A MODEL OF TRAFFIC FLOW AT RAMP ENTRIES

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by
Alexander Skabardonis, Dip.C.E., M.Sc.

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ABSTRACT

FACULTY OF ENGINEERING AND APPLIED SCIENCE
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Alexander Skabardonis

This Thesis describes the research done on the investigation of the traffic behaviour at grade separated intersections. The ramp entry is a critical area at an interchange and affects the operational characteristics both on motorway and ramp. The study is concerned with the effects of traffic and geometric variables on the traffic situation.

Past studies have been reviewed and critically commented on. The simulation has been adopted as the method of approach due to its advantages over the empirical methods, which require extensive data collection, and to its flexibility over analytical methods to overcome simplifying assumptions. A microscopic model has been developed to reproduce traffic flow on a motorway section with a ramp entry, based on detailed analysis of the elements comprising the traffic situation and integration with the aid of a specially constructed computer program.

Particular emphasis has been given to the Calibration and Validation of the model, using a large data base in terms of video films. Comparisons with measurements taken at different sites and with information from other sources have been shown that the proposed model is based on realistic assumptions and accurately represents the traffic behaviour on a wide range of traffic conditions.

The effect of geometric design features has been examined by applying the model to different geometric configurations. Further areas of potential applications in terms of geometric design or traffic control schemes are also discussed.

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CHAPTER 1

INTRODUCTION

Motorways and other high capacity roads often form important sections in both urban and rural networks and provide for the fast and safe movement of people and goods. The overall capacity of such links is largely governed by the interchanges, and they must therefore be designed to operate efficiently.

Traffic is continuously increasing and existing interchanges originally designed forforeseeableflows have difficulty accommodating the increased demand, as they have now reached or are approaching capacity. On the other hand there is a need for efficient design of new sites, and often design freedom is restricted by environmental and economic factors. The above problems prompted the research in this study and the objectives may be defined as :

Determination of inter-relationships between highway and traffic factors at grade separated interchanges.

Use the relationships to develop specific guidelines for new design and remedial measures.

The work is being concentrated on the ramp entry as we feel that this is clearly the most critical situation. Of the approaches open to us, we decided to concentrate on simulation, as it enables a wide range of conditions to be fully evaluated. The empirical methods require extensive measurements, which are expensive and time-consuming because the phenomenon is complex and interaction of vehicles takes place not at a specific point, but over a whole area. The use of analytical methods relies on many simplifying assumptions and ignores certain variables which we feel are important.

The analysis involves the formulation of a detailed microscopic model to examine the behaviour and response of traffic on various conditions. The validity of the simulation depends on the underlying assumptions and a careful assessment is essential. In this study particular emphasis is given to the calibration and validation of the model using sufficient information from actual traffic situations, in order to produce a model which can be used as a research tool to predict and evaluate alternative designs and traffic management schemes.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction - Definitions

The entry from a slip-road on a high volume and high speed motorway is one of the most difficult manoeuvres required of drivers. The entering driver must approach the motorway, evaluate the traffic stream on the main motorway lanes and select a suitable gap into which he can manoeuvre his vehicle. He must then adjust the speed of his vehicle and make an entry while giving due consideration to other vehicles on the ramp and main road, both ahead and behind, that may influence his entry. Thus merging is a very complex process requiring numerous and rapid driver decisions. The following definitions are essential in the understanding and analysis of the situation.

Merging : The process by which vehicles in two separate streams moving in the same general direction,combine or unite to form a single stream.

Acceleration lane : An added width of pavement adjacent to the main roadway traffic lanes enabling vehicles entering the main roadway to adjust their speed to the speed of through traffic before merging.

Ramp or slip road : A connecting roadway between two intersecting or parallel roadways, one end of which joins in such a way as to produce a merging manoeuvre.

Lag : The time-headway between an entering vehicle and the immediately succeeding motorway Lane 1 vehicle.

Lead time : The time headway between an entering vehicle and either the immediately preceding Lane 1 vehicle or the preceding entering vehicle, whichever is the smaller.

Gap : A major-stream headway that is evaluated by a slip-road vehicle desiring to merge.

Greenshields (Ref. 12) defined the average minimum acceptable gap as the gap which is accepted by 50 per cent of the drivers. Raff (Ref. 14) has defined the critical lag as the value such that the number of rejected lags longer than it is equal to the number of accepted lags shorter than it.

Because the efficient operation of the motorway network largely depends on the capacity of interchanges, especially the ramp-entry, several studies have been made in order to evaluate the factors affecting the situation, to estimate the delays to the entering vehicles, the capacity of the junction, and to establish design criteria.

2.2 Analytical Approach to the Problem of Merging

There have been attempts by some researchers towards a merging theory in terms of a mathematical model, although, due to the complexity of the situation, and the number of variables entering into the problem, many simplifying assumptions had to be made.

Haight et al., (Ref. 4) emphasised the dynamic aspect of merging where the driver on the acceleration lane is able to control the speed of the traffic stream into which he wishes to merge by changing his own speed, thereby increasing or decreasing his headway and spacing relative to the main stream. In dynamic merging the driver has a wide range of policies he could adopt because he can vary his speed, whereas at conventional at-grade intersections the driver can only accept or reject a gap but not 'change' it.

In the formulation of the model it was assumed that the driver's primary concern is the distance between his vehicle and the one in front. This distance should be large enough so that if the vehicle in front makes an emergency stop, there is enough room for the second car to make a safe stop. Vehicles were assumed to have the same acceleration and deceleration capabilities and the maximum values were considered for the calculation of the safe merging space S , defined as :

$$S = \max\left[|V-u| \cdot T,\ So\right]$$

where :

V, u : speeds of mainstream and merging vehicles respectively

So : vehicle length

T : safe time gap which has a minimum value fixed by So , plus minimum manoeuvring distance.

In delay calculations, Poisson arrivals on both traffic streams have been assumed. The probability of no delay as a function of relative speed and the probability that a merge requires more than distance d were estimated, and numerical results are shown in Fig. 2.1 and 2.2.

Mine and Mimura (Ref. 10) produced an analytical model, assuming that a vehicle entering from a slip road has a known function of speed $v(t)$ and can run on the acceleration lane until it can merge, i.e. the length of acceleration lane assumed to be infinite. It was also assumed that the gap acceptance function depends on the mainstream available gap t and the speed of the merging vehicle $v(\tau)$, τ is the time elapsed since the arrival of the merging vehicle at the junction. Based on the above assumptions and on the fact that mainstream vehicles have a constant speed $V\left(v(\tau) \leqslant V\right)$ and their gaps have a distribution $\phi(t)$, the general distribution of delay has been derived. As an example the following special case was examined : the speed function was assumed to be:

$$v(t) = V\left(1-e^{-\alpha t}\right)$$

the gap acceptance function :

$$\begin{aligned} \alpha\left[t,\ v(\tau)\right] &= 1 \quad \text{for} \quad t \geqslant \frac{1}{\alpha}\, e^{-\alpha\tau} \\ &= 0 \qquad\quad\ \ t < \frac{1}{\alpha}\, e^{-\alpha\tau} \end{aligned}$$

and

$$\phi(t) = \lambda e^{-\lambda t}$$

i.e. negative exponential. The model predicted that under these conditions a vehicle will merge with probability one, given that the acceleration lane has infinite length.

Blumenfeld and Weiss (Ref. 2) presented a model of merging from an acceleration lane, with finite length. The authors admitted that is a complex situation and many simplifying assumptions are necessary. In their study they assumed that the headways on the main stream form a negative exponential distribution and the speed of the merging vehicle on the acceleration lane remains constant until the end of the lane, at which time it changes instantaneously to zero. The gap acceptance model adopted, consisted of two step functions, one for the moving motorist and one for the stopped motorist. Assuming that the acceleration lane has length L, traffic on the main road has constant speed V, and merging vehicle has speed $v (< V)$, moving on the acceleration lane, the time T_L to reach the end of it, without merge is :

$$T_L = L/v$$

then the delay can be defined for merging at time t:

$$D = \frac{V-v}{V} t \qquad t < T_L \tag{2.1}$$

$$D = t - \frac{vT_L}{V} \qquad t > T_L \tag{2.2}$$

The expected value of delay is calculated based on the expression :

$$\bar{D} = \left(\frac{V - v}{V} \right) E(Dm) + (1 - Pm) E(Ds)$$

where :

Pm : Probability of merging before the end of acceleration lane
E(Dm) : The expected delay, when moving on the acceleration lane
E(Ds) : The expected delay, when stopped.

The average delay $\bar{D}$ and the probability of reaching the end of the acceleration lane as a function of L are shown in Fig. 2.3, 2.4.

It can be seen that the probability to coming to a stop before merge is quite small, except of the smallest values of L , the average delay is relatively insensitive to acceleration lane length, and most of the merging takes place very close to the start of the acceleration lane. Because the model is concerned with the single driver behaviour, it was pointed out that a study of the acceleration lane usage requires a close analysis of the queuing properties. Finally in that model, there is an increase in delay with L at the lowest traffic flow rates. This is due to the fact that when a car is delayed by a given headway, that headway tends to be fairly large. That phenomenon is an artifact to the model in which no speed adjustment is permitted.

Drew et al., (Ref. 5, 6) in the U.S.A. have done considerable research in order to evaluate the characteristics of merging and to find relationships between the geometric and traffic parameters entering into the problem. In the theoretical level, a Delay model was produced assuming that a driver waits to merge until he finds an acceptable gap T , greater than his critical gap. The Erlang distribution was assumed for the main road headways and the critical gap assumed to be constant for all the drivers or having a distribution of gamma type. In the general case the average delay to a merging vehicle was given as :

$$\bar{D} = \frac{1}{q}\left(\frac{a}{a - q\bar{T}}\right)^{a} - \bar{T} - \frac{1}{q}$$

where:

q : outside lane volume

a : Erlang parameter

$\bar{T}$: mean critical gap

In Fig. 2.5, 2.6, the delays for various values of parameters are shown. An expression for the mean queue length was also produced. The effect of speeds on gap acceptance was shown by calculating the theoretical minimum gap for ideal merge, where ideal merge is defined as the case when the merging vehicle enters the freeway stream without causing a freeway vehicle to reduce its speed or change lanes.

Assuming that driver's normal acceleration of merging vehicles is represented by the equation :

$$\frac{du}{dx} = a - bu$$

where :

a : maximum acceleration
a/b : free speed in merging area

and driver's speed is u_r at the beginning of merge, then the speed-time relationship and the equation at time-space curve are :

$$u = \frac{a}{b}\left(1 - e^{-bt}\right) + u_r e^{-bt}$$

$$x = \frac{a}{b}t - \frac{a}{b^2}\left(1 - e^{-bt}\right) + \frac{u_r}{b}\left(1 - e^{-bt}\right)$$

The acceleration time relationship for the merging manoeuvre is :

$$\frac{du}{dt} = \left(a - bu_r\right)e^{-bt} \qquad (2.3)$$

The theoretical minimum gap for merging is considered to be composed of three time intervals : a) a safe time headway between the merging vehicle and the freeway vehicle ahead T_r b) the time lost accelerating during the merging manoeuvre T_L and c) a safe time headway between the second freeway vehicle and the merging vehicle T_f .

The safe headway is :

$$T_r = \frac{L}{u} + \tau$$

where :

L : length of vehicle in front
τ : reaction time
u : speed of freeway vehicle

Solving the equation (2.3) for time, we obtain the time necessary to accelerate from speed u_r to freeway speed u :

$$T_2 = - \frac{1}{b} \ln \left(\frac{a - bu}{a - bur} \right) \qquad (2.4)$$

Subtracting the travel time T_1 to travel the same distance covered during merging, only at a constant speed u , from (2.4) gives the time lost T_L . Finally the expression for the theoretically required minimum gap is :

$$T = \frac{Lf + Lr}{u} + 2r + \frac{u - ur}{bu} + \left[\frac{(a/b) - u}{bu} \right] \ln \left(\frac{a - bu}{a - bur} \right)$$

where Lf,lr arelengths of freeway and ramp vehicles.
From the above formula it is apparent that a H.G.V. would require a larger gap due to its length Lr and lower acceleration, a .

In order to determine the representative values of minimum safe gaps the parameters a and b should be estimated. Knox (Ref. 18) has given for Australian conditions :

$$a = 4.8 \text{ mph/sec}$$
$$a/b = 80 \text{ mph}$$

Gap Acceptance Models : The gap acceptance concept constitutes an important consideration in the interaction of the traffic streams and a number of analytical studies have been concerned with the determination and distribution of critical gaps for the merging motorists. The original concept of gap acceptance was not in connection with freeway merging but with relation to cross-roads or T-intersections where a driver arrived on a minor-road at a stop-sign and then merged into major road traffic. It was assumed that a constant critical gap T , existed, the same for all drivers and situations (Ref. 17).

Later development of gap acceptance theory led to more sophisticated models, assuming that T has not a fixed value, but varies among the drivers. The model of consistent driver (Ref. 16) has been proposed, where T is a random variable distributed over the motorists, but for any particular motorist the value of T was fixed. McNeil and Morgan (Ref. 8) proposed a theoretical method for the calculation of the parameters of gap-acceptance distribution, considering an intersection without acceleration lane and negative exponential distribution for the headways of main stream. The method had applied to data collected at an intersection in Australia and the mean and s.dev. were :

$$m = 2.45 \text{ sec.} \qquad \sigma = 1.95 \text{ sec.}$$

for non-stopped motorists and

$$m = 2.97 \text{ sec.} \qquad \sigma = 1.19 \text{ sec.}$$

for stopped motorists.

In a second approach it was suggested T to be distributed random variable, but no distinction was made for different drivers. According to this model the driver's behaviour was assumed to be perfectly inconsistent (Ref. 15). McNeil and Smith (Ref. 9) considered a cross-intersection without acceleration lane and discussed the effect of the different gap-acceptance models on delay estimation. It was shown that the delays assuming the 'consistent behaviour' model were generally greater than those for the 'inconsistency model'. In a similar study Blumenfeld and Weiss (Ref. 3) considered the case of an uncontrolled junction and concluded that the average delay is an accurate estimate assuming consistent behaviour, the probability of no delay and the capacity of the minor road will be slightly overestimated and the variance of delay seriously overestimated. The form of the distribution of acceptable gaps for a driver could be of exponential or normal, based on the analysis of Ashworth and Bottom's data (Ref. 1), collected at a priority intersection (Fig. 2.7).

Because none of the proposed models on driver-behaviour represents the true life situation, Smith (Ref. 7) has attempted the formulation of an 'inconsistent consistency' model which takes into account both aspects of driver behaviour. Firstly there is a distribution $f(\bar{g})$ of mean critical gaps, $\bar{g}$, among motorists. This is distribution I ;

shown in Fig. 2.8. Then for each motorist in the population, with mean critical gap $\bar{g}$ there is another distribution $h(g)$ whose mean is $\bar{g}$, which is the distribution II. It was believed that this model would represent accurately the driver behaviour but the mathematical treatment and the calibration of such a model are very difficult. Smith (Ref. 11) similarly as Drew attempted to calculate the minimum gap for merging, based on the dynamic aspect of the situation, and tried to explain the effect of relative speed on gap acceptance. Assuming an acceleration equation of the form :

$$\frac{dv}{dt} = \lambda \left(\frac{V - v}{x_v - x_f} \right)$$

where :

V, v : constant speeds on motorway and ramp respectively

x_v : absolute position of ramp vehicle

x_f : absolute position of freeway kerb vehicle

λ : constant

the minimum gap was calculated :

$$T = \left[(L + S_R) e^{(V - v)/\lambda m} + S_F \right] \Big/ V$$

where :

L : vehicle length

S_R : rearward safety distance

S_F : forward safety distance

λm : min. acceleration constant given by L, S_R, S_F .

The effects on the main stream flow due to an entry from a ramp, were examined by Jewell (Ref. 13). It was assumed that a vehicle makes a forced entry into the main stream causing the main road vehicles to slow down. A queueing model was assumed, and the number of vehicles delayed and the duration of disturbance were calculated for the case of Poisson traffic. The minimal headway to force in order to maximise the merging rate was estimated. The conditions assumed in this study are not applicable to motorway ramp entry and no practical results have been obtained.

2.3 Empirical Studies

Merging is a very complex phenomenon and the relationships between the traffic components so difficult to define that it is necessary to have extensive data from actual traffic situations, to use either in the validation of any theoretical approach, or in the establishment of empirical models, usually formed by regression analysis for design purposes. In the U.S. a nationwide Freeway ramp capacity study has been undertaken (Ref. 28) and the analysis of data was used to establish the procedures for the Capacity of ramp-terminals, presented in the Highway Capacity Manual 1965 (Ref. 22). A series of equations were developed, according to the geometry of freeway section, predicting the lane 1 flow upstream of the merge. The estimated flow plus the ramp flow constitutes the merge flow which should not exceed a critical value for the desired level of service.

A number of empirical studies have been made to evaluate the fundamental parameters that affect the traffic situation, especially gap acceptance, driver characteristics and effect of geometrics. Worrall (Ref. 25) has made observations at two freeway entrance ramps in Chicago, U.S.A. He emphasised the 'dynamic' aspect of the phenomenon, where the gap acceptance and rejection should be measured at the actual point of entry of the ramp-vehicle, instead of the 'static' measurement of acceptable gaps at a fixed point, usually the ramp nose. He distinguished between lead and lag times, defining the 'gap-structure' instead of 'gap-size', especially important in the case of multiple entries. In Fig. 2.9 cumulative distributions of dynamic lead and lag times are shown, together with those statically measured. The relative speed has a considerable influence on the distribution of lags, the proportion of vehicles accepting small lags decreasing rapidly as the relative speed of the mainstream to the ramp vehicle increases, as it is shown in Fig. 2.10. The equivalent effect on accepted lead times is much less pronounced.

In England Ackroyd and Madden (Ref. 19, 20, 21) have made a series of observations at three interchanges on the M1 motorway under free flow conditions. The slip roads were two flyunders and one flyover.

The results have shown that the merging situation is dynamic, i.e. the merging point is not fixed but varies between vehicles. Some of them make a short merge, whilst others make full use of the acceleration lane and merge later. Under the flow conditions studied the pattern of gap acceptance and rejection is very complicated. Accepted gaps as small as 1.5 seconds and lead times of only 0.5 seconds were observed. The distribution of accepted gaps measured at the start of the merge exhibits positive skewness and a clearly defined modal value, for light vehicles; this modal value was about 9 - 10 seconds, as shown in Fig. 2.11. For heavy commercial vehicles the modal values are not so evident but show that H.G.V. generally accepted larger gaps. In Fig. 2.11, the distribution of lead and lag times is also shown. The differences in distributions and modal values of lead and lag times indicate that drivers are better at assessing lead times than lag times. This is not unexpected since drivers are considerably more familiar with perceiving motion in front than at their rear. The proportion of vehicles accepting small lead times was reduced significantly as the relative speed of the entering vehicles to lane 1 vehicles increased, this contrasts to the study by Worrall. For lag-times the reverse effect is shown, i.e. as the speed of the entering vehicles to lane 1 vehicles decreased the proportion of drivers accepting small gaps decreased. The results have shown that the relative effective grade between ramp and motorway has a significant influence on merging operation, flyunders have better operating conditions, and the drivers use the ramp to reach the speed of the motorway whereas the acceleration lane is primarily used for speed adjustment and positioning of vehicle in order to merge. It was recognised that the low angle of convergence and sufficient length of acceleration lane are important geometric factors.

In U.S.A. an extensive research study was undertaken at Texas A & M University. Data were collected using aerial photographic technique at 32 entrance ramps, to meet several geometric and traffic requirements; the freeway alignment was straight and level, because freeway geometrics were not considered. As part of the study Drew (Ref. 23) investigated the characteristics of gap acceptance, in order to determine the merging capacity of entrance ramps. Using regression analysis, the following relationship between critical gap t and geometrics was obtained:

$$t = 5.547 + 0.828\theta - 1.043L + 0.045L^2 - 0.042\theta^2 - 0.874S$$

where:

θ = angle of convergence in degrees
L = length of acceleration lane
S = shape factor :
1 for taper type
0 for parallel type

From the equation it can be seen that increasing the angle increases the critical gap while increasing the length decreases the critical gap as shown in Fig. 2.12. An interesting aspect is the difference of taper type and parallel lane type of acceleration lane. According to the analysis, based on observations of 13 taper type and 16 parallel lane type acceleration lanes, a tapered entrance terminal will, on average, have a critical gap that is about 0.9 seconds smaller than that of a parallel lane type acceleration lane, with the same length and the same angle of convergence.

In another phase of this study Wattleworth (Ref. 24) investigated the effect of some entrance ramp geometrics on freeway merging. The results have shown that the angle of convergence (θ) has an effect on speed at ramp nose, as θ increases the speed decreases for equal acceleration lane, and when acceleration lane increases, speed in general increases. The general trend is that the mean merging speed tends to decrease as acceleration lane decreases and angle of convergence increases. Speed changes on acceleration lane are greater in 'poor' designs, and with favourable conditions most of the vehicle's acceleration takes place on the ramp. The same result was obtained by Ackroyd's study in British conditions. For relative speeds the general trend seems to be that ramps with better designs have lower relative merging speeds. Long acceleration lane and low angle of convergence lead to small mean value of the number of gaps that a driver has to wait before merging, i.e. the driver selects the first or second gap offered which means that he selects his gap in advance. The merging points are affected by the volume and speed of freeway traffic and geometrics of entrance ramp and acceleration lanes. The

Beta and Gamma distributions were fitted to the measured data on merging distances and good agreement was obtained. In order to delineate those factors which affect the use of acceleration lanes as described by merging distance, a regression analysis was carried out using several geometric and traffic characteristics as independent variables. The most significant were found to be :

Acceleration lane length : Drivers tend to take a longer distance to merge on ramps with longer acceleration lanes.

Ramp length : A longer rampis associated with a shorter distance to entry point. This appears to result from the fact that drivers have a greater distance to adjust to freeway conditions, before reaching the acceleration lane.

Relative effective grade : for which an increase in relative grade is associated with an increase in distance to entry point. As the percent grade increases from minus to plus (the ramp lower than the freeway), ramp drivers cannot begin selecting gap, and adjusting speed until they get quite near the nose. Hence, they tend to make more of the adjustment on the acceleration lane.

Angle of convergence : An increase causes a decrease in merging distance. This apparently reflects a desire of drivers to continue a smooth, natural path of entry without having to perform the reverse curve, that would be necessary when this angle is large and the driver went further down the acceleration lane to enter the freeway. The drivers tend to stay on the path on which they are 'aimed' by the ramp. Similar results were obtained by Pinnell (Ref. 30), who reported that the short direct entries predominate on ramp terminals with high angles of convergence.

Ramp offset at nose : A larger offset distance causes an increase in distance to entry point. This is certainly to be expected, since the ramp vehicle must move a greater transverse distance.

The product of the 3 minute flow : (expressed on a v.p.h. basis) times the average speed, for which an increase causes an increase in distance to entry points. Variables found not significant included the percentage of trucks, the merge volume, and the percentage of ramp vehicles in the merge volume. The influence of geometric characteristics on merging paths, was also acknowledged in Ackroyd's study but no explicit relations were established. The conclusions of the Texas A & M University study are not very definitive, but highlighted the importance of geometrics on merging operation and the effect of speed on gap acceptance, which have not been covered in the Highway Capacity Manual.

In Great Britain, Seddon (Ref. 27, 29) has done an empirical study on merging at urban grade-separated interchanges. Observations have been made at 12 sites under high flow conditions using time lapse photography. The prime objective of the study was to derive equations of inside lane flow and the practical merge capacity. Using regression analysis the equation for the inside lane flow of the motorway was obtained :

$$q_1 = 1488 - 522N + 0.33Q - 0.33q_r + 22.6g_r \quad (R^2 = 0.79)$$

where:

N = number of main road lanes 2 or 3
Q = total main road flow Q : 460 - 3020 veh/h
qr = slip road flow 120 - 2160 veh/h
g_r = slip road gradient - 4.9 to + 5.9 per cent

The study has also attempted to predict merge capacity and to relate the critical gap with geometric parameters, but no statistically acceptable relationship was produced.

In Australia, Swed (Ref. 7) has done a field study to determine the effect of relative speed on gap acceptance. The data were grouped into relative speed intervals and the distribution of gaps in each interval was lognormal. It was found that the parameters of the distribution vary linearly with relative speed. The effect of relative speed on gap acceptance was also pointed out in the study of Edwards and Vardon (Ref. 11). In Knox's study (Ref. 18) it was shown that when relative speed increases, the minimum acceptable gap increases rapidly, having minimum value at zero relative speed. A 6 seconds gap was necessary when a car was forced to stop before starting to merge, although the most usual gaps observed were in the range of 3 to 4 seconds.

In England, Burrow (Ref. 26) has calculated the capacity of merging areas working similarly with the Highway Capacity Manual. Diagrams show the maximum flows that can be accommodated at a given 'level of service'. For the development of the diagrams, the maximum permitted flows were considered according to the road type, two or three-lane motorway, one or two-lane slip road. Two further limitations were set : the downstream total merged flow is restricted to 1600 veh/h/lane, and the entry flow should not exceed the upstream main carriageway flow. Comparisons with recorded flows suggest that the standards are realistic and also the diagrams provide a simplified method of estimating capacity than the Highway Capacity Manual.

2.4 Simulation Models.

There have been attempts at simulating the merging behaviour, having as an aim to overcome the simplifying assumptions made in the straight analytical approach and on the other hand to obtain practical results. The use of simulation in studying freeway traffic has been shown by Gerlough (Ref. 39) in 1956 and since then a number of working simulation models have been produced.

Wohl (Ref. 31, 32) presented a model for the simulation of freeway merging area, under the following assumptions : the merging area is considered to be sufficiently remote from traffic generation sources and traffic controls as to allow the freeway and ramp vehicles to approach the intersection randomly. No lane-changing is permitted within the merging area, so only the inside lane of freeway and the ramp is considered. The importance of relative speed was recognised and two types of acceptable gaps were defined. Those for the moving or undelayed vehicles and those for high relative speeds when drivers were stopped or delayed. The gap acceptance information was given in empirical form with the percentages of drivers accepting given gap value. The shifted negative exponential distribution was used for the generation of headways. The results in the form of mean delay to ramp vehicles versus total merging volume were compared to field data. No good agreement was obtained, but principal aim of the study was to show the potential of simulation in the solution of traffic problems.

Perchonok and Levy (Ref. 33) constructed a simulation model based on extensive data collection at interchanges in Chicago, U.S.A. The negative exponential distribution was fitted to the headway data, with close agreement. The gap acceptance distribution was not precisely defined,but the data indicated the effect of relative speeds and different values for the probability of acceptance were used according to the ramp vehicle speed. The speeds of vehicles were found normally distributed and regression equations were used to predict the lane distribution on the freeway. In the simulation model the freeway and ramp lanes were each divided up into 100 blocks of 17 ft length (the average car length) and the method of time scanning was used. Every one second of real time, each block was scanned for the presence of a vehicle and if the block was occupied,the vehicle was allowed to manoeuvre. The results have shown that the length of acceleration lane has an effect on the traverse time for ramp vehicles, as increases, the mean traverse time decreases,whereas it has no influence on traverse time for freeway vehicles. The results were not compared to field studies and other forms of validation were not carried out.

Glickstein et al. (Ref. 34) developed a simulation model to test alternative forms of interchange configurations. They were based on Perchonok and Levy's model and data collected at locations in Detroit and Huston, U.S.A. Linear relationship was assumed for speed-flow relationship and for the lane volume to total freeway volume. The gap acceptance was found to vary according to merging vehicle's speed and the type of manoeuvre (merging or weaving), but no statistical distribution was fitted, and it was pointed out that it is impossible to determine how the driver's choice was affected by the other gaps being presented to him either in advance or following the gap accepted. The simulation technique was the same as Perchonok's and the geometric arrangement was a 17,000 ft freeway section with two on-ramps and two off-ramps, to examine the significance of spacing between ramp-terminals. It was shown that there was little operational difference between the separation of 1530 ft and 3060 ft between on-ramps and off-ramps.

Dawson and Michael (Ref. 35) used the simulation to determine the ramp practical capacity. Two geometric situations were examined : ramp with no acceleration lane and stop or yield sign, and ramp with acceleration lane and no-sign control. All the vehicles in the system were assumed to have the geometric and operating characteristics of passenger cars with the same constant acceleration and deceleration capabilities. The gap acceptance model was assumed for ramps with acceleration lanes to be of the form:

$$P(\text{accept gap } t) = \ln\left(\frac{t}{tmin}\right) * \frac{1}{\ln\left(\frac{tmax}{tmin}\right)}$$

where:

t = any gap length between the limits of tmin and tmax

tmin = minimum acceptable gap

tmax = minimum gap length for which probability of acceptance is one.

The shifted negative exponential distribution was adopted for the headway distribution on freeway traffic and the double exponential distribution for the ramp headways; speed was assumed to vary with volume for shoulder lane vehicles and a fixed speed of 30 mph was assigned to each generated ramp vehicle, on the assumption that geometry governs speed regardless of traffic conditions. The practical capacity of a freeway on-ramp was defined as the maximum number of vehicles that could enter the freeway during one hour with 85 per cent of the drivers being able to leave the ramp without being delayed more than 60 seconds. The results of the simulation are shown in Fig. 2.13 in terms of average delay, practical capacity and probability that delay exceeds 60 seconds. An expression for the practical capacity was obtained :

$$y = e^{(a + bx + cx^2)}$$

where:

y = practical capacity (veh/h)
x = shoulder lane volume (veh/h)

Sinha and Dawson (Ref. 36) developed a model to simulate traffic flow on a freeway system with five through lanes, four on-ramps and six off-ramps. A car-following model was used for the movement and interactions of vehicles in the system, based on the fact that the vehicles do not come into physical contact and they maintain a minimum safe distance between them. The gap-acceptance distribution was assumed log-normal, and the headway distributions were assumed shifted negative exponential and Hyperlang for the through lanes and ramps respectively. The results provided information about the whole system, and the validation procedure consisted of comparing the headway distributions at specific points with empirical data.

Szwed (Ref. 37) in Australia constructed a simulation model having as aim to establish design criteria for the merging area. The traffic flow was represented by a Markovian traffic model with two groups of states : the car-following group for vehicles within a platoon, having a gamma headway distribution and the non-following group for unrestrained vehicles, having a composite headway distribution. The gap acceptance model of consistent driver was adopted with the critical gaps log-normally distributed. A field study has been done and the parameters of the gap acceptance distribution were found to vary linearly with relative speed. The merging strategy was as follows : ramp vehicles arrived at the merging lane at speed v. The freeway speed was $V (V > v)$. The ramp driver then travelled along at speed v, examining gaps in the kerb stream until he found one greater than his critical gap. He then accelerated into speed V and moved laterally into the accepted gap. If he failed to find a suitable gap before reaching the end of the merging lane, he merged, irrespective of the gap size. This disruptive effect on the kerb-lane flow was not simulated, only the number of forced merges was recorded. Lane-changing was not permitted in the merging area and a first come first served queue discipline was assumed for merging drivers.

The results have shown that the length of acceleration lane and the speed of merging vehicle are important factors. The probability of forced merges remains small until 1500 veh/h merging volume, and the percentage of drivers accepting a lag decreases linearly with volume.

Salter and El Hanna (Ref. 38) simulated the operation of a ramp-entry at an interchange in England, in order to investigate the junction performance, with and without control, and to show the benefits of a ramp-metering system. The vehicle characteristics such as length, speed, acceleration and deceleration were generated from normal distributions and the critical gap varied with the speed of merging vehicle according to field observations. Car-following procedures were adopted for the vehicle movements and the 'gap acceptance' ramp metering system was selected. It was shown that the introduction of a ramp control decreases the delays, the percentage of restrained vehicles and increases the speeds.

2.5 Conclusions

The minimization of delays at ramp-entries and the increase in capacity are key factors for the efficient operation of the whole motorway network. On the other hand the study of the merging process is one of the most difficult traffic problems due to interactions between vehicles and the number of traffic and geometric parameters involved. The studies reviewed in this Chapter are among the most significant and representative investigations into the problem. It is apparent that the theoretical approach, using an analytical model is almost untractable. Many simplifying assumptions are used and the results are inconclusive and of little practical value for the designer; the value of theoretical approach is that it may contribute to a better understanding of the underlying mechanisms of some factors affecting the situation, e.g. Haight's study, gap acceptance modelling. The empirical studies using sufficient data developed predictive relationships for the design of ramp-terminals. The results of those studies have clearly shown that merging is a 'dynamic' process and it is affected by certain geometric and traffic factors as relative speed, slip road geometry, acceleration lane length and angle of convergence. The results are not very definitive but highlight the situation and give quantitative criteria for design purposes. Simulation offers distinctive advantages over the rigid analytical approach and its use could lead to problem treatment overcoming many simplifying assumptions. Although several attempts have been made, the most important reviewed in this Chapter, the simulation models produced, have many shortcomings. Basically there are simplifications and unrealistic assumptions on the merging strategy, and in the modelling of traffic in general. On the other hand, the lack of sufficient calibration has resulted in the models not being fully validated.

It can be therefore seen that there is still a need for further investigating of the operation of grade separated intersections. The work done in this study will, we believe answer at least part of that need.

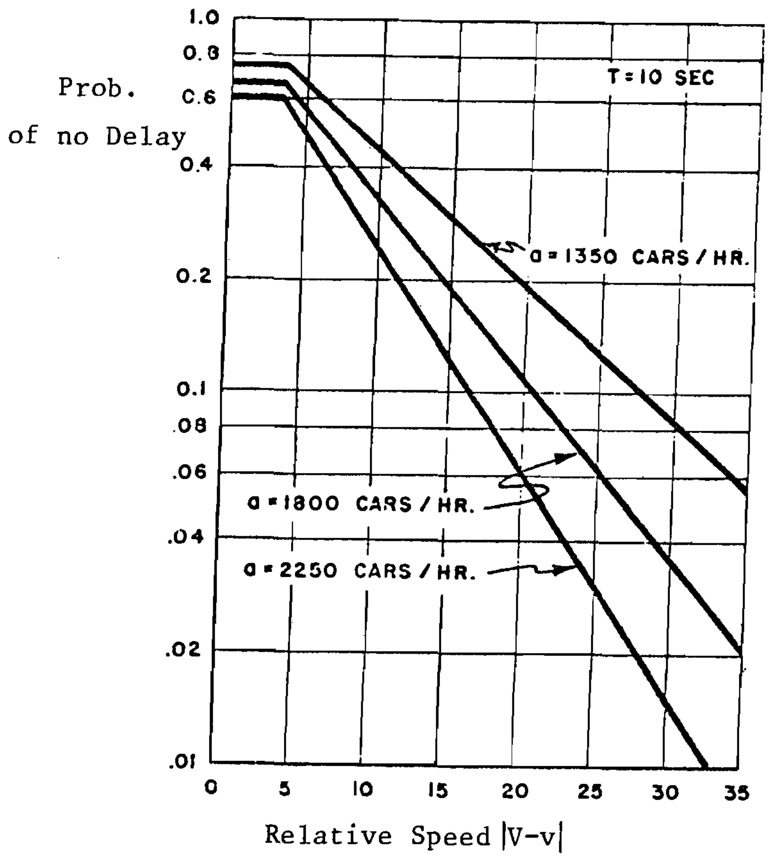


Figure 2.1: Probability of no Delay.vs.Relative Speed(Ref.4)

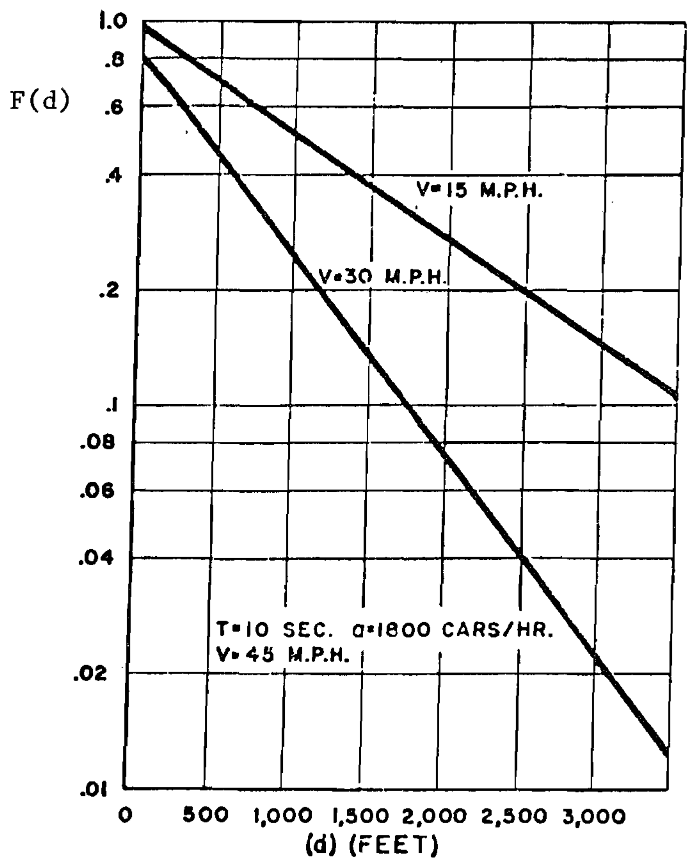


Figure 2.2: Probability that a merge requires distance d(Ref.4)

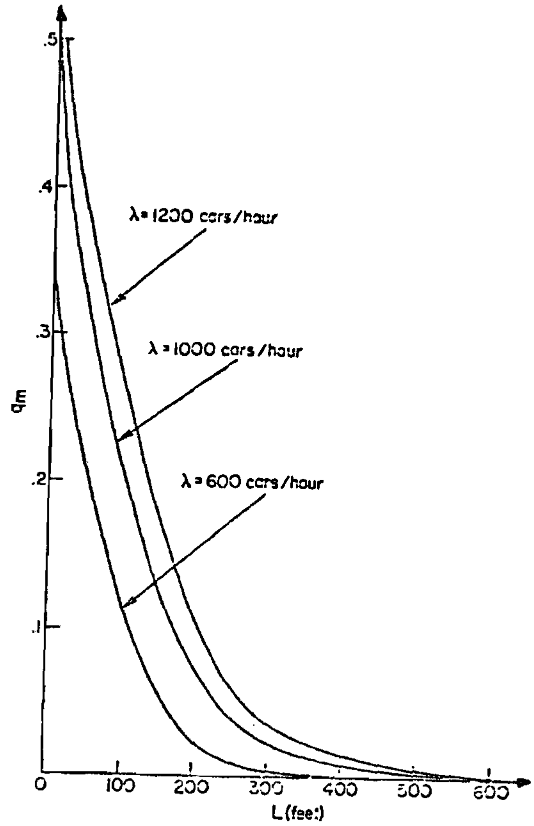


Figure 2.4:Probability of no Merge as a function of Acceleration Lane Length (Ref.2)

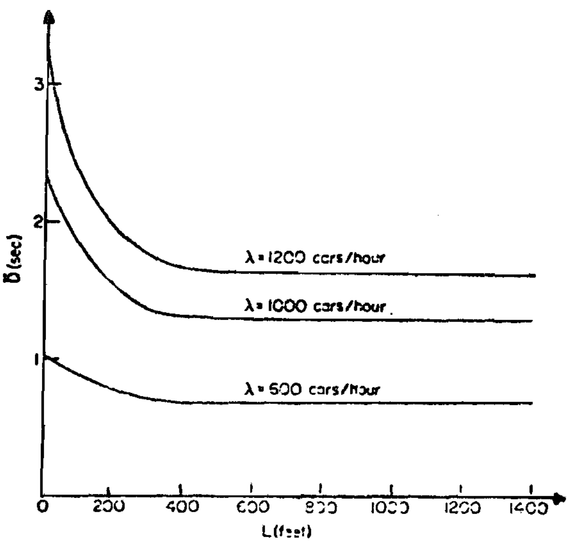


Figure 2.3:Average Delay as a function of Acceleration Lane Length (Ref.2)

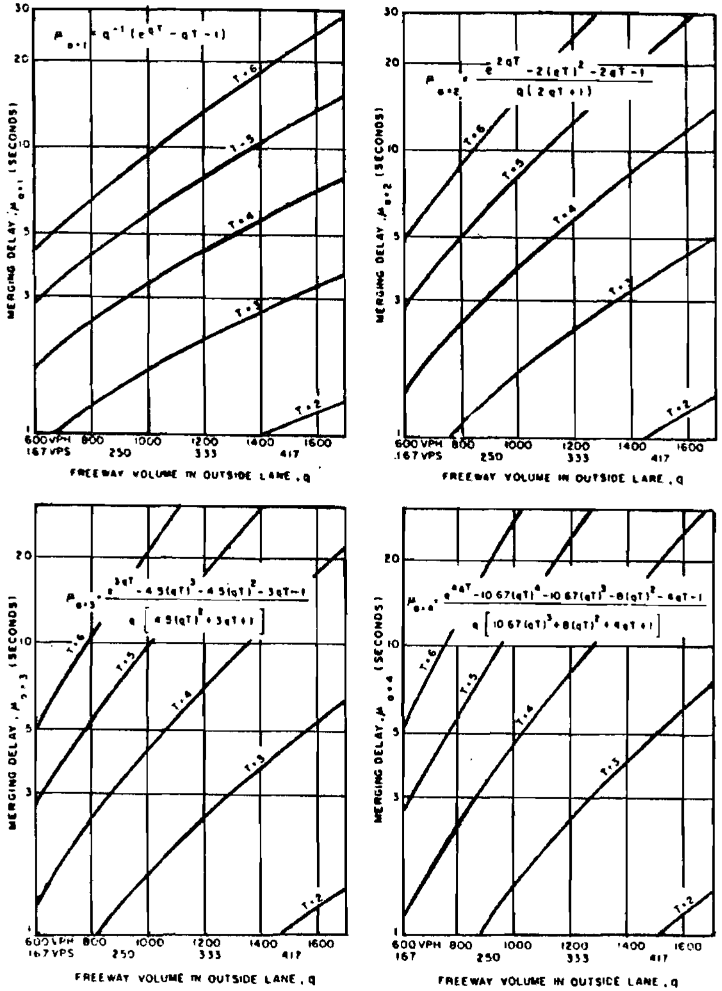


Figure 2.5:Average Merging Delay (Ref.5)

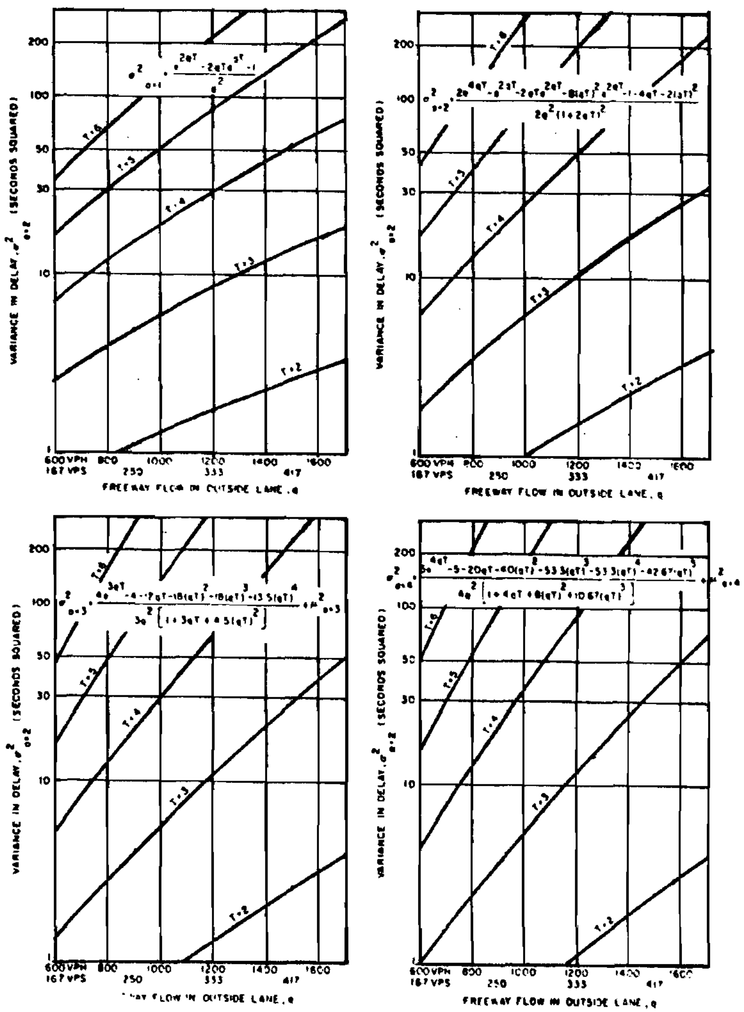


Figure 2.6:Variance in Merging Delay (Ref.5)

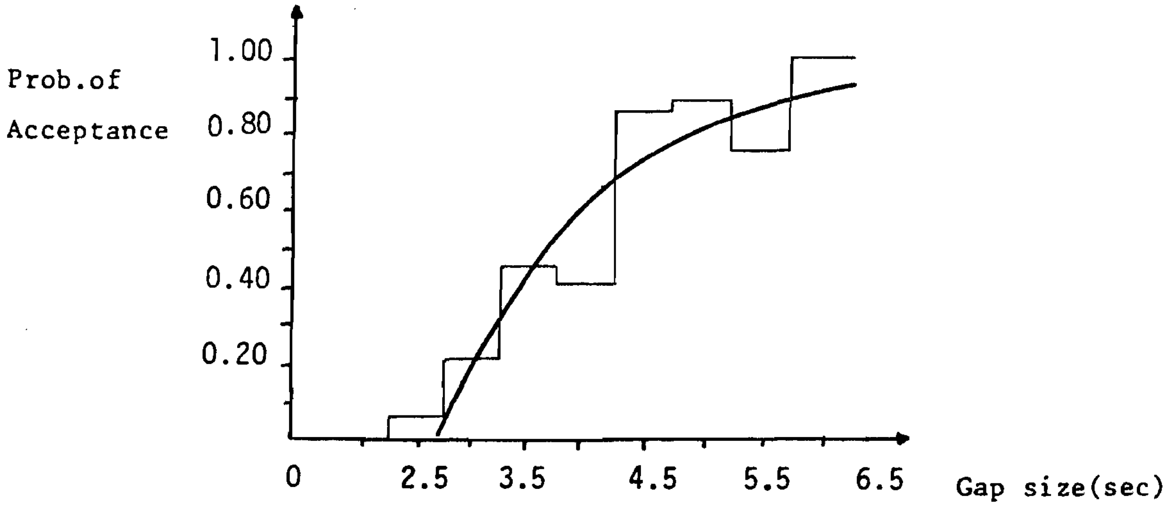


Figure 2.7:Gap Acceptance distribution for a certain driver (Ref.1)

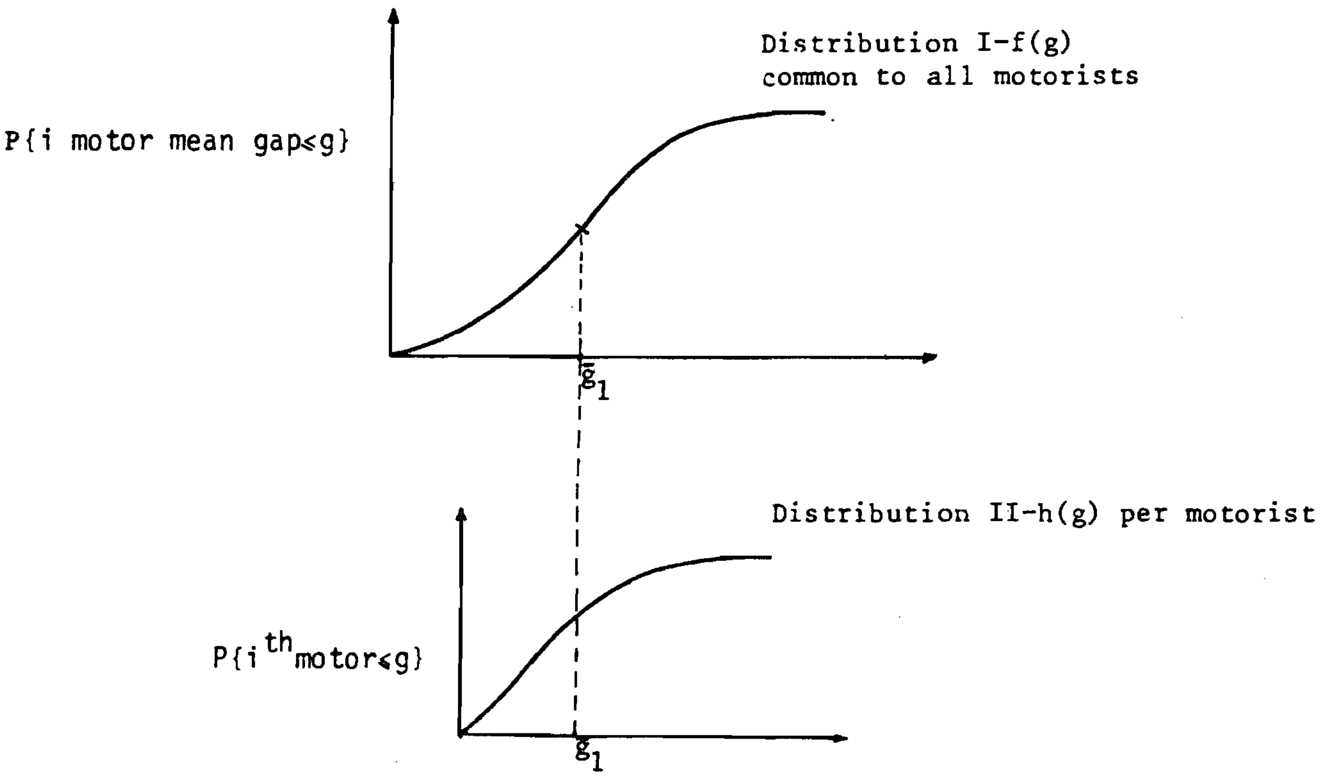


Figure 2.8:Illustration of the "Consistent-Inconsistent"

Gap Acceptance model (Ref.7)

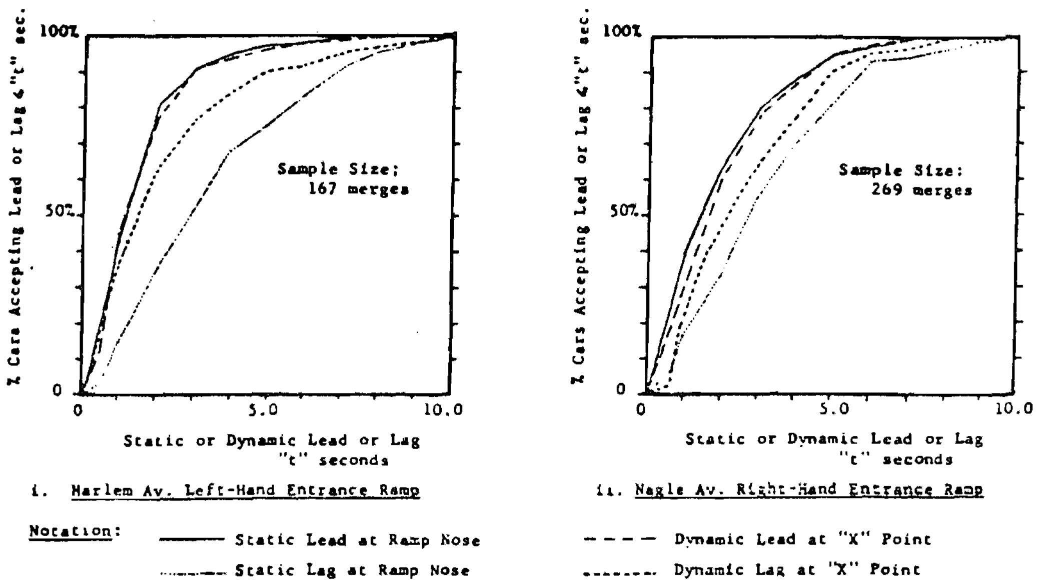


Figure 2.9:Distribution of Lead and Lag Acceptance Times (Ref.25)

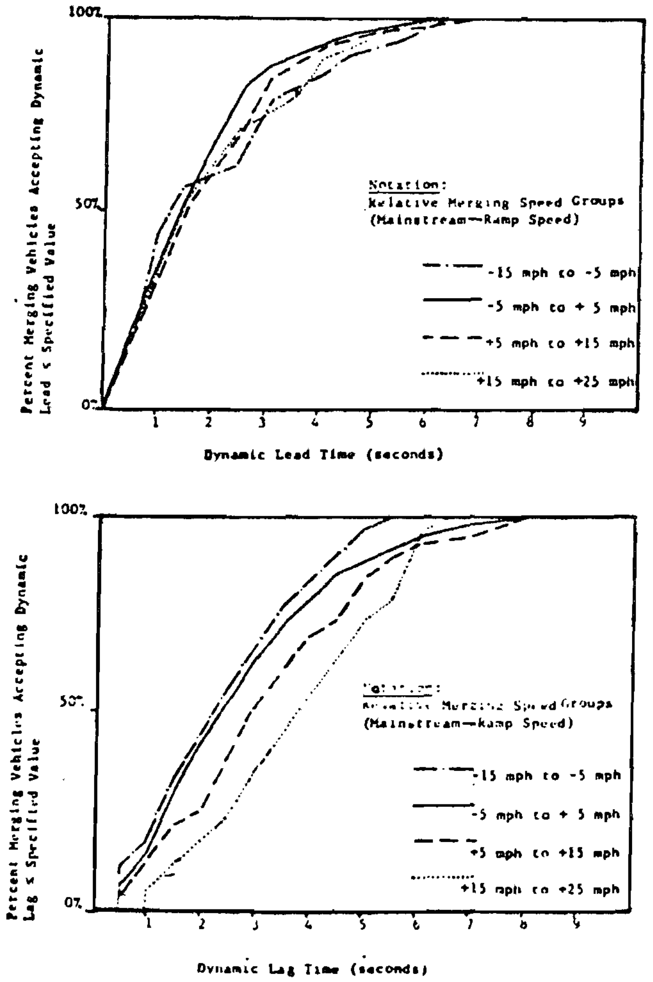
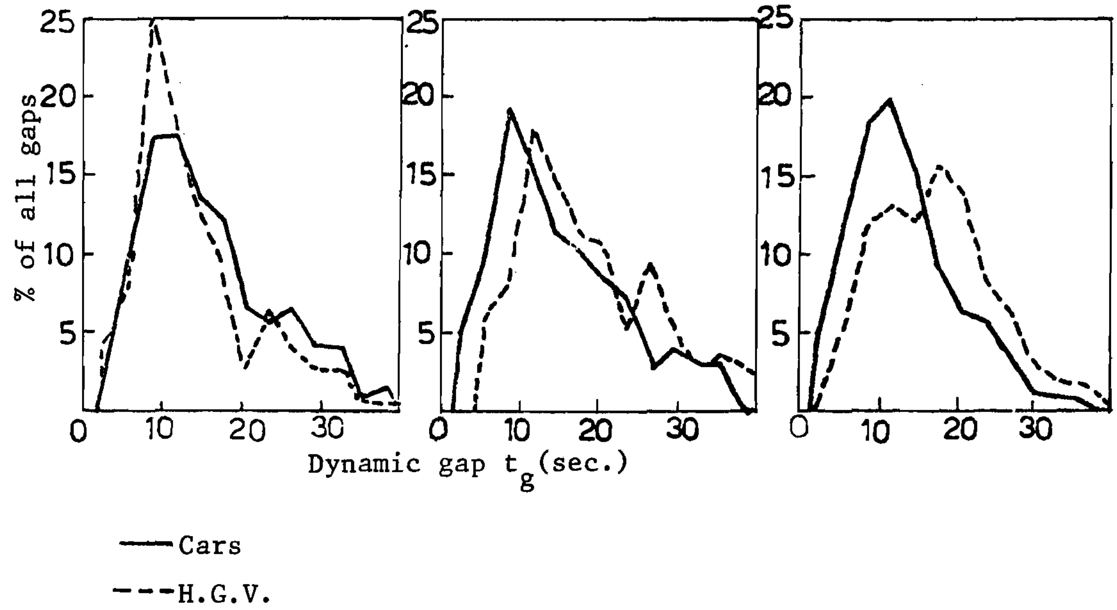
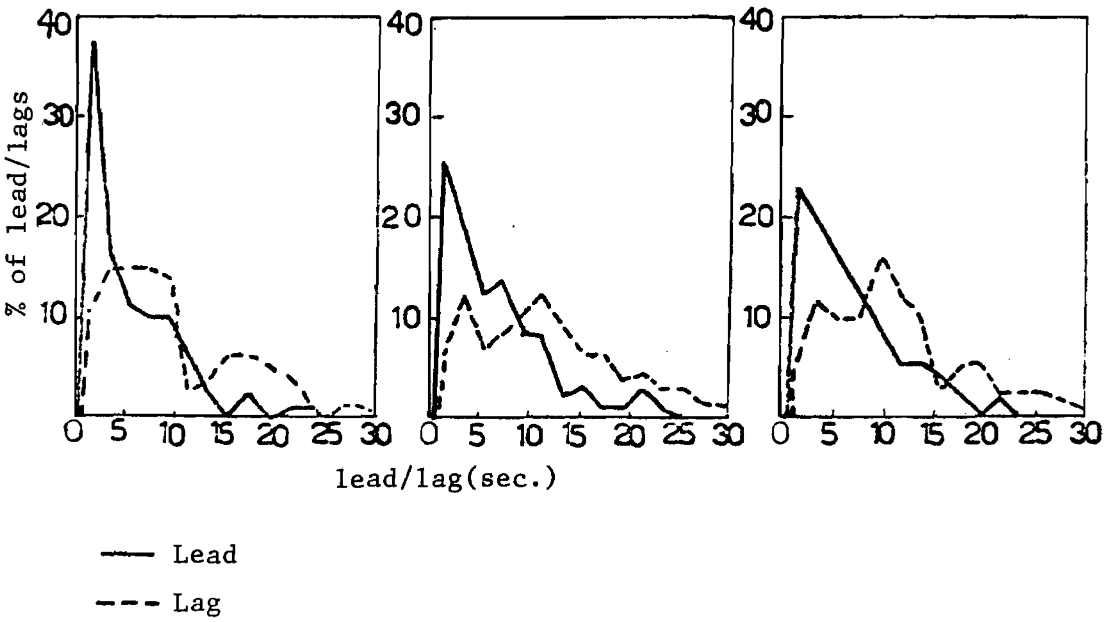


Figure 2.10:Effect of Relative Speed on Gap Acceptance (Ref.25)



a. Distribution of accepted dynamic gaps



b. Distribution of dynamic lead and lag times

Figure 2.11:Gap Acceptance at Motorway Interchanges (Ref.19)

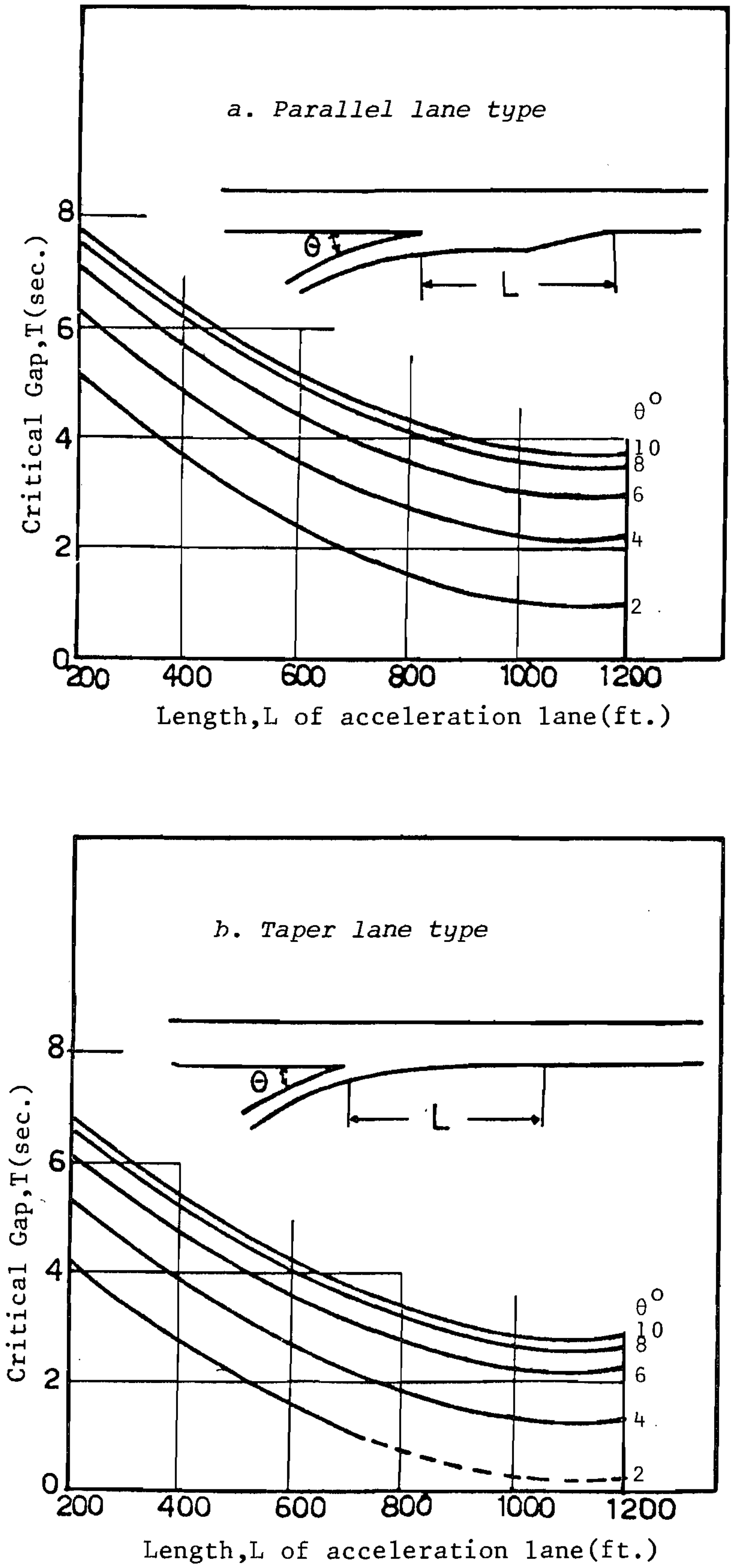
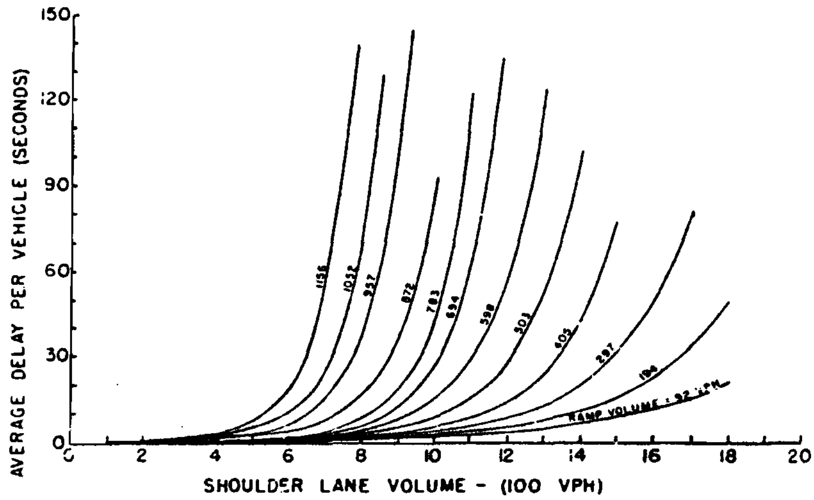
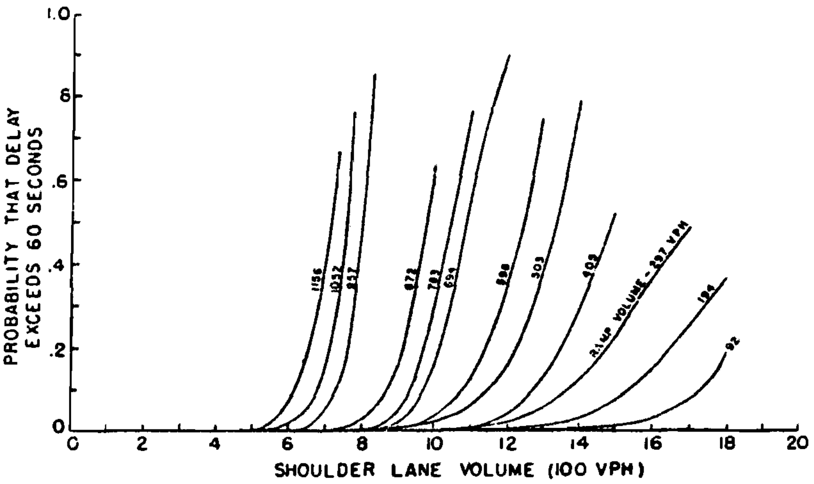


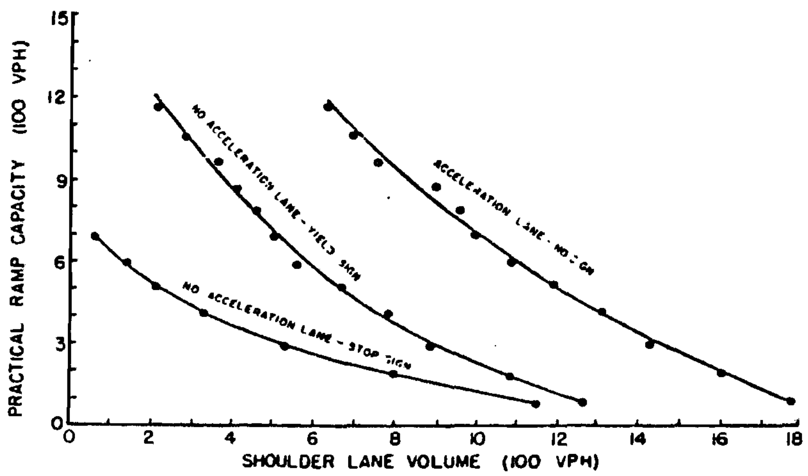
Figure 2.12:Relationship between Critical Gap and Ramp Geometrics (Ref.23)



Average delay to ramp vehicles with acceleration lane.



