A NEURAL INTEGRATED SIMULATION APPROACH FOR MODELLING OF MIXED TRAFFIC FLOW

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CERTIFICATE

This is to certify that the thesis entitled **"A Neural Integrated Simulation Approach for Modelling of Mixed Traffic Flow"** is a bonafide record of the work done by Sri Matha V. L. R. Anjaneyulu under my supervision and guidance. The thesis is submitted to the University of Calicut in fulfilment of the requirements for the award of the degree of Doctor of Philosophy in Civil Engineering. The matter contained in this thesis has not been submitted to any other University or Institute for the award of any degree.

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ABSTRACT

Traffic flow modelling has been the subject matter of traffic engineering research for several decades. Many approaches have been suggested for modelling the traffic flow phenomena, which include empirical models at one end of the scale to rigorous micro simulation models at the other end. Keeping pace with the developments in modelling of traffic flow phenomena by the researchers in developed countries, many researchers in India have also developed simulation models for both unidirectional and bi-directional movements.

However, many such valuable efforts have not been successful in developing a comprehensive theory for traffic flow phenomena in mixed mode environment. The main reason for this inadequacy lies in lack of clear understanding of vehicular movements in mixed traffic situation and perhaps crude approximations made in describing these movements. Quite often, heroic recommendations were made based on partial and imprecise data. Due to such decisions, several conjectures were created in the minds of the researchers. One such conjecture that had arisen was the reported bell shaped relationship between speed and flow.

The objective of the research work being reported in this thesis was to minimise these uncertainties to a greater extent through repeated experimentation using simulation model so that certain amount of consistency could be achieved in the conclusions.

The literature review has revealed that among many of the approaches available, simulation approach is by far the best method for modelling the traffic flow

because of the parsimony permitted by it in the data input. Interestingly these simulation models have been developed based on several sub models, which may vary in their rigour – ordinary regression models at one end to stochastic behavioural models at the other end. For many of these sub-models, the causative structure are not very clear and hence, both probabilistic and deterministic models may not be adequate. In such cases, Artificial Neural Networks (ANN) provide a promising alternative. In this thesis, an attempt has been made to integrate the ANN and simulation models to utilise the best advantages of both.

The following improvements have been incorporated into the simulation model:

- Introduction of a warm-up zone approach for vehicle arrivals so that the vehicles on the test section could achieve both temporal and spatial stability.
- Incorporation of models that describe the vehicle movement in a more realistic way through the use of neural networks for various component models.
- Incorporation of acceleration and deceleration models derived based on field observations either by ANN or regression models.
- Improvement in the placement of vehicles in the test section by closer monitoring of linear and lateral placements that could arise in the mixed traffic situation.
- Development of a graphic simulator to generate the images of vehicles on the computer screen following the intended logic so that inconsistent and illogical features could be traced and structure revised accordingly.

A neural integrated simulation model was built by incorporating all the above factors and was concurrently validated using the field data. This field validated simulation model was then used for conducting several experiments in mixed mode environment in a unidirectional traffic flow situation. The major conclusions resulting from the experiments carried out on simulation model are as follows:

i. There is no much justification in using models other than single regime linear model for modelling speed-density relationship in mixed traffic conditions. Consequently, the speed-flow and the flow-density models could be adequately described by parabolic relationships.

ii. The Passenger Car Unit (PCU) values at capacity flow conditions for two-wheeler, auto-rickshaw and bus have been worked out to be 0.65, 0.8 and 1.49 respectively in this study.

iii. The capacity values for two-lane unidirectional flow for single vehicle streams of two-wheelers, auto-rickshaws and buses have been found to be 4335, 3334 and 1890 vph respectively, in this study.

iv. A regression equation developed in this study for prediction of capacity flow in mixed traffic based on 51 mixed vehicle combinations is able to predict the capacity of the mixed traffic very close to the reality. However, it is to be used only in situations of mixed traffic and not for streams with only single type of vehicle.

v. Experiments conducted by varying the percentage of auto-rickshaws in the traffic stream while keeping the two-wheeler and car percentages constant, have indicated that the capacity flow Vs percentage auto-rickshaw is linear, but those linear graphs for varying percentages of cars may intersect with one another, suggesting that there may be more than one mix combination for which the capacity flows may be identical. Hence, vehicles do interact to modify the capacity flow.

vi. Capacity flow in traffic stream is highly correlated with bus and two-wheeler percentage, but in opposite sense. This means that larger the percentage of buses the lesser the capacity flow. However, larger the percentage of two-wheelers greater the capacity flow. Similar inference, as in above, is possible for jam density also. But the contribution of both the car and the auto-rickshaw are found to be statistically significant in jam density calculations.

vii. Equivalent passenger car units calculated in this study for different types of vehicles at various levels of service have indicated that the PCU values for two-wheelers and auto-rickshaws vary quite considerably over different levels of service. On the other hand, change in equivalent PCU values for bus was found to be not very significant

The major contributions of the study are:

i. Development of a Neural Integrated Simulation model for experimentation related to vehicular movements in mixed traffic environment.

ii. Further refinement of basic simulation models by incorporation of linear and lateral placement of vehicles, acceleration/deceleration characteristics of vehicles, vehicle manoeuvring logic, etc, through neural network approach.

iii. Introduction of warm up zone approach in simulation modelling of traffic flow.

iv. Simulation of vast possible scenarios for a given set of input situation by randomising the seeds for generation of random number sequences.

v. Graphic display of simulated movements of vehicles and introduction of appropriate corrections to the flow logic when found necessary.

vi. Determination of capacity and jam density values for single and multiple vehicle streams by the development of a simple linear regression equation.

vii. Demonstration of the suitability of linear speed-density and parabolic speed-flow and flow-density models even in mixed traffic situation.

viii. Study of sensitivity of capacity values and jam densities to composition of the vehicles in the stream.

ix. Development of equivalent passenger car units and service volumes at different levels of service and study of variable PCU values.

One of the limitations of the study is that the simulation model is useful only for one directional uninterrupted traffic movements under ideal geometric conditions. Separate models have to be developed for bi-directional movements and for study of influence of the geometrical factors. From the output of the simulation experiments, it has been observed that at the free flow and at the capacity flow conditions the scatter of points are large. In order to quantify this variability a quantity called q/ku was found out and these were plotted against density of traffic stream. It has been noticed that this ratio is not equal to unity and it is suspected that this ratio either may be a constant or a function of density itself. That ratio appears to be a proxy similar to porosity or void ratio in soil structure. Further research can perhaps bring about the nature of this q/ku relationship with density.

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CHAPTER - 1

INTRODUCTION

1.0 GENERAL

The study of characteristics of traffic stream such as speed, density, flow rate or traffic volume and their inter-relationships has been the subject matter of basic research in traffic engineering during the past several decades. While the earlier research work concentrated mostly on modelling of those inter-relationships in homogeneous traffic conditions, during the past three decades, considerable interest has been generated in modelling of traffic stream behaviour in mixed mode environment. Of all the models that have been developed, the relationship between the speed and the density of traffic stream has caught the attention of researchers because of its simplicity. Models have been developed for explaining the relationship between the speed and the density, the form of which has varied from simple linear function to multi-regime models. But in many such modelling efforts, lack of appropriate data over the whole domain of interest has been the bane of the researchers, particularly those modelling the mixed traffic behaviour. Many times the researchers have been compelled to take certain decisions and conclude the research work based on incomplete understanding of phenomena being modelled. One such relationship, which remains as a conjecture, is the suspicion that the speed-flow relationship in a mixed traffic flow situation may be bell shaped, rather than parabolic (Nagaraj *et. al.* 1990).

It is this kind of conjectures and uncertainties, which has been the motivating factor for taking up this research work in traffic stream modelling under mixed mode environment.

1.1 STATE OF THE ART

There are mainly three approaches for traffic flow modelling, viz., the empirical, the analogy and the simulation. In empirical approach, stream characteristics are expressed in terms of inter-relationships among the variables like the speed, the density and the flow by curve fitting to the observed data. However, there is a limitation to this empirical approach because of the wide variations in the influencing factors, which result in many traffic flow combinations. Quite often, the researchers will be forced to make many simplifying assumptions in the development of models. The models are generally deterministic in nature.

Analogy methods are also macroscopic in nature and in this approach, the stream characteristics are hypothesized to follow certain physical phenomenon. One such approach is the well-known fluid flow analogy. This approach appears to be appropriate only in those flow conditions which bare close resemblance to fluid flow. Like in empirical approach, it is not possible to use this approach also in a comprehensive sense due to interaction among several influencing variables.

Simulation models of traffic stream are based on microscopic approach in which individual vehicles behaviour is considered. It is possible to include many influencing factors and conduct experiments under controlled conditions. Further, they can be used to simulate a wide range of conditions with relative ease and without the need to collect the costly data. Many researchers in India (Marwah 1976, Ramanayya 1980, Badarinath 1993, Kuncheria 1995, etc) have adopted this approach to study mixed traffic flow at urban and rural mid blocks.

A typical simulation model consists of a number of component blocks, each representing a particular sub-system. Generally, many of these component models are developed based on empirical approach using the pre-processed data. However, many of these sub-models could be built to reflect stochastic and dynamic changes in the influencing variables through Artificial Neural Networks (ANN). In order to calibrate these ANN models it is necessary to train the network by the use of real time data reflecting the changes due to several influencing variables.

Gathering data for such a host of situations would be difficult for modelling traffic flow under all possible traffic conditions, different geometrical conditions, environmental conditions, etc, thus limiting the use of ANN approach. Still that approach can be made use of modelling sub-tasks for the main simulation program. Thus, a marriage between simulation and ANN approach would be a useful modelling effort for a study as complex as that of understanding the dynamics of mixed traffic.

1.2 REFINEMENTS NEEDED IN SIMULATION APPROACH

The following are some of the refinements needed in simulation approach:

i) The arrival times of vehicles at the entry to simulated stretch are generally estimated by sampling from headway models. In mixed traffic, the vehicles often move side by side due to unrestricted mixing. The distribution of headways depends upon the road width, the traffic composition, the flow level, etc., and hence, it becomes very difficult to identify a suitable headway model. Literature survey has often revealed that different researchers in the past have used different headway models even for nearly identical flow levels. This only indicates that the approach based on headway distributions may not be suitable for simulation of vehicle arrivals in mixed traffic. The warm-up zone approach (McLean 1989) is another approach, which has been suggested for controlling the vehicle arrival. This does not require the use of headway models and hence, appears to be a better alternative for generation of vehicle arrivals in mixed traffic simulation.

ii) Another aspect of simulation which needs attention of any researcher is the number of vehicles being simulated for a given road width. While, some of the researchers could simulate flows up to 1000 vehicles only, some could simulate even traffic flow of 12,000 vph for a road width of 7.0m. This excessively high value of flow could have been possible only when the observations might have been taken immediately after the entry point. The results in those cases are likely to be erroneous particularly when the observations are recorded immediately after starting of simulation. The traffic flow must be allowed to stabilise over time and space so that the vehicles get adjusted to the rhythm of movement. It is essential, therefore, to allow the system to stabilise over time and space before the observations are taken.

iii) The acceleration/deceleration values adopted by many researchers in simulation studies are either constant or based on limited field observations. In reality, a driver accelerates or decelerates at a rate, which he presumes to be safe and causes minimum discomfort to the passengers and based upon the vehicle characteristics. So, the acceleration/ deceleration characteristics vary from driver to driver and needs a better presentation in simulation model.

iv) In simulation, the vehicles are generated and moved through the simulated system as per the logic of the model. In the absence of the facilities for observing the movement of simulated vehicles, many inconsistencies such as two vehicles occupying the same position at the same time, etc., will go unnoticed. A visual display of the simulated vehicles as they pass through the system provides an opportunity to check and correct for any inconsistencies in the flow logic, which will not be possible otherwise.

v) The vehicle manoeuvring logic, the core of the traffic flow simulation model, is generally rule based. This also needs to be replaced by a better model for the reasons already discussed.

1.3 VISION STATEMENT

The ultimate goal of this research is to develop a comprehensive simulation model which is capable of being used for the prediction of stream characteristics such as speed, density and flow under all possible realistic traffic mix compositions, which would minimise the uncertainties and conjectures existing in the present state of understanding the traffic system. Such a model should be versatile and capable of initiating fundamental research in stream characteristics.

1.4 OBJECTIVES AND SCOPE OF THE WORK

The main objective of the work is to conduct traffic flow studies in mixed environment and characterise the traffic stream with the following specific objectives:

i) To identify the methodologies/approaches for describing various component processes of the traffic flow.

ii) To collect and process the required data and to derive models for component blocks.

iii) To develop the simulation model by integrating these component models and to validate the model.

iv) To conduct experiments on the validated simulation model and understand the characteristics of mixed traffic flow.

v) To derive equivalency factors for expressing the influence of various types of vehicles in terms of a representative vehicle in the traffic stream.

Keeping in view the time and other resources available and the complexity of the mixed traffic flow, the scope of the work is limited to study of unidirectional traffic flow at urban mid blocks in mixed mode environment. The idea is to specifically study the effect of composition of vehicles on the traffic stream parameters and to derive equivalency factors.

1.5 STRUCTURE OF THE THESIS REPORT

Including, 'Introduction' there are nine chapters in this thesis. Chapter - 2 on 'Traffic Flow Modelling Approaches', while reviewing the various approaches that have been followed for modelling the traffic flow, identifies the need for use of simulation approach in order to reduce the ordeal in data collection. In this chapter a subtle refinement is also suggested, like the possibility of introduction of ANN in development of sub models.

Chapter – 3 on 'Artificial Neural Networks', while presenting the various possible applications of ANN in the studies related to traffic engineering, describes nuances of ANN like, learning paradigm, network structure, etc. This chapter has concluded with a recommendation for the use of a multi-layer feed forward neural network, trained, using back propagation algorithm.

The development of component models is the subject coverage of Chapter – 4. In this chapter, several types of models have been developed starting from a simple regression model to the highly sophisticated stochastic models to describe the various components of the simulation model.

Apart from the above models, certain traffic situations are governed by the way the drivers react to the behaviour of the frontal vehicles, also called as leaders. These situations are aptly described by car-following theory. Chapter–5 describes the acceleration/deceleration characteristics of constrained vehicles, which are modelled using the above theory.

Development of acceleration and deceleration models in the free flow condition is the subject matter of Chapter – 6. The models in this have generally two components. While the first component can be called deterministic, the second, which reflects the driver behaviour, can appropriately be described by stochastic modelling.

Chapter – 7 on 'Development of Simulation Model' integrates all the sub models developed in previous chapters and provides a complete description of all possible movements achieved in simulation modelling.

All experiments carried out on the simulation model are described in Chapter - 8 on, 'Experiments Using Simulation Model'. Determination of capacity flow, jam density, flow levels corresponding to different levels of service and equivalent factors for different types of vehicles is the subject matter of this chapter.

The summary and conclusions of the study and indication of the scope for further study based on limitations of the present study are the subject matter of Chapter–9. Many interesting extensions to the present work to refine the basic understanding of the traffic flow phenomena in mixed mode environment are given in this chapter.

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CHAPTER - 2

TRAFFIC FLOW MODELLING APPROACHES

2.0 INTRODUCTION

The efficient movement of people and goods through physical road and street system is an intriguing problem. Modelling of traffic flow gives insight into the traffic behaviour and enables better roads to be built. Study of traffic systems has been a lively subject of research and debate for traffic engineers over the last five decades. This has resulted in a variety of models describing different aspects of traffic flow operations. Many researchers in India have attempted to characterise the traffic flow in mixed mode environment. An overview of traffic flow modelling research in general and of simulation studies carried out on mixed traffic flow is presented in this chapter.

2.1 TRAFFIC FLOW CHARACTERISTICS

Researchers have been attempting to understand the behaviour of traffic systems through development of traffic flow models relating the three fundamental parameters of traffic flow, viz., the speed, the volume and the density. The relationships between these variables vary depending on the characteristics of the traffic stream, which in turn are influenced by stream composition and the factors related to the vehicles, the roadway and the driver. The vehicular factors include the physical dimensions and the operating characteristics. The mixed traffic on our urban roads consists of vehicles belonging to different classes, whose physical and operating characteristics differ from one other considerably, resulting in very many flow conditions even at one location.

Researchers have broadly classified the traffic flow conditions as free flow, congested flow and transitional flow. In free flow condition, i.e. at very low densities, the vehicles will be unimpeded by other vehicles and will be moving freely. At high densities, vehicles will be influenced by the presence of other vehicles and will be constrained. While the interaction between vehicles is likely to be at minimum under free flow conditions, it will peak under congested flow conditions.

The concept of Level-Of-Service has been introduced in 1965 Highway Capacity Manual, dividing the operating conditions of traffic stream into six regions. While, Level-of-Service A corresponds to free flow condition, Levels-of-Service B & C represent the stable flow conditions. On the other hand, Levels-of-Service D & E are presented as unstable flow conditions. Level-of-Service F corresponds to forced flow conditions. The primary criterion for demarcating different Levels-of-Service has been the lane density, the other criteria used being, volume/capacity ratio and the operating speed. This indicates that interaction between vehicles vary depending on flow level.

Similarly, variations in operating characteristics can also be observed in mixed traffic. Indian Roads Congress (1990) has tentatively recommended six levels of service, more or less similar to that recommended in HCM (1965). However, the traffic, the roadway, the vehicular and the driver characteristics in India vary considerably from those prevailing in homogeneous and lane disciplined traffic conditions. In spite of the fact that many researchers have studied mixed traffic stream behaviour, the research carried out so far in India has not been successful to provide solutions to many practical problems. This can be due to the limitations of the approaches adopted by them. Hence, it becomes necessary to review the suitability of various methodologies available for modelling of mixed traffic flow and select a suitable methodology. A brief discussion of various methodologies used for study of traffic flow is given below

2.2 CATEGORISATION OF TRAFFIC FLOW MODELS

Research on the subject of traffic flow modelling started some fifty years ago, when Lighthill and Whitham (1955) presented a model based on the analogy of vehicles in traffic flow and particles in a fluid. Since then, mathematical description of traffic flow has been a lively subject of research and debate for traffic engineers. Traffic flow modelling is increasingly attracting the interest of scientists. This is partly due to the great economical and social relevance of the traffic flow problem and partly to the interesting features, which are emerging from its study (Campari and Levi, 2002). This has resulted in a broad range of models describing different aspects of traffic flow operations.

The achievements of traffic modelling research during the last five decades can be classified according to the following:

- Scale of the independent variables (continuous, discrete, semi-discrete);
- Level of detail (sub microscopic, microscopic, mesoscopic, macroscopic);
- Representation of the processes (deterministic, stochastic);
- Operationalisation (analytical, simulation);
- Scale of application (networks, stretches, links, and intersections).

2.3 TRAFFIC STREAM PARAMETERS

The fundamental traffic stream variables of interest are the flow, the speed and the density. Flow (q) is defined as the number of vehicles passing a specific point in a given period of time and is expressed as an hourly flow rate (vph). The unique flow parameter is the maximum flow or capacity flow (q_m). Speed (u) is defined as the average rate of motion of traffic stream expressed in kilometres per hour (kmph). From theoretical considerations, space-mean speed should be employed and the two unique parameters of speed are the free-flow speed (u_f) and the speed at capacity flow (optimum speed, u_o). Density (k) is defined as the number of vehicles occupying a section of a roadway and is expressed as vehicles per kilometre (vpkm). The two unique density parameters are the jam density (k_j) and the density at capacity flow (optimum density, k_o). The inter-relationships among these three parameters are known as the traffic stream models and typical relationships are presented in Fig 2.1.

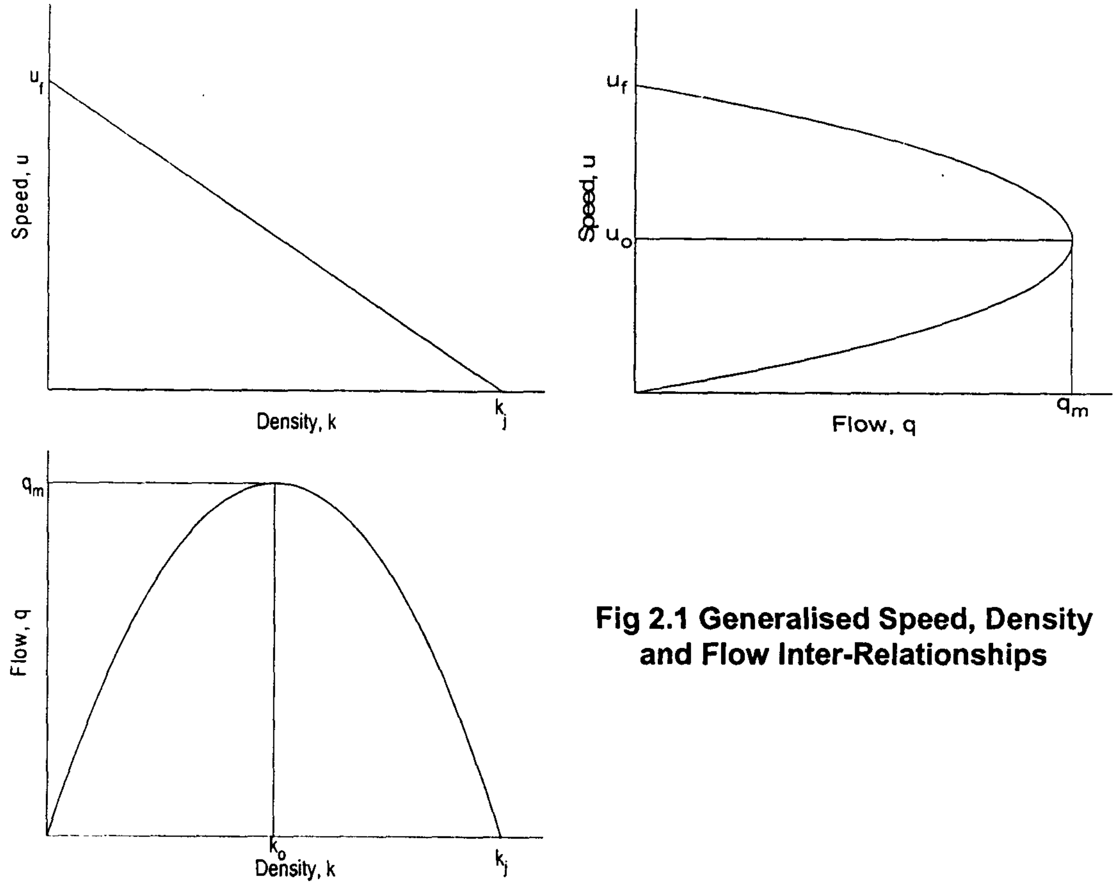

Fig 2.1 Generalised Speed, Density and Flow Inter-Relationships

A set of two independent equations is required to fully define the traffic flow in terms of these three basic variables. One equation is provided by the dimensional analysis relating the three variables:

$$q = k \cdot u \qquad \text{Eqn. ......2.1}$$

The other equation has to be obtained by empirical or other means. There are mainly three approaches for modelling of traffic flow, viz., the Deterministic, the Probabilistic and the Simulation approaches.

2.4 DETERMINISTIC APPROACH

In this approach, the relationships between the various quantifiable characteristics of the system under study are derived by empirical or analogy methods. The deterministic approach is the most appropriate for modelling the aggregate or macroscopic characteristics of traffic flow due to the simplicity and ease of application of models. If the random variations of the system are very small, then the system can be treated as deterministic and this approach can be applied to derive the models related to the system. Also, if suitable data can be obtained, this approach can be easily applied.

