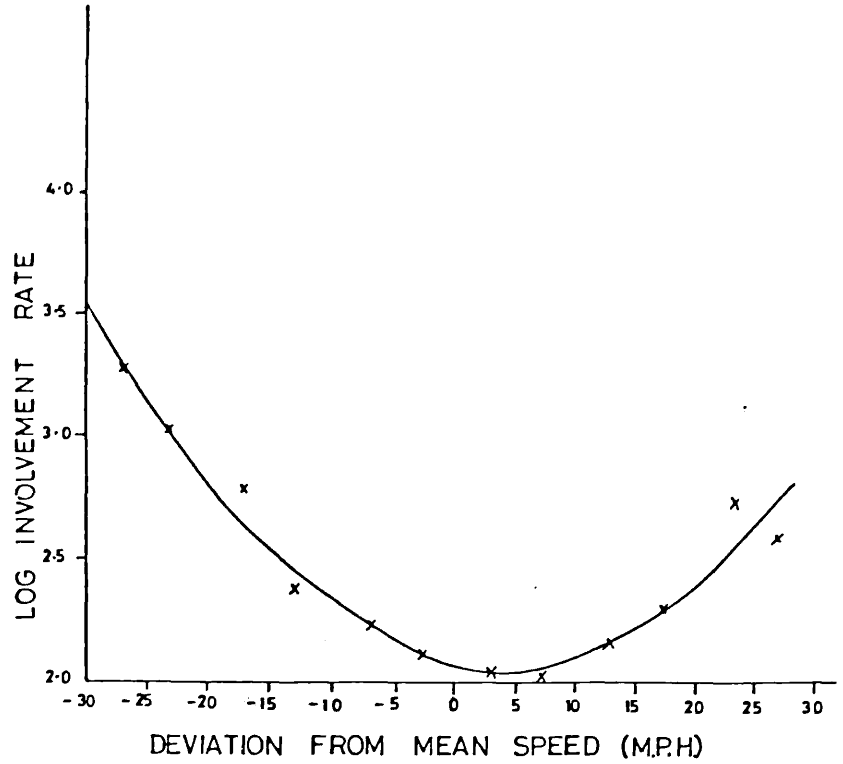

Figure 1. So'omon's data with the fitted quodratic curve.

the relationship between risk and vehicle flow may be different there.

The measurements required for the evaluation of the present risk factor are fairly straightforward. The method of calculating r_g has been discussed above. The parameters r_w and r_t are easily evaluated, from the Tables in Appendix 1. Calculation of the r_v and r_f terms requires data from continuous measurements of speed and flow at a site, since the speed distribution and the total flow in any time period must be known. If these measurements are readily available (as in Sussex), then the speed and flow parameters may be calculated. In general, at least one day's complete data on speed and flow at a site is required, if a risk factor is to be calculated for that site. This emphasises the need for portable data aquisition equipment.

This risk factor is suitable for use as a predictive tool. Values for speed distribution and flows at a given site may be predicted for the time period of interest if sufficient data from the past are available. Local weather conditions can be predicted with a certain amount of accuracy, especially if the forecast is for a short time ahead. If the variation in the time parameter is known for a particular area, it can be used for fairly accurate predictions of risk. The use of this risk factor for prediction purposes is discussed further in part 2, with particular reference to conditions in Sussex.

2. APPLICATIONS TO WEST SUSSEX

Chidham data

A sample set of data was obtained from the Chidham site, which is on the A27 west of Chichester. The data was for four hours on the 8 December 1976, and was listed in blocks. A block contains the data for 78 or 79 consecutively detected vehicles. The mean speed, flow and speed variance for each lane of traffic was calculated for each block of data. The blocks of data were then grouped into sections which corresponded to approximately 30 minutes measurement; the structure of the data prevented a more accurate time grouping.

The mean speed, flow and speed variance for each lane were calculated for these sections, and values of the flow and speed parameters in the risk factor were derived, using the formulae from Part I. Since the data are from one site only, r_g is constant. We can assume that the weather parameter is also constant, since a relatively short time period was considered. Then the risk factor R is given by

$$R = k r_v r_f ,$$

where k is a constant. The value of $\bar{R} = R/k$ was calculated for each 30 minute section of data. Typical results are shown in Figure 1.

Discussion on the use of this risk factor in Sussex

Values of the parameters r_v and r_f can be calculated from the continuous data on speeds and flow available for each on-line instrumented site in Sussex, and so the variation in the scaled risk factor $\bar{R}$ can be monitored for any *particular* site. To compare the risk between *different* sites, the road parameter r_g must be evaluated for each site. As has been discussed above, this is simple in principle, but requires historical accident data and suitable grouping of sites.

Sites may be grouped in various ways. If the groupings correspond to patrol zones, the difference in risk between different zones can be examined. However, such a grouping decreases the capability of the risk factor to distinguish between sites within the same zone, since r_g is now the same for all sites in that zone. An alternative method is to group the sites by similarity of physical characteristics (irrespective of their geographical position). If the measurements of speeds and flow at a site can be considered representative of an area around that site, then the risk factor calculated for that site can be considered to be characteristic of the area around the site, i.e. a risk factor for small areas within a zone can be calculated.

The use of the risk factor as a predictive tool has been noted. The previous discussion applies in Sussex, since considerable historical flow and speed data are available for each site. Once a large data base is available, long term predictions of risk can be made using standard time-series techniques. Until then, methods based on site accident statistics must be used.

The speed parameter considers speed distributions rather than individual vehicle's speeds, and contains no severity term (which would depend on mean speed). Consequently, changes in speed have a small effect on the risk factor, and it follows that police tactics which affect speed will not produce a very significant change in this risk factor, whichever of the two speed factors considered is used.

In contrast, the speed parameter was the dominant term in the previous risk factor used by Project 2001; it showed large variations when speed values varied, which meant that police tactics produced large effects on the risk measured in this way.

Our risk factor is concerned with non-junction sites. A specific term for mean speed would be included in a junction risk factor, and so the effect of police tactics on risk would be more noticeable at junctions.

A time parameter which reflects seasonal effects in Sussex needs to be developed, since one based on national data may not be accurate enough for a county with a high percentage of holiday traffic in summer.

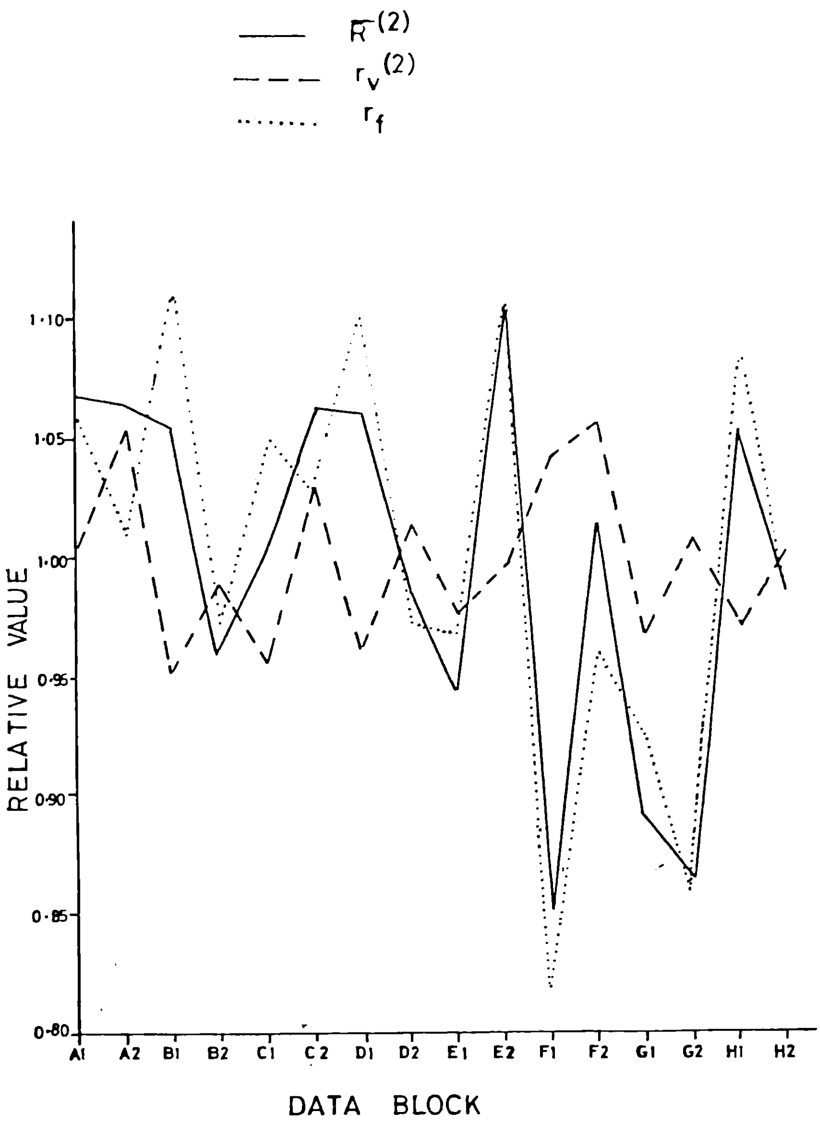

Figure 2 Value of $r_v^{(2)}$, r_f and $\bar{R}^{(2)}$ by data block (independently normalised to give average values of 1.0)