Probability that delay exceeds 60 sec with acceleration lane.



Practical capacities of freeway on-ramps.

Figure 2.13:Estimation of Delays and Determination of Ramp Capacity (Ref.35)

CHAPTER 3

THE METHOD OF APPROACH

3.1 General Considerations

The study of traffic behaviour at grade separated intersections is one of the most difficult traffic problems. For this kind of analysis, we decided to concentrate on microscopic simulation as it enables a wide range of conditions to be fully evaluated. The empirical methods require extensive measurements and the variability inherent in traffic data of this sort tend to mask any true operating mechanisms.

In general, simulation can be defined as "the imitation of a real situation by some form of model". The model which is used to represent the traffic situation is a special kind of mathematical model, not intended to be solved analytically, but to be implemented on a high speed electronic computer. In the computer, elements are handled in succession in a pre-determined manner. The elements in this case are numbers which may represent vehicle distribution patterns or such quantities as traffic volumes, speeds or headways. Successive calculations cause a flow of traffic within the computer, interaction takes place between the various elements in accordance with certain specified rules and the results are output in the form of computer variables which give a measure of the system's efficiency, e.g. average journey times through a particular intersection.

In this chapter, the adopted simulation procedure is broadly outlined and the basic elements of the technique are analysed.

3.2 Steps in the Simulation Procedure

The simulation procedure in this study consists of a series of stages :

Firstly, the definition of the key criteria and the determination of the basic components of the problem is essential. Because most of the variables entering into the problem are stochastic, their randomness needs to be formally expressed, the development of the mathematical distributions to represent the variability of the parameters is important. The generation of a stochastic variable is made by applying Monte-Carlo methods to the corresponding distribution.

The model development is the most important step and involves the detailed analysis of the elements comprising the traffic situation and the mathematical expression of the assumed logic and decision making process. The resultant relationships from the analysis are subsequently integrated with the aid of a computer program.

Calibration and validation are important steps in the procedure. The model is first calibrated using a data base in order to determine the values of the sensitive parameters and to indicate modifications and refinements necessary for the model results to agree with practice. At the validation stage, the results of the measures of effectiveness are compared with available information from similar situations to show the model's validity in reproducing the traffic situation. The validation may also be carried out by comparing the simulation model output with measurements taken at a site not used in the calibration.

3.3 Generation of Random Events

In the simulation procedure adopted in this study, it is essential the generation of random events, because most of the variables entering into the problem are stochastic. According to that approach, an event x, is associated with a probability of occurrence $P(x)$ having a probability density function $f(x)$. The generation takes place in two stages : Firstly a random number following a uniform distribution is generated in the interval $(0, 1)$. Secondly, this random number is treated as a probability to substitute into the appropriate distribution function in order to solve for the corresponding event.

3.3.1 Generation of Random Numbers

There have been developed various methods for the generation of random numbers (Ref. 55). The procedures used to generate random numbers are highly non-random, so the term "pseudo-random" more accurately characterises the generated numbers. But the statistical tests for these numbers do not exhibit any non-randomness. The procedure which is widely used is the simple multiplicative procedure (Ref. 44) :

$$R_m = pR_{m-1} \bmod b^n$$

where:

- R_m : the mth random number
- p : the constant multiplier
- n : number of digits (bits), in a normal word on the particular computer used.
- b : number base of computer ($b = 2$ or $b = 10$ for binary or decimal computers respectively).
- $\bmod b^n$: instruction to use only the low order or less significant half of the full (2n) digit product, i.e. the remainder after dividing the product by b^n .
- Ro : any odd number selected as a starting number.

The maximum period of generating random numbers, depends upon the word size of the computer and the selection of starting number and multiplier. Theoretically, for a binary computer, the maximum attainable period is :

$$h = 2^{n-2}$$

Also to avoid serial correlation between the random-numbers, the multiplier should be selected close to the 2^{n-2}. The formula adopted for the multiplier is :

$$p = 2^{n/2} \pm 3$$

a. Statistical Tests

The numbers generated by the above described procedure should be tested statistically, to ensure that they behave as truly random numbers. The following statistical tests are among the most widely used:

i. <u>The Frequency Test</u> : It is used to check the uniformity of a sequence of M consecutive sets of N pseudo-random numbers $r_1, r_2, r_3, \ldots, r_N$. The unit interval (0, 1) is divided into x equal sub-intervals, the expected frequency of random numbers is N/x in each sub-interval. Let f_j where j = 1, x denote the actual frequency of pseudo-random numbers $r_i (\ell = 1, N)$ in the sub-interval:

$$(j-1)/x \leqslant r_i \leqslant j/x$$

The statistic :

$$x_i^2 = \frac{x}{N} \sum_{j=1}^{x} \left(f_j - \frac{N}{x} \right)^2$$

has a chi-square distribution with x - 1 d.o.f. This statistic is then computed for all M consecutive sets of N pseudo-random numbers. We denote F_j the frequency of resulting M values of x_i^2 which lie between the (j-1)th and the jth quantile of the chi-square distribution with x-1 d.o.f. The following statistic is computed :

$$x_F^2 = \frac{u}{M} \sum_{j=1}^{u} \left(F_j - \frac{M}{u} \right)^2$$

If x_F^2 with u - 1 d.o.f. exceeds the critical value set by the desired level of significance, the hypothesis that the numbers generated in the sequences consisting of M sets of them are truly random, is rejected.

ii. <u>Other Tests</u> : The generated numbers in the interval (0, 1) are tested against the requirements :

$$\frac{1}{N} \sum_{i=1}^{N} r_i = 0.50 \qquad\qquad \frac{1}{N} \sum_{i=1}^{N} r_i^2 = 0.333$$

b. Description of the Selected Random Number Generator

In this study, the built-in routine for the generation of random numbers of the ICL 2970 computer at the University of Southampton was used (Ref. 54). This routine generates random fractions in the interval (0, 1) according to the multiplicative procedure, described in Section 3.3.1:

$$N = 13^{13} \times N \mathrm{Mod} 2^{59}$$

and the random fraction x is obtained by calling the real function :

$$x = \mathrm{GO5CAF(X)}$$

The initial random number has value :

$$N = 123456789 \times (2^{32} + 1)$$

The random number generator should be initialised at the start of each simulation run. The initialisation routine used has the form :

SUBROUTINE G$\emptyset$5CBF(I)

where I is an integer variable and it is used to calculate the value N used by the generation routine according to the expression :

$$N = 2 \times I + 1$$

In order to use these routines in the simulation model a special computer program, was written to check that the random numbers generated satisfy the requirements of randomness, according to the standard tests.

In table 3.1 the mean and mean square value of the random numbers are given. For the frequency test the values :

$$M = 100 \qquad N = 1000$$
$$X = 10 \qquad U = 10$$

were chosen and table 3.2 gives the frequency distribution of x_i^2

statistic calculated for each set $i (i = 1, M)$ the expected frequencies, and the statistic x_F^2 .

It can be seen that the hypothesis of randomness is accepted :

$$x_F^2 = 10.0 \leqslant x_{0.95,9}^2 = 16.92$$

TABLE 3.1

Tests for Random Numbers

RANDOM NUMBER GENERATED	MEAN VALUE	MEAN SQUARE VALUE
1000	0.510462	0.343018
10000	0.497570	0.331791
20000	0.496786	0.330465
30000	0.496888	0.330487
40000	0.496654	0.330138
50000	0.497473	0.331003
60000	0.497594	0.331965
70000	0.497975	0.331482
80000	0.498399	0.331758
90000	0.498783	0.331964
100000	0.498705	0.331757

TABLE 3.2

Frequency Test for Random Numbers

RANGE	ACTUAL FREQUENCY OF X_i^2	EXPECTED FREQUENCY	X_F^2
1.40 - 3.32	6	10	1.60
3.32 - 5.24	12	10	0.40
5.24 - 7.15	15	10	2.50
7.15 - 9.07	13	10	0.90
9.07 -10.99	10	10	0.00
10.99 -12.91	12	10	0.40
12.91 -14.83	10	10	0.00
14.83 -16.74	9	10	0.10
16.74 -18.66	6	10	1.60
18.66 -20.58	5	10	2.50
			10.00

3.3.2 Conversion to Random Deviates

Conversion of random numbers to random deviates satisfying the desired frequency distribution, can be done by two methods : the point distribution method and the method of inversion, a short outline of these methods is given below:

i. Point Distribution Method : Is used where the probability density function of a distribution is difficult to integrate or to invert. If the probability function is bounded by values x_{min} and x_{max}, two random numbers x_1, x_2 are generated satisfying the conditions:

$$x_1 = x_{min} + (x_{max} - x_{min})r_1$$

$$x_2 = f(x)_{max}r_2$$

in which r_1 and r_2 are uniformly distributed fractions of unity and $f(x)_{max}$ the maximum value of the probability density function $f(x)$.

We accept the random number x_1 as the desired random deviate if the following test is satisfied :

$$x_2 < f(x_1)$$

The number of successful tests in an interval Δx is proportional to the area under the curve in that interval, which is the desired distribution.

ii. Method of Inversion : The cumulative distribution function of a variable specifies the probability of obtaining a given value or less, from a distribution. If the density function $f(x)$ describing the distribution of a variable can be integrated and the resultant cumulative distribution function analytically inverted, a value of the cumulative function could be calculated for a selected uniform random fraction.

a. Random Deviates for the Exponential Distribution

The probability density function of the negative exponential distribution is given by the formula :

$$f(t) = \lambda e^{-\lambda t}$$

The cumulative distribution is :

$$P(h < t) = 1 - e^{-\lambda t} \qquad (1)$$

where λ : average rate of arrivals, t time interval between arrivals, $1/\lambda$ mean arrival time. From equation (1) we obtain :

$$e^{-\lambda t} = 1 - P(h < t)$$

or

$$t = \frac{1}{\lambda} \ln(1 - P(h < t)) \qquad (2)$$

If a random fraction r_i is generated and set (Ref. 53) :

$$r_i = 1 - P(h < t)$$

then from equation (2) we have the desired random deviate

$$t = -\frac{1}{\lambda}\ln(r_i)$$

A flow chart for the generation of exponentially distributed deviates is given in Fig. 3.1.

b. Random Deviates for the Erlang Distribution :

The probability density function of the Erlang distribution is given as :

$$f(t) = \frac{\lambda^k}{(k-1)!} t^{k-1} e^{-\lambda t}$$

Based on the fact that the Erlang variate is the sum of k exponential variates, with the same expected value :

$$x = \frac{\text{Mean}}{k}$$

we can generate deviates as for the case of the exponential distribution.

c. Random Deviates for the Normal Distribution

The probability density function of the normal distribution is given as :

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-(x-\mu)^2/2\sigma^2}$$

with mean μ and variance σ^2

If we have a sequence of random numbers $r_1, r_2, \ldots r_k$ defined over the interval (0, 1), then the random variable :

$$Z = \frac{\left(\sum_{i=1}^{k} r_i - K/2\right)}{\sqrt{k/12}}$$

is normally distributed according to the Central Limit Theorem (Ref. 44). The k number of random numbers chosen, is a question of accuracy and computational efficiency usually :

$$k = 12$$

The desired random deviate is :

$$x = \sigma Z + \mu$$

or

$$x = \left(\sum_{i=1}^{12} r_i - 6.00 \right) \sigma + \mu$$

A flow chart for a subroutine to produce normal deviates is given in Fig. 3.2.

d. Random Deviates for the Lognormal Distribution

A variable x is log-normally distributed, when the natural logarithm of x is normally distributed. Given the parameters of the lognormal distribution m, s, we calculate the parameters of the corresponding normal distribution μ, σ :

$$\mu = \ln(m) - \sigma^2/2$$
$$\sigma^2 = \ln(s^2/m^2 + 1)$$

a random variate y is then generated from the normal distribution $N(\mu, \sigma)$. The log-normally distributed random deviate is :

$$x = \exp(y)$$

3.4 Time Sequencing

One of the most important considerations in formulating a computer model for a particular traffic situation is the method used to move the system being simulated through time. Time sequencing not only controls the way in which time is processing but also has influence on all other stochastic processes. Because it is not possible to examine all parts of a system simultaneously, and time is divided into discrete units, it is necessary to scan the system by some method. There are two methods mainly used: Event scanning and periodic scanning :

a. Event Scanning : The procedure consists of determining a set of imminent significant events, and times at which they will occur, and selects the earliest event without any intervening scans (Ref. 42). This technique generally increases computing speed, being most suitable for simulation of large scale traffic systems.

b. Periodic Scanning : The duration of the simulated phenomenon is divided into a number of successive fixed time intervals (Δt) in which the computer program records the system's state. This technique is straightforward but has the disadvantage that the exact time of occurrence of an event may be lost, unless a small time increment is used.

In this study, the periodic scanning has been adopted as a method of updating the elements of the situation being modelled.

3.4.1 Time Increment

The selection of a suitable time increment for a periodic scanning simulation model is important. If the time increment (Δt) is too small many additional computations will be made, thereby increasing computation time and cost of the program. On the other hand, if a large Δt is used, it may affect significantly the accuracy of the results. The determination of the suitable time increment largely depends on the scope of the simulation program, i.e. if is concerned with a microscopic investigation of traffic flow then a small time increment is required.

In this study, the model was constructed to allow for variable time increment Δt to be used. The effect of Δt variation on the measures of effectiveness was tested by a series of simulation runs, and is shown in Fig. 3.3. It has also been suggested that the Δt should be a multiple of the driver's reaction time (T) . Based on the above considerations a typical value of

$$\Delta t = 0.50 \text{ sec.}$$

has been chosen.

3.4.2 Sampling Interval

Starting a simulation run of any process is complicated by possible bias introduced through using an arbitrary starting point such as an empty system. It is common practice to let simulation run for some time to allow it to reach 'equilibrium' before realistic vehicle data are collected and stored. Another important aspect is the determination of the length of simulation run and the sample of vehicles to be simulated.

It has been suggested (Ref. 40) that sufficient times of traffic operation should be simulated with constant geometric and traffic characteristics to analyse the pattern of measures of effectiveness and the variance that occurs, and the simulation time should be chosen on the basis of that analysis.

In the present model the selection of simulation time, equilibrium time or settling period, was based on a series of simulation runs. The results are shown in Fig. 3.4, 3.5. According to them the minimum settling period, FTIME, and the simulation time, STIME, were chosen to have the typical values:

FTIME = 3 min.

STIME = 1 h. (60 min.)

3.5 Simulation Advantages and Applications

The main advantage of simulation is that it can determine the effects of changes in the operating conditions on the evaluation parameters, without extensive, time consuming and expensive field experiments. A valid simulation model is a powerful tool which enables the engineer to 'predict' the behaviour of a system prior to its construction, also to test alternative designs under identical conditions. There are also some additional reasons making simulation most suitable for traffic studies :

(i) The effects of various traffic control devices as signals, speed limits, signs and access control can be studied in detail without confusing or alarming drivers (Ref. 58).

(ii) Simulation of complex traffic operations may provide an indication of which variables are important and how they relate. This may eventually lead to successful analytical formulations, or to check an uncertain analytical solution.

(iii) Simulation is relatively inexpensive and safe.

A comparison between analytical solutions, trial and error processes and simulation, in terms of some requirements, may be summarized below, based on Goode (Ref. 66).

	Analytical Solution	Simulation	Trial-Error
Cost	Least	Medium	Most
Generality of results	Most	Medium	Least
Reproducibility	Most	Medium	Least
Time	Least	Medium	Most
Realism	Least	Medium	Most

It can be seen that simulation is midway between the analysis and trial. But as the situation being studied becomes more complex the differences between the methods became more pronounced, until finally none of the extremes can be tolerated and simulation becomes the only feasible method.

Application of simulation due to the above reasons is extensive as a research and design tool. The study of intersection behaviour, traffic flow on motorways, coordinated signal systems, theory of traffic flow are some of the fields on which simulation is widely used (Ref. 48).

3.6 Historical Development

It is believed that the earliest simulation of road intersection was produced by Hillier, Whiting and Wardrop (Ref. 56) in 1952 and described as "The Automatic Delay Computer". This was subsequently developed by Webster for the analysis of traffic signals (Ref. 57).

In 1955 Mathewson, Trautman and Gerlough (Ref. 49) suggested two methods of using electronic computers to solve traffic problems at an intersection, by simulation. The first method was analog in character and made use of a special-purpose discrete-variable computer in which each vehicle was represented by a voltage pulse whilst the second used a general-purpose digital computer with arithmetic binary notation (series of o's and 1's) to represent vehicle distribution patterns. Arithmetic manipulation of these binary words caused an apparent 'flow' of vehicles through the intersection.

Gerlough (Ref. 39) presented an alternative method for vehicle representation in 1956. In this method, since referred to as memorandum representation, one or more words are assigned to each vehicle giving information such as location, lane number, actual speed, desired speed and the model is updated on event or periodic basis. Memorandum representation is amended to take account of any changes due to interactions between vehicles. A third method of representation was also proposed, called the mathematical notation. This is similar to the memorandum notation except that, in addition to its other characteristics, each vehicle is associated with its own position indicator. A vehicle's new position can at any time be computed as a function of its last position, its velocity, its acceleration, and the time increment. Spacings between vehicles are available from their respective coordinates and the vehicle length.

Following the rapid development of electronic computers, and the better understanding of traffic processes, simulation techniques were further improved and their use was rapidly increased as a tool in traffic systems evaluation.

The greater number of simulation models have been concerned with traffic behaviour at intersections, in order to evaluate the merits of alternative forms of control (stop-sign or traffic signals), and to estimate the junction capacity and delays under various conditions (Ref. 59, 67, 40, 69, 70). Driver behaviour in terms of perceptual, decision-making and response processes at an intersection was also studied by simulation (Ref. 60).

Examples of simulation models to study the merging process have been described in Chapter 2. Similar models for the merging situation have also been developed to study the effects of ramp metering systems (Ref. 38, 64, 65, 66) and in general the estimation of benefits for installation of surveillance and control systems on freeway sections with combinations of on and off ramps, a description and comparison of them is given by Hsu and Munjal (Ref. 51).

Simulation techniques were also used for the evaluation of the benefits of priority operations on the freeways such as bus-lanes and to give quantitative criteria on their performance (Ref. 50, 52). On the other hand, the movement of vehicles through a traffic network and the performance of the linked traffic signals have also been tested by simulation procedures (Ref. 62, 63).

Most of the simulation studies have been carried out on general purpose computers with memorandum representation of information, although in some cases special purpose simulators have been developed for intersection studies, as for example at the University of Manchester Institute of Science and Technology (Ref. 61). The predominant computer language in simulation programs is FORTRAN, although some special problem oriented languages have been introduced, e.g. GPSS, SIMCAR, SUMSCRIPT, DUNAMO (Ref. 48), to assist in the treatment of complex systems.

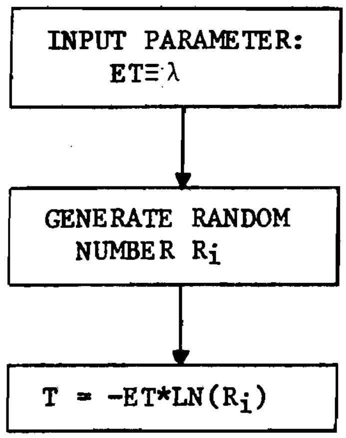


Figure 3.1: Generation of Exponential random deviates.

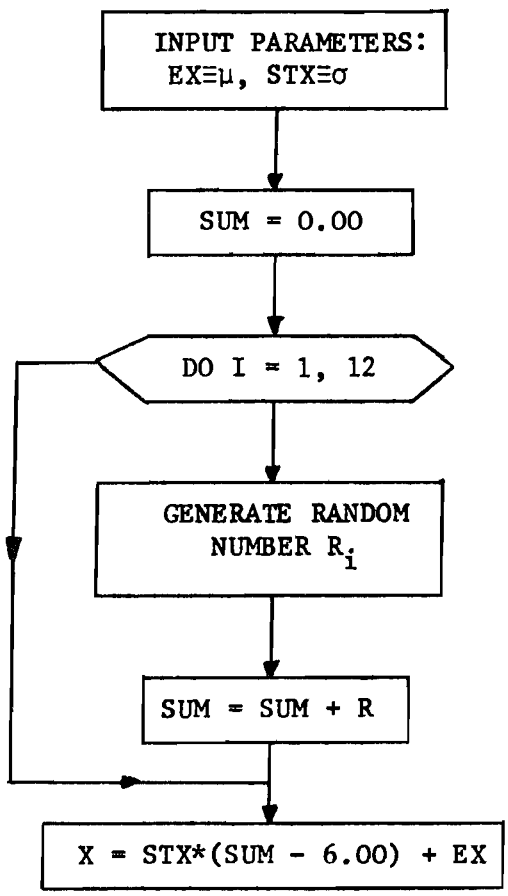


Figure 3.2: Generation of Normal random deviates.

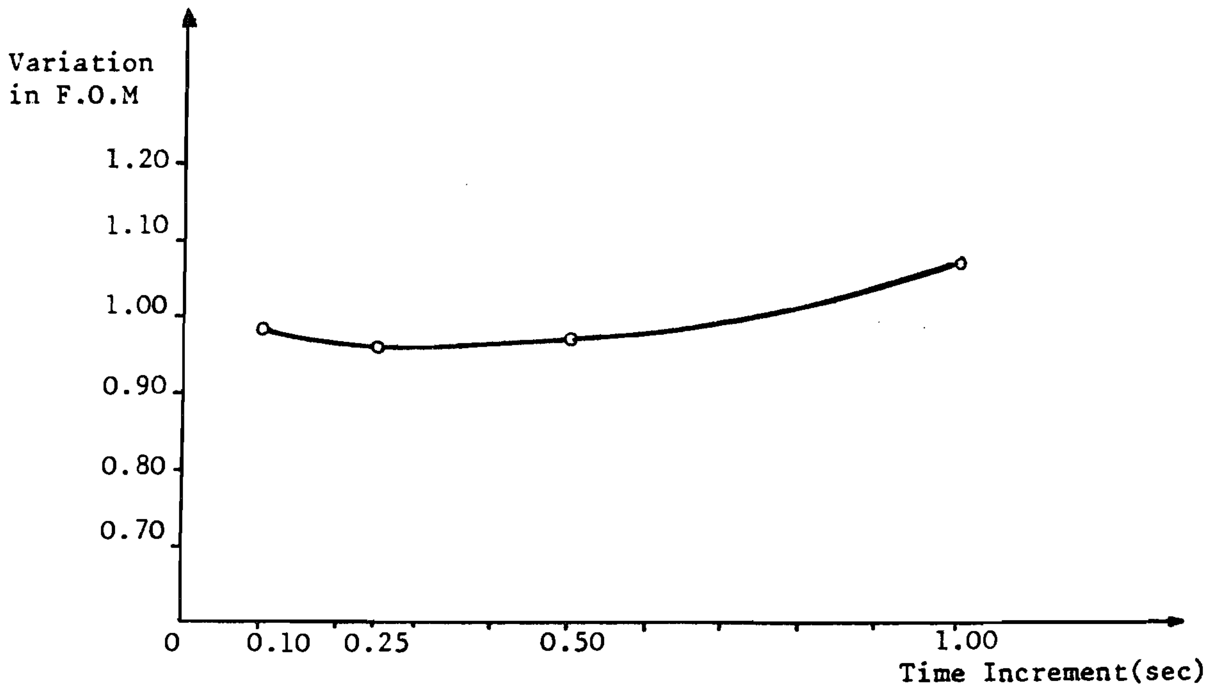


Figure 3.3:Sensitivity to Time Increment

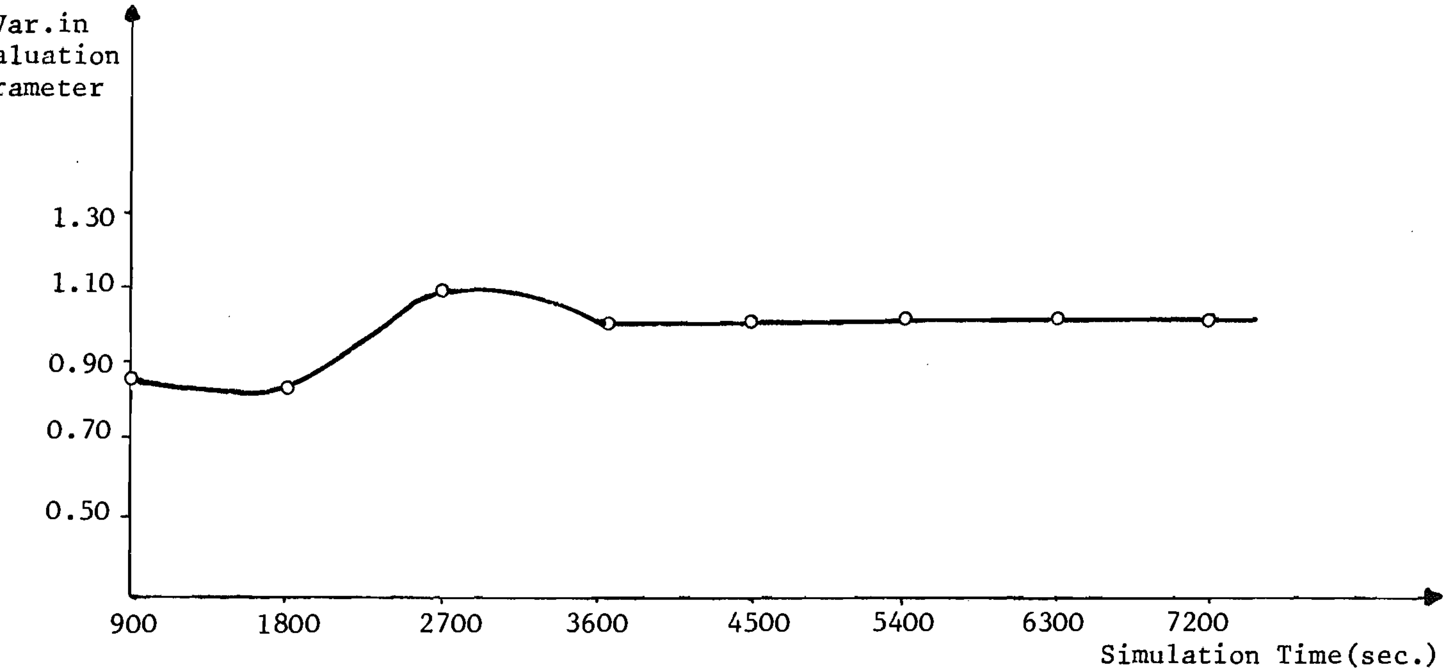


Figure 3.4: Sensitivity to Simulation Time

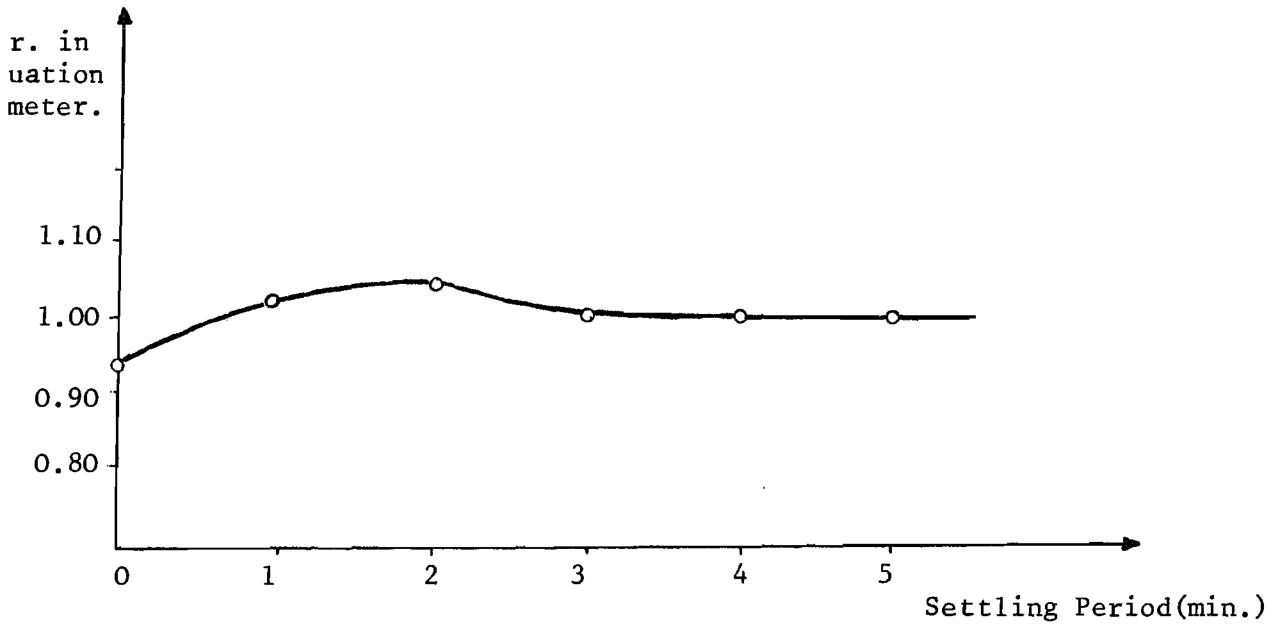


Figure 3.5: Sensitivity to Settling Period

CHAPTER 4

MODEL FORMULATION

4.1 General

The aim of the research study is to investigate the merging process, how it is affected by the geometric design and to estimate the delays and capacities at ramp entries, based on a microscopic model of traffic flow. The development of the model forms the major part of the study, as has been previously pointed out, we are dealing with a complex situation of inter-related geometric and traffic variables. Effort is needed to give a realistic formal representation of the real traffic behaviour. The assumptions which have to be made should not affect the realism and the underlying mechanisms of the situation and should be justified at theoretical and/or empirical levels.

In this chapter, the formulation of the model is described. Each component is analysed and the proposed modelling process is presented in detail. The measures of effectiveness are also defined.

4.2 Interchange Geometrics

Grade separation is an essential feature of intersections between motorways and all other roads. The interchange configuration is not standard, but varies depending on the type of intersecting roads, the physical conditions at the site and the direction and volume of traffic expected to use the junction. Geometric design is of great importance and affects the overall performance of the interchange. In this section, the design elements of the merging situation are outlined and the geometric characteristics used in the model are defined.

(i) Motorway (Freeway) : is defined as a road with complete control of access, has dual carriageways and its use is limited to motor vehicles. The geometric standards are governed by the traffic volumes and the high speeds attained by the vehicles. Special consideration should be given to the horizontal and vertical design components to provide a smooth and flowing alignment. The design standards are given in Table 4.1 (Ref. 89). The design flows under standard vehicle composition, i.e. 15 per cent of H.G. Vehicles, are given in Table 4.2 (Ref. 108). In the model a level and tangent section of a three-lane motorway has been assumed to represent the main road having an overall length O_1X_1, where O_1 is the generation point and X_1 the point where the vehicles leave the system as it is shown in Fig. 4.1. The distance O_1A between the generation point and the ramp nose has been chosen such that to allow for a realistic build-up and stabilization of mainstream traffic at the ramp entry. Typical values are :

$$O_1A = 450 \text{ m.}$$

$$O_1X_1 = 850 \text{ m.}$$

(ii) Slip-road : is the connecting link between the intersecting roads. It may either be straight, slightly curved or looped, depending on the layout of the junction. The design flow for one lane ramp should not exceed the volume of

$$Q = 1200 \text{ veh/h}$$

under standard vehicle composition. The geometric standards are in general lower than those for motorways. The design speed is usually taken as the 0.85 of the design of the motorway itself, with a minimum of about 0.4, depending on the ramp shape (Ref. 107). For flyover version interchanges, the gradients should preferably not exceed 5 per cent and should nowhere be steeper than 8 per cent. Where a slip road carries a large volume of heavy commercial traffic its gradient should be limited to 4 per cent (Ref. 90). On down-ramps, flyunders, the same maximum values apply but in special cases may be 2 per cent greater (Ref. 107). The overall width is set to 20 ft, with a carriageway of 14 ft. The length of the ramp provided is very important for the junction operation; a short ramp results in high relative speeds at ramp nose, whereas at the end of a sufficiently long ramp the merging vehicles can attain the motorway running speed. The angle of convergence θ for rural interchanges has normally a standard value of :

$$\theta = 1^{o}\ 26'$$

but may vary in different sites. In the model a straight section of one lane slip road is assumed, having a length AO_2, where O_2 is the generation point of the slip road vehicles and A the ramp nose. In this length is included the section AC referred to conventionally as 'ramp nose' (Fig. 4.1). Typical values used are :

$$AO_2 = 450 \text{ m.}$$

$$AC = 122 \text{ m.} \simeq 400 \text{ ft.}$$

The angle of convergence has the standard nominal value and the default values of the gradients are for the flyunders.

(iii) <u>The Acceleration Lane</u> : It is designed such that to enable the merging vehicles to reach the running speed of the main traffic stream, and to move traversely to the motorway lane at an acceptable safe rate. Its length is based on the following factors :

a) The speed at which drivers merge with through traffic.

b) The speed at which drivers enter the acceleration lanes.

c) The manner of accelerating or the acceleration rates.

It also depends on the relative volumes of through and entering traffic. The shape of the acceleration lane can be tapered or of parallel type. The current design procedures suggest a length of 800 ft. for rural motorway interchanges with lower values for urban sites (Ref. 89, 90). In the model a tapered acceleration lane has been chosen, with length AD measured from the ramp nose. The values of AD are in the range:

154 - 244 m., i.e. 500 - 800 ft.

TABLE 4.1

Motorway Design Elements

Design Speed	70 mph.
Min Stopping Sight Dist.	950 ft.
Min Desir. Radius	5000 ft.
K value (crest)	300 ft.
K value (sag)	250 ft.
Lane Width	12 ft.
Max. Superelevation	7%

TABLE 4.2

Design Flows for Motorways

SITE	MOTORWAY TYPE	PEAK HOUR FLOW (v/h/Carriageway)	
		STANDARD	MAX.WORKING
URBAN	Dual 2-lane	3600	3600
	Dual 3-lane	5700	5700
	Dual 4-lane	7600	7600
RURAL	Dual 2-lane	2400	3200
	Dual 3-lane	3600	4800
	Dual 4-lane	4800	6400

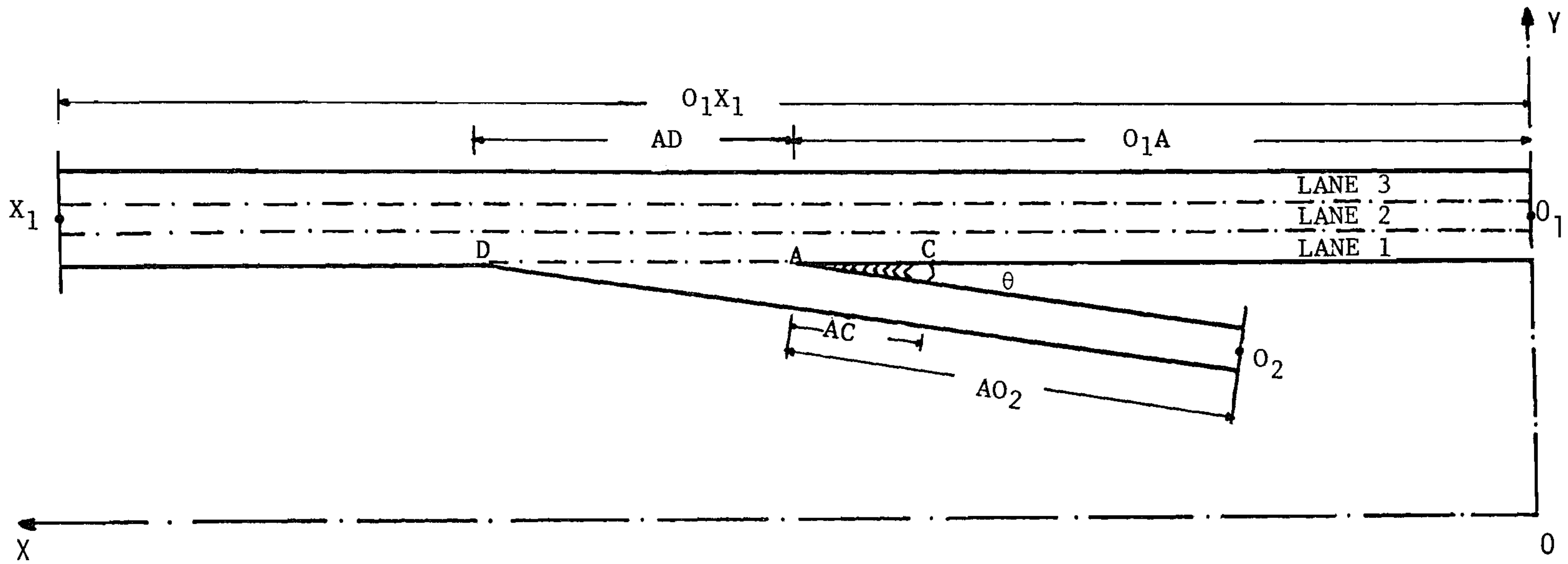


Figure 4.1:Typical Geometric Configuration,Definition of the Design Elements.

4.3 Driver - Vehicle Characteristics

The Driver and Vehicle characteristics have a considerable influence on the operation of a highway facility and should be taken into account in a microscopic investigation of the traffic process. Drivers' characteristics affect the magnitude of traffic parameters, e.g. gap acceptance, and are involved in the determination of safe-stopping sight distances, safe approach speeds at intersections, signs and delineation and other elements of motorway design (Ref. 87).

On the other hand, the type of vehicle and its characteristics such as length, engine capacity, acceleration and deceleration capabilities, should be taken into account because they affect the capacity of the road, the average running speed and headways (Ref. 86) and also the geometric design e.g. gradients, minimum turning radii at junctions, provision of climbing lanes.

4.3.1 Reaction Time

The road user in his every act is dependent on perception, intellection, emotion and volition (Ref. 92). Perception is the time required for a driver to realize that he must take action; perception may result to intellection, the formation of new ideas. Linked to the processes of perception and intellection are emotional sensations to the driver and the final decision requires the resolution of all impulses received into a volitional outgoing message which results in some definitive reaction.

The driver's response or reaction to a given situation takes time due to the above described psychological process. Reaction time is an important human parameter, it varies between the individuals and also depends strongly on the traffic situation e.g. an urban intersection or an open motorway, in the latter case being considerably smaller. The problem is that it is very difficult to be accurately measured. Haight (Ref. 4) has given a probability distribution of reaction time among drivers with mean value :

$$T = 0.73 \text{ sec.}$$

and a study on the braking reaction time has reported a range of values between 0.3 sec. and 2.00 sec., with a 50 percentile value of 0.90 sec. (Ref. 88).

Due to the lack of definitive distribution to express the variability of reaction time, in this study the reaction time is assumed constant for all the drivers, having typical value :

$$T = 1.00 \text{ sec.}$$

4.3.2 Vehicle Types

Vehicular traffic is composed of two broad classes of vehicles : Passenger vehicles such as cars, motorcycles, buses and Commercial vehicles which include a wide range of light vans to articulated lorries.

In the model motorcycles are excluded and two vehicle types are considered in terms of traffic behaviour : Cars, including light vans, and Heavy Good's Vehicles (H.G.V.) defined as follows :

Car : A passenger vehicle of a 1.4 litre petrol engine capacity on average, or a light goods vehicle under 1.5 ton unladen. Passenger cars and light vans have the same effect upon the capacity of roads and junctions. (Ref. 89, 90).

H.G. Vehicle : A general goods vehicle over 1.5 ton unladen and with an average carrying capacity of 10 ton (Ref. 93). Traffic composition is referred to as the percentage of H.G.V. in the traffic stream and in general varies with the site and traffic conditions. A proportion of 15 per cent is set as standard composition for design purposes. (Ref. 108).

In the model, the proportions of H.G.V. for the motorway and ramp vehicles p_m and p_r are input parameters and have typical values in the range :

p_m, p_r : 5 - 25 per cent.

Based on the p_m the proportions of H.G.V. on the motorway lanes 1 and 2 have typical values calculated from the expressions (Ref. 95) :

$$p_{m_1} = \frac{1200\ p_m Q}{(1200 + p_m Q) \cdot Q_1} \qquad (4.1)$$

$$p_{m_2} = \frac{(p_m Q)^2}{(1200 + p_m Q) \cdot Q_2} \qquad (4.2)$$

where :

Q : total motorway flow (veh/h)

Q_1, Q_2 : flow on lanes 1 and 2 (veh/h)

The right hand lane of the motorway (Lane 3) is only for cars. ($p_{m_3} = 0$). The determination of the vehicle type in the model is as follows : When a vehicle is generated, a random number R is generated and compared with the proportion of H.G.V., for the specific lane. If it is smaller or equal to that, the generated vehicle is commercial otherwise it is a car. A flow-chart for the above stated procedure is given in Fig. 4.2.

Once the vehicle type is determined, the subsequent characteristics of the vehicle are generated.

4.3.3. Vehicle Lengths.

The vehicles' dimensions vary due to their type and to the trends in motor industry. For highway design purposes according to AASHO (Ref. 87) dimensions of the 'design vehicle' are used, which is defined as having physical dimensions and a minimum turning radius longer than those of almost all vehicles in its class. Typical values are given in Table 4.3.

In England S. Van As (Ref. 91) has done field measurements and found that the normal distribution can describe the variation of the vehicle lengths.

The values of the parameters are given in the table 4.4. In the same table the average values estimated by Branston based on data taken from the M4 motorway are also shown (Ref. 86).

In the model the lengths of the vehicles are assumed to be normally distributed for each vehicle type. The parameters of the normal distributions are given in the table 4.5 for both cars and H.G.V. Each vehicle length is sampled from the corresponding distribution according to its type.

4.3.4 Acceleration and Deceleration Rates

The acceleration and deceleration of vehicles, i.e. the rate of speed change at a time interval, vary according to vehicle type, its mechanical capabilities, its speed and the road geometry especially gradients.

i. Maximum Acceleration : It varies inversely with vehicle speed, the maximum value being from standing start and depends on vehicle capacities. Typical values are given in table 4.6 for level roads (Ref. 88).

ii. Normal Acceleration : These acceleration rates take place when drivers are not influenced to accelerate rapidly, e.g. passing on dual carriageways. In table 4.7 the values for a typical passenger car are shown (Ref. 88).

iii. Normal Deceleration : Deceleration of motor vehicles occurs automatically when the accelerator pedal is released because of the retarding effect of motion resistance. For controlled or normal deceleration, vehicle brakes are used to restrain the vehicle motion. Typical values are also given in Table 4.7. (Ref. 88).

iv. Maximum Deceleration : It occurs when a vehicle comes to an emergency stop, or slows down rapidly to avoid collision. It has been observed that a deceleration rate of 2.5 m/s^2 is 'comfortable', a value of 3.4 m/s^2 is 'undesirable but not alarming', and a value of 4.2 m/s^2 is 'emergency stop'. (Ref. 94).

In the model the acceleration and deceleration rates are calculated according to the free and restrained, car following, behaviour (Section 4.5). The value of

$$DMAX = 4.2 \ m/s^2$$

is taken as the maximum deceleration.

TABLE 4.3

Design Vehicle Length (AASHO)

Vehicle	Length (ft-m)
Passenger Car	19 (5.791)
Single Truck	30 (9.144)
Semitrailer WB-40	50 (15.24)
Semitrailer WB-50	55 (17.764)
Semitrailer WB-60	65 (19.812)

TABLE 4.4

Measured Vehicle Lengths. (m.)

	Urban Site (Ref. 91)				M4 Data-mean values (Ref.86)	
Vehicle type	Mean	s. deviation	Minimum	Maximum	Nearside Lane	Offside Lane
Car	3.740	0.510	2.40	5.00	3.90	4.00
Light Van	3.950	0.530	2.80	5.20	4.30	4.20
Lorry	7.620	1.920	4.30	12.00	7.20	6.80
Articulated Lorry	-	-	-	-	12.30	9.10

TABLE 4.5

Parameters for the model (m.)

Vehicle	Mean	s. Deviation
Car	4.00	0.60
H.G.V.	11.00	2.40

TABLE 4.6

Maximum Acceleration Rates (m/s^2)

Vehicle Type	Speed (kph)					
	Stand to 24	Stand to 48	48	64	80	97
Car	3.58	2.222	2.083	1.694	1.25	0.861
H.G.V.	0.889	0.444	0.444	0.250	0.083	-

TABLE 4.7

Normal Acceleration and Deceleration Rates (m/s^2) for Passenger Cars

Speed Change (kph)	Acceleration (m/s^2)	Deceleration (m/s^2)
0 - 24	1.472	2.361
0 - 48	1.472	2.028
48 - 64	1.472	1.472
64 - 80	1.167	1.472
80 - 97	0.889	1.472
97 - 113	0.583	1.472

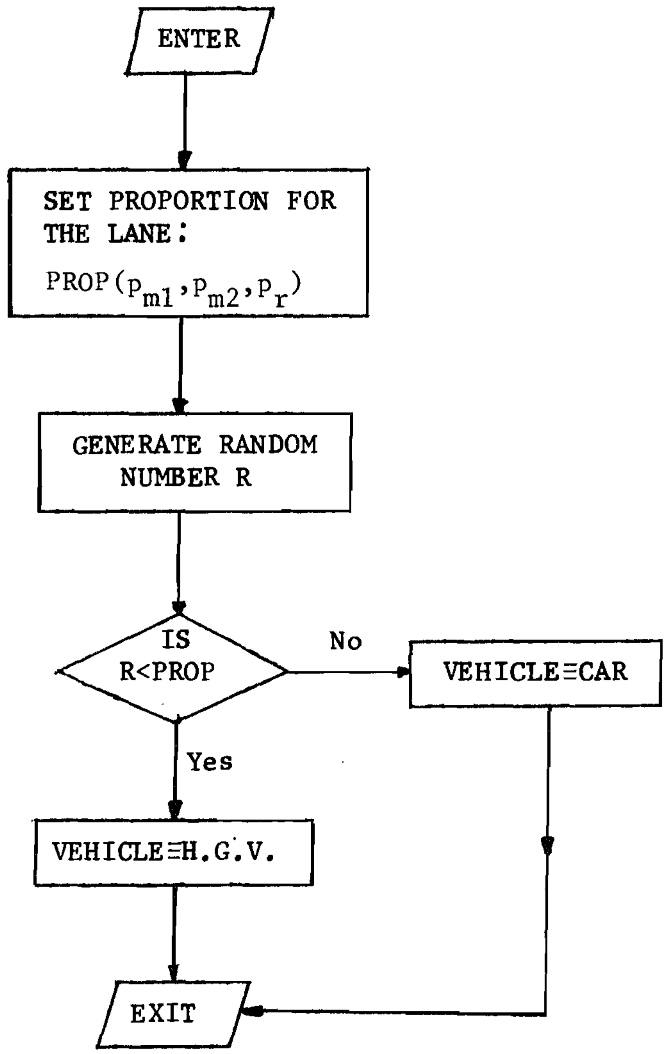


Figure 4.2: Determination of Vehicle Type.

4.4 Traffic Arrivals

Headways, or time spacings between successive vehicles, are one of the fundamental characteristics essential to the description of the traffic stream. The relations between flow, density, road capacities, delays and the effect of queueing on velocity distributions of free moving vehicles depend on the basic assumptions about inter-vehicle spacings (Ref. 72). On the other hand, one of the basic requisites of simulation is that traffic should be introduced to the simulated system, and traffic arrivals are regulated by the sequence of time intervals between successive vehicles in the stream. Due to the above stated reasons, the mathematical description of the arrival pattern, in terms of headway distribution has been the concern of many investigators and various models have been proposed in order to predict the headway distributions under different traffic conditions.

4.4.1 Random vs. Non-random Arrivals

It has been observed that under certain conditions vehicles arrive randomly at a point on a road, and since 1936, Adams (Ref. 82) has shown that freely flowing traffic corresponds to a random series of events, i.e. the arrival of a vehicle is independent of the arrival of any other vehicle and equal time intervals are equally likely to contain equal number of arrivals. Under these circumstances, vehicles arrive according to a Poisson process and headways are distributed according to the negative exponential distribution, stated as follows:

$$P(h \geqslant t) = e^{-\lambda t}$$

or

$$F(x) = P(h < t) = 1 - e^{-\lambda t}$$

where :

$P(h \geqslant t)$: probability of a headway h greater or equal to t.

λ : arrival rate, $\frac{V}{3600}$ (veh/sec)

V : Flow (veh/h)

The negative exponential distribution has been widely used in traffic studies because it is convenient to be analytically manipulated, numerical computations associated with it are relatively simple, and random variates can be easily generated for simulation. On the other hand, the application of the negative exponential distribution has many shortcomings. It predicts that headways in the range of values very near to zero will occur more frequently than those in any other range, but because vehicles have finite lengths, a minimum headway exists in the traffic stream, depending on the length of lead vehicle, to the minimum intervehicular spacing demanded by the trailing vehicle and to the speed and acceleration of the trailing vehicle (Ref. 82).

As traffic flow increases, vehicles interact and independence does not exist, as it has been shown by the car-following theory. There is a tendency for vehicles to operate in platoons and two broad categories of drivers can descriptively be defined as : those who can freely travel (leaders), and those who are restrained from reaching their desired speed (followers) (Ref. 71). At very high flows an element of regularity of arrival times has been observed (Ref. 73), and the assumption of random arrivals does not explain the phenomenon of 'saturation flow' or 'capacity'.

In order for these phenomena to be taken into account a number of theoretical headway models have been developed, consisting either of a single statistical distribution or of a mixture of two types.

4.4.2 Single Headway Models

a. The Shifted Exponential Distribution : This model has been derived from the exponential distribution with the introduction of a minimum headway τ , to reflect the existence of a real minimum time interval (Ref. 39). The mathematical expression is :

$$P(h \geqslant t) = e^{-(t - \tau)/(T - \tau)}$$

where :

τ : min. headway (sec)

T : mean headway = 3600/V (sec)

This distribution has given close agreement to motorway headway data for lane volumes up to 400 veh/h (Ref. 20), but it failed to give a satisfactory fit on high volumes (Ref. 38).

<u>b : The Pearson type III Distribution</u> : The observation of vehicular headways normally results in a distribution with small frequencies at minimum headways, increases to a maximum and then decreases exponentially for large headway values. A distribution with this general shape is the Pearson type III or Gamma, stated as :

$$f(t) = \frac{b^a}{\Gamma(a)} \cdot (t - \tau)^{a - 1} e^{- b(t - \tau)} \qquad \tau < t < \infty$$

where :

$$\Gamma(a) = \int_0^\infty Z^{a - 1} e^{- z} dZ \qquad \text{(Gamma function)}$$

τ : min. observed headway (sec)

The parameters of the theoretical distribution may be obtained from the sample estimates $\bar{t}$, S^2 as follows : (Ref. 43)

$$\bar{t} = \frac{a}{b} \qquad s^2 = \frac{a}{b^2}$$

This distribution has given a satisfactory fit to empirical data especially at large and intermediate headway values (Ref. 71).

As can be seen, the negative exponential and shifted negative exponential distributions are particular cases of the Pearson type III, if we set:

$$a = 1$$

If the value of a is integer, then we have the simplified form :

$$f(t) = \frac{b^a}{(a-1)!} t^{a-1} e^{-bt}$$

referred to as the Erlang distribution. The value of a can be assumed as an index of non randomness. The advantage of the Erlang distribution lies on the fact that it can describe different operating conditions from complete randomness ($a = 1$) to complete uniformity ($a = \infty$) (Ref. 46).

The corresponding counting distribution has been derived by Haight (Ref. 73) referred to as generalized Poisson.

c. The Lognormal Distribution : This is the distribution of a variable, whose logarithm obeys the normal law of probability (Ref. 76). The form of this distribution with parameters m, s^2 can be stated as follows (Ref. 77) :

$$f(t) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(\ln t-\mu)^2}{2\sigma^2}}$$

where :

$$\mu = \ln(m) - \sigma^2/2$$

$$\sigma^2 = \ln(s^2/m^2 + 1)$$

Greenberg (Ref. 78) has shown that this distribution is related to car-following theory and he had applied it to both freeway and tunnel headway data, with very close approximation, except for the extremes, in high flow conditions. Tolle (Ref. 77) has tested the distribution on a separate data base and although the Kolmorogov-Smirnof test produced statistically good fits, the chi-square test did not at higher volumes.

4.4.3 Composite Headway Models

The composite, or mixed headway distributions attempt to consider the platooning or bunching of vehicles and to reproduce the interactions in the traffic stream. They have as a general form the expression :

$$f(t) = \phi g(t) + (1 - \phi)h(t)$$

in which ϕ is the proportion of restrained drivers, and $h(t)$ and $g(t)$ the distributions for free and restrained headways respectively.

a. The Double Exponential Distribution : This distribution proposed by Schuhl (Ref. 85) was the first model which classified the vehicles into following and non-following, having the mathematical expression :

$$P(h \geq t) = \phi e^{\frac{-(t-\tau)}{\bar{t}_1-\tau}} + (1 - \phi)e^{-t/\bar{t}_2}$$

where :

$\bar{t}_1$: mean headway between restrained vehicles
$\bar{t}_2$: mean headway between unrestrained vehicles
τ : minimum headway

This distribution is suitable for highways where passing is restricted, such as two-lane two-way roads. The parameters were firstly calculated by Kell and by Grecco and Sword (Ref. 40, 75). It was found that the distribution is acceptable for low and medium flows (up to 700 veh/h) but does not seem to give a good fit at high flow rates.

b. The Hyperlang Model : This distribution has been proposed by Dawson and Chimini (Ref. 89) as a generalised headway model, formed as a linear combination of Erlang and negative exponential distributions, having as mathematical expression :

$$P(h \geq t) = (1-\phi)e^{-\frac{t-\delta 1}{\gamma 1-\delta_1}} + \phi e^{-K\frac{t-\delta 2}{\gamma 2-\delta 2}} \sum_{X=0}^{K-1} \left[K \frac{t-\delta 2}{\gamma 2-\delta 2}\right]^{x} \frac{1}{x!}$$

where :

γ_1, δ_1 : mean and minimum headway for free vehicles

γ_2, δ_2 : mean and minimum headway for restrained vehicles

K : an index denoting the degree of non-randomness in the distribution of restrained headways.

As can be seen the double exponential, Erlang and exponential distributions are special cases of the described model. The agreement with headway data up to high volume was very good, the only disadvantage being the number of parameters involved.

c. Moving Queue Models : These models have been derived assuming that the traffic stream forms a process comprising random bunches and gaps. The modification of the Poisson arrival process by the travelling queue technique is similar to that imposed by the classical queueing system with a single server (Ref. 84). The distribution of following headways on the road is analogous to the service time distribution in the queueing system. Each non-following headway t is the sum of a following headway x , drawn from $g(x)$ and a gap $(t-x)$ which is exponentially distributed such as :

$$h(t) = \int_0^t g(x)\lambda e^{-\lambda(t-x)}dx$$

This model was first proposed by Tanner (Ref. 80), assuming a constant following headway τ , and it is referred to as constant headway queueing model, having the form :

$$\begin{aligned} P(h \leq t) &= 0 \qquad t < \tau \\ &= 1 - (1-\phi)e^{-\lambda(t-\tau)} \qquad t > \tau \end{aligned}$$

A generalisation of the Tanner's model is the assumption that following headways form a distribution $g(t)$ instead of having a constant value. The normal, gamma and lognormal distribution were proposed to represent the $g(t)$ distribution and the best fit for a wide range of flows was achieved with the log-normal distribution (Ref. 84).

d. The Semi-Poisson Model : This headway model was proposed by Buckley (Ref. 80). The underlying assumption is that a 'zone of emptiness' exists behind each vehicle which is never entered by the following vehicle. The zone of emptiness constitutes the following headway distribution $g(t)$ and the non-following distribution $h(t)$ is exponential, modified such as to include only headways greater than a random variable sampled from $g(t)$. It follows that :

$$h(t) = \lambda e^{-\lambda t} \int_o^t g(z)dz/_B$$

where B is the probability that a random variate from $\lambda e^{-\lambda t}$ is greater than a random variate from $g(t)$.

$$B = \int_o^\infty \lambda e^{-\lambda x} \int_o^x g(z)\ dzdx$$

It was shown (Ref. 71) that if $g(t)$ is normal with parameters Θ, σ^2 then a non-following headway can be obtained by summing two random variates, one from an exponential function with parameter λ and one from a normal function with parameters : ε, σ^2 where :

$$\varepsilon = \Theta - \sigma^2\lambda$$

The model was tested against a data base of freeway lane headways, for a wide range of flows, with gamma and normal distributions of following headways. The best fit at high flows was obtained with the gamma distribution. The disadvantage of the model is the estimation of parameters; no physically sensible solutions could be found for the gamma distribution for flows less than 1260 veh/h and on the other hand the parameters for the normal distribution were not found consistent with the traffic stream, expressing a discontinuity at a flow of 900 veh./h, especially in the proportion of following vehicles.

4.4.4 Generation of Traffic Arrivals

In this study, it is necessary to generate vehicle arrivals at each lane of the motorway section and the ramp. The selected headway model should adequately describe the arrival pattern for a range of traffic conditions and on the other hand its parameters to be easily evaluated.

a. Headway Model for Motorway Lanes : Observations and measurements done during the study have shown that up to lane flows of 750 veh/h the shifted negative exponential distribution can fit reasonably to the observed data, whereas at the higher flow levels a more elaborate model is needed. Accordingly the simulation model allows for headway generation based on the input flow levels. For the high flow situation the generalized queueing model with lognormal distribution of following headways $g(x)$ has been adopted, as it has given the best fit to empirical data (Ref. 84). The parameters of the distribution $g(x)$, (α, σ) assumed to remain constant over the flow range, and typical values are given by Branston (Ref. 84).

$\alpha = 1.60$ sec $\quad \sigma = 0.40$ sec

$\alpha = 1.30$ sec $\quad \sigma = 0.40$ sec

for the slow and fast motorway lanes respectively. These assumptions are in agreement with the work of Wasielewski (Ref. 81), who found values of 1.32 sec and 0.52 for the parameters of the followers distribution independent of the flow rate. The mean interbunch gap $1/\lambda$ and the proportion ϕ of the following vehicles are given by the expressions :

$$\lambda = \lambda^* - 0.5\lambda^{*1.5} \tag{4.3}$$

$$\phi = \rho - 0.5(\rho-1)\lambda^{*0.5} \tag{4.4}$$

where :

λ^* : flow rate

ρ = traffic intensity = $\dfrac{\alpha}{1/\lambda^*}$

In the simulation model the parameters of the distribution for each lane are calculated according to the equations (4.3), (4.4) assuming that the offside lanes (lane 2 and lane 3) of the motorway section have the same values for the mean following headway. A following headway is randomly generated according to the lognormal distribution.

A random number R is then generated and compared with the proportion of following vehicles ϕ . If R is found to be equal or smaller to ϕ , the next arrival is a follower, otherwise a random variate is generated according to the negative exponential distribution with parameter λ and added to the following headway to obtain a non-follower headway. The arrival of a vehicle is computed as the arrival time of the previous vehicle plus the generated headway. In Fig. 4.3 a flow-chart is given for the generation of arrivals according to the above procedure, and in Fig. 4.4, the theoretical and simulated headway distribution for a motorway lane, is presented.

b. Headway Model for the Ramp : for the generation of vehicle arrivals on the slip road, a composite model seems to be most appropriate because the geometry, one lane link, does not allow for overtaking, and on the other hand the interchange configuration, e.g. roundabouts or signals at diamond interchanges, affects the randomness in arrivals. The hyperlang model has been adopted as the headway distribution of ramp-vehicles and the parameters are given in table 4.8, based on Ref. 82.

The generation process is as follows : firstly a random number R is generated and compared with the proportion of restrained vehicles α_2 . If it is smaller then the headway is generated according to the Erlang distribution. If it is found greater then the headway is generated as a random variate from a negative exponential distribution. In Fig. 4.5, the procedure is shown in the flow chart and also the simulated and theoretically computed probability distribution for a flow rate of 957 veh/h is given in Fig. 4.6.

TABLE 4.8

Typical values of the parameters for the Hyperlang Distribution (Ref. 82)

FLOW RATE	α_1	γ_1	δ_1	k	α_2	γ_2	δ_2
158	0.86	22.34	0.69	1	0.14	2.88	1.65
251	0.70	22.09	0.35	1	0.30	2.90	1.44
353	0.61	16.75	0.74	1	0.39	3.35	1.12
450	0.64	9.81	0.61	2	0.36	2.81	0.70
547	0.56	11.05	0.70	2	0.44	2.73	0.90
651	0.43	11.06	0.79	2	0.57	2.81	0.71
746	0.40	8.35	0.88	3	0.60	2.92	0.57
836	0.20	11.57	0.95	3	0.80	3.23	0.52
957	0.53	4.58	1.06	6	0.47	2.71	0.72

where:

α_1 : proportion of free vehicles

α_2 : proportion of restrained vehicles

γ_1 : mean headway for free vehicles

δ_1 : minimum headway for free vehicles

γ_2 : mean headway for restrained vehicles

δ_2 : minimum headway for restrained vehicles

k : index, denoting the degree of non-randomness in the restrained headway distribution

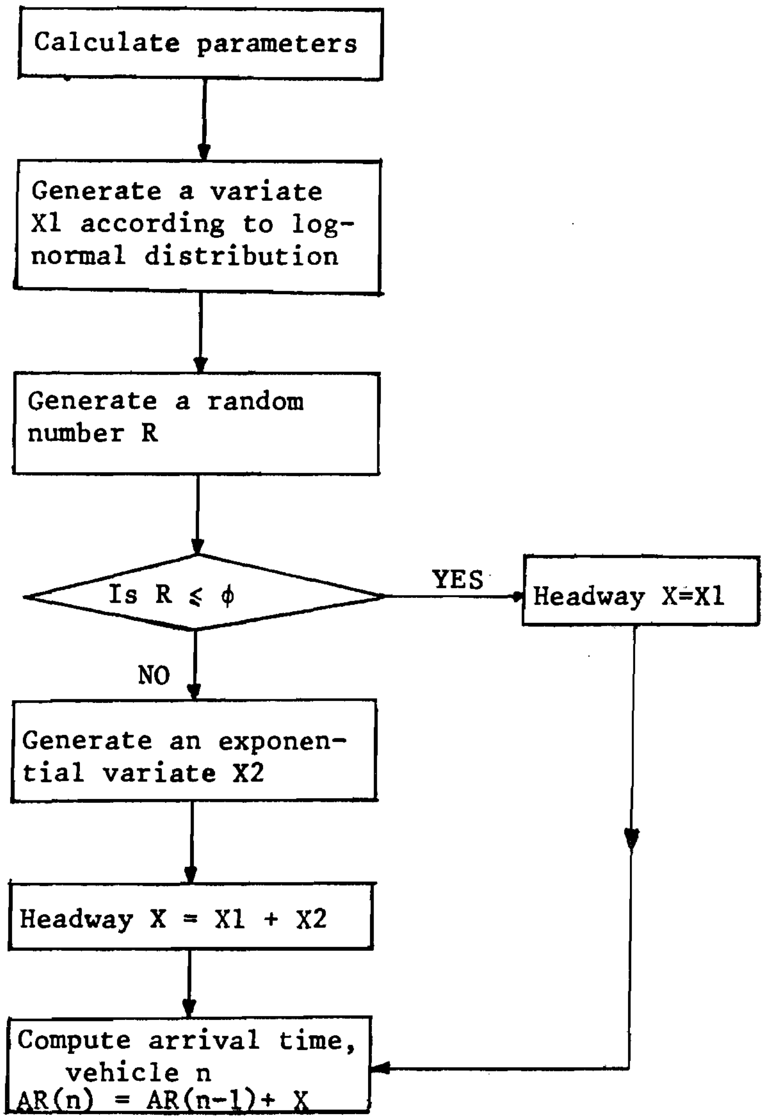


Figure 4.3: Generation of Headways according to the Queueing Model.