2.4.1 Empirical Models

The models of Greenshields (1934) (Linear model), Greenberg (1959) (Logarithmic model), Underwood (1961) (Exponential model) and Drew (1968) (generalised model) are some of the empirical models which relate speed and density. Haight (1963) proposed the following five boundary conditions to be satisfied by a traffic stream model:

i) $q = 0$ for $k = 0$

ii) $q = 0$ for $k = k_j$

iii) $u = 0$ for $k = k_j$

iv) $u = u_f$ for $k = 0$

v) $\lim_{k \to 0} \frac{du}{dk} = 0$

Except the May and Keller's (1967) model, all the other relationships fail to satisfy one or more conditions. It means that a single model may hardly be able to describe the traffic flow over complete range of flow conditions.

Many researchers (Edie (1961), Underwood (1961), Ceder, (1976), Easa (1983)) reported the discontinuities, as shown in Fig 2.2, in freeway speed-flow-density data. They attributed this to the different mechanisms that affect the flow at different density levels and suggested the use of multi regime models to properly describe traffic behaviour. Hall *et. al.* (1986) concluded that cause of gaps in data could be backing of queue into the location where the flow is less than

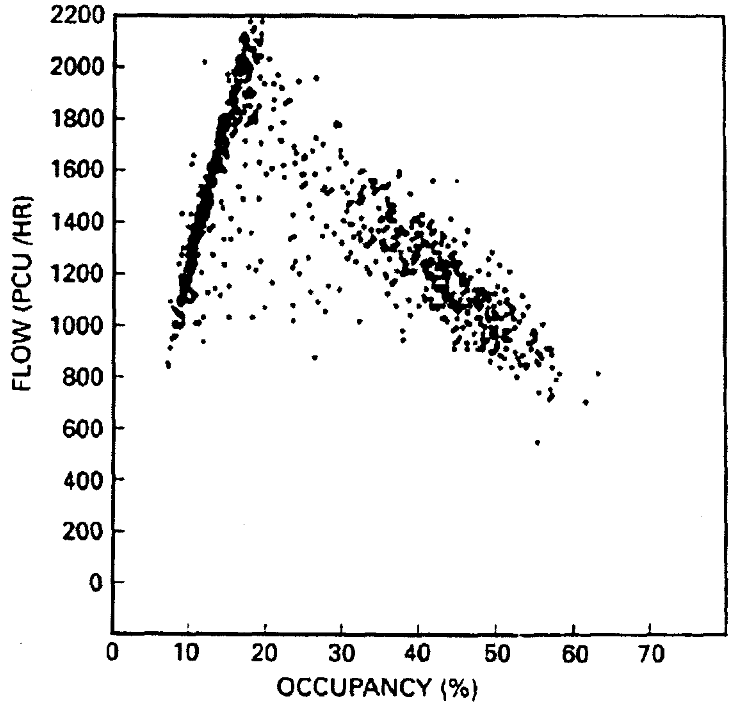

Fig 2.2 Flow – Occupancy Data

capacity. Hall and Hall (1990) observed that the conventional parabolic relationship exists between speed and flow for a section upstream of a capacity restriction, whereas, at downstream the speed remains constant until a queue is formed upstream and then a vertical drop to lower speeds occurs at nearly same flow rate.

Chin and May (1994) observed that serious misinterpretations of speed-flow relationships could result if unsuitable data reduction and analytical procedures are employed and if the influence of study location on the nature of data is disregarded.

Modelling of mixed traffic by empirical approach had been the topic of study of many researchers [Mony *et. al.* (1978), Gupta *et. al.* (1979), Central Road Research Institute (1982), Thuladhar (1987), Sarna *et. al* (1988), Nagaraj *et. al.* (1990), Kadiyali *et. al* (1991), Kumar (1994) and Sahoo *et. al.*(1996)] in India. A review of those works revealed that there have been no unified and generally acceptable relationships between the traffic stream parameters. This can be due to wide variations in traffic mix, vehicular and roadway characteristics that result in very many combinations of flow conditions. Development of a generalised flow model encompassing all the possible flow conditions may not be rational. Thus it becomes essential to divide the flow conditions into homogeneous regions and develop models for each of the regions separately.

As the flow conditions vary from location to location and over time, development of traffic flow models by empirical approach requires huge data, the collection of which may be uneconomical in our conditions. Further, it may be hardly possible to observe and record the data for all the ranges of flow conditions. Also, these models, being deterministic in nature, may not reflect adequately the individual driver-vehicle unit. Hence, the models developed based on empirical approach are likely to be of limited use for development of standards. Further, the availability of sufficiently long uniform sections, such that the influence of adjacent sections is minimum, in urban areas is quite rare.

2.4.2 Car-Following Models

Car-following models are microscopic models and are based on the dynamic behaviour of a following vehicle. Car-following models determine the acceleration or deceleration rate of the following vehicle in a given time interval based on the actions of the lead vehicles. Once the acceleration or deceleration rate of the following vehicle is determined, equations of motion are used to compute the speed and the position of the following vehicle for any given time interval. Many different car-following models have been proposed to describe driver behaviour in a traffic stream. Brackstone and McDonald (2000) provides a historical review of car-following behaviour modelling. Driver behaviour literature suggests four different types of car-following models. These four types are:

- Stimulus – response car-following models;
- Safety-distance or behavioural car-following models;
- Psycho-physical car-following models; and
- Fuzzy logic-based car-following models.

Many researchers, Gazis *et. al.* (1961) May and Keller (1967), etc., derived the macroscopic stream models from car-following models for steady state conditions. Even though, a microscopic treatment of traffic stream is possible by this approach, the car-following models are having certain limitations. In reality, the drivers get clues not only from their respective lead vehicles but also from other sources. So the response of the following driver will be much more complex than what is assumed in these models. Further, the theory applies only when the vehicles follow the lead vehicles and hence, can be applied to only high-density flow conditions. Hence, the car-following models fail to describe the traffic stream over the full range of flow conditions. In mixed traffic, vehicles hardly observe lane discipline and often move side by side in the same lane. So the application of car-following models for mixed traffic appears to be under a cloud. Brackstone et al (2002) concluded that much work remains to be done before car following can be even partially understood.

2.4.3 Fluid Flow Analogy Approach

Another approach of obtaining traffic stream models is fluid flow analogy approach. In this approach, the traffic flow is considered to be analogous to fluid flow. The continuity equation, which is an expression of conservation of matter, is the basis for the relationship between speed and density. Lighthill and Whitham (1955), Drew (1968) and Pipes (1968), used this approach to derive the macroscopic models of traffic flow. Some of the other notable works in this direction are by Prigogine and Herman (1971) Dendrinos (1978), Hall (1987), Daganzo (1995), Angelis (1999), Schadschneider (2000), Bellomo et al (2002), Bonzani and Mussone (2002), Cho and Lo (2002a & b), Lo (2002), Kerner (2002), Marasco (2002) and Waldeer (2002).

Many of these approaches to model traffic flow have not developed beyond the stage of formulation, may be due to the complexity of the formulation and the variety of the parameters required to describe the three processes. Despite the limitations, these approaches have provided valuable insight into various aspects of traffic behaviour and flow modelling (Gartner et al 1997).

Some of the dissimilarities between vehicular flow and the flow of molecules in a fluid or a gas are anisotropy, unaffected slow-vehicles, driver's personality, finite space requirements, consideration of the velocity variance and finite braking times and reaction times.

These models have the advantage that they enable description behaviours of individual vehicles, without the need to describe their individual time-space behaviour. However, the resulting equations have been criticised for having too many parameters and high dimensionality, hampering calibration and their real-time applicability. But, they are ideally suited as a foundation to derive macroscopic flow models. Moreover, using particle discretisation approaches, these models can be microscopically discretised.

Macroscopic models are suited for large scale, network-wide applications, where macroscopic characteristics of the flow are of prime interest. Generally, calibration of macroscopic models is relatively simple (compared to microscopic and mesoscopic models). However, macroscopic models are generally too coarse to correctly describe microscopic details and impacts, for instance caused by changes in roadway geometry. Due to the availability of closed analytical solutions, these are suitable for application in model-based estimation, prediction, and control of traffic flow.

2.5 PROBABILISTIC APPROACH

Stochastic models are built up from the causal mechanisms and their merit lies in providing an understanding of the flow process as related to fundamental characteristics of the drivers, the vehicles and the roadway. The variable nature of these characteristics gives rise to the stochastic nature of the various processes occurring in a traffic stream, viz., the speeds, the headways, the overtaking and the platooning.

The commonly used theory for traffic flow analysis is the queuing theory. In a traffic stream, the vehicles travel at different speeds, so the fast vehicles catch up the slow vehicles, resulting in formation of platoons. These platoons can be considered as moving queues. But there are certain differences between traffic operations and standard queuing system. The traffic still continues to flow even when the traffic intensity (ratio of arrival rate to service rate) is greater than unity. When one platoon catches another platoon, queues get amalgamated. Also, an overtaking may occur from any position in platoon and the specification of queue discipline becomes difficult. Further, the overtaking process is affected by driver gap acceptance behaviour.

Some of the probabilistic models of traffic flow are the *Fast-vehicle Models* (Tanner, 1958), *Two-speed model* (Gordon and Newell (1964), Daganzo (1975)), *Integral Equation Model* (Erlander(1968), Jacobs (1974)), *Equilibrium Platoon Models* (Miller(1965), Gipps(1976) and Kallberg(1980)) and *No overtaking Models.* All these models included one or at the maximum two random variables in the formulation, whereas, traffic flow is a result of a number of random processes which interact in various ways. Attempts to include more random variables in the models have generally failed due to mathematical intractability. So a general model of traffic flow applicable to all situations is not yet available. However, the probabilistic approach provides an understanding of the causal mechanisms. So probabilistic models can be used for formulation of the models and the statistical methods can be applied to estimate the parameter values.

2.6 SIMULATION APPROACH

Simulation models are based on the (sub)microscopic approach in which the behaviour of individual components of the system is considered. The computer simulation of the system is made possible by integrating several models that describe the operating characteristics of the components. In simulation study, it is possible to include any number of factors, which may not be possible in mathematical models. A comprehensive simulation can be regarded as the equivalent of an idealised laboratory experiment. Simulation enables the experimenter to control the variables as in conjunction with observing real traffic. So it is possible to study the effect of independent variables on dependent variables. Another advantage of simulation modelling is the ability to simulate a wide range of conditions with relative ease and without the expenses necessary to obtain the field data. It is also possible to create combinations of road and traffic conditions that are hardly observed but which are felt necessary to be simulated by researchers. Because of these advantages, many researchers have adopted computer simulation for modelling of traffic flow. Traffic flow simulation models can be broadly classified as microscopic simulation models, sub-microscopic simulation models and cellular automation models.

Gerlough (1956) was the first to use simulation technique for the study of traffic flow. Cassel and Janoff (1968) developed a model known as Franklin Institute Research Laboratories (FIRL) model to simulate traffic flow on a two-lane two-way roadway. Some of the other researchers to work on simulation modelling of traffic flow are Heimbach *et. al.*(1973), Gynnerstedt (1977), John and Kobett (1978), Robinson (1980), Wu and Heimbach (1981). McLean (1989) discussed in detail many of these simulation models.

2.6.1 Microscopic simulation models

The availability of fast computers has resulted in an increasing interest in complex micro-simulation models. These models distinguish and trace single cars and their drivers. Driver's behaviour is generally described by a large set of if-then rules (production-rule systems). From driver behaviour and vehicle characteristics, position, speed and acceleration of each vehicle are calculated for each time step. A large number of microscopic simulation models have been developed.

2.6.2 Sub microscopic simulation models

In addition to describing the time-space behaviour of the individual entities in the traffic system, sub microscopic simulation models describe the functioning of specific parts and processes of vehicles and driving tasks. For instance, a sub microscopic simulation model describes the way in which a driver applies the brakes, considering among other things the driver's reaction time, the time needed to apply the brake, etc. These sub microscopic simulation models are highly suited to model the impacts of driver support system on the vehicle dynamics and driving behaviour.

2.6.3 Cellular automaton models

A more recent addition to the development of microscopic traffic flow theory are the so-called *Cellular Automaton* (CA) or *particle hopping* models. CA-models describe the traffic system as a lattice of cells of equal size. A CA-model describes in a discrete way the movements of vehicles from cell to cell (Nagel (1996,1998), Campari and Levi (2000), Schreckenberg et al (2001), Gundaliya et al (2004)). The size of the cells are chosen such that a vehicle driving with a velocity equal to one moves to the next downstream cell during one time step.

Using this *minimal set* of driving rules, and the ability to apply parallel computing, the CA-model is very fast, and can consequently be used both to simulate traffic operations on large-scale motorway networks, as well as for traffic assignment and traffic forecasting purposes. CA-models aim to combine the advantages of complex micro-simulation models, while, remaining computationally efficient. However, the car-following rules of both the space-oriented and time-oriented CA-models lack intuitive appeal and their exact mechanisms are not easily interpretable from the driving-task perspective. Moreover, they are too crude to describe and study microscopic details of traffic flow (e.g. overtaking and merging) sufficiently accurate from a single driver's perspective. The dimensions of vehicles in mixed traffic vary widely resulting in two vehicles moving side by side in the same lane. The representation of vehicles as cells is a big question to be answered before this technique can be used for study of mixed traffic.

A large number of computer models have been developed for homogeneous traffic conditions in western countries. Algers *et al.* (1997) identified 58 microscopic simulation models and analysed 32 models, given Table 2.1, as part of the 'Simulation Modelling Applied to Road Transport European Scheme Tests (SMARTEST)' project. They analysed the models based on Scale of application, Objects and phenomena modelled, Indicators, Interface, Control strategies and algorithms, Validation and limitations. They concluded that imperfect simulation of human behaviour is still an open-ended research problem.

Table 2.1 List of Microscopic Simulation Models

CASIMIR	DRACULA	HUTSIM	MICSTRAN
NEMIS	NETSIM	PADSIM	SIGSIM
SIMNET	SITRA-B+	SITRAS	THOREAU
AUTOBAHN	FREEVU	FRESIM	MIXIC
SISTM	AIMSUN2	CORSIM	FLEXSYT II
INTEGRATION	MELROSE	MICROSIM	MITSIM
PARAMICS	PLANSIM-T	TRANSIMS	VISSIM
ANATOLL	PHAROS	SHIVA	SIMDAC

2.7 SIMULATION STUDIES IN MIXED TRAFFIC

The first work in simulation study of mixed traffic in India is that of Marwah (1976). He developed a simulation model to study the behaviour of mixed traffic on two-lane two-way roads. The minimum spacing between the vehicles was assumed to be deterministic. The overtaking logic used in his study permits overtaking of bunch of any number of vehicles, which may not be feasible in real life situation. The model could simulate only two or three categories of vehicles at a time.

Another important work is that of Ramanayya (1980), wherein he developed models to depict the operations on single lane one-way, two-lane one-way and two-lane two-way roads. He suggested only three Levels of Service for mixed traffic conditions. A new conversion unit called Equivalent Design Vehicle Unit (EDVU) to convert the different classes of vehicles including passenger cars to a standard or design vehicle (passenger car of developed countries) was suggested in his study. The details of the model are discussed in Table 2.2.

Palaniswami (1983) developed a simulation model by modifying Sweedish VTI model to accommodate slow moving vehicles and to assess the effect of congestion on road user costs. He used the model to study the mixed traffic behaviour on single lane, intermediate lane and two lane roads in rural areas. The models were calibrated by comparing the speed distributions. The event based scanning technique was used to scan and update the system. The simulation model was coded in SIMULA.

Bandyopadhyay and Marwah (1986) developed a simulation model for traffic flow on a city road. The simulated road system consisted of four lanes, with the two central lanes meant for trams, intersections and transit vehicle STOPs. Five types of vehicles including trams were considered for study. The vehicle manoeuvring logic included three broad categories of flow status of vehicles and is based on lane concept. The scanning procedure adopted was a combination of time scanning and event scanning techniques.

Chalapathi (1987) developed a simulation model for multi lane unidirectional traffic on rural roads. His model is based on Sweedish VTI model, which was earlier modified by Palaniswamy (1983). The model outputs included the road user costs for various alternate highway conditions.

Badarinath (1993) developed a model to simulate the mixed traffic flow on one-way roads of widths ranging from 3.5 m to 6.5 m at 1 m. increments. The position of vehicles across the width of the road was determined based on Convolution Theorem. The behaviour of driver in positioning vehicle in a dynamic traffic stream was represented by means of a new concept based on Information Theory. This process involved conversion of the information received by the drivers into bits and comparing these bits of information with the threshold values derived from the previous accident data. However, this concept needs to be validated. A detailed discussion of the model is presented in Table 2.2

Kuncheria (1995) developed simulation models to study the mixed traffic flow on one-way roads, at bottlenecks, merging and diverging locations and on two-way roads in urban areas. The overtaking logic in this study was based on probability of accepting the available space in lateral direction. He reported that the simulation outputs such as the stream speeds and the flows remain practically unaltered, irrespective of the headway distribution model used for generation of vehicle arrivals, if the observations are taken after sufficiently long distance from the entry. He reported that the model could not replicate the high-density flow conditions. Some of the details of this model are presented in Table 2.2.

Table 2.2 Summary of Three Simulation Models of Traffic Flow on Urban Mid-Blocks in Mixed Mode Environment

Name of Researcher / Description	Ramanayya (1980)		Badarinath (1993)	Kuncheria (1995)	
Scanning Technique	Time scanning, 1 sec		Time Scanning, 1 sec.	Time Scanning, 1 sec.	
No. of Vehicle classes	8 Classes		6 classes,	5 classes	
Entry Process	Based on headway distribution models up to 500 vph - Exponential 500 - 650 vph: Shifted Exponential 650 - 900 vph: Log-Normal		Based on Headway distribution Models Upto 500 vph: Exponential 500 - 800 vph: Shifted Exponential 800 - 1100 vph: Log-Normal 1100-1500 vph: Erlang 1500-2000 vph: Triple Exponential	Based on Headway Distributions Up to 500 vph: Exponential 500 -2000 vph:Shifted Exponential 2000-3000 vph: Erlang & Composite models	
Road Geometry	Straight & Horizontal		Straight & Level	Straight & Level	
Type of Flow	One-way & Two-way		One Way	One-way & two-way	
Free Speeds(kmph)	Normal Distribution		Normal Distribution	Normal Distribution	
Type	Mean	S.D.	Four Wheelers: Mean - 52.5	Mean	S.D.
Cars	55	10.0	S.D.- 8.15	50.5	10.2
Bus	50	10.0	Two/Three Fast Vehicles: Mean-47.5	50.1	10.5
Truck	40	6.50	S.D.-7.30	-	-
Auto-rickshaw	45	5.0	Two/Three Slow Vehicles: Mean-12.0	40.4	6.5
Two-wheeler	45	5.0	S.D.- 2.50	45.1	8.5
Cycle	10	1.5		16.1	4.33
Acceleration/ Deceleration Values	Uniform acceleration values as given by IRC Deceleration values from curves reported in Transportation Engg. Hand Book		Zero jerk acceleration models developed based on limited observations	Non-linear models Model parameters derived from limited observations	
Flow Logic	Based on available & required linear & lateral clearances Generalised car following model		Based on available & required linear & lateral clearances and risk the vehicle is subjected to. Generalised car following model	Based on available & required linear & lateral clearances for different combination pairs of vehicles	
Clearances Required	Considered in terms of percent accepted or rejected 'x' times the width of the vehicle. Based on limited observations			Exponential models derived from field observations	

Table 2.2 Summary of Three Simulation Models of Traffic Flow on Urban Mid-Blocks in Mixed Mode Environment

(contd.)

Internal Book Keeping	List Processing or Chaining			List processing or chaining				Circular Array concept		
Outputs	Avg. speed of traffic stream & each class of vehicles Distribution of headways Classified volume count Concentration Avg. delay for each class of vehicles Number of Overtaking			Headways Average speeds Volume Group size (platoon headway: 2 sec.)				Headways Average speeds of stream and each class of vehicles Volume Density Composition		
Validation Criteria	Speed distribution (6.6% to 8.6%) Average Travel Time Distribution of Headways Average Delay			Field validation Extent of platooning				Average Stream Speed (- 2.6 to 5.9 %)		
Results of Experimentation										
Stream Composition (%FMV & %SMV)	90-10	70-30	50-50	% of four wheelers						
				50	35	25	10			
Volume (vph)	800	800	800	800						
Speed - Average.	34.25	29.25	18.00	47	48	48.5	48.5			
Maximum	42.50	37.25	30.00	56.5	58	58	58	At Capacity		
Density	23.35	27.3	41.60					Flow	Speed	Density
Capacity Values- Cars	2044	880	385					400	27	148
Buses	2270	1190	600					2000	26	70
Trucks	2089	991	466					-	-	-
Auto-rickshaws	1865	910	445					4400	26	165
Two-wheelers	3290	1370	541					8000	27	290
PCU Values at Capacity Car	0.98	2.27	5.19	Not suggested PCU Values				Range of Variable PCU Values for One-way flow		
Bus	0.88	1.68	3.33					1.21 - 2.20		
Truck	0.96	2.02	4.29	Level-of-Service		Group Size		-		
Auto-rickshaw	1.07	2.20	4.49	A		<6		0.76 - 1.63		
Two-wheeler	0.61	1.46	3.70	B		6-10		0.51 - 1.00		
				C		10-13				
				D		>13				

Arasan and Koshy (2003) developed a discrete event simulation model with time scanning technique to simulate the traffic flow on urban roads of different widths for deriving capacity and service volume standards. They validated the model based on headway distribution and speeds. They found a decreasing trend in PCU values as their proportion in the traffic stream increased. They derived the capacity values for urban roads for three typical traffic compositions.

Even though a good number of simulation models have been developed by researchers, the research so far has not been able to describe the behaviour of mixed traffic on urban mid blocks. This may be due to the complexity of the mixed traffic, lack of instrumentation for collection of data required for development of component models, lack of understanding of the vehicular interaction process or lack of high-speed computer facilities with sufficient memory for simulation of such a dynamic and complex system.

The following conclusions are possible to be drawn from the above discussion:

i) The empirical approach may not be able to provide solutions to the mixed traffic flow for the reason that it:

- Involves collection of huge data, which is impracticable due to the complexity of the system and lack of proper instrumentation.
- Does reflect only aggregate behaviour
- Is deterministic in nature and does not reflect the driver behaviour.

ii) The probability approach to traffic flow study results in mathematically intractable models due to the several inherent stochastic processes under mixed traffic.

iii) The mixed traffic is not amenable to study by any of the above two approaches

iv) The simulation approach seems to be more appropriate for study of mixed traffic, for the following reasons:

- It is based on integration of easily understandable component models.
- Experiments can be conducted under controlled conditions.

Hence, it has been decided to study the characteristics of unidirectional traffic flow in mixed mode environment through simulation approach.

2.8 NEED FOR AN INTEGRATED APPROACH

Many of the relationships governing the individual vehicle manoeuvres in traffic flow simulation models are generally mathematical or empirical in nature. Also, in simulation approach, the simulated system is scanned or updated at regular intervals of time in accordance with stored instructions or rules of the model. The method of driver decision-making in simulation is rule based. However, in real life situation, a driver continually reviews his driving situation. As he scans the environment, his brain receives a stream of images or patterns defining the surrounding traffic situation. The data are fed to the brain in their raw state without any pre-processing. The driver's behaviour resulting from this input is probably based on a learned reaction to a particular situation described by the input, and therefore can be represented by a technique that follows this process more closely.