3. SUMMARY

An analysis of the literature has suggested the formulation of a mathematical expression for an accident risk factor at non-junction sites in a non-urban environment. It is a product of five parameters. These are a road parameter r_g, a weather parameter r_w, a time parameter r_t, a speed parameter r_v and a flow parameter r_f.

The value of r_g must be calculated for each site from accident data, as described in detail in the body of the report. Tables giving first estimates of the values of r_w and r_t are presented, but refined values await the availability of continuous measurements of flow at the specific sites of interest. We have shown that the value of r_v can be calculated from the locally measured speed distribution using

$$r_v^{(1)} = a\sigma^2 + c \quad \text{or} \quad r_v^{(2)} = Ae^k$$

and that of r_f is simply related to the locally measured flow by

$$r_f \quad I/100.$$

The risk factor for a particular site over the time period (T,T') is then given by

$$R = r_g \int_T^{T'} r_w r_t r_v r_f \, dt.$$

While it is in principle possible to make accurate measurements of the data needed to evaluate r_g, r_t, r_v and r_f, there is little agreement on what measurements are required to define a particular weather condition, and hence r_w. For example, while it is agreed that wet roads are more hazardous than dry, the depth of water which constitutes wetness at a particular site is not established. Automatic measurement of water depth and road temperature at a particular site is possible (but may be costly), and combination of such measurements with current flow and speed data with automatic data logging has not been developed.

The risk factor used in this paper has been evaluated for one site in Sussex, and it has been shown that the major part of the observed time variation is due to flow rather than speed. Because of the way in which we have formulated the speed dependence of the risk factor, this is likely to be a general result. A severity term has not been included, because it makes R a disbenefit factor rather than a risk factor. The methodology of incorporating such a term has been described, and so severity may be included if desired.

The risk factor suggested here can in principle be used predictively. Data collection requirements for this have been discussed in detail.

This investigation has suggested that development of a useful, predictive accident risk factor for specific sites is possible. No results have been obtained for non-urban junctions or for urban areas, but the methodology should be capable of extension to these areas. It appears that further development, which will require considerable effort, would be worthwhile in providing an additional tool to all those concerned with road safety. The possibility of this development has depended on the availability for the first time of automatic recording of flows and speeds at many sites. If the development of such automatic data collection techniques for the road traffic system is to be properly exploited, work of this nature on a risk factor or equivalent approaches must be pursued.

REFERENCES

ALMOND, J. (1963). Speed measurements at rural census points. *Traff. Eng. Control*, 5 (5), pp.290–294. (Cited in TRRL Report LR 598).

CODLING, P. J. (1974). Weather and road accidents. In *Climatic resources and economic activity*. Newton Abbot (David and Charles Holdings), pp.205—222.

COOPER, D. F., P. A. STORR and J. WENNELL (1977). Traffic studies at T-junctions: 4. The effect of speed on gap acceptance and conflict rate. *Traff. Eng. Control*, 18 (3), March 1977, 110—112.

DAWSON, R. F. F. (1971). Current costs of road accidents in Great Britain. TRRL Report LR396, Transport and Road Research Laboratory, Crowthorne.

DUNCAN, N. C. (1974). Rural speed/flow relations. TRRL Report LR651, Transport and Road Research Laboratory, Crowthorne.

JOKSCH, H. C. (1975). An empirical relation between fatal accident involvement per accident involvement and speed. *Acc. Anal. Prevention*, 7, 129—132.

LEFEVE, B. A. (1955). Relation of accidents to speed habits and other driver characteristics. *Highway Research Bulletin* No. 120, Highway Research Board, Washington, DC, 6—30.

MUNDEN, J. M. (1967). The relation between a driver's speed and his accident rate. RRL Report LR88, Road Research Laboratory, Crowthorne.

PROJECT 2001: Summary report of the 3-year research programme (Draft Issue 1) d.S.D.B., Home Office, London.

RESEARCH TRIANGLE INSTITUTE (1970). Speed and accidents. Vol. 1, Final Report, Research Triangle Park. (Cited in summary report on Project 2001.)

SATTERTHWAITE, S. P. (1976a). Monthly variations in road accident frequency. *Traff. Eng. Control*, 17 (1), January 1976, 18—20.

SATTERTHWAITE, S. P. (1976b). A note on the distribution of road accidents over the days of the week. Transport Studies Group, University College London (unpublished).

SATTERTHWAITE, S. P. (1976c). A note on the distribution of road accidents over the hours of the day. Transport Studies Group, University College London (unpublished).

SMEED, R. J. (1958). Theoretical studies and operational research on traffic and traffic congestion. *Bull. Inst. Internat. Statist.*, 36 (4), pp.357—375. (Cited in TRRL Report LR598.)

SOLOMON, D. (1964). Accidents on main rural highways related to speed, driver and vehicle. U.S. Department of Commerce, Bureau of Public Roads, Washington, DC.

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APPENDIX 1

Values for environmental parameter

Table I. Weather parameter (w_c)

Weather condition	Value of c	w_c
Wet	1	1·53
Snow/Ice	2	1·49
Fog	3	1·45
Clear and dry	4	1·00

Table II. Month parameter (t_y)

Month	y	t_y
January	1	1·19
February	2	1·08
March	3	1·02
April	4	0·93
May	5	0·96
June	6	0·91
July	7	0·87
August	8	0·79
September	9	0·87
October	10	1·01
November	11	1·23
December	12	1·27

Table III. Day parameter (t_w)

Day	w	t_w
Sunday	1	0·89
Monday	2	0·98
Tuesday	3	0·94
Wednesday	4	0·91
Thursday	5	0·96
Friday	6	1·15
Saturday	7	1·16

Table IV. Hour parameter (t_d)

Hour of day	d	t_d
Midnight–1 am	1	1·32
1–2 am	2	1·64
2–3 am	3	1·59
3–4 am	4	1·74
4–5 am	5	0·92
5–6 am	6	0·62
6–7 am	7	0·55
7–8 am	8	0·87
8–9 am	9	0·99
9–10 am	10	0·67
10–11 am	11	0·62
11–noon	12	0·77
noon–1 pm	13	1·04
1–2 pm	14	1·03
2–3 pm	15	0·87
3–4 pm	16	1·00
4–5 pm	17	1·23
5–6 pm	18	1·19
6–7 pm	19	1·01
7–8 pm	20	0·99
8–9 pm	21	0·85
9–10 pm	22	0·93
10–11 pm	23	1·46
11–Midnight	24	1·95

APPENDIX 2

Derivation of speed factor r_v.

A quadratic function was fitted to the daytime data given in Solomon's Fig. 7. The values assigned to each point are given below.

Difference from average speed (in mph)	Involvement rate (vehicles per 100 million vehicle miles)
− 37	19,000
− 33	42,000
− 27	1,800
− 23	1,100
− 17	620
− 13	230
− 7	170
− 3	130
+ 3	110
+ 7	100
+ 13	140
+ 17	190
+ 23	530
+ 27	380

The curve was plotted on a semi-logarithmic scale, so the quadratic function to be fitted is

$$y = \log_{10}z = a(v-\mu)^2 + b(v-\mu) + c \qquad (1)$$

where z is the involvement rate and $(v-\mu)$ is the difference from the average speed.