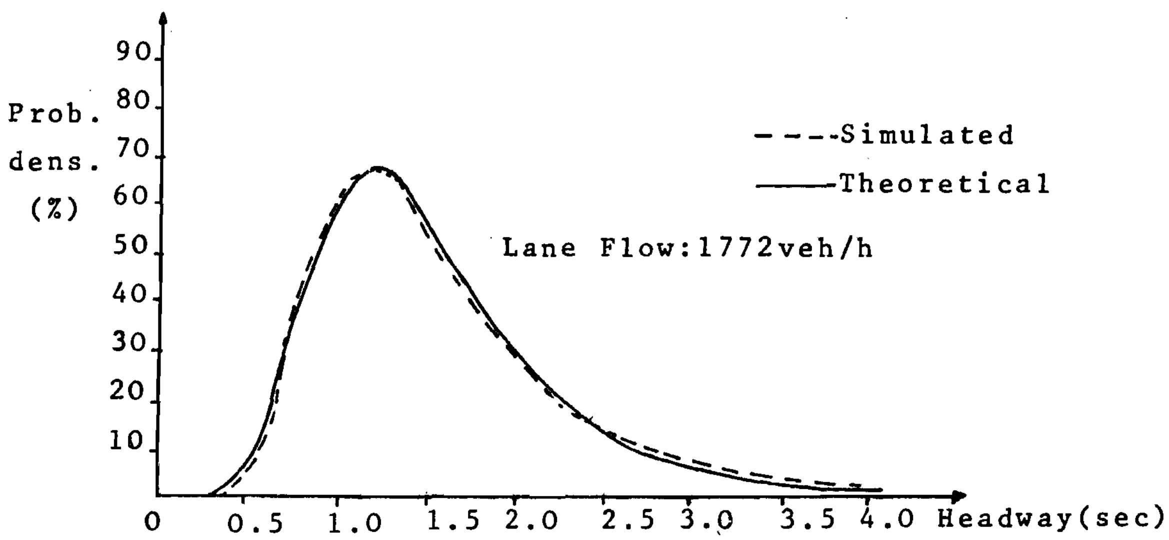


Figure 4.4: Headway distribution on Motorway Lane,based on Queueing Model.

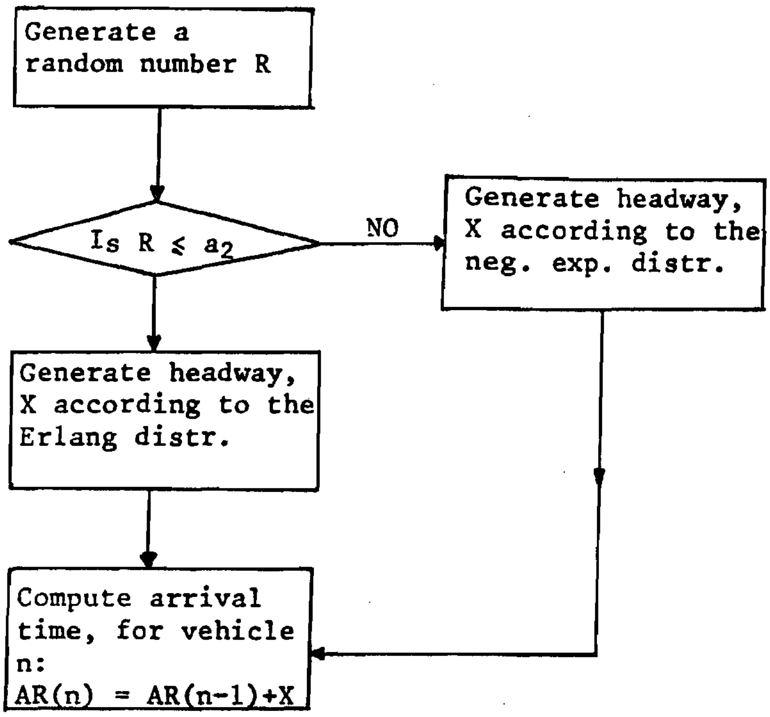


Figure 4.5: Generation of Headways for Ramp Vehicles.

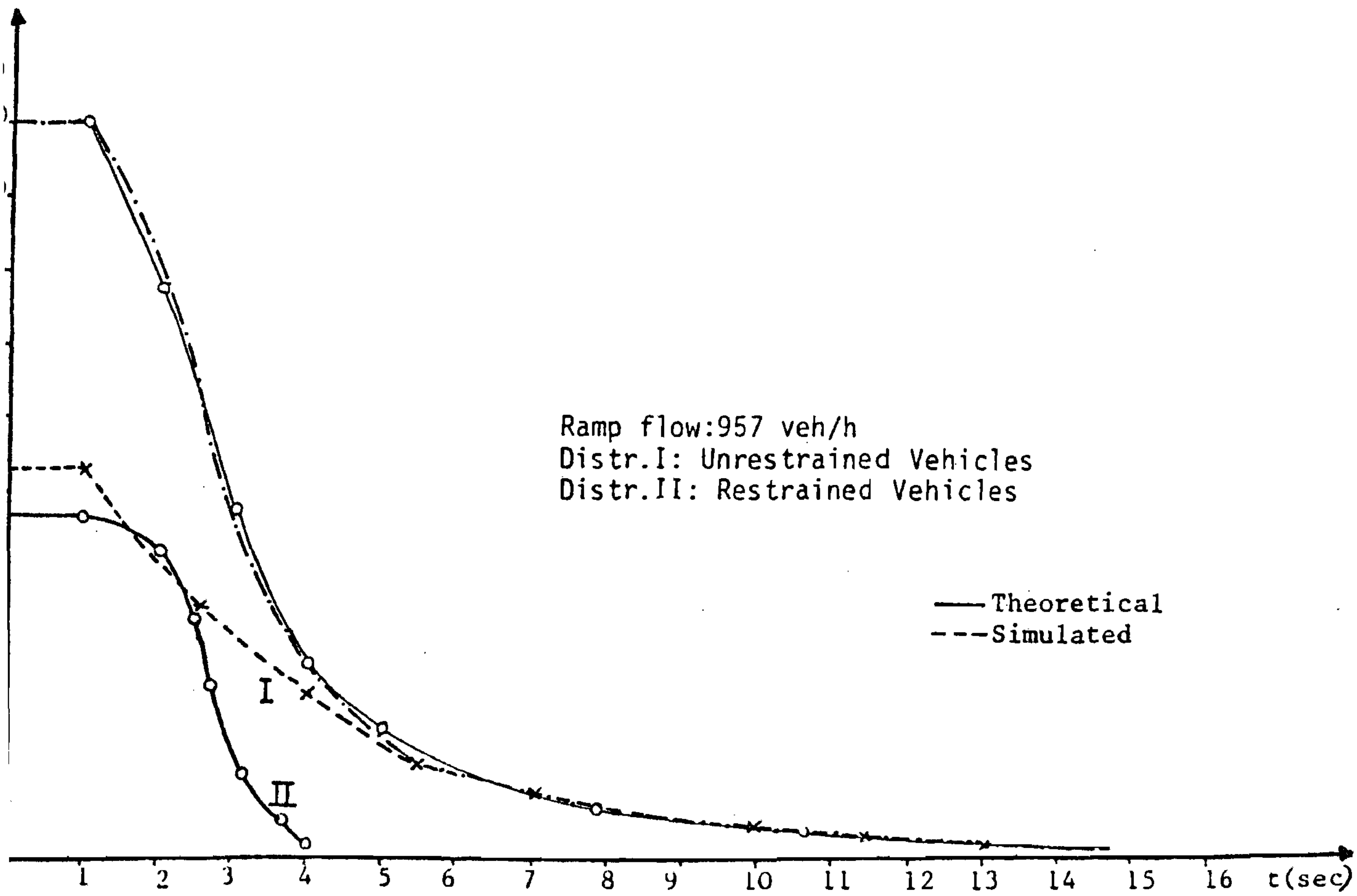


Figure 4.6: Ramp Headways according to the Hyperlang Model.

4.5 Motorway Traffic Stream

The modelling of the traffic behaviour at a junction requires the adequate description of the movement and interactions of vehicles of the intersecting traffic streams. Basically, there are two approaches in examining the traffic phenomena : the macroscopic approach, which deals with average quantities such as flow, speed and concentration, or density, of vehicles and attempts to establish relationships between them, and the microscopic approach which is involved with the behaviour of each individual vehicle. In this study the model, because it is intended to give detailed information for each vehicle, is primarily concerned with the second approach, but the macroscopic theory is also considered because it gives parameters estimation and it has been shown that under certain conditions, the two approaches are interrelated.

4.5.1 Macroscopic Theory

The macroscopic approach attempts to describe the traffic stream based on its average parameters. The relationship between speed, flow and density is fundamental in the traffic flow theory and can be stated as : (Ref. 109)

$$Q = K \cdot U \qquad (4.5)$$

where :

Q = mean flow rate, number of vehicles per unit time
K = mean density, number of vehicles per unit road length
U = space mean speed, mean of vehicle speeds travelling over a length of road and weighted according to the time spent travelling that length.

From the equation (4.5) the following terms are defined as shown in the table 4.9 below :

TABLE 4.9

Parameters of Traffic Stream

Parameter	Operating Conditions	Derivation
K_M	Optimum Density	$\frac{dQ}{dU} = 0$
Q_M	Maximum Flow	$Q_M = U_M K_M$
U_M	Speed at Q_M = characteristic speed	$\frac{dQ}{dK} = 0$
K_j	Maximum Density $U = Q = 0$	
U_f	Free Speed $K = Q = 0$	

The relationship between flow and density is called the fundamental diagram of traffic (Ref. 110) and several studies have been made to express it quantitatively. One of the early models assumed that the speed of the traffic stream is a linear function of the density :

$$U = U_f \left(1 - \frac{K}{K_j}\right)$$

which leads to a parabola for the q-K curve :

$$q = U_f \cdot K \left(1 - \frac{K}{K_j}\right)$$

In order for the phenomena of high density traffic to be explained and quantitatively expressed, the Hydrodynamic analogy was used, i.e. the traffic is analogous to the continuous fluid. The theory was used by Lighthill and Whitham (Ref. 123) in the formulation of shock wave theory, particularly useful in describing traffic at bottlenecks. Greenberg (Ref. 112) assumed the equation of motion for one dimensional fluid :

$$\frac{du}{dt} = -\frac{a^2}{K}\frac{\partial K}{\partial x}$$

where :

$u = u(x, t)$ speed as a function of location and time and the continuity equation :

$$\frac{\partial k}{\partial t} + \frac{\partial q}{\partial x} = 0$$

Using the assumption that the speed of the traffic stream at a point is only a function of the density, i.e.

$$u = u(k)$$

he obtained the equations :

$$u = u_m \ln\left(\frac{k_j}{k}\right)$$

$$q = u_m k \ln\left(\frac{k_j}{k}\right)$$

The Greenberg's model gave a realistic fit to empirical data from tunnels especially at the high flow regime, whereas it fails at low densities predicting infinite speed at zero density. Other macroscopic models are particularly suitable for non congested traffic (Ref. 114).

Because traffic changes from very light to heavy flows it is very difficult to find an unique macroscopic model to provide accurate descriptions for all the traffic conditions, it has also been observed that as flow builds up a discontinuity occurs and two regimes can be considered : free flow and congested flow (Ref. 97).

Macroscopic theory provides a sound basis for understanding and predicting the behaviour of groups of vehicles, especially in a qualitative way. Some of the models are applicable to real traffic situations. A weakness of macroscopic theory is that it does not incorporate driver, vehicle, and roadway parameters in an explicit way. These aspects attempts to cover the microscopic approach, which has been adopted in this study.

4.5.2 Microscopic Theory

The microscopic theory examines the behaviour of single vehicles based on the assumption that every driver who finds himself in a single lane traffic situation is assumed to react mainly to a stimulus from his immediate environment according to the relationship :

$$(\text{Response})_{t+T} = \text{Sensitivity} \times (\text{stimulus})_t \qquad (4.6)$$

where T reaction time lag, the combined effect of the sluggishness of the driver and his car. As response the acceleration of the vehicle was considered and the stimulus was assumed to be a function of the vehicle and its neighbour positions and speeds. The relative speed was found to be the strongest stimulus and the expression (4.6) becomes (Ref. 100) :

$$\frac{d^2xn(t+T)}{dx} = \lambda \left[\frac{dx_{n-1}(t)}{dx} - \frac{dx_n(t)}{dx}\right]$$

referred to as the 'car-following model',

where:

λ = sensitivity coefficient

x_{n-1} = position of (n-1) vehicle, at time t

x_n = position of n vehicle, at time t

T = reaction time

Firstly, a constant sensitivity λ was proposed and the linear car-following model was obtained (Ref. 79). In further investigations it was shown that sensitivity λ is a function of vehicle speed, and the spacing between the vehicles. A generalized expression was given (Ref. 100):

$$\lambda = \frac{\alpha \left[\frac{dx_n(t+T)}{dx}\right]^{\ell}}{\left[x_{n-1}(t) - x_n(t)\right]^m}$$

where :

m, ℓ : constants

Various proposed car-following models can be derived from different combinations of values m, ℓ , e.g.

$\ell = 0$ $m = 1$ Inverse spacing model
$\ell = 1$ $m = 2$ Edie's model (Ref. 114)

Ceder (Ref. 98), using an extensive data base evaluated the car-following models for different values of the parameters ℓ, m against the requirements of maximum flow, free speed and jam concentration. None of the single models can represent the data sets at near capacity conditions indicating that there are two flow regimes free flow and congested traffic, as it was also shown macroscopically. The application of the two flow-regime models has shown that the inverse spacing car-following model is appropriate for the high flow situation and for the free flow traffic there is a tendency for values :

$$m = 3 \qquad \ell = 0$$

for the freeway data sets.

In the model, the discontinuity of the traffic stream is taken into account by modelling separately the 'free' and 'restrained' vehicles. The determination of the free or restrained vehicles is based on their relative distance; assuming a mean following headway T_F , then the 'catching up' distance can be estimated :

$$\text{DISTA} = T_F \cdot \text{Speed}(n)$$

The unrestrained vehicles proceed according to the equation :

$$\frac{d^2 xn(t+T)}{dt} = AC\left(DVEL(n) - \frac{dxn(t)}{dt}\right) \tag{4.7}$$

where :

DVEL(n) : desired velocity of vehicle n
AC : proportionality coefficient

For the following or restrained vehicles, the inverse spacing car-following model has been adopted to represent their behaviour having the mathematical expression :

$$\frac{d^2xn(t+T)}{dt} = \alpha_o \frac{1}{x_{n-1}(t) - xn(t)} \left[\frac{d_{xn-1}(t)}{dt} - \frac{dx_n(t)}{dt}\right] \qquad (4.8)$$

Assuming steady state conditions, i.e. ignoring the time lag T the equation (4.8) can be written :

$$\frac{du}{dt} = \left(\frac{\alpha_o}{s}\right) \frac{ds}{dt}$$

Integration of the equation yields:

$$u = \alpha_o \ln s + c_1$$

where $s = s_j$ $u = 0$ thus : $c_1 = - \alpha_o \ln s_j$

Finally we obtain :

$$u = \alpha_o \ln\left(\frac{s}{s_j}\right) = \alpha_o \ln\left(\frac{k_j}{k}\right) \qquad (4.9)$$

which is the Greenberg's model derived from the hydrodynamic analogy and :

$$\alpha_o = U_M = \text{characteristic speed}$$

The jam spacing s_j is given

$$s_j = \frac{Q_M}{e^{U_M}}$$

In the model, at each time increment Δt the accelerations or decelerations of vehicles after time T are calculated based on the equations (4,7), (4.8). The calculated accel. or decel. rates should not exceed the minimum or maximum values being :

$$DMAX = -\ 4.2\ m/s^2 \quad \text{for decelerations}$$

$$DAGL = AC\left(DVEL(n) - \frac{dx(n)}{dt}\right) \quad \text{for accelerations}$$

A flow chart for the procedure is given in Fig. 4.7. The updating of vehicle positions at each time movement Δt is done assuming that the acceleration is uniform and equal to the average of the accelerations at the start and end of the time slice Δt. Thus the following equations are used for the calculation of vehicle position, velocity and acceleration at time t .

$$\frac{dx_n(t)}{dt} = \frac{dx_n(t-\Delta t)}{dt} + 0.5\left[\frac{d^2x_n(t-\Delta t)}{dt} + \frac{d^2x_n(t)}{dt}\right]\Delta t \qquad (4.10)$$

$$x_n(t) = x_n(t-\Delta t) + \frac{dx_n(t-\Delta t)}{dt}\,\Delta t + 0.5\left\{0.5\left[\frac{d^2x_n(t-\Delta t)}{dt} + \frac{d^2x_n(t)}{dt}\right]\right\}\Delta t^2$$

$$x_n(t) = x_n(t-\Delta t) + 0.5\left[\frac{dx_n(t-\Delta t)}{dt} + \frac{dx_n(t)}{dt}\right]\Delta t \qquad (4.11)$$

$$\frac{d^2x_n(t)}{dt} = \frac{d^2x_n(t-T)}{dt}$$

When a vehicle is generated and is unrestrained, it proceeds constantly on the roadway having its desired speed. In case it is a follower its actual speed is calculated from the equation 4.9.

4.5.3 Parameters Estimation

Basic parameters for the modelling of the motorway traffic stream are the free speed of the vehicles and the characteristic speed of the traffic stream. Those can be derived from the speed-flow relationship, which in fact is of great importance for the design and evaluation of the highway facilities. The main factors affecting the situation are traffic composition, road layout, average hilliness and/or bendiness and urban or rural environment. (Ref. 96). For motorways, the speed flow relationship can adequately be represented by straight lines and it has been observed that speed does not fall until flow reaches a high level. In the model the relationship shown in Fig. 4.8 has been adopted as a typical speed-flow, based on the results of the Martin and Voorhees study (Ref.111).

Assuming average geometric conditions the analytical expression becomes :

$$S = 99-5Q$$

where:

S = speed of all vehicles (km/h)

Q = vehicles/h/lane up to 1200 veh/h.

The characteristic speed can be obtained from short duration measurements during peak periods. Based on Ref.105, a typical value has been chosen as :

$$a_o = 50 \text{ kph} = 13.89 \text{ m/sec}$$

Desired speed or target velocity is defined as the cruise speed at which each driver wishes to travel, unimpeded from other vehicles. The desired speed is affected from the geometric and environmental conditions and varies between the motorists.

The estimation of the free speed of a vehicle is directly related with the determination of the position of the vehicle in the traffic stream. (leader or follower). According to the definition given by Duncan (Ref. 96), the average desired speed for all vehicles can be estimated from the speed-flow relationship at a flow :

$$Q = 300 \text{ veh/h/standard lane}$$

The variability of the free speeds of drivers can be formally expressed by a speed distribution, taking also into account that the drivers in the offside lanes have, in general, higher speeds than the shoulder lane motorists. Observations have shown that the free speed distribution is normal (Ref. 103, 106) for the whole traffic stream, and for each lane too.

In the model, it is assumed that vehicles per lane i have a normal distribution of free speeds $N_i(\mu_i, \sigma_i)$ with range of values $(\mu_i - 2\sigma_i, \mu_i + 2\sigma_i)$. The parameters of the distributions are provided as input data.

Typical or default values of the parameters μ_i, σ_i can be derived assuming that the whole traffic stream has a normal distribution of free speeds $N(\mu, \sigma)$ where μ can be taken from speed flow curve according to the free speed definition :

$$\mu = 97.5 \text{ kph } (27.083 \text{ m/sec})$$

and the value of the standard deviation σ is taken as

$$\sigma = 15 \text{ kph} = (4.17 \text{ m/sec})$$

according to the observed results by Burrows (Ref. 113). The values of the μ_i, σ_i can then be assumed as :

Lane	Mean	s.dev.
1	$\mu-\sigma$	$2\sigma/3$
2	μ	$2\sigma/3$
3	$\mu+\sigma$	$2\sigma/3$

In the simulation model, when a vehicle is generated then a desired speed is sampled from the speed distribution according to its lane, and remains a permanent characteristic of the vehicle during the simulation.

4.5.4 The Lane Distribution

The distribution of traffic between lanes on a multi-lane highway is important for the merging problem. The rate of the shoulder-lane flow directly affects the capacity of ramp-entry and the delays of vehicles and it is essential to be adequately predicted. The lane distribution depends on traffic conditions such as total flow, traffic composition, and on geometric factors, number of lanes, site characteristics. In USA empirical studies have been undertaken to predict the lane distribution, especially on proximity with ramp-terminals. The results were in the form of nomographs for the prediction of the inside lane flow on various ramp-terminals (Ref. 22) and regression equations for the lane flow over a length of road (Ref. 22, 34). In Fig. 4.9 a typical lane distribution for 3-lane freeway is shown (Ref. 22). In Britain (Ref. 95) no explicit relationships exist and there is variation between sites, although as total flow Q increases the general pattern is:

(i)	flow in lane 1 > flow in lane 2 > flow in lane 3	$Q < 1000$ veh/h
(ii)	flow in lane 2 > flow in lane 1 > flow in lane 3	$1000 < Q < 3000$
(iii)	flow in lane 2 > flow in lane 3 > flow in lane 1	
(iv)	flow in lane 3 > flow in lane 2 > flow in lane 1	$Q > 3000$ veh/h

In order to provide input flows for each lane in the model, the average values of lane flow were calculated based on the graphs for each site, from Ref. 95. The numerical values are given in table 4.10 and Fig. 4.10 shows the resultant lane distribution.

TABLE 4.10

Average Values of Lane Flows (% total flow).

Total Flow	Lane 1	Lane 2	Lane 3
1000	0.410	0.432	0.158
1500	0.356	0.429	0.215
2000	0.332	0.410	0.258
2500	0.307	0.396	0.297
3000	0.246	0.376	0.378
3500	0.217	0.330	0.453
4000	0.217	0.347	0.436
4500	0.219	0.341	0.440

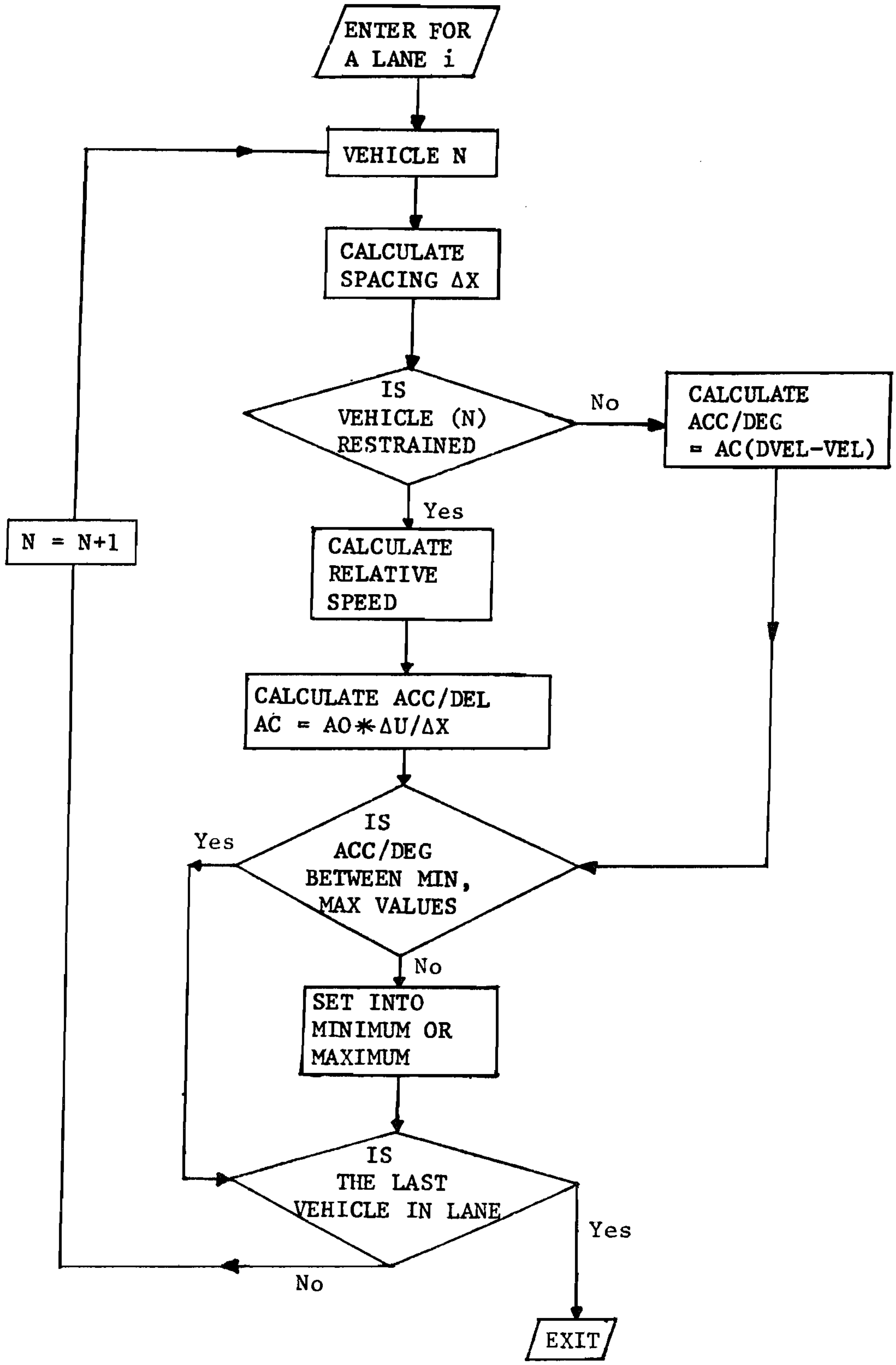


Figure 4.7: Calculation of acceleration deceleration rates, free and restrained behaviour.

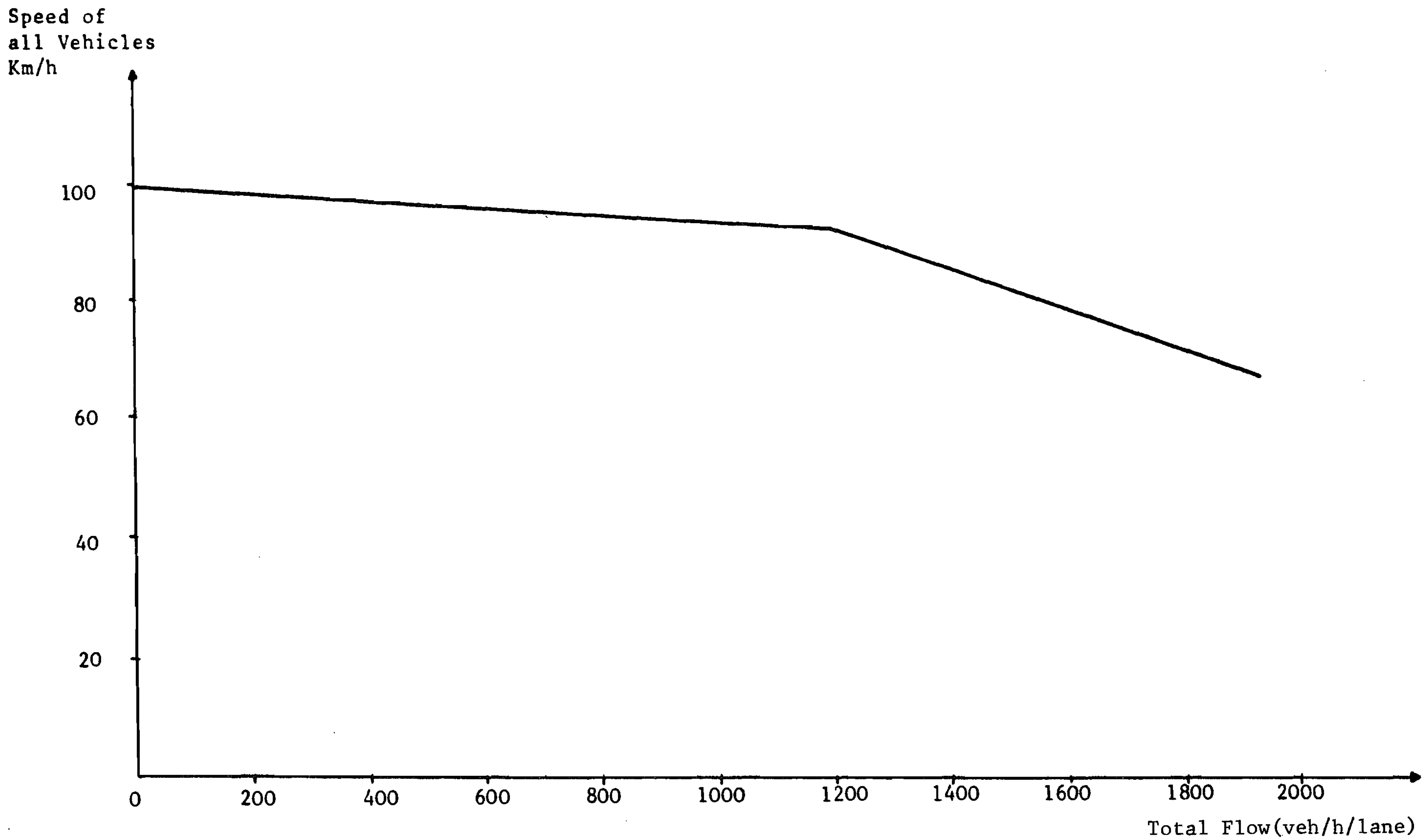


Figure 4.8:Typical Speed-Flow Relationship (Ref.111)

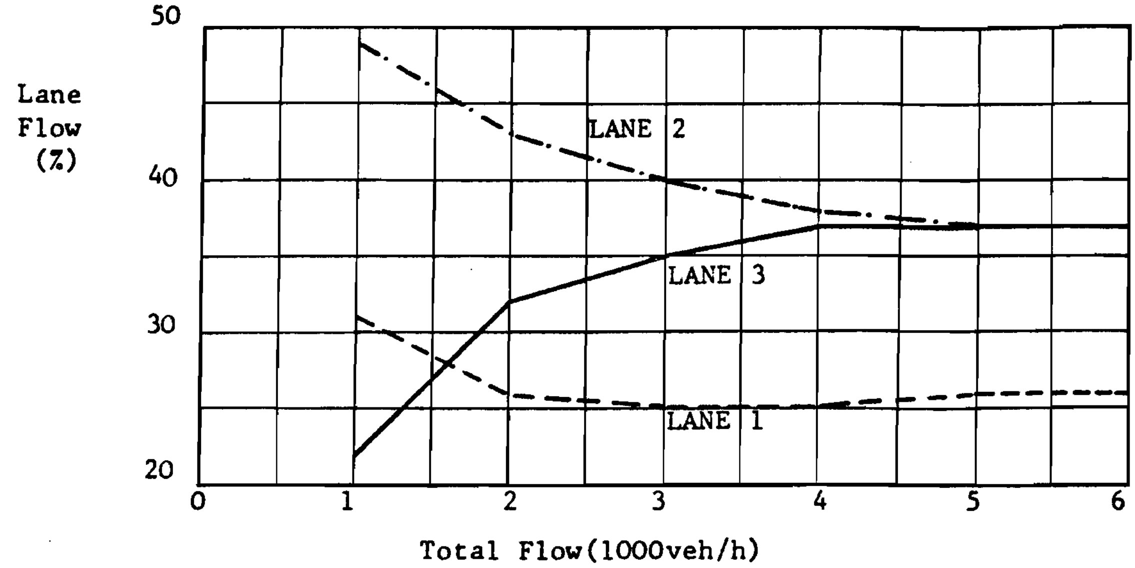


Figure 4.9:Lane Distribution (H.C.Manual-1965)

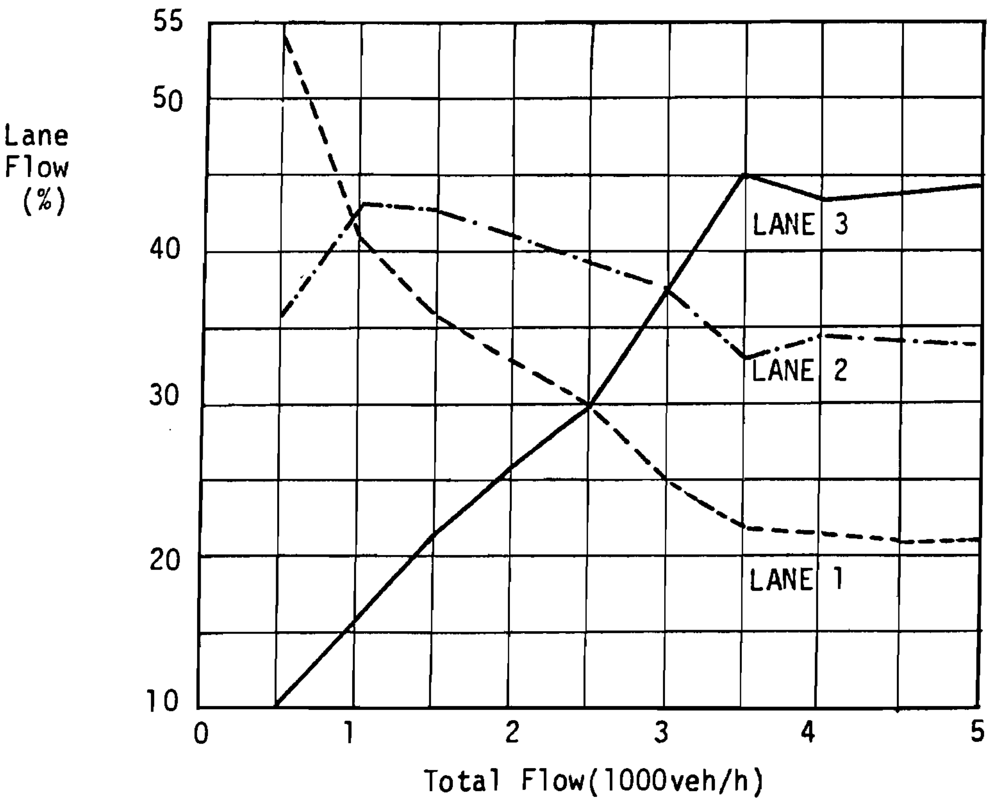


Figure 4.10:Lane Distribution-British Conditions (Ref.95)

4.6 Lane-Changing

Lane changing is defined as the transfer of a vehicle from a lane to an adjacent lane. It is a common and complex phenomenon in multilane highways. There may be a variety of reasons why a driver changes lanes : driver's lane preference, local traffic density, restraint by slower vehicles in front, proximity with interchanges. It is impossible to model lane changing mathematically taking into account all the causes for a lane change. Most of the studies are concerned with macroscopic consideration of the process and models have been developed to predict the average number of lane changes between lanes at a given distance during a fixed time period (Ref. 102).

In a microscopic investigation of traffic, simulation model, we are interested in the actual mechanics of the lane changing manoeuvre expressed in terms of time and distance required to complete the manoeuvre, gap acceptance behaviour, speed and distance of the lane changer relative to its neighbouring vehicles. In Fig. 4.11 those elements are shown, and analysed in the following paragraphs :

i) 'Catching-up distance' : It has been observed that the main reason for lane changing is the speed difference between vehicles. Faster motorists tend to move to a higher speed lane when they are restricted by a slow vehicle. The distance x_1 , i.e. the space headway, between the vehicle attempting to change lane and the vehicle in front varies according to the traffic volume, the speed difference (V_1-V_2) and the lane. Typical values are given below, based on Ref. 103.

Mean x_1 Distance (in ft)

Total Flow	Lane 1	Lane 2	Lane 3
up to 2800 veh/h	224	144	171
up to 4000 veh/h	141	104	103

ii) Gap Acceptance : The lane-changing manoeuvre is subject to the availability of acceptable gap in the adjacent lane. The driver, in order to move to the next lane, must evaluate the lead and lag times

t_1 , t_2 and to proceed if they are greater than its critical value. Limited information exists for minimum acceptable lead and lag times. Field observations have given values of 1 and 3 seconds respectively (Ref. 38) and in USA Worall (Ref. 104) has reported values in the range of 0.3 to 0.70 seconds for lead times and 0.30 to 0.60 secs for lags, measuring at the start of the manoeuvre, whilst the mean time required to complete the lane change was 1.25 seconds for the head movement, from the straight path in lane i to first intercept the dividing lane line, and 1.95 seconds for the tail movement, vehicle to return in the straight path in the new lane j . The gap-acceptance function is very difficult to formulate because it is not possible to identify a rejected gap in this case, and many variables affect the situation; although the critical gap varies between the drivers no adopted distribution seems to exist.

4.6.1 Modelling

In this study, the following assumptions have been made in order to deal with the lane-changing process :

The desire for lane-changing depends solely, if the driver is restricted on his way to travel. If the driver's target velocity is higher than the speed of the vehicle in front and has 'caught up' the slower vehicle, then he attempts to change lane. The 'catching-up' distance T_c corresponds to the mean restrained headway T_F , as defined in the section (4.5.2). Further to the above main assumption, the following considerations have also been taken into account based on observations of various motorway sites :

The vehicle, which changes lane, maintains its permanent characteristics and returns in its original lane, if the conditions are favourable i.e. after overtaking. H.G. vehicles do not move to the right lane (lane 3) in order to overtake; In most of the cases the relative speeds are : (Fig. 4.11)

$$V_1 - V_3 > 0 \qquad V_1 - V_4 < 0$$

The gap acceptance distribution has the form of step function, for both lead and lag times as it is shown in Fig. 4.12. Typical values for t_{ℓ} and t_g are

$$t_{\ell} = 1.00 \text{ sec.}$$

$$t_g = 2.00 \text{ sec.}$$

The above critical times include the transient period of manoeuvre completion, i.e. the vehicle when changes lanes is immediately transferred to the next lane, ignoring the time and distance during the manoeuvre. This simplifying assumption is made to eliminate the difficulty in modelling the vehicles interaction during the transient period, based on the fact that the realism of the model is not affected.

At each time increment one lane change per vehicle is permitted, i.e. movement from lane 1 to lane 3 is not allowed at the same time.

A flow chart for the modelling of the situation is given in Fig. 4.13.

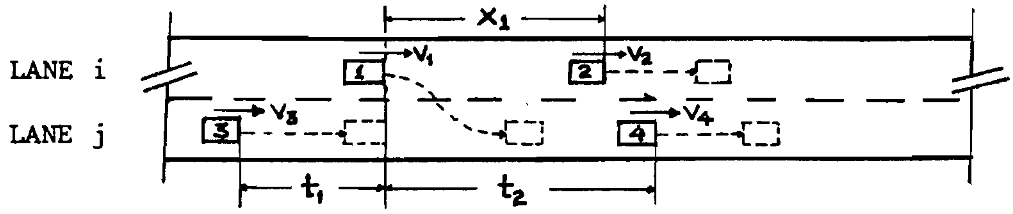


Figure 4.11:Elements of the Lane-Changing manoeuvre.

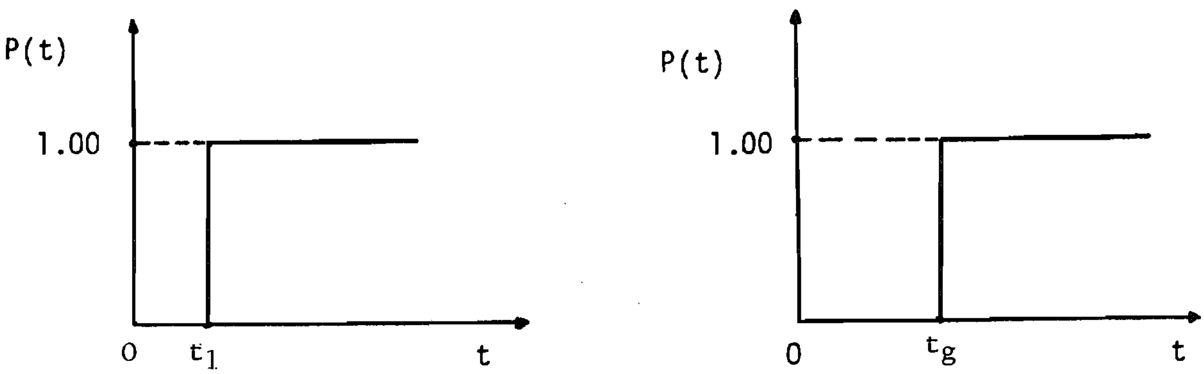


Figure 4.12:Lead and Lag times distributions for Lane Changing.

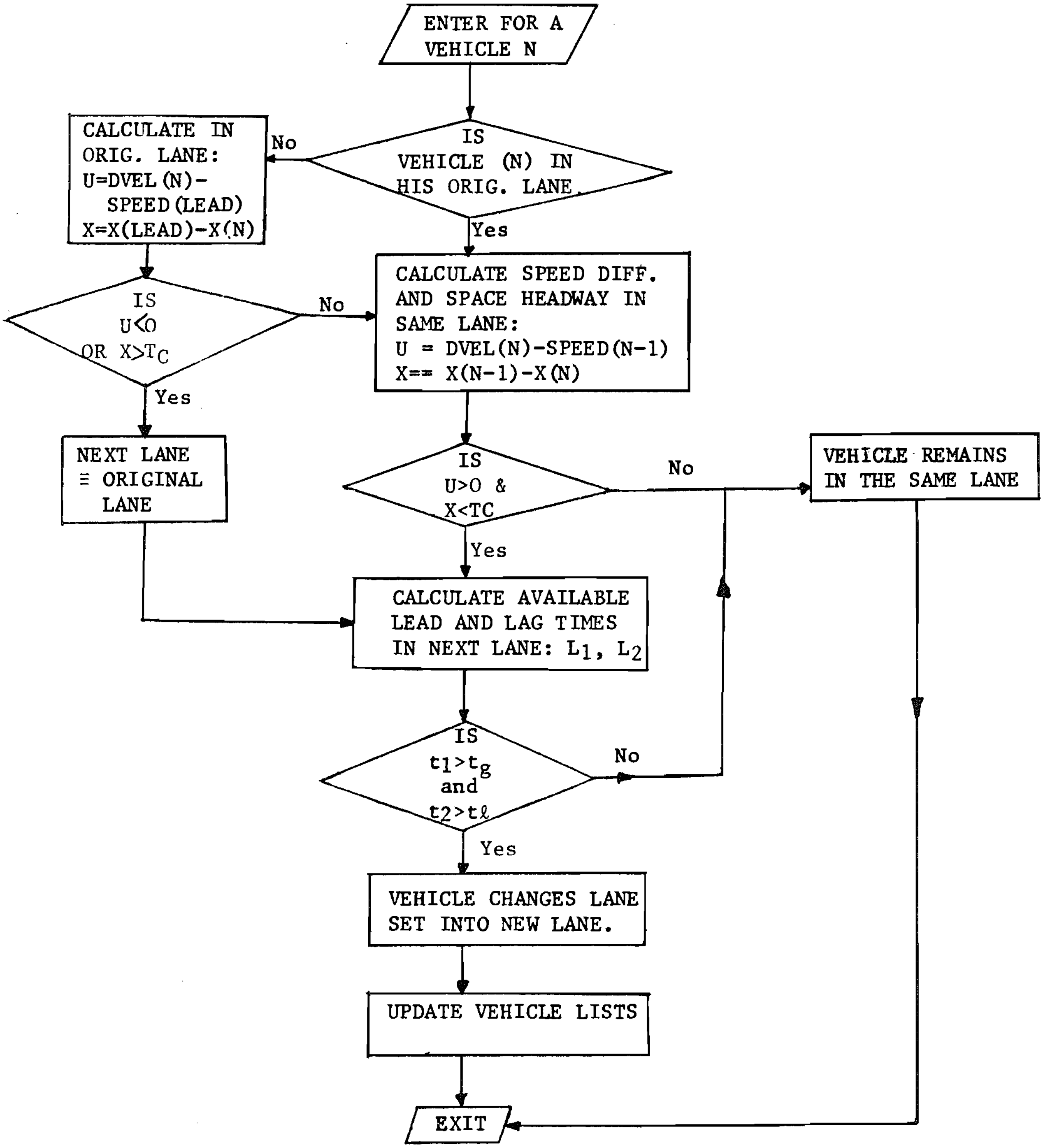


Figure 4.13: Flow Chart for the simulation of Lane-Changing.

4.7 Traffic Stream on the Slip Road

Traffic on the ramp consists of vehicles wishing to merge into the motorway. In this study the modelling of traffic behaviour on the slip road is based on the following assumptions:

i) Traffic Interaction : Because a one-lane slip road has been assumed, lane changing or overtaking is not allowed and a typical 'single lane no passing highway' situation exists. The vehicle's response, acceleration or deceleration is determined based on the equations of free and restrained behaviour described in section 4.5.

The determination of a restrained or follower vehicle is based on the distance headway :

$$DISTR = T_R \cdot speed(veh.)$$

where:

T_R : average mean following headway, having typical values in the range [3.00 - 4.00] sec.

The maximum flow level has been taken as the value of 1200 veh/h, which is suggested as the maximum working flow for the design of single lane links (Ref. 108).

Under free flow conditions the driver's behaviour on the ramp is different than travelling on a single lane, e.g. tunnel. The aim of the slip road driver is to reach the speed of the vehicles on the inside lane of the motorway in which it intends to merge; according to that a 'target' velocity is assigned to each slip road vehicle, with average typical value the mean free speed in the motorway lane 1, for the calculation of free behaviour from equation 4.7.

ii) Speeds of the Slip Road Entry : The speed of a vehicle entering the slip road is mainly affected by the type of the vehicle, traffic conditions and the configuration of the particular interchange e.g.

diamond signalized or roundabout type. In the model the free speeds of the vehicles entering the ramp are assumed to be normally distributed with parameters (μ, σ) . The mean speed of vehicles for roundabout type interchange has been reported (Ref. 101) as 35 mph and the speed of H.G.V. 3-5 mph lower than that of the cars. Accordingly typical values used in the model are:

$$\mu = 35 \text{ mph} = 15.64 \text{ m/sec.} \quad \sigma = 5 \text{ mph} = 2.235 \text{ m/sec.}$$

The sampling interval is taken as $(\mu - 2\sigma, \mu + 2\sigma)$ for cars and $(\mu - 2\sigma, \mu + \sigma)$ for H.G. vehicles.

iii) Acceleration Rates : These are determined according to the described traffic interaction. When a slip road vehicle is generated, and it is restrained, its speed and acceleration is calculated based on the interaction of vehicles; if is free then its speed is the 'free speed' and acceleration is estimated from the free behaviour (eq. 4.7).

The value of the proportionally coefficient AC can be determined from simulation of slip road vehicles and comparisons with actual speed distance profiles. Typical values are in the range:

AC : 0.10 - 0.20

4.8 The Merging Process

The entry from a slip road on the motorway traffic stream is the critical part of the whole model. The modelling of the situation and the underlying assumptions directly affect the measures of effectiveness and the overall model's ability to represent accurately the behaviour of the system under study. In order to proceed with the problem, the following basic considerations are taken into account.

4.8.1 Single Driver Behaviour

We consider a driver M approaching the junction from a slip road at a speed of U_M, depending on his attitudes, the vehicle's characteristics and the traffic and geometric conditions; It is assumed that the magnitude of speed U_M expresses the 'driver-vehicle' combination in terms of traffic behaviour. As the motorist approaches the ramp nose he faces a series of gaps on the main stream and has to decide on to which he is able to merge. The point of decision is not constant, as it has been conventionally taken as the ramp nose, but varies and the sight distance has a considerable influence on that.

When the driver arrives at the decision point, he evaluates the 'gap-structure' immediately available to him, i.e. lead and lag times and if these are greater than his critical values, he merges accelerating at a rate f_M depending on his speed U_M. Otherwise he proceeds on the acceleration lane and because he can adjust his speed, he may adopt different policies:

If he is a 'fast driver', high U_M and f_M, he may accelerate to create an acceptable gap, or to merge in front of the lead vehicle if the gap is acceptable. In the model the theoretically required acceleration f_T for an acceptable lag is calculated based on the restrictions of available length, which cannot exceed the acceleration lane boundaries, and a minimum available lead time and compared with the vehicle's value f_M. If

$$f_M > f_T$$

then the driver accelerates to create an acceptable gap, as shown in Fig. 4.14b. Otherwise he decelerates at a constant rate :

$$f_d = \frac{U_M^2}{2(L-x_M)}$$

where :

x_M = position of the vehicle

L : distance from the end of acceleration lane.

as is shown in Fig. 4.14c. If unsuccessful to merge facing a series of unacceptable gaps, due to high mainstream flows he comes to a stationary position, where an 'at grade T-junction' situation applies.

4.8.2 Queueing-Multiple Entries

One of the fundamental differences between the modelling of merging vehicles at grade intersections and ramp entries is the treatment of queueing vehicles. At ramp entries the queueing vehicles can evaluate gaps and even merge before the lead vehicle. On the other hand the acceleration lane is used as a storage lane and queueing vehicles have the opportunity to examine the mainstream gaps without a time-lag or move-up time, in order to reach the stop line, as it usually happens at priority intersections. In order to deal with the situation it can be assumed that each driver acts independently irrespective to his position in the queue. This in fact leads to large, unacceptable delays to individual vehicles, which being delayed by unacceptable mainstream gaps, then are further delayed, because a series of queueing vehicles have already merged, changing the pattern of the mainstream traffic.

A series of observations at motorway junctions have been made and the following mechanism has been adopted in the model. When a vehicle arrives at the merging area and it is queueing, then its behaviour depends on the previous vehicle. If the leading vehicle has decided to merge, then it can evaluate mainstream gaps otherwise its acceleration is determined from the following behaviour (eq. 4.8) and proceeds. A safety stopping distance is assumed, in case the vehicles come to a stop position:

$$S_{STOP} = LE(n-1) + SE$$

where

LE(n-1) : length of vehicle n-1

SE : stopping error.

The perception of the leading vehicle's behaviour is based on the assumption that the queueing vehicle can evaluate if it accelerates or decelerates.

In the case of a multiple entry, when a large mainstream gap occurs then the queueing vehicles can enter simultaneously accepting the same lag, having a minimum following headway between them. A flow chart for the modelling is given in Fig. 4.15.

4.8.3 Parameters

Because we are concerned with a microscopic investigation of traffic flow, gap acceptance consists a basic parameter in the modelling of the merging process. In the model, it has been assumed that the drivers behave consistently, i.e. the gap acceptance function for a driver has the form of a step function :

$$\begin{aligned} H(t - L_{CR}) &= 1 \quad t > L_{CR} \\ &= 0 \quad t < L_{CR} \end{aligned}$$

The critical lag is also assumed to vary between the drivers and also to depend on the relative speed, defined as the speed difference between the merging vehicle and the immediate succeeding main road vehicle. Various gap acceptance distributions have been proposed (reviewed in Chapter 2). In the model, normal, lognormal or exponential distributions have been assumed to represent the variability between the drivers, and typical values for the parameters are shown in table 4.11, based on Ref. 25.

When a vehicle is forced to stop then its critical lag increases and a typical value :

$$TSTOP = 4.00 \text{ sec.}$$

is assumed in this case (Ref. 18).

The critical lead times have been assumed constant for all the drivers, having the typical value:

$$T_{CR} = 1.00 \text{ sec.}$$

In the case of multiple entries the lead time between merging vehicles can be assumed :

$$T_{CR} = H_{MIN}$$

where :

H_{MIN} : mean minimum headway.

and for a stopped vehicle can be taken as the equivalent of a vehicle length.

TABLE 4.11

Typical Values for Gap-Acceptance Parameters (Ref. 25)

Relative Speed (mph)	Mean (sec)	S.Dev. (sec)
$U-U_M \leq -5$mph	2.310	1.000
$-5 \leq U-U_M \leq +5$mph	2.460	1.000
$U-U_M \geq +5$mph	3.000	1.000

The acceleration rates f_M are assumed to have a range of values (a_1, a_2) and normal or lognormal distributions can be fitted to represent their variability.

It is further assumed that the sampling of the acceleration rate depends on the vehicle speed U_M. Assuming that U_M is normally distributed over the range $(\mu-2\sigma, \mu+2\sigma)$, we can define ranges $\Delta_i (i = 1, 3)$ where a distribution of acceleration $N_i(\mu_i, \sigma_i)$ applies, and the cumulative distribution of the $N_i(\mu_i, \sigma_i)$ is the above defined on the range (a_1, a_2). If the speed U_M falls in the range Δ_i then the acceleration rate is sampled from the corresponding distribution $N_i(\mu_i, \sigma_i)$ as illustrated in Fig. 4.16.

Typical values assumed, are as follows

$$(a_1, a_2) \equiv (0, 0.3 \cdot g)$$

where:

$$g = 9.81 \text{ m/s}^2$$

$$\Delta_i = \frac{4\sigma}{3} \quad i = 1, 3$$

The parameters μ, σ can be determined from the simulation of vehicles on the slip road and the resultant speed-distance profile.

4.8.4 Delays, Journey Times and Merging Paths

A vehicle merging from the slip road on the motorway may be subject to delay due to the presence of mainstream vehicles. The definition of delay is not standard but varies according to the traffic situation. On conventional at-grade intersections, delay is usually defined as the difference between the time a vehicle accepts a gap on the mainstream and its arrival time at the stop line. At the ramp entry the situation is different because the vehicle continues to move as it searches for gaps on the main roadway. According to that the following definition of delay is proposed.

$$D = t_M - L_M/U_M \tag{4.12}$$

where :

t_M : difference in time between the time when vehicle merges and its arrival at the decision point.

L_M : distance travelled on the acceleration lane when vehicle was looking for an acceptable gap.

U_M : speed of the merging vehicle.

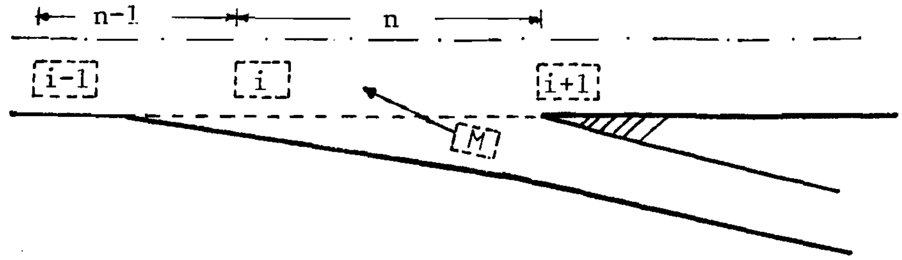
If the above defined delay is due to traffic conditions and does not include the delay due to geometric conditions, or the time needed for a vehicle to complete the manoeuvre, an instant merge has been assumed. Due to the above reasons the model allows for a constant value of delay D_S to be assigned for each merging vehicle, in addition to the calculated delay from equation 4.12. The value of D_S is input to the model, and a typical value can be chosen as :

$$D_S = 1.00 \text{ sec.}$$

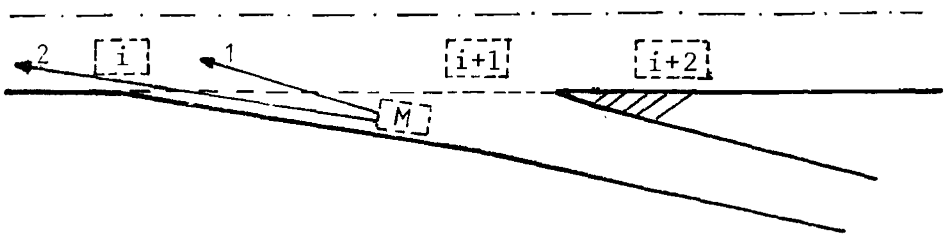
The time spent when a vehicle is queueing, is also recorded and the pattern of queueing delays is investigated.

Journey Time for a merging vehicle is defined as the difference in time when leaving the entire simulated section and its arrival time at the start of the slip road. It is a useful measure of the operating conditions occurring at the junction and can be used for comparisons of junction configurations having the same lengths but different design characteristics. The journey times can also be used as an estimate of delay assuming 'with' and 'without junction' situation.

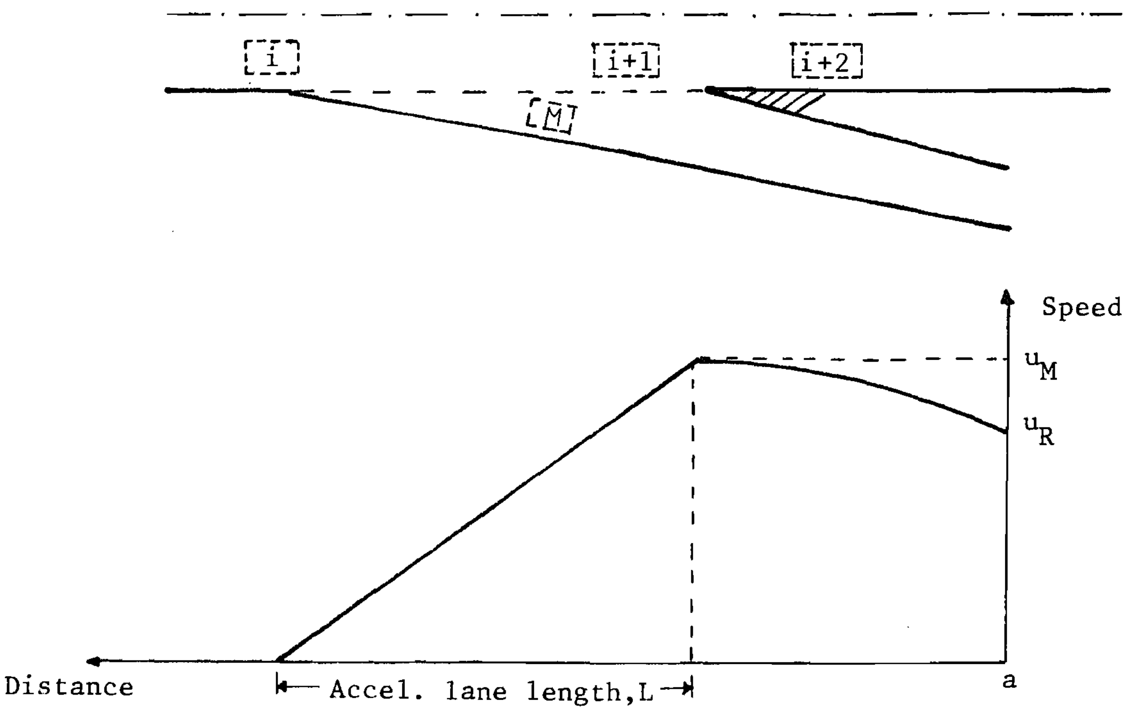
Merging path is defined as the distance between the point when the oncoming vehicle enters the shoulder lane and the ramp nose. The merging vehicles do not follow a standard pattern and a wide variety of paths have been observed under different conditions. The average values of merging paths and their distribution consist a measurement of the acceleration lane usage and in that relies on their significance for the design. In the model the paths of vehicles are recorded and their averages and pattern are investigated.



a. Direct merge into immediate available Gap(n)



b. Effect of Speed-Acceleration
1.Create an acceptable lag
2.Merging in the gap in front (n-1)



c. Unsuccesful merge and the corresponding Speed-Distance Profile

Figure 4.14: Driver's behaviour at merge

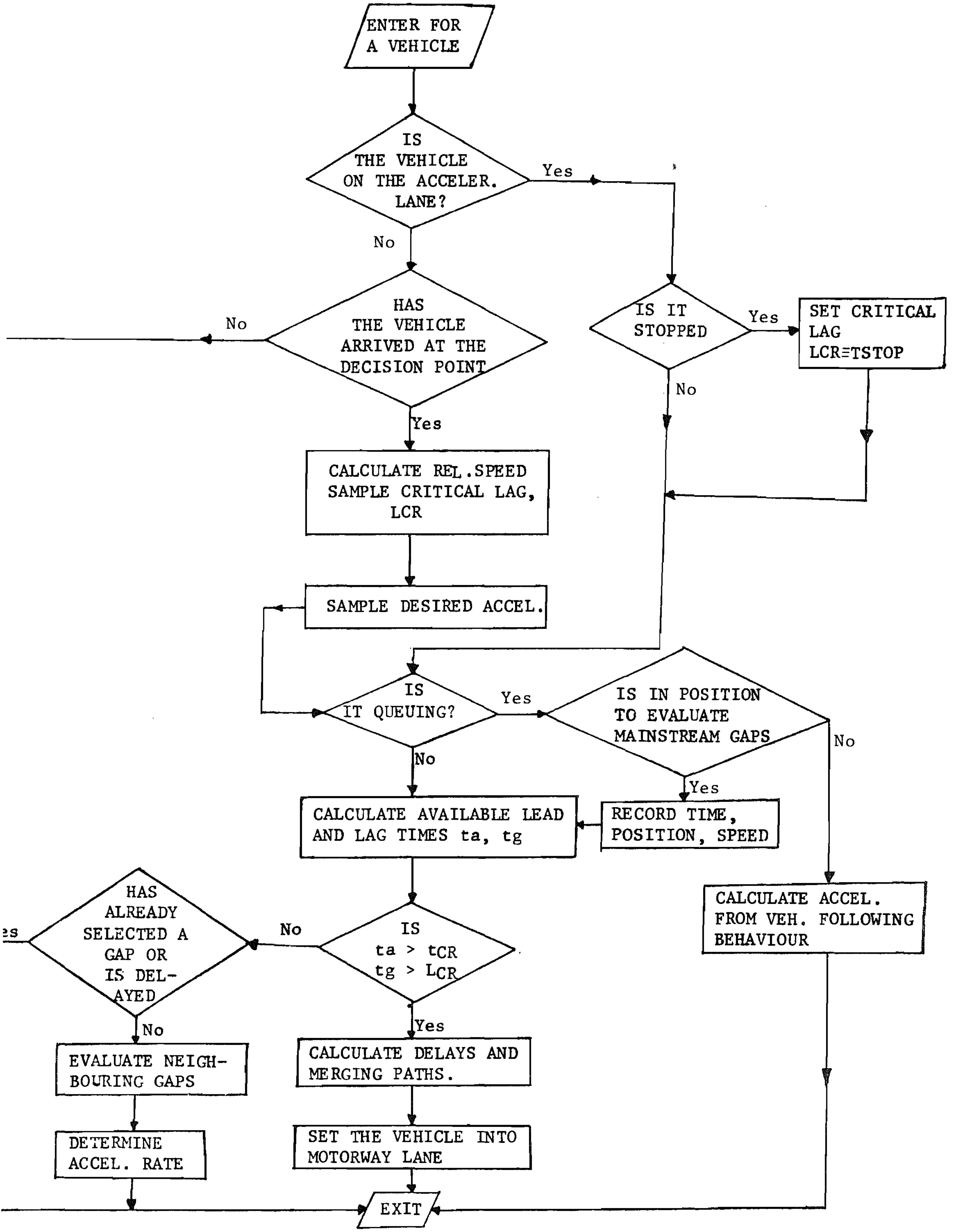


Figure 4.15: Flow Chart for the simulation of the Merging Process

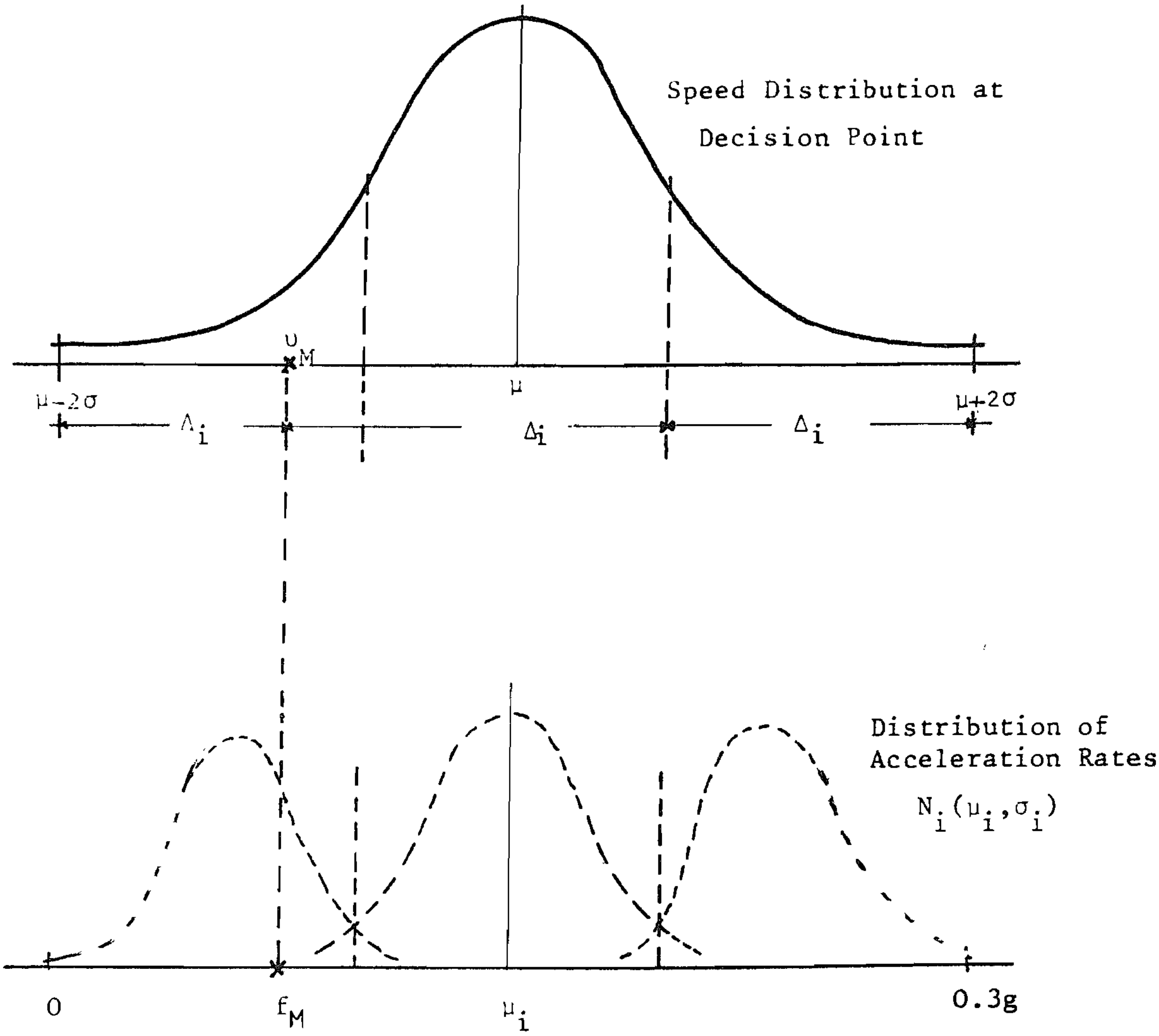


Figure 4.16: Sampling of acceleration rates f_M

4.9 Measures of Effectiveness

Measures of effectiveness or figures of merit (FOM) are the parameters for the evaluation of the operation and performance of the traffic situation, under study. Their relations with the variables of the problem are used to determine criteria for their optimization and as an aid to the design.