Artificial Neural Networks, which attempt to mimic the functionality of the human brain in a fundamental manner, provide the basis for a possible alternative modelling technique. In this approach, the collected data are presented directly to the model without any processing and thus the approach has the potential to avoid the bias, which might be introduced due to subjective interpretation of the model builders. Neural network, considered as a black box for all practical purposes, by being exposed to a large data, identifies the relationships between a number of input parameters and the corresponding output parameters. Hence, it is proposed to use neural networks for describing some of the driving subtasks in this study.

2.9 CONCLUSIONS

An overview of the research on traffic flow modelling during the last five decades has been presented. The complex nature of traffic systems makes them an excellent application environment for simulation. Review of the mixed traffic flow simulation models revealed that the relationships used to represent the various subtasks are deterministic and do not reflect the driver behaviour. It is proposed to develop a simulation model of mixed traffic flow by integrating neural network models of some of the subtasks of driving.

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CHAPTER - 3
ARTIFICIAL NEURAL NETWORKS

3.0 INTRODUCTION

A simulation model consists of a number of models that describe the various component of the simulated traffic system. The logic of manoeuvring individual vehicles through the simulated system forms the core of the traffic flow simulation models. The models used to represent the vehicle movement in the present day simulation models are mostly rule based and are derived by empirical methods from pre-processed data. As the driver behaviour is stochastic, the rule based models fail to represent the real life situation. The techniques, which have a performance close to human behaviour, are likely to represent the process better than rule-based models. For many reasons, neural computing offer a more realistic tool for modelling the highly probabilistic behaviour of drivers. An overview of artificial neural networks and their applications in the field of traffic engineering is the subject matter of this chapter.

3.1 APPLICATIONS OF NEURAL NETWORKS FOR TRAFFIC ENGINEERING STUDIES

Many researchers have reported that the neural networks offer a potential alternative tool for traffic engineering studies. Dougherty *et al.* (1993) used neural network to predict and classify traffic congestion. They made use of the data collected via an on line computer link to the SCOOT traffic control system and concluded that neural computing is well suited for analysis of large volumes of data. Hsio *et al.* (1994) successfully applied fuzzy logic and neural networks to predict the traffic congestion due to roadway incidents. Ledoux (1997) proposed a cooperation based neural network traffic flow model that is being integrated into a real time adaptive urban traffic control system. The model is expected to simulate the flow at an isolated intersection surrounded by neighbouring intersections. Fair performance of neural networks to predict the queue lengths for the next 60 sec was reported. Dharia and Adeli (2003) presented a neural network model for forecasting the freeway link travel time using the counter propagation neural network. The performance of the model was compared with that of a model using the back propagation algorithm. The model based on the counter propagation algorithm was reported to be faster than the back propagation network. The proposed freeway link travel-forecasting model finds use for real-time advanced travel information and management systems.

Pant and Balakrishnan (1994) studied the gap acceptance behaviour of drivers at stop-controlled intersections using neural network models. They compared the results of neural network model with that of the Binary-Logit model and concluded that the neural networks predicted the accepted or rejected gaps better than the Binary-Logit model.

Hunt and Lyons (1994) used the neural networks for modelling the dual carriageway lane changing process. In a later paper, Lyons(1995) concluded that neural models performed exceptionally well, in predicting lane-changing decisions of drivers on dual carriageway.

Mussone et. al. (1999) described a method based on the use of artificial neural networks (ANN) in order to work out a model that relates to the analysis of vehicular accidents in Milan. They used feed-forward neural networks with a back-propagation learning paradigm. The degree of danger of urban intersections under different scenarios was quantified by using ANN model. The main difficulty of including into a regression model the class variables such as day/night, type of intersection, class of roadbed, weather and, above all, type of accident can be easily overcome by using neural network models. Srinivasan et al (2004) evaluated the performance of a neural network model, originally developed for a freeway site in Singapore, for detection of incidents on a freeway site in California and observed that the neural network models are portable.

Awad (2004) developed models for estimating capacity of weaving segments of freeways using regression and neural networks. The multi-layer feed-forward neural network model was trained using an improved back propagation algorithm with momentum and adaptive learning rate. Although, linear regression technique showed satisfactory results, neural network technique outscored linear regression in the prediction performance, and generalization ability. The trained neural network architecture represented by weight and bias values for each layer can be simply used to predict capacity for weaving segments under new conditions.

Jiang and Zhang (2001) applied ANN technology to map the relationship of the traffic flow and average-space speed using the data from an arterial road in

Changchun, China. They concluded that this method make the predictions more reliable, more cost-efficient and easier.

Artificial neural networks (ANNs) have been extensively studied and used in time series forecasting. Yasdi (1999) designed and trained a neural network based on recurrent Jordan architecture popular in the modelling of time series to predict the future values of the traffic time series using its past values. He considered three types of forecasting: weekly, daily and hourly predictions as long-term, mid-term and short-term predictions respectively. The weekly prediction relies on the historic pattern stored in the knowledge base, while for daily prediction, for each day of the week and for each predicted event, a reference pattern is stored which can be used for corresponding predictions. The hourly prediction, however, is a short time prediction, which gives a view of the current traffic situation. The trained values were compared with the corresponding actual values, and they were found to be in close agreement. These results were reported to be better than that of the compared methods. The model improved the forecasting by about 20% for the road traffic flow. The neural network method requires a database to perform the learning task successfully. Therefore, the database has to be updated frequently.

Messai et al (2002) proposed a new short term traffic flow prediction model based on a feed forward neural network and observed good performance of the model by comparing with simulated and real life data. Xiao and Wang (2003) and Xiao (2004) applied radial basis function neural network with feedback for traffic flow modelling and concluded that ANN has the excellent capability of describing the highly non-linear dynamic characteristics of the traffic flow.

Zhang (2003) proposed a hybrid methodology that combines both ARIMA and

ANN models to take advantage of the unique strength of ARIMA and ANN models in linear and non-linear modelling. The major advantage of neural networks is their flexible non-linear modelling capability. With ANNs, there is no need to specify a particular model form. Rather, the model is adaptively formed based on the features presented from the data. This data-driven approach is suitable for many empirical data sets where no theoretical guidance is available to suggest an appropriate data generating process. Chen et al (2003) proposed Grey Neural Network, a novel method that combines grey system theory with neural network, to forecast highway traffic flow and concluded that the GNN model outperformed the other methods of traffic flow forecasting. Hence, neural networks have great potential for application in traffic engineering.

3.2 ARTIFICIAL NEURAL NETWORKS

Artificial neural networks consist of large number of simple, inter-connected processing units or nodes called neurons. The neuron consists of three basic elements as described below:

i. A set of *synapses or connecting links*, each of which is characterised by a weight or strength of its own. A signal at the input of the synapse connected to the neuron is multiplied by its synaptic weight. The weight is positive if the associated synapse is excitary; it is negative if the synapse is inhibitory.

ii. An *adder* for summing the input signal, weighted by the respective synapses of the neuron.

iii. An *activation function* for limiting the amplitude of the output of neuron

The model of a neuron is presented in Fig. 3.1. A neuron takes in a set of inputs and computes an output according to a transfer function. The inter neuron

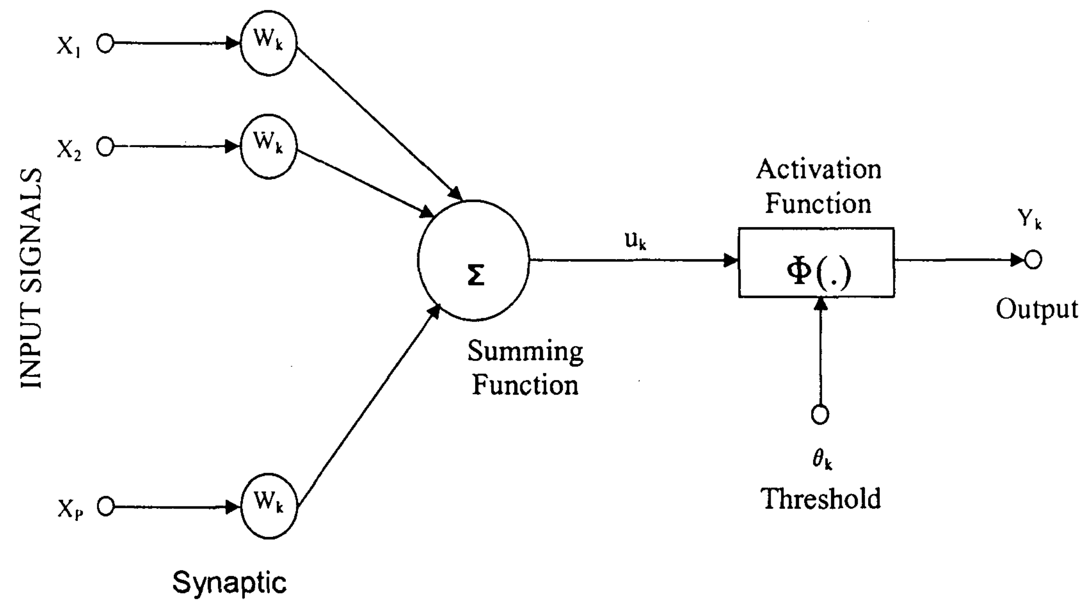

Fig 3.1 Model of a Neuron

connections will be of varying strength; each connection having a weight associated with it. The output state of a neuron will be either 'off' or 'on'. A change from one state to the other is triggered when the sum of the inputs (weighted by the strength of their respective connections) crosses some threshold. This threshold is usually represented by a transfer function. The neurons will be generally arranged in layers, with nodes in adjacent layers connected to each other, either partially or fully. A neural network thus has an input layer, an output layer and possibly one or more hidden layers. Thus, a neural network is a massive parallel-distributed processor that has a natural propensity for storing experimental knowledge and making it available for use.

The properties of Neural Networks viz., the non-linearity, the input-output mapping, the adaptivity, the fault tolerance, the evidential response, the neurobiological analogy and others make it possible to solve complex problems using this approach.

3.3 NETWORK ARCHITECTURE

The manner in which the neurons of a neural network are structured is known as the network architecture. The network architecture can be broadly classified as: single layer feed forward networks, multi layer feed forward networks, recurrent networks and lattice structures. Multi-layer feed-forward networks are the most widely used.

3.3.1 Multi-layer Feed Forward Networks (MFNN)

This class of a feed forward network distinguishes itself by the presence of one or more intervening layers between the input and output layers called hidden layers. By adding one or more hidden layers the network acquires a global perceptive despite its local connectivity by virtue of the extra set of synaptic connections and the extra dimensions of neural interactions. The MFNN is said to be fully connected if each node in each layer of the MFNN is connected to every other node of adjacent forward layer. If some of the synaptic connections are missing the network is said to be partially connected.

3.3.2 Recurrent Neural Networks (RNN)

A RNN distinguishes itself from a feed forward neural network, in that it has at least one feedback loop. The feedback connections can originate from hidden layers as well as outer layers or from either of these layers. The presence of feedback loops has a profound impact on the learning capability of the network and on its performance. Moreover, the feedback loops involve the use of particular branches composed of unit delay elements, which result in a non-linear dynamic behaviour by virtue of the non-linear nature of neurons. Partial recurrent neural networks are those which have feedback loops from only one of the layers. Of the common partial recurrent neural networks used, Elman network is popular. Elman network is effective in representing both temporal and spatial patterns.

3.4 LEARNING PROCESS

Learning is a process by which the free parameters of neural network are adapted through a continuing process of stimulation by the environment in which the network is embedded. The learning process involves the following steps:

i. The neural network is stimulated by an environment.

ii. The neural network undergoes changes as a result of this stimulation.

iii. The neural network responds in a new way to the environment, because of the changes that have occurred in its internal structure.

The learning paradigms are basically classified as supervised learning, reinforcement learning and self organised learning. The supervised learning is performed under the supervision of an external teacher. Reinforcement learning involves the use of a critic that evolves through a trial and error process. Unsupervised learning is performed in a self-organised manner in that no external teacher or critic is required to instruct synaptic changes in the network. The back-propagation algorithm, which is a supervised learning algorithm, is the most successful and widely used algorithm for the design of multi-layer feed-forward networks used to represent large-scale systems. Fig. 3.2 shows the architecture of a feed-forward network with two hidden layers and one output layer.

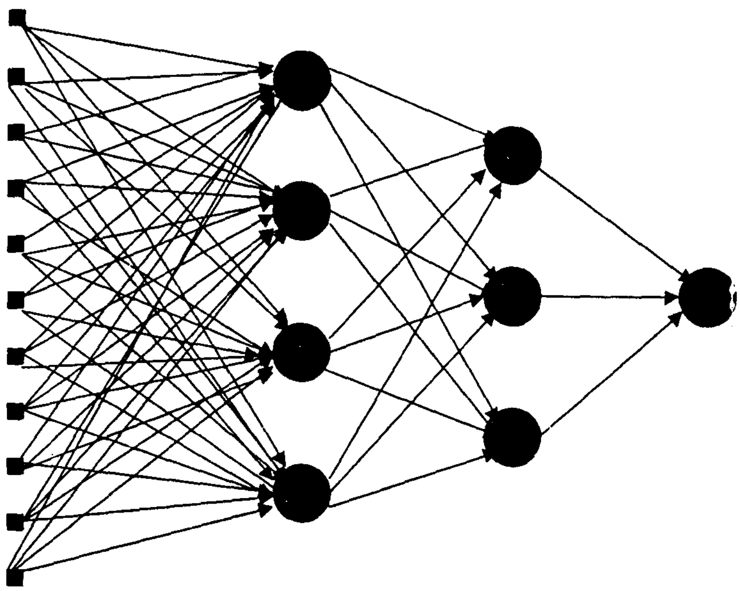

Fig 3.2 Multi-layer Feed-Forward Network

3.4.1 Concept of Back-Propagation Algorithm

In multi layer feed forward networks, the input signal propagates through the network in a forward direction, on a layer by layer basis. The error back-propagation process consists of two passes through different layers of network: a forward pass and a backward pass. In the forward pass, an activity pattern is applied to the sensory nodes of the network, and its effect propagates through the network layer by layer. Finally, a set of outputs is produced as the actual response of the network. During the forward pass the synaptic weights of the network are all fixed. The actual response of the network is compared with the desired response and error signal is calculated. The error signal is then propagated backward through the network. The synaptic weights are adjusted so as to make the actual response of the network being closer to the desired response. During the backward process, the synaptic weights are all adjusted in accordance with the error-correction rule. Detailed description of back-propagation algorithm is available in many text books (Wasserman (1989), Haykin (1994)).

The correction $\Delta w_{ij}(n)$ to be applied to the weight W_{ij} is defined by the *delta* rule as

$$\Delta w_{ij}(n) = -\eta \frac{\partial \xi(n)}{\partial w_{ij}(n)} \qquad \text{Eqn. 3.1}$$

where: η: rate of learning.

$\xi(n)$: instantaneous sum of error squares.

W_{ij}: weight of the synapse connecting the neuron 'i' and neuron 'j' at iteration 'n'.

3.4.2 Rate of Learning

The training algorithm provides an approximation to the trajectory in weight space computed by the method of steepest descent. The smaller the learning rate parameter 'η', the smaller will be changes to the synaptic weights in the network from one iteration to the next and the smoother will be the trajectory in weight space. However, this is at the cost of slower rate of learning. If, on the other hand, the learning rate parameter 'η' is too large so as to speed up the learning, the resulting changes in the synaptic weights assume such a form that the network may become unstable. A simple method of increasing the rate of learning and yet avoiding the danger of instability is to modify the delta rule by including a momentum term.

Considering its frequent use and record of success, a multi layer feed-forward neural network, trained using back propagation algorithm has been selected in the present study.

3.5 CONCLUSIONS

A review of the applications of neural networks has revealed that neural networks offer a potential alternate method of modelling traffic flow in mixed mode environment. An overview of the neural networks was presented in this chapter. It is suspected that neural networks might not be able to provide total solutions to complex systems, such as mixed traffic flow, as they need prior exposure in the form of training. Rather, they appear to be more useful for representing some of the sub-systems in the simulation model. Hence, it is proposed to use neural networks for modelling some of the component blocks of the traffic flow simulation model.

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CHAPTER – 4

DEVELOPMENT OF COMPONENT MODELS

4.0 GENERAL

A traffic flow simulation model consists of a number of component models that represent the essential processes in the system being simulated. The development of various component models along with a brief review of related literature, data collection and analysis is the subject of this chapter. The validation of these component models is also presented in this chapter.

4.1 DESIRED LINEAR AND LATERAL CLEARANCES

In mixed traffic, the interactions among vehicles take place not only in linear direction, but also, in lateral direction. Models of linear and lateral clearances are useful for many purposes such as finding influence areas, determining PCUs and development of geometrical design standards. The minimum linear and lateral clearances desired by individual driver-vehicle units, to continue their movement, form the most important ingredient of simulation model. In homogeneous traffic conditions, wherein, the lane discipline is strictly followed, the linear clearance (or spacing) maintained by the vehicles with respect to the vehicles in front may be sufficient to describe the vehicle manoeuvring process. However, in mixed traffic, the lane discipline is absent and the vehicles travel abreast due to the variations in their size and non-standard widths; so it becomes essential to consider the clearances in lateral direction in addition to that in linear direction for defining the flow logic.

4.1.1 Literature Review

There are not many references available on the study of linear and lateral clearances of vehicles for homogeneous traffic. There are numerous attempts to describe the linear and lateral clearances maintained by vehicles in mixed traffic. Chari *et. al* (1990) developed relationships between speed and lateral spacings between pairs of vehicles. It was observed that the lateral clearances increased with increase in speed. The lateral clearances have therefore pronounced effect on behaviour of vehicles. Nagaraj *et. al.* (1990) studied the linear and the lateral spacings of vehicles using data from Calicut city. They found that spacings show a fluctuating trend with respect to speed. They used the values of linear and lateral spacings for estimation of the Influence Areas of vehicles, which in turn were used to determine the demarcation criteria for different levels of service and to estimate the PCU values of different classes of vehicles. But they did not propose any mathematical models, which could be used in simulation studies.

Kuncheria (1995) used the models of linear and lateral space gaps for different combinations of pairs of vehicles, derived from field observations, in his simulation study of mixed traffic. He reported that linear space gaps vary linearly with speeds. These relationships were derived by considering the mean values of space gaps for different speed ranges and thus his study reflects the average characteristics of vehicles, unlike the characteristics of individual vehicles required for a simulation work.

The placement of a vehicle with respect to the surrounding vehicles in a traffic stream depends on a number of factors. Some of these factors are the type and the speed of the vehicle, the type and the speed of influencing vehicle, the stream speed, the flow, the density and the traffic composition and the driver characteristics, etc. The literature on linear and lateral spacings that has been discussed so far is mostly confined to development of macro level parameters which could at best be used for deriving some broad scale factors like PCU or for an overall assessment of level of service, but those are of limited use in simulation modelling. Akbar (1995) tried to describe the linear and lateral clearances of vehicles in mixed traffic by probability models, like, the Exponential, the Erlang, the Gamma and the Log-Normal distributions, but found that none of those could be a good fit.

4.1.2 Data Collection

Considering the kind of information required for modelling the linear and lateral clearances and the advantages of the video graphic technique, video graphic technique was adopted for the conduct of field studies. The video equipment consisted of a video camera cum recorder, portable video cassette player and a T.V monitor. The field recording was done at four typical urban mid-blocks in Calicut city. The study sections selected were straight and uniform width and free from the influence of bus stops, intersections, parking etc.

Video camera mounted on a tripod was installed on the top of the adjacent tall building, such that at least 30 meters length of the road for full width was clearly visible. Paint markings were made on carriageway at 5 meters interval for 30 meters and at 1 meter interval in transverse direction for the full width of the carriageway. The recording was done for nearly three hours during both morning and evening periods so that wide range of flow conditions are covered.

4.1.3 Data Retrieval

The required information was retrieved by replaying the recorded cassette in laboratory and adopting the following procedure. The grid points marked on the carriageway were transferred on to a transparent sheet pasted on the monitor. The grid points were then joined using a marker pen to obtain the grid lines.

The information about the frontal spacings was obtained using a program written in 'C'. For a pair of vehicles, one behind the other at close spacing, the information recorded included:

i. Type, time of entry and exit of front vehicle

ii. Type, time of entry and exit of following vehicle

Using the above information the speeds of front and following vehicles were calculated. The time headway between the two vehicles was obtained by taking the difference in entry times of following and front vehicles. The time headway and speed of front vehicle were used to find the front spacing. In all, data of a total of 5000 vehicles was collected and the data was stored in the format as shown in Table 4.1 for future use. This procedure was adopted to avoid the judgmental errors in estimating the frontal clearances from grid marks.

Table 4.1 Sample of Data Used for Modelling of Frontal Spacings

S.No.	Type of Front Vehicle	Type of following vehicle	Speed of front vehicle (kmph)	Speed of following vehicle (kmph)	Front Spacing (m)
1	6	6	38.136	42.056	15.678
2	5	4	34.092	34.884	3.126
3	5	1	30.406	38.962	7.433
4	6	2	36.438	34.22	17.207
5	5	6	30.821	33.458	12.671
6	6	5	34.22	32.144	14.068
7	5	2	32.142	34.883	6.875
8	5	5	34.884	34.091	6.977
9	5	5	33.332	34.091	6.666
10	4	5	32.727	31.579	4.545

4.1.4 Regression Analysis of Frontal Spacings

The data collected from video recordings was used to describe the frontal spacings in terms of different variables such as the speed of subject (following) vehicle, relative speed (speed of subject vehicle minus the speed of front vehicle). A plot of the frontal spacings Vs speed of subject vehicle is shown in Fig 4.1. It can be seen from the plot that even though there is an increasing trend in frontal spacings with increase in speed, the points are very widely scattered. To reduce the scatter, the frontal spacings were classified based on the type of subject vehicle. The plots of the frontal spacings for various types of vehicles are shown in Fig 4.2. It can be observed from these plots that there is not much reduction in the scatter of the points. Hence, the frontal spacings for each combination of front – following vehicles are considered. Plots of frontal spacings for various combinations of vehicles were prepared and selected plots are presented in Fig 4.3. The increasing trend of spacings with respect to increase in speeds can be observed from these figures. Even though the scatter is less at low speeds, the scatter is more at higher speeds.