Substituting the values from the table in equation (1) gives 14 simultaneous equations for a, b and c. Solving these, and taking the mean value of the constants, gives $a = 1{\cdot}3 \times 10^{-3}$; $b \equiv 1{\cdot}1 \times 10^{-2}$; $c = 2{\cdot}07$.

This curve fits the data well, as can be seen in Figure 1. Hence a measure of the risk due to differences from the mean speed is given by

$$r(u) = au^2 + bu + c.$$

A speed distribution may be represented by the normal function

$$y(v) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(-\frac{(v-\mu)^2}{2\sigma^2}\right)$$

where v is the speed, μ is mean speed and σ is the standard deviation.

Let $u = v - \mu$, the difference from mean speed. Then

$$y(u) = \frac{1}{\sigma\sqrt{\pi}} \exp\left(-\frac{u^2}{2\sigma^2}\right).$$

The total risk due to the speed distribution is given by r(u).y(u) . . . (2). This can be calculated by integrating (2) over the entire range of possible speeds

$$\text{i.e. } r_v^{(1)} = \int_{-\infty}^{\infty} r(u)\, y(u)\, du.$$

This is a symmetric integral, so the contribution of odd functions is zero. The contribution of even functions is twice the value of the integral over half the range

$$\therefore \quad r_v^{(1)} = \frac{2}{\sigma\sqrt{2\pi}} \int_0^{\infty} (au^2 + c)\, e^{-u^2/2\sigma^2}\, du.$$

$$\text{Now} \int_0^{\infty} e^{-hmu^2} u^2 du = \tfrac{1}{4}\sqrt{\left(\frac{\pi}{h^3m^3}\right)}$$

$$\text{and} \int_0^{\infty} e^{-hmu^2} du = \tfrac{1}{2}\sqrt{\left(\frac{\pi}{hm}\right)}$$

Hence

$$r_v^{(1)} = \frac{2}{\sigma\sqrt{2\pi}} \left(\frac{a}{4}\sqrt{\pi 8\sigma^6} + \frac{c}{2}\sqrt{\pi 2\sigma^2}\right)$$

which simplifies to give

$$r_v^{(1)} = a\sigma^2 + c.$$

Different limits of integration may be chosen, in which case the integral must be evaluated using approximate or numerical techniques. However, if symmetric limits are used (e.g. $\pm 2\sigma$), then the expression for r_v is similar to that obtained above, with only slight changes in the constants.

The introduction of a severity factor changes the form of the speed parameter. If a vehicle is involved in an accident, then in general the severity of that accident is greater if the vehicle has a higher speed. Both from the physics of the situation and from empirical results, a quadratic relationship between severity and speed seems to be appropriate. (We note again that the choice of a scale of severity involves subjective judgement).

If a vehicle with speed v is involved in an accident, an estimate of the severity of the accident is $\phi(v)$. The chance of this vehicle being involved in an accident is r(u), and so the risk to this vehicle, taking severity into account, is r(u) $\phi(v)$. Then a risk factor r_s which includes severity may be calculated as

$$r_s = \int_{-\infty}^{\infty} r(u)\, y(u)\, \phi(v)\, du$$

This may be evaluated in a similar manner to r_v above, giving

$$r_s = A\sigma^4 + Q_1(\mu)\sigma^2 + Q_2(\mu)$$

where A is a constant and $Q_1(\mu)$ and $Q_2(\mu)$ are quadratic functions of the mean speed μ.

The speed factor $r_v^{(1)}$ and c derived earlier does not give a linear measure of risk, since the curve fitted to Solomon's data was based on semi-logarithmic axes. An alternative measure of risk is derived by taking the antilog of r(u) before performing the integration.

From (1), writing $u = v - \mu$ and converting to base e,

$$\log_e z = c_1u^2 + c_2u + c_3$$

where c_1, c_2 and c_3 are constants.

Then

$$r(u) \equiv z = \exp(c_1u^2 + c_2u + c_3)$$

$$r_v^{(2)} = \int_{-\infty}^{\infty} r(u)\, y(u)\, du$$

$$= \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{\infty} \exp(c_1u^2 + c_2u + c_3) \exp\left(\frac{-u^2}{2\sigma^2}\right) du$$

$$= \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{\infty} \exp(k_1u^2 + k_2u + k_3)\, du$$

where $k_1 < 0$, $k_2 < 0$ and $k_3 > 0$. Complete the square by setting

$$z = (-k_1)^{\frac{1}{2}}u - \frac{k_2}{2(-k_1)^{\frac{1}{2}}}$$

Then

$$r_v^{(2)} = c \int_{-\infty}^{\infty} \exp(-z^2 + k)\, dz$$

where c and k are constants w.r.t. the integration.

Hence

$$r_v^{(2)} = ce^k \int_{-\infty}^{\infty} (\exp^- z^2)\, dz.$$

But

$$\int_{-\infty}^{\infty} (\exp^- z^2) dz = 2\int_0^{\infty} (\exp^- z^2)\, dz = \sqrt{\pi}$$

$$\therefore r_v^{(2)} = A\, e^k$$

where A and k depend on σ^2.

The values of A and k are given by:

$$A = [1 - (5{\cdot}98 \times 10^{-3})\sigma^2]^{-\frac{1}{2}}$$

$$k = \frac{9{\cdot}52 - (5{\cdot}63 \times 10^{-2})\sigma^2}{2 - (1{\cdot}19 \times 10^{-2})\sigma^2}.$$

Models of gap acceptance by queues at intersections

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Introduction. The acceptance of gaps by queues of vehicles is important in assessing the capacity of junctions and freeway entrance ramps, particularly in peak periods. By queue acceptance we mean the acceptance of a large gap in a major road traffic stream by two or more waiting minor road vehicles, where the minor road queue is not exhausted.

Pearson and Ferreri[1] examined queue acceptance in terms of the percentage of gaps of a given size accepted by streams of vehicles entering a freeway. From their gap acceptance distributions, they derived a linear relationship between N, the number of vehicles entering, and T, the gap-length in seconds:

$$N = 0{\cdot}28T-1{\cdot}07 \qquad \text{......(1)}$$

They claim a high correlation coefficient for this relationship, but the method of derivation is not clear.

Bendtsen[2] studied queues of turning vehicles at the intersection of a freeway exit ramp with a primary road. His main concern was with measurement techniques and the intervals between successive vehicles.

Uber[3] considered the behaviour of queues of turning vehicles moving into large gaps at a T-junction controlled by a STOP sign. He derived an expression relating T and N based on the median start-up times of the first and subsequent vehicles making the turn and the 'median remainder rejected lag':

$$N = 0{\cdot}29T-0{\cdot}74 \qquad \text{......(2)}$$

This paper compares two methods for deriving queue acceptance relationships and illustrates their use for two different turning manoeuvres. The collection of empirical data at two non-urban T-junctions in southern England is described briefly next. A direct linear relationship and an explanatory model are then presented, and afterwards are compared and discussed in detail. Finally, we conclude that the explanatory model is the better one.

Data collection

As part of Project 2001 of the Home Office Police Scientific Development Branch (PSDB), we have been involved in studies of several non-urban T-junctions in southern England. Our observations have included the use of portable television equipment to film these junctions; the resulting video-tapes have been analysed to produce lists of events and their times of occurrence, from which details of gap acceptance and vehicle movements can be derived. The observational techniques are described more fully elsewhere[4].

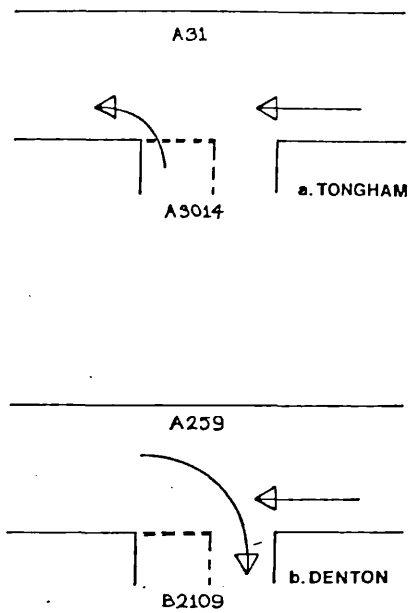


Fig 1. Outline of (a) the merging manoeuvre at Tongham; and (b) the simple crossing turn at Denton. (Note that traffic keeps to the left in the U.K.)