The definition of measures of effectiveness is very difficult because of the traffic and geometric conditions affecting the situation. The 'delay' for a merging vehicle due to mainstream flow may be not significant but the geometric conditions may induce a considerable increase in its journey time as it traverses the entire system. On the other hand, journey times are difficult to be compared due to variation in geometry at interchanges, such as slip road length.

The evaluation parameters chosen to show the junction performance under different conditions are : average delay due to main road flow, and average total journey time of merging vehicles. The typical values of journey times could be used in the comparison of systems having the same roadway lengths, but vary in terms of other design elements. It is believed that journey time is a more representative parameter than the delays at specific points, but the difficulty of an explicit definition is obvious.

The developed model, as a research study tool, can also provide information and predict the behaviour of all the components of the problem, such as : the speed distance profile for ramp vehicles, the number of merging vehicles delayed and/or stopped, the distribution of merging paths as a measure of acceleration lane usage, the lane distribution of the motorway vehicles at a point downstream, the number of lane changes between lanes and the headway distribution upstream of ramp nose for the inside motorway lane.

4.10 The Computer Program

The modelling process is integrated with a construction of a computer program to simulate the traffic behaviour. The computer program SIRAM (Simulation of Ramp Merging) is written in FORTRAN IV language for dynamic processing on the ICL 2970 computer at the Univerity of Southampton.

The program is completely modular in structure to allow for flexibility and simplicity. According to that, possible future additions or changes can be easily made or algorithms may be exchanged without changing the main program's structure. The following main subroutines are used for the simulation process, each of which utilizes a number of modules:

INPUT : Provides the values of traffic and geometric parameters for the simulation run.

INIT : Initialises the variables of the system, the random number generator and placement of vehicles on the motorway lanes and the ramp.

MERG : Examines the merging process, calculates the available gaps or lags, determines the acceleration rates for the merging vehicle, estimates delays and merging paths.

CARFOL : Simulates the movements and the interactions of vehicles on the motorway and on the ramp. Examines if any vehicle has passed the system's boundaries and updates the vehicles currently in the system.

TIMSEQ : Sets and continuously increments the time. Determines the collection and storage and data on the measures of effectiveness.

GENER : Determines if a vehicle has arrived on the motorway or ramp generation point and generates its characteristics. Generates headways and computes vehicle arrivals.

RESULT : When the simulation time expires this module analyses and prints the results of the simulation run.

The modules used by the main subroutines are listed in table 4.12. A flow chart of the whole program is given in Fig. 4.17, where the operational relationships of the modules are illustrated. The program listing is given in Appendix I.

The program uses the memorandum representation, computer words are employed to describe the vehicle's characteristics. In addition the mathematical notation is used, where each vehicle is associated with its own position indicator. Its position is therefore continuous within the accuracy of the computer. A vehicle's new position can at any time be computed as a function of its last position, its velocity, its acceleration and the time increment. Spacings between vehicles are available from their respective coordinates and the vehicle lengths. In the following paragraphs the program's logic is outlined :

The simulation starts with the reading of input data, the calculation of the required parameters and the initialisation of the system variables. The system is loaded with one vehicle per lane. The time is advanced and at each time increment the merging process is simulated with the subroutine MERG based on the assumptions described in Section 4.8.

The subroutine CARFOL calculates the acceleration of vehicles according to the procedure analysed in Section 4.5 and then updates the vehicle's position and speed based on the equation (4.10) and (4.11) for both the motorway lanes and the ramp.

The lane-changing process is subsequently examined and if a vehicle changes lane, it is set on to the new lane and the vehicle lists on the interacting lanes are rearranged. Finally, it examines if a vehicle has passed the system boundaries. If this occurs, the characteristics and journey times are recorded and the vehicle is removed from the vehicle list with an updating technique to save memory space in the computer. The program examines if a vehicle has arrived at the system boundaries, calling the main subroutine GENER. When the current time is greater or equal to the arrival time of a vehicle, then the vehicle has arrived in the system, its position is set equal to zero and its characteristics are sampled. A new headway is generated and the new arrival time is calculated. The procedure is done for all the motorway lanes and the slip road.

Data on the measures of effectiveness are not collected during the settling period, which is required for the stabilization of the section. During that period the simulation proceeds normally loading the system; when that time expires, the data on the evaluation parameters are collected and accumulated and the program analyses and writes the final results when the time exceeds the specified simulation time, calling the main module RESULT.

A number of WRITE statements have been incorporated into the program and the program can print, on request detailed information of the individual vehicle movements during the simulation, so the procedure can be thoroughly checked and debugged.

TABLE 4.12

Modules of the Simulation Program

PARAM	Calculation of parameters from input data
NORMAL	Generation of normal deviates
EXAM	Estimation of available lag and lead times
RELSP	Calculation of relative speed for a merging vehicle
AGAP	Generation of a critical gap for a merging vehicle
DELA	Calculation of merging delays
PACAL	Calculation of merging path
CALC	Estimation of theoretically required acceleration to merge
MERGST	Determination of merging driver's policy
QUEUVE	Queueing vehicle's behaviour
RSPED	Generation of desired acceleration at merge
ALMT	Vehicle movement on the acceleration lane
UPDATE SETR	Set a merged vehicle into the mainstream, update vehicle's arrangement on main lane
RESETR	Update the vehicle list on ramp
REAM	Calculate the acceleration of vehicles on the motorway
MOTION	Calculation of motorway vehicles position and speed
CARAMP	Calculation of acceleration of a ramp vehicle
UPDATR	Calculation of ramp vehicle's position and speed
SDPROF	Recording of ramp vehicle's speed on slip road points
PROFILE	Estimation of the average speed-distance profile
HESP	Recording of headways and speeds for the motorway Lane 1 at ramp nose
INLANE	Estimation of motorway inside lane headway distribution
CHANGE	Examines the lane-changing process
LANECH	Determines if a vehicle desires to change lanes
LANEGAP	Calculation of available gaps in the new lane
SETLC	Set a lange-change vehicle into new lane
RESET	Update the vehicles arrangement in the new lane
CONTROL	Examines if a vehicle leaves the system
CONTROL1	Records the exiting vehicle's characteristics
SET	Updates the vehicle list in a lane when vehicle exits or changes lane

TABLE 4.12 (contd.)

SDATA	Accumulation of journey times, flows, speeds of vehicle.
HMOT	Generation of headways and calculation of vehicle's arrivals on motorway lanes
HRAMP	Generation of headways and calculation of arrivals for ramp vehicles
APROP	Determination of vehicle type
ALMC	Generation of vehicle length
DEVEL	Generation of free speeds for motorway vehicles
RVELO	Generation of free speeds for ramp vehicles
ASSIGN	Set motorway vehicle characteristics
INVEL RRST	Calculates initial speed for restrained vehicles on motorways and the ramp
ARAMP	Determination of arrival of a ramp vehicle, set its characteristics
COLLECT	Estimation of average flow, journey speeds and times
STORE	Calculation of lane changes over fixed periods
CALSTAT	Statistical analysis of the evaluation parameters
MINFLOW COMP	Calculation of minute flow input-output at the ramp entry and estimation of the cumulative demand
CHECK	Check the vehicle spacings

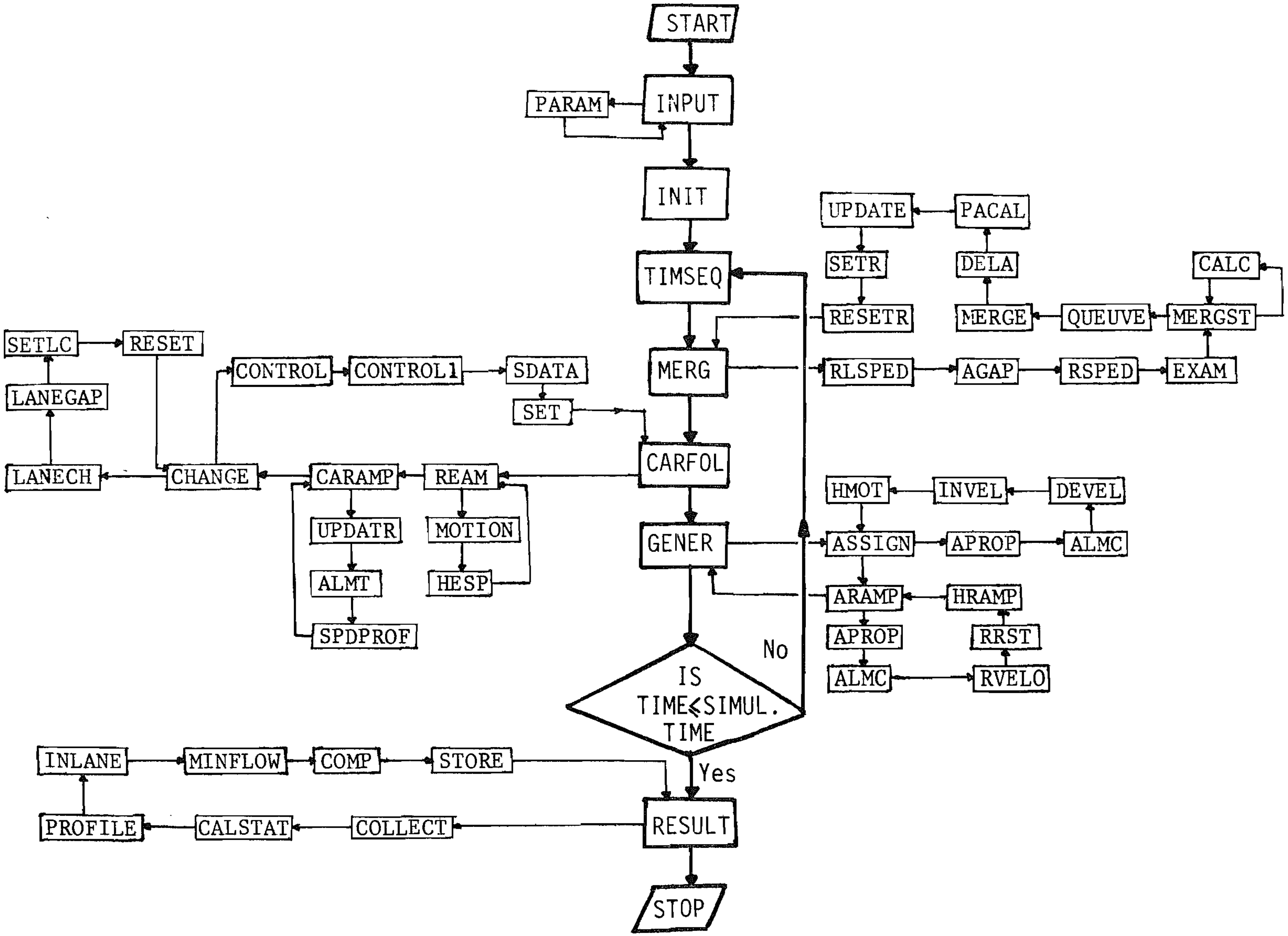


Figure 4.17: Flow-Chart for the Computer Programme SIRAM

CHAPTER 5

DATA ANALYSIS

5.1 Introduction

The aim of the research study is the development of a model which can adequately represent the traffic behaviour, enabling the evaluation of alternative designs. It is therefore necessary that the model results are carefully assessed using information from actual traffic situations. In order for that to be done, sufficient data from existing interchanges should be measured and analysed.

This chapter is concerned with the data analysis process of the study. The analysis procedure consists of the measurement of the data and their statistical analysis. The results will subsequently be used for the model calibration and validation. The stages of the analysis are shown in the flow diagram in Fig. 5.1.

5.2 The Data Base

One of the distinctive characteristics of the merging situation is that interaction of vehicles takes place not on a specific point, but over a whole area. That makes the data collection difficult, expensive, and time consuming. Although the data collection tools were available, it was considered impracticable in terms of time and budget the installation of loop detectors on specific interchanges for the measurement of traffic parameters, or the use of a number of technical staff to collect the required data.

In the context of the study, the only appropriate methods seem to be time-lapse photography or Video recording, which have the advantage of providing a permanent and comprehensive record of traffic movement, which can readily be used for extensive analysis (Ref. 121,122).

Having the availability of such equipment, the problem was the selection of suitable sites. At that time a large number of video tapes collected for the Department of Transport as part of a study (Ref. 123) were available to us.

Initial detailed viewing of the tapes has shown that they were suitable for analysis purposes and it was decided to form the data base of the study. In table 5.1 the characteristics and details of the tape recordings are given.

It can be seen that the tapes cover a range of geometric and traffic conditions at motorway interchanges, and were taken at peak periods.

5.3 Equipment-Method of Analysis

The method used for data extraction consists of slow playback of the tape on a monitor screen, scaled such that distances on it correspond to distances on the ground. Recording the times of vehicle arrivals at certain reference lines enables various traffic variables to be measured.

The drawing of the reference lines on the screen has been made using standard reference points, such as lamp posts and road markings. Detailed plans of the junctions were available and based on those, distances on the screen were calculated, according to the theory of cross-ratio as discussed by Hallert (Ref. 124). This principle of projectivity states that a cross-ratio of distances between four points on a straight line is invariable under projection. Consequently if the distance between four collinear points on the pavement is known and three of these points are identifiable on the image, then the location of the missing fourth point can be computed. The reference lines were drawn on a very thin tracing paper which was superimposed on the monitor screen. A typical site is shown in Fig. 5.2.

One of the advantages of videotape recording, compared with time lapse photography, is that it operates at the standard speed of 50 field/sec, so the events can be registered with an accuracy of 0.02 sec. at the slowest rate of playback, whereas time lapse camera cannot accurately operate at speeds in excess of 4 frames/sec and thereby requires tedious interpolations between successive exposures. In order to take advantage of this and to facilitate the procedure, a digital clock has to be superimposed on the screen for direct recordings of vehicle timings.

Most of the available video tapes (type V62 SONY high density) had no built in time base so it was decided to be re-recorded into standard 3M UMATIC S tapes, compatible with the available video recorder, with a superimposed digital clock. The available DIGITEL D4031 number generator, provides a crystal controlled time (hours, minutes, seconds and 1/100 sec.) output which can be superimposed at any position and with a number of alternative formats on the video recording. The equipment was tested for possible differences between recording speed and playback speed, using a tape with built-in time base and no discrepancy was observed.

The video recorder used in the analysis was a SONY UMATIC VO4800 PS portable recorder, with a remote pause controller to enable various rates of slow-playback, i.e. from normal speed up to 0.02 sec. Two monitors were used : a PHILLIPS 12" for tests and recording and a PYE 26" for the analysis. In Fig. 5.3 the schematic arrangement of the equipment is shown.

<u>a. Data of the Motorway Traffic Stream</u> : These were recorded as arrivals of vehicles per lane, on specific reference points. Basically three reference lines M_1, M_2, M_3 were used as shown in Fig. 5.2. Line M_1 was the headway line and lines M_2, M_3 having a distance D m. apart, used for speed estimation. If the arrival times of a vehicle n are $x_{i,n}$ (i = 1, 3), it follows :

$$\text{Headway, } h = x_{1,\, n+1} - x_{1,\, n} \text{(sec)}$$

$$\text{speed}(n) = 3.6 * D/_{(x_{3,n} - x_{2,n})} \text{ (kph)}$$

The distance D is usually taken as the constant distance between two lamp posts, i.e:

$$D \simeq 40 \text{ m}.$$

Information such as vehicle types, lane changes were also noted.

b. Data on the Merging Process : In order to obtain detailed information of the behaviour of the merging vehicles, recordings of arrivals at certain points of the ramp vehicle and the corresponding mainstream vehicles were made, as it is shown in Fig. 5.2. The accepted lead and lag times were measured at the point of entry, as time differences between the arrival of the ramp vehicle and the interacting mainstream vehicles. In case of multiple entries, the lead times are the time headways between successive merging vehicles at the last common reference line. The type of the vehicle and its path were also recorded on the data sheets. Because the merging phenomenon is dynamic, and due to the traffic conditions occurring, gap rejection was very difficult to be identified and also the actual point of decision. In some cases, the accepted lag was difficult to be estimated, because the oncoming mainstream vehicle changes lane, creating a large gap for the entering vehicles.

5.4 An Alternative Method of Data Extraction

The main data extraction has been made according to the method described in the previous section. For the case of certain traffic variables such as headway data or free speeds of vehicles on the motorway lanes an alternative method was used, having a main advantage that reduces substantially the time required for data recording.

According to that method, a multichannel event recorder was used, and the data were recorded at real time video playback. The event recorder has built-in a digital clock with accuracy of 0.01 sec. and the register of an event is made by pressing the time button on the handset provided. Up to four channels can be used simultaneously and also information such as vehicle types can be input using numeric codes on the handset. The data are automatically transferred, with the aid of a serializer , on a PG2100 Data recorder and stored on a magnetic tape. The data can be listed and checked through a terminal and subsequently punched into standard format paper tape for computer use. The method is diagramatically shown in Fig. 5.4.

5.5 Data Processing

This phase of the data analysis process consists of tranferring the extracted data from the video tapes into readily available form for statistical analysis. The information from the data sheets were punched on cards and also the data on magnetic tapes into standard format paper tapes and were input to the computer as the first data files.

A number of computer programs were written to calculate the various traffic variables such as speeds, headways, accepted lead and lag times, check the data for any input errors and make the necessary corrections. The resultant output was a detailed information for each vehicle, i.e. type, headway, speed, for motorway traffic, and type, headway, entry speed, final speed, average acceleration, lead time accepted, lag accepted, path, relative speed for ramp vehicles.

The output was stored in files, which form the final data files for the main analysis.

TABLE 5.1

Data Base

TAPE	TIME OF DAY	DATE	LOCATION	DURATION
1	p.m. peak	Friday 15 Sept.,1978	Gr. Bar northbound entry to M6	60 min.
2	a.m. peak	Wednesday	M5/M6 southbound	30 min.
3		17 Jan., 1979	merge	30 min.
4	a.m. peak	Monday	Bentley southbound	50 min.
5		16 Oct., 1978	merge (M6)	30 min.
6	p.m. peak	Friday 30 June 1978	M5/M6 northbound merge (Ray Hall)	40 min.
7	p.m. peak	Monday 10 July 1978	M5/M6 northbound merge (Ray Hall)	60 min.
8	p.m. peak	Friday 14 July 1978	M5/M6 northbound merge (Ray Hall)	60 min.
9	p.m. peak	Wednesday 12 July 1978	A38(M)/M6 southbound merge	60 min.
10	a.m. peak	Friday 14 July 1978	Gravelly Hill/ Entry to A38(M)	60 min.
11	a.m. peak	Thursday 13 July 1978	A38(M)/M6 slip roads-weaving section	60 min.
12	p.m. peak	Friday 14 July 1978	A38(M)/M6 slip roads-weaving section	60 min.
13	a.m. peak	Friday 14 July 1978	A38(A)/M6 slip roads-weaving section	40 min.
14	a.m. peak	Wednesday 12 July 1978	A38(M)/M6 slip roads-weaving section	60 min.

INITIAL ASSESSMENT
OF THE TIME BASE

↓

RE-RECORDING OF
ORIGINAL VIDEO
TAPES

↓

DATA EXTRACTION
FROM TAPES

↓

DATA PROCESSING
INITIAL DATA FILES

↓

CALCULATION OF
TRAFFIC PARAMETERS
DATA CHECK AND
CORRECTION

↓

FINAL DATA FILES

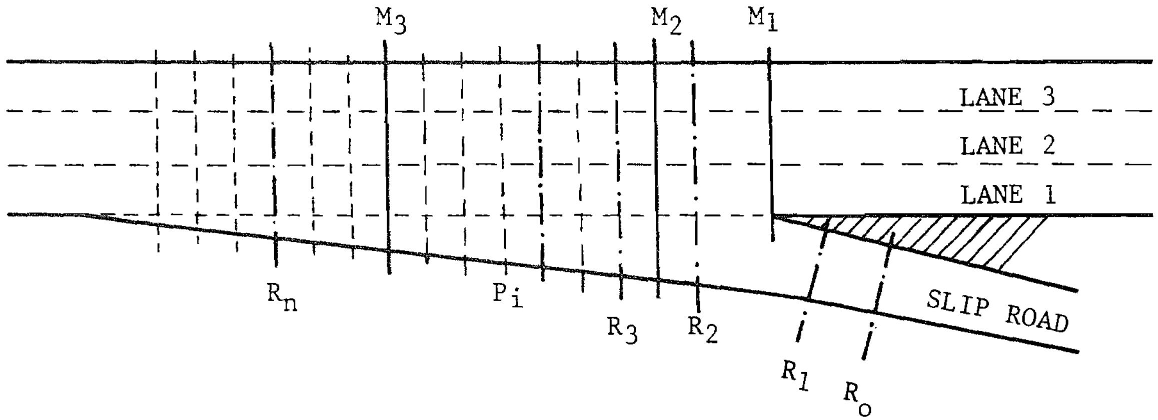
↓

INITIAL STATISTICAL
ANALYSIS

↓

DETAILED STATISTICAL
ANALYSIS-PARAMETERS
ESTIMATES

Figure 5.1: Stages of the Data Analysis procedure.



Legend

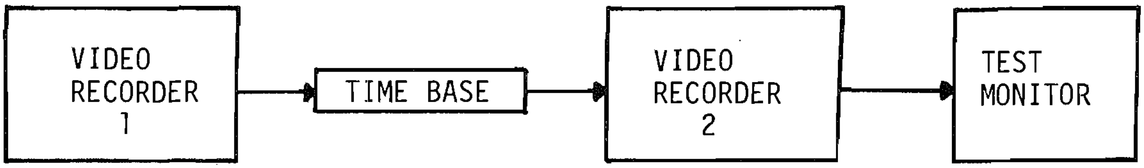
——— M_i:Reference Lines for Motorway Vehicles

—·—· R_i:Reference Lines for Merging Vehicles,where

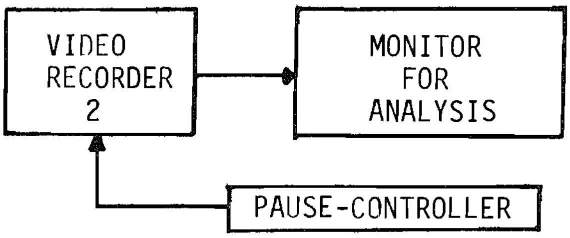
R_o,ref. line for headways.

- - - - P_i:Reference Lines for estimation of Merging Paths

Figure 5.2:Reference Lines used for Data Recording(Plan of a typical site)



a. Tape Re-recording



b. Tape Projection

Figure 5.3:Video Equipment used for Data Recording.

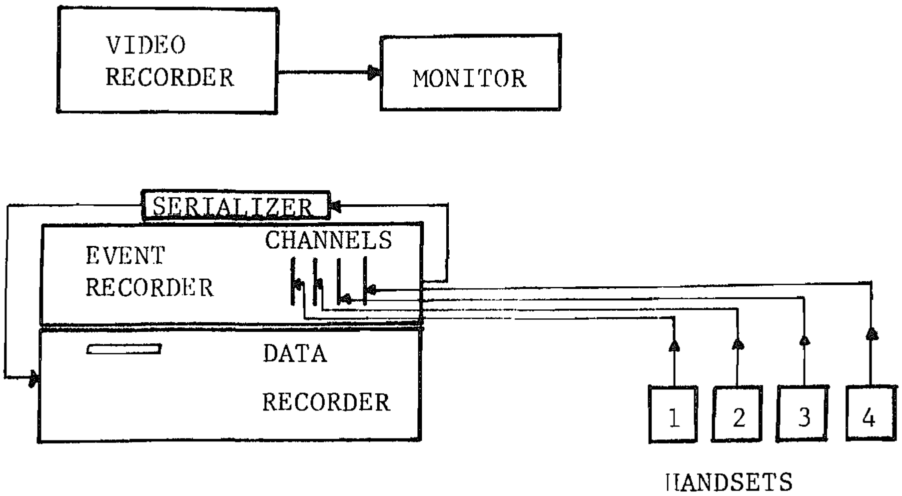


Figure 5.4:Data Recording,based on the alternative method.

5.6 Main Analysis and Results

In this phase detailed statistical analysis of the data was done in order to estimate the typical values of the measured traffic parameters, and to investigate their variability, searching for mathematical distributions to express their pattern. In this section the method of analysis is outlined and the results are presented, for the two basic sets of data: Mainstream traffic and the entering stream.

5.6.1 Mainstream Traffic Data

a. Flows-Vehicle Composition : Motorway lane flows upstream of the ramp nose, were computed on average of classified minute counts and the vehicle composition is given as percentage of H.G. Vehicles. Consequently the total motorway flow and the percentage of total flow per lane were calculated. These are given in tables 5.2, 5.3 for three lane motorway sections and also for one two lane site. It can be seen that the range of total flow Q_M is :

$$1869 \leqslant Q_M \leqslant 3528 \text{ (vph)}$$

which corresponds to the level of 'standard design flow' (Ref. 108).

The lane distribution for all the sites, plotted in Fig. 5.5, shows an increased proportion of flows on the offside lanes and as the total flow increases the flow on the inside lane decreases, having in most sites the value:

$$Q_1 = 500 - 600 \text{ vph.}$$

The difference to the general pattern is in the site Great Bar, (table 5.2), where we have a higher inside lane flow but that can be explained due to the very high proportion of H.G. Vehicles.

b. Headway Pattern : Headways per lane were initially analysed and frequency distributions were obtained. Data were sampled during periods when the flow was constant, in order to eliminate variations and disturbances of average flow that may produce biased distributions.

The determination of constant flow periods was made using a method developed by Breiman and Lawrence (Ref. 115), known as the 'area test'. According to that method, if $N(t)$ the number of cars counted up to time t, then in order to evaluate if a change in flow occurs, over a time interval $(0, \xi)$ we use the statistic:

$$Z(\xi) = \frac{1}{\sigma}\int_0^{\xi} N(t)\,dt - \frac{\xi}{2\sigma}N(\xi) \tag{5.1}$$

where :

σ^2 : Variance $(N(1))$

If there are N cars in time ξ and x_k denotes headway of the k car, we have :

$$\int_0^{\xi} N(t)\,dt \simeq \sum_{k=1}^{N} kX_k$$

and from (5.1) it follows:

$$Z(\xi) = \frac{1}{\sigma}\sum_{k=1}^{N} kX_k - \frac{\xi}{2\sigma}N(\xi)$$

The statistic $Z(\xi)$ has been shown to be normally distributed with $(0, \xi^3/12)$. Define :

$$U(\xi) = \sqrt{12}\,Z(\xi)/\xi^{3/2}$$

so $U(\xi)$ has a $N(0, 1)$ distribution. We fix a level of significance and run the test until the first time that :

$$|U(\xi)| \geqslant \alpha$$

Then we stop and back up the last previous value, ξ_0, for which

$$|U(\xi)| \lesssim 1$$

The constant flow period is taken up to ξ_o . The test then restarts from the end of the first interval and used to detect a second change in flow. Typical values of α is

$$\alpha = 3.5$$

which gives a probability of falsely rejecting a constant flow period less than 4 per cent.

Statistical models were fitted to the headway data, and the shifted negative exponential distribution, was found to describe adequately, the empirical distributions. The chi-squared test of goodness of fit was used throughout this study to indicate the fit of models to data. Classes for use in the test were obtained by dividing the observed data in each sample into ranges with approximately equal frequencies (Ref. 116) the number of classes k being given by

$$k = 2.4(n - 1)^{0.4} \tag{5.2}$$

where:

n : sample size $n \geq 200$

Large chi-squared values often seemed to contradict good visual fits to large samples, and for this reason the significance level of the chi-squared test was allowed to vary with sample size. A value of 5 percent was used for samples with less than 500 observations,a value of 1 percent being used for larger samples. This decision did not imply that the changed level of significance related to extra confidence in the data, but rather was used as an alternative estimate of the goodness of fit.

In most of the samples the shifted exponential distribution was statistically acceptable, with minimum headway :

$\tau = 0.50$ sec for the offside lanes

$\tau = 1.00$ sec for the inside lane

The exception was the case of motorway lane 3 at higher flows, where a high proportion of headways in the range of (2.00 - 3.00) sec. occurred and the fitted distribution failed to give an acceptable fit. It was observed that a number of drivers were forced to use that lane in order to overtake slower vehicles moved on the middle lane at the merging area.

The double exponential, lognormal, and generalized queueing distributions were tested,but none gave a better overall fit, than the shifted exponential.

In tables 5.4, 5.5, 5.6 the measured headway parameters and the parameters of the fitted distribution are given on a per lane basis for data on the three lane motorway sites, table 5.7 is referred to the inside lane of the three lane section at Bentley site and table 5.8 is referred to the two lane section at the M6 southbound site. The empirical and fitted probability distributions for all the sites are plotted in Fig. 5.6. up to Fig. 5.10.

c. Speeds of Motorway Vehicles : Speeds of vehicles per lane were calculated according to the method described in Section 5.3. Typical values such as mean and s.deviation were estimated for cars and H.G.V. separately and for all the vehicles combined. These are summarized in tables 5.9 and 5.10 for the two sites. In the other sites the camera position did not permit accurate estimation of vehicle speeds. It can be seen that there was no significant difference between samples, and sites too. As the lane number increases, the mean speed increases and the s.deviation decreases.

d. Free Speeds of Vehicles : Because the free speeds of motorway vehicles is an essential input to the simulation model, it is necessary to estimate the desired speed distribution from actual traffic data.

The usual method of determining the distribution of free speeds at a site is to collect data during periods of light flow when the proportion of vehicle interaction is small (Ref. 117). The application of this method is subject to two possible drawbacks.

First, for some sites flows may seldom be sufficiently low to determine the distribution in this way; secondly, even when low flow conditions do occur, there is no guarantee that the distribution of free speeds measured for that population of drivers will be the same as that for a population of drivers travelling at higher flows. It was decided that the free speeds distribution to be estimated from all the speed data collected, which also had the advantage of not collecting additional data. A method proposed by Branston (Ref. 118) was used, based on measurements of speeds of vehicles passing a point. The method requires the determination of an impeded vehicle and two criteria were used : time headway T_C and relative speed to the vehicle in front in the same lane. When a vehicle had a time headway greater than T_C , or a relative speed outside a predetermined range R_C it was classified as unimpeded.

The estimation of T_C was based on the following considerations : Firstly the Highway Capacity Manual 1950 method, (Ref. 119) states that the s.deviation of relative speeds decreases very slowly with decreasing headway until the critical T_C is reached, whence it rapidly decreases with decreasing time headway as the proportion of impeded vehicles increases. The application of the method is shown in Fig. 5.11, where the straight lines are plotted with the method of least squares, and the headway, where the lines intersect, gives the value of T_C :

$$T_C = 3.3 \text{ sec.}$$

for the motorway middle lane. When applied to all samples it gave T_C values in the range of (3.00 - 4.0) sec. Alternatively the T_C can be determined from the cut-off point of the headway distribution, at which the fit of the shifted negative exponential distribution becomes acceptable at the 5 per cent level of statistical significance (Ref. 120). The problem with that method is that under the flows occurring in the measured sites, gives a T_C value less than 2 sec., which is less than it is expected. In case of flows greater than 1100 veh/h it produced a critical value of 3.00 sec at motorway lane 3.

Relative speed criteria were obtained from the mean m and s.deviation, s of relative speeds of vehicles having headway less than 1 sec, defined as :

$$R_C = m \pm \ell . s$$

The sensitivity of the criteria chosen was tested by estimating the free speed distributions for various combinations of T_C, ℓ . A sensitivity analysis for the tape 8 is given in table 5.11 and as it is shown the mean free speed seems to be fairly insensitive to the above criteria, the maximum variation D being about 1 per cent. Similar results were obtained and for the other samples :

$D \simeq 1.2$ per cent tape 7

$D \simeq 1$ per cent tapes 1A, 1B.

The normal distribution was fitted to the resultant free speed distributions, and the goodness of fit was accepted in all the samples. In tables 5.12, 5.13 the parameters of the free speed distributions per lane and for the whole traffic stream, are given, based on the criteria :

$$T_c = 3.50$$

$$\ell = 2.0$$

and in Fig. 5.12, 5.13 their frequency distributions are plotted.

Free speeds were also estimated and for the two lane motorway section at site M5/M6 southbound merge and the results are given in tables 5.14, 5.15. The probability distributions of the free speeds are plotted on normal probability paper in Fig. 5.14.

TABLE 5.2

Average Flows on Motorway. (vph).

SITE	TAPE	VEH TYPE / LANE	CARS	H.G.V.	% VEH. COMPOSITION	ALL VEH.	% TOTAL FLOW
M5/M6 Ray Hall	7	LANE 1	331	166	33.40	497	21.80
		LANE 2	839	168	16.58	1007	44.20
		LANE 3	773	0	0.0	773	34.00
		TOTAL	1943	334	14.62	2277	100.00
	8	LANE 1	360	163	31.16	523	17.80
		LANE 2	884	260	22.70	1145	39.00
		LANE 3	1270	0	0.0	1270	43.20
		TOTAL	2514	424	14.43	2938	100.00
Great Bar (M6)	1A	LANE 1	306	394	56.28	700	30.20
		LANE 2	719	205	22.18	924	39.90
		LANE 3	694	0	0.0	694	29.90
		TOTAL	1719	599	25.80	2318	100.00
	1B	LANE 1	287	350	55.00	637	28.18
		LANE 2	723	203	22.00	926	41.00
		LANE 3	697	0	0.0	697	30.82
		TOTAL	1707	553	24.46	2260	100.00
A38/M6 Southb. merge	9	LANE 1	349	127	26.68	476	25.46
		LANE 2	738	94	11.30	832	44.52
		LANE 3	561	0	0.0	561	30.02
		TOTAL	1648	221	11.82	1869	100.00

TABLE 5.3

Average Flows on Motorway. (vph).

SITE	TAPE	VEH TYPE / LANE	CARS	H.G.V.	% VEH. COMPOSITION	ALL VEH.	% TOTAL FLOW
Bentley Southb. merge (M6)	4	LANE 1	366	201	35.45	567	16.20
		LANE 2	729	595	44.94	1324	37.84
		LANE 3	1608	0	0.0	1608	45.96
		TOTAL	2703	796	22.75	3499	100.00
	5	LANE 1	369	199	35.00	568	16.10
		LANE 2	747	586	43.96	1333	37.78
		LANE 3	1627	0	0.0	1627	46.12
		TOTAL	2743	785	22.25	3528	100.00
M6 Southb. merge	3	LANE 1	398	293	42.4	691	50.5
		LANE 2	618	59	8.70	677	49.5
		TOTAL	1016	352	25.74	1368	100.00
	2	LANE 1	447	256	36.42	703	51.0
		LANE 2	616	59	8.74	675	49.0
		TOTAL	1063	315	22.85	1378	100.00

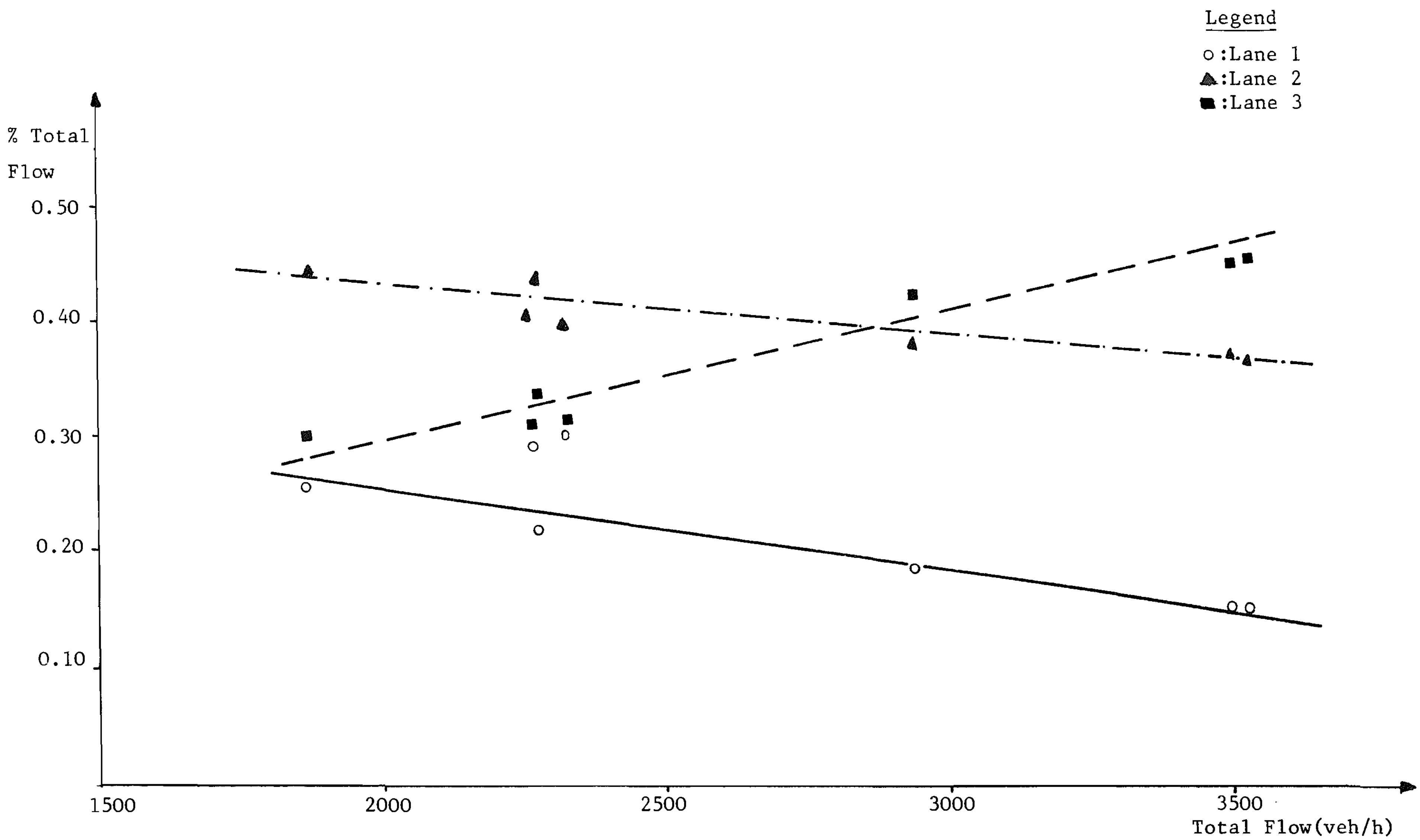


Figure 5.5:Lane Distribution on Motorway Section (All Sites)

TABLE 5.4

Headways on Motorway Lanes

Site: M5/M6 Northbound merge

				MEASURED		FITTED EXP. DISTR.	
TAPE	LANE	CONST. FLOW (vph)	SAMPLE	MEAN (sec)	MINIMUM (sec)	MEAN (sec)	MINIMUM (sec)
7	LANE 1	500	495	7.19	0.52	7.19	1.00
	LANE 2	1005	998	3.58	0.50	3.58	1.00
	LANE 3	860	755	4.19	0.50	4.19	0.50
8	LANE 1	523	377	6.89	0.66	6.89	1.00
	LANE 2	1145	825	3.14	0.50	3.14	1.00
	LANE 3	1320	941	2.73	0.50	2.73	0.50

TABLE 5.5

Headways on Motorway Lanes

Site: Gr: Bar.

				MEASURED		FITTED EXP. DISTR.	
TAPE	LANE	CONST. FLOW (vph)	SAMPLE	MEAN (sec)	MINIMUM (sec)	MEAN (sec)	MINIMUM (sec)
1A	LANE 1	698	335	5.16	0.60	5.16	0.50
	LANE 2	930	519	3.87	0.50	3.87	0.50
	LANE 3	840	383	4.28	0.50	4.28	0.50
1B	LANE 1	637	256	5.65	0.90	5.65	0.50
	LANE 2	956	365	3.76	0.50	3.75	0.50
	LANE 3	793	272	4.54	0.50	4.54	0.50

TABLE 5.6

Headways on Motorway Lanes

Site: A38/M6

				MEASURED		FITTED EXP. DISTR.	
TAPE	LANE	CONST. FLOW (vph)	SAMPLE	MEAN (sec)	MINIMUM (sec)	MEAN (sec)	MINIMUM (sec)
9	LANE 1	480	399	7.44	1.02	7.50	1.50
	LANE 2	800	666	4.52	1.00	4.50	1.00
	LANE 3	720	477	4.98	0.52	5.00	1.00

TABLE 5.7

Headways on Motorway Inside Lane

Site: Bentley

			MEASURED		FITTED EXP. DISTR.	
TAPE	CONST. FLOW (vph)	SAMPLE	MEAN (sec)	MINIMUM (sec)	MEAN (sec)	MINIMUM (sec)
4	580	488	6.18	1.02	6.20	1.50
5	568	243	6.34	1.00	6.34	1.50

TABLE 5.8

Headways on Motorway Lanes

Site: M6 Southbound (two lane section)

				MEASURED		FITTED EXP. DISTR.	
LANE	TAPE	CONST. FLOW (vph)	SAMPLE	MEAN (sec)	MINIMUM (sec)	MEAN (sec)	MINIMUM (sec)
LANE 1	3	690	334	5.22	0.60	5.22	1.00
	2	700	423	5.14	0.50	5.14	1.00
LANE 2	3	680	330	5.24	0.50	5.24	0.50
	2	675	422	5.32	0.50	5.32	0.50

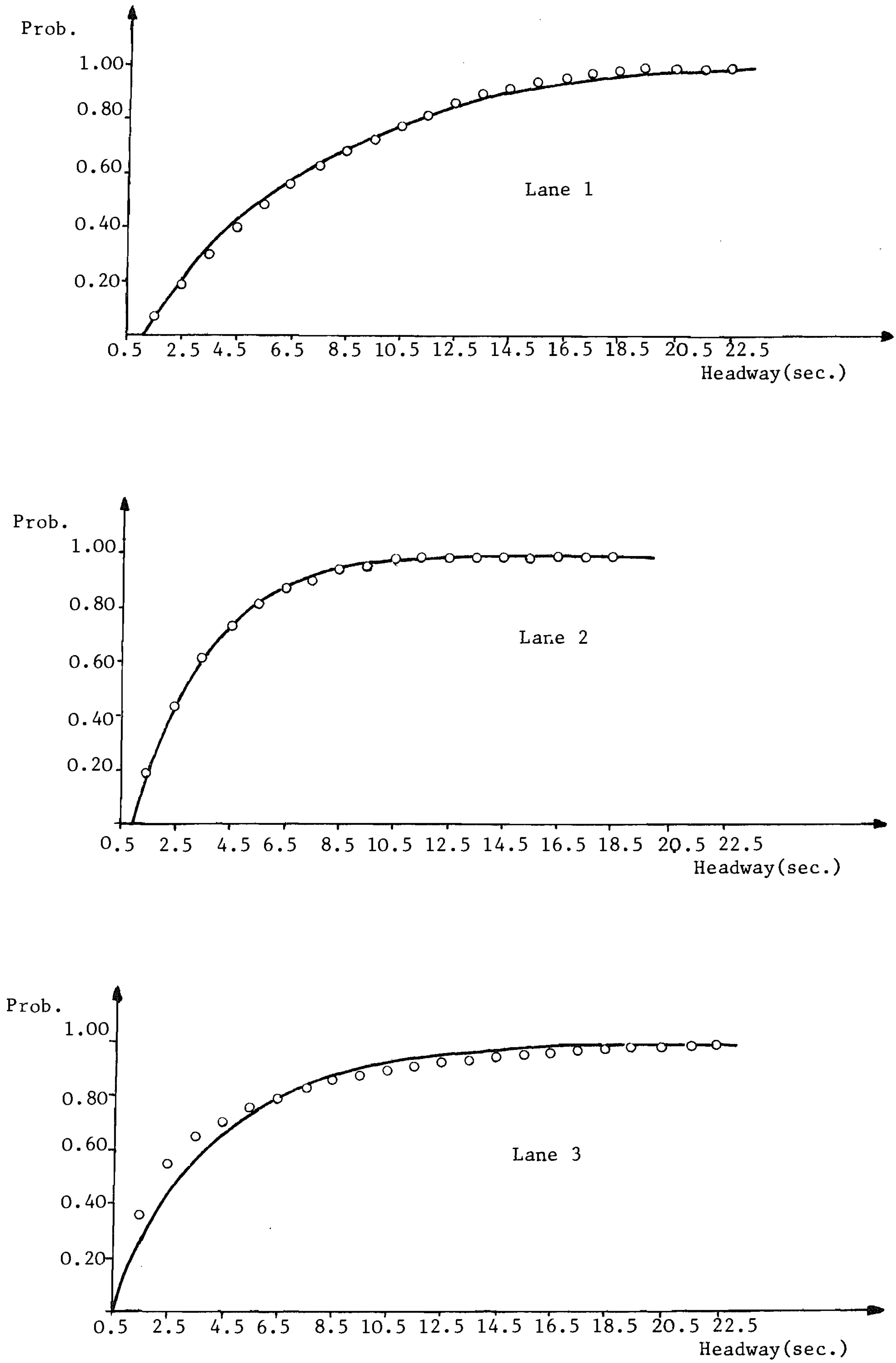


Figure 5.6: Probability distr. of Headways on Motorway Lanes.
Site:M5/M6,Ray Hall,Tape 7.

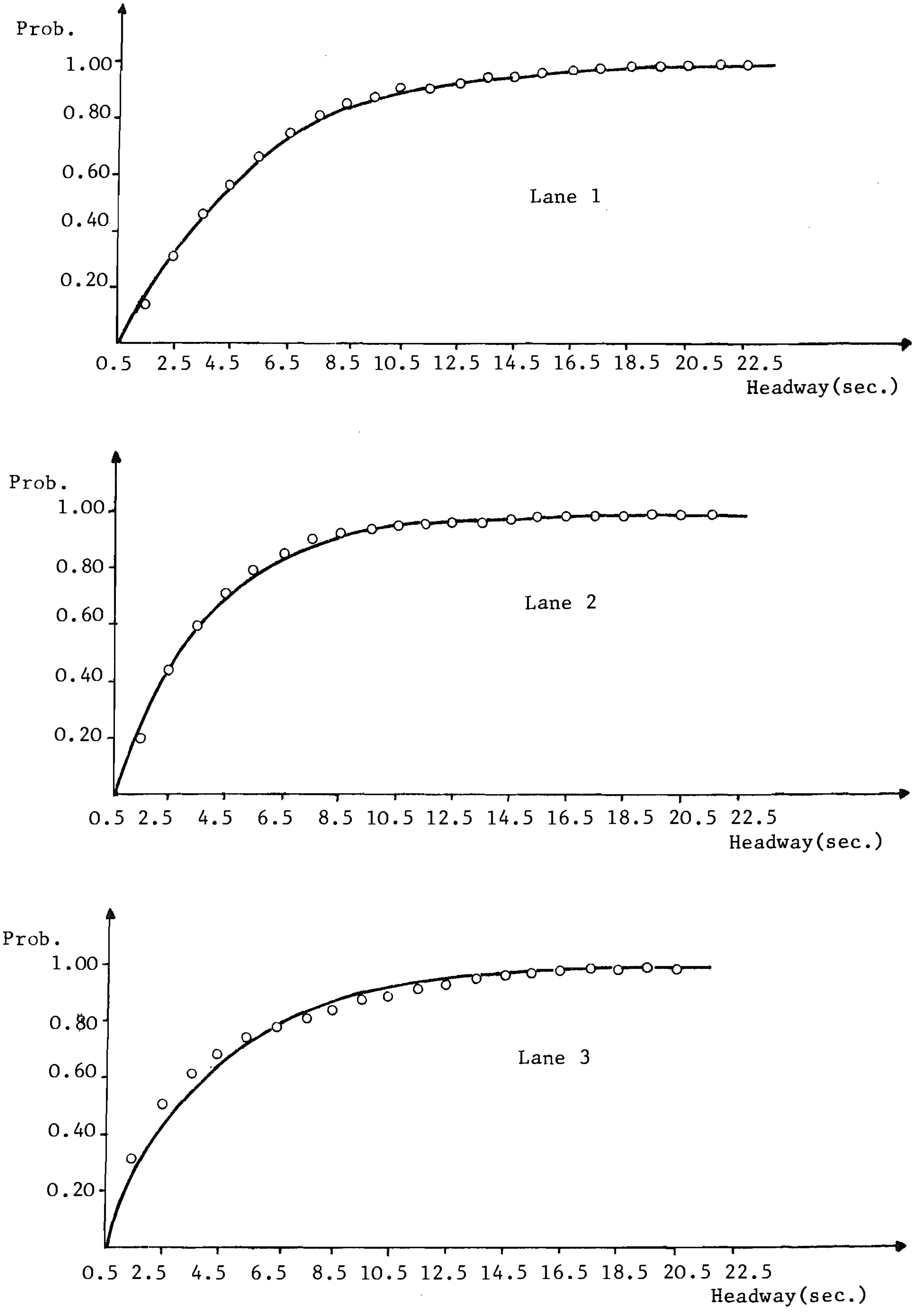
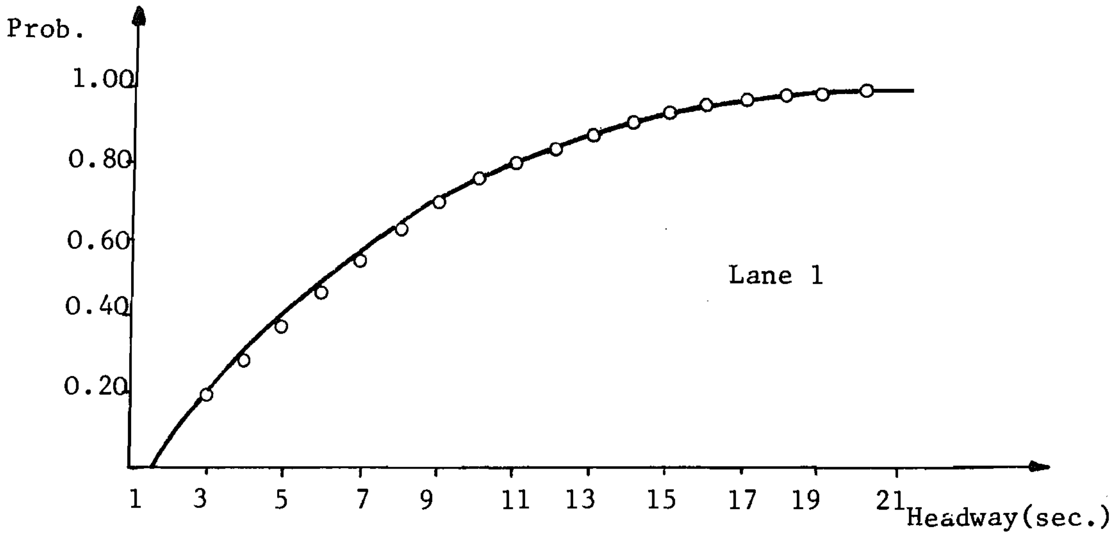
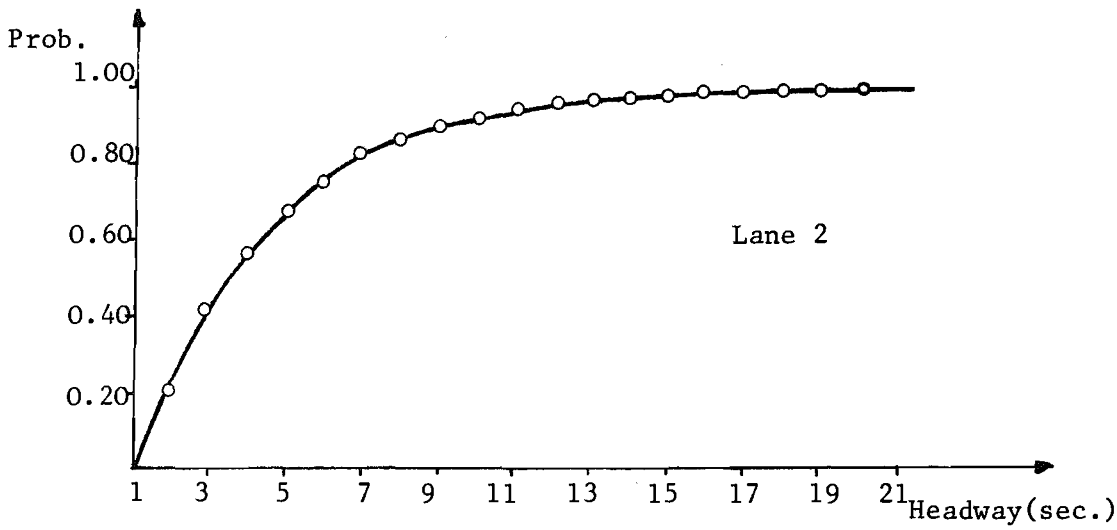


Figure 5.7: Probability distr.of Headways on Motorway Lanes.

Site:Great Bar,Tape 1A.





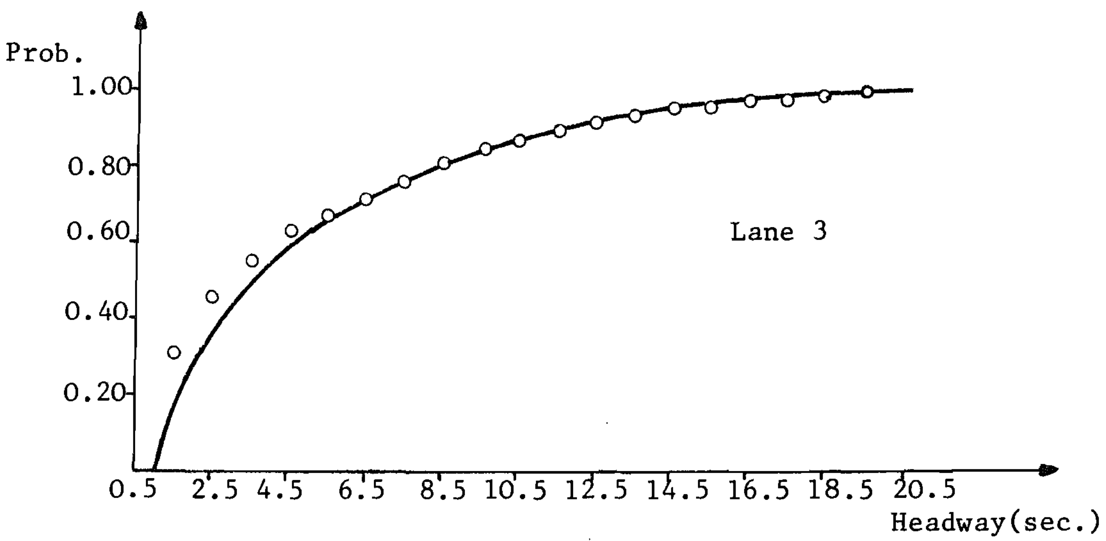


Figure 5.8: Probability distr. of Headways on Motorway Lanes.

Site:A38/M6,Tape 9.

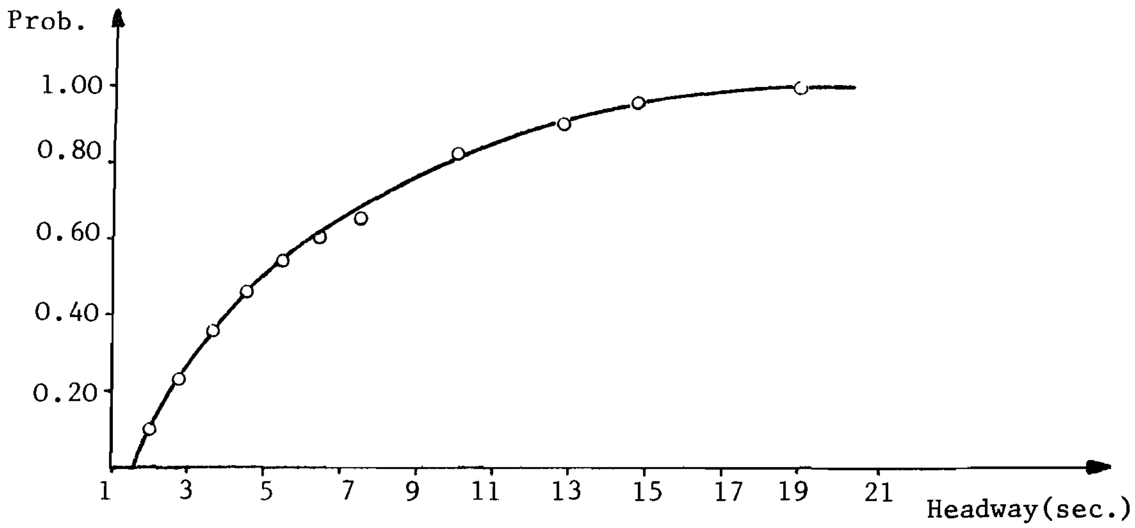


Figure 5.9:Probability distribution of Headways on Motorway Lane 1(Site:Bentley)

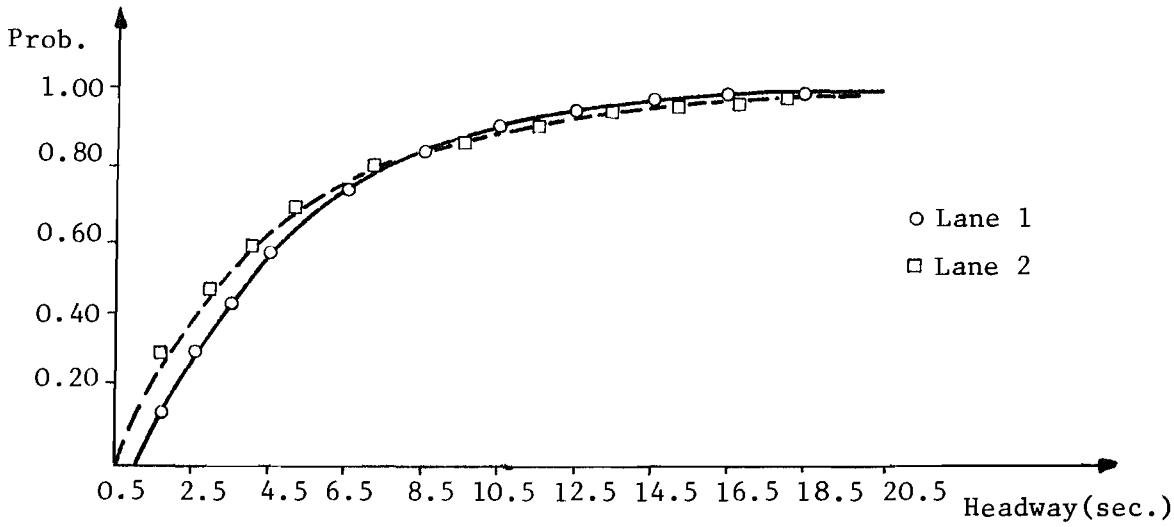


Figure 5.10:Probability distributions of Headways on Motorway Lanes(Site:M6 Southbound Merge)

TABLE 5.9

Observed speeds on Motorway

Site: M5/M6

TAPE 7						
LANE	VEH. TYPE	SAMPLE	MEAN (kph)	S.DEV.(kph)	MIN.	MAX.
LANE 1	CARS	320	83.752	10.050	63.158	118.033
	H.G.V.	164	78.124	9.190	52.747	107.464
	ALL VEH.	484	81.845	10.115	52.747	118.033
LANE 2	CARS	818	91.323	11.421	64.865	135.850
	H.G.V.	162	82.161	11.100	64.573	111.073
	ALL VEH.	980	89.809	11.861	64.573	135.850
LANE 3	CARS	742	107.054	8.514	72.727	133.330
	H.G.V	-	-	-	-	-
	ALL VEH.	742	107.054	8.514	72.727	133.330
TAPE 8						
LANE	VEH. TYPE	SAMPLE	MEAN (kph)	S.DEV.(kph)	MIN.	MAX.
LANE 1	CARS	255	81.318	10.341	61.276	114.286
	H.G.V.	118	80.434	9.482	51.797	104.350
	ALL VEH.	373	81.037	10.072	51.797	114.286
LANE 2	CARS	630	89.862	9.612	65.455	122.034
	H.G.V.	190	82.721	8.230	54.961	104.350
	ALL VEH.	820	88.207	9.782	54.961	122.034
LANE 3	CARS	865	102.569	8.259	72.727	133.333
	H.G.V.	-	-	-	-	-
	ALL VEH.	865	102.569	8.259	72.727	133.333

TABLE 5.10

Observed speeds on Motorway

Site: Gr. Bar.

TAPE 1A						
LANE	VEH. TYPE	SAMPLE	MEAN (kph)	S. DEV. (kph)	MIN.	MAX.
LANE 1	CARS	146	79.666	13.108	51.062	112.500
	H.G.V.	172	75.240	10.898	51.797	103.597
	ALL VEH.	318	77.272	12.146	51.062	112.500
LANE 2	CARS	348	92.446	9.892	62.068	126.317
	H.G.V.	98	83.440	10.596	62.608	108.270
	AL VEH.	446	90.467	10.689	62.068	126.317
LANE 3	CARS	321	107.270	9.624	81.818	126.316
	H.G.V.	-	-	-	-	-
	ALL VEH.	321	107.270	9.624	81.818	126.316
TAPE 1B						
LANE	VEH. TYPE	SAMPLE	MEAN (kph)	S. DEV. (kph)	MIN.	MAX.
LANE 1	CARS	120	77.978	12.417	51.43	108.907
	H.G.V.	127	74.862	10.035	53.334	98.932
	ALL VEH.	947	76.376	11.339	51.43	108.907
LANE 2	CARS	244	91.192	10.031	62.608	119.980
	H.G.V.	62	81.889	10.123	55.148	103.680
	ALL VEH.	306	89.307	10.710	55.148	119.980
LANE 3	CARS	224	108.646	9.748	74.484	138.304
	H.G.V.	-	-	-	-	-
	ALL VEH.	224	108.646	9.748	74.484	138.304

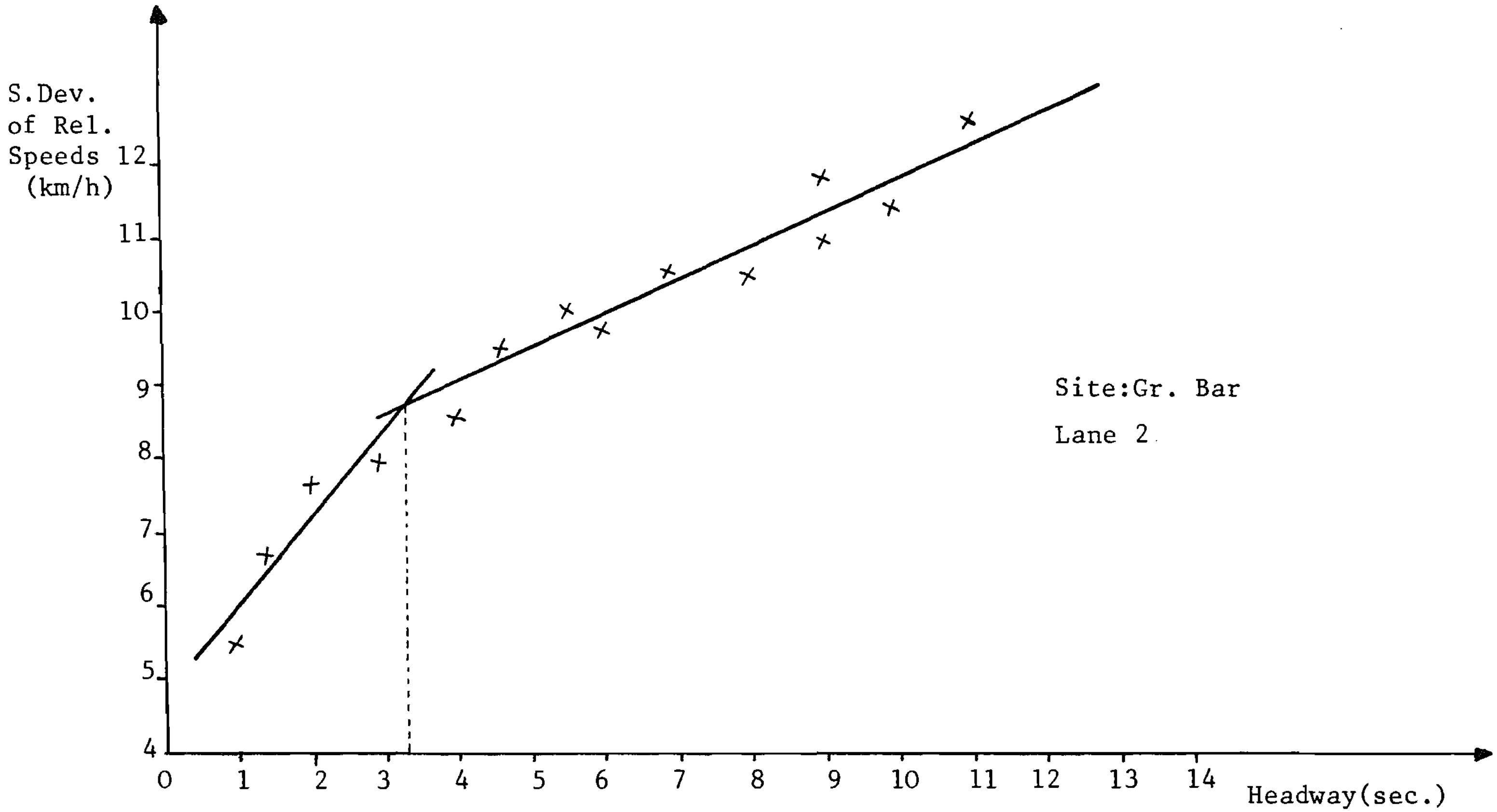


Figure 5.11:Determination of headway citerion T_c

TABLE 5.11

Sensitivity to the criteria for the Determination of the free speed distribution.