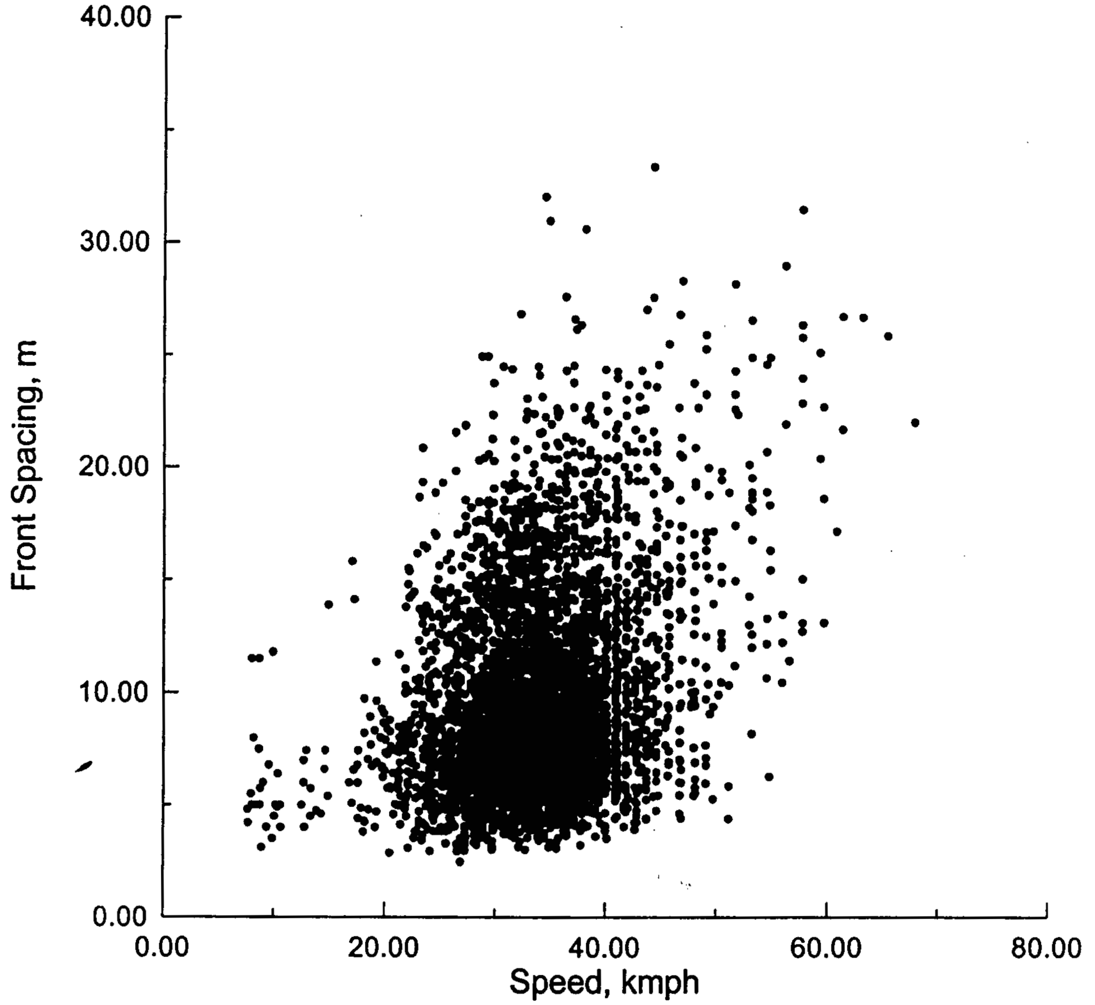


Fig 4.1 Frontal Spacing Vs Speed for All Vehicles

The front spacings were modelled using various variables as explanatory variables and different model formulations. The models tried for car-car combination are given in Table 4.2. The performance of all the models was poor. Among the models tried, the polynomial model with second order term was having highest R^2 value, but the improvement over first order model was negligible. Hence, first order linear models in terms of speed of subject vehicle are considered for further investigation.

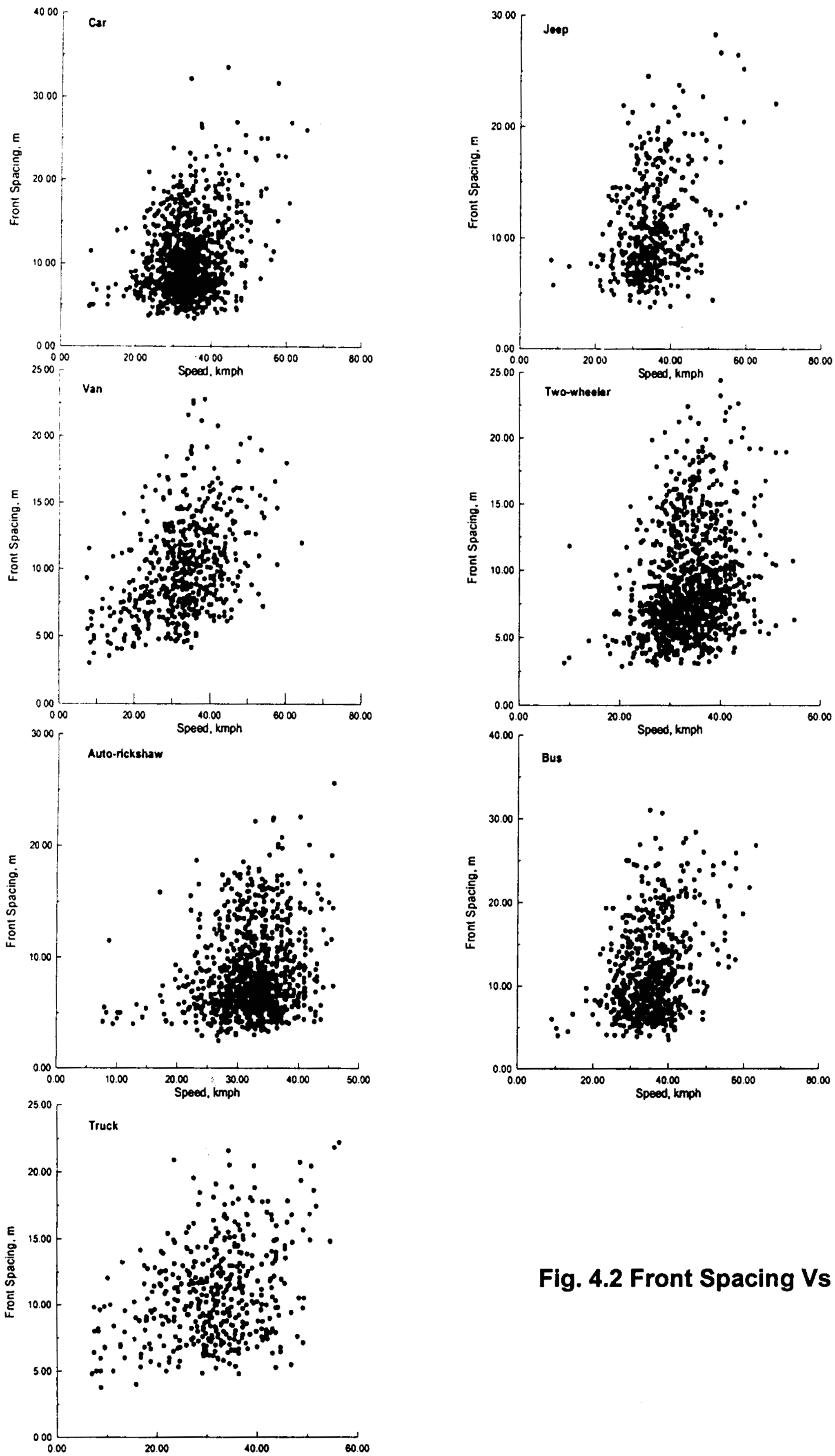
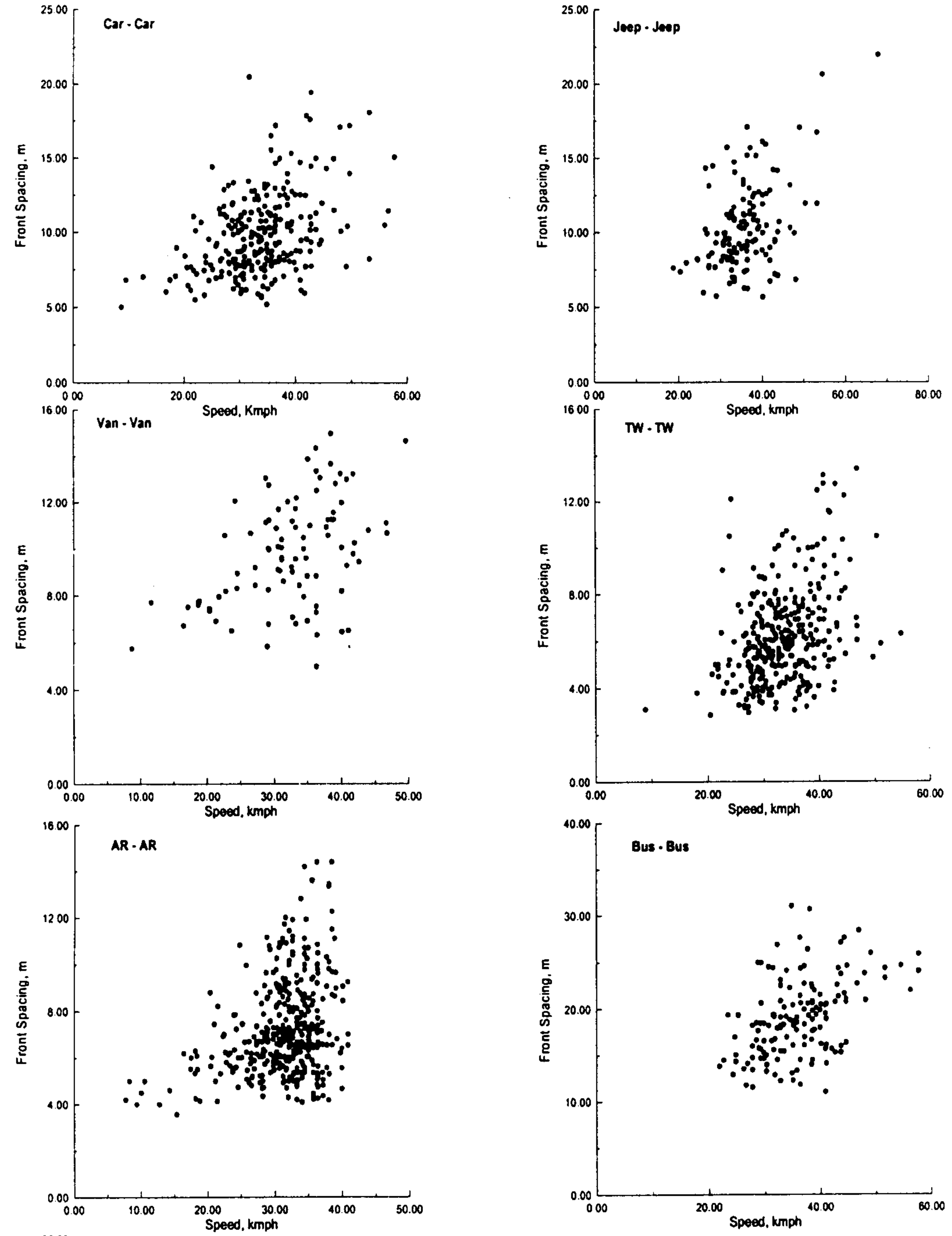


Fig. 4.2 Front Spacing Vs Speed



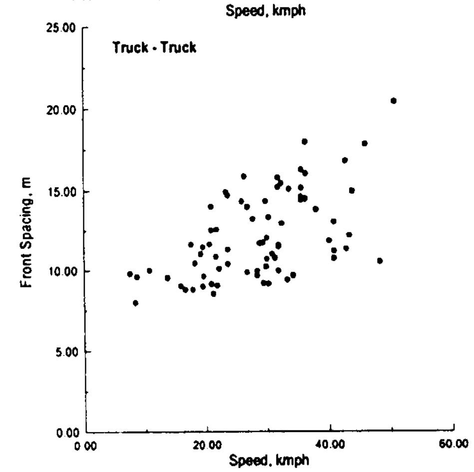


Fig 4.3 Front Spacing Vs Speed

(Selected Combinations of Vehicles)

Table 4.2 Regression Models for Frontal Spacings (Car – Car Combination)

S.No.	Model	R^2 - value
1	Fsp = 2.821 +0.1276 * Speed	0.090
2	Fsp = 6.20 – 0.0645 * Speed + 0.0027 * Speed^2	0.093
3	Fsp = 7.313 + 0.0825 * Relative Speed	0.012
4	Fsp = 4.049 + 0.844 * $e^{0.0377 * Speed}$	0.092

The general procedure adopted in order to improve the R^2 – value is to develop models for average values in terms of explanatory variables. The spacings data was classified based on speed and average spacing was calculated for each speed class. Models were built for average spacings in terms of speed using regression analysis and the results are given in Table 4.3. The available data was partitioned into two sets one set being used for model development and the other set being used for validation of models. The data set used for model building comprised of two-thirds of data records selected at random and the validation data set comprising the remaining one-third of data.

Models were also built considering the individual spacing values. Performance of these models was compared with that of models based on average values by calculating the %RMSE for both the data sets, calibration data set and validation set. The %RMSE values of models for two-wheeler are given in Table 4.3.

Models built are of the form:

$y = a_a + a_1 * x$

where y – front spacing (mean front spacing) in m.

x – speed (mean speed) in kmph

Table 4.3 Comparison of Models based on Average & Individual Data for Two Wheeler (TW)

Front vehicle	a_0	a_1(t – value)	R^2 – value	F – value	%RMSE	
					Calibration Data Set	Validation Data Set
Car	4.238	0.127 (4.764)	0.195	22.69	2.257	1.965
	3.537	0.147 (14.023)	0.975	196.65	2.260	1.962
Jeep	4.648	0.117 (3.283)	0.109	10.77	1.936	1.797
	3.314	0.154 (19.278)	0.987	371.63	1.949	1.829
Van	4.574	0.119 (4.156)	0.229	17.27	1.351	1.299
	3.732	0.146 (6.257)	0.907	39.07	1.365	1.398
TW	1.990	0.125 (7.277)	0.139	52.94	1.847	1.840
	2.394	0.112 (9.397)	0.957	88.29	1.848	1.843
Auto	4.425	0.089 (4.368)	0.058	19.08	1.934	1.896
	4.162	0.095 (9.716)	0.959	94.39	1.936	1.885
Bus	8.570	0.199 (5.899)	0.150	33.97	2.553	2.261
	9.986	0.148 (9.143)	0.954	83.59	2.586	2.271
Truck	7.886	0.211 (6.465)	0.230	41.79	2.306	2.042
	7.726	0.207 (10.267)	0.946	105.41	2.324	2.069
While the first row corresponds to the model based on individual data, the second row corresponds to the model based on average data.						

The models based on average values are having very high R^2 - values compared to the models based on individual values. But a comparison of %RMSE of these two types of models revealed that the predictive ability of both these types of models is almost the same. Hence, it is decided to use the models based on the individual values instead of the models based on average values. The values of regression coefficients are given Table 4.4.

Table 4.4 Parameters of Regression Models for Front Spacings

	a_0	a_1 (t-test value)	R^2 – value	F – test value
Reference Vehicle	Subject Vehicle - CAR			
Car	4.558	0.152 (6.497)	0.210	42.210
Jeep	4.680	0.143 (5.761)	0.278	33.180
Van	5.183	0.162 (6.986)	0.378	48.803
Two wheeler	2.985	0.128 (6.576)	1.996	43.246
Auto	4.265	0.123 (4.859)	0.114	23.610
Bus	8.544	0.239 (8.712)	0.274	75.890
Truck	7.027	0.163 (4.869)	0.282	23.704
Mini-Truck	5.987	0.148 (4.793)	0.225	22.969
Reference Vehicle	Subject Vehicle – JEEP			
Car	3.249	0.201 (7.305)	0.277	53.357
Jeep	3.271	0.198 (6.314)	0.222	37.628
Van	4.123	0.184 (5.425)	0.255	29.428
Two wheeler	2.417	0.130 (8.531)	0.461	72.773
Auto	3.787	0.123 (7.916)	0.315	62.760
Bus	9.366	0.222 (6.494)	0.299	42.172
Truck	7.810	0.271 (8.682)	0.301	75.370
Mini-Truck	5.400	0.158 (12.434)	0.691	154.534
Reference Vehicle	Subject Vehicle – VAN			
Car	4.185	0.174 (6.695)	0.299	44.825
Jeep	5.617	0.131 (3.764)	0.172	14.167
Van	5.411	0.136 (4.926)	0.212	24.264
Two wheeler	3.138	0.116 (7.226)	0.341	52.214
Auto	3.492	0.127 (8.806)	0.351	77.541
Bus	10.667	0.174 (3.902)	0.233	15.228
Truck	8.773	0.119 (5.103)	0.383	26.045
Mini-Truck	5.199	0.168 (5.553)	0.299	19.156
Reference Vehicle	Subject Vehicle – TWO WHEELER			
Car	4.238	0.127 (4.764)	0.195	22.699
Jeep	4.548	0.117 (3.283)	0.109	10.777
Van	4.574	0.119 (4.156)	0.229	17.273
Two wheeler	1.990	0.125 (7.277)	0.139	52.948

Auto	4.425	0.089 (4.368)	0.058	19.083
Bus	8.570	0.199 (5.899)	0.158	33.974
Truck	7.886	0.211 (6.465)	0.230	41.790
Mini-Truck	5.877	0.124 (4.131)	0.164	17.065
Reference Vehicle	Subject Vehicle – AUTO RICKSAHAW			
Car	4.160	0.127 (7.867)	0.255	61.897
Jeep	4.331	0.140 (5.981)	0.238	35.773
Van	5.004	0.143 (3.543)	0.180	12.554
Two wheeler	2.122	0.119 (6.798)	0.145	46.211
Auto	3.654	0.112 (6.462)	0.131	41.762
Bus	9.257	0.180 (4.468)	0.125	19.908
Truck	7.133	0.193 (3.738)	0.163	13.979
Mini-Truck	5.824	0.092 (4.650)	0.272	21.622
Reference Vehicle	Subject Vehicle – BUS			
Car	4.395	0.162 (7.926)	0.268	62.819
Jeep	4.946	0.142 (5.330)	0.262	28.414
Van	4.720	0.153 (6.395)	0.318	40.891
Two wheeler	3.326	0.100 (4.335)	0.186	18.789
Auto	4.323	0.115 (4.700)	0.114	22.086
Bus	9.115	0.272 (6.226)	0.209	38.769
Truck	7.595	0.197 (5.492)	0.331	30.166
Mini-Truck	5.924	0.209 (6.289)	0.361	35.549
Reference Vehicle	Subject Vehicle – TRUCK			
Car	3.986	0.176 (4.969)	0.295	24.688
Jeep	4.511	0.159 (5.256)	0.336	27.629
Van	4.791	0.155 (5.566)	0.323	30.982
Two wheeler	3.280	0.142 (4.935)	0.253	24.359
Auto	4.187	0.143 (4.292)	0.248	18.240
Bus	9.734	0.189 (6.061)	0.352	36.733
Truck	7.680	0.156 (5.653)	0.310	31.957
Mini-Truck	6.537	0.157 (5.814)	0.346	33.799
Reference Vehicle	Subject Vehicle – MINI TRUCK			
Car	4.725	0.128 (5.775)	0.214	13.646
Jeep	5.033	0.134 (5.297)	0.264	18.053

Van	4.615	0.165 (5.917)	0.422	35.009
Two wheeler	3.489	0.105 (6.199)	0.365	38.431
Auto	3.983	0.140 (6.209)	0.421	38.553
Bus	9.949	0.176 (6.613)	0.431	38.776
Truck	7.781	0.153 (7.843)	0.345	32.545
Mini-Truck	5.284	0.208 (5.986)	0.354	20.827

The poor performance of these regression models may be due to the large variations in the frontal spacings maintained by drivers even for a particular combination of vehicles. The large variations can be attributed to the driver effect. Regression models being deterministic in nature fail to capture such behaviour of drivers.

There are two approaches to overcome this difficulty. One is to introduce a random component into the regression models to reflect the driver behaviour and the other is to use an artificial intelligence technique that mimics the behaviour of human beings. Both these approaches are tried in the present study.

4.1.5 Regression Models Reflecting Driver Behaviour

In the attempts presented earlier, to model the frontal spacings maintained by a driver of a particular class of vehicles, the type of vehicle on the front and speed of subject vehicle were considered as the explanatory variables. Those models reflect the average behaviour of a particular class of vehicles whereas the spacings vary from driver to driver and time to time. The frontal spacings depend on the type of subject and front vehicles, speed of subject vehicle and the driver behaviour. While, separate models for each combination of vehicles take care of the effect of types of vehicles, the regression models in terms of speed take care of speed effect. So a term, which reflects the driver behaviour, needs to be incorporated in the regression models. Thus, an equation of the form given below may be able to reproduce the spacings as observed in reality.

$$fsp_i = fsp_{ie} + \epsilon_i$$

where: fsp_i – frontal spacing of i^{th} class of vehicle

fsp_{ie} – estimated frontal spacing value using regression model for the i^{th} class of vehicle

ϵ_i - term representing the driver behaviour for I^{th} class of vehicle

As the human behaviour is stochastic in nature, the values of ϵ also follow some stochastic distribution. The human behaviour is generally known to be normally distributed. As shown above, the frontal spacings depend on the type of vehicle, the speed and the driver behaviour. To capture the effect of driver behaviour on the frontal spacings, the spacings for each combination of vehicles were classified based on speed. The class interval was determined using Sturges' formula. Frequency distribution plots were prepared for each speed class for each combination of vehicles. The details of analysis for two wheeler – two wheeler combination are given in Tables 4.5 & 4.6.

Table 4.5 Statistical Parameters for TW – TW Combination

Parameter	Speed, kmph	Front spacing, m
Minimum values	20.45	0.957
Maximum value	54.82	11.260
Mean value	33.65	4.061
Standard deviation	6.146	1.880

Table 4.6 Statistical Analysis Results of Frontal Spacings for various Speed Classes (TW - TW Combination)

Speed Class	No. of Observations	Minimum Value	Maximum Value	Mean Value	Standard Deviation	COV
20-25	14	0.957	7.142	3.030	1.488	0.491
25-30	55	1.061	7.221	3.361	1.502	0.447
30-35	66	1.226	8.817	3.903	1.499	0.384
35-40	47	1.167	10.601	4.294	2.184	0.509
40-45	27	2.014	11.260	5.686	2.650	0.466
45-50	4	3.403	7.584	4.963	1.829	0.369
50-55	3	3.985	8.60	5.667	2.549	0.450

It can be observed from the above that the spacings maintained by drivers vary about a mean value and also follow a normal distribution. This is what we generally observe as aggressive drivers maintain lesser spacing, conservative drivers, slow to react, maintain larger spacing and majority of the drivers follow the front vehicle in a narrow range of distance. The percent frequency distribution plot of frontal spacings for selected combinations of vehicles and speeds are shown in Fig 4.4. In all these figures it can be observed that frontal spacings follow normal distribution. Hence, it was taken that the driver behaviour can be described by normal distribution and the value of the term representing the driver behaviour can be taken as normal variate with zero mean and specified standard deviation.

A difficulty with this approach is to have the mean and standard deviation values estimated based on sufficient sample for each speed class and for each combination of vehicles. It may be tedious and time consuming to determine these values from field observations. To overcome this difficulty it is proposed to estimate the mean value using the regression model and to estimate the standard deviation based on this mean value and coefficient of variation. For this purpose a value of coefficient of variation that represents the general behaviour of human beings is to be obtained. An analysis of coefficient of variation values for various combinations of vehicles and speed classes, indicated that these values varied in the range of 0.090 to 0.438 with mean and median values being 0.227 and 0.224 respectively. The coefficient variation values were also observed to follow normal distribution. Hence, a coefficient variation of 0.225 is taken for estimating the standard deviation, knowing the mean value. The standard deviation values were then used to determine the value of random term in the model for estimating the frontal spacings.

4.1.6 Neural Network Models of Frontal Spacings

The placement of a vehicle with respect to other surrounding vehicles in a traffic stream depends on a number of factors like: the type and the speed of the subject vehicle, the type and the speed of the influencing vehicle, the stream speed, the flow, the traffic composition, the traffic controls and the road geometrics. Also, it varies from driver to driver and from time to time. The effect of many of these influencing variables is difficult to be directly observed and quantified, in addition to being non linear.