Simple turning movements were studied at two intersections. A merging manoeuvre was examined at the A31/A3014 junction near Tongham in Surrey (Fig 1a), while a simple crossing turn was investigated at the A259/B2109 junction near Denton in East Sussex (Fig 1b). Both intersections are T-junctions controlled by GIVE WAY signs (but this feature is irrelevant for the crossing manoeuvre at Denton).

A direct linear relationship

The simplest approach to queue acceptance would appear to be a direct examination of the length of the time gap T accepted by N vehicles from a queue. The results of previous workers[1,3] suggest that there is a linear relationship between T and N. The simple linear form, when fitted to the empirical data, gave:

$$T = 2{\cdot}8N+4{\cdot}9 \qquad \text{......(3)}$$

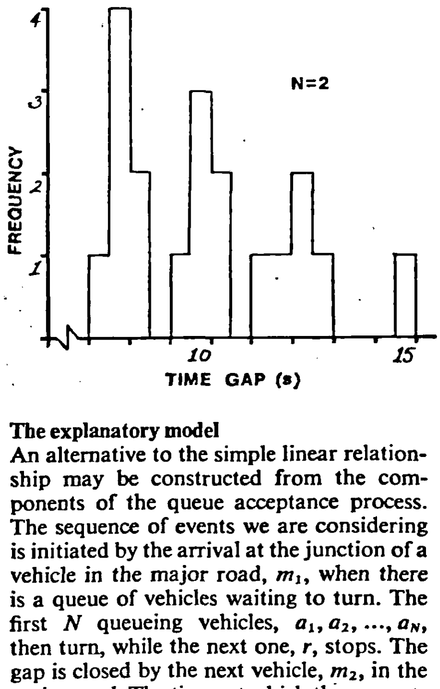
for the merging manoeuvre at Tongham ($r = 0{\cdot}76$, $r^2 = 0{\cdot}58$, $d.f. = 34$, $p < 0{\cdot}01$), and

$$T = 3{\cdot}8N-0{\cdot}1 \qquad \text{......(4)}$$

for crossing at Denton ($r = 0{\cdot}95$, $r^2 = 0{\cdot}90$, $d.f. = 14$, $p < 0{\cdot}01$). Note that we have treated N as an independent variable and T as a dependent variable; to regard T as independent is not appropriate for these data, which are sampled from continuous distributions of gap sizes for fixed, integer, values of N.

A simple examination of the correlation coefficients would imply that a linear relationship is reasonable. However, it must be noted that, like many distributions that occur in the study of traffic, the distribution of the lengths of gaps accepted by a given number of vehicles is markedly skew (see, for example, Fig 2). Thus the normality assumptions of any linear regression model are likely to be violated; indeed, we prefer not to use the term 'regression'. Because the direct linear relationship may still be suspect, we look for a different kind of model to describe these data.

Fig 2. The distribution of time gaps accepted by two turning vehicles from a queue (merging, Tongham).



The explanatory model

An alternative to the simple linear relationship may be constructed from the components of the queue acceptance process. The sequence of events we are considering is initiated by the arrival at the junction of a vehicle in the major road, m_1, when there is a queue of vehicles waiting to turn. The first N queueing vehicles, $a_1, a_2, \ldots, a_N$, then turn, while the next one, r, stops. The gap is closed by the next vehicle, m_2, in the major road. The times at which these events occur are $t(m_1), t(a_1), \ldots, t(a_N)$, $t(r)$ and $t(m_2)$.

The intervals between the events in this sequence can be classified in three distributions: the start-up time of the first turning vehicle, $t(a_1)-t(m_1)$; the move-up time of subsequent vehicles, $t(a_i)-t(a_{i-1})$, $i = 2, \ldots, N$, $t(r)-t(a_N)$; and the residual lag which is rejected, $t(m_2)-t(r)$. All these distributions are skew, and, in line with most simple gap acceptance measurements[5,6] and good statistical practice, we choose the median value for our calculations. We can now construct the time gap T accepted by the N vehicles as the median start up time S, plus the median move-up time M for each of the next N vehicles, plus the median residual lag R:

$$T = S + N.M + R \qquad ......(5)$$

We can use more data to form these distributions than are available from analysis of the queue acceptance process alone. For example, there is evidence that the gap acceptance behaviour of a turning vehicle does not depend on the presence of vehicles waiting behind it[7], and it is unlikely that the starting behaviour will be affected either. Thus the start-up times for *all* vehicles which accept gaps may be included in the distribution. Similarly, all rejected lags may be included in the residual lag distribution.

We have assumed above that the move-up times of queueing vehicles are independent of whether or not they turn immediately. Previous results of Pearson and Ferreri[1], Bendtsen[2], Greenshields (quoted by Bendtsen[2]) and Uber[3] are by no means consistent. Our own results also show some variation. Under these circumstances, our assumption does not seem unreasonable. It enables us to include in the move-up time distribution data derived from queues of two or more vehicles of which only the first one turns. Move-up times are considered again in more detail in the discussion below.

Empirical results are shown in Table I. From these we derive the relationships:

$$T = 3{\cdot}0N+3{\cdot}0 \qquad ......(6)$$

for the merging manoeuvre at Tongham, and:

$$T = 2{\cdot}4N+2{\cdot}9 \qquad ......(7)$$

for the crossing turn at Denton.

The explanatory model is more appropriate to the data and produces a more useful result. It enables the effect of changes in the individual components of queue acceptance on the overall relationship to be evaluated, and so provides a useful tool for traffic engineers. For example, S and R may be affected by structural changes to a junction or by improvements in visibility, while M may be affected by changes in the performance of the vehicle or its driver.

Flow effects. In many gap acceptance studies, the proportion of gaps of a given size that are accepted is of interest. The proportions are obtained from observations of drivers, each driver contributing a sequence of rejected gaps before he turns, and a single accepted gap when he makes his turn. Under conditions of high flow there are more small gaps presented to the turning driver than at low flows, and, in general, he will reject more of them before one that is long enough is presented to him and accepted. Thus the derived gap acceptance distribution depends on the distribution of presented gaps, which in turn depends on the traffic volume in the major road. Ashworth[8] has shown how to calculate the flow bias in the observed gap acceptance functions which results from this effect.

It is fortunate that neither of the two models presented in this paper leads to bias effects of this kind. The direct linear relationship is based on acceptances, which do not depend on the presented gap distribution, while the explanatory model uses the residual lag distribution, again independent of flow. However, there are some flow effects, not related directly to the presented gap distribution, which should be considered.

Wagner[9] examined the mean start-up times for vehicles accepting both lags and gaps in peak and off-peak periods. In peak periods, when the flow was presumably high, he observed significantly shorter start-up times in each case. Uber[3] investigated flow effects explicitly: he found no relationship between the start-up of the first turning vehicle and flow, but the move-up times of subsequent vehicles decreased with increasing major road flow. Thus both the S and M terms in Equation (5) may be flow dependent. This is not a bias effect in Ashworth's sense, arising in the methods employed for observation and analysis, but a behavioural effect.

There are two possible explanations for these observations: there may be a true change in behaviour in a single population; or the observations may be of different populations. Wagner's results, derived in peak and off-peak periods, are likely to reflect different driving populations. The peak-period population is composed largely of males travelling to and from work, while the off-peak population might contain more housewives on shopping and school trips, and more professional and commercial drivers[10]; these groups are known to differ in their driving characteristics [7,11]. It is harder to apply this argument to Uber's results, which are all derived from observations made at or near the morning peak period. The change in move-up times appears to be a true behavioural change, possibly caused by a desire to turn quickly rather than risk an extended wait for a longer acceptable gap. Why, then, was no similar effect observed in the start-up times of the first queueing vehicle to turn? Further empirical evidence is needed in this area.