Tape 8 (site M5/M6)

LANE 1

T_C	ℓ	SAMPLE	MEAN (kph)	S. DEV. (kph)	X^2	$X^2_{0.95}$
3.00	1.50 2.00 2.50	296 290 286	81.500 81.537 81.583	10.421 10.298 10.266	21.19 11.86 17.31	22.36 12.59 21.03
3.50	1.50 2.00 2.50	280 274 268	81.491 81.530 81.663	10.421 10.291 10.248	11.023 15.87 14.36	12.59 22.36 21.03
4.00	1.50 2.00 2.50	271 264 257	81.409 81.364 81.449	10.442 10.298 10.256	1.96 22.10 21.04	5.99 22.36 22.36

TABLE 5.11 (cont.)

LANE 2

T_C	ℓ	SAMPLE	MEAN (kph)	S. DEV. (kph)	X^2	$X^2_{0.95}$
3.00	1.50	333	91.035	9.649	6.59	12.59
	2.00	325	91.143	9.162	3.26	12.59
	2.50	323	91.113	9.138	3.13	12.59
3.50	1.50	284	91.095	9.739	11.05	12.59
	2.00	275	91.177	9.151	2.26	11.07
	2.50	273	91.141	9.123	2.21	11.07
4.00	1.50	232	91.429	9.911	10.23	12.59
	2.00	223	91.543	9.198	2.86	11.07
	2.50	221	91.502	9 165	2.90	11.07

LANE 3

T_C	ℓ	SAMPLE	MEAN (kph)	S. DEV. (kph)	X^2	$X^2_{0.95}$
3.00	1.50	272	105.148	8.282	17.74	18.30
	2.00	247	105.278	7.885	5.74	9.48
	2.50	241	105.337	7.720	5.81	7.81
3.50	1.50	247	105.218	8.513	9.75	11.07
	2.00	220	105.358	8.135	5.14	9.48
	2.50	214	105.426	7.961	5.13	7.81
4.00	1.50	218	104.789	8.682	9.85	11.07
	2.00	190	104.911	8.303	5.51	9.48
	2.50	181	104.864	8.094	6.97	9.48

TABLE 5.12

Free speeds of Motorway Vehicles (kph)

Site: M5/M6 Ray Hall.

TAPE	LANE	SAMPLE	MEAN	S. DEV.	MIN.	MAX.	X^2	$X^2_{0.95}$
7	LANE 1	351	82.503	10.618	52.747	118.033	10.81	12.59
	LANE 2	437	91.304	10.755	64.573	135.850	14.38	16.92
	LANE 3	293	109.533	9.568	74.227	133.333	12.19	12.59
	ALL LANES	1081	93.387	14.797	52.747	135.850		
8	LANE 1	274	81.530	10.291	61.726	114.286	15.78	22.36
	LANE 2	275	91.177	9.151	68.246	122.034	2.26	11.07
	LANE 3	220	105.358	8.135	75.393	133.333	5.14	9.48
	ALL LANES	769	91.797	13.285	61.277	133.333		

TABLE 5.13

Free speeds of Motorway Vehicles (kph)

Site: Great Bar.

TAPE	LANE	SAMPLE	MEAN (kph)	S. DEV. (kph)	MIN.	MAX.	X^2	$X^2_{0.95}$
1A	LANE 1	198	79.857	12.137	54.735	112.500	5.42	12.59
	LANE 2	179	92.318	10.155	62.608	126.316	2.983	14.07
	LANE 3	137	108.915	9.71	81.818	126.316	10.575	16.919
	ALL LANES	514	91.941	15.833	54.375	126.316		
1B	LANE 1	150	79.319	12.085	53.333	108.908	10.75	14.07
	LANE 2	151	91.713	10.088	62.608	119.980	4.33	15.51
	LANE 3	101	109.912	9.893	77.605	138.304	14.81	15.51
	ALL LANES	402	91.657	16.069	53.333	138.304		

TABLE 5.14

Free speeds of Motorway Vehicles

Site:M6 Southbound Merge.

LANE	TAPE	VEH.TYPE	SAMPLE	MEAN (kph)	S.DEV. (kph)	MIN.	MAX.	X^2	$X^2_{0.95}$
LANE 1	3	CARS	96	80.255	12.085	57.341	108.832	7.94	9.48
		H.G.V.	63	71.002	8.686	54.695	88.146	4.40	11.07
		ALL VEH.	159	76.589	11.749	54.695	108.832	7.84	11.07
	2	CARS	129	85.248	13.050	57.035	115.931	9.65	12.59
		H.G.V.	95	74.690	10.036	55.170	103.35	3.35	11.07
		ALL VEH.	224	80.716	13.002	55.170	115.931	11.08	15.51
LANE 2	3	CARS	123	100.424	11.311	71.104	128.502	2.24	11.07
		H.G.V.	10	76.952	11.484	63.112	95.227	-	-
		ALL VEH.	133	98.660	12.818	63.112	128.502	3.69	12.59
	2	CARS	83	97.613	10.383	75.110	130.068	4.07	9.48
		H.G.V.	10	79.014	10.421	64.434	98.755	-	-
		ALL VEH.	93	95.613	11.843	65.434	130.068	5.28	11.07

TABLE 5.15

Free speeds of Motorway Vehicles

(Tapes 3 and 2 combined).

LANE	VEH.TYPE	SAMPLE	MEAN (kph)	S.DEV. (kph)	MIN.	MAX.	X^2	$X^2_{0.95}$
1	CARS	225	83.118	12.860	57.035	115.931	13.34	14.06
	H.G.V.	158	73.579	9.705	54.695	103.550	5.02	16.92
	ALL VEH.	383	79.183	12.562	54.695	115.931	24.39	25.00
2	CARS	206	99.292	11.008	71.104	130.068	5.95	12.59
	H.G.V.	20	77.985	10.725	63.112	98.755	3.56	5.02
	ALL VEH.	226	97.406	12.526	63.112	130.068	13.95	14.07

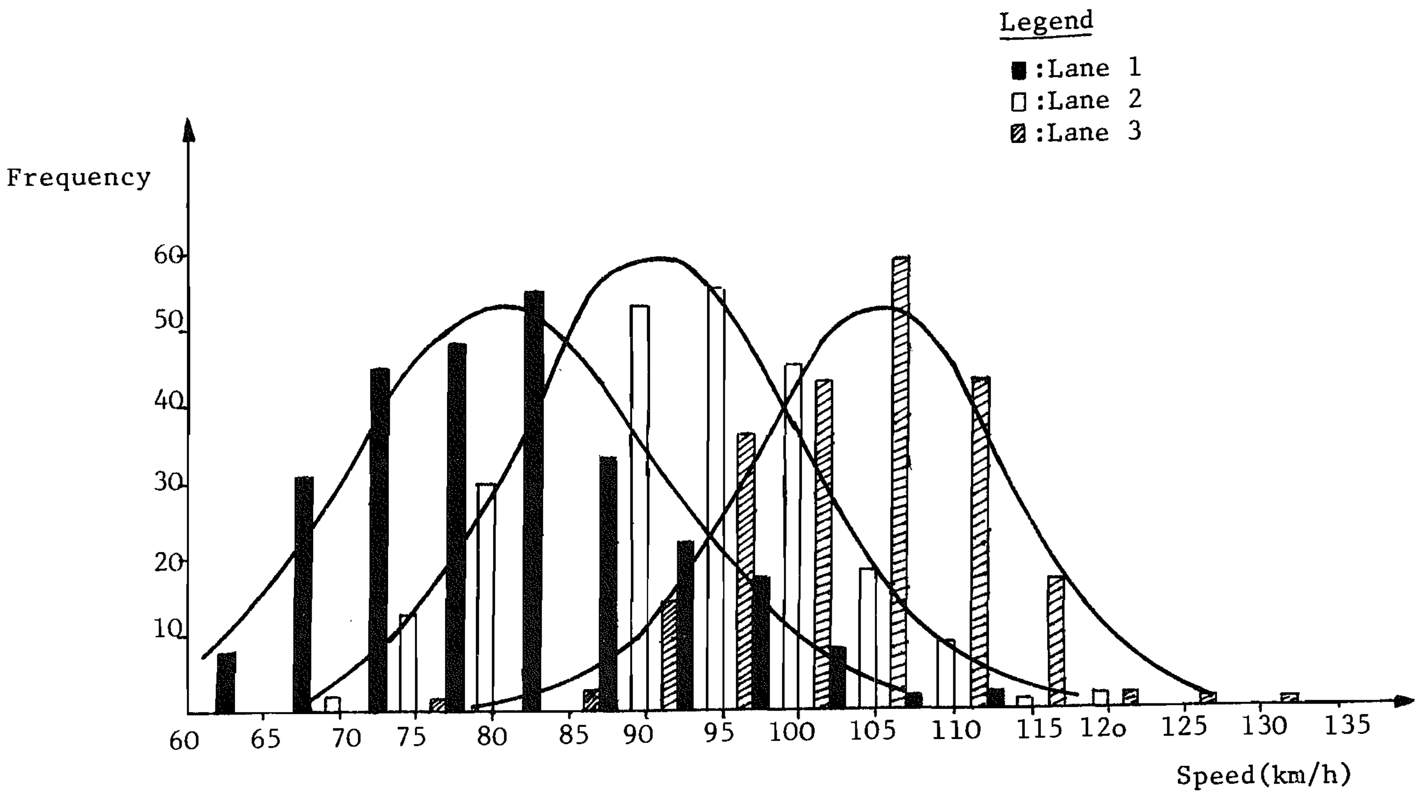


Figure 5.12:Free Speed distributions of Motorway Vehicles
(Site:M_5/M_6,tape 8)

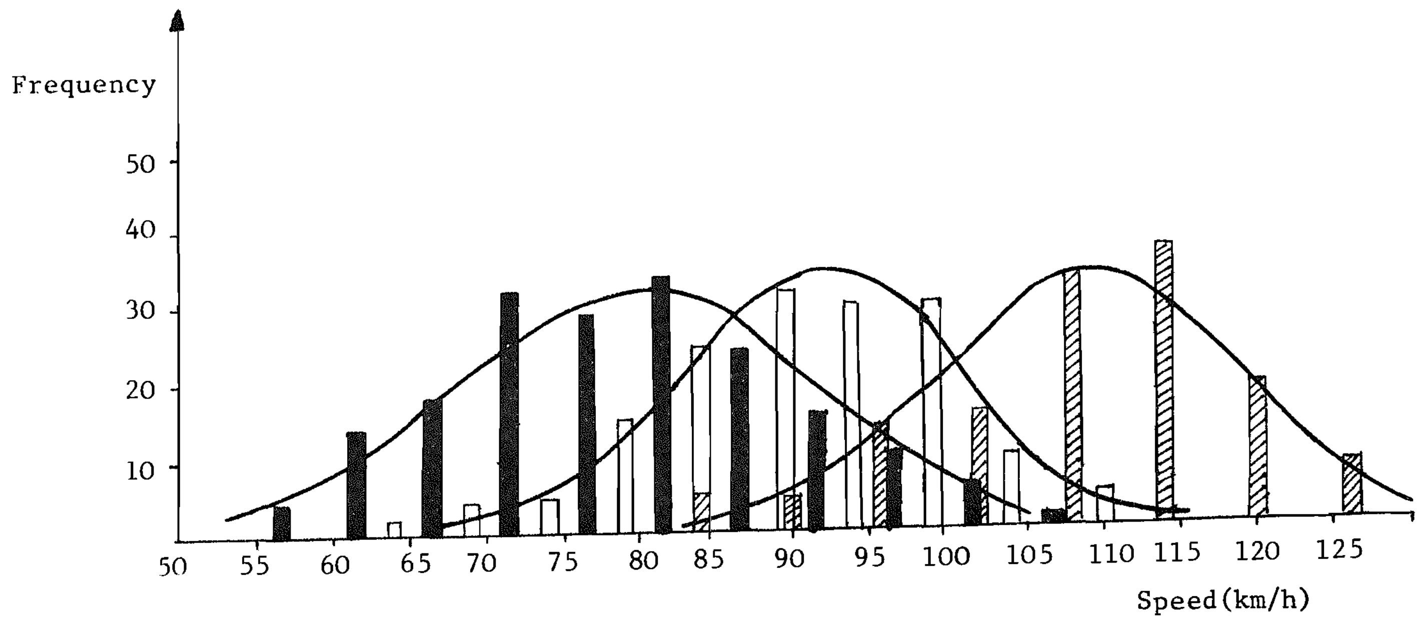


Figure 5.13:Free Speed distributions of Motorway Vehicles
(Site:Gr.Bar,Tape 1A)

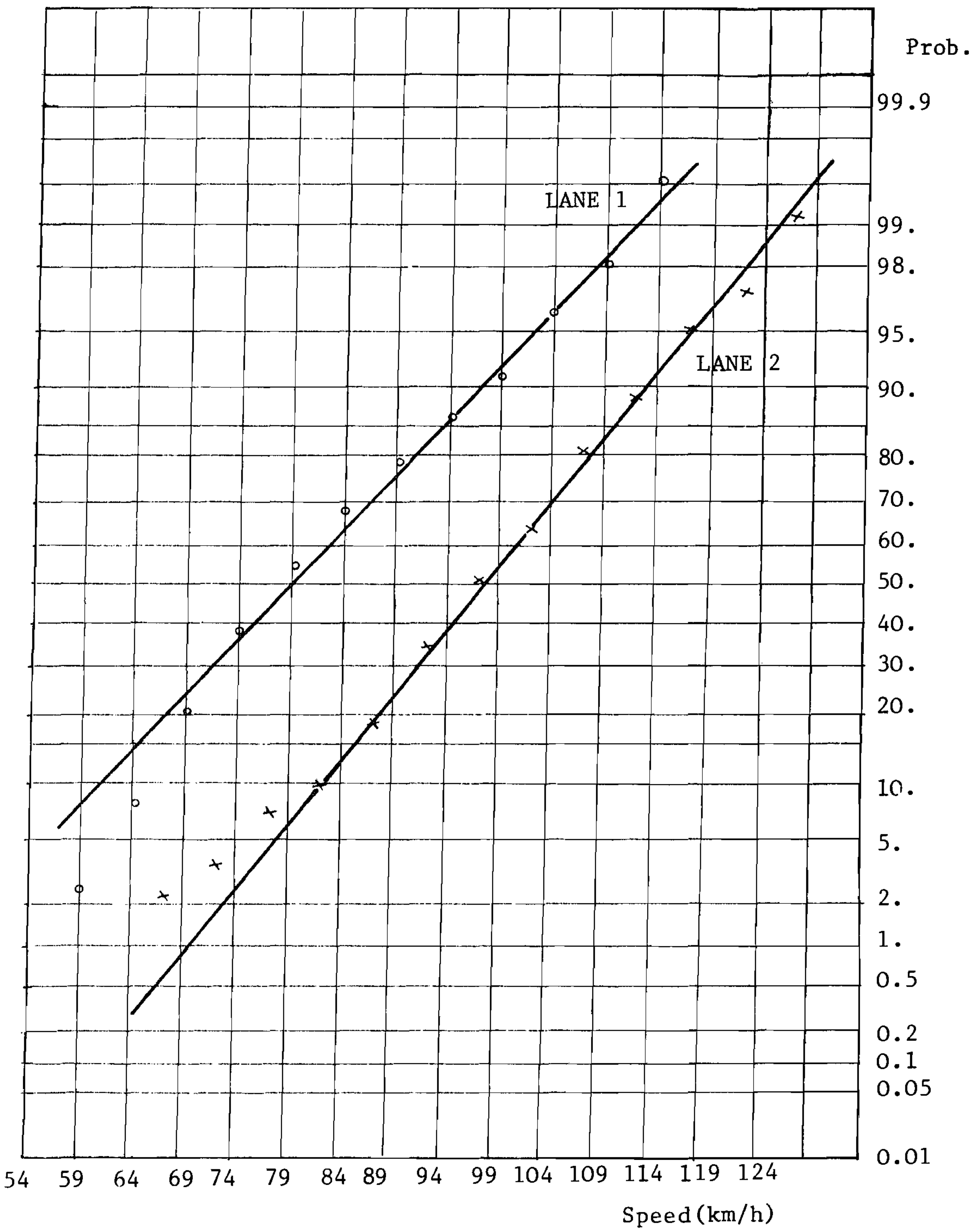


Figure 5.14:Free speeds of Motorway Vehicles(Site:M6,Southbound merge). Fitted Normal distributions.

5.6.2 The Entering Stream and the Merge

Data on the merging process were analysed in order to estimate parameters for the model and to investigate factors affecting the merging behaviour. From the available data base, information on most traffic variables has been extracted for two sites the M5/M6 Ray Hall interchange, and the Great Bar entry to M6 interchange. Details of the sites geometry is given in Fig. 5.15, 5.16. Additional data has also been collected at other sites, and also observations have been made to the whole data base available.

a. Flows and Arrival Pattern : Average flows and vehicle compositions were estimated as averages of classified minute counts, and the headways were calculated as time differences of vehicle arrivals approximately 20 m. from the ramp nose. Using the method described in Section 5.6.1, constant flow periods were determined and the obtained headway samples were subsequently analysed. Typical values are given in table 5.16.

It can be seen that in all sites we have a high ramp flow, and there is a high proportion of small headways in the range of 2-3 sec., as shown in the plotted headway distributions in Figs. 5.17 - 5.19. Various statistical distributions have been fitted to the data such as negative exponential, lognormal, double exponential and hyperlang model and the best fit was obtained using the hyperlang distribution, having the form given in Section 4.4.3b.

The proportion of restrained headways ϕ was estimated by fitting exponential distribution to the tails of the overall headway distribution using graphical and analytical techniques. In most of the samples the exponential distribution was accepted for headways greater than a value in the range of (3.00 - 3.50) sec., and the probability of headways less than that value gives the ϕ.

Once the mean γ_1 of the distribution of free headways is calculated, the mean γ_2 of the distribution restrained headways is estimated from the relationship:

$$m = \phi\gamma_2 + (1 - \phi)\gamma_1$$

The rest of the parameters were estimated using the method of minimum chi-square. In most of the cases the value of the Erlang parameter K was

$$K = 2$$

and the minimum headways δ_1, δ_2 had values:

$$\delta_1 = 1.00 \text{ sec.}$$

$$\delta_2 = 0.50 \text{ sec.}$$

Tables 5.17 gives the parameters of the distribution for each sample and in Fig. 5.17 - 5.19, the observed and fitted distributions are plotted.

b. Gap Acceptance : Data on gap acceptance were measured as described in Section 5.3.b. The subdivision into lead and lag times accepted at the point of entry was adopted as more realistic than a 'static' analysis where gap acceptance of the series of mainstream headways is measured at a fixed point, conventionally the ramp nose. The dynamic approach followed by Worall (Ref. 25) also attempts to take into account the relative motions and paths of the merging vehicles.

The analysis of the data, excluded values greater than 9 sec., because it was assumed that above that threshold , there is no interference between two following vehicles (Ref. 135).

Typical values such as mean, s. deviation, 50 percentile accepted, minimum and maximum are given in tables 5.18, 5.19 for the lead and lag times respectively, and in Fig. 5.20, 5.21 their frequency distributions are plotted.

It can be seen that both distributions exhibit positive skewness and the modal values are about 1 sec and 2 sec for lead and lag times respectively. Although the sites have different geometric characteristics there are no significant differences on the average values and the shape of the distributions.

Because on site Great Bar, there is a higher motorway inside lane flow (700 veh/h) than on site Ray Hall (500 veh/h), there is a higher proportion of drivers accepting smaller lags, as shown in Fig. 5.21.

On both sites about 45 per cent of the drivers accept a lead time of 1 sec., and that may be considered as a reasonable estimate of the minimum dynamic lead time required by a merging vehicle. The minimum values observed were 0.50 sec for lead and 0.88 sec for lag.

The effect of the relative speed on the gap acceptance was also examined, the relative speed defined as:

$$RV = V_{M1} - U$$

where :

V_{M_1} : speed of the interacting mainstream vehicle.

U : speed of the merging vehicle.

Tables 5.20, 5.21 give the typical values for accepted lag and lead times for three ranges of relative speeds for both sites and Fig. 5.22, 5.23 show the probability distributions of accepted leads and lags. As far as lag times are concerned, as RV increases the proportion of drivers accepting smaller gaps decreases, indicating that the faster the merging vehicle, the higher the probability of accepting a smaller lag. The difference in the distributions for RV $\geq$ +5 mph, in the two sites is due to the presence of a significant proportion of lags in the range of 7 sec., in site Ray Hall.

The reverse effect is shown for the lead times (Fig. 5.23), the smaller leads accepted by drivers having small relative speeds, whereas the fast merging vehicles (RV $\leq$ - 5 mph) require larger leads, as has been also reported elsewhere (Ref. 101).

Distribution of Accepted Lags : In order to have a statistical representation of the accepted lags different distributions were fitted to the data, such as normal, exponential, Gamma and lognormal distribution. The best fit was obtained with the lognormal distribution, which was acceptable at all the sites and samples, as shown in table 5.20.

The probit method was also used in the analysis of accepted lags. According to that method (Ref. 130), the lognormal function is transferred into the linear form:

$$Y = \alpha + bx$$

where :

x : $\log(t)$, t : lag accepted

Y : Probit of P, P probability of accepting a lag, defined as the absissa which corresponds to the probability of P in a normal distribution $N(5.0, 1.0)$. Accordingly the probit of the expected proportion accepting a time lag is related to the time lag by the following equation:

$$Y = 5.00 + \frac{1}{\sigma}(x-\mu)$$

where :

μ, σ : mean and s. deviation of x .

Probits were calculated and straight lines were fitted to the data points, for the three relative speed classes, as shown in Fig. 5.24 for the site Ray Hall. The lag accepted at 50 per cent can be estimated from x when $Y = 5.00$.

The method of analysis of gap acceptance data cannot obviously identify the point of decision for a driver, nor the perceived gap at that point. As it has also been pointed out the gap rejection is very difficult to be readily identified, and due to the medium mainstream flow merging vehicles accept directly large available gaps, the determination of the critical values is complicated.

On the other hand, the analysis has given valuable information about the driver behaviour and how it is affected by certain parameters.

c. Speeds and Accelerations : Speeds of merging vehicles were estimated, recording their arrivals at specific points on the approach to the junction and on the acceleration lane. Acceleration rates were computed as average values over the distance travelled by the slip-road vehicle during the merging manoeuvre. The entry speeds were calculated at a distance approximately 20 m. from the ramp nose and the results are summarized in table 5.22. It can be seen that geometric configuration has a marked effect on the speeds. The high speeds at the site Ray Hall is because the offside lane of the slip road was recorded, whereas at Great Bar is a typical one lane slip road.

The normal distribution was found to fit the observed data, and the speed distributions on both sites are plotted in Fig. 5.25 on normal probability paper.

The free speed distribution of merging vehicles at nose was also calculated by applying criteria similar to those for the determination of the free speeds on the motorway, and also that the accepted lags should be greater than 9 sec. in order to avoid speed adjustments. The parameters of the free speed distributions for the site Ray Hall are:

$$m = 82.88 \text{ (kph)}$$

$$\sigma = 8.28 \text{ (kph)}$$

and the speeds are normally distributed. The mean value is almost identical with the mean value of the motorway inside lane speeds.

The 'free' speeds of vehicles at a distance approximately 100 m from the start of the slip road were also measured at a site, with two lane slip roads and normal distributions were fitted to the data. Table 5.23 gives typical values of the observed speeds. The mean value (64.963 $\simeq$ 40.35 mph) is similar with that measured by Ackroyd (Ref. 101) at motorway interchanges.

The acceleration rates were analysed by grouping the data into 10 kph speed ranges for all the rates and for accelerations and decelerations separately. The results are shown on tables 5.24, 5.25, 5.26. Normal distributions were fitted to all values (accelerations - decelerations) and lognormal distributions to accelerations rates only, per each speed interval, as it is shown in Fig. 5.26.

In table 5.27 it is shown that vehicles have higher accelerations as their speed increases. These relationships are based on the average speeds and acceleration rates for each speed interval , and these are not directly related to each individual vehicle.

<u>Acceleration Rates and Merging Process</u> : The relationship between acceleration and gap acceptance was investigated by grouping the data based on criteria according to the merging process and traffic interaction on the slip road. Four vehicle types were classified as follows :

i. Free vehicles : Vehicles which are unimpeded during their manoeuvre, i.e. not restrained from other ramp vehicles in front and merge into a large gap. These vehicles are considered to adopt their desired acceleration during the manoeuvre.

ii. Restrained-free entry : Vehicles which have a small headway with merging vehicles in front (queueing) and merge into large gaps.

iii. Free-forced entry : Vehicles which are unimpeded from other ramp vehicles, but accept small lags.

iv. Restrained-forced entry : Queueing vehicles accepting small lags.

The values of time headway and acceptable lag, for which we have no interaction are chosen to have the typical values of 3.5 sec. and 9 sec. respectively.

The results from the analysis are shown in table 5.28. It can be seen that single merging vehicles accepting small lags have on average higher acceleration rates. The decelerations of the free entry unimpeded vehicles can be explained due to their speed adjustments during the manoeuvre. The relationship between acceleration rate and accepted lag was examined and the following relationships were obtained :

$$A = 0.638 - 0.074 * L \quad (R = -0.161) \quad \text{for type iii vehicles}$$

$$A = 0.645 - 0.090 * L \quad (R = -0.166) \quad \text{for type iv vehicles}$$

where :

A : acceleration (m/s^2)

L : accepted lag(sec)

The above relationships indicate the existence of a negative correlation between acceleration and accepted lag, but additional data and analysis are need in order to establish a definitive relationship.

The distribution of desired acceleration was obtained excluding the deceleration rates and the parameters were estimated :

$$m = 0.626 \ m/s^2$$

$$\sigma = 0.451 \ m/s^2$$

and a lognormal distribution was fitted to the data (Fig. 5.27)
No definitive relationship was found to exist between speed and desired acceleration, the best obtained from the data had the form :

$$A = -0.634 + 0.056 * S \qquad (R = 0.263)$$

where :

S : speed (kph).

<u>d. Merging Paths</u> : The distances travelled by the ramp vehicles during the merging manoeuvres were measured with reference to the ramp nose and their frequency distributions were obtained for 10 m. intervals and plotted in Fig. 5.28. It can be seen that under the traffic conditions occurring, most of the vehicles have a short direct merge. In site Great Bar approximately 2 per cent of the vehicles merged before the nose, whereas the proportion of vehicles which merge beyond the end of acceleration lane was less than 1 per cent on both sites.

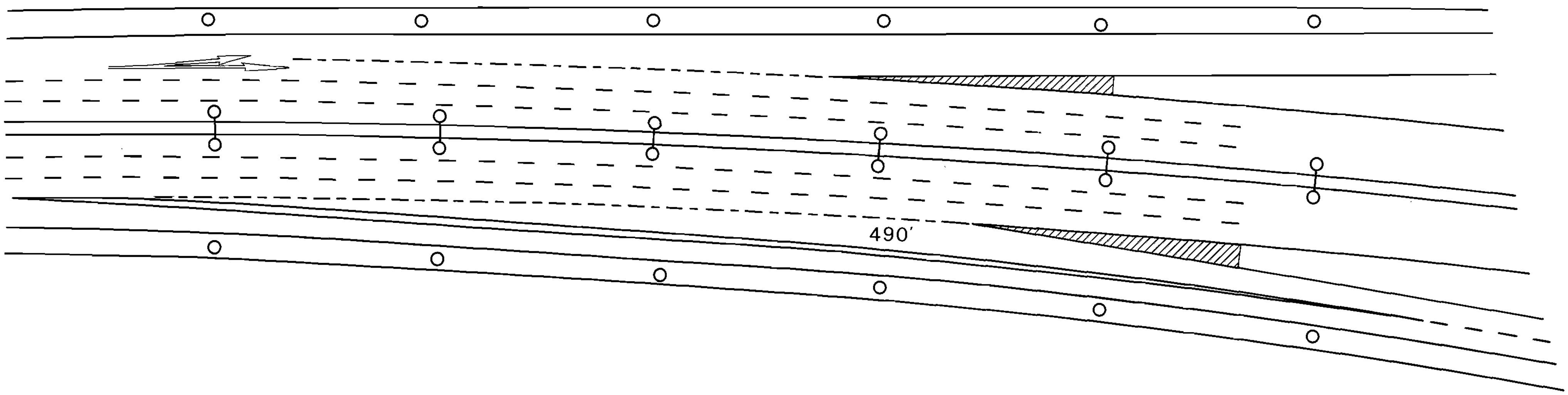


Figure 5.15 : Plan of site Ray Hall

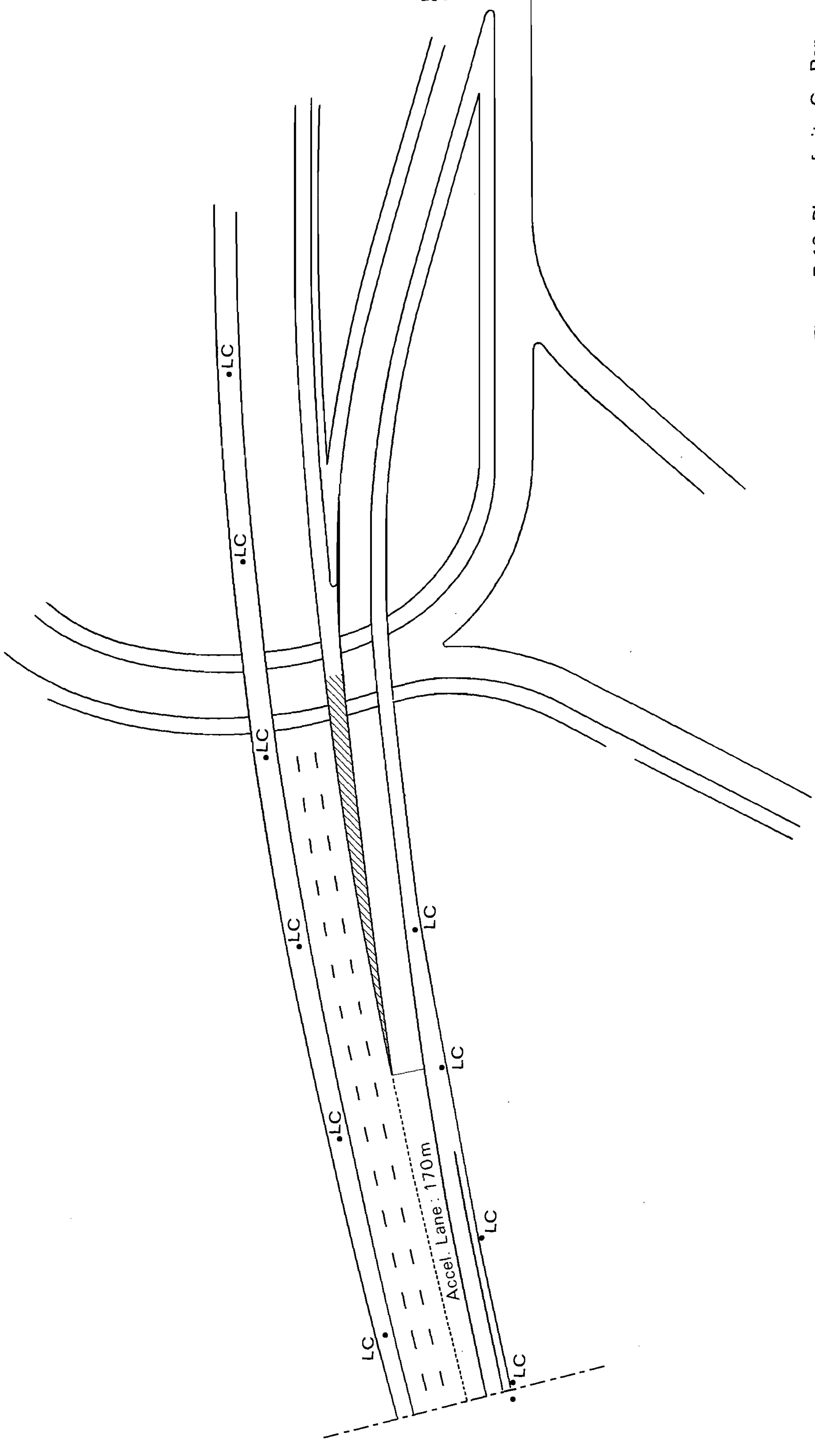


Figure 5.16: Plan of site Gr. Bar.

TABLE 5.16

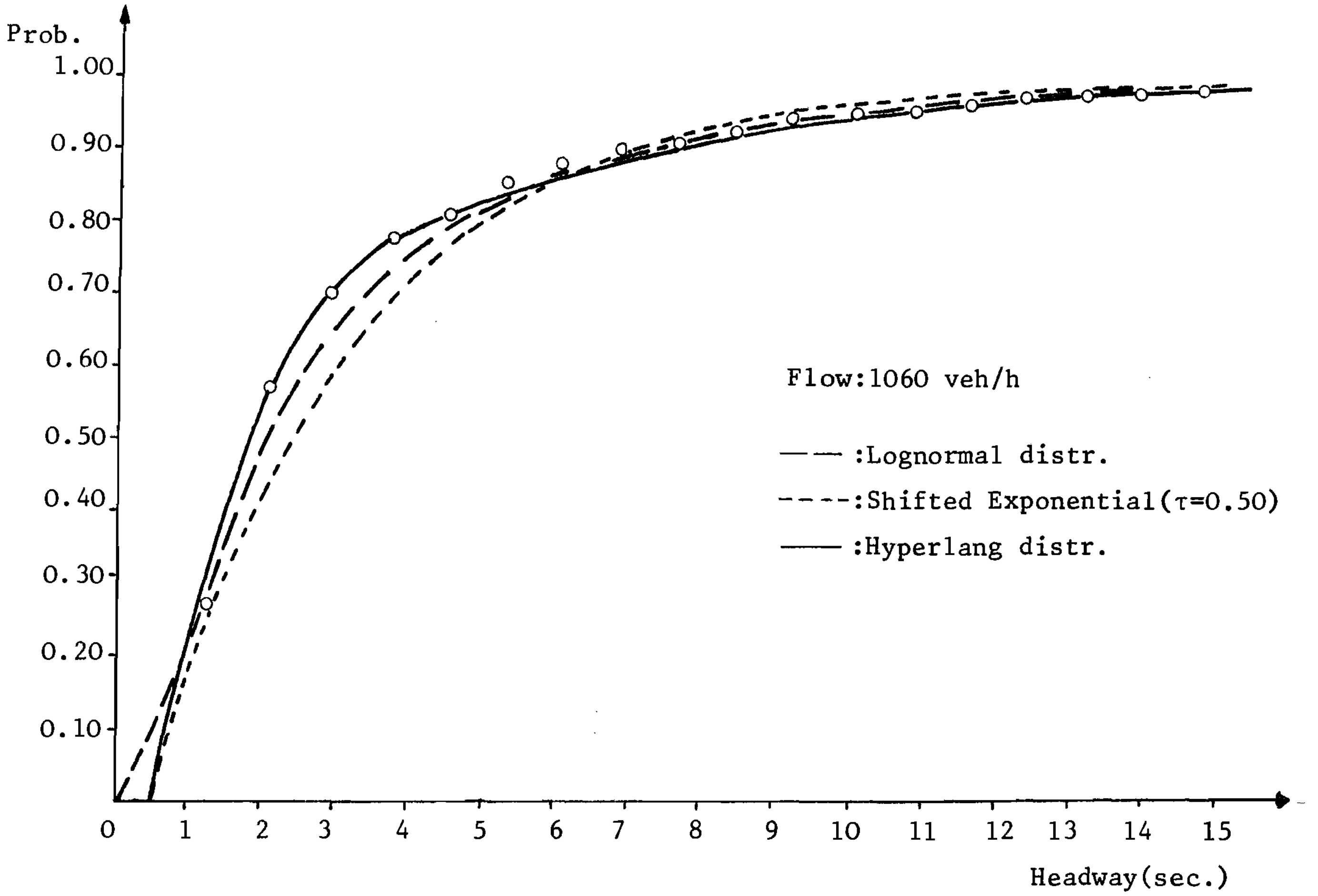
Headways of merging vehicles (sec)

SITE	TAPE	CONSTANT FLOW (vph)	% H.G.V.	SAMPLE	MEAN	S. DEV.	MIN.
M5/M6	7	912	6.67	877	3.948	4.414	0.52
Ray Hall	8	1100	6.10	768	3.271	3.926	0.46
Great	1A	1048	13.20	498	3.436	3.741	0.46
Bar	1B	960	7.79	384	3.746	4.256	0.48
Bentley	4	1064	11.30	931	3.382	3.284	0.40
South. merge	5	945	8.60	413	3.801	3.703	0.40

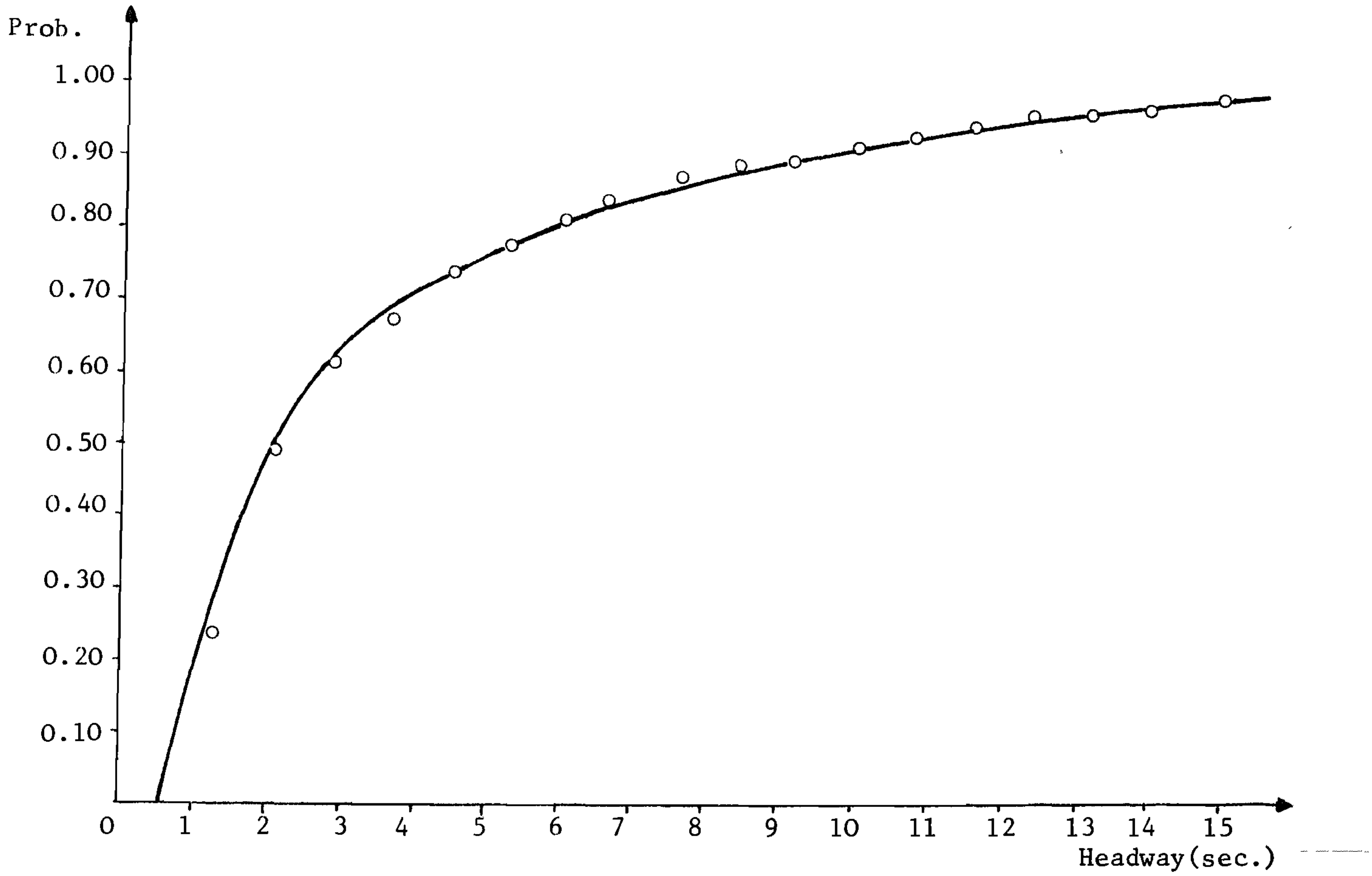
TABLE 5.17

Parameters of the Hyperlang distribution

SITE	TAPE	SAMPLE	ϕ	γ_1	δ_1	γ_2	δ_2	K
M5/M6	7	877	0.560	7.141	1.00	1.439	0.50	2
Ray Hall	8	768	0.645	6.613	1.00	1.431	0.50	2
Great	1A	498	0.729	8.166	1.00	1.677	0.50	2
Bar	1B	384	0.691	7.559	1.00	2.040	0.50	2
Bentley	4	931	0.693	7.054	1.00	1.755	0.50	2
South. merge	5	420	0.562	5.802	1.00	2.241	0.50	1



a. Tape 8



b. Tape 7

Figure 5.17:Headway distribution of Ramp Vehicles(Site:M5/M6,Ray Hall)

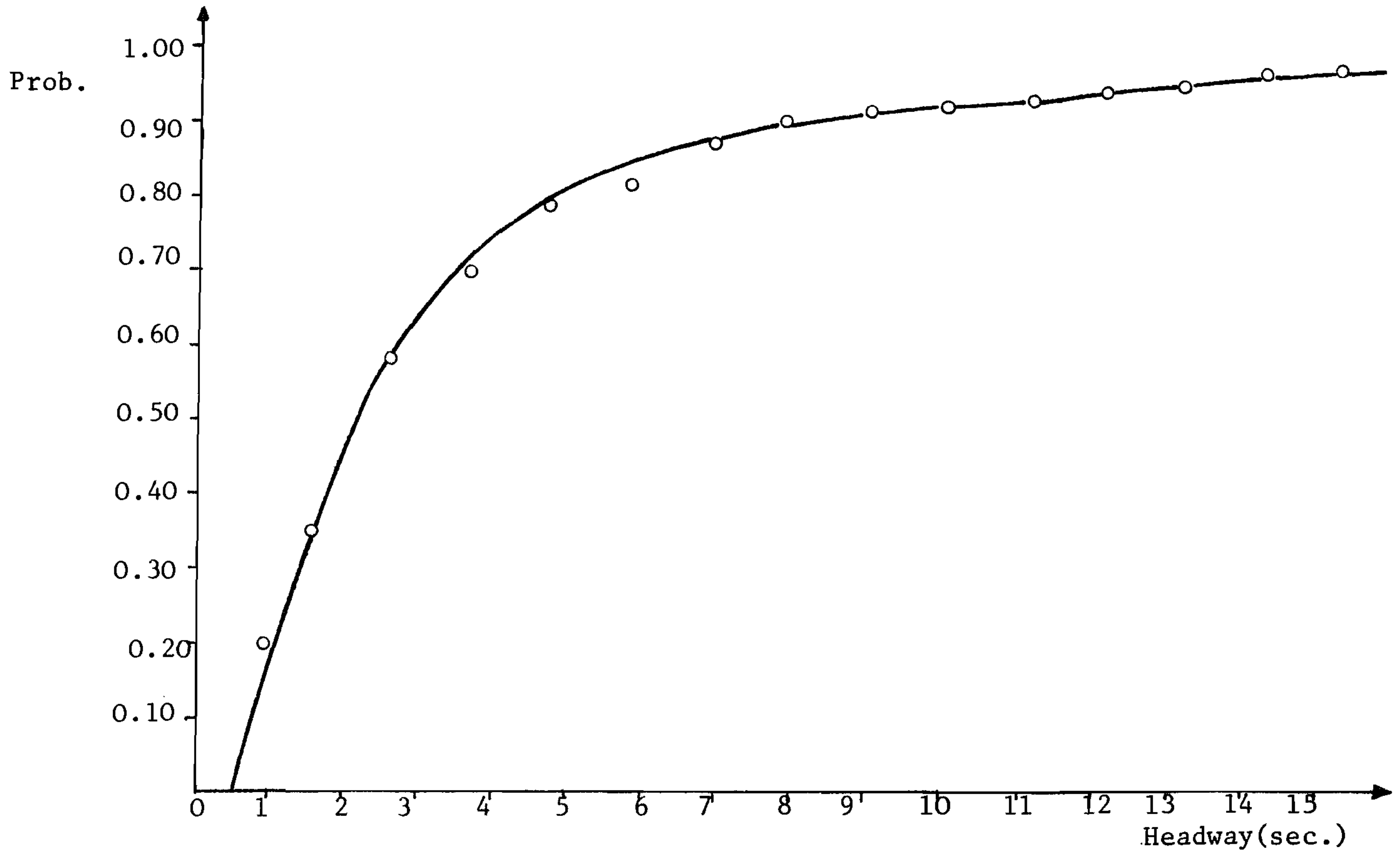


Figure 5.18: Probability distr. of Headways Of merging veh(Site:Gr. Bar,tape 1B)

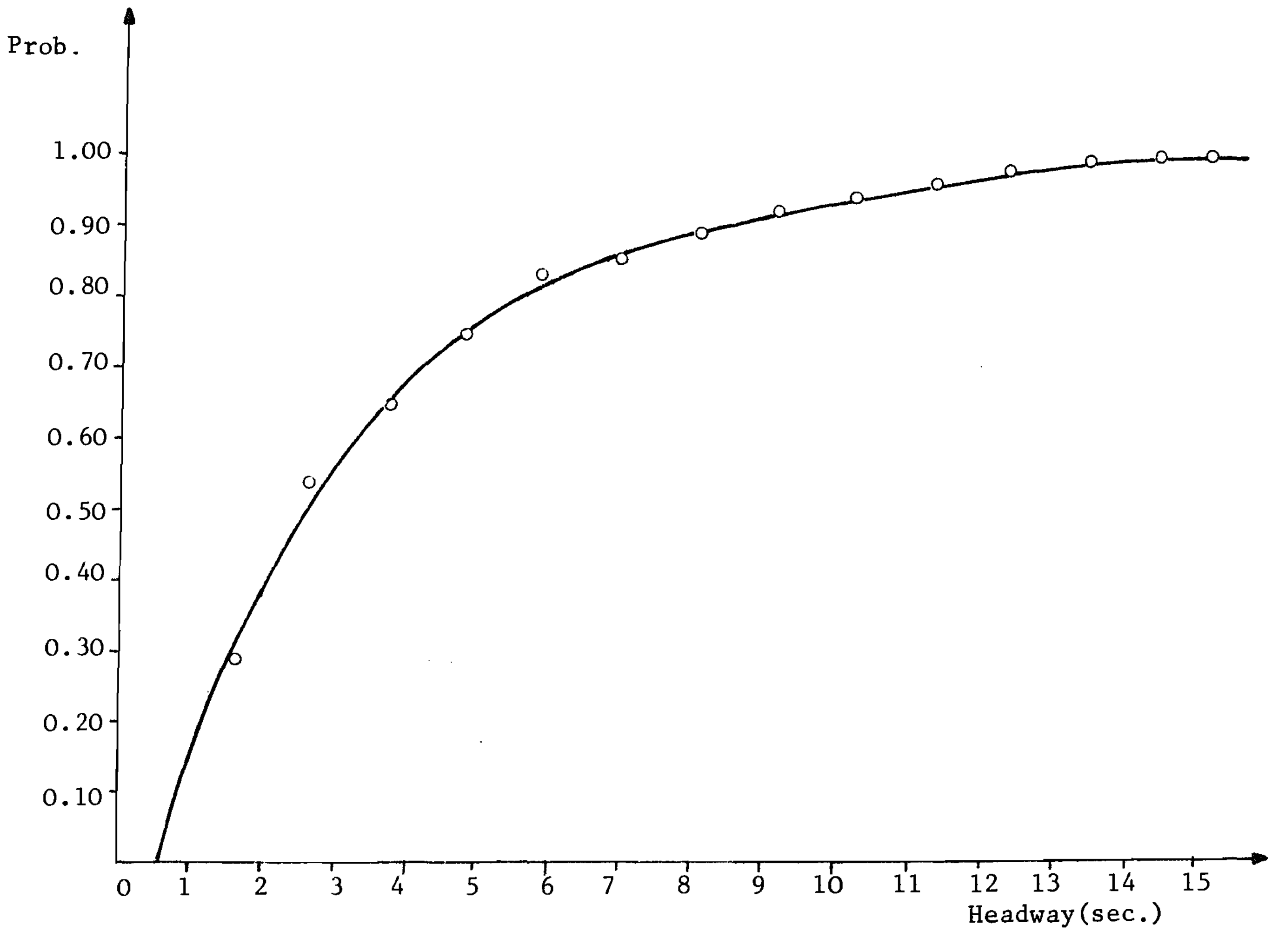


Figure 5.19: Probability distr. of Headways of merging veh(Site:Bentley)

TABLE 5.18

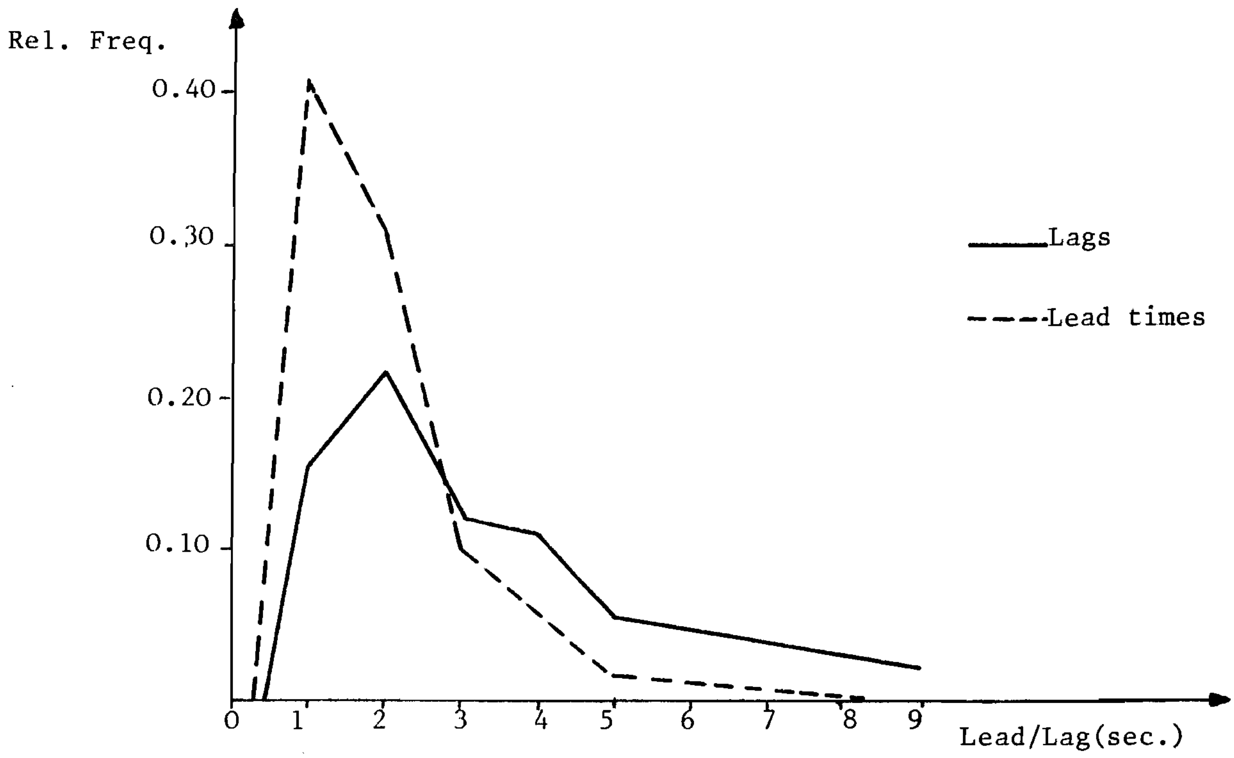
Accepted lags at the point of merge

SITE	TAPE	SAMPLE	MEAN (sec)	S.DEV. (sec)	50% (sec)	MINIMUM (sec)	MAXIMUM (sec)
M5/M6	7	604	3.959	2.275	3.26	0.88	8.96
Ray Hall	8	529	4.011	2.286	3.30	0.94	8.98
Great	1A	407	3.889	2.252	3.10	0.90	8.96
Bar (M6)	1B	318	3.924	2.238	3.28	0.96	8.98

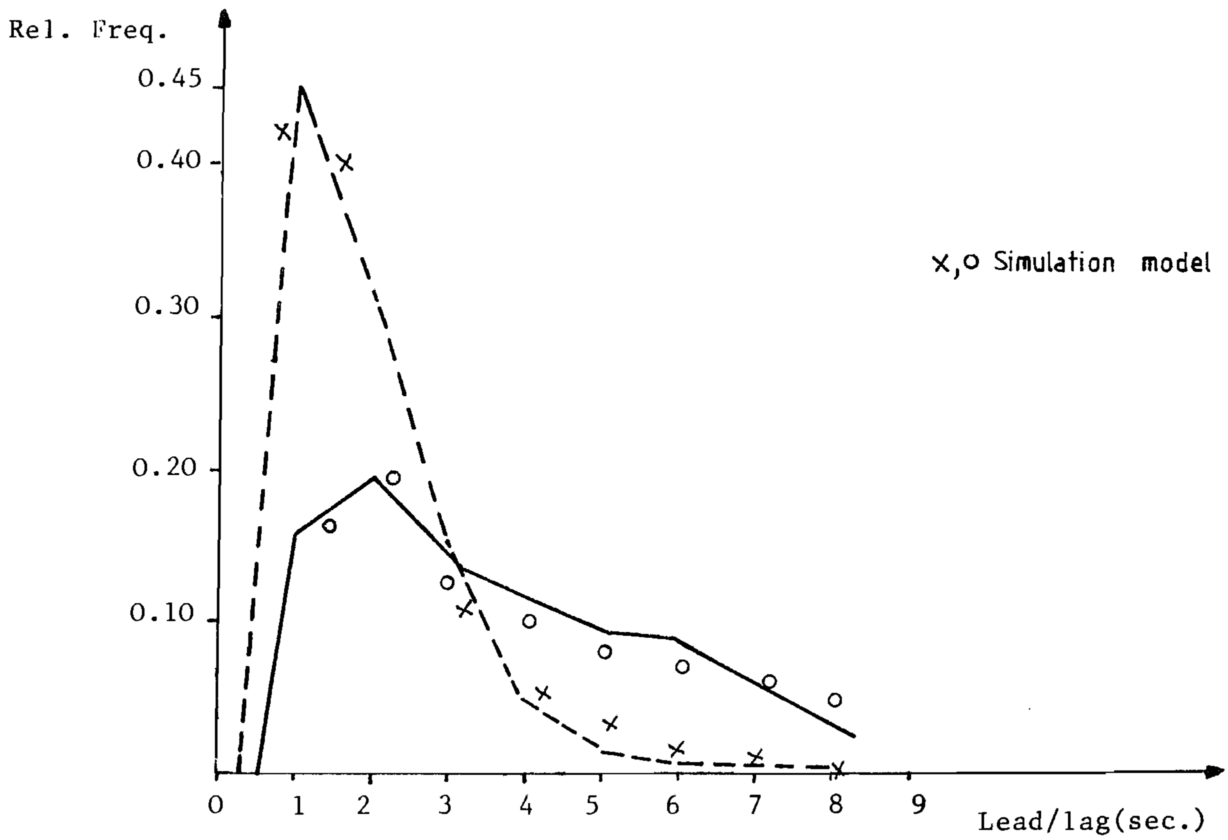
TABLE 5.19

Accepted lead times at the point of merge

SITE	TAPE	SAMPLE	MEAN (sec)	S. DEV. (sec)	50% (sec)	MINIMUM (sec)	MAXIMUM (sec)
M5/M6	7	824	2.200	1.606	1.48	0.50	8.86
Ray Hall	8	721	2.048	1.470	1.40	0.520	8.66
Great	1A	451	1.894	1.398	1.25	0.50	8.42
Bar (M6)	1B	142	2.190	1.710	1.32	0.50	8.50

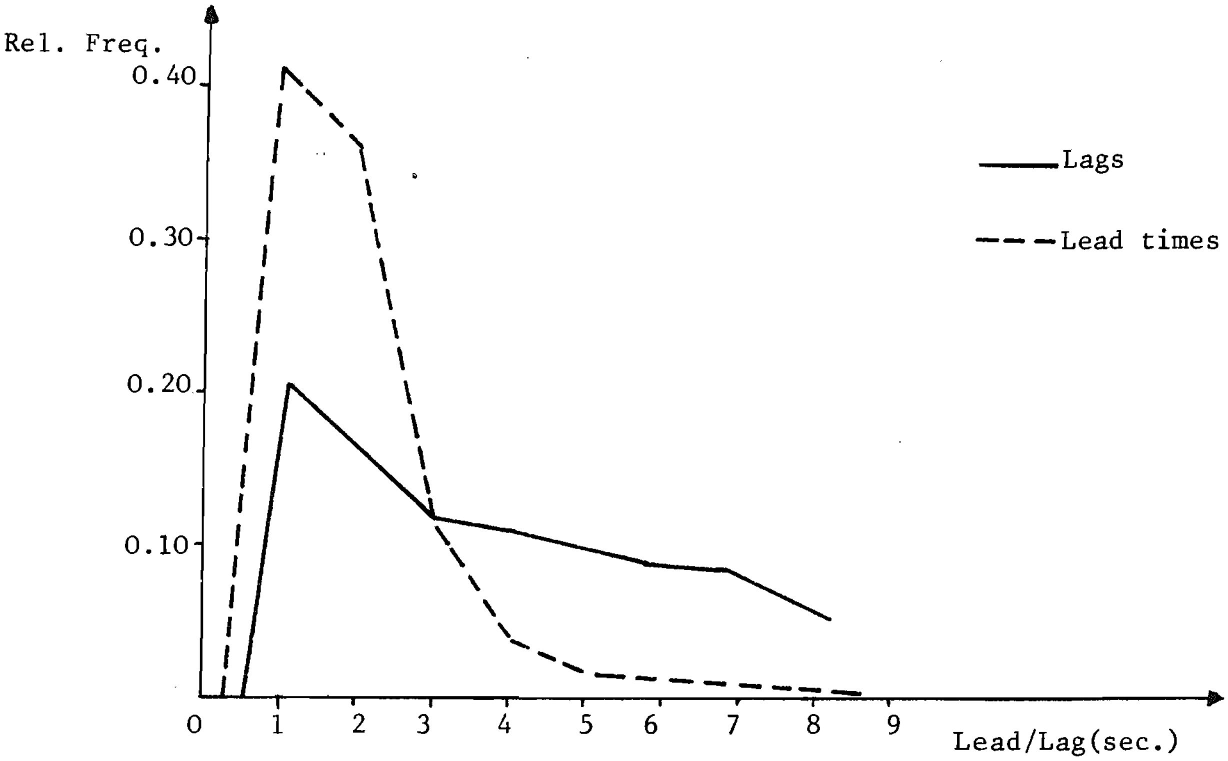


a. Tape 7

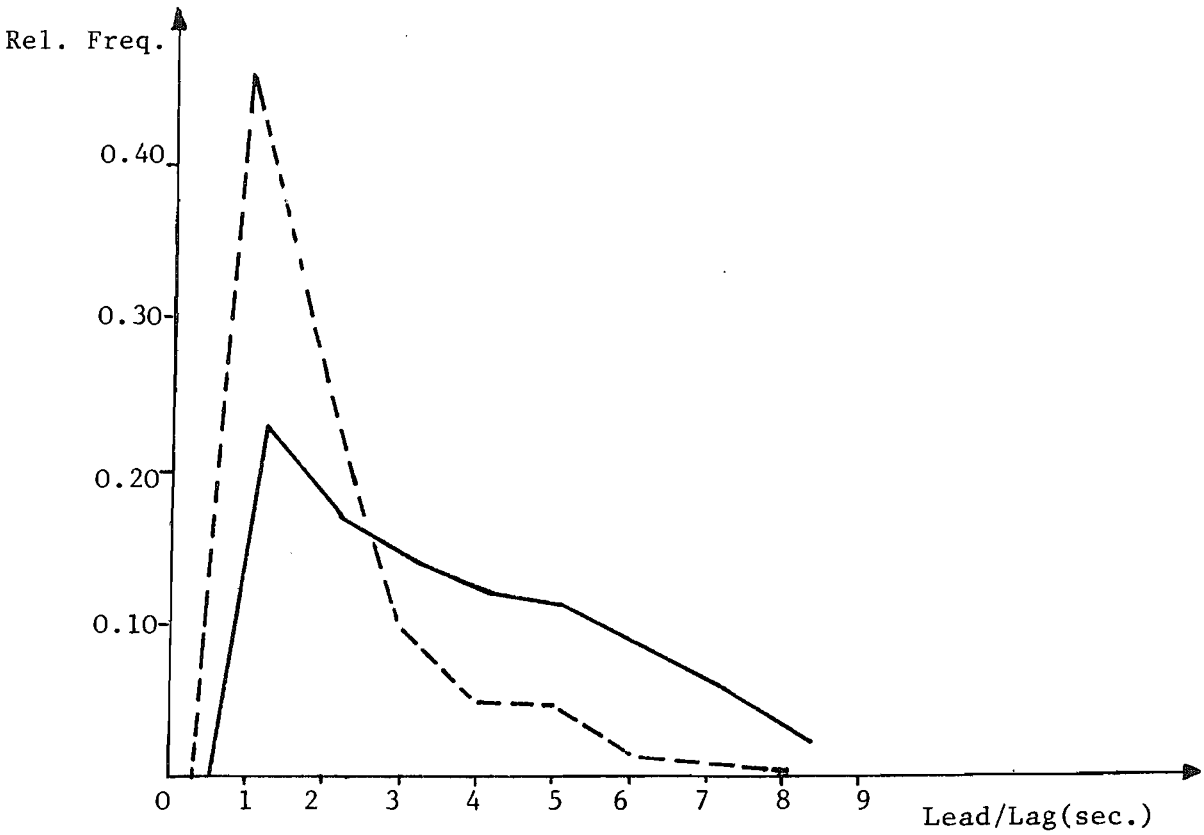


b. Tape 8

Figure 5.20:Distributions of accepted Lead and Lag Times(Site: Ray Hall)



a. Tape 1A



b. Tape 1B

Figure 5.21:Distributions of accepted Lead and Lag Times(Site:Gr. Bar)

TABLE 5.20

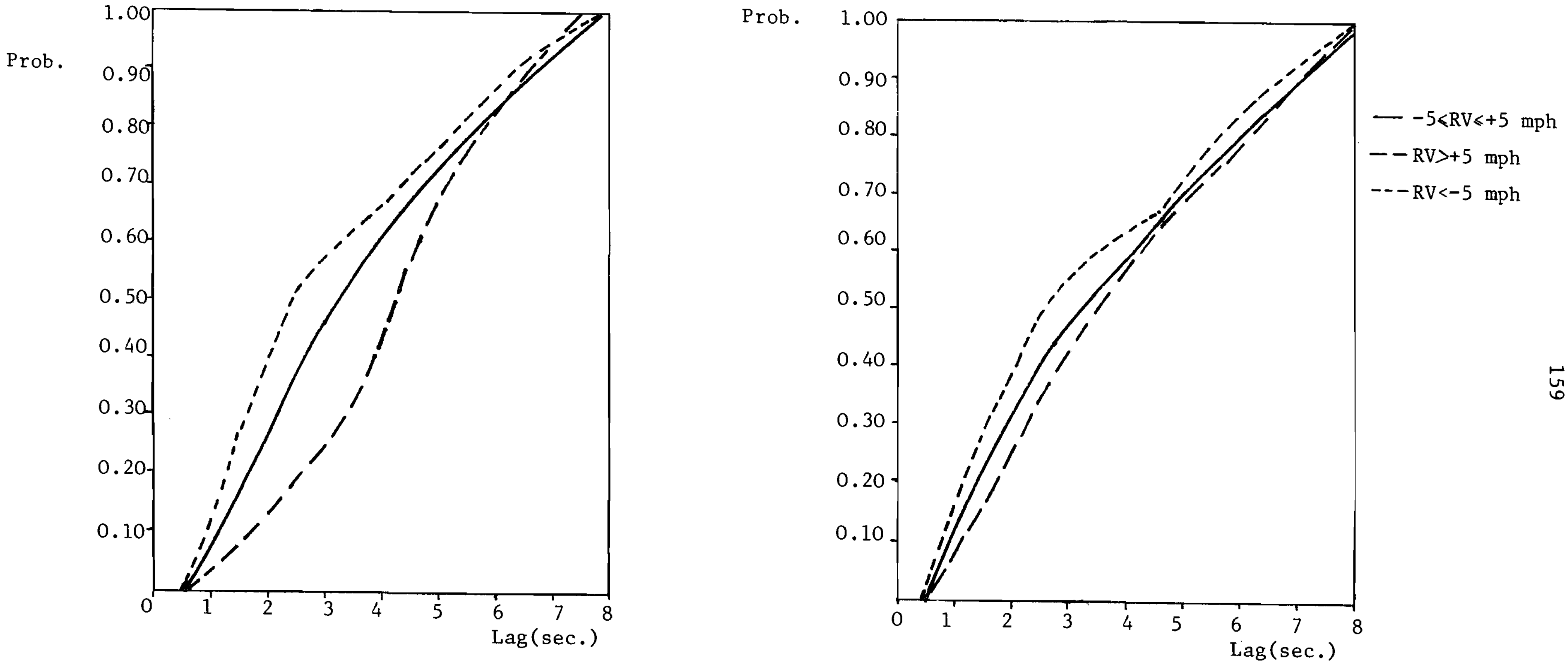
Effect of Relative speed on accepted lags.