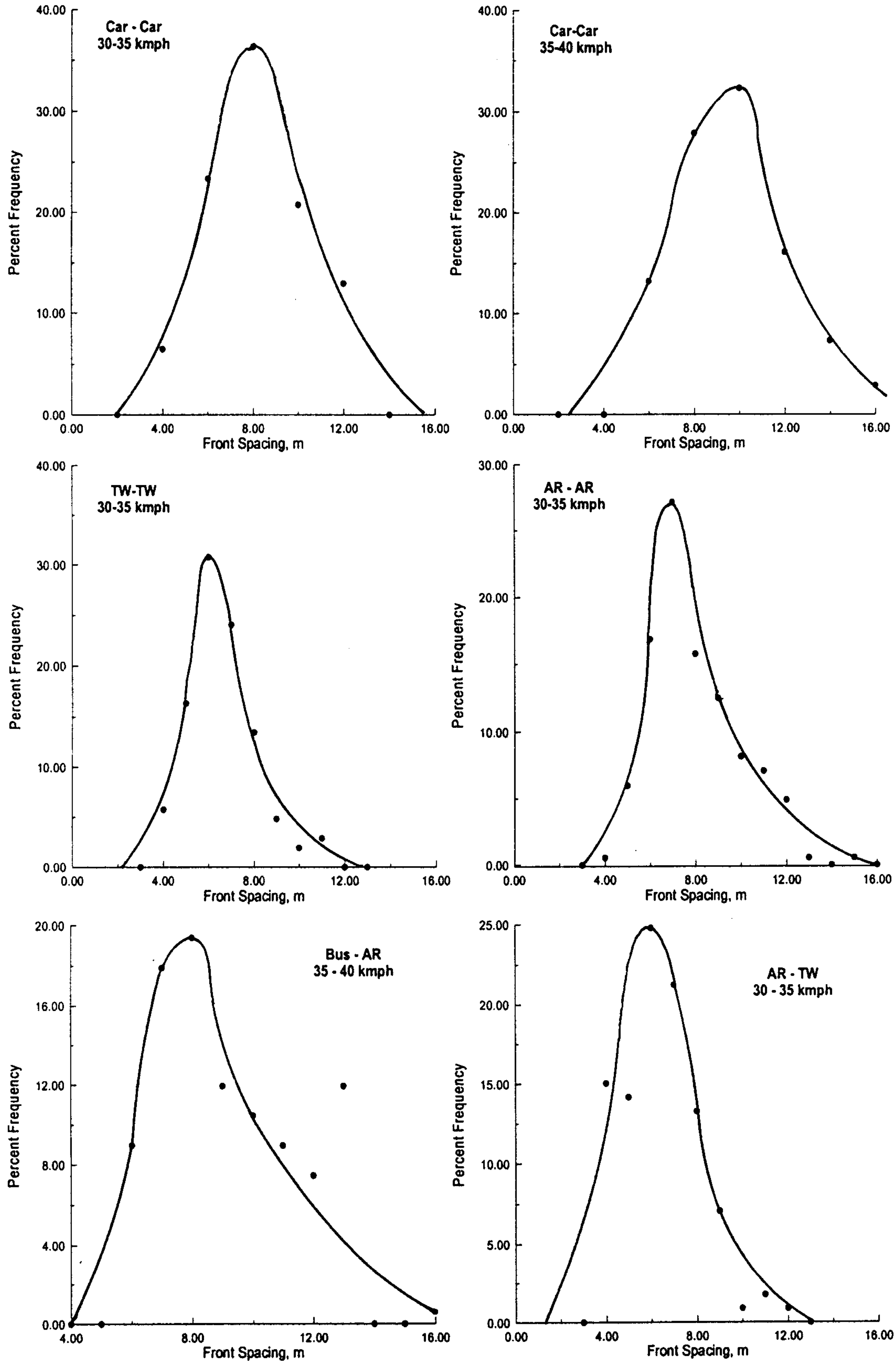


Fig 4.4 Percent Frequency Distribution of Front Spacings

Neural networks, made up of massively interconnected but very simple processing units called neurons, emulate human behaviour. Neural networks by being exposed to a large number of examples during the training process, identifies the input/output relationships, without any preconception or bias for storing experimental knowledge and making it available for use.

Feed forward networks with one or more hidden layers and trained in supervised manner, with error back propagation algorithm, have been successfully applied to solve many difficult and diverse problems (Haykin,1994). The feed forward neural network with a back-propagation learning paradigm and with two layers has been used for modelling the frontal spacings. The tool box of the Neural Networks in the Windows based MATLAB (MATrix LABoratory) software package, by Mathworks Inc., U.S. was used to develop models of frontal spacings maintained by different classes of vehicles. Feed forward networks up to three layers can be created and simulated using this package. Several variations of back propagation algorithms are available in the package. The algorithms tried in this study include the simple algorithm (gradient descent), with momentum learning, with momentum and adaptive learning, resilient back propagation algorithm and Levenberg-Marquardt algorithm. The transfer function in the first layer is tan-sigmoid, and the output layer transfer function is linear. For this study, the back propagation algorithm with momentum and adaptive learning rate was used. The Levenberg-Marquardt algorithm was found to be most suitable.

The performance of a neural network depends on the number of neurons in hidden layer. To evolve the number of hidden neurons, experimental runs were made by varying the number of neurons in hidden layer. Fig. 4.5 illustrates the percentage RMS value with respect to number of neurons in hidden layer.

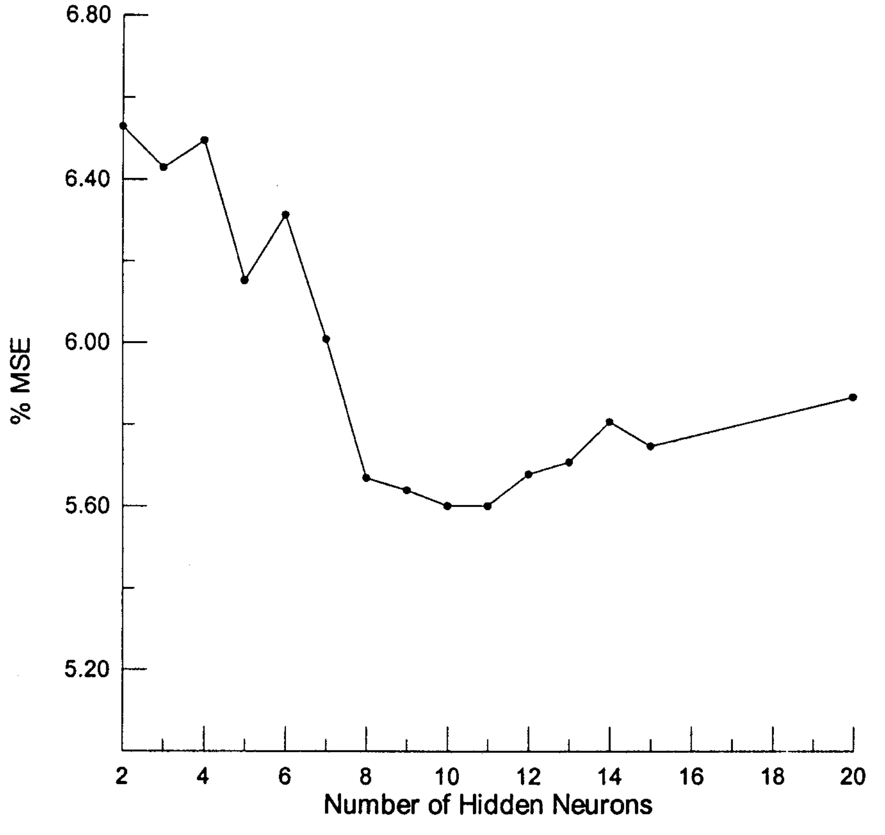


Fig 4.5 Performance Curve for varying Number of Hidden Neurons

It can be observed that the RMS error decreases with the increase in number of neurons up to certain level and then starts increasing. So two layered network architecture with one hidden layer and ten neurons in hidden layer, selected based on experimental results, was used for modelling frontal spacings. The input included the type of front and subject vehicles and speed of front and subject vehicles. The output is the front spacing. The trained network was subjected to the validation data set and the estimated front spacing values were compared with the observed values. MATLAB provides a tool to measure the performance of the trained network. The tool returns the value of the correlation coefficient (R-value) between the outputs and targets. (It is a measure of how well the variation in the output is explained by the targets. If this number is equal to 1, then there is perfect correlation between targets and outputs. A value very close to 1 indicates a good fit). The values of correlation coefficient (r) and RMSE (Root Mean Square Error) for training data set and validation data set are given in Table 4.7.

Table 4.7 Comparison of Performance of Neural Network & Regression Models (All Vehicle Combinations)

Model	Training Data Set		Validation Data Set	
	r	RMSE	r	RMSE
Neural Network Model	0.8460	5.602	0.8403	5.650
Regression Model	0.7490	14.30	0.7990	13.76

A comparison of the estimated and observed values for the validation data set by both the methods is given in Fig 4.6. It can be noted that the regression models over estimate the values, while the values determined using neural models are distributed on both the sides.

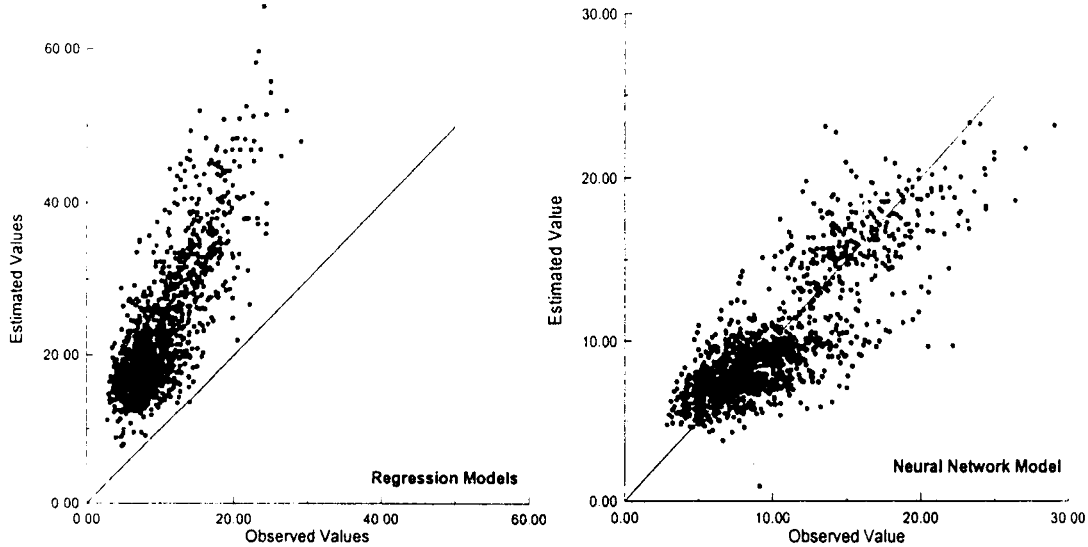


Fig 4.6 Comparison of Performance of Regression and Neural Network Models – Validation Data Set

As the performance of neural models in predicting the frontal spacings is far better than regression models, neural models were used to estimate the frontal spacings in simulation model. The synaptic weights of the neural network were stored for use in simulation model.

4.2 LATERAL CLEARANCES

Another input required for simulation modelling of mixed traffic is the clearances maintained by vehicles on left and right. If the vehicles follow lane discipline strictly, then the clearances on left/right are not important, as, only one vehicle is assumed to be present at a given position in a lane. However, in mixed traffic, the lane discipline is absent and the vehicles travel abreast due to the variations in their size and non-standard widths; so it becomes essential to consider the clearances in lateral direction in addition to that in linear direction for defining the flow logic.

4.2.1 Data Collection

The information required for deriving models for lateral clearances was obtained by analysing the videotapes used for obtaining the data for frontal spacings. The required information were retrieved by playing the recorded cassettes in laboratory and adopting the following procedure. The grid points marked on the carriageway were transferred on to a transparent sheet pasted on the monitor. The grid points were then joined to obtain the grid lines, using a marker pen. The observations noted, by playing and replaying the cassette, included:

i. Type and speed of subject vehicle.
ii. Type of vehicle on the left, its speed and lateral clearance between it and the subject vehicle.

iii. Type of vehicle on the right, if any, its speed and lateral spacing between it and the subject vehicle.

The speed values were determined using a program written for this purpose. The information pertaining to nearly 1800 vehicles were retrieved. The retrieved information were coded and stored in a specific format for easy handling of the data using computers.

4.2.2 Regression Analysis of Lateral Clearances

Regression analysis was carried out with the lateral clearances (left clearance or right clearance) as the dependent variable. The explanatory variables included the type and the speed of both the vehicles and the relative speed of the two vehicles. Of the many models tried, it was found that the models for different pairs of vehicle groups were having better R^2- value than the models for all types of vehicles considered together. It was also observed that the clearances maintained by vehicles depend mainly on the speed of the vehicles. The final selected models, which formed main component of vehicle movement logic, are of the linear form with the speed of subject vehicle as the explanatory variable. The values of the coefficients and statistical analysis results are given in Table 4.8 & 4.9.

Left/Right Clearance = $a_0 + a_1 * $ Speed of subject vehicle

Table 4.8 Parameters of Regression Models for Left Clearances

	a_0	a_1 (t-test value)	R^2 – value	F – test value
Reference Vehicle	Subject Vehicle - CAR			
Car	0.357	0.0183 (5.628)	0.258	31.673
Jeep	0.310	0.0179 (6.385)	0.439	40.772
Van	0.348	0.0165 (4.373)	0.277	19.119

Two wheeler	0.497	0.0119 (4.232)	0.148	17.911
Auto	0.546	0.0180 (3.002)	0.057	9.013
Bus	0.485	0.0093 (3.209)	0.102	10.299
Truck	0.430	0.0115 (3.576)	0.238	12.787
Reference Vehicle	Subject Vehicle - JEEP			
Car	0.468	0.0140 (3.819)	0.198	14.583
Jeep	0.369	0.0156 (4.488)	0.242	20.146
Van	0.390	0.0161 (8.275)	0.248	21.654
Two wheeler	0..278	0.0173 (5.019)	0.292	25.191
Auto	0.338	0.0168 (5.905)	0.342	34.871
Bus	0.402	0.0136 (3.867)	0.238	14.954
Truck	0.359	0.0191 (4.644)	0.334	21.570
Reference Vehicle	Subject Vehicle – VAN			
Car	0.393	0.0135 (4.122)	0.221	16.991
Jeep	0.376	0.0163 (4.984)	0.319	24.843
Van	0.402	0.0138 (5.611)	0.341	31.482
Two wheeler	0.349	0.0114 (5.118)	0.353	26.198
Auto	0.373	0.0142 (4.404)	0.297	19.398
Bus	0.478	0.0102 (4.350)	0.296	18.923
Truck	0.465	0.0110 (4.359)	0.279	18.997
Reference Vehicle	Subject Vehicle – TWO WHEELER			
Car	0.348	0.0156 (3.935)	0.260	15.487
Jeep	0.322	0.0164 (3.132)	0.395	9.806
Van	0.385	0.0147 (4.507)	0.270	20.317
Two wheeler	0.288	0.0137 (4.032)	0.092	16.257
Auto	0.386	0.0118 (4.268)	0.082	18.213
Bus	0.422	0.0123 (2.725)	0.93	7.425
Truck	0.415	0.0110 (3.379)	0.164	3.379
Reference Vehicle	Subject Vehicle – AUTO			
Car	0.344	0.0103 (4.826)	0.290	23.291
Jeep	0.335	0.0101 (5.385)	0.372	28.997
Van	0.321	0.0112 (4.895)	0.356	24.881
Two wheeler	0.296	0.0149 (6.105)	0.246	37.272
Auto	0.319	0.0099 (4.539)	0.220	20.604

Bus	0.402	0.0082 (4.187)	0.184	17.532
Truck	0.422	0.0099 (3.613)	0.189	13.504
Reference Vehicle	Subject Vehicle – BUS			
Car	0.279	0.0199(3.123)	0.127	9.754
Jeep	0.248	0.0178 (3.679)	0.166	13.357
Van	0.287	0.0181 (3.037)	0.180	9.224
Two wheeler	0.23	0.0220 (4.376)	0.237	19.150
Auto	0.465	0.0111 (3.737)	0.103	13.962
Bus	0.441	0.0114 (4.387)	0.179	19.197
Truck	0.487	0.0094 (3.314)	0.189	10.984
Reference Vehicle	Subject Vehicle - TRUCK			
Car	0.296	0.0156 (4.392)	0.310	19.268
Jeep	0.305	0.0147 (3.090)	0.154	9.550
Van	0.286	0.0152 (4.904)	0.343	24.049
Two wheeler	0.204	0.0220 (4.730)	0.301	22.371
Auto	0.417	0.0139 (4.359)	0.252	19.008
Bus	0.379	0.0144 (4.464)	0.317	19.390
Truck	0.409	0.0127 (4.475)	0.308	20.029

Table 4.9 Parameters of Regression Models for Right Clearances

	a_0	a_1 (t-test value)	R^2 – value	F – test value
Reference Vehicle	Subject Vehicle - CAR			
Car	0.554	0.0156 (3.401)	0.134	11.565
Jeep	0.438	0.0063 (3.587)	0.230	12.863
Van	0.466	0.0069 (4.492)	0.249	20.176
Two wheeler	0.382	0.0072 (4.656)	0.265	21.682
Auto	0.405	0.0062 (3.920)	0.273	15.363
Bus	0.522	0.0091 (4.272)	0.239	18.253
Truck	0.473	0.0089 (3.575)	0.229	12.779
Reference Vehicle	Subject Vehicle – JEEP			
Car	0.471	0.0154 (4.471)	0.270	19.989
Jeep	0.455	0.0081 (3.983)	0.206	15.867
Van	0.413	0.0082 (3.633)	0.258	13.198
Two wheeler	0.402	0.0076 (3.930)	0.251	15.444

Auto	0.394	0.0094 (4.157)	0.318	17.285
Bus	0.470	0.0095 (3.558)	0.177	12.662
Truck	0.465	0.0097 (3.007)	0.175	9.040
Reference Vehicle	Subject Vehicle – VAN			
Car	0.530	0.0139 (2.616)	0.13	6.845
Jeep	0.411	0.0066 (4.439)	0.267	19.702
Van	0.399	0.0694 (4.171)	0.262	17.398
Two wheeler	0.368	0.0704 (4.461)	0.273	19.927
Auto	0.483	0.0085 (4.620)	0.203	15.340
Bus	0.511	0.0076 (3.761)	0.221	14.441
Truck	0.445	0.0099 (3.360)	0.197	11.289
Reference Vehicle	Subject Vehicle – TWO WHEELER			
Car	0.319	0.0073 (4.057)	0.212	16.456
Jeep	0.425	0.0095 (4.417)	0.245	19.503
Van	0.383	0.0077 (4.538)	0.241	20.596
Two wheeler	0.308	0.0099 (4.800)	0.245	23.044
Auto	0.311	0.0078 (5.108)	0.202	26.089
Bus	0.410	0.0089 (4.333)	0.273	18.773
Truck	0.412	0.0080 (3.920)	0.228	15.369
Reference Vehicle	Subject Vehicle – AUTO			
Car	0.336	0.0069 (4.621)	0.236	21.357
Jeep	0.381	0.0075 (4.600)	0.248	21.156
Van	0.354	0.0119 (5.157)	0.322	26.594
Two wheeler	0.401	0.0061 (3.091)	0.180	19.554
Auto	0.388	0.0084 (4.219)	0.175	19.803
Bus	0.499	0.0062 (3.689)	0.175	13.608
Truck	0.500	0.0078 (3.460)	0.210	11.974
Reference Vehicle	Subject Vehicle – BUS			
Car	0.437	0.0085 (5.046)	0.233	25.460
Jeep	0.513	0.0047 (3.417)	0.206	11.617
Van	0.462	0.0080 (4.733)	0.266	22.784
Two wheeler	0.412	0.0079 (3.967)	0.179	15.733
Auto	0.441	0.0085 (4.030)	0.191	16.240
Bus	0.424	0.0097 (4.348)	0.213	18.904

Truck	0.485	0.0079 (3.618)	0.233	13.091
Reference Vehicle	Subject Vehicle – TRUCK			
Car	0.457	0.0059 (3.613)	0.198	3.613
Jeep	0.445	0.0041 (2.928)	0.193	13.574
Van	0.451	0.0049 (3.011)	0.211	11.541
Two wheeler	0.406	0.0072 (4.346)	0.243	18.888
Auto	0.429	0.0078 (3.762)	0.217	14.149
Bus	0.495	0.0093 (3.402)	0.205	11.577
Truck	0.465	0.0075 (3.232)	0.195	10.499

The low R^2 values may be due to the effect of driver behaviour, which the linear regression models are not able to capture. Another point to be noted is that individual values are used instead of average values. The values of the coefficient of speed variable in the equations developed for left and right clearances are very small indicating that effect of speed on lateral clearances is marginal. This may be correct as the vehicles, if aligned properly, accept smaller lateral clearances. That is a vehicle whose path is straight will accept smaller clearance, whereas a vehicle that has to travel in curved path can accept only larger clearances. If the clearance available is less than required, then the driver will slow down such that the available clearance can be made use of for safe manoeuvring of vehicle.

4.2.3 Models Reflecting Driver Behaviour

Analysis of left/right clearances for each speed class for each pair of vehicles, as done in case of frontal spacings, revealed that the lateral clearances also follow normal distribution for a given speed range and for a particular pair of vehicle classes. A summary of the mean, standard deviation and coefficient variation values calculated for various speed ranges of different pairs of vehicle classes are given in Table 4.10. The mean and standard deviation values were smaller compared to those of frontal spacings. It can be observed from the table that the mean and median values of coefficient of variation are almost same as those for frontal spacings. Hence, the coefficient of variation value was taken as 0.250 for estimating the value of term representing the driver behaviour.

Table 4.10 Summary of Mean, Standard Deviation and COV values

Parameter	Mean	Standard Deviation	Coefficient of Variation
Minimum	0.463	0.071	0.051
Maximum	1.400	0.432	0.879
Mean	0.871	0.222	0.262
Median	0.932	0.228	0.257

4.3 LATERAL PLACEMENT OF VEHICLES

Lateral placement (position) is position of vehicle across the width of the carriageway. Lateral placement information is useful for arriving at geometric design standards, safety evaluation, evaluation of pavement strength characteristics, etc. Models of lateral placement of vehicles form one of the components of traffic flow simulation models.

4.3.1 Literature Review

Evens (1950) stated that the transverse position of a vehicle on the highway when the driver is not affected by other traffic was influenced by the speed of travel than by any other characteristic. Summala and Marisalo (1978) reported the mean distance of the axis of passenger cars from the centre line as a function of speed and driving conditions. With the increasing speed the position of the car moves towards the centre line and the shift towards the centre line is more for night time driving than daytime driving. Miller and Steuart (1982) observed lateral placement of vehicles depends on speed, lane width and size of vehicle. Nagarai *et. al.* (1990) found the average spacing on the left to increase with increase in speed up to some value and then decrease. Badarinath (1993) observed that the traffic stream dispersal across the road for various classes of vehicles could be modelled by convolution theorem. The marginal differences between the field observations and the theoretical predictions were attributed to the discrete distribution used in convolution theorem. Kuncheria (1995) used ratio of distance from the edge of the road to the centre of the vehicle and the width of carriageway to model the free lateral position of vehicles. He observed that the free lateral positions of vehicles follow normal distribution and the distance from the edge of the road to the centre of the vehicle to decrease with increase in volume.

The position occupied by a vehicle across the width of the road depends on the speed, the type of vehicle, the volume and the width of road. Normally, a vehicle which is not under the influence of any other vehicle, i.e., a vehicle under free flow condition occupies a particular position across the road. This position shifts away from the edge of road with the increase in speed for safety reasons. The vehicles shift from this free lateral position in the presence of adjacent vehicles for safety reasons. Sometimes the vehicles move up to the edge of road to give way for overtaking vehicles. For simulation modelling of traffic flow, models that describe the lateral placement of vehicles under free flow condition are required. The position of vehicles in the presence of adjacent vehicles depends on the clearances to be maintained with respect to other vehicles and the edge of the road.