Traffic volume in the major road will influence the results obtained by both models. However, with the explanatory model we can examine flow effects in greater detail, and explain more satisfactorily the phenomena we observe.

Data use. The two models we have proposed use different amounts of the large quantity of data available from intersection observations. The direct linear relationship makes use of the information about queues only: the number of turning vehicles in each queue and the size of the gaps they accept. Much of these data, at least in the observations we conducted, relate to small queues of two or three turning vehicles. The explanatory model, as we have noted, uses far more information about traffic behaviour, drawn from a wider data base. Each of the components of the model—the start-up time distribution, the move-up time distribution and the residual lag distribution—may contain data from manoeuvres other than queue acceptance. In its use of data, the explanatory model appears preferable, enabling reliable and more representative results to be obtained from shorter periods of observation.

The Pearson and Ferreri method[1] makes quite different use of the available data. Like the simple linear relationship, it examines queue acceptance directly, but it uses both accepted gaps *and* rejected gaps to derive gap acceptance relationships for queues of different lengths. As the method they use to derive the linear relationship between T and N (Equation (1)) is not specified in detail, the Pearson and Ferreri technique cannot be compared directly with the two methods outlined in this paper. In addition, their result is not corrected for the flow bias which, as we have noted above, is present in all empirical gap acceptance distributions, and so a comparison of the numerical values is not possible either.

Merging and crossing turns. The explanatory model enables the results from the merging queue acceptance at Tongham to be compared with the crossing acceptance at Denton. Table I indicates that S, M and R all have different values at the two sites. The differences between the start-up times, and between the residual lags, may be due entirely to the method of data collection. As we have noted previously[4], the measurement of lags depends critically on the point within the junction at which the arrival of a major road vehicle is recorded: for example, observations at A and B in

Table I. Medians of the distributions contributing to the queue acceptance process

Distribution	Tongham Median	Tongham Sample size	Denton Median	Denton Sample size
Start-up time for first in gap *(S)*	1·7	417	1·1	572
Move-up time for second in gap	2·9	236	2·3	268
Move-up time for third in gap	3·2	133	2·6	88
Move-up time for fourth and following vehicles in gap	2·9	156	2·9	71
Move-up time of all following vehicles in gap *(M)*	3·0	525	2·4	427
Residual lag *(R)*	1·3	432	1·8	546

Discussion

The linear form appeared to be the simplest method for estimating the queue acceptance relationship directly. Although it seemed to produce good results, a closer inspection of the data revealed that the normality assumptions required for a regression model were violated, and so any attempt to regard this as a linear regression model would be quite wrong. This illustrates the danger of using linear regression as a convenient tool (in terms of statistical arithmetic) without ensuring that the assumptions of this particular statistical model are satisfied. In addition, the results given by this method are merely *descriptive* of the queue acceptance relationship, and cannot be used for detailed analysis.

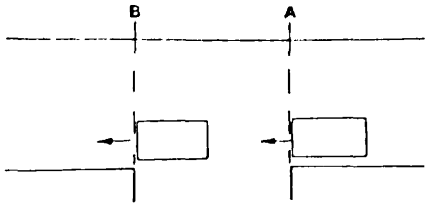


Fig 3. Recording positions for major road vehicles.

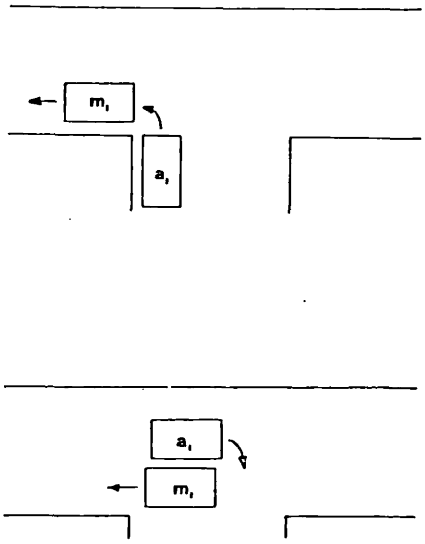
Fig 3, say 22 ft apart, would lead to differences in measurement of 0·5 s at speeds of 30 mile/h in the major road. Consistency between recordings of major road vehicle arrivals at any *specific* junction is relatively easy to achieve; here, errors of this kind are negligible. However, variations in experimental techniques and camera positions at *different* junctions mean that inconsistencies of this kind may arise when different junctions are compared. The situation is further complicated by the fact that the turning vehicle is at different locations within the junction for merging and crossing: even if the recording of major road vehicles is consistent, a merging vehicle must wait longer for the major-road vehicle to clear its path than a crossing vehicle (Fig 4), affecting the measured start-up times. Thus neither *S* nor *R* should be compared in Table I.

It is reasonable, however, to compare the move-up times for the two manoeuvres, as these do not depend on the position of the major road vehicle. The distributions of move-up times are not markedly skew (at least for Tongham, Fig 5), and it is possible to conduct the analysis in terms of the normal distribution. The relevant data are shown in Table II; the means are significantly different at the two sites ($z = 5 \cdot 65$; $p < 0 \cdot 001$).

There are a number of explanations for the difference in the move-up times for merging and crossing. First, the merging manoeuvre is a more difficult task than the crossing manoeuvre, since it depends to a greater extent on the speed of approaching major-road vehicles[12]. As a result, the driver's decision time may be longer for merging, so increasing the interval between consecutive turns. The geometry of the intersection provides a second explanation: a driver making the simple crossing turn we are considering (Fig 1b) is located in the centre of the major road, generally with a better view of approaching vehicles than a merging driver (Fig 1a). Thus the crossing driver may be able to make his decision to turn or not before he reaches the location at which the turn physically commences. On the other hand, the merging driver may have problems seeing the oncoming major-road traffic before he reaches the GIVE WAY line, particularly if there are minor-road vehicles beside him waiting to turn right, and he may be unable to anticipate.

A third, less obvious, explanation is related to the detailed movements of crossing vehicles as they turn. Often, the leading vehicle in a queue, a_1, has moved well forward into the junction before it is able to turn, and the second vehicle, a_2, is also within the junction. When a_1 is able to turn, he turns sharply; a_2 is able to start his turn almost at once, as he generally follows a different line through the junction, 'cutting the corner' as in Fig 6. Subsequent vehicles may also cut the corner. As a result, the move-up time of the second vehicle to turn, $t(a_2)-t(a_1)$, may be artificially short. Since the majority of move-up times are derived from the second vehicle in the queue (Table I), the median move-up time will tend to be shorter. This effect cannot occur in the case of merging.

Fig 4. The earliest possible start of a turn.



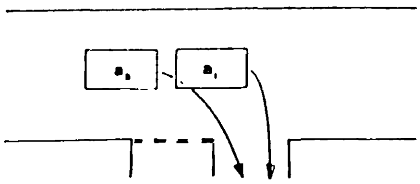


Fig 6. Possible crossing paths.