SITE	REL.SPEED CLASS (mph)	SAMPLE	MEAN (sec)	S.DEV. (sec)	50% (sec)	MIN. (sec)	MAX. (sec)	X^2	$X^2_{0.95}$
M5/M6 Ray Hall	$-5\leq RV\leq +5$	237	3.98	2.251	3.15	1.00	8.98	19.51	19.68
	$RV\geq +5$	111	4.716	2.106	4.20	0.98	8.86	7.26	7.81
	$RV\leq -5$	181	3.621	2.348	2.80	0.94	8.88	4.44	7.81
Great Bar(M6)	$-5\leq RV\leq +5$	146	3.725	2.302	3.15	0.90	8.56	14.69	19.68
	$RV\geq +5$	558	3.954	2.229	3.55	0.96	8.94	10.39	11.07
	$RV\leq -5$	22	3.447	2.409	2.50	0.96	8.98	-	-

TABLE 5.21

Effect of Relative speed on accepted lead times

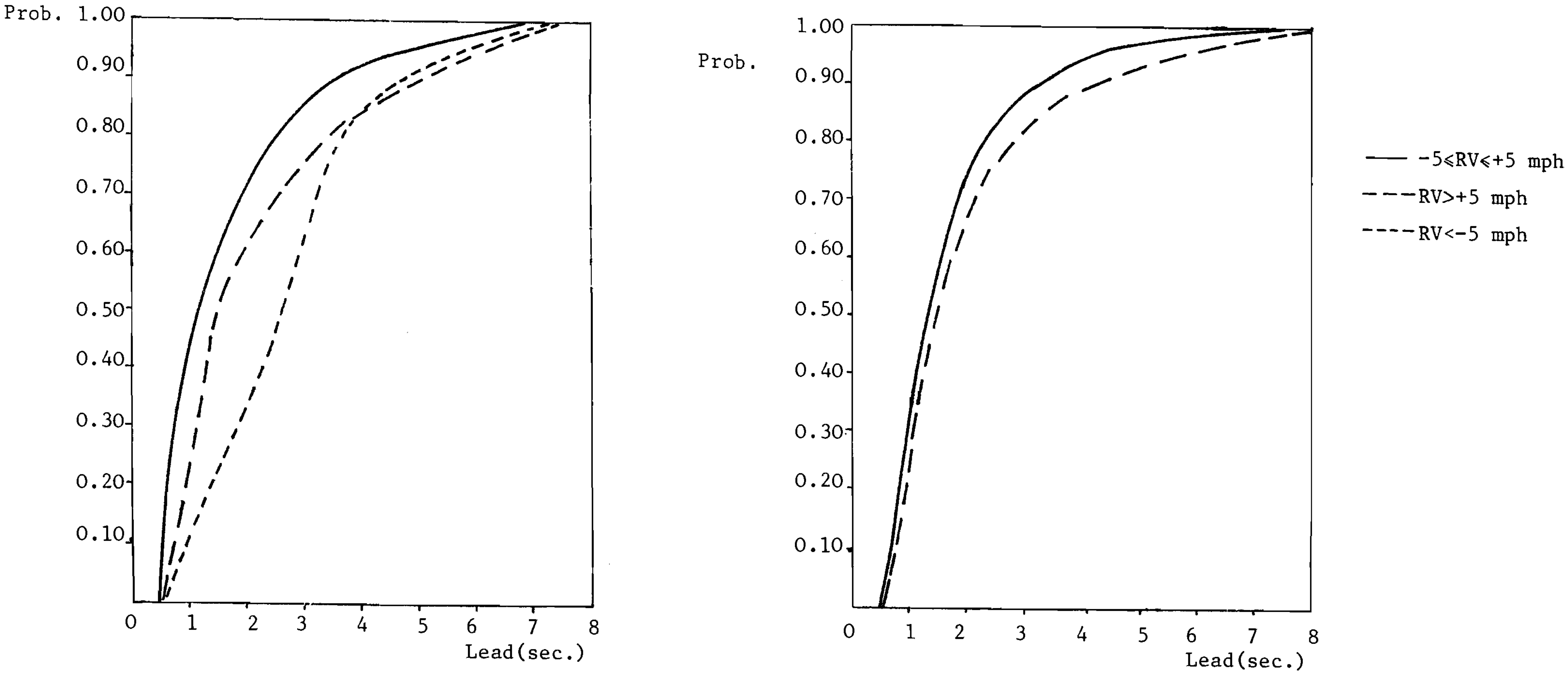
SITE	REL SPEED CLASS (mph)	SAMPLE	MEAN (sec)	S.DEV. (sec)	50% (sec)	MIN. (sec)	MAX. (sec)
M5/M6 Ray Hall	$-5\leq RV\leq +5$	109	1.821	1.415	1.20	0.60	7.40
	$RV\geq +5$	43	2.263	1.932	1.45	0.60	8.66
	$RV\leq -5$	65	2.909	1.722	2.60	0.70	8.54
Great Bar (M6)	$-5\leq RV\leq +5$	37	1.926	1.689	1.30	0.50	8.26
	$RV\geq +5$	254	1.963	1.534	1.45	0.50	8.34
	$RV\leq -5$	-	-	-	-	-	-



a. Site M5/M6, Ray Hall

b. Site Great Bar

Figure 5.22: Effect of Relative Speed on accepted Lags



a. Site M5/M6,Ray Hall

b. Site Great Bar

Figure 5.23:Effect of Relative Speed on accepted Lead Times

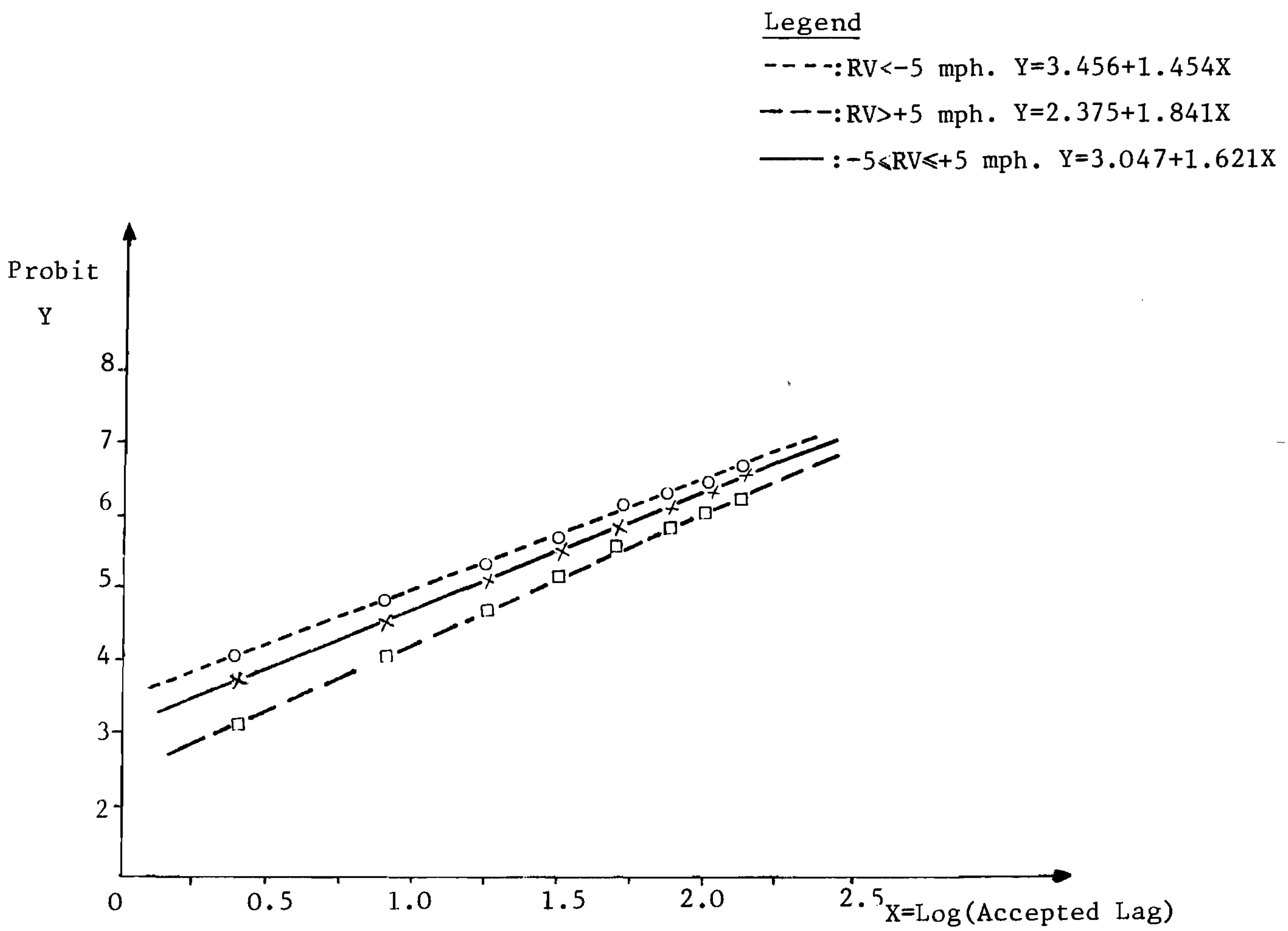


Figure 5.24:Probit Analysis of Accepted Lags (Site:M5/M6,Ray Hall)

TABLE 5.22

Speeds of Merging vehicles at nose (kph)

SITE	TAPE	AVERAGE FLOW(vph)	VEHICLE TYPE	SAMPLE	MEAN	S.DEV.	MIN.	MAX.
M5/M6 Ray Hall	7	910	CARS	831	79.207	10.479	51.82	109.44
			H.G.V.	50	74.692	6.93	59.48	87.70
			ALL	881	78.591	10.360	51.82	109.44
	8	1100	CARS	723	77.630	9.642	54.92	106.047
			H.G.V.	47	73.775	8.277	57.96	91.20
			ALL	770	77.394	9.604	54.92	106.047
Great Bar (M6)	1A	1050	CARS	434	62.060	9.080	37.231	100.935
			H.G.V.	64	60.562	7.785	41.80	72.00
			ALL	498	61.867	8.937	37.231	100.935
	1B	1000	CARS	355	62.63	8.920	40.00	86.30
			H.G.V.	30	61.43	8.931	40.310	80.23
			ALL	385	62.50	8.915	40.00	86.30

TABLE 5.23

Free speeds on merging Vehicles at the start of slip road (kph).

Site: A38/M6 merge.

LANE	SAMPLE	MEAN	S.DEV.	MIN.	MAX.	X^2	$X^2_{0.95}$
LANE 1	302	62.314	7.647	50.00	92.308	12.57	16.91
LANE 2	248	68.253	8.176	50.706	94.738	11.83	15.51
ALL LANES	560	64.963	8.393	50.00	94.738		

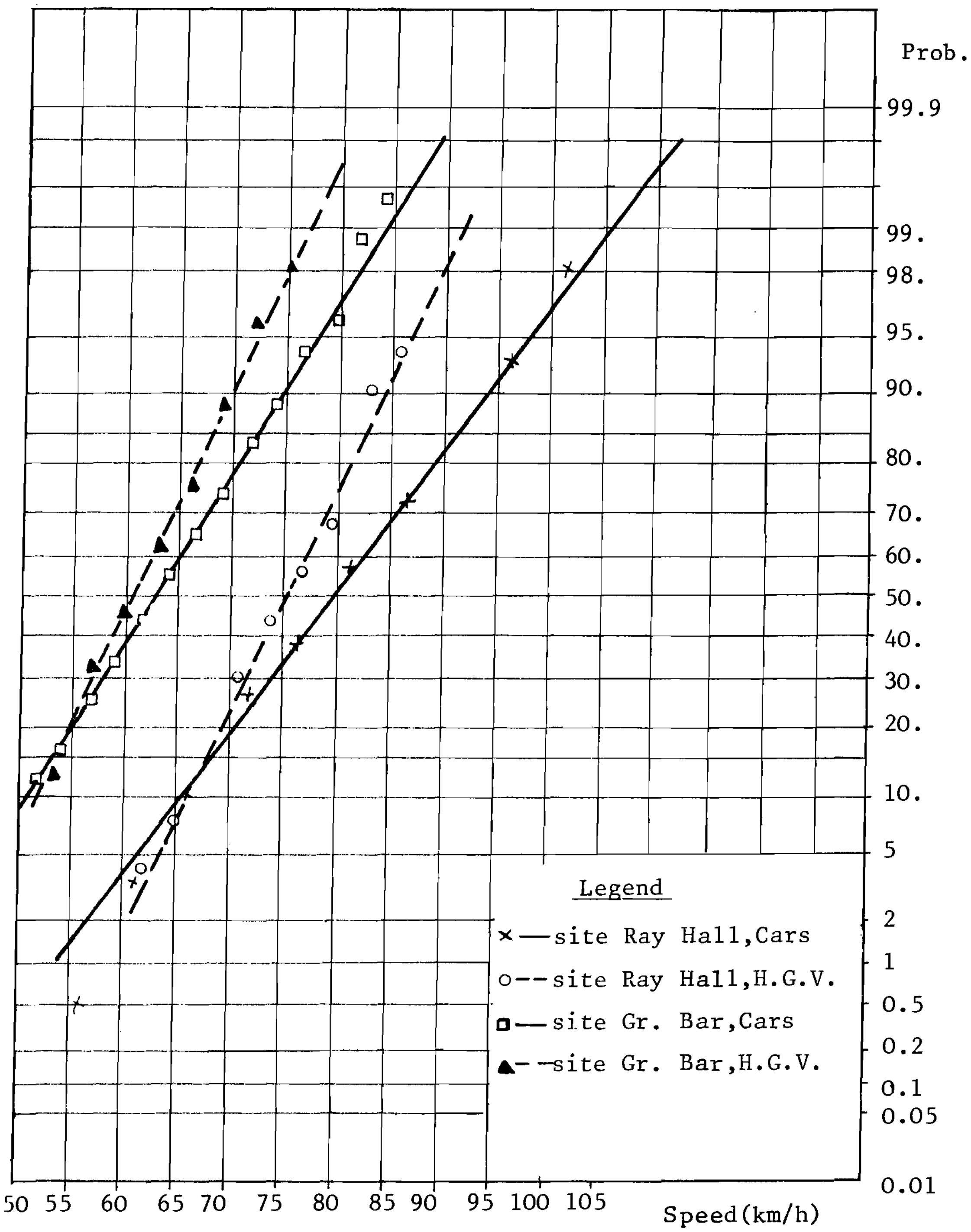


Figure 5.25: Speeds of Merging Vehicles at nose.
Fitted Normal distributions.

TABLE 5.24

Average accelerations of merging Vehicles (m/s^2)
Site: M5/M6 Ray Hall.

SPEED — CLASS (kph)	SAMPLE	MEAN	S.DEV.	MIN.	MAX.	NORMAL FIT X^2	NORMAL FIT $X^2_{0.95}$
54.72 - 64.72	61	0.261	0.52	-0.948	1.676	1.13	11.07
64.72 - 74.72	199	0.343	0.418	-1.442	1.850	18.91	19.66
74.72 - 84.72	253	0.459	0.666	-2.205	2.227	28.13	28.87
84.72 - 94.72	145	0.460	0.683	-1.887	2.650	22.37	23.68
94.72 - 104.72	25	0.469	0.833	-1.109	2.507	-	-
ALL VALUES	683	0.406	0.607	-2.205	+2.650	5.54	11.34

TABLE 5.25

Acceleration rates of merging Vehicles (m/s^2)
Site: M5/M6 Ray Hall.

SPEED - CLASS (kph)	SAMPLE	MEAN	S.DEV.	MIN.	MAX.	LOG-NORMAL FIT X^2	LOG-NORMAL FIT $X^2_{0.95}$
54.72 - 64.72	42	0.530	0.364	0.040	1.676	2.029	3.841
64.72 - 74.72	168	0.461	0.312	0.001	1.850	1.08	5.02
74.72 - 84.72	199	0.683	0.532	0.011	2.227	13.25	14.07
84.72 - 94.72	116	0.696	0.489	0.013	2.650	12.07	12.59
94.72 - 104.72	17	0.888	0.611	0.076	2.507	-	-
ALL VALUES	542	0.612	0.468	0.001	2.65		

TABLE 5.26

Deceleration rates of merging Vehicles (m/s^2)
Site: M5/M6 Ray Hall.

SPEED - CLASS (kph)	SAMPLE	MEAN	S.DEV.	MIN.	MAX.
54.72 - 64.72	19	0.334	0.238	0.031	0.948
64.72 - 74.72	31	0.299	0.330	0.001	1.442
74.72 - 84.72	54	0.366	0.417	0.014	2.205
84.72 - 94.72	29	0.486	0.504	0.039	1.877
94.72 - 104.72	8	0.422	0.435	0.027	1.109
ALL VALUES	141	0.379	0.403	0.001	2.205

TABLE 5.27

SPEED - ACCELERATION RELATIONSHIPS	R
Acceleration rates: A =-2.002+0.015*V	0.90
Deceleration rates: A =+0.092+0.0036*V	0.903
All Values: accel/dec: A =-0027 +0.005*V	0.91
where:	
A : average accel/decel in m/s^2	
V : average speed in speed class (kph)	

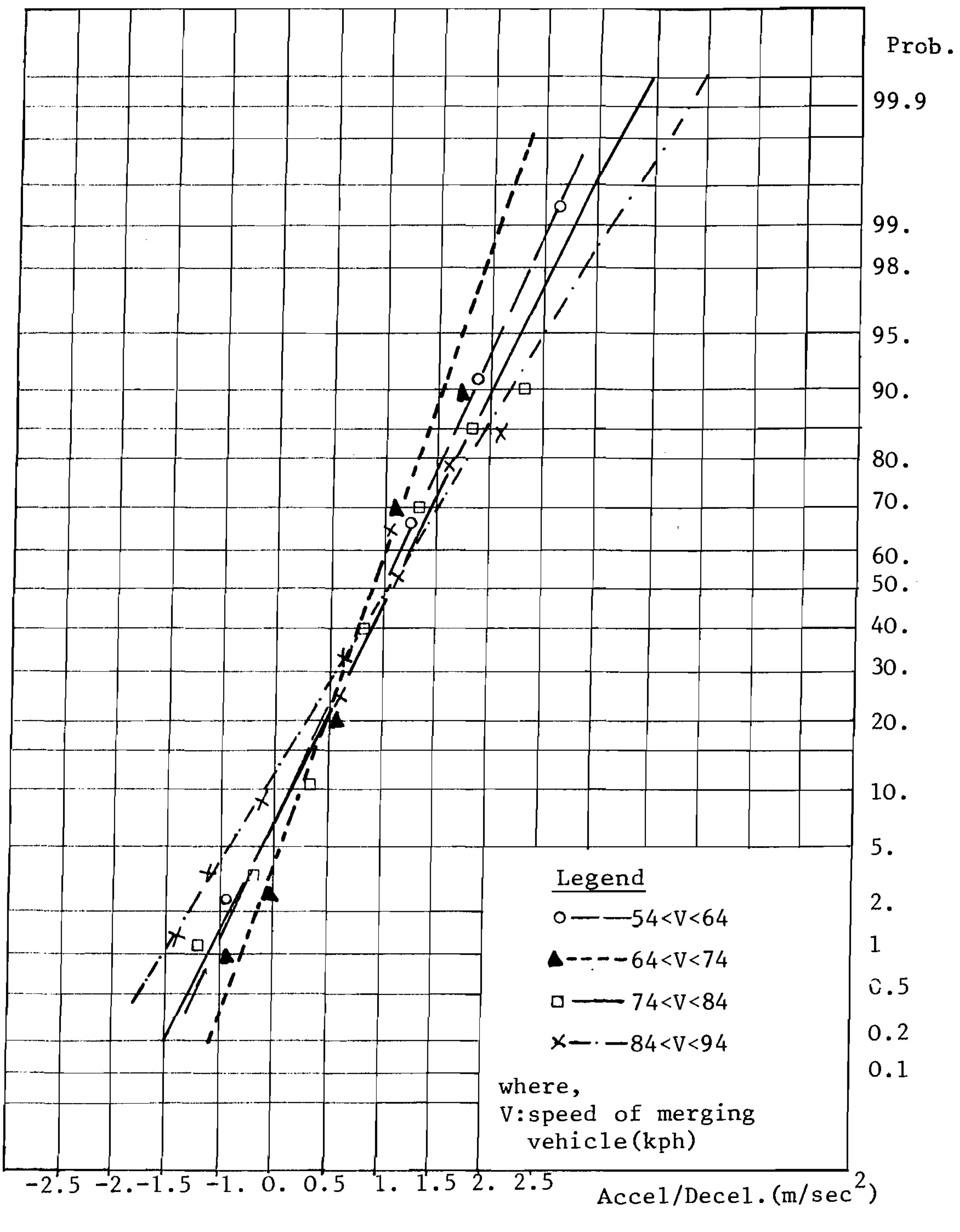


Figure 5.26a: Accelerations-Decelerations of Merging vehicles, for 10 kph. speed intervals.Fitted Normal distributions.

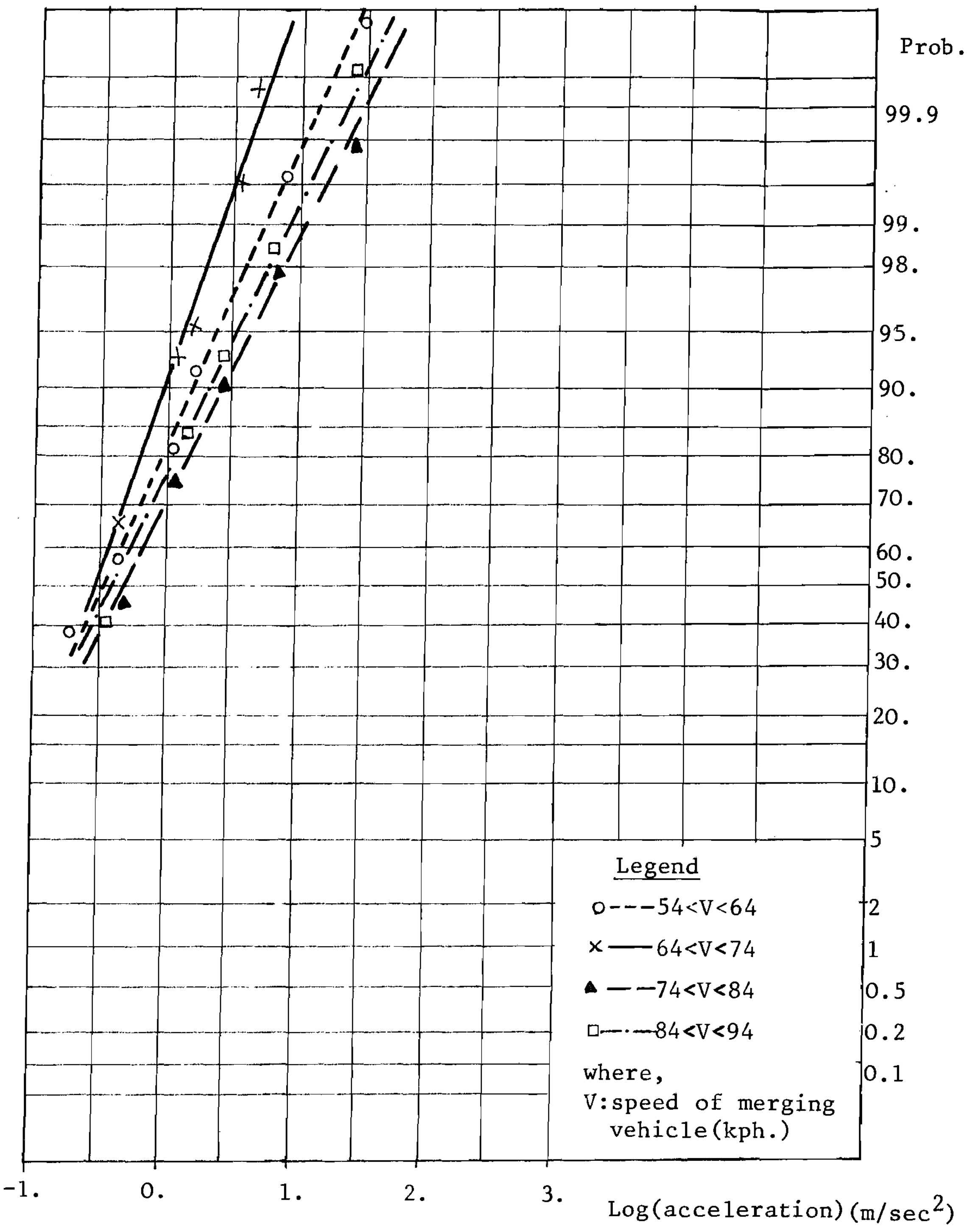
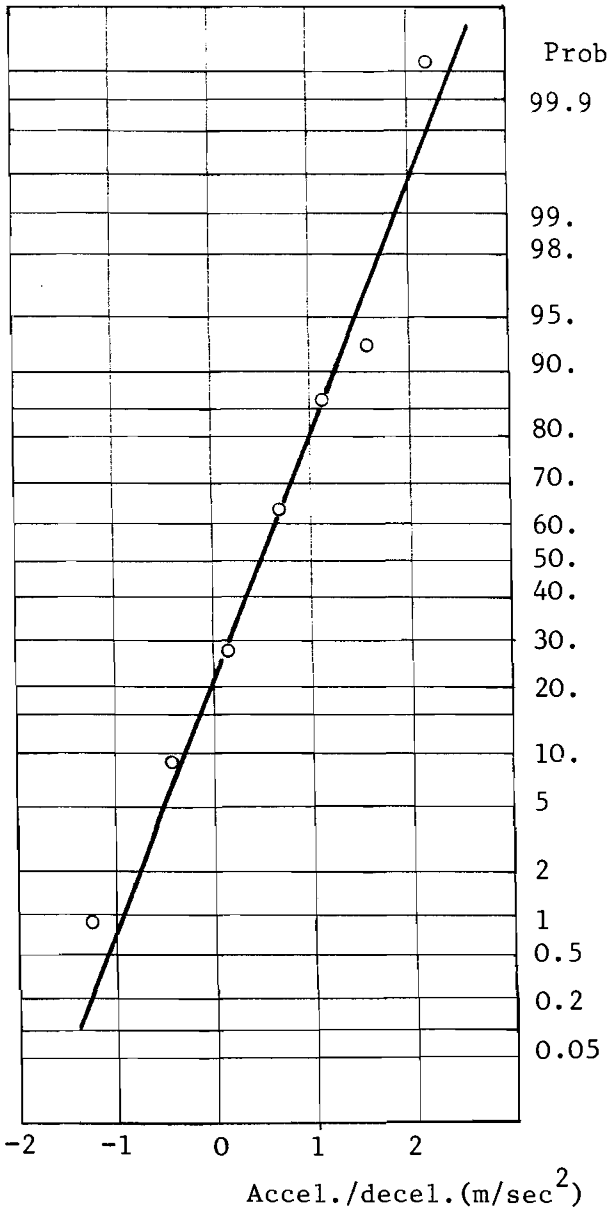


Figure 5.26b: Acceleration rates of Merging vehicles,for 10 kph. speed intervals.Fitted Lognormal distributions.

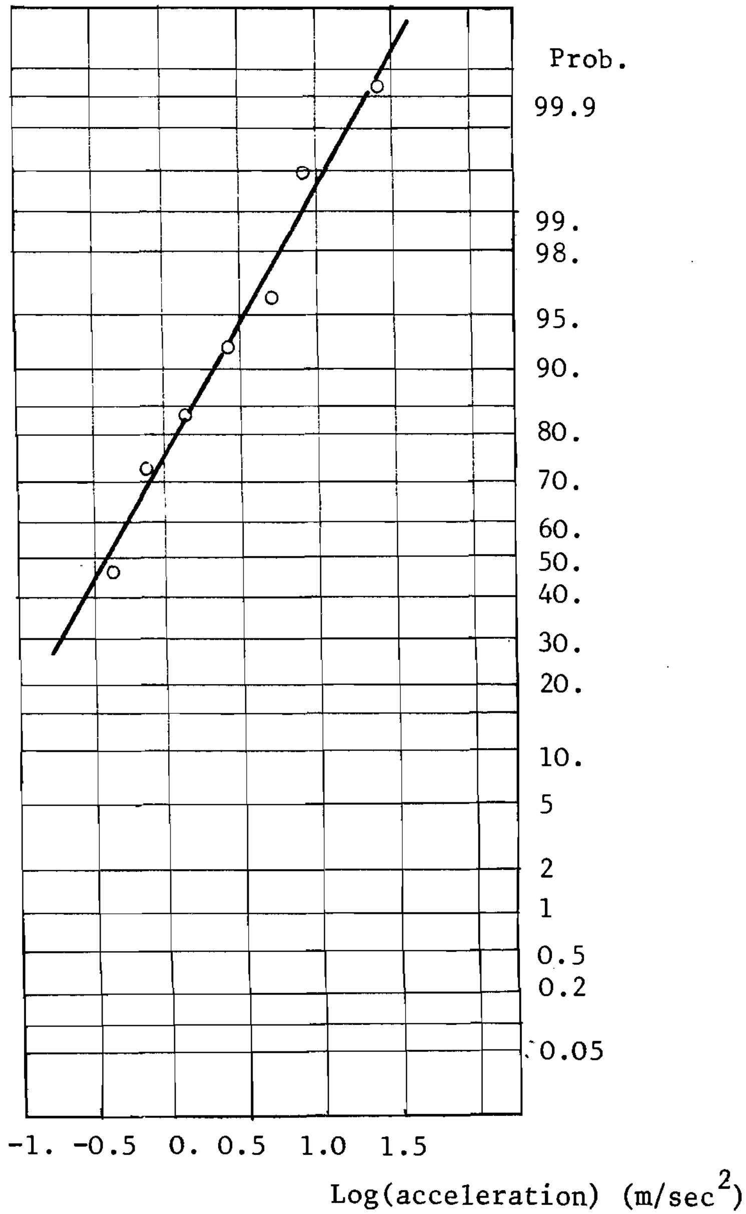
TABLE 5.28

Accelerations/Decelerations related to the merging process (m/s^2)
Site: Ray Hall.

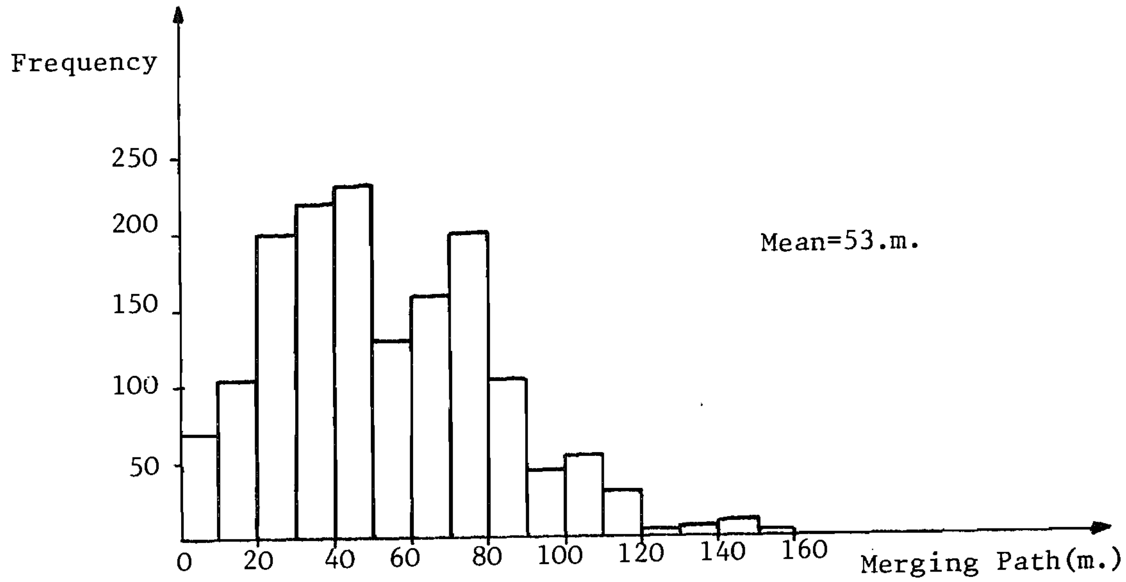
VEH. CLASSIFICATION	SAMPLE	MEAN	S. DEV.	MIN.	MAX.	NORMAL FIT	
						X^2	$X^2_{0.95}$
FREE	89	0.414	0.635	-1.862	2.082	6.62	9.48
RESTR-FREE ENTRY	255	0.380	0.592	-1.877	2.536	16.10	16.92
FREE-FORCED	132	0.445	0.557	-1.109	2.507	5.31	11.07
RESTR-FORCED	208	0.409	0.645	-2.205	2.650	25.73	26.29



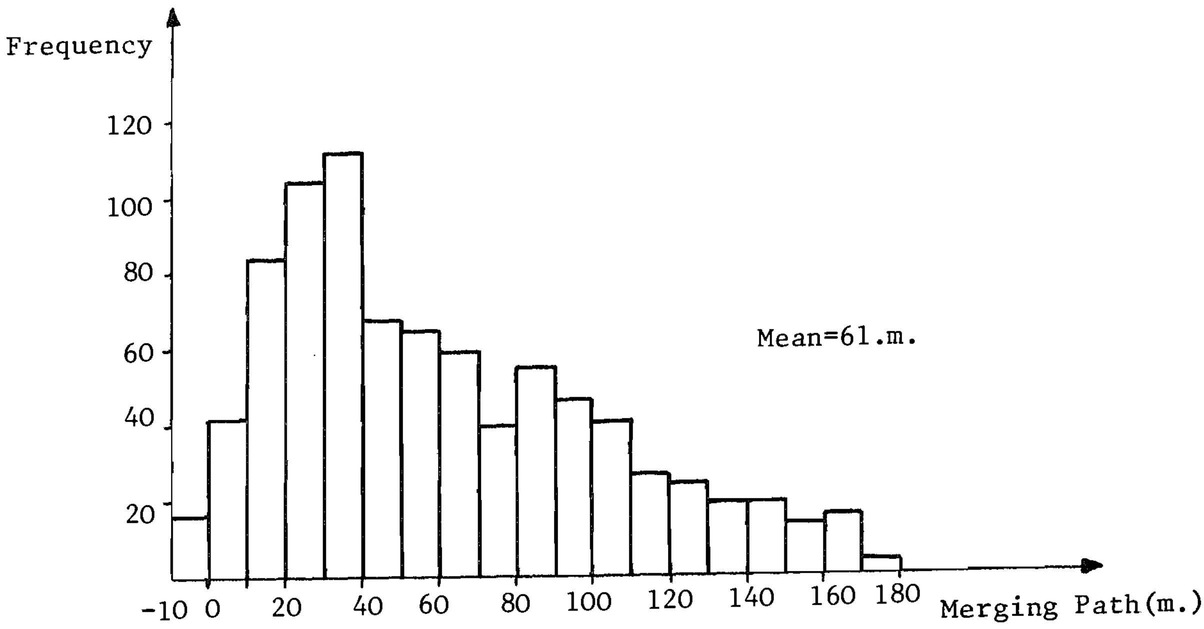
a. Free Accelerations Decelerations of merging veh.
Fitted Normal distribution.



b. Free Accelerations of merging veh.
Fitted Lognormal distribution.



a. Site: Ray Hall



b. Site: Great Bar

Figure 5.28:Frequency distributions of Merging Paths.

CHAPTER 6

CALIBRATION AND VALIDATION

6.1 General

The ultimate objectives of the study are to evaluate the operation and performance of a ramp entry and to suggest design procedures. In order to achieve these objectives, a microscopic model of traffic flow has been developed to investigate the merging behaviour at motorway interchanges. Calibration and Validation are considered as important steps in the simulation procedure adopted in this study, in order that the performance of the model to be assessed on the basis that accurately represents the traffic situation and therefore to enable alternate geometric or traffic management effects to be examined.

6.2 Calibration

The calibration stage is closely related to the data analysis described fully in the previous chapter. Results from the analysis are used as typical values of traffic parameters and for mathematical description of their variability, i.e. statistical distributions. On the other hand qualitative observations from the data base, and further theoretical study have also suggested a series of refinements and improvements in the modelling of the process, since its original version (Ref. 132), especially the merging and lane changing behaviour. In parallel, computer runs were made to test and evaluate model parameters not estimated from data such as : characteristic speeds, vehicle lengths, reaction time, acceleration constants, and the 'stabilization' motorway section upstream.

In its final version the model has as input two sets of data : site independent, which are permanent estimates of the parameters used in the model, e.g. gap acceptance values, free speed distributions, as they are given in the previous chapters; and site dependent which can vary according to the different geometric and traffic conditions. These include : Flows and traffic composition on the motorway lanes and slip road, geometric characteristics of the ramp entry and headway parameters on traffic streams.

The model has been tested against the data of calibration site, i.e. the M5/M6 Ray Hall interchange where the delays of merging vehicles were estimated from the measurements according to the equation (4.12). The observed and predicted from the model delay values are in close agreement,as it is shown in Fig. 6.1.*

6.3 Validation

The validation procedure consists of comparing the results from the model with available information from a site not used in the calibration (Ref. 134), and with other studies and data. In order for that to be done, the delays of merging vehicles at the site Great Bar were estimated from the measurements and the junction was simulated using as input parameters the geometric and flow conditions. The actual and predicted delay distributions are plotted in Fig. 6.2 and it can be seen that they are in good agreement.

The proportion of merging vehicles having negative delays, also noted in the calibration site is due to the delay definition, in practice some ramp vehicles having accepted a gap continue to move on the acceleration lane with increased speed whereas the modelling assumes direct merge.

Further to the above independent test the model output was compared with the analytical soultion proposed by Blumenfeld and Weiss (Ref. 2) as described in section 2.2. The program was modified such as to calculate delays according to the definition given in equation (2.1), (2.2) and the queueing vehicles were ignored. The numerical values were :

$$V = 60 \text{ mph} \qquad v = 20 \text{ mph}$$

for mainstream and ramp vehicle speeds and

$$T_m = 4 \text{ sec.} \qquad T_s = 4 \text{ sec.}$$

* The simulated and actual distributions of the accepted lag and lead times, are also in close agreement as it can be seen from Fig. 5.20b.

as values of the critical gaps for the moving and stopped driver respectively.

The results of the simulation model are very close to those given by the analytical solution, the maximum difference being 5 per cent in delay estimation and 8 per cent for the probability of a vehicle being stopped, as it is shown in Fig. 6.3, 6.4.

6.4 Effect of Traffic Conditions

The model has been used to predict the operation of the ramp entry on a wide range of traffic conditions assuming constant geometry. The geometric features correspond to the current design standards for rural sites :

Acceleration lane length : 244 m. (800 ft.)
Slip road length = 450 m., including 122 m. 'nose', flyunder.

The results from the model for the evaluation parameters are summarised below :

a. Delays at entry : The estimated average delays for various combinations of upstream motorway flow Q_M and ramp flow Q_R are plotted in Fig. 6.5 for flow levels $(Q_M + Q_R)$ up to 4800 veh/h. It can be seen that under these traffic conditions the junction operates satisfactorily under low delays. The delays follow the pattern to increase as mainstream and ramp flow increase.

The average delays for merging vehicles were also plotted on the merging diagram, shown in Fig. 6.6, which is used for design purposes (Ref. 108, 26). The mean delays for each flow regime are calculated and presented in table 6.1. Although the delays increase as flow increases,the greater increase and variability in delay occurs at the highest flow levels, near the typical capacity, but it can be seen that the entering flows suggested for design(Ref. 26) can be accomodated under reasonable delays.

b. Journey Times : The average journey times of merging vehicles as defined in section 4.8.4 predicted by the simulation model for a distance up to 450 m. downstream, are plotted in Fig. 6.7 as a function of the total flow downstream ($Q_M + Q_R$). Table 6.2 gives their average values and other measures for certain flow regimes. The results show that for a flow range :

$$Q_M + Q_R \leq 3600 \text{ veh/h}$$

which corresponds to the 'standard design flow', the journey times are similar for all the combinations of Q_M and Q_R . For the flow regime :

$$3600 \leq Q_M + Q_R \leq 4800 \text{ veh/h}$$

which is the level of 'maximum working flow', the journey times have a slight increase, and do not again exhibit great variations from the mean value. As flows approach the 'capacity' greater variations and increased journey times occur, especially at high Q_R . These results are very close with the patterns observed during Transport and Road Research Laboratory measurements on M4 interchanges (Ref. 125).

c. Speeds on Ramp : The average speed distance profile of merging vehicles on the slip road predicted from the simulation model, shown in Fig. 6.8, indicates that most of the acceleration of merging vehicles takes place on the slip road under the given geometric conditions leading to small relative speeds at the ramp nose. H.G. Vehicles have in general higher relative speeds due to their mechanical capabilities. This is in agreement with observed speed characteristics under similar geometric conditions (Ref. 101).

6.5 Concluding Remarks

During this study a microscopic simulation model has been developed and calibrated. Using measured traffic data and available information from other sources, the model performance has been assessed and it was shown that adequately represents the traffic behaviour at motorway interchanges.

It is therefore a research tool for evaluation of alternate geometric design features, traffic control schemes or detailed analysis of certain traffic elements, such as modelling the traffic flow on single or multilane links.

The model has been applied on a wide range of traffic conditions assuming standard geometry. The results have shown that the delays of merging vehicles are low up to high flow levels, within the range of geometrics considered.
The flyunder slip road with sufficient length leads to higher speeds of merging vehicles at the entry and acceptance of smaller gaps.
In this case the acceleration lane is basically used as 'speed-adjustment' lane. As the flow levels approach the 'typical capacity', there is a significant increase and greater variability on the mean delays which indicates a need for monitoring the interacting traffic streams.

Comparisons with the suggested design flows have been shown that are realistic and can be accommodated on the specific layout of the interchange.

TABLE 6.1

Average delays (sec.)

FLOW REGIME	MEAN	S.DEVIATION	MINIMUM	MAXIMUM
ST.DESIGN FLOW	0.110	0.043	0.048	0.210
MAX. WORKING FLOW	0.218	0.084	0.181	0.520
ABOVE MAX. WORKING FLOW	2.165	2.978	0.419	9.863

TABLE 6.2

Average journey times (min.)

TOTAL FLOW DOWNSTREAM	MEAN	S.DEVIATION	MINIMUM	MAXIMUM
$\leq$3600	0.700	0.021	0.673	0.728
3600 - 4800	0.718	0.021	0.691	0.746
$\geq$4800	0.780	0.050	0.714	0.893

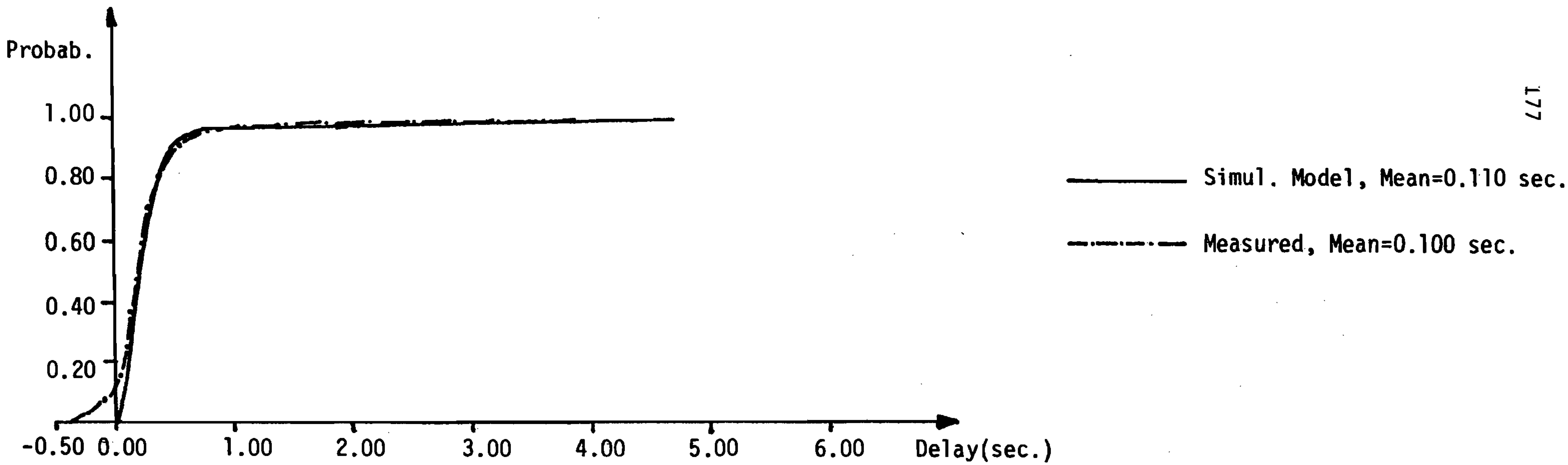


Figure 6.1:Probability distribution of Delay at entry(Calibration Site)

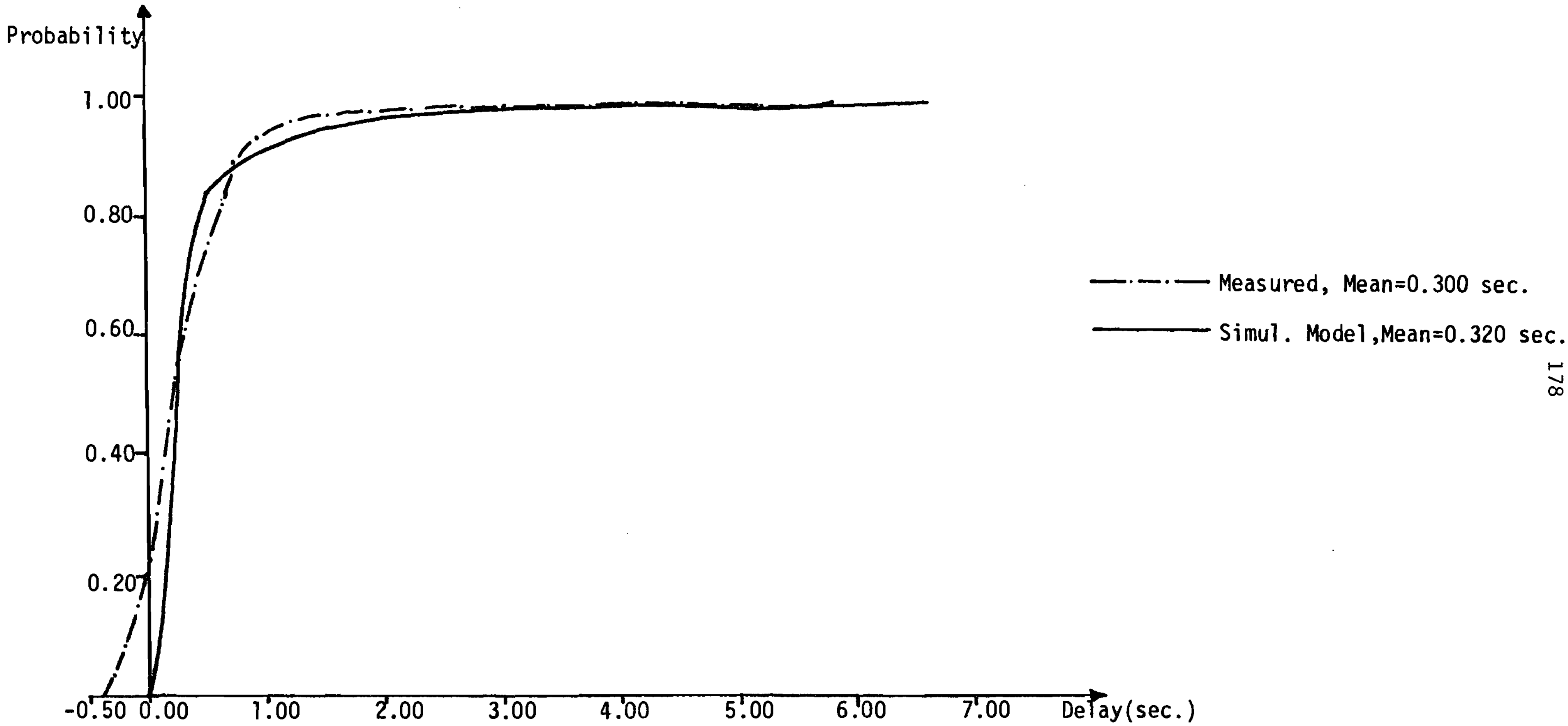


Figure 6.2:Delay distribution(Indepedent Site)

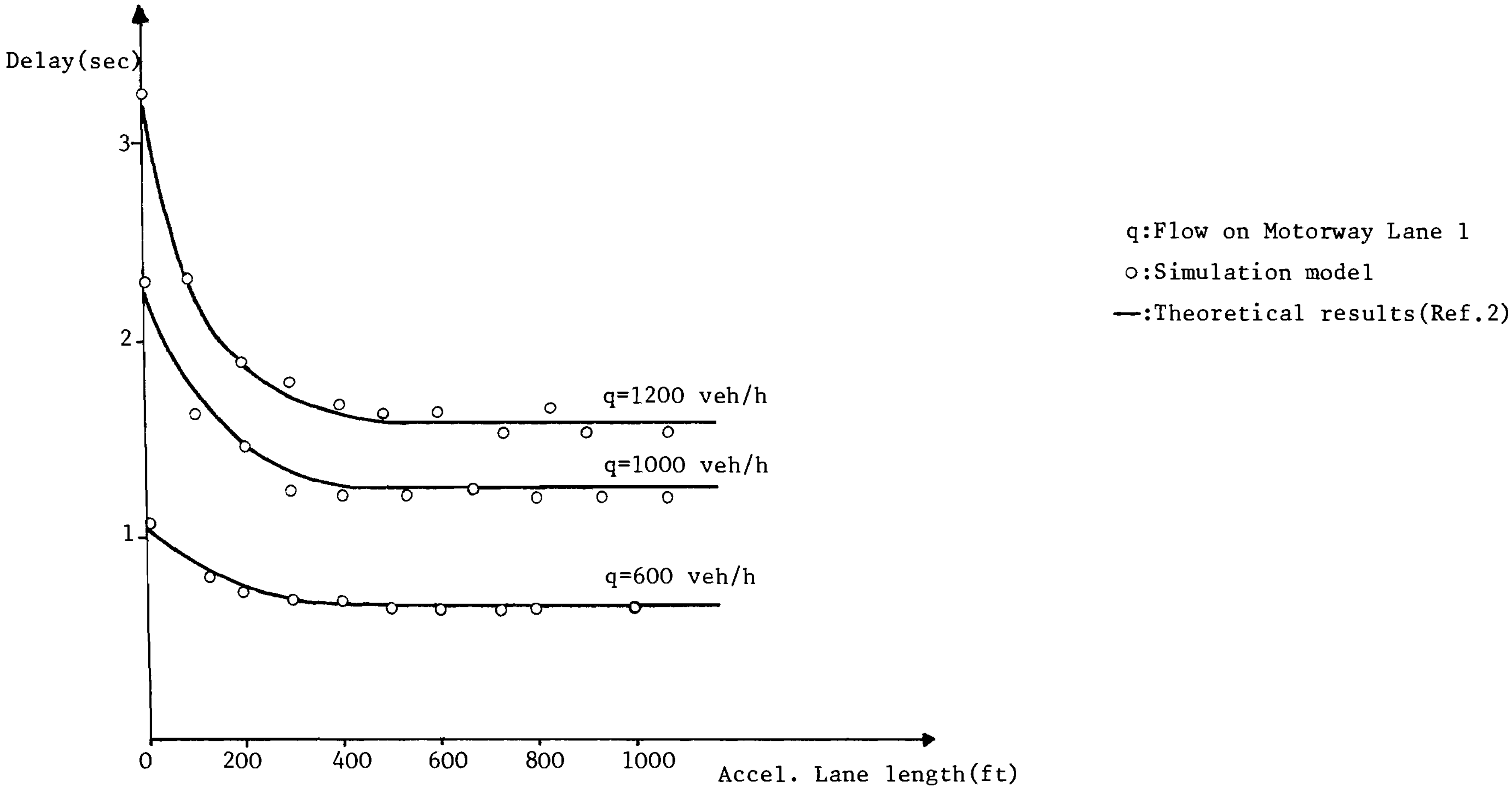


Figure 6.3: Average Delay of Merging Vehicles,as a function of the Acceleration Lane length.

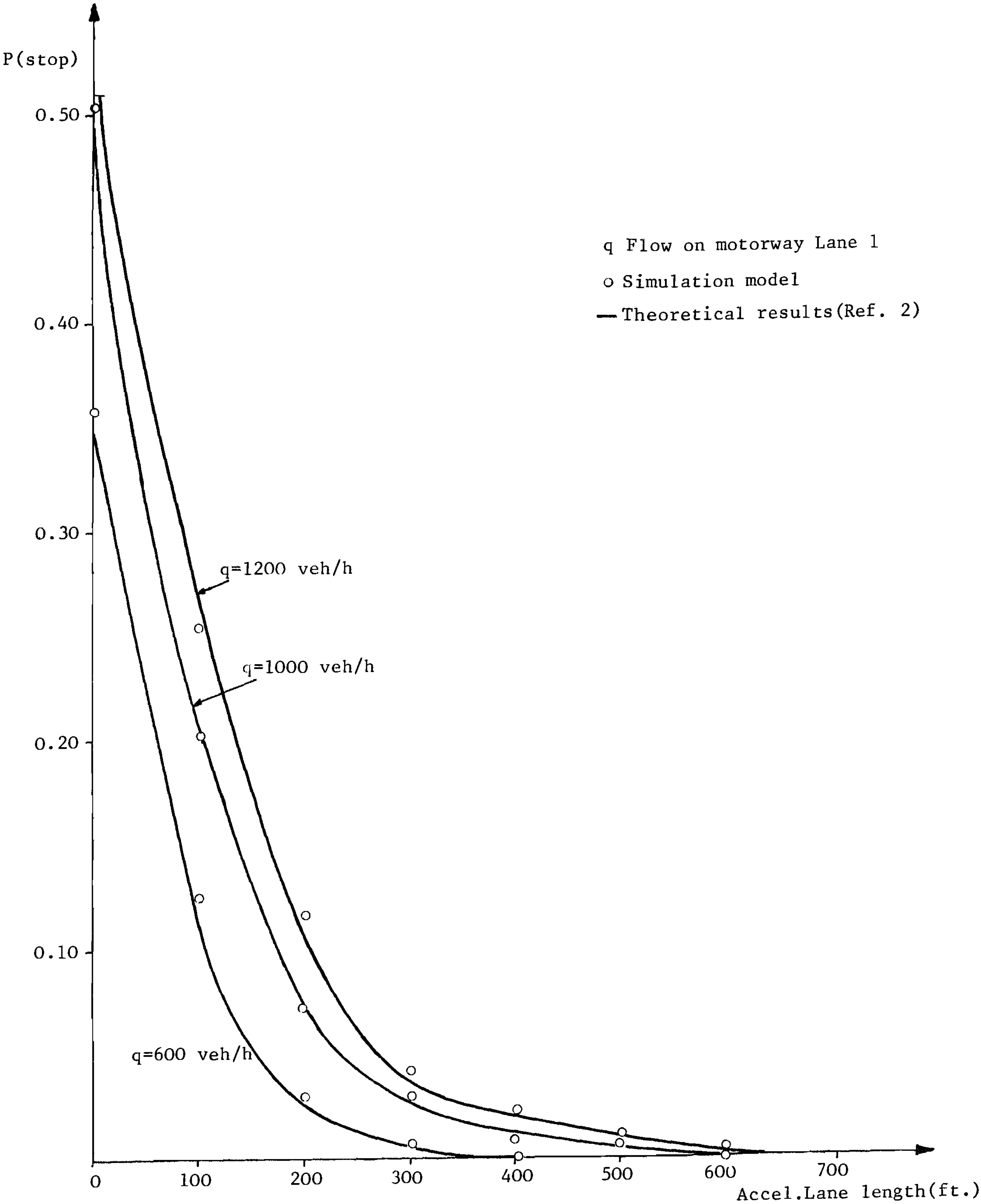


Figure 6.4: Probability of a vehicle being Stopped, as a function of the Acceleration Lane length.

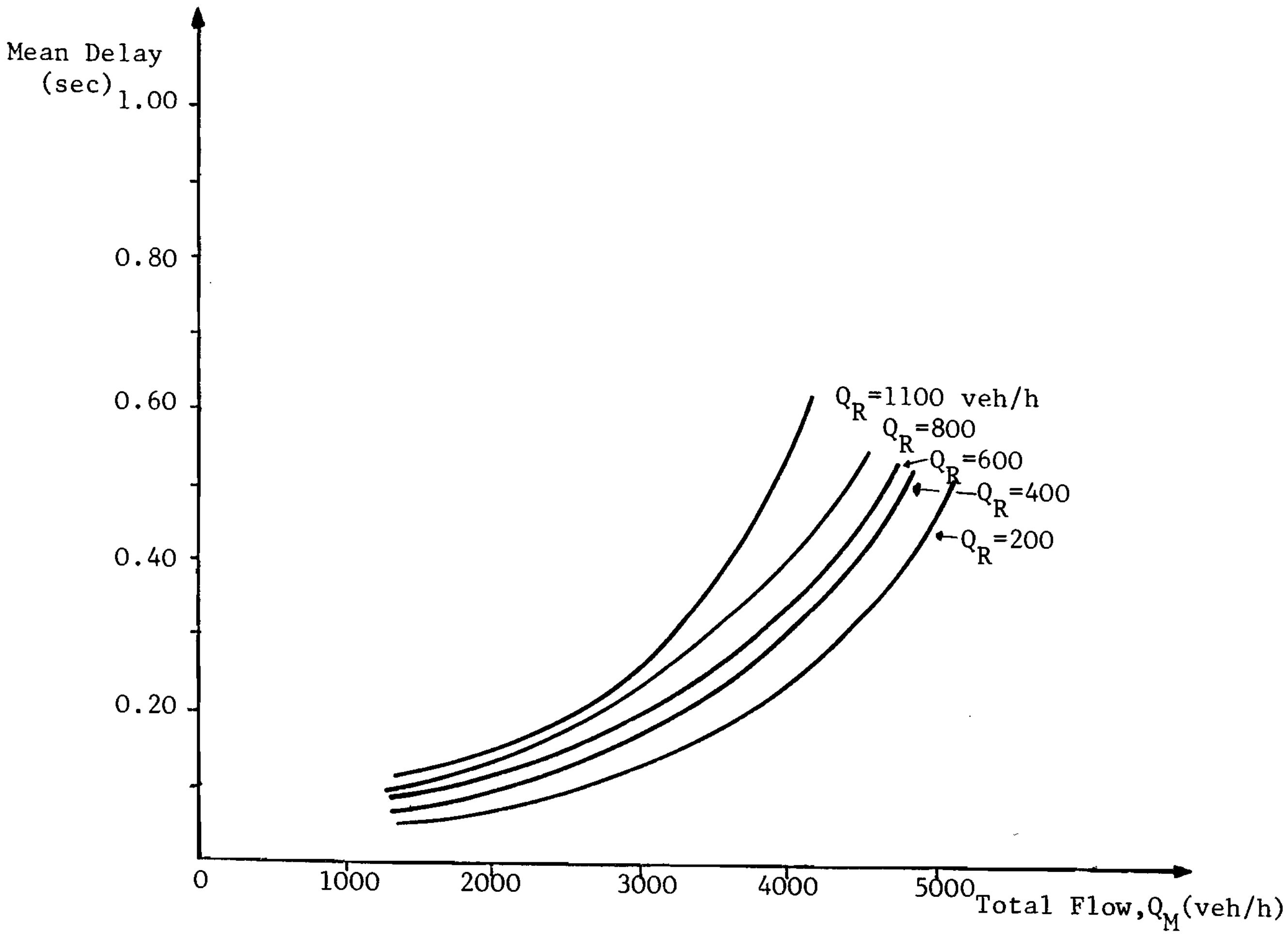


Figure 6.5: Average Delays to Merging Vehicles.