4.3.2 Data Collection and Analysis

In the present study, models ware developed for free lateral position of different types of vehicles. The data required for this purpose was collected on four urban mid blocks of varying widths. In order to eliminate the external influences on the lateral placement, tangent sections with minimum grade and with no barriers were selected. For the purpose of obtaining the speeds of vehicles a trap of 30 m length was marked on the section and for the purpose of lateral position of vehicles marks were made across the width of the road at 0.25 m interval. The data was collected by manual method with one person noting down the lateral position of vehicle and another person recording the time taken by the vehicle to cover the trap length. The average and standard deviations of lateral placement of different types of vehicles are given in Table 4.11. The lateral placement is taken as the distance from the edge of the roadway to the left edge of the vehicle in case of four wheelers, while in case of two wheelers and three wheelers the lateral placement is taken from the left edge of the road to the centre of vehicle. It can be noticed that average values vary depending on the type of vehicle. Also, in majority of the cases, the lateral placement was found to increase with increase in road width. Another observation was an increase in the standard deviation of the distances maintained by any class of vehicles from the edge of the road with increase in road width. This indicates that more the freedom given, more will be the variation. If road width is less, then the driver will have less choice regarding the placement of vehicle. This indicates that vehicle placement depends on driver characteristics.

Plots were prepared for lateral placement of vehicles in terms of speed for each class of vehicles and road width. The plots of lateral placement in terms of speed for two wheelers for various road widths are shown in Figs 4.7 to 4.10.

Table 4.11 Mean & Standard Deviation (SD) of Lateral Placement of Vehicles

Type of Vehicle	Location 1 (width-4.00 m)		Location 2 (width-5.00 m)		Location 3 (width-6.50 m)		Location 4 (width-7.50 m)	
	Mean	SD	Mean	SD	Mean	SD	Mean	SD
Car	1.053	0.212	2.35	0.301	3.01	0.379	3.30	0.894
Jeep	1.116	0.347	2.65	0.241	3.00	0.520	3.57	0.772
Van	1.098	0.216	2.45	0.341	2.94	0.460	3.41	0.810
Two wheeler	1.794	0.398	2.49	0.54	2.65	0.906	2.61	1.080
Auto	1.800	0.299	2.40	0.45	2.69	0.804	2.65	1.061
Bus	0.831	0.212	1.71	0.252	2.20	0.332	2.69	0.633
Truck	0.623	0.189	1.64	0.281	2.20	0.352	2.90	0.550
Mini Truck	0.870	0.220	1.81	0.229	2.53	0.381	3.82	0.503
Mini Bus	0.915	0.276	1.87	0.262	2.25	0.341	2.91	0.675

The correlation values between lateral placement and the causal variables, speed and road width, for different classes of vehicles are given in Table 4.12.

Table 4.12 Correlation Coefficient Values between Free Lateral Placement & Causal Variables

Type of Vehicle	Speed	Width of road
Car	0.637	0.708
Jeep	0.693	0.709
Van	0.543	0.623
Two wheeler	0.362	0.158
Auto	0.558	0.628
Bus	0.719	0.849
Truck	0.758	0.837
Mini Truck	0.859	0.782
Mini Bus	0.715	0.853

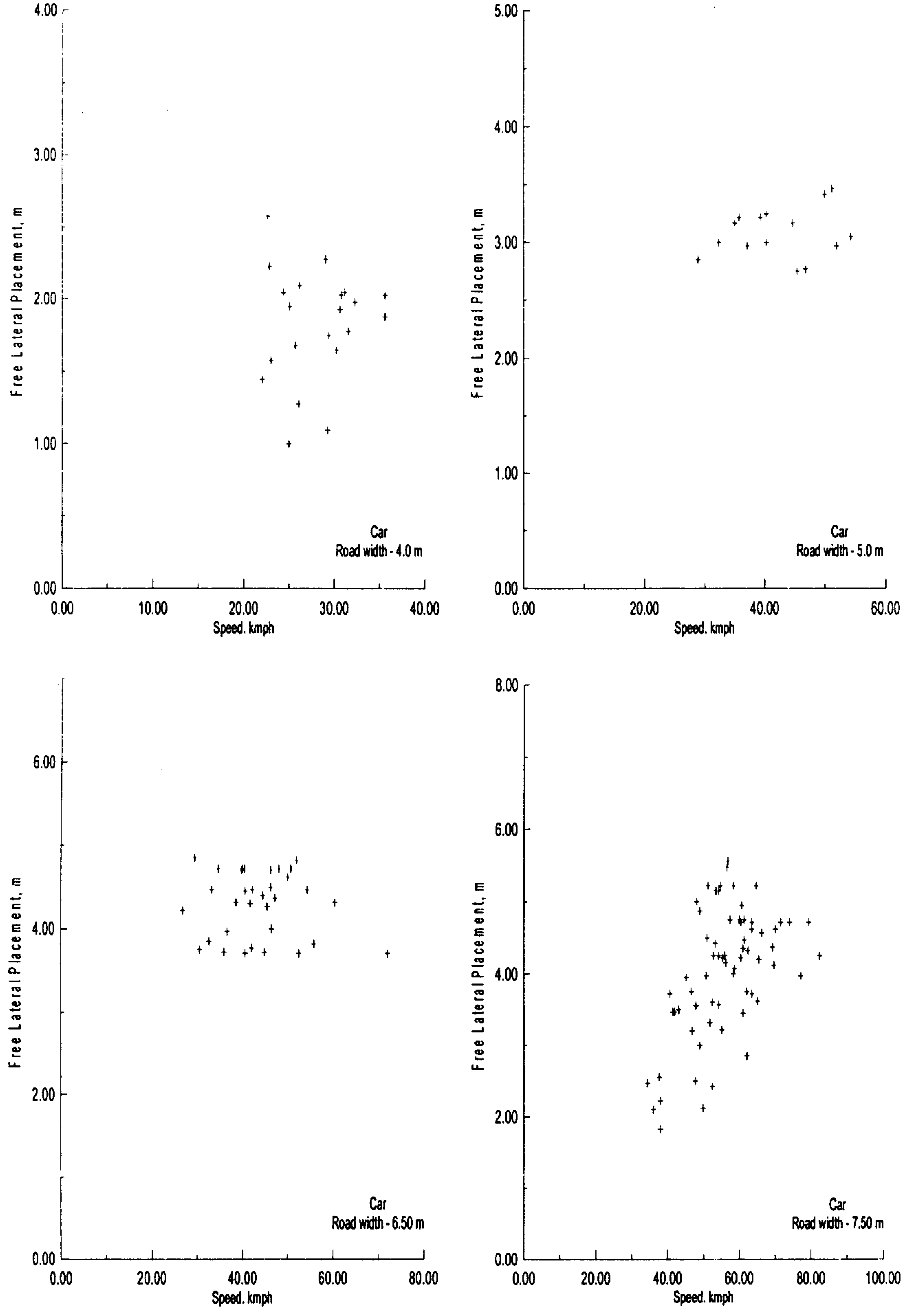


Fig 4.7 Free Lateral Placement of Car

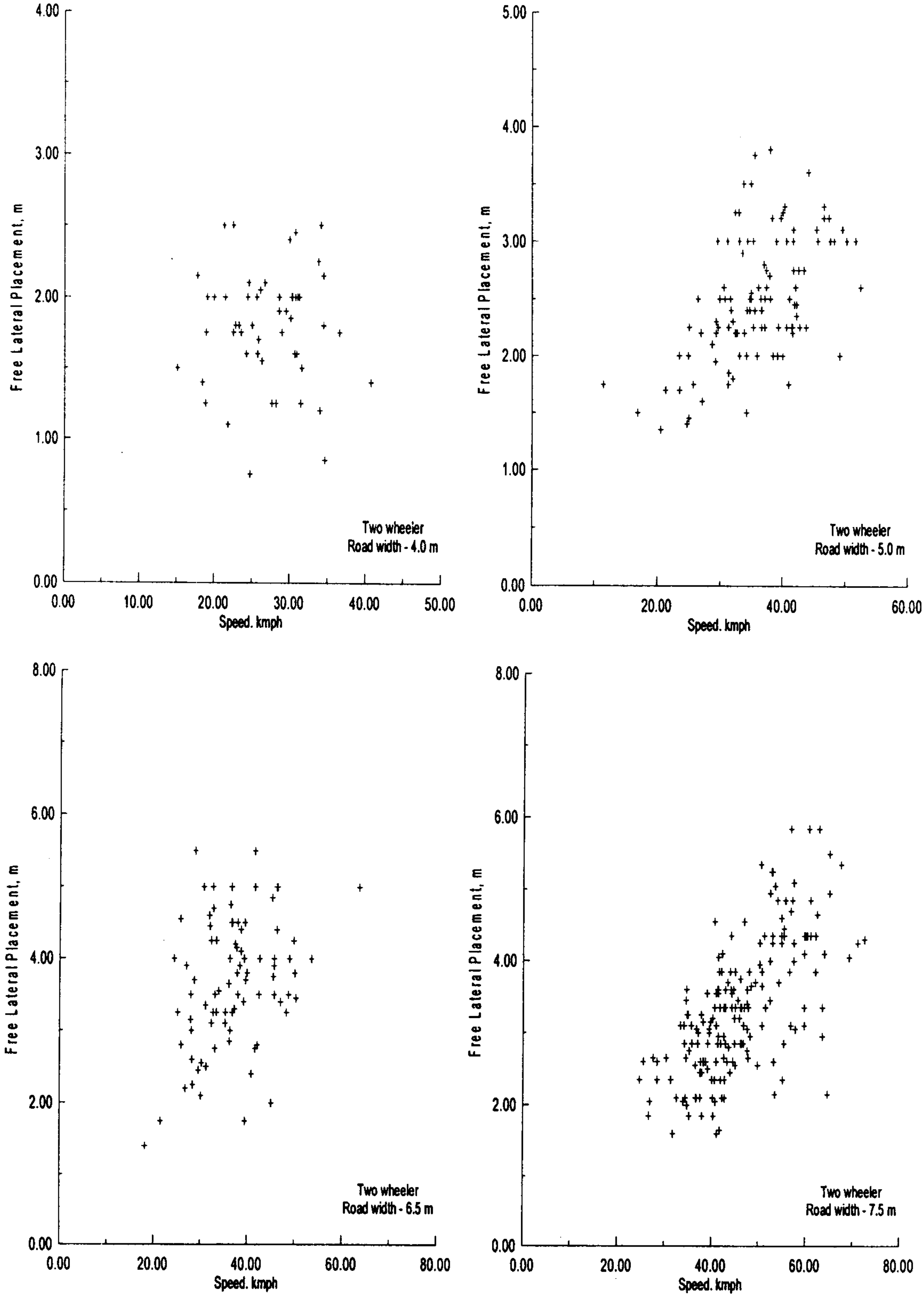


Fig 4.8 Free Lateral Placement of Two-wheeler

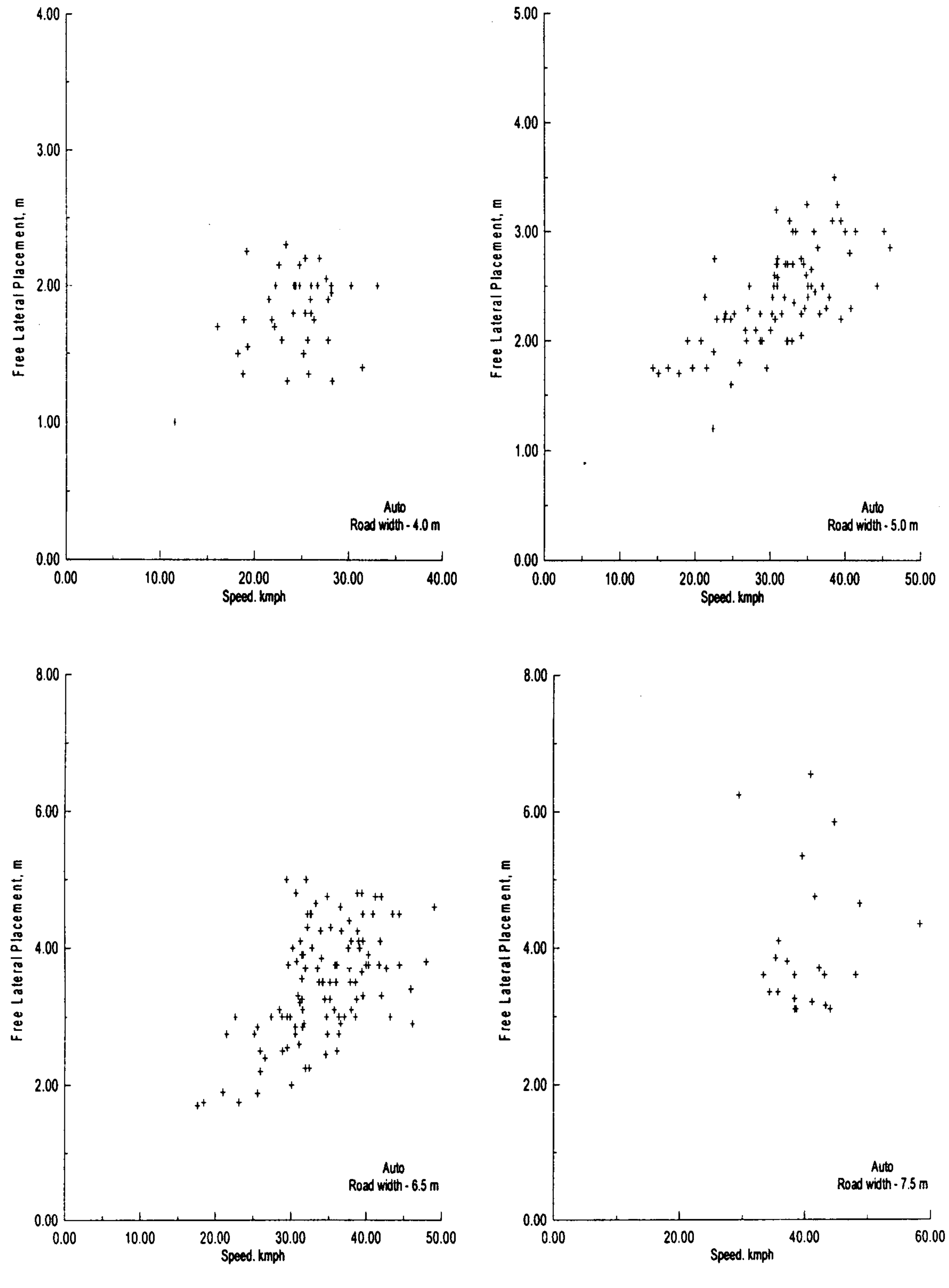


Fig 4.9 Free Lateral Placement of Auto-rickshaw

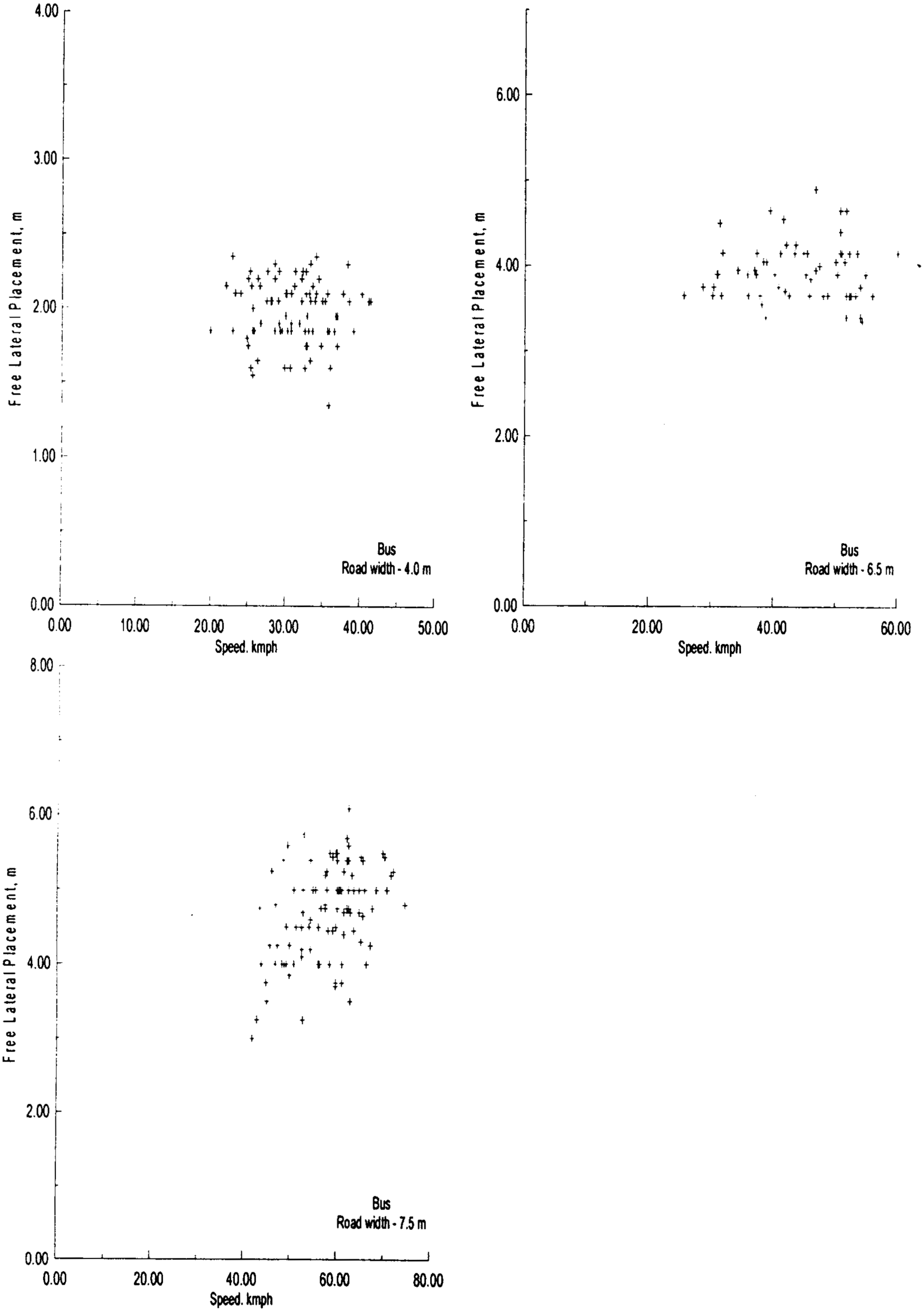


Fig 4.10 Free Lateral Placement of Bus

4.3.3 Regression Models of Free Lateral Placement of Vehicles

From the above results and discussion it can be said that the lateral placement of vehicle depends on the type of vehicle, speed of vehicle, the road width and the driver behaviour. Hence, models were developed for lateral placement of vehicles in terms of speed and road width. The models finally selected are of the following form:

$$flp = a_0 + a_1 * Speed + a_2 * Road\ width$$

The value of the coefficients along with sign and other relevant statistics are given in the following Table 4.13.

Table 4.13 Results of Regression Analysis of Free Lateral Placement of Vehicles

Type of Vehicle	a_0	a_1 (t-test values)	a_2 (t-test value)	R^2 - value
Car	-0.509	0.0260 (5.010)	0.313 (5.274)	0.362
Jeep	3.009	0.0211 (4.657)	0.238 (3.536)	0.289
Van	0.145	0.0211 (2.216)	0.238 (2.548)	0.271
Two wheeler	0.855	0.0334 (6.601)	0.0545 (1.65)	0.229
Auto	1.779	0.0431 (11.452)	-0.144 (6.26)	0.294
Bus	-0.992	0.027 (7.307)	0.299 (8.831)	0.540
Truck	-0.600	0.0227 (2.686)	0.283 (3.859)	0.435
Mini Truck	-0.659	0.0235 (2.452)	0.248 (4.532)	0.395
Mini Bus	-0.895	0.0315 (3.524)	0.313 (4.115)	0.412

4.4 HEADWAY DISTRIBUTION

The arrival process consists of the generation of arrival times of the vehicles at the entry to the simulated stretch. The arrivals are generally simulated by sampling from specified headway distributions. Headway is defined as the time that elapses between consecutive vehicles. The distribution of these headways has been studied, with the primary interest being application to capacity estimation, safety analysis, and generation of vehicles in microscopic simulations.

It is well established that the headways in free flow conditions could be described by the exponential or the shifted exponential distribution but it is not the same in platoon flow conditions. Several models have been used by the researchers for describing the distribution of headways—for example, semi-Poisson model, Cowan's M3 model, log-normal distribution, and double displaced negative exponential distribution (Sullivan and Troutbeck 1994), Erlang distribution and gamma distribution (Al-Ghamdi, 2001). Al-Ghamdi (2001) observed negative exponential, shifted exponential, and gamma distributions to reasonably fit the low and medium states of flow on freeways, the Erlang distribution to properly fit the high traffic flow state. On the other hand, the gamma distribution gave a decent fit for a large range of flows on arterials (around 600–1,200 vph). Al-Ghamdi (2001) also established boundaries for the three states of flow based on the distribution families that fit the three groups of flow on freeways. The flow ranges for low flow state, medium flow state and congested state are less than 400 vph, 400–1,200 vph and above 1,200 vph respectively.

Researchers have used different distributions depending on the flow levels, which appear to have been derived from empirical observations. Based on the preceding discussion and as tabulated in Table 2.1, it can be observed that no specific distribution/s will be able to describe the headways over all the ranges of flows in mixed traffic. Also, this approach may be correct in homogeneous traffic conditions, where, the vehicles strictly follow the lane discipline. In mixed traffic conditions, where, the vehicles can travel side by side in same lane, the headways depend on several factors, like, the flow level, the width of lane, the traffic composition, etc. This calls for the need to adopt a different methodology for describing the vehicle arrival process.

Another approach for generation of vehicle arrivals at the entry of the study section is warm-up zone approach, in which suitable lengths of hypothetical road sections are added before the simulated stretch. The vehicle arrivals at the entry to the warm-up zone can be generated by sampling from any headway model, generally the exponential distribution. The vehicles thus generated are moved through the warm-up zone as per the flow logic of the model.

This method is a sound means of generating the vehicle arrival process, provided the flow reaches an appropriate state of statistical equilibrium by the time the vehicles arrive at the entry point of simulated stretch. Experiments have to be conducted to determine the length of warm-up zone and warm-up time required for the system to stabilise.

4.5 FREE FLOW SPEEDS

Free flow speed is the speed at very low densities and flows. Free flow speed is influenced by factors such as the design speed of section or roadway, the frequency of intersections, general density of development and complexity of driving environment and speed limits. Desired speed is defined as the speed at which the drivers would choose to travel under prevailing conditions if unimpeded by other vehicles or roadway constraints. Thus, desired speeds vary depending on the flow conditions. The free flow speed information is useful for a variety of applications like traffic flow research, assessment of speed limit compliance, checking the design speed assumptions and routine monitoring of speed trends, The free flow speeds provide information about how dangerous drivers consider a certain road stretch to be. Free flow speeds are generally used in traffic flow simulation studies. In simulation modelling, the generated vehicles are assigned the free speeds by sampling from speed distributions.

4.5.1 Literature Review

The free flow speeds depend on the driver behaviour and type of vehicle. Several stochastic models were used by researchers to describe the free speeds of vehicles, among which, normal distribution is the most common. Even in mixed traffic also many researchers like Ramanayya (1980), Sarna et al (1988) Kadiyali *et al* (1981 & 1991) have reported that free flow speeds of vehicles follow *Normal* distribution. Kuncheria (1995) tried normal and log-normal distributions to describe free flow speeds of different vehicles in mixed traffic and concluded that normal distribution describes better the free flow speeds.