Conclusions

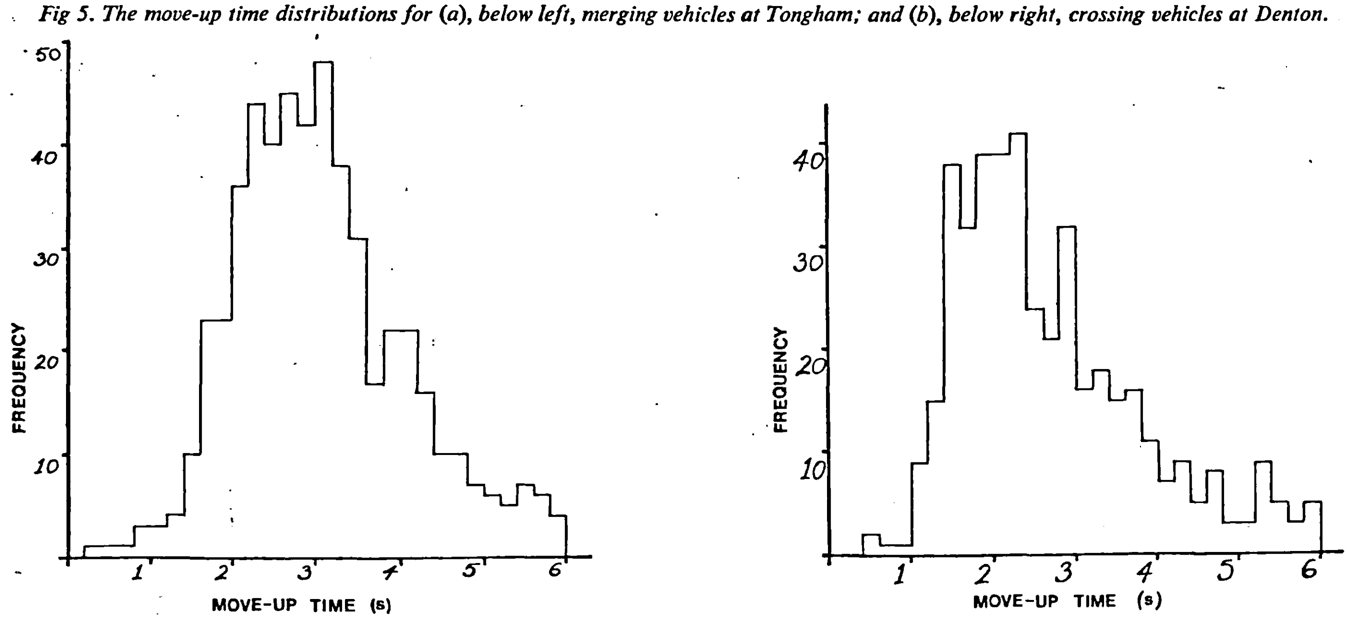
The explanatory model is better than the direct linear relationship for the analysis of queue acceptance: it is more appropriate to the data, as the distributions of accepted gaps are skew, violating any regression assumption; it is more useful for diagnosis and the prediction of the effects of changes in the intersection and its environment, since the individual components of the process are included explicitly; and it makes better use of the available data. The explanatory model may also be more useful than that of Pearson and Ferreri[1], since it does not contain bias effects due to the flow level of traffic in the major road. Traffic volume

Continued on page 185

Table II. Move-up time distributions

	Tongham (merging)	Denton (crossing)
Median *(M)*	3·0	2·4
Mean	3·1	2·7
Variance	1·06	1·28
Sample size	525	427

Fig 5. The move-up time distributions for (a), below left, merging vehicles at Tongham; and (b), below right, crossing vehicles at Denton.



MODELS OF GAP ACCEPTANCE BY QUEUES AT INTERSECTIONS

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appears to have other, behavioural effects apart from causing an observational bias; with the explanatory model these effects can be identified more clearly. The need for further observations on the effect of flow is noted.

Using the explanatory model, the time gap T seconds required by N vehicles turning from a queue was found to be:

$$T = 3{\cdot}0N+3{\cdot}0 \qquad \text{......(6)}$$

for the merging manoeuvre at Tongham, and

$$T = 2{\cdot}4N+2{\cdot}9 \qquad \text{......(7)}$$

for the crossing manoeuvre at Denton. Although it is unwise to compare these relationships numerically with regard to the start-up and residual lag components, a closer examination of the move-up component for the two cases provides interesting insights into the detailed functioning of the intersections, and indicates some of the reasons for the different results for the merging and crossing turns.

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REFERENCES

[1]Pearson, R. H. and M. G. Ferreri. Operational study—Schuylhill Expressway. *HRB Bulletin 291*, Highway Research Board, Washington, DC, 1961, 104–123.

[2]Bendtsen, P. H. Starting performance of queues at unsignalised T-junctions. *Traff. Engng Control*, 14 (5), September 1972, 214–217, 220.

[3]Uber, C. B. Start-up times and queue acceptance of large gaps at T-junctions. *Traff. Engng Control*, 19 (4), April 1978, 174–177.

[4]Cooper, D. F. Traffic studies at T-junctions: 3. Observations of gap acceptance using television. Paper presented at the Second European Congress on Operations Research, EURO II, Stockholm, Sweden, November 29–December 1, 1976.

[5]Ashworth, R. The analysis and interpretation of gap acceptance data. *Trans. Sci.*, 4 (3), August 1970, 270–280.

[6]Miller, A. J. Nine estimators of gap acceptance parameters. In: *Traffic Flow and Transportation* (Ed. G. F. Newell), pp 215–236. Elsevier, New York, 1972.

[7]Ebbesen, E. B. and M. Haney. Flirting with death: variables affecting risk taking at intersections. *J. App. Soc. Psych.*, 3 (4), 1973, 303–324.

[8]Ashworth, R. A note on the selection of gap acceptance criteria for traffic simulation studies. *Trans. Res.*, 2, 1968, 171–175.

[9]Wagner, F. A. Jr. An evaluation of fundamental driver decisions and reactions at an intersection. *Highway Research Record 118*, Highway Research Board, Washington, DC, 1966, 68–84.

[10]Storie, V. J. Male and female car drivers: differences observed in accidents. TRRL Report LR761, Transport and Road Research Laboratory, Crowthorne, 1977.

[11]James, J. E. and M. Goldman. Driving patterns for different sized vehicles. *Ergonomics*, 14, 1971, 363–369.

[12]Cooper, D. F., P. A. Storr and J. C. Wennell. Traffic studies at T-junctions: 4. The effect of speed on gap acceptance and conflict rate. *Traff. Engng Control*, 18 (3), March 1977, 110–112.

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A microprocessor-based system for traffic data collection

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Introduction. For some time, the Operational Research Group at Royal Holloway College has been studying driver behaviour at non-urban T-junctions. During this research, we have been involved in a number of data collection exercises using videotape recordings of traffic behaviour at the junction. We are particularly interested in deriving gap acceptance parameters from the data to use as input to a simulation model[1,2].

To obtain gap acceptance functions and to study their dependence on other traffic parameters, we need detailed records of the events which occur at the junction, and the times at which they occur. Events of interest are the arrival time of turning and non-turning vehicles at the junction, the commencement of a turn and the completion of a crossing manoeuvre. We also record the type of vehicle, the speed of main-road vehicles and, on occasions, various other descriptions on the vehicle and its occupants.

In our previous work[3, 4, 5], videotapes of the T-junctions were made and a digital clock image incorporated later. The tapes were analysed by running them in slow motion, stopping them at an event of interest and noting the event and clock time. The final output from the process was a list of events and the times at which they occurred to the nearest tenth of a second. This information was transferred to punched cards and analysed by computer.

Video techniques have the advantage of providing a complete record of events. However, the extraction of detail from the videotapes and the subsequent transfer to punched cards is extremely time-consuming and may be very tiring for the analyst. The time required for this phase of the analysis is of the order of 20 times the observation period.

This paper describes the design and construction of an alternative system to collect traffic data and transfer them to a central computer for processing. The use of this new system in recent observations of traffic behaviour at T-junctions is discussed and compared with the video techniques described above.

Requirements for the data collection system

We wished to develop a system of traffic data collection which would be capable of recording detailed data on driver behaviour over periods of several hours to cover, for example, morning and evening peak periods. For this purpose, the system must incorporate some of the properties associated with our previous video techniques. It must be portable, having its own power supplies, and be physically small enough to be transported in a car. It must provide an accurate time-base so that traffic parameters of reasonable accuracy can be obtained from the data. It must be able to record the times of vehicles passing points on the road and further information on the vehicle and its occupants. To do this we required inputs of two kinds: data input manually by observers, and data from automatic sensors (e.g. pressure tubes, induction loops).

In trying to eliminate the serious disadvantage of our video systems—the lengthy analysis phase—we needed to store the data internally and allow later direct transfer of these data to a mainframe computer. Punched paper tape was ruled out as being bulky, noisy, dependent on mechanical parts and easily damaged. Two possibilities remained: either random access memory (RAM) or magnetic tape storage. As our intention was to put the raw (unprocessed) data onto a mainframe computer for later analysis, and the quantity of data collected at any one time was expected to be relatively large, permanent storage on RAM would be prohibitively expensive.

Choice of equipment

To design and build our own hard-wired system was not possible because we had neither the resources nor the necessary experience. We decided that a microprocessor-based system would be more appropriate for our needs, and such a system could relatively easily be altered should our requirements change.