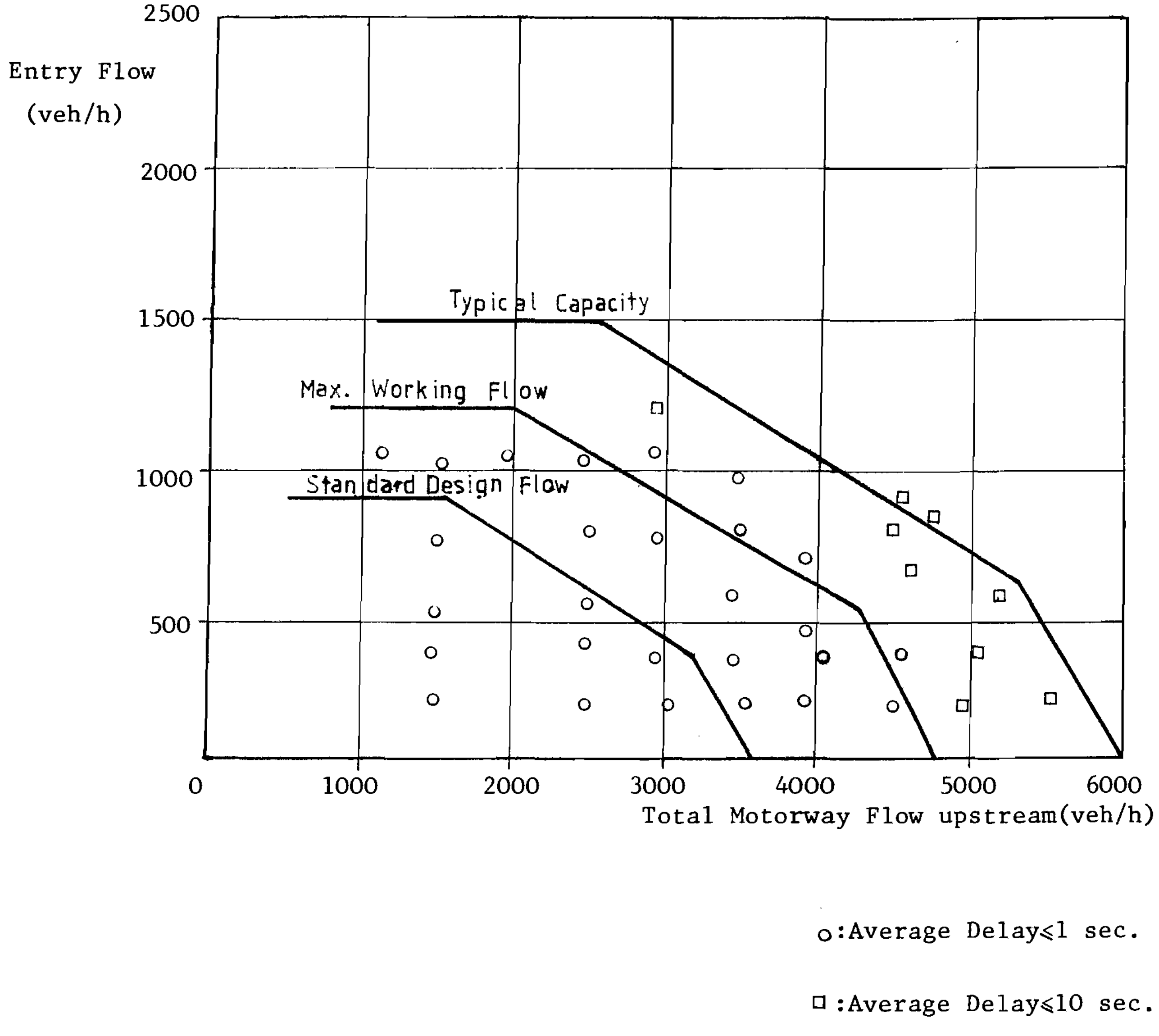


Figure 6.6: Estimated Delays from the model plotted on the Merging diagram(Ref.26)

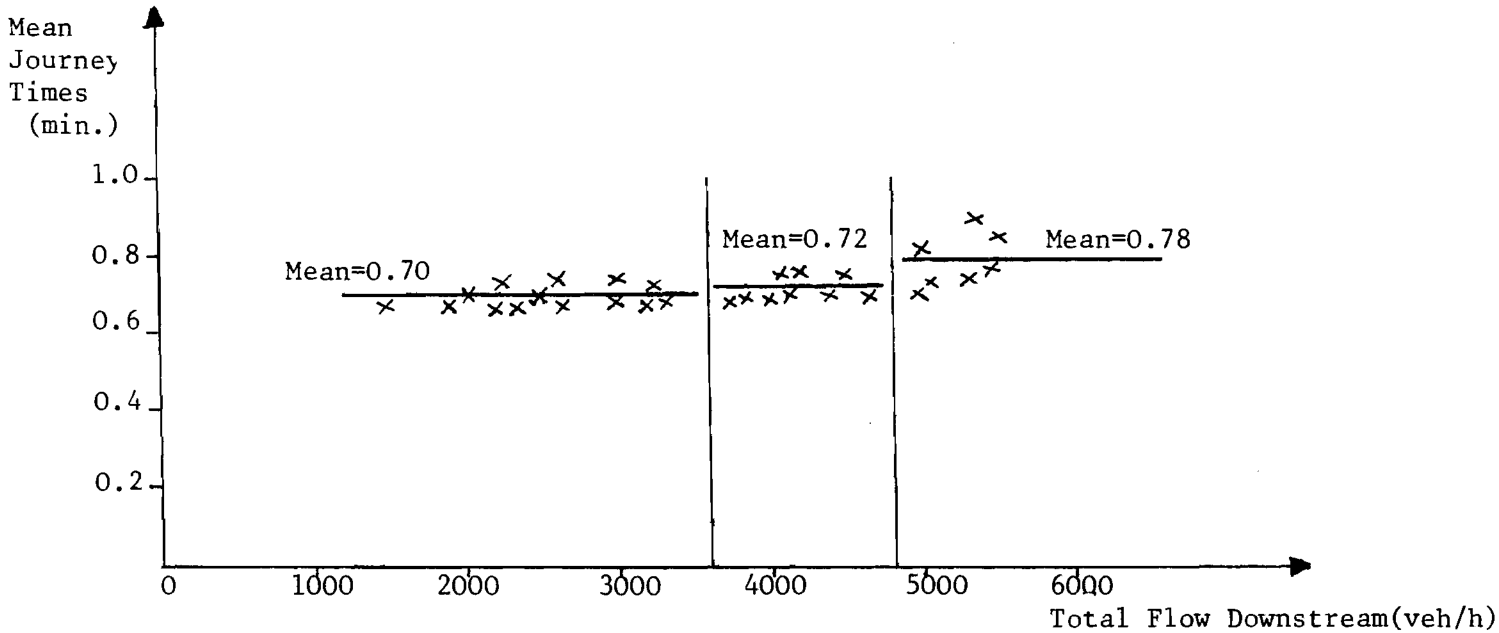


Figure 6.7: Average Journey times of Merging Vehicles

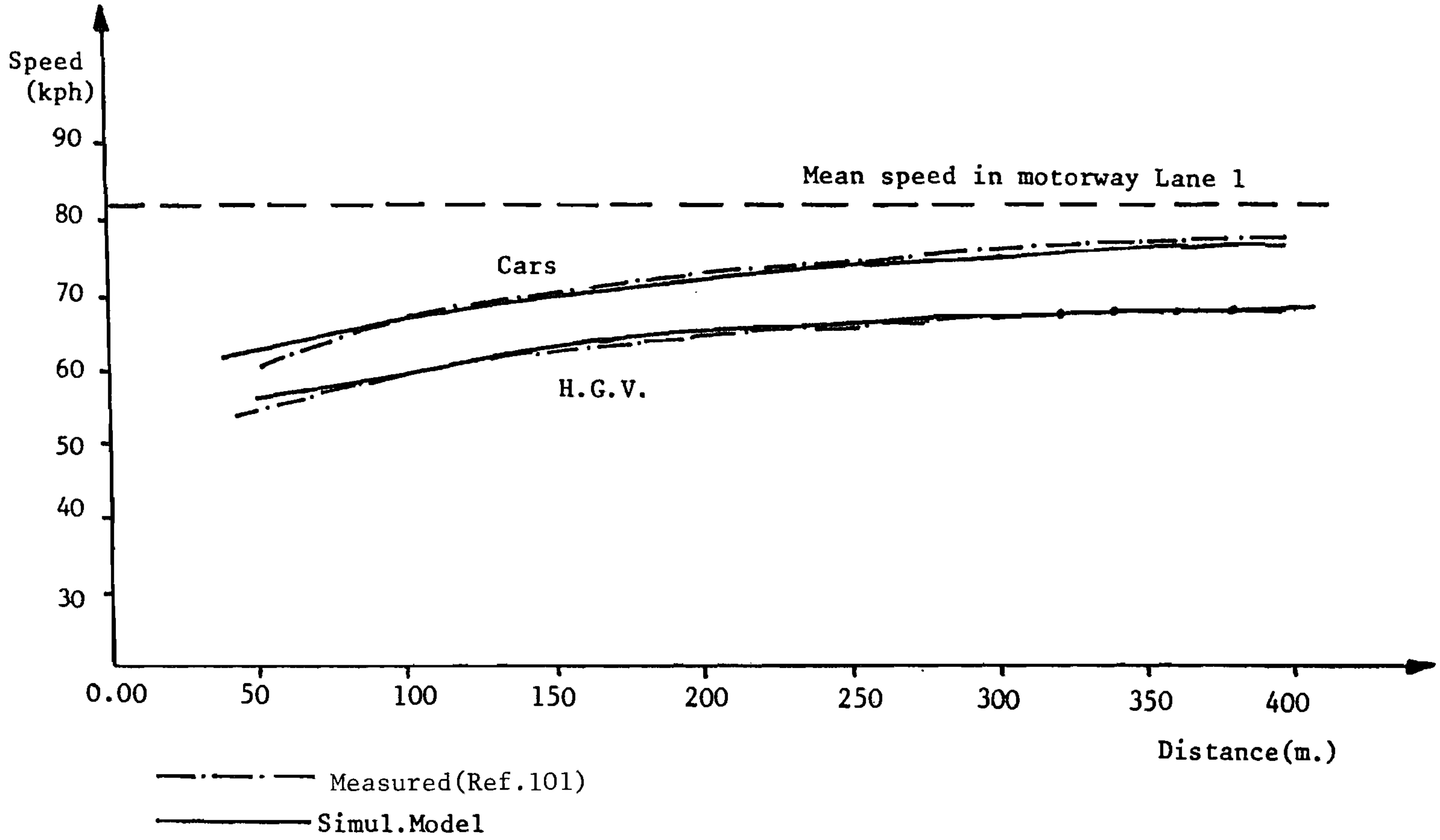


Figure 6.8:Speed Distance Profile for Ramp Vehicles

CHAPTER 7

APPLICATIONS

7.1 Introduction

In this study a model of traffic flow at ramp entries has been successfully formulated, calibrated and validated. With its aid the traffic behaviour at grade separated interchanges can be investigated and alternative designs can be evaluated. One of the model advantages is that it has been constructed with considerable flexibility, so specific designs can be examined without main modifications.

Basically there are two areas of applications : geometric design and traffic management schemes. The geometric design includes the examination of the significance of certain elements in the traffic behaviour such as : acceleration lane length, slip road length, gradient and curvature, angle of convergence, main road alignment and also two lane merge. The traffic management schemes can be broadly defined as main flow control, such as lane closure upstream of the merge, or ramp flow control, ramp metering. The control mechanism such as signals or roundabout at the slip road entry, can also be investigated. Another interesting aspect at motorway interchanges is the spill back of exiting vehicles into the main stream, due to congestion at the end of the exiting slip road. Various control mechanisms for such situations can be examined by the model.

7.2 Effect of the Acceleration Lane Length.

The length L of the acceleration lane is examined assuming the slip road layout as given in section 6.4, i.e. long straight ramp. The objective is to show if a reduction in L , which is desirable in terms of cost or inevitable due to site restrictions significantly affects the operation of an interchange.

A range of acceleration lane length values was chosen:

$$154 \text{ m.} \leq L \leq 244 \text{ m.} \quad \text{i.e.} \quad 500\text{-}800 \text{ ft.}$$

and the junction was simulated for various combinations of mainstream and ramp flows. The mean and s. deviation of average delays and journey times are given in table 7.1 for each flow regime and plotted in Fig. 7.1, 7.2.

The results show that in terms of average values we do not have significant increase as the length of acceleration lane decreases for flows up to maximum working flow. Delays at entry are more sensitive to higher flow levels but again we do not have significant reduction in capacity.

The length of acceleration lane should also be related to the safety aspect; an increased number of vehicles being stopped at the end of acceleration lane is a potentially hazardous situation. In order that this aspect be examined the probability of a single vehicle being stopped due to merging process, as a function of L was calculated from the model. The results are plotted in Fig. 7.3 for total mainstream flows above 4000 veh/h. Below that level the number of vehicles stopped was found negligible. It can be seen that the percentage of unsuccessful merges is small and insensitive to the length of acceleration lane above the value of $L = 184$ m. (600 ft.).

7.3 Effect of Slip Road Gradient

The gradient on the slip road affects the driver-vehicle characteristics and in connection to the short length leads to high relative speeds at the ramp nose. A realistic representation of traffic on grades is required in order for these effects to be examined. It is assumed in the model that gradient affects the desired acceleration of vehicles as they travel along the slip road because of the extra force required to overcome the grade resistance. This is generally expressed as :

$$Rg = g.W$$

where :

W : vehicle's weight (tons)

g : gradient (percent)

Rg : resistance (Kgr.)

The desired or free acceleration is calculated according to the equation (4.7), where the proportionality coefficient can be derived for cars and H.G. Vehicles, from data on maximum accelerations according to the grade and initial speed (Ref. 88). For testing and calibration, simulation runs have been made and the resultant speed distance profile of the merging vehicles is plotted in Fig. 7.4 together with available data measured at a site (Ref. 126). It can be seen that good agreement was obtained for both cars and H.G. Vehicles. The site, being the M4/A34 junction at Newbury, is a flyover with gradient 4.3 percent and length 330 m. measured from the start of the slip road up to the start of the acceleration lane.

The current design standards (Ref. 89, 90) suggest for slip roads a value of 4 per cent for gradients as normal maximum and 5 per cent as absolute maximum for rural sites. For urban conditions the gradient should not exceed 8 per cent and should be limited to 4 per cent in case of high proportion of H.G. Vehicles. Thus, in order that the effects of gradients be examined, the model has been applied to a ramp entry with a slip road having gradient 4 per cent and length 300 m., including the 122 m. nose. A series of simulation runs for various input flow rates have been made and the delays and journey times of merging vehicles were calculated.

The delays at entry increase significantly and also the number of vehicles delayed, as it is shown in Fig. 7.5 where the two geometric configurations are compared. The effect is more pronounced at the higher flow levels, i.e. at and above 'maximum working flow'. These results can be explained with the aid of Fig. 6.8, 7.4 where the speed distance profiles are plotted for the two situations. For the same length of slip road the mean relative speed for cars is 7 kph (4.4 mph) for flyunder and 12 kph (7.5 mph) for flyover. In the latter case the merging vehicles accept in general larger gaps as it has also been shown in the data analysis. For H.G. Vehicles the difference is more distinctive as the relative speeds are 15 kph (9.3 mph) and 26 kph (16 mph) for flyunder and flyover versions respectively.

Variation of acceleration lane length : The effect of gradient on the length of the acceleration lane L is examined by calculating the delays and journey times of merging vehicles for various flow rates. Because gradient particularly affects the performance of H.G. Vehicles due to their lower mechanical capabilities various proportions p of H.G. Vehicles on the merging flow were taken into account. The results have been shown that delays at entry increase rapidly as p increases and L decreases (Fig. 7.6).

The mean journey times of merging vehicles for a total downstream flow above the maximum working flow are plotted in Fig. 7.7 as a function of the parameters L and p . Comparisons with the results produced assuming a flyunder type of slip road as shown in Fig. 7.1, 7.2 indicate that the acceleration lane length becomes more sensitive as the geometrics of slip road become less favourable. Therefore it may not be claimed that the traffic operation is insensitive to length L for flyovers as it was shown above a certain threshold for downgrade ramps.

Effect on Mainstream Traffic : The effect of merging volume on the mainstream is also examined by comparing the estimated from the model lane changes from the inside lane to the offside, downstream of the ramp nose. The predicted values given as net lane changes for 3 min. intervals are plotted in Fig. 7.8 as a function of merging vehicles composition, for a typical flow regime. It is indicated that as H.G. Vehicles flow increases, i.e. more lower speed vehicles arrive at the entry, more motorway vehicles move over to the offside lanes. The effect is much less pronounced at the flyunder version ramp entries.

TABLE 7.1

Effect of acceleration lane length on delays and journey times.

a. Mean and S. Dev. of average delay (sec.)

FLOW REGIME / ACCEL.LANE (m)	ST. DES. FLOW	MAX. WORKING FLOW	ABOVE MAX. WORKING FLOW
154	0.130-0.041	0.300-0.019	2.986-4.115
184	0.128-0.046	0.282-0.140	2.808-3.890
214	0.124-0.052	0.250-0.110	2.550-3.58
244	0.110-0.043	0.218-0.084	2.165-2.978

b. Mean and S. Dev. of journey times (min.)

ACCEL. LANE (m)	TOTAL FLOW DOWNSTREAM		
	≤3600	3600-4800	≥4800
154	0.704-0.020	0.730-0.021	0.808-0.070
184	0.704-0.022	0.725-0.029	0.784-0.048
214	0.703-0.020	0.720-0.020	0.781-0.049
244	0.70-0.021	0.718-0.021	0.780-0.050

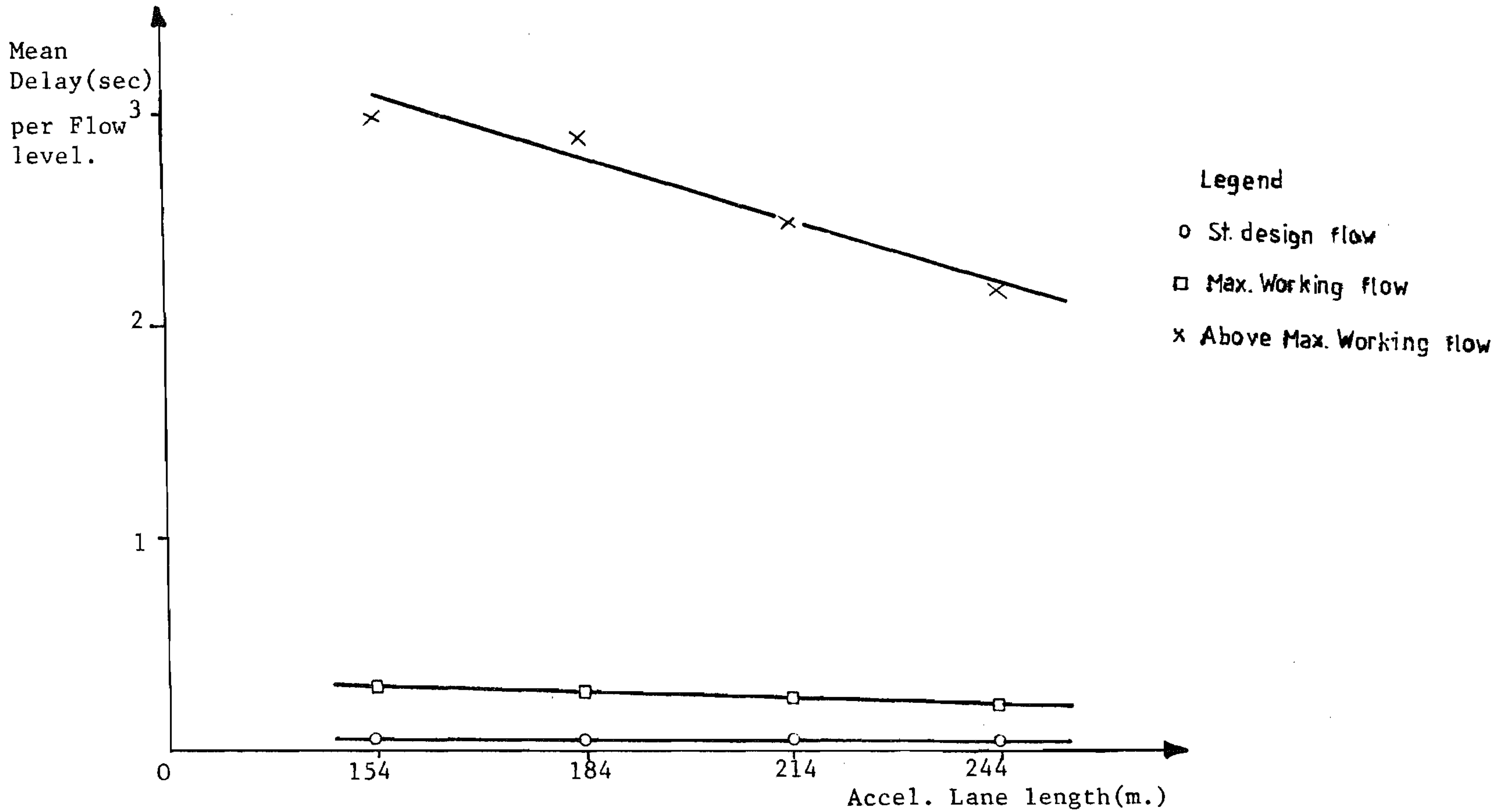


Figure 7.1 : Effect of Accel. Lane Length on Average Delay, for different Flow Regimes.

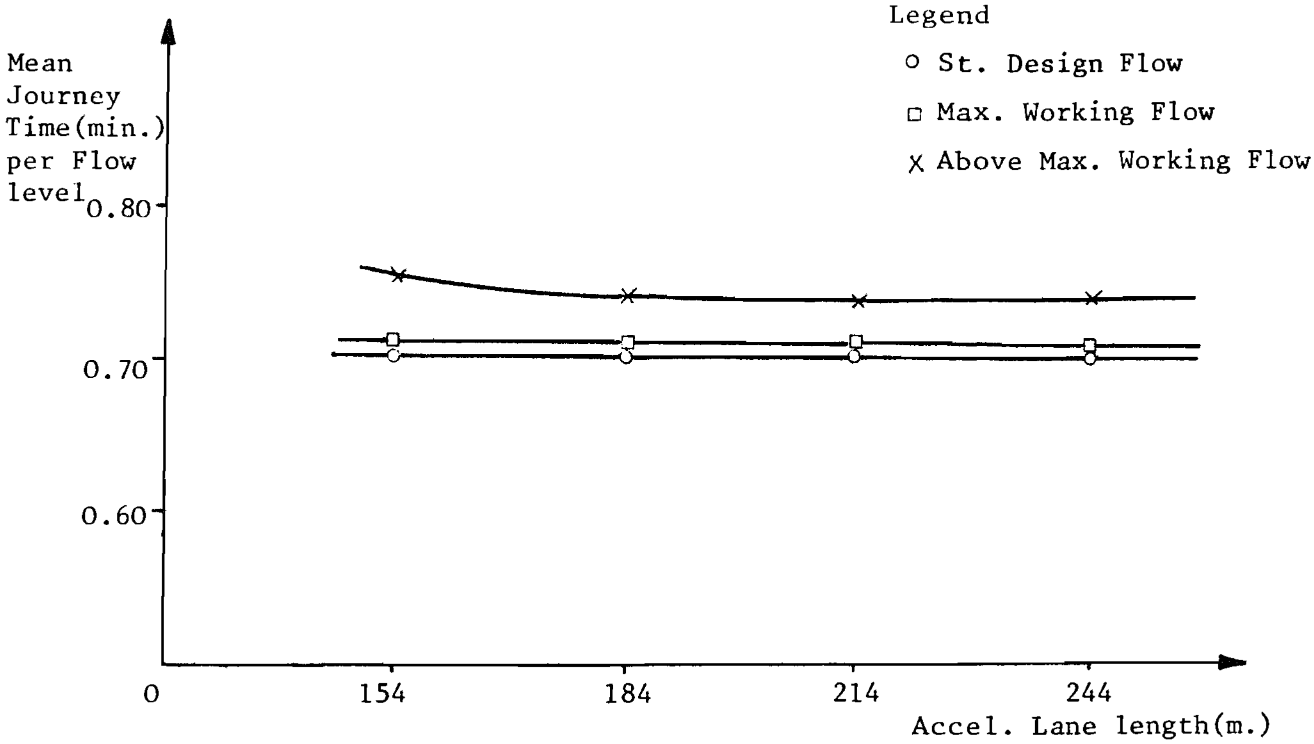


Figure 7.2: Effect of Accel. Lane Length on Average Journey Time of merging vehicles,for different Flow Regimes.

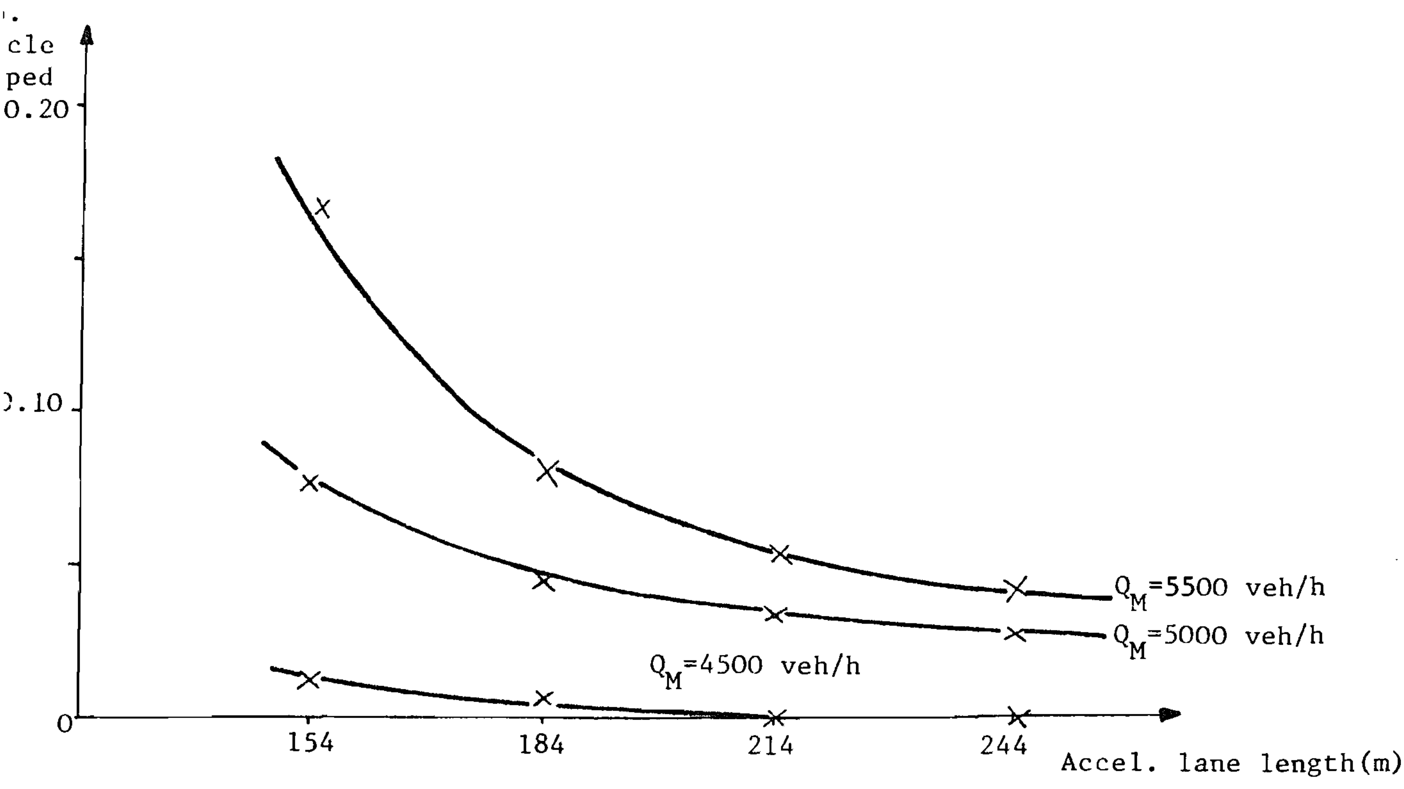


Figure 7.3: Probability of a vehicle being stopped due to mainstream flow Q_M as a function of the acceleration lane length.

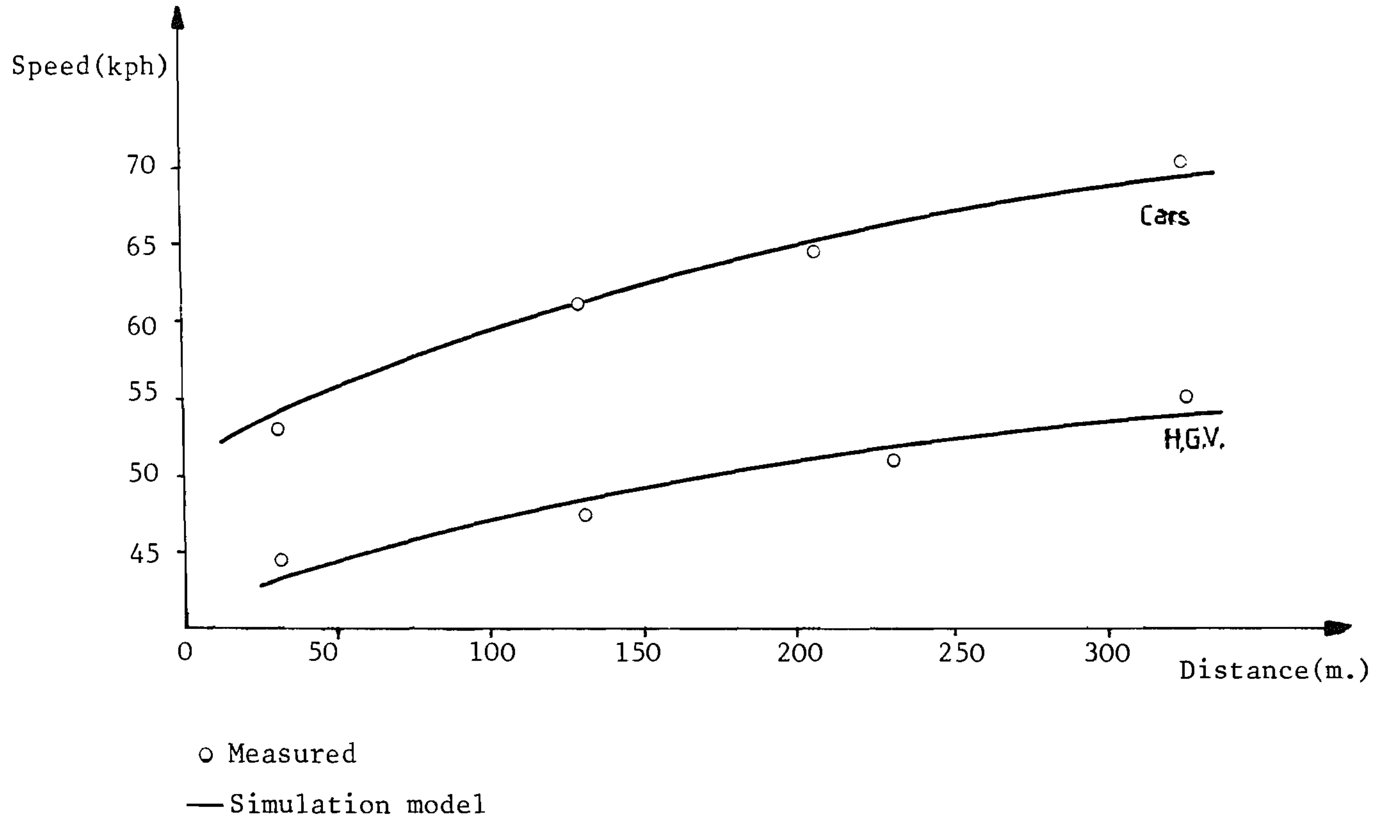


Figure 7.4:Speed-Distance Profile of slip road vehicles on uphill gradient.

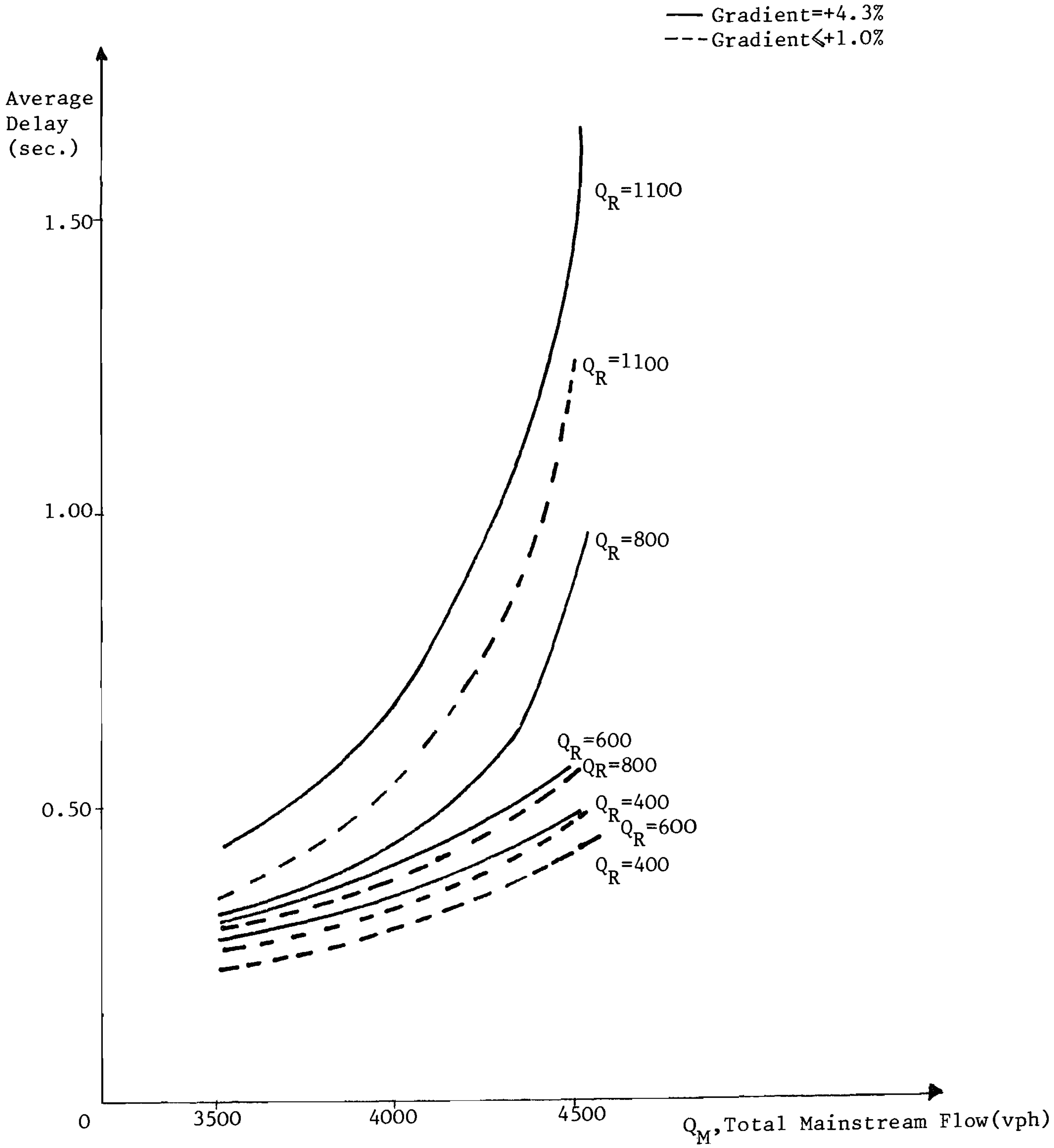


Figure 7.5: Effect of Gradient on average Delay of Merging Vehicles.

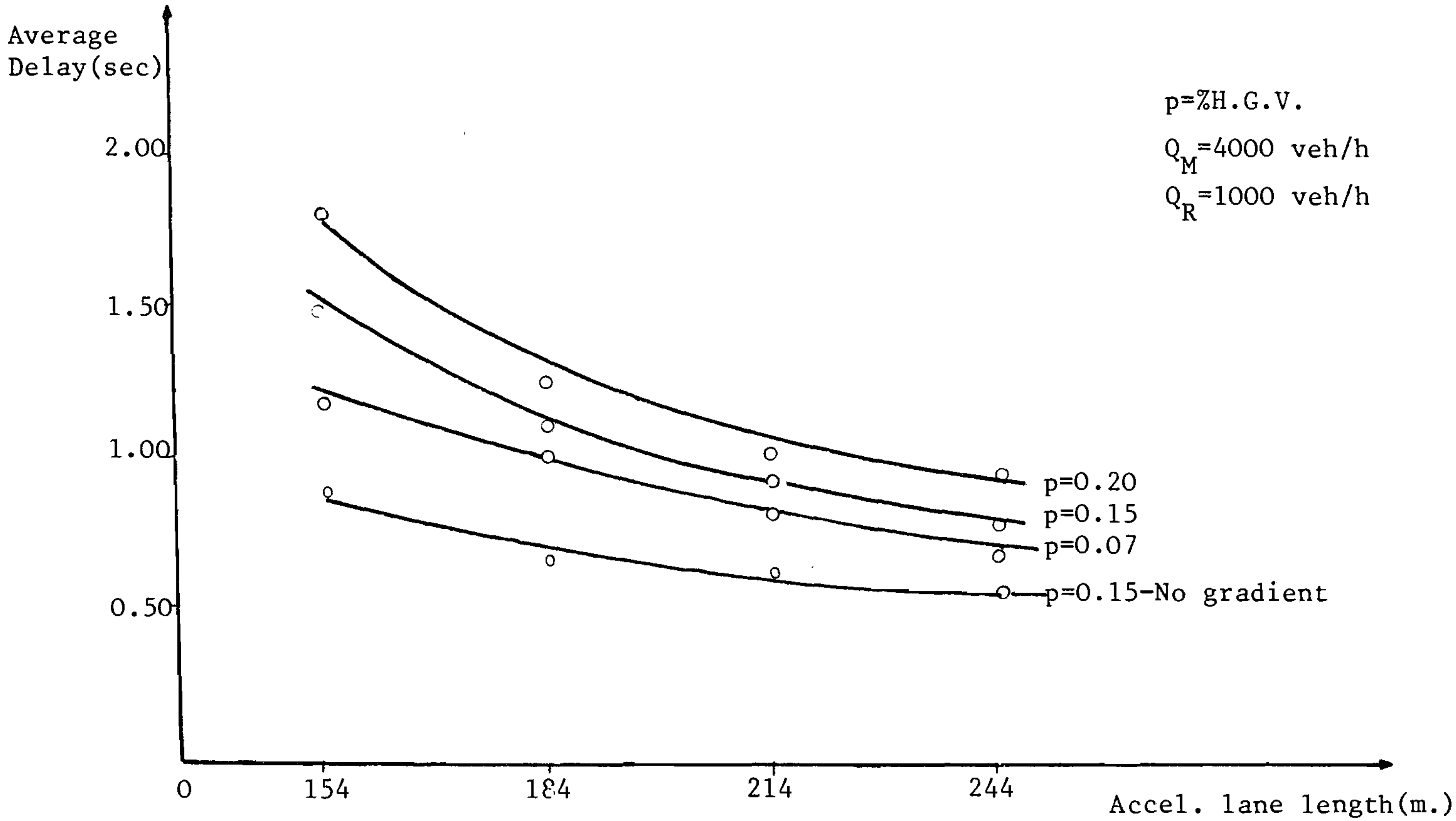


Figure 7.6: Effect of Gradient and Length of Accel. Lane on Average Delay, for different proportions of H.G.V. on merging flow.

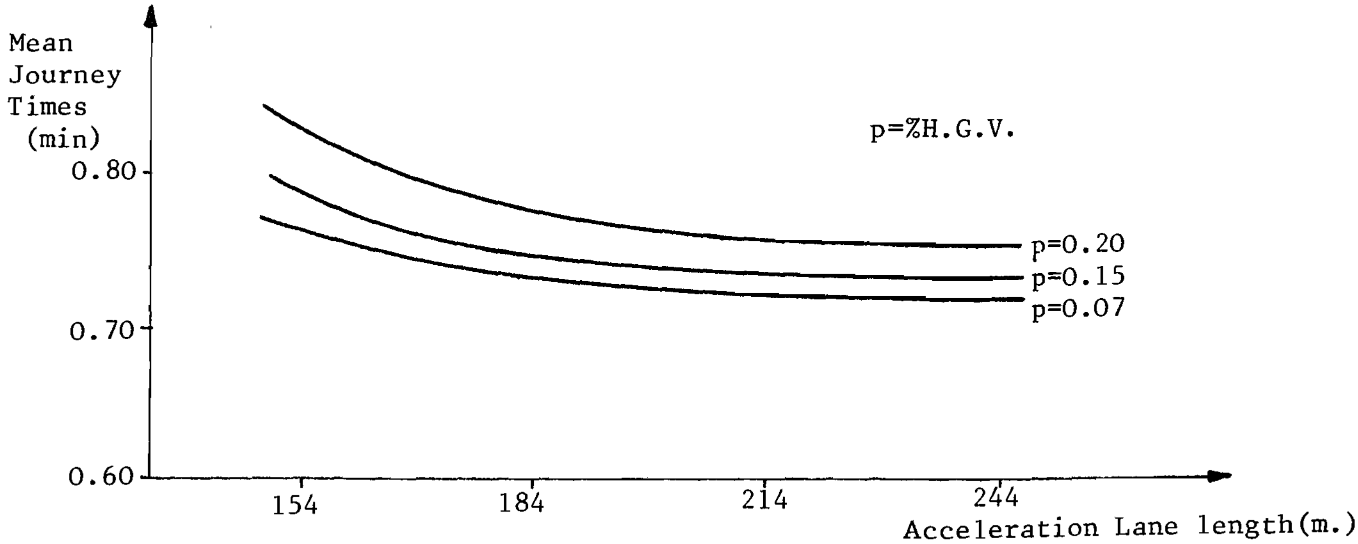


Figure 7.7:Mean Journey Times of merging vehicles for total downstream flow$\geqslant$4800 veh/h

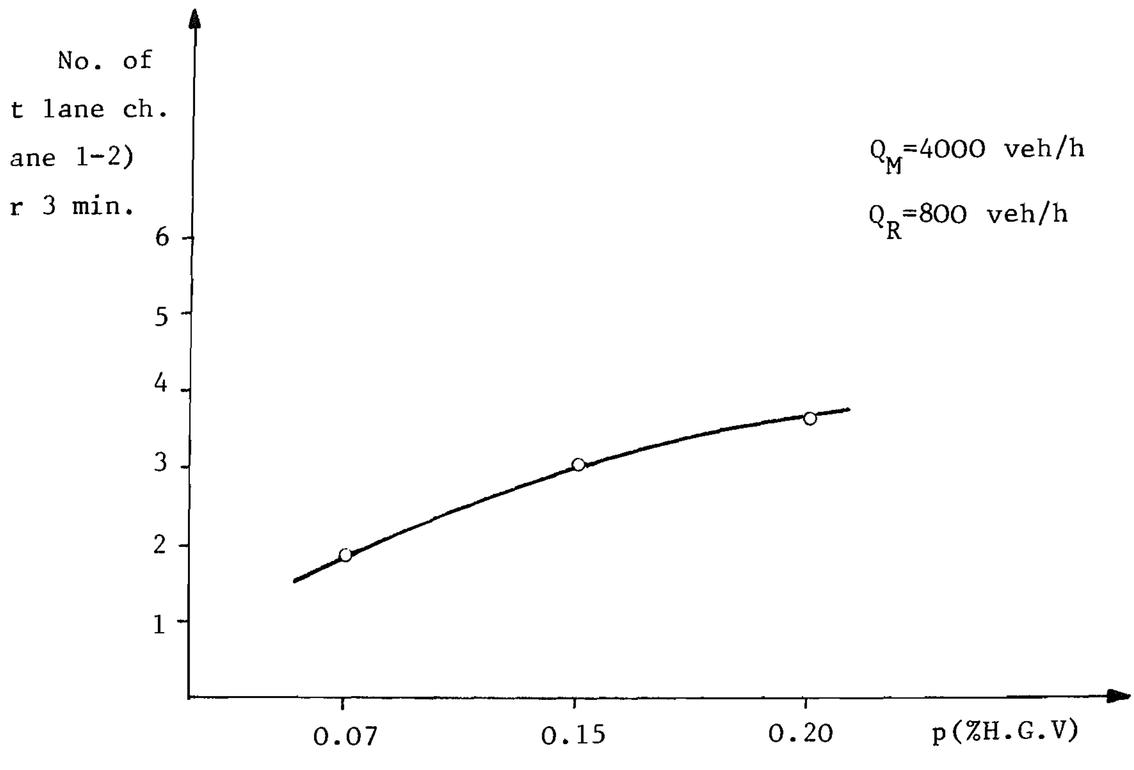


Figure 7.8: Effect of merging flow composition on mainstream flow.

CHAPTER 8

COMMENTS AND CONCLUSIONS

The work described in this thesis is concerned with the traffic behaviour at grade separated intersections, especially the ramp entry as we feel that is the most critical situation. The research has been prompted by problems which exist in two areas : Operation of existing interchanges which are approaching capacity and the need for efficient design at new sites. In order for these problems to be studied, the relationships between geometric and traffic variables should be comprehensively investigated.

The problem of merging has been studied by various researchers using a variety of methods of approach, and the most representative studies have been reviewed in Chapter 2. Empirical results have been shown that the merging process is a 'dynamic phenomenon' and it is affected by certain geometric and traffic factors as relative speed, gap acceptance, acceleration lane length. The analytical formulations use many simplifying assumptions and ignore certain variables which we feel are important. A number of simulation models have also been produced but they have shortcomings, particularly concerning the merging strategy. The lack of sufficient calibration has also resulted in the models not being fully validated. The design standards currently used are based on calculation of maximum allowable flows for given operating conditions, using empirical equations. It can therefore be seen that there is still a need for further investigation into the operation of grade separated interchanges, and the research described in this thesis can answer at least part of that need.

The method of approach is described in Chapter 3. The use of simulation as a method of tackling the problem is justified because of the complexity of the situation and also we are interested in a wide range of conditions to be fully evaluated. The approach has distinctive advantages over the empirical one, which requires extensive measurements and the variability in traffic data of this sort tend to mask any true operating mechanisms. There also may be unacceptable operating conditions or constructions for an empirical test, e.g. reduction in length of acceleration lane. The simulation procedure in this study is microscopic, involving the detailed analysis of the elements comprising the traffic situation and the integration of the relationships from the analysis with the aid of a completely modular computer program.

In the simulation procedure the model formulation consists of the most important step. The model simulates the traffic flow on a three lane motorway section with a ramp entry of a single lane slip road. It takes into account the variability of driver's characteristics, as desired speed and acceleration, gap acceptance; vehicle characteristics as type, length, acceleration capabilities; the stochastic nature of the interacting traffic streams, headway distributions, the driver's response on different flow regimes, free and restrained or car-following movement, lane changing on the motorway and merging behaviour. It is assumed that a vehicle wishing to merge can adopt a range of policies according to its characteristics and the prevailing traffic conditions. The underlying assumptions, parameters and relationships are fully described in Chapter 4. A series of tests (Ref. 132) indicated that the developed model is working correctly.

In order for the proposed model to adequately represent the traffic behaviour, particular emphasis has been given to its Calibration and Validation. A large number of video tapes collected for the Department of Transport have been fully analysed using video and event recording techniques. Sufficient measurements were taken and analysed statistically to determine estimates of the necessary parameters and distributions of the problem variables. Both macroscopic elements such as flows, vehicle composition and microscopic elements such as accepted lead and lag times, headways, speeds and accelerations were evaluated. The data base, the method of analysis and results are described in Chapter 5. In parallel, qualitative observations from the data base and further theoretical study have suggested a series of refinements and improvements in the model.

The model has been tested against the data from the Calibration site and good agreement was obtained. The same good agreement was observed by applying the model to another site not used in the Calibration. Comparisons with theoretical solutions, design procedures and information from other sources have also shown the validity of the proposed model in a wide range of traffic conditions as it is described in Chapter 6. The model was subsequently applied to simulate the traffic operation at ramp entries with different geometric characteristics in order to examine the significance of certain design elements. The results are shown and discussed in Chapter 7.

The following conclusions can be drawn from the study :

The microscopic simulation model has been shown to accurately represent the true situation at a number of sites. Good correlation has also been obtained with other sources.

The delays of merging vehicles under 'ideal' geometric conditions, i.e. flyunder slip road and acceleration lane length 244 m., are low up to high flow levels, and under these conditions the suggested design flows can be accommodated under reasonable delays. At the flow regime near the 'typical capacity' a significant increase and variability of delays and journey times occurs indicating a need for monitoring the interacting traffic streams. The sufficient length of slip road, 450 m. including the 122 m. nose, allows for the merging vehicles to attain the running speed at the motorway lane 1, thus using the acceleration lane as merging or speed-adjustment lane. The relative speeds are still low for a slip road length of 300 m., which is indicated as typical design values (Ref. 90).

The length of acceleration lane does not seem to significantly affect the traffic behaviour under the above stated slip road layout, above a certain threshold. Above a length of 184 m., the delays and journey times are insensitive for all the flow regimes, implying an effective reduction of 60 m. in the acceleration lane length of 244 m. The safety aspect i.e. number of vehicles stopped, has also been examined and it was found unaffected in the variation of the acceleration lane length in the range 184-244 m. Heavy Goods Vehicles have in general lower acceleration capabilities and higher relative speeds at the nose as it is shown in the predicted speed distance profiles, and therefore require a greater acceleration lane length. In the case of high proportion of H.G. Vehicles in the entering stream a reduction of 30 m. (100 ft) in the length of acceleration lane was found to be more rational.*

The gradient on the slip road was found to affect significantly the merging behaviour. At a gradient of 4 per cent which is used as a typical maximum design value, the delays at entry are significantly higher than in flyunder version interchanges. Comparisons of the two situations suggest that gradient can be expressed as an equivalent increase in entry flow but this factor cannot be taken as constant, as it has been suggested (Ref. 108), but increases as mainstream traffic increases and queues are formed.

The importance of the slip road length becomes more pronounced in the case of uphill gradients. The short slip road leads to high relative speeds at the point of merge, and acceptance of larger gaps as it has been shown from the analysis of data. It should be noted that the current design standards do not provide specific values for the length of ramp. An adequate length of slip road should be provided such as the relative speeds to be minimised. If site restrictions occur a longer length of the 'nose' will provide better visibility and will contribute to better and safer traffic operation.**

The effect of acceleration lane length for flyover version ramps was examined assuming various proportions of the H.G. Vehicles, as they are most affected from the gradient. In all the cases a decrease in acceleration lane length caused an increase in delays and journey times and it is indicated that a reduction of the acceleration lane length cannot be considered.

The effect of merging flow on mainstream traffic was investigated by calculating the number of lane changes at the merging area and the lane distribution upstream and downstream from the ramp nose. It was found that the 'move-over' on the main carriageway increases as entry flow increases. The analysis of empirical data has not produced an acceptable statistical relationship due to the complexity of the situation. However it has been shown that the vehicle composition of merging vehicles associated with an uphill gradient have a significant effect on the number of lane changes from the inside motorway lane to the offside, in order that conflicts with slower entering vehicles can be avoided.

* The new standards(Ref. 136) suggest a length of 220 m. for acceleration lane for entering flows less than 800 veh/h. It has been shown from the applications of the model that a flyunder type of interchange can accomodate higher entry flows,e.g. 1100 veh/h, under the same accel.lane length.
** Provision is given by the new standards(Ref. 136) for extra accel. lane length,i.e. 340 m. in total, but no consideration is given to the slip road.

Suggestions for Future Work

During this study effort has been made in order that a realistic model of traffic flow be developed, calibrated and validated. The applications have shown the effects of certain critical design elements in the operation of grade separated interchanges.

The next step in the research in this area seems to be the examination using the model of the traffic management schemes as have been outlined in section 7.1. This is closely related to research into the driver's response on these traffic control features.

Further research is also required into the effects of entry flow to the mainstream traffic and the investigation of conflicts and forced merges.

The effects of roadworks and lane closures on the motorway into the traffic operation is an interesting problem area and the developed model provides a framework for the analysis and evaluation of the situation.