4.5.2 Data Collection and Analysis

The parameters of distribution (mean and variance) depend on the characteristics of the traffic stream under study. Also, as the simulation model has to be validated with field data, it is necessary to conduct field studies and collect the data of free speeds of different classes of vehicles. In the present study, field surveys were conducted at four typical urban mid blocks to obtain the free speeds of different vehicles. The selected sections were level, straight, of uniform width and free from influence of intersections and bus stops. The free speeds of the vehicles were collected by manual methods during early morning hours when the flows were very less.

To arrive at the probability distribution model for describing the free speeds of vehicles, the frequency distribution plots of observed plots were prepared and compared with that of standard distributions. The observed frequency distribution plots were found to be very close to that of *Normal* distribution. So the normal distribution was selected for explaining the free speeds of vehicles and Chi-square test was conducted to check the goodness of fit at 5% significance level. The results of chi-square test, which indicated that *Normal* distribution was a good fit in all the cases, are presented in Table 4.14. It can also be observed that the free speeds of cars are the highest and that of auto-rickshaws are the lowest.

Table 4.14 Results of Goodness of Fit Test for Free Speed Data

S. No.	Type of Vehicle	Mean Speed	Standard Deviation	Sample Size	χ^2_{cal}- value	χ^2_{tab}- value
1	Car	58.24	5.41	305	3.55	5.99
2	Jeep	54.31	4.83	268	2.86	5.99
3	Van	53.07	6.94	214	2.62	3.84
4	T-W	45.75	6.05	346	3.98	6.82
5	Auto	42. 23	5.58	301	2.11	7.89
6	Bus	57.40	6.96	308	1.92	8.99
7	Truck	52.43	5.69	244	7.23	7.78

The free flow speeds of the vehicles in the simulation were determined by sampling from *Normal* distribution for the specified mean and standard deviation values of the free speeds of different classes of vehicles. The normal variates were generated by a procedure based on central limit theorem. Separate seed values were used to generate random number generators for assigning the speeds of vehicles of different classes.

4.6 THRESHOLD HEADWAY FOR FREE FLOWING VEHICLES

Drivers drive their vehicles at free flow speed or accelerate to free speed when there is no other vehicle in front of them within sufficient distance. In other words free flow speed is the speed of vehicle when the vehicle is not under the influence of any other vehicle. In simulation model it is necessary to assess whether a vehicle is constrained or not, so that it can be driven at the proper speed. Headway is generally used to distinguish free flowing and following vehicles. Unfortunately, there is little agreement in the literature about the point at which vehicles travelling in the same direction influence each other, and little consistency in the approaches adopted to discover it. The point where additional headway does not further increase the speed would thus represent the threshold that distinguishes following vehicles from free vehicles.

4.6.1 Literature Review

Evans and Wasielewski (1983), on the basis of a mathematical model of headway distributions on freeways proposed 2.5 sec time headway as the upper limit for interacting vehicles. Pasanen and Salmivaara (1993), employing an alternative approach, suggested that 3 sec should be used as the threshold for determining whether a vehicle was "free", but provide no theoretical foundation for this value. Highway Capacity Manual (HCM, 2000) has suggested a threshold headway of 3 sec. Vogel (2002) based on analysis of headway and speed data of more than 1,00,000 vehicles concluded that a temporal separation of 6 sec as the best criterion for separating free and interacting vehicles.

4.6.2 Data Collection and Analysis

To arrive at the threshold headway for determining whether a vehicle is free from the influence of front vehicle or not, headway and speed data of vehicles which are not moving side by side were collected using the video tapes recorded at different locations in Calicut city. The data was classified based on headway. For each headway class the cumulative percent frequency distribution of speeds was obtained and the diagrams was prepared. The cumulative percent frequency distribution diagram for 1.5 sec headway class is shown in Fig 4.11. From the cumulative percent frequency distribution plots the 85th and 98th percentile values were determined. These 85th and 98th percentile speeds for headway class were plotted as shown in Fig4.12. It can be observed that at a headway of 3 sec, both the 85th and 98th percentile speeds achieve maximum value and even if headway increases further, the increase in speed is very marginal. Hence, a headway of 3 sec was taken as threshold for determining whether a vehicle is freely moving or constrained.

4.7 RATE OF LATERAL MOVEMENT

Drivers steer their vehicles across the width to overtake another vehicle or to take the desired position across the width of the road. The drivers steer the vehicle in such a way that no collision takes place. The rate of lateral movement depends on many factors such as the clearances available, turning radius, forward speed and driver behaviour. An attempt has been made in this study to determine the rate of lateral movement for different vehicles.

4.7.1 Data Collection and Analysis

The required information was collected at locations where the vehicles will be forced to change their path because of the presence of some obstruction. Grid markings were made along and across the road at intervals of 10 m and 0.25 m respectively. The time of passing and the position occupied across the road at each ten metre mark were noted manually. For this purpose, observers were positioned at each of ten meter marks. The collected data was analysed to obtain the rate of lateral movement of different classes of vehicles.

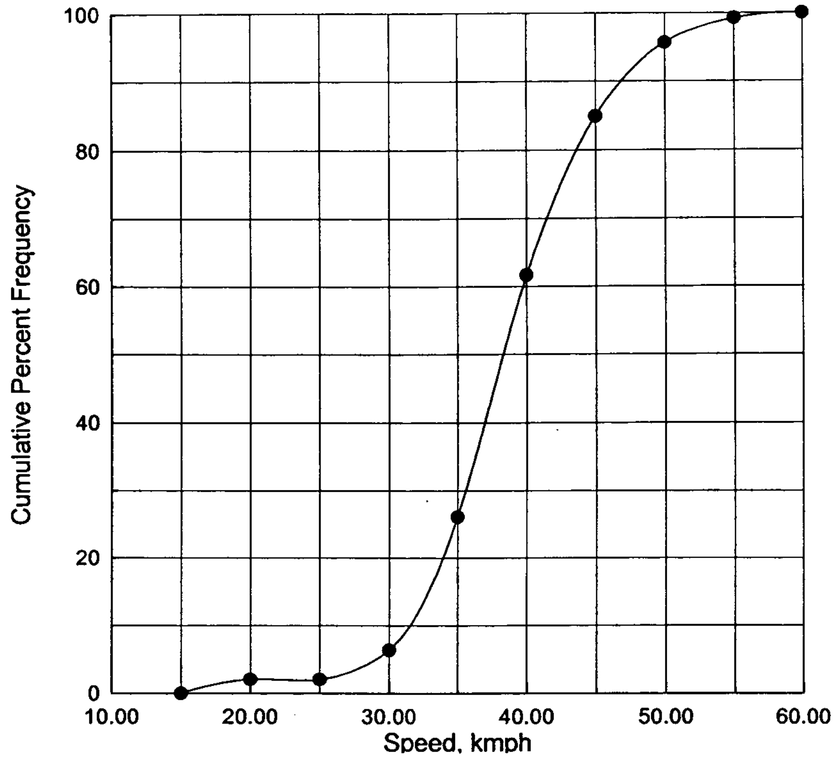


Fig 4.11 Cumulative Percent Distribution of Speeds for 1.5 sec Headway

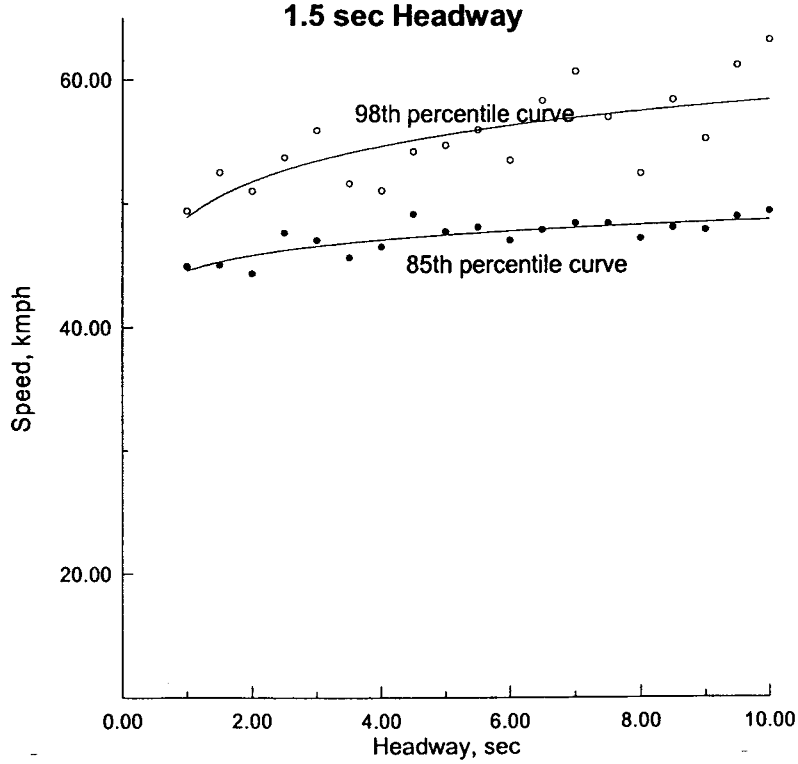


Fig 4.12 Speed Vs Average Headway

The maximum and average values of lateral speeds are given in Table 4.15. The values are based on limited observations. In simulation model, the rate at which a vehicle is to be moved laterally is determined based on the frontal clearances, distance to be moved and time available for reaching the target position. These values are limited to the maximum values observed in the field. The average values are used to check whether sufficient clearances are available to safely manoeuvre the vehicle to the required position.

Table 4.15 Rate of Lateral Movement for Various Classes of Vehicles

Type of Vehicle	Average value, m/sec	Maximum value, m/sec
Car	0.648	0.806
Jeep	0.882	1.451
Van	0.630	0.756
Two wheeler	0.656	0.728
Auto	0.703	1.228
Bus	0.594	1.010
Truck	0.537	1.048
Mini Truck	0.724	1.450
Mini Bus	0.642	1.213

4.8 CONCLUSIONS

Development of models to represent some of the subtasks based on real life data has been the subject matter of this chapter. Regression analysis of frontal spacings revealed that the frontal spacings depend on the type of front vehicle, subject vehicle and the speed of subject vehicle. Performance of neural models was found to be better than that of regression models in case of frontal spacings. Speed of the subject vehicle emerged as main influencing factor in case of clearances maintained on left and right. The deviations of individual drivers from mean values were found to follow normal distribution for both linear and lateral clearances.

The lateral placement of vehicle under free flow conditions was found to be influenced by the speed of vehicle and the width of road. Both the factors were found to have positive correlation with free lateral placement of vehicles. Free flow speeds of different classes of vehicles were found to follow normal distribution. Threshold headway for defining a free flowing or following vehicle was also estimated.

CHAPTER – 5

DEVELOPMENT OF CAR FOLLOWING MODELS

5.0 GENERAL

In traffic stream a vehicle may be forced to follow another vehicle for many reasons. At the same time, drivers will be looking for opportunities to overtake the vehicles ahead of them. In order to avoid collision in such kind of overtaking manoeuvres, the vehicles will be forced to follow the lead vehicle. Thus, in order to study traffic flow characteristics it is necessary to model both these car following and acceleration/deceleration characteristics of vehicles. Thus, these two manoeuvres are decided to be studied and these are presented in two separate chapters. The current chapter deals with the development of car following models, while, the chapter to follow is reserved for discussion on modelling acceleration/deceleration characteristics of vehicles in mixed traffic environment.

Car following models, which describe the behaviour of drivers reacting to the behaviour of their leaders, is another important component of traffic flow simulation model. In conventional methods, the driver behaviour is studied as response in the form of acceleration/deceleration of the following driver using response-stimuli equations. These models are deterministic and thus do not fully represent the imprecise driver reaction. Neural networks, which have a performance close to human behaviour, are likely to represent the process better than purely mechanical systems. This chapter presents the efforts made to study the car-following behaviour of drivers using conventional response-stimuli method and neural network. A comparison of the two methods is also presented along with review of literature, data collection and analysis and modelling.

5.1 CAR FOLLOWING MODELS

The principal goal of a driver is to guide the vehicle from origin to destination in a safe manner, along with additional goals such as arriving at destination at the earliest possible time etc. The task of accomplishing the driver's goals can be split into different categories of action such as perception, judgment, decision and control. The driver's control actions are limited to the control of heading (steering) and control of acceleration. To accomplish the steering function, the driver attempts to maintain the vehicle within the error criterion limits assessed based on experience, throughout the duration of the steering job. The acceleration control subtask consists of detecting the differences in velocity or spacing with adjacent vehicles and taking actions that will prevent unsafe conditions and fulfil the driver's goal of proceeding at a particular speed. The physical action that a driver performs after perceiving and judging the situations is to accelerate, decelerate or move at the same speed. Thus, in driver behaviour studies, the response or the output parameter is considered as acceleration or deceleration.

As the vehicle moves on the roadway, the driver gathers information about the surrounding environment required for performing the different vehicle manoeuvres. These information act as the stimuli for the decision making process of the driver. The driver positions the vehicle in a traffic stream such that safe clearances are maintained with respect to the surrounding vehicles or objects and drives the vehicle at a speed perceived as safe. If the available safe clearances are less than the required safe clearances, then he decelerates. Otherwise, accelerates to the desired speed. In most of those studies, the driver behaviour in a single lane traffic stream was modelled by examining the manner in which the individual vehicles followed one another. The most pertinent car-following theories in the chronological order of their development are briefly discussed in the following articles.

5.2 LITERATURE REVIEW

Pipes (1953) characterised the motion of vehicles in traffic stream, following the rule suggested in the California Motor Vehicle Code, that for following another vehicle at safe distance is allow to yourself at least the length of a car between your vehicle and the vehicle ahead for every 10 miles per hour of speed at which you are travelling. He described the distance headway as a function of speed and derived the car following model. The limitation of the model is that the compatibility between the predicted values and the actual field measurements exists only in the midranges of speeds. However, considering the simplicity of models, the close agreement with real-life observations is astonishing (Pignataro (1973)).

Forbes et al (1958) approached car following behaviour of driver by considering the reaction time needed for the following vehicle to perceive the need to decelerate and apply the brakes. According to this theory, the minimum time headway is considered to be equal to the reaction time and the time required for the lead vehicle to travel a distance equal to its length. Similar to Pipe's theory, there is close agreement between the values predicted by Forbes' model and the field observations in mid range speeds only.

The General Motors' models were based on the premise that the reaction of the driver of following vehicle at time 't' depends on the sensitivity of the driver of the following vehicle and the strength of the stimulus at time ($t-\tau$). In these models, the strength of the stimulus is measured in terms of the relative velocity between the lead vehicle and the following vehicle and the reaction of the following vehicle is taken as the acceleration or deceleration rate. The time difference τ, is equal to the perception reaction time and the sensitivity term (α_0) maps the unit of a stimulus to a reaction.

The researchers at General Motors developed five models, known as GM Models, that have the same general structure, but differ from one another with respect to the sensitivity term. The third generation model proposed by Gazis et al (1961) is the most widely used as it is simple and more realistic (Gipps, 1981). It is represented as follows:

$$a_n(t+\tau) = \alpha_0 \left[\frac{v_{n-1}(t) - v_n(t)}{x_{n-1}(t) - x_n(t)} \right] \qquad \text{Eqn. 5.1}$$

where

$a_n(t+\tau)$ - acceleration/deceleration of the following vehicle at time $t+\tau$

α_0 - Sensitivity parameter

$v_{n-1}(t) - v_n(t)$ - Relative speed between the two vehicles at time t.

$X_{n-1}(t) - X_n(t)$ - Distance between the two vehicles at time t.

The drawbacks of these models are the following:

- The interaction between stimulus and reaction has a one-to-one correspondence.
- The imprecise driver's reaction pattern is not fully represented.
- Representation of a human behavioural pattern may be better explained by an approximate reasoning process than a deterministic model.
- The following vehicle reacts even to minute changes in relative velocity between lead vehicle and the following vehicle in a deterministic manner.

Psycho-physical models describe driving in a way very close to reality. Research into a perceptual psychology has shown that drivers are subjected to certain limits on the stimuli to which they respond [Leutzbach, 1988]. The basis of such models is,

1. at large spacing, the driver of a following vehicle is not influenced by the size of speed difference.
2. at small spacing, there are combinations of relative speeds and distance headway for which there is, as in (1), no response of driver of the following vehicle because the relative motion is too small.

This model thus considers the existence of perceptual thresholds. Only when these thresholds are reached the driver of a following vehicle will be able to perceive the change in the apparent size of the lead vehicle and will be able to react to the changes in the kinematic variables. These models include substantially more realistic assumptions concerning traffic flow phenomenon. A model for traffic simulation by Fritschze (1994) based on psycho-physical modelling shows that the phase area of relative distance is differentiated into a number of thresholds defined by rules. These rules are directly incorporated in traffic simulation. But these rules can be made only if the information about the psychological evaluation of the drivers of the available spacing is known. Since such form of data is very difficult to collect, the validation of these models is difficult.

Lloyd and Gerlough (1976) attempted to simulate more significant characteristics of a car following driver–vehicle system, using control system theory techniques. The model was implemented by a computer program, which outputs time histories of driver–vehicle response to step perturbations. The major parameters considered were related to response, time perception threshold and sensitivities. But the results showed that more work was needed to make it more realistic. The characteristic of self-adaptability is lacking in this approach.

In recent years, fuzzy logic has been applied to many practical problems involving control and decisions under the environment of the imprecise human reasoning process. Kikuchi and Chakroborthy (1992) proposed a fuzzy rule based car following model that assumed that a driver's decision is the result of fuzzy reasoning process and predicted the possibilities of the reaction of the following vehicles. The predicted range was found to be reasonable. Also, it was applied to analyse the traffic stability and speed–density relationship that gave better results when compared with the deterministic models. Chakroborthy and Kikuchi (1999) compared the performance of GM models and fuzzy inference logic based models using real world data and concluded that fuzzy model possessed many desirable properties of car-following models, which were not found in GM models.

A review of the theories developed on the car following behaviour and the studies conducted reveals that, while the deterministic models do not reflect the stochastic nature of driver behaviour, attempts involving fuzzy logic, which approximates the human reasoning process, were more successful in describing the driver behaviour. Artificial Neural Networks is another branch of Artificial intelligence, which tries to mimic the functionality of brain in a fundamental manner. It maps the different relationships by being exposed to a set of examples of the behaviour concerned, by allowing the data to speak for itself.

In reality, the driver continually reviews the driving situation. As tne driver moves, his/her brain receives a stream of consecutive images representing a stream of traffic patterns, which is directly processed in the brain. The driver behaviour resulting from the input is based on a learned reaction to a particular situation described by the input and can be represented by a modelling technique that closely follows this process. Hence, the neural network approach is possibly a better solution in modelling the driver behaviour, as the network itself would develop the relationships.

5.3 DATA COLLECTION

The information required to be collected for development of car following models include type and speed of the subject vehicle, type and speed of the lead vehicle and distance between the two vehicles at successive time intervals. Video graphic method was adopted for collection of data as it offers many advantages. A road section free from interruptions such as cross roads, bus stops, etc, was selected. The video camera was set up on the top of an adjacent seven storey building and was adjusted to get a clear view for about 50 m length of the road. To facilitate easy retrieval of accurate information regarding speed, linear and lateral clearances, paint markings were made at intervals of 5 m in the longitudinal direction for 50 m length and 1m in the lateral direction on the chosen road surface. For the purpose of retrieval of required data, the video input was fed to a computer provided with a frame grabber card and software that enables to capture and digitise the frames of taped video. The sequences of captured images were then stored as AVI (Audio Video Interweaved) files. The following details were noted from each frame, making use of the grids marked on transparent sheet, which was pasted on the monitor.

i. Longitudinal position of subject vehicle and its type.

ii. Longitudinal position of the vehicle in the front and its type.

The speeds of vehicles in metres per second were determined by finding the difference in respective positions of vehicles in the subsequent frames. The determined speeds were then converted to kilometres per hour.

5.4 RESPONSE – STIMULUS MODELS

Car-following models, which are of the form of stimulus-response equation, are generally used to describe the behaviour of drivers in a traffic stream. These models give the acceleration of the vehicle depending on its relative position with respect to the lead vehicle. These models are of the form:

Response $(t + \tau)$ = sensitivity × stimulus   Eqn. 5.2

The third generation model among the models proposed by General Motor's research team was used in this study for the comparative evaluation of the performance of neural network models. Equation 5.1 gives this model.

The model was developed by the regression analysis of the observed data for which the dependent variable was the acceleration of the following vehicle in the present interval and the independent variables were the relative speed and the relative headway between the lead and the following vehicles. The values of the sensitivity parameter estimated during calibration were used along with explanatory data to predict the acceleration values. These acceleration values were then used to determine the corresponding speeds for comparing the same with that of the outputs of neural network model.

5.5 NEURAL NETWORK MODEL

The neural network models of driver behaviour with regard to speed changes were developed using the software package called MATLAB (Matrix laboratory), of Mathworks Inc. For this purpose, a partially recurrent neural network proposed by Elman (1990) was used. Elman Recurrent Networks (ERN) are two layer back propagation networks with the addition of a feedback connection from the output of hidden layer to its input. This feedback path allows Elman networks to learn, to recognise and generate temporal patterns as well as spatial patterns. Using Neural Net toolbox of MATLAB, the ERNs can be trained with momentum and adaptive learning rate.

To arrive at the appropriate network architecture, trial models were developed by varying the number of neurons in the hidden layer. It was observed, as shown in Fig. 5.1, that the performance of the neural networks improved with the increase in number of neurons up to certain level and then deceased. It was also observed that when the number of hidden neurons exceeded 100, the network showed a tendency to get entrapped in local minima and to reach premature saturation. Hence, for large sized data sets, the number of hidden neurons was limited to 50.

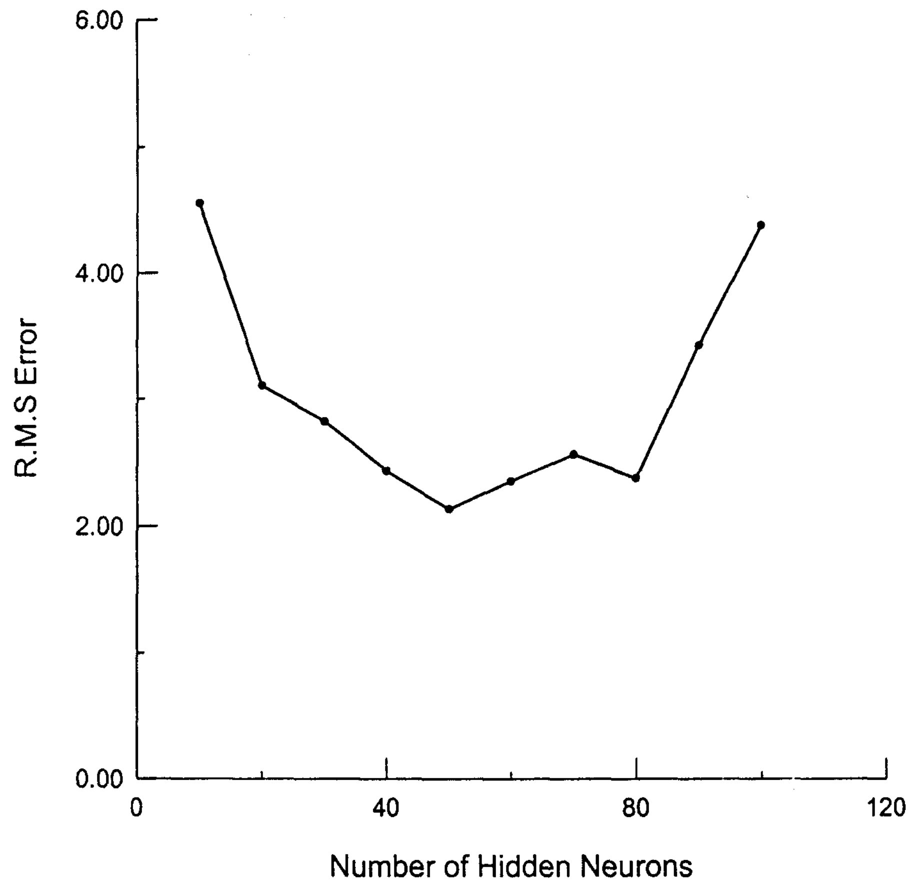


Fig 5.1 Performance Curve for Various Number of Hidden Neurons

5.5.1 Representation of Vehicles in Neural Model

Experiments were also done by representing the vehicles by simple numbers, by PCU values or by binary representation. When the type was represented in the binary form, each type representation required 1 input node, i.e. if eight modes are considered then eight nodes will be required. The network was thus unable to understand the function as the number of input nodes increased considerably and the performance was very poor. As presented in Table 5.1, when the vehicle type was represented by a simple number, the performance of models both at the time of training and testing was better than that of other representations.