We considered some commercial systems and decided to base our data collection equipment on the Golden River MK4 system which could be used for a variety of traffic applications (for example, see Dalgleish and Tuthill[6]). This is a modular system based on a microprocessor. It has facilities for using RAM and Programmable Read-Only Memory (PROM), a real-time clock, a number of input and output (I/O) facilities and its own power supply. The relevant specifications for the modules used in our system are summarised in the Appendix.

Our choice of I/O port (the MK4/12 I/O Port B) works on an 8-bit byte switch closure input, and an 8-bit byte output. With suitable programming, the system can easily handle eight distinct on/off inputs for each such I/O port in the system.

Some applications of the Golden River MK4 systems have used a digital cartridge recorder. Such devices are costly and a cheaper alternative was sought. Computer Workshop market the SWTPC (Southwest Technical Products Corporation) 'AC-30' cassette interface. This is a mains-operated unit designed as an interface between a 300-baud UART (teletype) port and one or two audio cassette recorders. If a cassette recorder with a remote stop-start facility is used, the AC-30 is capable of stopping and starting the cassette motor according to signals from the connected computer (or microprocessor). Therefore the MK4 could be used to send a signal to start the tape-recorder motor, output data to the cassette and then stop the motor on a second signal. Data could be transferred in a continuous mode from cassette tape via the AC-30, to a UART port on a mainframe computer, provided the latter could handle the input rate of the data stream. This proved possible on the CDC 6600 we used, and is further discussed below.

A drawback of the AC-30 is that it is a mains-operated unit. However, its specifications showed that only low voltages and fairly low power were needed to run the device. Golden River agreed to modify the AC-30 from the kit provided to operate with a rechargeable battery supply.

Initial system

The initial system consisted of an MK4 microprocessor-based system including four 8-bit pushbutton input units and a UART I/O port; the modified AC-30 cassette interface; and a National Panasonic portable cassette recorder with a remote stop/start facility. It proved possible to use commercial audio cassette tapes with this configuration. A brief description of the individual modules is given in the Appendix, and a block diagram of this initial system is shown in Fig 1.

Inputs

Data are input to the system via four of the MK4/12 I/O Ports B. Each of these ports has two connectors in parallel. The connectors may be used to receive input from

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several types of source. We use two, handsets and automatic sensors. Each handset has eight pushbuttons as inputs to the MK4; each button has a corresponding light emitting diode (LED) indicator powered from the MK4. Any type of automatic sensor, e.g. pneumatic tubes or induction loops, may be used provided its output can be converted to a switch closure signal. We have been using coaxial cable and a suitable interface[7] to detect the passage of vehicles and input the information to the I/O port.

Outputs
Data are output from the system through the UART I/O port. For our purpose, this output is transferred via the AC-30 to the cassette recorder. The cassette motor is controlled by two output bits of a specified I/O port—the *control port*.

Software
The MK4 system uses the COSMAC microprocessor as its central processing unit (CPU). We have written and documented a program to collect, format and output the data (Golden River kindly allowed us the use of their development systems to write and test the programs). The development systems included an editor, assembler and various debugging aids.

The program was written in assembly language and is described in detail elsewhere[8]. The program begins by initialising counters, registers and constants and clearing memory, which takes about one millisecond. When this process is complete a signal is sent to a designated control handset via the appropriate I/O port as a message to the user that the system is ready to accept data. Subsequently, every 10 ms the program checks the status of the input lines on those I/O ports to which handsets may be connected. The state of the input lines is shown on the LED indicators of the corresponding handset, except for the two bits on the control handset which are reserved for cassette motor control. If, since the previous search, any input bit has changed from 0 to 1 (e.g. a button has just been pressed), the time, the handset number and the button number are recorded in ASCII code. This information, together with a parity symbol, is stored in RAM and forms one record.

Twenty such records form one block of data which is then output to tape. The output line on one bit (bit 7) of the control port is momentarily set high (= 1). This port is also connected to the cassette interface and the signal is used to start the cassette motor. A software delay of 1·25 s then occurs to allow the tape to reach its full speed. The program then outputs a START BLOCK message followed by the 20 records, and an END BLOCK message. The output line on bit 6 of the control port is then set high; this signal stops the cassette motor. It takes about eight seconds to output one block of 20 records.

While the output procedure is taking place the program continues to check the inputs and record events in a buffer area in RAM. In the present system the buffer can hold five blocks (100 records) of data. Before beginning to record a new block, the program checks that there is sufficient space in the buffer to record the data. If this is not the case a signal is output to the control handset to indicate that an overflow has occurred. The program must then be restarted and any data not already transferred to the cassette recorder are lost.

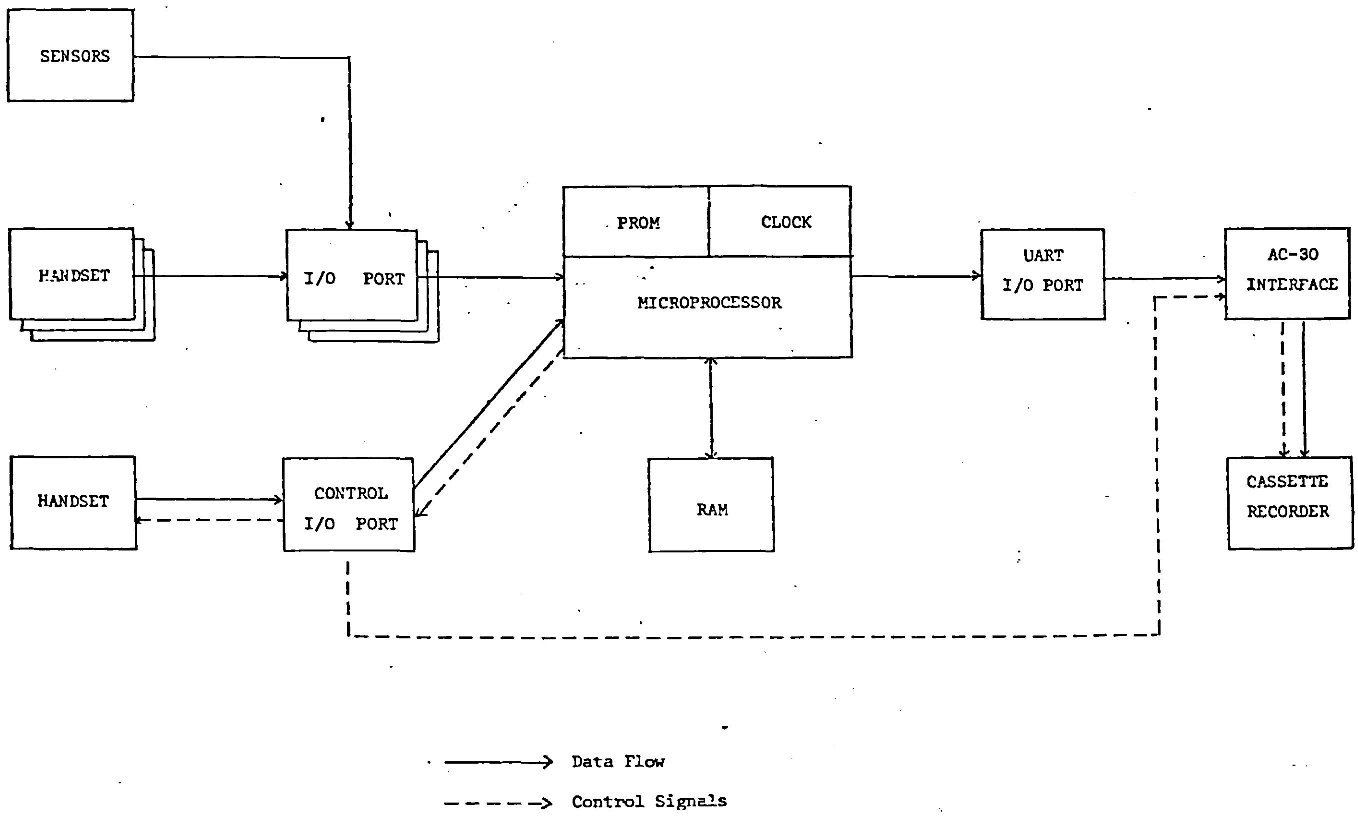
In order for an overflow to occur, the access rate must average more than 150 events per minute. For present purposes the access rate is low enough not to overflow the system. However, the system may be extended to cope with a higher access rate.