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APPENDIX I

Listing of Program SIRAM

```
C***SIRAM***********************************************************
C A MICROSCOPIC MODEL TO INVESTIGATE THE TRAFFIC BEHAVIOUR
C AT GRADE SEPARATED INTERSECTIONS
C FINAL VERSION-JULY 1981
C*******************************************************************
      PROGRAM SIRAM
C*******************************************************************
      COMMON/B405/STIME/B501/TIME
      CALL INPUT
      CALL INIT
    1 CALL TIMSEQ
      CALL MERG
      CALL CARFOL
      CALL GENER
      IF(TIME.LT.STIME) GO TO 1
      CALL RESULT
      STOP
      END
C*******************************************************************
      SUBROUTINE INPUT
C*******************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/C22/RACC,SRAC/C25/TA/C26/T1,T2/C29/DISTA/B4/PROP(4)
      COMMON/C1/AD1,ADX,AD3,AD4/C20/RV,SR,EV,SV/G2/BR/E10/Q,AF(3)
      COMMON/C11/FTIME/C12/GAP1,SGAP1/C13/GAP2,SGAP2/C14/GAP3,SGAP3
      COMMON/B10/EV1(3),SV1(3)/C40/A1,G1,D1,K1,A2,G2,D2
      COMMON/B2/REACT,TINK/B5/ALM(2),STM(2)/RAM/AC1,DISTA1/DEL/DMIN
      COMMON/CAL/TIMER/B405/STIME/B24/AC/B25/CK/ELK/AMM(3),TM(3)
      COMMON/SPO/GSTOP/ACL/ET
C************************************DATA***************************
      READ(1,*) REACT,CK,AC,DISTA,ET
      READ(1,*) AC1,DISTA1
      READ(1,*) (ALM(J),J=1,2),(STM(J),J=1,2)
      READ(1,*) TINK,STIME,FTIME,TIMER
      READ(1,*) AD1,AD4
      READ(1,*) RV,SR,BR,RACC,SRAC
      READ(1,*) TA,T1,T2,GSTOP,DMIN
      READ(1,*) GAP1,SGAP1,GAP2,SGAP2,GAP3,SGAP3
      READ(5,*) ADX,AD3
      READ(5,*) Q,(AF(J),J=1,3)
      READ(5,*) (PROP(J),J=1,4)
      READ(5,*) (EV1(J),J=1,3),(SV1(J),J=1,3),EV,SV
      READ(5,*) A1,G1,D1,K1,A2,G2,D2
      READ(5,*) (AMM(J),J=1,3),(TM(J),J=1,3)
C*******************************************************************
      WRITE(6,98)
      WRITE(6,3)REACT,CK,AC,DISTA,ET
      WRITE(6,4) AC1,DISTA1
      WRITE(6,5) (ALM(J),J=1,2),(STM(J),J=1,2)
      WRITE(6,5)TINK,STIME,FTIME,TIMER
      WRITE(6,4) AD1,AD4
      WRITE(6,3) RV,SR,BR,RACC,SRAC
      WRITE(6,3) TA,T1,T2,GSTOP,DMIN
      WRITE(6,6) GAP1,SGAP1,GAP2,SGAP2,GAP3,SGAP3
      WRITE(6,26)
   26 FORMAT(27X,'SITE DEPENDENT DATA')
      WRITE(6,40) ADX,AD3
      WRITE(6,66) Q,(AF(J),J=1,3)
```

```
   66 FORMAT(5X,'%FLOW PER MOT. LANE',1X,F10.3,3F6.3)
      WRITE(6,12) (PROP(J),J=1,4)
      WRITE(6,55) (EV1(J),J=1,3),(SV1(J),J=1,3),EV,SV
   55 FORMAT(5X,'SPEED PARAMETERS',1X,6F8.3)
      WRITE(6,70)A1,G1,D1,K1,A2,G2,D2
      WRITE(6,80) (AMM(J),J=1,3),(TM(J),J=1,3)
      CALL PARAM
   80 FORMAT(5X,'MOT.LANE.HEAD-DATA',5X,6F6.3)
   98 FORMAT(27X,'SITE INDEPENDENT DATA')
    3 FORMAT(1X,5F10.3)
    4 FORMAT(1X,2F10.3)
    6 FORMAT(1X,6F10.3)
    5 FORMAT(1X,4F10.3)
   12 FORMAT(5X,'VEHICLE COMPOSITION',1X,3F6.3)
   40 FORMAT(5X,'SITE GEOMETRY',1X,2F10.3)
   70 FORMAT(5X,'RAMP HEAD DATA'1X,3F7.3,I5,3F7.3)
      RETURN
      END
C**********************************************************************
      SUBROUTINE PARAM
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B10/EV1(3),SV1(3)/ARX/EM1(3),EMA1(3)/B400/IC,IV
      COMMON/GAPS/AG1,AG2,AG3,AG4,AG5,AG6/C12/GAP1,SGAP1/B24/AC/CN/DJ
      COMMON/C13/GAP2,SGAP2/C14/GAP3,SGAP3/C22/RACC,SRAC/C23/AO,AB
      COMMON/C20/RV,SR,EV,SV/E3/IK,IL,IO,IF,IQ/C93/IR1/C21/BX1,BX2,AX1
      COMMON/C1/AD1,ADX,AD3,AD4/C45/W1,W2/C51/APO,AP1/C46/DF/D1/RS
      DATA IC,IV,IK,IL,IO,IF,IQ/1,2,3,4,5,6,7/
C
      DO 1 J=1,3
      EM1(J)=EV1(J)-2.*SV1(J)
    1 EMA1(J)=EV1(J)+2.*SV1(J)
      AO=RACC+2.*SRAC
      AB=RACC-2.*SRAC
      BX1=RV+2.*SR
      BX2=RV-2.*SR
      AX1=EV-RV
      W1=AD1+AD4
      W2=AD1-AD3
      DF=50.
      APO=AD1+ADX
      AP1=AD1-DF
      R1=AD3/DF-1.
      IR1=INT(R1)
      AG1=GAP1+SGAP1
      AG2=GAP1-SGAP1
      AG3=GAP2+SGAP2
      AG4=GAP2-SGAP2
      AG5=GAP3+SGAP3
      AG6=GAP3-SGAP3
      DJ=(AC*3600.)/(EXP(1.)*2000.)
      RS=(5.*1.609)/3.6
      RETURN
      END
C**********************************************************************
      SUBROUTINE INIT
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
```

```
      COMMON TR(10000,3)
      COMMON/BVEH/M1,MIC2,MIC3/BB/K/B501/TIME/FL/KE/A1/NZ
      COMMON/BAL/SUM(9),SUMA(9)/BCC/TRR(5000)/UP/X1/OLA/KZ
      COMMON/F10/L1(3)/XX/SUMD/E56/IT/BDD/LA/B16/R/C50/IDV
      COMMON/B19/P(3,3),SUP(3,3)/B2/REACT,TINK/IRA/IW,IG
      COMMON/A33/FLOW(3,2),FLM(3,2),SP1(3),DTM(3),SPM1(3)
      COMMON/BXX/FAP(3)/REJ/IRJ,ILJ/AL2/KX(9),KY(9)
      COMMON/DIO/IML,IFL,ICL
C
      DATA K,M1,MIC2,MIC3/0,0,0,0/,KE,NZ,IML,IFL,ICL/0,0,0,0,0/
      DATA IT,LA,X1,KZ/0,0,0.,0/,R,IW,IG,IDV/0.,0,0,0/
      DATA FLOW,FLM,SP1,DTM,SPM1/6*0.,6*0.,3*0.,3*0.,3*0./
      DATA FAP,IRJ,ILJ/3*0.,0,0/,L1,SUMD/3*0,0./
      DATA SUM,SUMA,KX,KY/9*0.,9*0.,9*0,9*0/
      DATA P,SUP/9*0.,9*0./
      TRR(1)=0.00
      DO 3 JL=1,3
    3 TR(1,JL)=0.00
      TIME=-TINK
      CALL G05CBF(0)
      RETURN
      END
C***********************************************************************
      SUBROUTINE TIMSEQ
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B2/REACT,TINK/B501/TIME/B16/R/CAL/TIMER
C
      TIME=TIME+TINK
      IF(TIME.GE.TIMER) WRITE(6,1) TIME
    1 FORMAT(5X,'TIME=',1X,F10.3)
      R=R+1.
      TIM=180.*R
      IF(TIME.EQ.TIM) GO TO 10
      R=R-1.
      RETURN
   10 CALL STORE
      WRITE(6,1) TIME
      RETURN
      END
C***********************************************************************
      SUBROUTINE MERG
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/D20/SLAG(50)/D40/GAP(50)/E3/IK,IL,IO,IF,IQ/BB/K
      COMMON/FA/SALD(50)/C25/TA/B9/N/B99/VEL(50)/B2/REACT,TINK
      COMMON/B102/POS(50)/C90/ART(50,5)/B501/TIME/C1/AD1,ADX,AD3,AD4
      COMMON/C200/ACCD(50)/BC/ARR(50)/FP/I3(50)/B21/POS1(80,3)
      COMMON/E5/IA(50,5)/FL/KE/B98/ACCEL(50)/SPO/GSTOP/CAL/TIMER
      COMMON/C11/FTIME/E30/ALMR(50)/REJ/IRJ,ILJ/CN/DJ
C
      IF(K.LT.1) RETURN
      KM=0
      KT=KE+1
      IF(K.LE.KT) KT=K
      DO 50 N1=1,KT
      N=N1-KM
```

```
      IF(IA(N,5).EQ.IF) GO TO 50
      IF(IA(N,3).EQ.IQ) GO TO 120
      IF((IA(N,4).EQ.IO) .AND. (N1.EQ.KT)) GO TO 165
      IF(IA(N,4).EQ.IO) GO TO 9
      IF((IA(N,2).EQ.IK) .OR. (IA(N,2).EQ.IL)) GO TO 10
      GO TO 166
  165 IF(POS(N).GT.AD1) GO TO 9
  166 AY=AD1-VEL(N)*TINK
      IF(POS(N).LT.AY) GO TO 50
      ART(N,1)=TIME
      ART(N,2)=VEL(N)
      ART(N,3)=POS(N)
      KE=KE+1
      CALL AGAP
      CALL RSPED
      IF(TIME.GE.TIMER) WRITE(6,300) N,(ART(N,J),J=1,3),GAP(N),
     *ACCD(N)
  300 FORMAT(1X,'VEH. AT MERGE',10X,I5,5F10.3)
      IF(N.EQ.1) GO TO 800
      IF(IA(N-1,5).NE.IF) GO TO 90
  800 CALL EXAM(TN)
      IF(TN.LT.DJ) GO TO 616
      IF(SLAG(N).LT.0.0) GO TO 70
      IF(SLAG(N).GT.GAP(N)) GO TO 70
  616 IF(IA(N,3).EQ.IQ) GO TO 6010
      IF(ARR(N).GT.FTIME) IRJ=IRJ+1
   60 CALL MERGST
      GO TO 50
 6010 IF((GAP(N)-SLAG(N)).GT.0.050) GO TO 60
   70 IF(SALD(N).GE.TA) GO TO 7001
      IF(IA(N,3).EQ.IQ) GO TO 6001
      IF(ARR(N).GT.FTIME) ILJ=ILJ+1
      GO TO 60
 6001 AR=TA-SALD(N)
      IF(AR.GT.0.05) GO TO 60
 7001 IA(N,2)=IK
      GO TO 610
    9 GAP(N)=GSTOP
   10 CALL EXAM(TN)
      IF(TN.LT.DJ) GO TO 50
      IF(SLAG(N).LT.0.0) GO TO 619
      AV=GAP(N)-SLAG(N)
      IF(AV.GT.0.05) GO TO 50
  619 AVA=TA-SALD(N)
      IF(AVA.GT.0.05) GO TO 50
  610 CALL MERGE(KM)
      GO TO 50
   90 IA(N,3)=IQ
      IF(IA(N-1,2).EQ.IK) GO TO 1010
      GO TO 180
 1010 J1=I3(N-1)
      JL=1
      IF(POS(N).LT.POS1(J1,JL)) GO TO 900
      IF(J1.EQ.1) GO TO 180
      IF(POS(N-1).GT.POS1(J1-1,JL)) GO TO 900
  180 CALL QUEUVE
      GO TO 50
  120 IF(N.EQ.1) GO TO 700
```

```
      IF(IA(N-1,5).EQ.IF) GO TO 700
      IF((IA(N,2).EQ.IK) .OR. (IA(N,2).EQ.IL)) GO TO 10
      GO TO 50
  700 IF(IA(N,4).EQ.IO) GO TO 191
      IF((IA(N,2).EQ.IL) .OR. (IA(N,2).EQ.IK)) GO TO 10
  900 ART(N,4)=TIME
      ART(N,5)=POS(N)
      GO TO 800
  191 IF(IA(N,2).EQ.IL) GO TO 9
      ART(N,4)=TIME
      ART(N,5)=POS(N)
      IA(N,2)=IL
      GO TO 9
   50 CONTINUE
      CALL SETTLE
      RETURN
      END
C***********************************************************************
      SUBROUTINE AGAP
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B9/N/C12/GAP1,SGAP1/C13/GAP2,SGAP2/C14/GAP3,SGAP3
      COMMON/GAPS/AG1,AG2,AG3,AG4,AG5,AG6/DIO/IML,IFL,ICL
      COMMON/D40/GAP(50)/B501/TIME/F11/FTIME/D1/RS
C
      CALL RELSP(RVEL)
      CALL NORMAL(AIT)
      IF(RVEL.GT.RS) GO TO 40
      IF(RVEL.LT.-RS) GO TO 50
      GAP(N)=SGAP2*(AIT-6.00)+GAP2
      IF(GAP(N).GT.AG3) GAP(N)=AG3
      IF(GAP(N).LT.AG4) GAP(N)=AG4
      IF(TIME.GT.FTIME) IML=IML+1
      RETURN
   40 GAP(N)=SGAP1*(AIT-6.00)+GAP1
      IF(GAP(N).GT.AG1) GAP(N)=AG1
      IF(GAP(N).LT.AG2) GAP(N)=AG2
      IF(TIME.GT.FTIME) IFL=IFL+1
      RETURN
   50 GAP(N)=SGAP3*(AIT-6.00)+GAP3
      IF(GAP(N).GT.AG5) GAP(N)=AG5
      IF(GAP(N).LT.AG6) GAP(N)=AG6
      IF(TIME.GT.FTIME) ICL=ICL+1
      RETURN
      END
C***********************************************************************
      SUBROUTINE RELSP(RVEL)
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B99/VEL(50)/B21/POS1(80,3)/B102/POS(50)/B9/N
      COMMON/B211/VELX(80,3)/BVEH/M1,MIC2,MIC3
C
      JL=1
      IF(M1.LT.1) GO TO 2
      IF(POS1(M1,JL).GT.POS(N)) GO TO 2
      DO 1 J=1,M1
      IF(POS1(J,JL).LT.POS(N)) GO TO 5
    1 CONTINUE
```

```
    5 RVEL=VELX(J,JL)-VEL(N)
      RETURN
    2 RVEL=-3.00
      RETURN
      END
C*****************************************************************************
      SUBROUTINE EXAM(TN)
C*****************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B102/POS(50)/B99/VEL(50)/D20/SLAG(50)/B211/VELX(80,3)
      COMMON/B21/POS1(80,3)/B9/N/FA/SALD(50)/FP/I3(50)/D1/RS
      COMMON/VEH/I(80,3,3)/B6/ALMA(80,3)/CAL/TIMER/B501/TIME
      COMMON/E5/IA(50,5)/SPO/GSTOP/E3/IK,IL,IO,IF,IQ/BVEH/M1,MIC2,MIC3
C
      JL=1
      IF(M1.LT.1) GO TO 60
      IF(POS(N).GT.POS1(1,JL)) GO TO 16
      IF(POS(N).LT.POS1(M1,JL)) GO TO 70
      DO 50 J=2,M1
      IF(POS1(J,JL).LT.POS(N)) GO TO 40
   50 CONTINUE
   40 TN=POS(N)-POS1(J,JL)
      SLAG(N)=TN/(VELX(J,JL)-VEL(N))
   80 A2=POS1(J-1,JL)-POS(N)
      U=VELX(J-1,JL)-VEL(N)
      IF(U.GT.RS) GO TO 19
      A3=ALMA(J-1,JL)+GSTOP
      IF(U.GT.0.0) GO TO 10
      IF(I(J-1,JL,3).EQ.1) GO TO 10
      SALD(N)=A2/VEL(N)
      GO TO 3
   19 SALD(N)=A2/ALMA(J-1,JL)
      GO TO 3
   10 SALD(N)=A2/A3
    3 I3(N)=J
      GO TO 2
   70 SLAG(N)=20.00
      TN=20.00
      J=M1+1
      GO TO 80
   16 TN=POS(N)-POS1(1,JL)
      SLAG(N)=TN/(VELX(1,JL)-VEL(N))
      GO TO 25
   60 SLAG(N)=20.00
      TN=20.00
   25 SALD(N)=20.000
      I3(N)=1
    2 IF(TIME.LT.TIMER) RETURN
      WRITE(6,4) N,SALD(N),SLAG(N)
    4 FORMAT(16X,'AVAILABLE LEAD-LAG',1X,I5,2F10.3)
      RETURN
      END
C*****************************************************************************
      SUBROUTINE MERGST
C*****************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B9/N/B99/VEL(50)/D20/SLAG(50)/FP/I3(50)/D40/GAP(50)
      COMMON/FA/SALD(50)/B98/ACCEL(50)/F5/IA(50,5)/C25/TA
```

```
      COMMON/CAL/TIMER/E3/IK,IL,IO,IF,IQ/C51/APO,AP1/B501/TIME
      COMMON/B102/POS(50)/FL/KE/BVEH/M1,MIC2,MIC3
C
      JL=1
      J1=I3(N)
      IF(J1.EQ.1) GO TO 190
      JP=J1-1
      CALL CALC(JP,JL)
      IF(IA(N,2).EQ.IK) GO TO 90
      IF(SALD(N).LT.TA) GO TO 8
  190 JP=J1
      CALL CALC(JP,JL)
      IF(IA(N,2).EQ.IK) GO TO 90
    8 CP=2.*(APO-POS(N))
      CA=(VEL(N)**2)/CP
      ACCEL(N)=-CA
      IA(N,2)=IL
   90 IF(TIME.GE.TIMER) WRITE(6,89) N,ACCEL(N)
   89 FORMAT(70X,'ACCEL. VEH. ON MERG. LANE',1X,I4,1X,F10.3)
      IF(N.GE.KE) RETURN
      IF(IA(N,3).NE.IQ) RETURN
      NT=N+1
      DO 1 J=NT,KE
      N=J
      CALL QUEUVE
    1 CONTINUE
      RETURN
      END
      SUBROUTINE CALC(JP,JL)
C***************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B9/N/B99/VEL(50)/D40/GAP(50)/C25/TA/C51/APO,AP1
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B98/ACCEL(50)
      COMMON/B102/POS(50)/B6/ALMA(80,3)/SPO/GSTOP/B2/REACT,TINK
      COMMON/E5/IA(50,5)/E3/IK,IL,IO,IF,IQ/C200/ACCD(50)
C
      IF(JP.EQ.1) GO TO 1
      A1=(VELX(JP,JL)-GAP(N)*VELX(JP-1,JL))/(TA+ALMA(JP-1,JL))
      TB=TA+ALMA(JP-1,JL)
      A2=APO-POS1(JP,JL)-(VELX(JP,JL)+POS1(JP-1,JL)-APO)/TB
      T2=A2/A1
      CO=(2.*(APO-POS(N)-VEL(N)*T2))/(T2**2)
      IF(ACCD(N).LT.CO) RETURN
    1 CA=ACCD(N)
      V1=VEL(N)-VELX(JP,JL)
      SA=POS(N)-POS1(JP,JL)-GAP(N)*V1
      V=V1+CA*GAP(N)
      TO=(-V+SQRT((V**2)-2.*CA*SA))/CA
      IT=INT(TO/TINK)
      T=FLOAT(IT)*TINK+TINK
      APX=POS(N)+T*(VEL(N)+0.50*CA*T)
      IF(APX.LE.APO) GO TO 10
   14 CA=CO
      GO TO 3
   10 IF(JP.EQ.1) GO TO 3
      A1=POS1(JP-1,JL)+VELX(JP-1,JL)*T-APX-ALMA(JP-1,JL)
      AZ=VEL(N)+CA*T
      IF(AZ.LT.VELX(JP-1,JL)) GO TO 11
```

```
      A2=A1/AZ
      IF(A2.GE.TA) GO TO 3
  120 OX=ABS(CA-ACCD(N))
      IF(OX.LT.0.001) GO TO 14
      RETURN
   11 S2=ALMA(JP-1,JL)+GSTOP
      IF(A1.GE.S2) GO TO 3
      GO TO 120
    3 ACCEL(N)=CA
      IA(N,2)=IK
      RETURN
      END
C*****************************************************************
      SUBROUTINE QUEUVE
C*****************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B9/N/B99/VEL(50)/C25/TA/SPO/GSTOP/E30/ALMR(50)
      COMMON/B102/POS(50)/CAL/TIMER/B501/TIME/B98/ACCEL(50)
      COMMON/E5/IA(50,5)/E3/IK,IL,IO,IF,IQ/C51/APO,AP1
      COMMON/B2/REACT,TINK/FA/SALD(50)
C
      S1=POS(N-1)-POS(N)
      S2=ALMR(N-1)+GSTOP
      IF(ACCEL(N-1).GT.0.00) GO TO 600
      IF(VEL(N-1).LT.1.00) GO TO 24
   61 XA=APO-POS(N)-S2
      CA=(VEL(N)**2)/(2.*XA)
      IF(ACCEL(N-1).LT.-0.0) GO TO 10
      CB=0.0
      GO TO 101
   10 XO=(VEL(N-1)**2)/(2.*ABS(ACCEL(N-1)))+S1-S2
      CB=(VEL(N)**2)/(2.*XO)
  101 C=AMAX1(CA,CB)
      GO TO 60
   24 IF(S1.LT.S2) GO TO 612
      C=(VEL(N)**2)/(2.*(S1-S2))
   60 ACCEL(N)=-C
      GO TO 612
  600 IF(SALD(N-1).GT.TA) GO TO 61
      V=VEL(N-1)-VEL(N)
      AZ=S1+TINK*(V+0.5*TINK*(ACCEL(N-1)+ACCEL(N)))
      IF(AZ.GT.S2) GO TO 100
      SX=(2.*(S1-S2+TINK*(V+0.5*ACCEL(N-1)*TINK)))/(TINK**2)
      ACCEL(N)=SX
      GO TO 612
  100 ACCEL(N)=ACCEL(N)
  612 IF(TIME.GE.TIMER) WRITE(6,89) N,ACCEL(N)
   89 FORMAT(80X,I3,1X,F10.3)
      RETURN
      END
C*****************************************************************
      SUBROUTINE SETTLE
C*****************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B102/POS(50)/B99/VEL(50)/B98/ACCEL(50)
      COMMON/C51/APO,AP1/FL/KE/B501/TIME/CAL/TIMER/BB/K
      COMMON/E5/IA(50,5)/E3/IK,IL,IO,IF,IQ
C
      IF(KE.LT.1) RETURN
```

```
      IF(IA(1,3).NE.IQ) RETURN
      IF(IA(1,4).NE.IO) RETURN
      AT=APO-POS(1)
      IF(AT.LT.5.0) RETURN
      DO 5 J=1,KE
    5 POS(J)=POS(J)+AT
      IF(TIME.GE.TIMER) WRITE(6,7) (J,POS(J),VEL(J),ACCEL(J),J=1,KE)
    7 FORMAT(50X,I10,3F10.3)
      KT=KE+1
   10 IF(K.LT.KT) RETURN
      IF(IA(KT,4).NE.IO) GO TO 29
      POS(KT)=POS(KT)+AT
      IF(TIME.GE.TIMER) WRITE(6,7)KT,POS(KT),VEL(KT),ACCEL(KT)
      KT=KT+1
      GO TO 10
   29 RETURN
      END
C***********************************************************************
      SUBROUTINE RSPED
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/C22/RACC,SRAC/C23/AO,AB/B9/N/B99/VEL(50)
      COMMON/C200/ACCD(50)
C
      CALL NORMAL(AIT)
      ACCD(N)=SRAC*(AIT-6.00)+RACC
      IF(ACCD(N).LT.AB) ACCD(N)=AB
      IF(ACCD(N).GT.AO) ACCD(N)=AO
      RETURN
      END
C***********************************************************************
      SUBROUTINE MERGE(KM)
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B9/N/B99/VEL(50)/C200/ACCD(50)/OUS/VELR(50)
      COMMON/E5/IA(50,5)/B98/ACCEL(50)/E3/IK,IL,IO,IF,IQ
      COMMON/ARX/EM1(3),EMA1(3)
C
      IF(VEL(N).LT.EM1(1)) GO TO 72
      IF(VEL(N).LT.VELR(N)) GO TO 1081
      VEL(N)=VELR(N)
      GO TO 40
   72 VEL(N)=EM1(1)
   40 ACCEL(N)=0.00
      GO TO 50
 1081 IF(IA(N,2).EQ.IK) GO TO 1082
      ACCEL(N)=ACCEL(N)
      GO TO 50
 1082 ACCEL(N)=ACCD(N)
   50 IA(N,5)=IF
      CALL PACAL
      CALL DELA
      CALL UPDATE
      KM=KM+1
      RETURN
      END
C***********************************************************************
      SUBROUTINE UPDATE
C***********************************************************************
```

```
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B102/POS(50)/B21/POS1(80,3)/B9/N/FP/I3(50)
      COMMON/BVEH/M1,MIC2,MIC3
C
      JL=1
      IF(M1.LT.1) GO TO 650
      IF(POS(N).LT.POS1(M1,JL)) GO TO 650
      M1=M1+1
      M8= M1-I3(N)
      M=0
      MC=M1
      CALL RESET(MC,JL,M,M8)
      M1=MC
      JJ=I3(N)
      CALL SETR(JJ,JL)
      CALL RESETR
      RETURN
  650 M1=M1+1
      M1=I3(N)
      JJ=M1
      CALL SETR(JJ,JL)
      M1=JJ
      CALL RESETR
      RETURN
      END
C*********************************************************************
      SUBROUTINE PACAL
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/G5/PATH(5000)/B9/N/B501/TIME/CAL/TIMER
      COMMON/C11/FTIME/B2/REACT,TINK/C1/AD1/E56/IT/BC/ARR(50)
      COMMON/B102/POS(50)/B99/VEL(50)/B98/ACCEL(50)
C
      IF(POS(N).GT.AD1) GO TO 1
      PAT=POS(N)+(VEL(N)+0.5*ACCEL(N)*TINK)*TINK-AD1
      GO TO 8
    1 PAT=POS(N)-AD1
    8 IF(TIME.GE.TIMER)  WRITE(6,9) PAT
    9 FORMAT(60X,'PATH=',1X,F10.3)
      IF(ARR(N).LE.FTIME) RETURN
      IT=IT+1
      PATH(IT)=PAT
      RETURN
      END
C***************************************************************
      SUBROUTINE DELA
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/BC/ARR(50)/B501/TIME/E56/IT/CAL/TIMER/C11/FTIME/B9/N
      COMMON/C90/ART(50,5)/IRA/IW,IG/DEL/DMIN/B102/POS(50)
      COMMON/OUP/DELS(5000)/OPS/DELQ(3000)/LAP/HMA(5000)
      COMMON/E55/DELAY(5000)/XAP/SPR(5000)/XEP/HRA(5000)
      COMMON/E5/IA(50,5)/E3/IK,IL,IO,IF,IQ
C
      IF(IA(N,3).EQ.IQ) GO TO 20
      IF(ART(N,2).LT.0.001) GO TO 21
      DELAT=(TIME-ART(N,1))-(POS(N)-ART(N,3))/ART(N,2)+DMIN
      GO TO 24
   21 DELAT=TIME-ART(N,1)+DMIN
```

```
   24 DELĀM=0.00
      DELM=DELAM+DELAT
      GO TO 100
   20 IF(ART(N,2).LT.0.001) GO TO 31
      DELAM=(ART(N,4)-ART(N,1))-(ART(N,5)-ART(N,3))/ART(N,2)
      DELM=(TIME-ART(N,1))-(POS(N)-ART(N,3))/ART(N,2)+DMIN
      GO TO 35
   31 DELAM=ART(N,4)-ART(N,1)
      DELM=TIME-ART(N,1)+DMIN
   35 DELAT=DELM-DELAM
  100 IF(TIME.GE.TIMER)  WRITE(6,85) DELM,DELAT,DELAM
   85 FORMAT(70X,'T-DELAY',1X,F10.3,'MERGE',1X,F10.3,'QUE.',1X,F10.3)
      IF(ARR(N).LE.FTIME) RETURN
      DELAY(IT)=DELM
      SPR(IT)=ART(N,2)
      HRA(IT)=ART(N,1)
      HMA(IT)=TIME
      IF(IA(N,3).EQ.IQ) GO TO 105
      IW=IW+1
      DELS(IW)=DELAT
      RETURN
  105 IG=IG+1
      DELQ(IG)=DELM
      RETURN
      END
C***************************************************************
      SUBROUTINE SETR(JJ,JL)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B212/ACCELX(80,3)/B9/N
      COMMON/BVEH/M1,MIC2,MIC3/B102/POS(50)/B99/VEL(50)/B98/ACCEL(50)
      COMMON/BC/ARR(50)/EAA/AR(80,3)/B40/VEL1(80,3)/OLA/KZ
      COMMON/VEH/I(80,3,3)/B6/ALMA(80,3)/E30/ALMR(50)/E5/IA(50,5)
      COMMON/B30/ACCEX(80,3,10)/B25/CK/B24/AC/OUS/VELR(50)
C
      POS1(JJ,JL)=POS(N)
      I(JJ,JL,2)=JL
      VEL1(JJ,JL)=VELR(N)
      ALMA(JJ,JL)=ALMR(N)
      AR(JJ,JL)=ARR(N)
      I(JJ,JL,1)=IA(N,1)
      I(JJ,JL,3)=1
      VELX(JJ,JL)=VEL(N)
      IF(JJ.EQ.1) GO TO 9
      ACO=POS1(JJ-1,JL)-POS1(JJ,JL)
      IF(ACO.GT.10.0) GO TO 9
      ACCELX(JJ,JL)=0.00
      ACCEX(JJ,JL,KZ)=0.00
      GO TO 10
    9 ACCELX(JJ,JL)=ACCEL(N)
      ACCEX(JJ,JL,KZ)=ACCEL(N)
   10 IF(KZ.GT.1) GO TO 90
      ACCEX(JJ,JL,KZ+1)=0.0
      RETURN
   90 ACCEX(JJ,JL,KZ-1)=0.0
      RETURN
      END
C***************************************************************
```

```
      SUBROUTINE RESETR
C*****************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B9/N/B102/POS(50)/B99/VEL(50)/B98/ACCEL(50)/E30/ALMR(50)
      COMMON/C90/ART(50,5)/OUS/VELR(50)/C200/ACCD(50)/FL/KE/BB/K
      COMMON/E5/IA(50,5)/B103/ACCELT(50,10)/BC/ARR(50)/D40/GAP(50)
C
      KM=1
      K=K-KM
      IF(K.LT.1) GO TO 41
      IF(N.GT.K) GO TO 41
      DO 1 J=N,K
      POS(J)=POS(J+KM)
      VEL(J)=VEL(J+KM)
      ACCEL(J)=ACCEL(J+KM)
      ALMR(J)=ALMR(J+KM)
      VELR(J)=VELR(J+KM)
      ARR(J)=ARR(J+KM)
      DO 3 I1=1,5
    3 IA(J,I1)=IA(J+KM,I1)
      DO 14 I2=1,2
   14 ACCELT(J,I2)=ACCELT(J+KM,I2)
    1 CONTINUE
   41 IF(N.EQ.KE) GO TO 51
      KE=KE-KM
      IF(KE.LT.1) RETURN
      DO 2 J=N,KE
      GAP(J)=GAP(J+KM)
      ACCD(J)=ACCD(J+KM)
      DO 5 I1=1,5
    5 ART(J,I1)=ART(J+KM,I1)
    2 CONTINUE
      RETURN
   51 KE=KE-KM
      RETURN
      END
C*******************************************************************
      SUBROUTINE CARFOL
C*****************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/BVEH/M1,MIC2,MIC3
      COMMON/OLA/KZ/B2/REACT,TINK/UP/X1/B501/TIME
C
      X1=X1+1.
      TP=X1*REACT
      TP1=ABS(TIME-TP)
      IF(TP1.LE.0.001) GO TO 1
      KZ=KZ+1
      GO TO 2
    1 KZ=1
    2 MA=M1
      JL=1
      CALL REAM(MA,JL)
      MA=MIC2
      JL=2
      CALL REAM(MA,JL)
      MA=MIC3
      JL=3
      CALL REAM(MA,JL)
```

```
      CALL CHANGE
      CALL CARAMP(TP1)
      CALL CONTROL
      CALL CHECK
      RETURN
      END
C**********************************************************************
      SUBROUTINE REAM(MA,JL)
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B212/ACCELX(80,3)
      COMMON/B30/ACCEX(80,3,10)/B24/AC/B25/CK/B2/REACT,TINK
      COMMON/C29/DISTA/ACL/ET/B501/TIME/CAL/TIMER/CN/DJ
      COMMON/B40/VEL1(80,3)/OLA/KZ
C
      IF(TIME.GE.TIMER) WRITE(6,41) MA,JL
      IF(MA.LT.1) RETURN
      DO 1 J=1,MA
      ACCE1=ACCEX(J,JL,KZ)
      CALL MOTION(J,JL,ACCE1)
    1 CONTINUE
      IF(TIME.LT.TIMER) GO TO 4
      WRITE(6,2) (POS1(J,JL),VELX(J,JL),ACCELX(J,JL),J=1,MA)
    4 ACCEX(1,JL,KZ)=CK*(VEL1(1,JL)-VELX(1,JL))
      IF(MA.LT.2) RETURN
      DO 5 J=2,MA
      AT=CK*(VEL1(J,JL)-VELX(J,JL))
      S=POS1(J-1,JL)-POS1(J,JL)
      TO=S/VELX(J,JL)
      IF(TO.GT.DISTA) GO TO 3
      U=VELX(J-1,JL)-VELX(J,JL)
      ACC=(AC*U)/S
      AM=(U/REACT)-ABS(ACCELX(J-1,JL))
      IF(AM.LT.0.0) ET1=AMAX1(AM,ET)
      IF(AM.GE.0.0) ET2=AMIN1(AT,AM)
      IF((ACC.LT.0.0) .AND. (ACC.LT.ET1)) ACC=ET1
      IF((ACC.GE.0.0) .AND. (ACC.GT.ET2)) ACC=ET2
      ACCEX(J,JL,KZ)=ACC
      GO TO 5
    3 ACCEX(J,JL,KZ)=AT
    5 CONTINUE
    2 FORMAT(1X,12F10.3)
   41 FORMAT(30X,'MOTORWAY VEHICLES',1X,I10,'IN LANE',1X,I3)
      RETURN
      END
C**********************************************************************
      SUBROUTINE MOTION(J,JL,ACCE1)
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B212/ACCELX(80,3)
      COMMON/B2/REACT,TINK
C
      VA=VELX(J,JL)
      VELX(J,JL)=VA+0.5*TINK*(ACCELX(J,JL)+ACCE1)
      POS1(J,JL)=POS1(J,JL)+0.5*TINK*(VA+VELX(J,JL))
      ACCELX(J,JL)=ACCE1
      CALL HESP(J,JL)
      RETURN
      END
```

```
C********************************************************************
      SUBROUTINE HESP(J,JL)
C************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B21/POS1(80,3)/B501/TIME/BXA/TIN(3000)/C1/AD1
      COMMON/B2/REACT,TINK/B211/VELX(80,3)/C11/FTIME/BXX/FAP(3)
      COMMON/EAA/AR(80,3)/KAM/SPM(3000)/C46/DF
C
      IF(AR(J,JL).LE.FTIME) RETURN
      ADA=AD1-DF
      XOS=POS1(J,JL)-VELX(J,JL)*TINK
      IF(POS1(J,JL).LT.ADA) RETURN
      IF(XOS.GE.ADA) RETURN
      FAP(JL)=FAP(JL)+1.
      IF(JL.GT.1) RETURN
      IX=INT(FAP(JL))
      TIN(IX)=TIME
      SPM(IX)=VELX(J,JL)
      RETURN
      END
C************************************************************************
      SUBROUTINE CARAMP(TP1)
C************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/BB/K/B102/POS(50)/B99/VEL(50)/G2/BR/FL/KE/UP/X1
      COMMON/B103/ACCELT(50,10)/B98/ACCEL(50)/CAL/TIMER/OA/IO
      COMMON/ACL/ET/RAM/AC1,DISTA1/OUS/VELR(50)/B501/TIME
      COMMON/OLA/KZ/E5/IA(50,5)/B24/AC/C27/T2/SPO/GSTOP
      COMMON/E30/ALMR(50)
C
      KT=KE+1
      IF(K.LT.KT) GO TO 5
      IF(TIME.GE.TIMER) WRITE(6,518)
  518 FORMAT(40X,'VEH. ON RAMP')
      DO 31 I=KT,K
      IF(IA(I,4).EQ.IO) GO TO 900
      ACCE=ACCELT(I,KZ)
      CALL UPDATR(I,ACCE)
      IF(VEL(I).GT.0.00) GO TO 900
      VEL(I)=0.0
      ACCEL(I)=0.00
      DO 23 I1=1,2
   23 ACCELT(I,I1)=0.00
      IA(I,4)=IO
  900 CALL SDPROF(I)
   31 CONTINUE
    5 CALL ALMT
      IF(TP1.GT.0.001) X1=X1-1.
      IF(K.GT.KT) GO TO 1900
      IF(K.LT.KT) RETURN
      I=KT
      IF(IA(I,4).EQ.IO) RETURN
      AT=BR*(VELR(I)-VEL(I))
      GO TO 310
 1900 DO 2 I=KT,K
      IF(IA(I,4).EQ.IO) GO TO 2
      AT=BR*(VELR(I)-VEL(I))
      IF(I.EQ.KT) GO TO 310
```

```
      F1=POS(I-1)-POS(I)
      FM=F1/VEL(I)
      IF(FM.GT.DISTA1) GO TO 3
      U1=VEL(I-1)-VEL(I)
      ACC=(AC1*U1)/F1
      IF(ACC.LT.ET) ACC=ET
      IF(ACC.GT.AT) ACC=AT
      ACCELT(I,KZ)=ACC
      GO TO 2
  310 IF(KE.GT.0) GO TO 555
    3 ACCELT(I,KZ)=AT
      GO TO 90
  555 A4=POS(KE)-POS(I)
      A1=A4/VEL(I)
      IF(A1.GE.T2) GO TO 3
      IF(IA(KE,4).EQ.IO) GO TO 666
      IF(A1.GE.DISTA1) GO TO 3
      ACCELT(I,KZ)=AC1*(VEL(KE)-VEL(I))/A4
      GO TO 90
  666 AN1=AC1*(VEL(KE)-VEL(I))/A4
      AN=-(VEL(I)**2)/(2.*(POS(KE)-POS(I)-ALMR(KE)-GSTOP))
      ACCELT(I,KZ)=AMAX1(AN1,AN)
   90 IF(K.EQ.KT) RETURN
    2 CONTINUE
      RETURN
      END
C*******************************************************************
      SUBROUTINE UPDATR(I,ACCE)
C**************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B102/POS(50)/B2/REACT,TINK/B98/ACCEL(50)/B99/VEL(50)
      COMMON/B501/TIME/CAL/TIMER
C
      V1=VEL(I)
      VEL(I)=V1+0.5*TINK*(ACCE+ACCEL(I))
      POS(I)=POS(I)+0.5*TINK*(V1+VEL(I))
      ACCEL(I)=ACCE
      IF(TIME.GE.TIMER) WRITE(6,30) I,POS(I),VEL(I),ACCEL(I)
   30 FORMAT(30X,I10,3F10.3)
      RETURN
      END
C*******************************************************************
      SUBROUTINE ALMT
C**************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B103/ACCELT(50,10)/B99/VEL(50)/OA/IO/OB/IF/OP/IQ
      COMMON/B98/ACCEL(50)/B102/POS(50)/E5/IA(50,5)/FL/KE
      COMMON/CAL/TIMER/B501/TIME/C50/IDV/BC/ARR(50)/C11/FTIME
C
      IF(KE.LT.1) RETURN
      DO 301 J=1,KE
      IF((IA(J,5).EQ.IF) .OR. (IA(J,4).EQ.IO)) GO TO 301
      DO 131 I1=1,2
  131 ACCELT(J,I1)=ACCEL(J)
  301 CONTINUE
      IF(TIME.GE.TIMER) WRITE(6,31) KE
   31 FORMAT(20X,'VEHICLES ON ACCEL. LANE',1X,I10)
      DO 302 I=1,KE
      IF((IA(I,5).EQ.IF) .OR. (IA(I,4).EQ.IO)) GO TO 302
```

```
      ACCE=ACCEL(I)
      CALL UPDATR(I,ACCE)
      IF(VEL(I).GT.0.01) GO TO 302
      IA(I,4)=IO
      IF((ARR(I).LE.FTIME) .OR. (IA(I,3).EQ.IQ)) GO TO 10
      IDV=IDV+1
   10 VEL(I)=0.00
      ACCEL(I)=0.00
      DO 23 I2=1,2
   23 ACCELT(I,I2)=0.00
      IF(TIME.GE.TIMER) WRITE(6,28) I,POS(I),VEL(I),ACCEL(I)
   28 FORMAT(15X,'VEH. STOPPED',1X,I10,3F10.3)
  302 CONTINUE
      RETURN
      END
C*****************************************************************
      SUBROUTINE SDPROF(I)
C*******************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B102/POS(50)/E5/IA(50,5)/B400/IC,IV/B99/VEL(50)
      COMMON/B2/REACT,TINK/BAL/SUM(9),SUMA(9)/C11/FTIME
      COMMON/AL2/KX(9),KY(9)/BC/ARR(50)/C93/IR1/C46/DF/C45/W1,W2
C
      IF(ARR(I).LE.FTIME) RETURN
      DO 1 J=1,IR1
      AJ=FLOAT(J)
      ADF=W2+(AJ*DF)
      IF(POS(I).LT.ADF) RETURN
      FF=POS(I)-VEL(I)*TINK
      IF(FF.LT.ADF) GO TO 5
      IF(J.GE.IR1) RETURN
    1 CONTINUE
    5 IF(IA(I,1).EQ.IV) GO TO 6
      KX(J)=KX(J)+1
      SUM(J)=SUM(J)+VEL(I)
      RETURN
    6 KY(J)=KY(J)+1
      SUMA(J)=SUMA(J)+VEL(I)
      RETURN
      END
C*******************************************************************
      SUBROUTINE CHANGE
C*******************************************************************
      IMPLICIT INTEGER(I-N)
      COMMON/BVEH/M1,MIC2,MIC3
C
      IF(M1.LE.1) GO TO 150
      IF(MIC2.LT.1) GO TO 260
      JL=1
      MB=M1
      MC=MIC2
      M=1
      CALL LANECH(JL,MB,MC,M)
      IF(MC.LE.MIC2) GO TO 150
      MIC2=MC
      M1=MB
      RETURN
  150 IF(MIC2.LE.1) GO TO 250
      IF(MIC3.LT.1) GO TO 260
```

```
      JL=2
      MB=MIC2
      MC=MIC3
      M=1
      CALL LANECH(JL,MB,MC,M)
      IF(MC.LE.MIC3) GO TO 200
      MIC3=MC
      MIC2=MB
      RETURN
  200 MC=M1
      IF(MC.LT.1) GO TO 250
      MB=MIC2
      M=-1
      CALL LANECH(JL,MB,MC,M)
      IF(MC.LE.M1) GO TO 250
      M1=MC
      MIC2=MB
      RETURN
  250 IF(MIC3.LE.1) GO TO 260
      IF(MIC2.LE.1) GO TO 260
      JL=3
      MB=MIC3
      MC=MIC2
      M=-1
      CALL LANECH(JL,MB,MC,M)
      MIC2=MC
      MIC3=MB
  260 RETURN
      END
C**********************************************************************
      SUBROUTINE LANECH(JL,MB,MC,M)
C************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B211/VELX(80,3)/B21/POS1(80,3)/B40/VEL1(80,3)
      COMMON/VEH/I(80,3,3)/B400/IC,IV/C29/DISTA
C
      IF(M.LT.1) GO TO 1000
      DO 602 J=2,MB
      TO=(POS1(J-1,JL)-POS1(J,JL))/VELX(J,JL)
      IF(TO.GT.DISTA) GO TO 602
      IF(VEL1(J,JL).LE.VELX(J-1,JL)) GO TO 602
      IF(I(J,JL,1).EQ.IV) GO TO 4
      GO TO 6
    4 ID=JL+M
      IF(ID.EQ.3) GO TO 602
    6 JO=J
      CALL LANEGAP(JO,JL,M,MB,MC)
  602 CONTINUE
      RETURN
 1000 DO 1003 J=1,MB
      IF(I(J,JL,2).EQ.JL) GO TO 1003
      JO=J
      CALL LANEGAP(JO,JL,M,MB,MC)
 1003 CONTINUE
      RETURN
      END
C************************************************************************
```

```
      SUBROUTINE LANEGAP(JO,JL,M,MB,MC)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B211/VELX(80,3)/B21/POS1(80,3)/VEH/I(80,3,3)
      COMMON/C25/TA/C26/T1,T2/C29/DISTA/CAL/TIMER/B501/TIME
      COMMON/C51/APO,AP1/B19/P(3,3),SUP(3,3)
      COMMON/C11/FTIME/EAA/AR(80,3)
C
      J=JO
      IF(POS1(J,JL).GT.POS1(1,JL+M)) GO TO 612
      IF(POS1(J,JL).LT.POS1(MC,JL+M)) RETURN
      DO 603 J1=2,MC
      IF(POS1(J,JL).GT.POS1(J1,JL+M)) GO TO 604
  603 CONTINUE
  604 TT=(POS1(J1-1,JL+M)-POS1(J,JL))/VELX(J,JL)
      IF(TT.LT.TA) RETURN
       IF(VELX(J,JL).GT.VELX(J1-1,JL+M)) RETURN
  407 T=(POS1(J,JL)-POS1(J1,JL+M))/(VELX(J1,JL+M))
      IF(T.LT.T1) RETURN
      IF(T.GE.DISTA) GO TO 6051
      IF(VELX(J,JL).LT.VELX(J1,JL+M)) RETURN
 6051 IF(TIME.GE.TIMER) WRITE(6,7) I(J,JL,2),JL,M,T,T2,TIME
    7 FORMAT(1X,'OR.LA',1X,I5,'CUR-LA',1X,I5,1X,I5,3F10.3)
      GO TO 605
  612 J1=1
      GO TO 407
  605 MC=MC+1
      M8=MC-J1
      CALL RESET(MC,JL,M,M8)
      JT=J1
      JB=J
      CALL SETLC(JT,JB,JL,M)
      J=JB
      J1=JT
      IF((AR(J,JL).LE.FTIME) .OR.(I(J,JL,3).EQ.1)) GO TO 750
      IF((POS1(J,JL).LT.AP1) .OR. (POS1(J,JL).GT.APO)) GO TO 750
      P(JL,JL+M)=P(JL,JL+M)+1.
  750 MB=MB-1
      IR=1
      DO 630 JA=J,MB
  630 CALL SET(JA,IR,JL)
      RETURN
      END
C****************************************************************
      SUBROUTINE SETLC(JT,JB,JL,M)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/EAA/AR(80,3)/B40/VEL1(80,3)/VEH/I(80,3,3)
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B212/ACCELX(80,3)
      COMMON/B6/ALMA(80,3)/B30/ACCEX(80,3,10)/OLA/KZ/B25/CK
C
      VELX(JT,JL+M)=VELX(JB,JL)
      POS1(JT,JL+M)=POS1(JB,JL)
      VEL1(JT,JL+M)=VEL1(JB,JL)
      DO 1 I1=1,3
    1 I(JT,JL+M,I1)=I(JB,JL+M,I1)
      ALMA(JT,JL+M)=ALMA(JB,JL)
      AR(JT,JL+M)=AR(JB,JL)
```

```
      ACCELX(JT,JL+M)=ACCELX(JB,JL)
      ACCEX(JT,JL+M,KZ)=CK*(VEL1(JT,JL+M)-VELX(JT,JL+M))
      IF(KZ.EQ.1) GO TO 2
      ACCEX(JT,JL+M,KZ-1)=ACCEX(JB,JL,KZ-1)
      RETURN
    2 ACCEX(JT,JL+M,KZ+1)=ACCEX(JB,JL,KZ+1)
      RETURN
      END
C**********************************************************************
      SUBROUTINE RESET(MC,JL,M,M8)
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/EAA/AR(80,3)/B40/VEL1(80,3)/VEH/I(80,3,3)
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B212/ACCELX(80,3)
      COMMON/B6/ALMA(80,3)/B30/ACCEX(80,3,10)
C
      DO 670 K2=1,M8
      J3=MC-K2
      POS1(J3+1,JL+M)=POS1(J3,JL+M)
      VELX(J3+1,JL+M)=VELX(J3,JL+M)
      ACCELX(J3+1,JL+M)=ACCELX(J3,JL+M)
      VEL1(J3+1,JL+M)=VEL1(J3,JL+M)
      DO 2 I1=1,3
    2 I(J3+1,JL+M,I1)=I(J3,JL+M,I1)
      ALMA(J3+1,JL+M)=ALMA(J3,JL+M)
      AR(J3+1,JL+M)=AR(J3,JL+M)
      DO 1 I1=1,2
    1 ACCEX(J3+1,JL+M,I1)=ACCEX(J3,JL+M,I1)
  670 CONTINUE
      RETURN
      END
C**********************************************************************
      SUBROUTINE SET(JA,IR,JL)
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/EAA/AR(80,3)/B40/VEL1(80,3)/VEH/I(80,3,3)
      COMMON/B6/ALMA(80,3)/B30/ACCEX(80,3,10)
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B212/ACCELX(80,3)
C
      POS1(JA,JL)=POS1(JA+IR,JL)
      VELX(JA,JL)=VELX(JA+IR,JL)
      ACCELX(JA,JL)=ACCELX(JA+IR,JL)
      VEL1(JA,JL)=VEL1(JA+IR,JL)
      ALMA(JA,JL)=ALMA(JA+IR,JL)
      AR(JA,JL)=AR(JA+IR,JL)
      DO 2 I1=1,3
    2 I(JA,JL,I1)=I(JA+IR,JL,I1)
      DO 1 I2=1,2
    1 ACCEX(JA,JL,I2)=ACCEX(JA+IR,JL,I2)
      RETURN
      END
C**********************************************************************
      SUBROUTINE CONTROL
C**********************************************************************
      IMPLICIT INTEGER(I-N)
      COMMON/BVEH/M1,MIC2,MIC3
C
      MF=M1
```

```
      JL=1
      CALL CONTROL1(MF,JL)
      M1=MF
      MF=MIC2
      JL=2
      CALL CONTROL1(MF,JL)
      MIC2=MF
      MF=MIC3
      JL=3
      CALL CONTROL1(MF,JL)
      MIC3=MF
      RETURN
      END
C*************************************************************************
      SUBROUTINE CONTROL1(MF,JL)
C*************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B21/POS1(80,3)/B40/VEL1(80,3)/B501/TIME
      COMMON/VEH/I(80,3,3)/EAA/AR(80,3)/C11/FTIME/C45/W1,W2
C
      IR=0
      IF(MF.LT.1) RETURN
      DO 2 J=1,MF
      IF(POS1(J,JL).LT.W1) GO TO 2
      IR=IR+1
      IF(AR(J,JL).LE.FTIME) GO TO 2
      DT=TIME-AR(J,JL)
      IF(I(J,JL,3).EQ.1) GO TO 5
      SP=POS1(J,JL)/DT
      DK=POS1(J,JL)/VEL1(J,JL)
      DE=DT-DK
      GO TO 7
    5 SP=(POS1(J,JL)-W2)/DT
      DK=(POS1(J,JL)-W2)/VEL1(J,JL)
      DE=DT-DK
    7 CALL SDTATA(J,JL,DT,SP)
    2 CONTINUE
      MF=MF-IR
      IF(MF.LT.1) RETURN
      DO 3 JA=1,MF
    3 CALL SET(JA,IR,JL)
      RETURN
      END
C*************************************************************************
      SUBROUTINE SDTATA(J,JL,DT,SP)
C*************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/VEH/I(80,3,3)/B400/IC,IV/XX/SUMD
      COMMON/A33/FLOW(3,2),FLM(3,2),SP1(3),DTM(3),SPM1(3)
C
      IF(I(J,JL,1).EQ.IV) GO TO 1
      FLOW(JL,IC)=FLOW(JL,IC)+1.
      GO TO 2
    1 FLOW(JL,IV)=FLOW(JL,IV)+1.
    2 IF(I(J,JL,3).EQ.1) GO TO 3
      DTM(JL)=DTM(JL)+DT
      SP1(JL)=SP1(JL)+SP
      RETURN
```

```
    3 IF(I(J,JL,1).EQ.IV) GO TO 4
      FLM(JL,IC)=FLM(JL,IC)+1.
      GO TO 5
    4 FLM(JL,IV)=FLM(JL,IV)+1.
    5 SPM1(JL)=SPM1(JL)+SP
      SUMD=SUMD+DT
      RETURN
      END
C***********************************************************************
      SUBROUTINE CHECK
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B21/POS1(80,3)/B211/VELX(80,3)/B212/ACCELX(80,3)
      COMMON/BVEH/M1,MIC2,MIC3/B501/TIME/B102/POS(50)/BB/K
C
      IF(TIME.LE.5.) RETURN
      JL=1
      IF(M1.LE.2) GO TO 14
      DO 10 J=2,M1
      IF(POS1(J,JL).GE.POS1(J-1,JL)) GO TO 80
   10 CONTINUE
   14 JL=2
      IF(MIC2.LE.2) GO TO 16
      DO 11 J=2,MIC2
      IF(POS1(J,JL).GE.POS1(J-1,JL)) GO TO 80
   11 CONTINUE
   16 JL=3
      IF(MIC3.LE.2) GO TO 700
      DO 12 J=2,MIC3
      IF(POS1(J,JL).GE.POS1(J-1,JL)) GO TO 80
   12 CONTINUE
  700 IF(K.LT.2) RETURN
      DO 66 J=2,K
      IF(POS(J).GE.POS(J-1)) GO TO 800
   66 CONTINUE
      RETURN
   80 WRITE(6,54) TIME,JL
   54 FORMAT(10X,'ERROR!',1X,'AT TIME=',1X,F10.3,'IN LANE',1X,I5)
      WRITE(6,55) POS1(J,JL),VELX(J,JL),ACCELX(J,JL)
      WRITE(6,55) POS1(J-1,JL),VELX(J-1,JL),ACCELX(J-1,JL)
      WRITE(6,555) J
   55 FORMAT(10X,3F10.3)
      GO TO 700
  800 WRITE(6,55) TIME,POS(J),POS(J-1)
  555 FORMAT(20X,I10)
      RETURN
      END
C*****************************************************************
      SUBROUTINE GENER
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON TR(10000,3)
      COMMON/BVEH/M1,MIC2,MIC3/F10/L1(3)/B501/TIME
C
      JL=1
      L1(JL)=L1(JL)+1
      L=L1(JL)
      IF(TIME.LT.TR(L,JL)) GO TO 1020
```

```
      M1=M1+1
      MK=M1
      CALL ASSIGN(MK,JL)
      GO TO 1060
 1020 L1(JL)=L1(JL)-1
 1060 JL=2
      L1(JL)=L1(JL)+1
      L=L1(JL)
      IF(TIME.LT.TR(L,JL)) GO TO 1030
      MIC2=MIC2+1
      MK=MIC2
      CALL ASSIGN(MK,JL)
      GO TO 1070
 1030 L1(JL)=L1(JL)-1
 1070 JL=3
      L1(JL)=L1(JL)+1
      L=L1(JL)
      IF(TIME.LT.TR(L,JL)) GO TO 1050
      MIC3=MIC3+1
      MK=MIC3
      CALL ASSIGN(MK,JL)
      GO TO 1021
 1050 L1(JL)=L1(JL)-1
 1021 CALL ARAMP
      RETURN
      END
C*************************************************************************
      SUBROUTINE ASSIGN(MK,JL)
C*************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B211/VELX(80,3)/B212/ACCELX(80,3)/B21/POS1(80,3)
      COMMON/VEH/I(80,3,3)/B501/TIME/CAL/TIMER/C29/DISTA
      COMMON/B40/VEL1(80,3)/B30/ACCEX(80,3,10)/EAA/AR(80,3)
C
      AR(MK,JL)=TIME
      I(MK,JL,2)=JL
      CALL APROP(MK,JL)
      CALL ALMC(MK,JL)
      CALL DEVEL(MK,JL)
      IF(MK.EQ.1) GO TO 40
      A=POS1(MK-1,JL)/VEL1(MK,JL)
      IF(A.GT.DISTA) GO TO 40
      CALL INVEL(JL,MK)
      GO TO 50
   40 VELX(MK,JL)=VEL1(MK,JL)
      ACCELX(MK,JL)=0.00
      DO 32 I1=1,2
   32 ACCEX(MK,JL,I1)=ACCELX(MK,JL)
   50 POS1(MK,JL)=0.00
      I(MK,JL,3)=0
      CALL HMOT(JL)
      IF(TIME.LT.TIMER) RETURN
      WRITE(6,60) MK,(I(MK,JL,I1),I1=1,3),AR(MK,JL),POS1(MK,JL),
     *VEL1(MK,JL),VELX(MK,JL),ACCELX(MK,JL)
   60 FORMAT(10X,'MOTORWAY VEHICLE CHARACT.'/1X,4I10,
     *1X,5F10.3,I10)
      RETURN
      END
```

```
C*********************************************************************
      SUBROUTINE APROP(MK,JL)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B4/PROP(4)/B400/IC,IV/VEH/I(80,3,3)
      COMMON/E5/IA(50,5)
C
      IF(JL.EQ.4) GO TO 100
      X=G05CAF(X)
      IF(X.LE.PROP(JL)) GO TO 301
      I(MK,JL,1)=IC
      RETURN
  301 I(MK,JL,1)=IV
      RETURN
  100 X=G05CAF(X)
      IF(X.LE.PROP(JL)) GO TO 101
      IA(MK,1)=IC
      RETURN
  101 IA(MK,1)=IV
      RETURN
      END
C*********************************************************************
      SUBROUTINE ALMC(MK,JL)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B5/ALM(2),STM(2)/B400/IC,IV/B6/ALMA(80,3)
      COMMON/VEH/I(80,3,3)/E5/IA(50,5)/E30/ALMR(50)
C
      CALL NORMAL(AIT)
      I1=1
      IF(JL.EQ.4) GO TO 6
      IF(I(MK,JL,1).EQ.IV) I1=I1+1
      ALMA(MK,JL)=STM(I1)*(AIT-6.00)+ALM(I1)
      RETURN
    6 IF(IA(MK,1).EQ.IV) I1=I1+1
      ALMR(MK)=STM(I1)*(AIT-6.00)+ALM(I1)
      RETURN
      END
C*********************************************************************
      SUBROUTINE DEVEL(MK,JL)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B10/EV1(3),SV1(3)/B40/VEL1(80,3)
      COMMON/ARX/EM1(3),EMA1(3)
C
      CALL NORMAL(AIT)
      VEL1(MK,JL)=SV1(JL)*(AIT-6.00)+EV1(JL)
      IF(VEL1(MK,JL).GT.EMA1(JL)) VEL1(MK,JL)=EMA1(JL)
      IF(VEL1(MK,JL).LT.EM1(JL)) VEL1(MK,JL)=EM1(JL)
      RETURN
      END
C*********************************************************************
      SUBROUTINE NORMAL(AIT)
C*********************************************************************
      IMPLICIT REAL(A,X)
      AIT=0.00
      DO 1 J=1,12
      X=G05CAF(X)
```

```
      AIT=AIT+X
    1 CONTINUE
      RETURN
      END
C**********************************************************************
      SUBROUTINE INVEL(JL,MK)
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B21/POS1(80,3)/B6/ALMA(80,3)/OLA/KZ
      COMMON/B30/ACCEX(80,3,10)/B212/ACCELX(80,3)
      COMMON/B24/AC/B40/VEL1(80,3)/B211/VELX(80,3)
C
      U=AC/POS1(MK-1,JL)
      IF(VELX(MK-1,JL).GE.VEL1(MK,JL)) GO TO 73
      AT=U*(VELX(MK-1,JL)-VEL1(MK,JL))
      AIV=VEL1(MK,JL)+AT
      SA=AIV-VELX(MK-1,JL)
      IF(SA.LT.0.0) GO TO 11
   13 VELX(MK,JL)=VELX(MK-1,JL)
      GO TO 12
   11 IF(SA.LT.-1.00) GO TO 13
      VELX(MK,JL)=AIV
      GO TO 12
   73 VELX(MK,JL)=VEL1(MK,JL)
   12 ACCELX(MK,JL)=0.0
      DO 2 I1=1,2
    2 ACCEX(MK,JL,I1)=0.0
      RETURN
      END
C**********************************************************************
      SUBROUTINE HMOT(JL)
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON TR(10000,3)
      COMMON/E10/Q,AF(3)/F10/L1(3)
      COMMON/ELK/AMM(3),TM(3)
C
      T=TM(JL)
      AM=AMM(JL)
      I2=L1(JL)+1
      X=G05CAF(X)
      XB=-(AM-T)*ALOG(X)+T
      TR(I2,JL)=TR(I2-1,JL)+XB
      RETURN
      END
C**********************************************************************
      SUBROUTINE ARAMP
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/BB/K/B102/POS(50)/B99/VEL(50)/B98/ACCEL(50)/BC/ARR(50)
      COMMON/CAL/TIMER/RAM/AC1,DISTA1/B103/ACCELT(50,10)/OLA/KZ
      COMMON/E5/IA(50,5)/B501/TIME/E30/ALMR(50)/BDD/LA
      COMMON/BCC/TRR(5000)/G2/BR/OUS/VELR(50)/C45/W1,W2
C
      LA=LA+1
      IF(TIME.LT.TRR(LA)) GO TO 7
      K=K+1
      IF(K.LE.50) GO TO 11
```

```
      K=K-1
    7 LA=LA-1
      RETURN
   11 ARR(K)=TIME
      MK=K
      JL=4
      CALL APROP(MK,JL)
      CALL ALMC(MK,JL)
      CALL RVELO(VE)
      IF(K.LE.1) GO TO 20
      EL=(POS(K-1)-W2)/VE
      IF(EL.GE.DISTA1) GO TO 20
      CALL RRST(VE)
      GO TO 30
   20 ACCEL(K)=0.00
      DO 10 I1=1,2
   10 ACCELT(K,I1)=BR*(VELR(K)-VE)
      VEL(K)=VE
   30 POS(K)=W2
      DO 1 I1=2,5
    1 IA(K,I1)=0
      CALL HRAMP(LA)
      IF(TIME.LT.TIMER) RETURN
      WRITE(6,2) K,IA(K,1),ARR(K),ALMR(K),POS(K),VEL(K),VE,
     *ACCEL(K),ACCELT(K,1),ACCELT(K,2)
    2 FORMAT(30X,'RAMP VEH. IN SYSTEM'/1X,2I5,8F10.3)
      RETURN
      END
C*********************************************************************
      SUBROUTINE RVELO(VE)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/C20/RV,SR,EV,SV/BB/K/B400/IC,IV/E5/IA(50,5)
      COMMON/OUS/VELR(50)/C21/BX1,BX2,AX1
C
      CALL NORMAL(AIT)
      VE=SR*(AIT-6.00)+RV
      IF(VE.LT.BX2) VE=BX2
      IF(IA(K,1).EQ.IV) VE=0.90*VE
      IF(VE.GT.BX1) VE=BX1
      VELR(K)=VE+AX1
      RETURN
      END
C*********************************************************************
      SUBROUTINE RRST(VE)
C*********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/BB/K/B102/POS(50)/RAM/AC1,DISTA1/B99/VEL(50)
      COMMON/C45/W1,W2/E30/ALMR(50)/B98/ACCEL(50)
      COMMON/G2/BR/OUS/VELR(50)/B103/ACCELT(50,10)/OLA/KZ
C
      AO=POS(K-1)-W2
      TP=AC1/AO
      IF(VE.LT.VEL(K-1)) GO TO 12
      A=TP*(VEL(K-1)-VE)
      VEM=VE+A
      SA=VEM-VEL(K-1)
      IF(SA.LT.-1.00) GO TO 11
```

```
      VEL(K)=VEM
      GO TO 16
   11 VEL(K)=VEL(K-1)
      GO TO 16
   12 VEL(K)=VE
   16 ACCELT(K,KZ)=TP*(VEL(K-1)-VEL(K))
      AP=BR*(VELR(K)-VEL(K))
      IF(ACCELT(K,KZ).LE.AP) GO TO 14
      ACCELT(K,KZ)=AP
   14 IF(KZ.GT.1) GO TO 19
      ACCELT(K,KZ+1)=0.00
      GO TO 20
   19 ACCELT(K,KZ-1)=0.00
   20 ACCEL(K)=0.00
      RETURN
      END
C***********************************************************************
      SUBROUTINE HRAMP(LA)
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/C40/A1,G1,D1,K1,A2,G2,D2/BCC/TRR(5000)
C
      IF(K1.LT.1) GO TO 1
      XL=(G2-D2)/FLOAT(K1)
    1 M=LA+1
      IF(K1.LT.1) GO TO 2
      X=G05CAF(X)
      IF(X.LT.A2) GO TO 19
    2 X=G05CAF(X)
      HDD=-(G1-D1)*ALOG(X)+D1
      GO TO 300
   19 TR=1.00
      DO 6 I=1,K1
      X=G05CAF(X)
    6 TR=TR*X
      HDD=-XL*ALOG(TR)+D2
  300 TRR(M)=TRR(M-1)+HDD
      RETURN
      END
C***********************************************************************
      SUBROUTINE RESULT
C***********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      DIMENSION X(5000),A1(3),F1(3),C(3),V(3)
      COMMON/C33/TF(3),F2(3)/REJ/IRJ,ILJ/B400/IC,IV
      COMMON/E55/DELAY(5000)/E56/IT/IRA/IW,IG/XX/SUMD
      COMMON/DIO/IML,IFL,ICL/OUP/DELS(5000)/XAP/SPR(5000)
      COMMON/OPS/DELQ(3000)/BXX/FAP(3)/XEP/HRA(5000)
      COMMON/A33/FLOW(3,2),FLM(3,2),SP1(3),DTM(3),SPM1(3)
      COMMON/C50/IDV/G5/PATH(5000)/KAM/SPM(3000)
C
      WRITE(6,5)
      WRITE(6,99)
   99 FORMAT(33X,'RESULTS'/33X,'-------')
      WRITE(6,12)
   12 FORMAT(1H0,23X,'A.MOTORWAY TRAFFIC STREAM'/24X,'-----------------
     *-------'//29X,'A.1 FLOWS(VEH/H)'//
     *31X,'LANE 1',10X,'LANE 2',10X,'LANE 3')
```

```
      DO 205 I=1,3
  205 TF(I)=FLOW(I,IC)+FLOW(I,IV)-FLM(I,IC)-FLM(I,IV)
      DO 1 I=1,3
    1 A1(I)=(FAP(I)-TF(I))*100./FAP(I)
      WRITE(6,100) (FAP(I),I=1,3),(TF(I),I=1,3),(A1(I),I=1,3)
  100 FORMAT(10X,'UPSTREAM',10X,F6.0,10X,F6.0,10X,F6.0/10X,'DOWNSTREAM',
     *8X,F6.0,10X,F6.0,10X,F6.0/10X,'%IFFERENCE',8X,F6.3,10X,
     *F6.3,10X,F6.3)
      DO 184 I=1,3
      C(I)=FLOW(I,IC)-FLM(I,IC)
  184 V(I)=FLOW(I,IV)-FLM(I,IV)
      WRITE(6,104) (C(I),I=1,3),(V(I),I=1,3)
  104 FORMAT(1H0,25X,'A2.VEHICLE COMPOSITION'//10X,'CARS',14X,F6.0,
     *10X,F6.0,10X,F6.0/10X,'H.G.V.',12X,F6.0,10X,F6.0,10X,F6.0)
      DO 164 I=1,3
  164 F2(I)=FLM(I,IC)+FLM(I,IV)
      CALL COLLECT
      WRITE(6,101) (SP1(I),I=1,3)
  101 FORMAT(1H0,26X,'MEAN SPEED(KPH)'/26X,F10.3,8X,F10.3,8X,
     *F10.3)
      WRITE(6,105) (DTM(I),I=1,3)
  105 FORMAT(1H0,27X,'A.4 MEAN JOUR. TIMES'/26X,F10.3,8X,F10.3,8X,
     *F10.3)
      CALL STORE
      WRITE(6,103) (F2(I),I=1,3),(FLM(I,IC),I=1,3),(FLM(I,IV),
     *I=1,3)
  103 FORMAT(1H0,30X,'MERGING VOLUME'//28X,F6.0,10X,F6.0,
     *10X,F6.0/10X,'CARS',14X,F6.0,10X,F6.0,10X,F6.0/10X,'H.G.V.',
     *12X,F6.0,10X,F6.0,10X,F6.0)
      WRITE(6,21) (SPM1(I),I=1,3)
   21 FORMAT(1H0,29X,'MEAN SPEED(KPH)'//26X,F10.3,8X,F10.3,8X,F10.3)
      AJTIM=SUMD/(F2(1)+F2(2)+F2(3))
      WRITE(6,3002) AJTIM
 3002 FORMAT(28X,'AVER.JOURNEY TIME=',1X,F8.3)
      DO 116 I=1,3
  116 F1(I)=TF(I)+F2(I)
      WRITE(6,106) (F1(I),I=1,3)
  106 FORMAT(1H0,25X,'TOTAL FLOW DOWNSTREAM'//28X,F6.0,10X,F6.0,10X,
     *F10.0)
      WRITE(6,5)
      CALL MINFLOW
      WRITE(6,5)
      CALL INLANE
      WRITE(6,601)
  601 FORMAT(30X,'SPEED DISTR. ON MOTORWAY')
      N=INT(FAP(1))
      DO 602 I=1,N
  602 X(I)=3.6*SPM(I)
      A=5.
      CALL CALSTAT(X,N,A)
      WRITE(6,5)
      CALL PROFILE
      WRITE(6,5)
      WRITE(6,20)
   20 FORMAT(29X,'DELAY STATISTICS'/29X,'----------------')
      WRITE(6,210)
  210 FORMAT(32X,'TOTAL DELAY')
      N=IT
      DO 15 J=1,N
   15 X(J)=DELAY(J)
      A=0.50
```

```
      CALL CALSTAT(X,N,A)
      WRITE(6,220)
  220 FORMAT(25X,'DELAY FOR SINGLE DRIVER')
      N=IW
      DO 225 J=1,IW
  225 X(J)=DELS(J)
      A=0.50
      CALL CALSTAT(X,N,A)
      IF(IG.LT.3) GO TO 1030
      WRITE(6,227)
  227 FORMAT(27X,'DELAY FOR QUEING VEH.')
      N=IG
      DO 228 J=1,N
  228 X(J)=DELQ(J)
      A=0.50
      CALL CALSTAT(X,N,A)
 1030 MX=0
      DO 85 I=1,IT
      IF(DELAY(I).LE.0.) GO TO 85
      MX=MX+1
      X(MX)=DELAY(I)
   85 CONTINUE
      PR1=FLOAT(MX)/FLOAT(IT)
      PR=FLOAT(IDV)/FLOAT(IT)
      WRITE(6,303) IDV,PR,MX,PR1
  303 FORMAT(10X,'NO. OF VEH. STOPPED',1X,I7,5X,
     *'PROB. VEH STOP',1X,F10.3//10X,'NO. OF VEH DELAYED',
     *1X,I7,5X,'PROB. VEH DELD',1X,F10.3)
      IF(MX.LE.3) GO TO 999
      WRITE(6,817)
  817 FORMAT(29X,'DELAYED VEHICLES')
      N=MX
      A=0.50
      CALL CALSTAT(X,N,A)
  999 WRITE(6,5)
      WRITE(6,22)
   22 FORMAT(25X,'MERGING PATH STATISTICS')
      N=IT
      DO 75 J=1,N
   75 X(J)=PATH(J)
      A=10.
      CALL CALSTAT(X,N,A)
      WRITE(6,5)
      WRITE(6,700)
  700 FORMAT(25X,'SPEED DISTR. OF RAMP VEH.')
      N=IT
      DO 701 I=1,N
  701 X(I)=3.6*SPR(I)
      A=5.
      CALL CALSTAT(X,N,A)
      WRITE(6,702)
  702 FORMAT(30X,'RAMP HEADWAYS')
      N=IT
      DO 703 I=2,N
  703 X(I-1)=HRA(I)-HRA(I-1)
      N=N-1
      A=1.00
      CALL CALSTAT(X,N,A)
```

```
      WRITE(6,409) IML,IFL,ICL
  409 FORMAT(20X,'REL. SPEED CLASS',1X,3I10)
      WRITE(6,810) IRJ
  810 FORMAT(27X,'NO.OF REJECTED LAGS',1XI10)
      WRITE(6,824) ILJ
  824 FORMAT(27X,'NO.OF REJECTED LEAD TIMES',1X,I10)
      WRITE(6,5)
    5 FORMAT(1X,'--------------------------------------------------------
     *----------------------')
      RETURN
      END
C**********************************************************************
      SUBROUTINE COLLECT
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/A33/FLOW(3,2),FLM(3,2),SP1(3),DTM(3),SPM1(3)
      COMMON/C33/TF(3),F2(3)
C
      DO 1 I=1,3
    1 SP1(I)=(3.6*SP1(I))/(TF(I))
      DO 2 I=1,3
      IF(F2(I).LE.0.0) GO TO 2
      SPM1(I)=(3.6*SPM1(I))/F2(I)
    2 CONTINUE
      DO 4 I=1,3
    4 DTM(I)=DTM(I)/TF(I)
      RETURN
      END
C**********************************************************************
      SUBROUTINE INLANE
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      DIMENSION X(5000)
      COMMON/BXX/FAP(3)/BXA/TIN(3000)
C
      IM1=INT(FAP(1))-1
      DO 2 IN=1,IM1
    2 X(IN)=TIN(IN+1)-TIN(IN)
      WRITE(6,5)
    5 FORMAT(23X,'HEADWAY DISTRIBUTION-LANE 1')
      N=IM1
      A=1.
      CALL CALSTAT(X,N,A)
      RETURN
      END
C**********************************************************************
      SUBROUTINE MINFLOW
C*******************************************************************
      IMPLICIT REAL(A-H,O-W,Z),INTEGER(I-N,Y)
      DIMENSION F(5000),M(300),MR(300),MM(300),MOL(300),MF(300)
      COMMON/E56/IT/XEP/HRA(5000)/BXA/TIN(3000)
      COMMON/BXX/FAP(3)/LAP/HMA(5000)
C
      DO 10 I=1,IT
   10 F(I)=HRA(I)
      N=IT
      CALL COMP(Y,M,F,N)
      N1=Y
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```
      DO 40 I=1,N1
   40 MR(I)=M(I)
      DO 11 I=1,IT
   11 F(I)=HMA(I)
      N=IT
      CALL COMP(Y,M,F,N)
      N2=Y
      DO 50 I=1,N2
   50 MM(I)=M(I)
      IX=INT(FAP(1))
      DO 60 I=1,IX
   60 F(I)=TIN(I)
      N=IX
      CALL COMP(Y,M,F,N)
      N3=Y
      DO 70 I=1,N3
   70 MF(I)=M(I)
      N=MIN0(N1,N2,N3)
      DO 700 I=1.N
  700 MOL(I)=MM(I)+MF(I)
      WRITE(6,85)
   85 FORMAT(5X,'MINUTE',6X,'FLOW-ARR.',6X,'FLOW-MERGE',
     *6X,'FLOW-MOT',6X,'TOTAL FLOW')
      WRITE(6,86) (J,MR(J),MM(J),MF(J),MOL(J),J=1,N)
   86 FORMAT(2X,I8,4X,I8,6X,I8,6X,I8,8X,I8)
      DO 100 I=2,N
      MM(I)=MM(I-1)+MM(I)
      MR(I)=MR(I-1)+MR(I)
      MF(I)=MF(I-1)+MF(I)
      MOL(I)=MOL(I-1)+MOL(I)
  100 CONTINUE
      WRITE(6,85)
      WRITE(6,86) (J,MR(J),MM(J),MF(J),MOL(J),J=1,N)
      RETURN
      END
C**********************************************************************
      SUBROUTINE STORE
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/B19/P(3,3),SUP(3,3)/B16/R
      COMMON/B501/TIME/C11/FTIME/B405/STIME
      DIMENSION SUPA(3)
C
      IF(TIME.LE.FTIME) GO TO 9
      IF(TIME.GE.STIME) GO TO 10
      DO 2 I=1,3
      IF(I.EQ.3) GO TO 3
      SUP(I,I+1)=SUP(I,I+1)+P(I+1,I)-P(I,I+1)
    2 CONTINUE
    3 SUP(I,I-1)=SUP(I,I-1)+P(I-1,I)+P(I,I-1)
    9 DO 12 I=1,3
      DO 12 J=1,3
   12 P(I,J)=0.0
      RETURN
   10 DO 11 I=1,3
      IF(I.EQ.3) GO TO 121
      SUPA(I)=SUP(I,I+1)/R
   11 CONTINUE
```

```
  121 SUPA(I)=-(SUPA(I-1)+SUPA(I-2))
      WRITE(6,1) (SUPA(J),J=1,3)
    1 FORMAT(25X,'NO. OF NET LANE CHANGES'//28X,F6.3,10X,F6.3,10X,
     *F6.3)
      RETURN
      END
C**********************************************************************
      SUBROUTINE CALSTAT(X,N,A)
C**********************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      DIMENSION X(N),RFR(200),TFR(200),CR(200),PR(200),WT(5000)
      DIMENSION IFR(200)
      COMMON/C1/AD1,ADX,AD3,AD4
C
      CALL G01AAF(N,X,0,WT,XBAR,S2,S3,S4,XMIN,XMAX,WTSUM,IFAIL)
      CV=S2/XBAR
      NA=N/2
      NB=INT(FLOAT(N)/2.+0.50)
      IF(NA.EQ.NB) GO TO 3
      FMD=X(NB)
      GO TO 4
    3 FMD=(X(NA-1)+X(NA+1))/2.
    4 WRITE(6,5) N,XBAR,S2,XMIN,XMAX,CV,S3,S4,FMD
    5 FORMAT(5X,'SAMPLE=',1X,I10/5X,'MEAN=',1X,F10.3,3X,'S.DEV=',1X,
     *F10.3,3X,'MINIM='1X,F10.3,3X,'MAXM=',1X,F10.3/5X,'CVAR=',1X,
     *F10.3,3X,'SKENS=',1X,F10.3,3X,'KYRTS=',1X,F10.3,3X,'MEDV=',
     *1X,F10.3)
      IF(A.EQ.10.) GO TO 85
      D=XMAX-XMIN
      CR(1)=XMIN
      D1=D/A+1.
      K=INT(D1)+1
      IF(K.GT.200) RETURN
      DO 7 I=2,K
    7 CR(I)=CR(I-1)+A
      GO TO 140
   85 CR(1)=0.
      AK=ADX/A+2.
      K=INT(AK)
      DO 180 J=2,K
  180 CR(J)=CR(J-1)+A
  140 K1=K-1
      IF(K1.LE.1) RETURN
      DO 10 I=1,K1
   10 IFR(I)=0
      ISF=0
      DO 11 I=1,N
      DO 12 J=1,K1
      IF((X(I).GE.CR(J)) .AND. (X(I).LT.CR(J+1))) GO TO 13
      GO TO 12
   13 IFR(J)=IFR(J)+1
      GO TO 11
   12 CONTINUE
   11 CONTINUE
      DO 14 I=1,K1
      TFR(I)=(CR(I)+CR(I+1))/2.
   14 ISF=ISF+IFR(I)
      CMF=0.
      DO 17 I=1,K1
      RFR(I)=FLOAT(IFR(I))/N
   17 CMF=CMF+RFR(I)
```

```
      PR(1)=RFR(1)
      DO 19 I=2,K1
   19 PR(I)=PR(I-1)+RFR(I)
      WRITE(6,20)
      WRITE(6,21) (CR(I),CR(I+1),TFR(I),IFR(I),RFR(I),PR(I),I=1,K1)
      WRITE(6,22) ISF,CMF
   20 FORMAT(10X,'C L A S S',8X,'MID.POINT',8X,'FREQUENCY',4X,
     *'REL.FREQ.',3X,'PROBABILITY')
   21 FORMAT(1X,2F10.3,4X,F10.3,8X,I6,8X,F6.3,8X,F6.3)
   22 FORMAT(43X,I6,8X,F6.3)
      RETURN
      END
C*************************************************************************
      SUBROUTINE COMP(Y,M,F,N)
C***************************************************************
      IMPLICIT REAL(A-H,O-W,Z),INTEGER(I-N,Y)
      DIMENSION M(N),F(N)
      COMMON/C11/FTIME
C
      DO 2 J=1,300
    2 M(J)=0
      Y=0
    8 Y=Y+1
      R=FTIME+60.*FLOAT(Y)
      R1=FTIME+FLOAT(Y-1)*60.
      IF(R.GE.F(N)) GO TO 12
      DO 10 I=1,N
      IF(F(I).GT.R) GO TO 8
      IF(F(I).LE.R1) GO TO 10
      M(Y)=M(Y)+1
   10 CONTINUE
   12 Y=Y-1
      RETURN
      END
C*******************************************************************
      SUBROUTINE PROFILE
C*************************************************************************
      IMPLICIT REAL(A-H,O-Z),INTEGER(I-N)
      COMMON/BAL/SUM(9),SUMA(9)/AL2/KX(9),KY(9)
      COMMON/C45/W1,W2/C93/IR1/C46/DF
C
      WRITE(6,3)
    3 FORMAT(17X,'SLIP-ROAD VEHICLES:SPEED DISTANCE PROFILE')
      WRITE(6,4)
    4 FORMAT(25X,'CARS',22X,'H.G.V.'/17X,'DISTANCE-M',2X,'SPEED-KPH',1X,
     1'DISTANCE-M'2X,'SPEED-KPH')
      DO 1 J=1,IR1
      AJ=FLOAT(J)
      ADF=W2+(AJ*DF)
      SX=(SUM(J)/FLOAT(KX(J)))*3.6
      IF(KY(J).GT.0) GO TO 9
      SXA=0.00
      GO TO 10
    9 SXA=(SUMA(J)/FLOAT(KY(J)))*3.6
   10 WRITE(6,6) ADF,SX,ADF,SXA
    6 FORMAT(20X,F4.0,7X,F6.3,5X,F4.0,7X,F6.3)
    1 CONTINUE
      RETURN
      END
C************************************************************************
```