Data analysis
The raw data are played back from tape via the AC-30 through a UART port at 300 baud to files on a CDC-6600 computer. The accuracy of the transfer is checked using the parity symbol on each record and the data are reformatted for use with our existing analysis programs.

Discussion
The data collection system described here satisfies all of our initial requirements. Although the equipment is portable it is larger than was first envisaged, due mainly to the size of the AC-30 and the batteries used as its power supply. A second version is being designed which should eliminate the need for the AC-30.

Most automatic traffic detectors in present use are based on induction loops or pneumatic tubes. Induction loops are generally used for permanent installations, and are relatively expensive. Pneumatic tubes provide a cheaper alternative, and may be used with the MK4 system. We have used coaxial cables (with a suitable interface) in preference to pneumatic tubes because we found them much easier and quicker to install. Trials have shown this method of detecting vehicles to be successful, provided that the vehicles are travelling quickly enough (of the order of 10 mile/h

Fig 1. The data collection system.



and above) to 'hit' the cables with sufficient force to produce a signal.

An observer presses a button on a handset to indicate that a specific event has occurred. The present system records the time the button was pressed to an accuracy of 100th of a second. The accuracy of the data recorded is limited by the accuracy of the observers.

The existing system is very versatile. It accepts data from switch closure inputs, formats them and outputs them to cassette tape. Thus it could be used for many real-time data collection purposes—not necessarily confined to traffic studies—provided analysis programs were available or could be written on a mainframe computer. Indeed a mainframe computer is not necessary. The data could be fed back into the MK4 system and a program written so that the microprocessor could analyse them. The main limitations are on the data acquisition rate and the data transfer rate.

Although, at present, we use the system merely to collect data, the system could be adapted for many computing tasks. By connecting a teletype, VDU or similar device to the UART port of the MK4 system, one has a self-contained computer system. The cassette interface can accommodate two cassette recorders, allowing program and data storage, and the possibility of developing editing programs.

When using the existing system it is a simple matter to check visually that the cassette recorder is stopping and starting. LED indicators on the AC–30 show when it is receiving data. There is, however, no check that the data are reaching the cassette recorder, or that such data are meaningful. An external earphone can be connected to the cassette recorder to check that some sort of signal is reaching the recorder. To check that the data are meaningful it would be necessary to connect some form of digital display between the interface and the recorder or to the output socket on the recorder. This is not thought to be worthwhile for our present purposes as data can be quickly checked on return from the field, and the site can always be revisited. However, if the system is used for other tasks, such a check may be required.

The data collection system, as described in this paper, has been used at several T-junctions. As with almost all new systems, there have been some problems. These have arisen primarily from two sources: bad connections between the cassette interface and the cassette recorder; and the power supplies on the cassette interface and cassette recorder have sometimes become too low to operate effectively. The latter problem is caused by human error in leaving power switches on or in not keeping the equipment fully charged. The second version of this equipment (now being designed) will eliminate both these shortcomings, as the cassette interface and cassette recorder are being replaced by a cassette unit within the MK4 system, thus eliminating the need for connecting wires and separate power supplies.

Despite the 'teething troubles', we now have a considerable amount of data on the files of a mainframe computer. This new system of collecting traffic data requires, as was expected, much less time between observations and results than the previous video methods.

Conclusions

(1) The system has been used to collect traffic data from several T-junctions. Analysis of these data is comparatively easy, and much quicker than using video techniques.

(2) The accuracy of the data is limited by the accuracy of observers in a real-time situation.

(3) The equipment, although fairly bulky, is portable; a second version is being designed which will be physically smaller than the initial system.

(4) The system can be used for other data collection purposes or, with modifications, it can be used to perform many computing tasks.

(5) At present, data collected on site cannot be checked there. Checking the data on site is thought to be feasible, but not necessary for our present purposes.

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REFERENCES

[1] COOPER, D. F. and N. FERGUSON. Traffic studies at T-junctions: 2. A conflict simulation model. *Traff. Engng Control*, 17 (7), July 1976, 306–309.

[2] COOPER, D. F., P. A. STORR and J. WENNELL. Traffic studies at T-junctions: 4. The effect of speed on gap acceptance and conflict rate. *Traff. Engng Control*, 18 (3), March 1977, 110–112.

[3] COOPER, D. F., W. SMITH and V. BROADIE. Traffic studies at T-junctions: 1. The effect of approach speed on merging gap acceptance. *Traff. Engng Control*, 17 (6), June 1976, 256–257.

[4] COOPER, D. F. Traffic studies at T-junctions: 3. Observations of gap acceptance using television. Paper presented at the Second European Congress on Operations Research EURO II, Stockholm, Sweden, Nov. 29 to Dec. 1, 1976 (unpublished).

[5] WENNELL, J., P. A. STORR, D. F. COOPER and M. R. C. MCDOWELL. Conflict Modelling Studies: Part IV. The Tongham experiment. Internal Report to Police Scientific Development Branch, Home Office, January 1977.

[6] DALGLEISH, M. J. and R. D. TUTHILL. Development of a microprocessor-based vehicle classifier. *Traff. Engng Control*, 19 (3), March 1978, 116–119.

[7] MEYER, M. Details of electronics construction for road traffic detection system. Internal Report RHC/MOR/TRRL/6 to TRRL Road User Characteristics Division, November 1978.

[8] STORR, P. A. Documentation of the program RHCT used by the Royal Holloway College data collection system. Internal Report RHC/MOR/TRRL/5 to TRRL Road User Characteristics Division, December 1978.

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APPENDIX: Equipment

(a) Golden River equipment

(i) 1 MK4/1 Card Frame. This houses and connects all the various modules in an MK4 system up to 17 in. in total width.

(ii) 1 MK4/2 Case. This accommodates one MK4/1 Card Frame to provide a finished housing for an MK4 system in a protected environment. An environmental case is also available, but the lighter (in weight) and cheaper MK4/2 is sufficient for our purposes as we do not intend collecting data in the rain nor do we leave our equipment unattended.

(iii) 1 MK4/4 Power Supply. This consists of rechargeable nickel-cadmium cells. Provision is made for operation and charging from 240V AC or a 12V DC supply, e.g. a car battery.

(iv) 1 MK4/4 Microprocessor. The microprocessor provides all the logic to perform the instructions stored in program memory.

(v) 1 MK4/6 Random Access (1 024) Bytes. This is used as a temporary data storage space and provides a 'working area' for the program.

(vi) 1 MK4/7 Programmable Read-Only Memory. This is used as read-only memory for areas of program or data which must not be altered in the course of program execution.

(vii) 4 MK4/12 I/O Port B. These units each take eight switch-closure inputs. Outputs are used for LED indicators on the MK4/19 handsets and, in our configuration, two bits of one MK4/12 output are used for the cassette motor control. Each MK4/12 has two parallel I/O sockets.

(viii) 1 MK4/13 RS232C Interface. This is the UART port which allows for devices such as modems, teletypes, VDUs to exchange data with the processor and memory.

(ix) 1 MK4/18 Real-Time Clock. This provides a time base for maintaining an accurate software-based clock by generating an interrupt cycle every 1, 10, 100 or 1 000 ms.

(x) 4 MK4/19 Handheld Digital I/O Units. Each of these consists of eight pushbuttons as inputs to the MK4; eight LED indicators as outputs from the MK4; four toggle switches to control the state of four external flag lines (not used); and one toggle to switch off the LED indicators to conserve power.

(b) SWTPC AC–30 Cassette Interface

This is a mains-powered unit which has been modified to run off rechargeable batteries. Its purpose is to connect a computer (or microprocessor) to one or two audio cassette recorders for the purpose of program or data transfer. Signals from the computer can be used to stop and start the cassette recorder motors. Software delays must be included in the controlling program to allow the motors to attain full speed before data transfer.

(c) Cassette Recorder

This is a National Panasonic cassette recorder, Model RQ–212DAS; it has a remote jack socket which can be connected to a remote control to stop and start the cassette motor.