Table 5.1 % R.M.S. error for different ways of representing vehicle type

Description	Training	Testing
Sample Size	85	25
All variables excluding vehicle type	0.44	75.69
All variables (Vehicle type represented by simple numbers)	0.44	28.35
All variables (Vehicle type represented by PCU values (Nagaraj et al, 1990)	0.44	32.37
All variables (Vehicle type by binary representation)	0.44	56.96

Models were developed by both neural networks and conventional car following method (GM Model), using the data for all types of vehicles. The input vector for neural models consisted of data regarding all variables in previous interval and the output was the speed of the selected vehicle in the present interval. The performance curve given in Fig. 5.2 shows the change in error with respect to the number of iterations (epochs) during training.

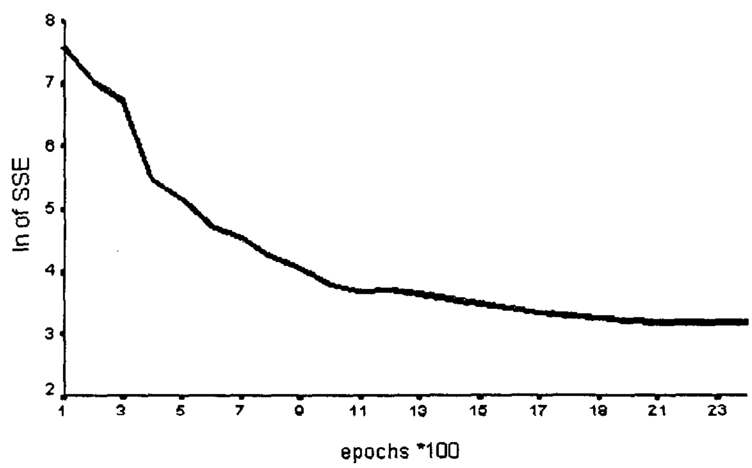


Fig. 5.2 Performance curve considering all types of vehicles during

5.6 COMPARISON OF NEURAL AND CAR FOLLOWING MODELS

The models thus developed were used to predict the speed values for the validation data set. The percentage RMS error for predicted speeds of vehicles were computed and comparison was made for the performance of both the models.

Table 5.2 gives the results obtained from neural and car-following models. It could be observed that the overall performance of neural models was better than that of car-following models. The performance of both models in predicting the speeds of each vehicle type was also compared. It was found that neural models performed better than car-following models during both training and validation stages. Thus the neural network models were found to be superior to the conventional car-following models.

Table 5.2 Comparison of Neural and Car-Following Models for All Types of Vehicles Together

Description	Neural Models	Car-Following Models
Training sample size	1166	883
Test sample size	350	289
No. of training epochs	2300	-
Sensitivity of car-following model	-	0.4018
% RMS error for training data	15.57	21.55
% RMS error for test data		
All types of vehicles combined	18.11	23.63
Auto-rickshaw	19.31	21.05
Two-wheelers	13.73	14.58
Car	16.78	15.99
Jeep	23.60	31.99
Van	23.70	28.32
Bus	23.61	32.11
Trucks	17.55	20.08

Validation of the models was also carried out by comparing the frequency distribution diagrams of predicted and observed speeds for test data. The predicted speeds obtained from neural models came, almost, in the same range as the actual speeds, whereas for car-following models, the variations were more pronounced for each category.

The experiments conducted to understand the speed changing decisions of the vehicle drivers indicated that the speed of the subject vehicle in the previous interval is the most influencing variable. However, in order to model the acceleration/deceleration characteristics, it was thought necessary to give the speed of subject vehicle for one more than previous time periods, so that the effect of previous time history could be taken care of. It is because of this experiments were repeated by giving information pertaining to the subject vehicle for previous three time intervals (interval of one second each).

Corresponding to the number of previous time period data which were supplied for modelling three models could be built. In Model 1, apart from information on other influencing variables, only the speed in the previous one second of the subject vehicle was provided. In Model 2, the speeds of the subject vehicle in the two previous time intervals were also given. Similarly, in Model 3 the speeds of the subject vehicle in the previous 3 second intervals were also supplied. Table 5.3 shows the percent RMS errors of the three models.

Table 5.3 Comparison of the three models

Description	Model 1	Model 2	Model 3
Training Data Sample	420	420	420
Test Data Sample	70	70	70
% RMS error for training data	17.91	17.91	17.91
% RMS error for test data	28.3	23.67	19.85

It could be clearly seen that model 3, which has the information on the speeds of the subject vehicle in the three previous intervals, had a lesser percent RMS error, confirming that improved driver behaviour models could be built with better information support from the past travel history. Comparison of frequency distribution diagrams of predicted and observed speeds further reinforced this idea. From Fig. 5.3 it can be observed that when the speed data of the previous three intervals are included in the input, the predictive ability of the model has improved remarkably.

However, this inference could not be generalised, because it was noticed that the models became more and more disturbed with information other than speeds in the previous few time intervals were also supplied. Perhaps, this is what could be expected from a human mind which is bombarded with lot of information, but at the same time because of its limitation it is unable to assimilate the complete information.

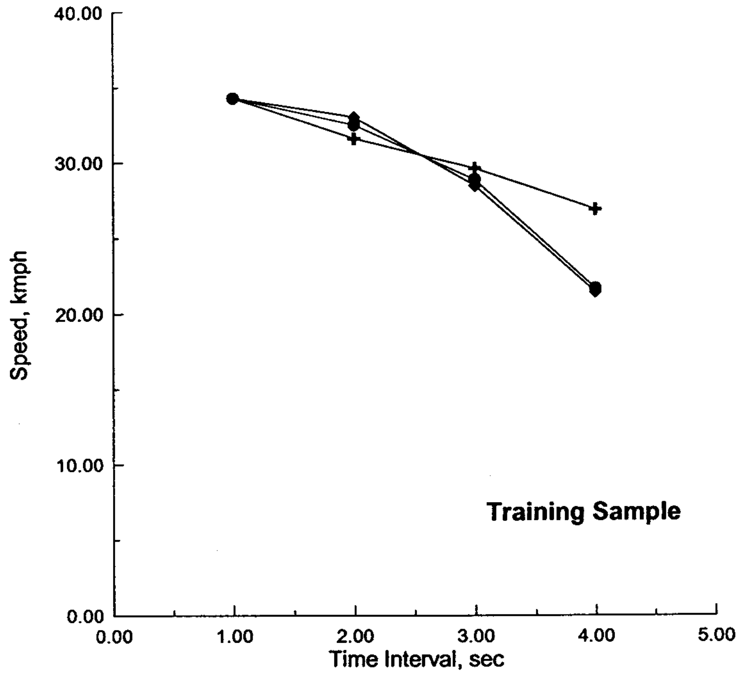


Fig 5.3a Speeds of a car at successive intervals

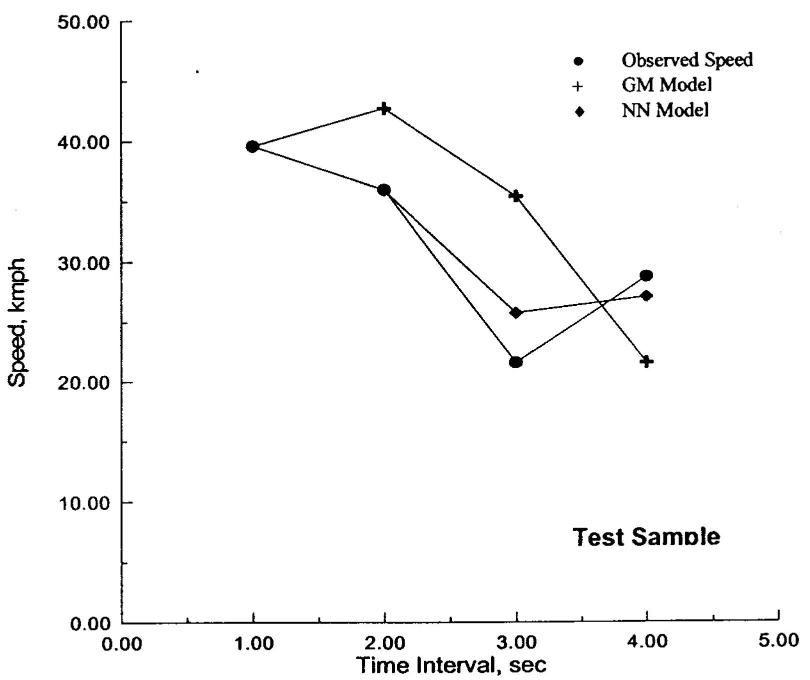


Fig 5.3b Speeds of a car at successive intervals

5.7 CONCLUSIONS

Development of car following models based on real life data and using the response-stimuli approach and the neural network technique is presented in this chapter. The neural models have proven to give more realistic results than car-following models in describing the driver behaviour while following another vehicle. The performance of neural network model improved when presented with history of speeds in previous intervals.

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CHAPTER – 6
DEVELOPMENT OF ACCELERATION/DECELERATION MODELS

6.0 GENERAL

Acceleration and deceleration operations are quite common in traffic flow. Car following models are good for describing the acceleration operations of vehicles under constrained conditions. But these models fail to describe the acceleration characteristics of unconstrained vehicles. Separate models are required to describe these operations. Development of models for describing accelerations and decelerations of various classes of vehicles in mixed traffic environment under free flow conditions is presented in this chapter.

6.1 ACCELERATION AND DECELERATION MODELS

Acceleration and deceleration operations are quite common in traffic flow. Models of acceleration/deceleration characteristics of vehicles are useful for many traffic engineering applications such as traffic flow modelling, estimation of fuel consumption and modelling of air pollution. Models of acceleration/deceleration are essential for microscopic simulation of traffic flow.

In early studies, the acceleration and deceleration values were assumed to be constant for all vehicles, for all drivers and for all ranges of speeds. Realising that the acceleration rate is governed by the vehicle transmission system, weight and horsepower, separate values were suggested for different classes of vehicles. Later, it was found that the acceleration rate varied inversely with the speed, and non-uniform acceleration models were used to describe the acceleration characteristics of vehicles. Many researchers represented the acceleration characteristics as constant values or by means of simple equations, such as linear decreasing acceleration models in their studies. Even though these approaches are very simple and easy for implementation, such simplifications are unrealistic.

In reality, the acceleration characteristics of vehicles vary widely depending on the drivers, the technological characteristics of vehicles and the weather conditions. Also, the drivers accelerate or decelerate at a rate that they presume to be safe and which causes minimum discomfort to passengers (Zero jerk). Data collected by researchers (Tomlin et. al. (1983) and Jarvis (1982)) indicated S-shaped speed profiles, which confirm to zero acceleration and zero jerk at the start and end of acceleration. Researchers have tried many models to capture the combined influence of the variables. Akcelik and Biggs (1987) have observed that polynomial models satisfy the realistic conditions of zero jerk. Even though those models represent the realistic conditions of zero jerk, they fail to reflect the driver behaviour. Hence, their suitability for microscopic simulation modelling of traffic flow is questionable.

6.2 LITERATURE REVIEW

Very few studies have been conducted so far to understand the acceleration and deceleration characteristics of vehicles in mixed traffic flow conditions. The Transportation and Traffic Engineering Handbook (1982) appears to be the only document which gives the acceleration and deceleration values of vehicles in developed countries. But those values are based on studies conducted under National Highway Cooperative Research Program (NCHRP), USA before 1971 and are given in Table 6.1.

Table 6.1 Normal Acceleration and Deceleration rates for passenger car

Speed change, kmph	Acceleration, kmph/sec	Deceleration, kmph/sec
0 – 24	5.3	8.5
24 – 48	5.3	7.3
48 – 64	5.3	5.3
64 – 80	4.2	5.3
80 – 97	3.2	5.3
97 – 113	2.1	5.3

In India, the Indian Roads Congress has suggested a set of acceleration values for use in calculation of overtaking sight distance and are presented in Table 6.2. While, Ramanayya (1980) used the values given in Transportation Engineering Handbook (1982), Badarinath (1993) made use of the polynomial acceleration and deceleration models, recommended by Akcelik and Biggs (1987). Kuncheria (1995) developed linear models based on limited observations to describe acceleration and deceleration characteristics. Hence, for all practical purposes, realistic values or models of acceleration and deceleration characteristics of vehicles are not available. This could be due to lack of proper instrumentation necessary for data collection.

Table 6.2 Acceleration values as recommended by IRC

Speed, kmph	25	30	40	50	65	80	100
Acceleration, kmph/s	5.0	4.8	4.5	4.0	3.28	2.56	1.92

Acceleration rate is governed by the vehicle transmission system, weight and horse power. The acceleration rate also varies with speed, being high at low speeds and low at high speeds. The deceleration rate that a driver utilizes is generally less than what the vehicles can permit. Only in emergency, the drivers attempt to fully utilize the maximum deceleration. The deceleration rates adopted under different conditions are given in Table 6.3.

Table 6.3 Normal Deceleration Rates, (Wilson (1940))

Sl. No.	Description	Deceleration (m/sec^2)
1.	Comfortable to passengers and preferred by drivers	2.62
2.	Undesirable, but not alarming to passenger; Driver would rather not use this	3.39
3.	Severe and uncomfortable to passengers; objects slide off the seats and Drivers use this figure only in emergency stop conditions	4.26

It can be noted from the above tables that the acceleration/ deceleration values can be expressed as a function of speed and in the form given below.

$$\frac{dv}{dt} = \alpha - \beta v$$

where

$\frac{dv}{dt}$ – rate of change in speed (acceleration/deceleration)

α, β – model parameters

v – speed of vehicle

The above equation known as non-uniform model of acceleration has been used in many studies. Inspection of the above equation indicates that α is the maximum acceleration attainable and α/β is the maximum possible speed.

Kuncheria (1995) has measured the speed values at regular intervals of time using a test vehicle and has derived the values of α and β, which are given in Table 6.4 for selected classes of vehicles.

Table 6.4 α, β values of different classes of vehicles (Kuncheria (1995))

S. No.	Vehicle Class	α, m/sec^2	β,sec^{-1}	α/β, m/sec
1	Passenger car	1.75	0.0870	20.36
2	Two-wheeler	1.09	0.0600	18.36
3	Bus	0.64	0.0400	16.24
4	Autorickshaw	1.01	0.0700	13.29

Even though this model is simple and easy to adopt, the model assumes a high initial acceleration value, which is not realistic. It can be noted from the above discussion that the study by Kuncheria (1995) is the only study based on field observations for mixed traffic conditions as prevalent in our country. Absence of studies on acceleration characteristics of vehicles in our country may be due to non-availability of suitable instrumentation.

6.3 Data Collection

Acceleration values can be obtained from either the speed values or the cumulative distances travelled by the vehicle at regular intervals of time. In order to study acceleration and deceleration characteristics of vehicles in road traffic, change in the speed of vehicles is to be obtained with the maximum possible accuracy. Various techniques are being used for collection of speed data to obtain acceleration values.

6.3.1 Methods of Data Collection

In Chase-car technique a vehicle is randomly selected in the traffic stream ("target" vehicle) and is followed in a vehicle equipped with devices for continuous recording of speed. The devices for obtaining speed information continuously include a pen recorder or an electronic distance measuring instrument.

The basic disadvantage in chase-car technique is that the following vehicle is assumed to accelerate or decelerate in the same way as that of target vehicle. But, because the driver behaviour of following vehicle is also involved, the following vehicle's acceleration or deceleration characteristics may be different from that of the target vehicle. So, the data obtained by Chase-Car technique may not be true representation of the characteristic of target vehicle. Also, the total distance to be travelled by test vehicle will be quite large to get sufficient data and so the method is uneconomical.

Even though, the radar speed meter gives the instantaneous speed of the target vehicle, it cannot be used for obtaining the speed profile of a vehicle, as it is very difficult to manually note down the speed values at very short time intervals during the acceleration/deceleration of the vehicle. Otherwise the data has to be transferred/stored at small time intervals on real time basis. Also, the readings are likely to be distorted due to presence of other vehicles.

For many studies, videographic method has proved to be a valuable tool to the traffic engineer. The video recording in field was carried out by setting up video camera by the side of road with orientation of camera as parallel as possible to the roadway so that an approaching or a leaving vehicle can be observed for a long distance. The video recording provides a continuous picture of the front/rear of the approaching/leaving vehicle. This method has the following advantages:

i. Marking control points on the roadway is not necessary

ii. Tall structure adjacent to the road way for setting up video camera is not warranted

iii. Setting up the equipment in the field is fairly easier, and

iv. Behaviour of driver remains unaffected.

By knowing the size of the image, the position of the vehicle with reference to video camera can be calculated at regular intervals of time. The travel time-speed history of vehicles can be obtained from the known positions of vehicles at regular intervals of time.

6.3.2 Data Retrieval Process

The procedure adopted for retrieval of data from recorded tapes is as follows:

- Play back the video tape on a VCR, which is directly connected to a computer though a frame grabber card.
- Using the frame grabber card and associated software bring the video display on to the monitor. (The resolution of the frame grabber card was kept constant through out the data retrieval process.)
- Capture the video display using any of the screen capturing softwares. (HyperCAM, one of the popular Windows based screen capturing software, compresses the captured images and stores as AVI (Audio Video Interleaved) file. The main advantage is that the images will not be stored as separate files but as a single file and also the size of file will be

very less. Another advantage of this software is that it is possible to define screen area to be captured. Using this software, it is also possible to capture the video images at an user selectable rate (between 10 frames per second to 1 frame per minute). The AVI file can be played back using Media Player, a standard utility in Windows 95/NT, at selected rate and the required information can be noted.)

- Note down the image size (width or height) depending on the type of vehicle.

Image size (width and height in pixels) of an object on a computer screen from a pre-recorded video tape depends on the distance of the object from the camera and the properties of camera lens. Consider the image of an object recorded on the videotape at constant zoom position. In this case, the size of the image of the object depends on the distance of the object from the camera. In other words, the actual distance from the video camera to the object depends on the ratio of the size of the image in pixels to the size of that object. As no information regarding optics of video camera is available, empirical relationships between the ratio of image size to object size and the distance of object from video camera were derived. As the resolution of the computer display is different in horizontal and vertical directions, separate models were developed for horizontal and vertical measurements.

The models relating the distance of object from video camera as the independent (X) variable and ratio of image size to actual size as dependent (Y) variable are presented in Table 6.5. The R^2- values obtained are above 0.9 in all the cases indicating the very good predictive capability of the models.

Table 6.5 Equation System Used for Estimation of Object Distance

Zoom Position	Horizontal Direction		Vertical Direction	
	Equation	R^2 - value	Equation	R^2 - value
8	Ln(Y)=8.7494-0.9981*Ln(X)	0.999	Ln(Y)=8.7435-0.9975*Ln(X)	0.999
7	Ln(Y)=8.4966-0.9752*Ln(X)	0.999	Ln(Y)=8.4432-0.9636*Ln(X)	0.997
6	Ln(Y)=8.3319-0.9732*Ln(X)	0.998	Ln(Y)=8.2949-0.9686*Ln(X)	0.998
5	Ln(Y)=8.2782-1.0060*Ln(X)	0.998	Ln(Y)=8.2264-0.9969*Ln(X)	0.999

6.3.3 Estimation of Positions of Vehicles

After the retrieval process, the data available include the image size, which is the distance between reference points of the vehicles, at every one second interval as they pass through the study section and the actual distance between the reference points for various types of vehicles. Using these, the ratio of image size to actual size was computed and this quantity was used in the equation developed and presented earlier, to find out the actual position of the vehicle with respect to the position of the video camera.

6.3.4 Estimation of Speed and Acceleration Profiles

From the known positions of the vehicle at every one second interval, the distance travelled by the vehicle in one second was calculated. The difference between the position of the vehicle in the previous second and its position at present second was taken as the distance travelled by the vehicle in one second. Similarly, after getting the speed values, the acceleration values were calculated, as the difference between the speeds in the previous interval and the present interval. This procedure was repeated for the whole data.

6.4 DEVELOPMENT OF ACCELERATION MODELS

Attempts were made to describe the observed acceleration profiles by means of the simple models, the constant acceleration and linear decreasing models, and the polynomial model as suggested by Akcelik and Biggs (1987). For each class of the vehicles, the parameters of the model were estimated using half of the data available. The remaining half of the data was used for validation of the models. The comparison of relative errors in the prediction of acceleration distance was the basis of evaluation. The results were not satisfactory and hence, the methodology presented below was adopted to develop acceleration and deceleration models of vehicles.

As the rate of acceleration depends on the accelerating capability of vehicle which in turn vary depending on the type of vehicle, separate plots of acceleration values Vs speed for each type of vehicles were drawn. The plots of acceleration values Vs speed for Two-wheelers, Cars, Auto-rickshaws, Buses, Trucks and Jeeps are presented in Fig. 6.1.

It can be observed from the above figures that the acceleration values are widely vary even for a particular speed value. This may be due to the fact that the rate of acceleration depends not only on the characteristics of the vehicle but also on the driver behaviour. So, to study the combined effect of vehicular characteristics and human behaviour on the acceleration rate, the following procedure was adopted.

- Estimation of the acceleration values using the derived equations and the observed speed values.

- Computation of the difference between estimated and observed acceleration values.
- Verify whether these deviations can be explained by stochastic models.

To develop the models for predicting the average acceleration values, the observed acceleration values were classified into different groups depending on speed values. Based on the minimum and maximum observed speeds and using the Sturges formula, the class interval was fixed as 5 kmph.

After fixing the speed ranges, the acceleration values were classified into different groups. For each speed range, the average acceleration value was calculated. These average values were then plotted against mid values of corresponding speed classes. This procedure was repeated for each class of vehicles separately. Fig. 6.2 shows the average acceleration values Vs speed for Cars, Two-wheelers, Auto-rickshaws, Jeeps, Buses and Trucks respectively. It can be observed from those figures that except for Auto-rickshaws, the acceleration rates decrease with increase in speed and follow exponential pattern. In case of Auto-rickshaws, the average acceleration values were found to fluctuate without following any specific trend. This may be due to the complex behaviour of Auto-rickshaw, which enjoys two-degrees of freedom.

Regression analysis was carried out to derive models of average acceleration values in terms of speed for each class of vehicles separately. The derived models are tabulated in Table 6.6.

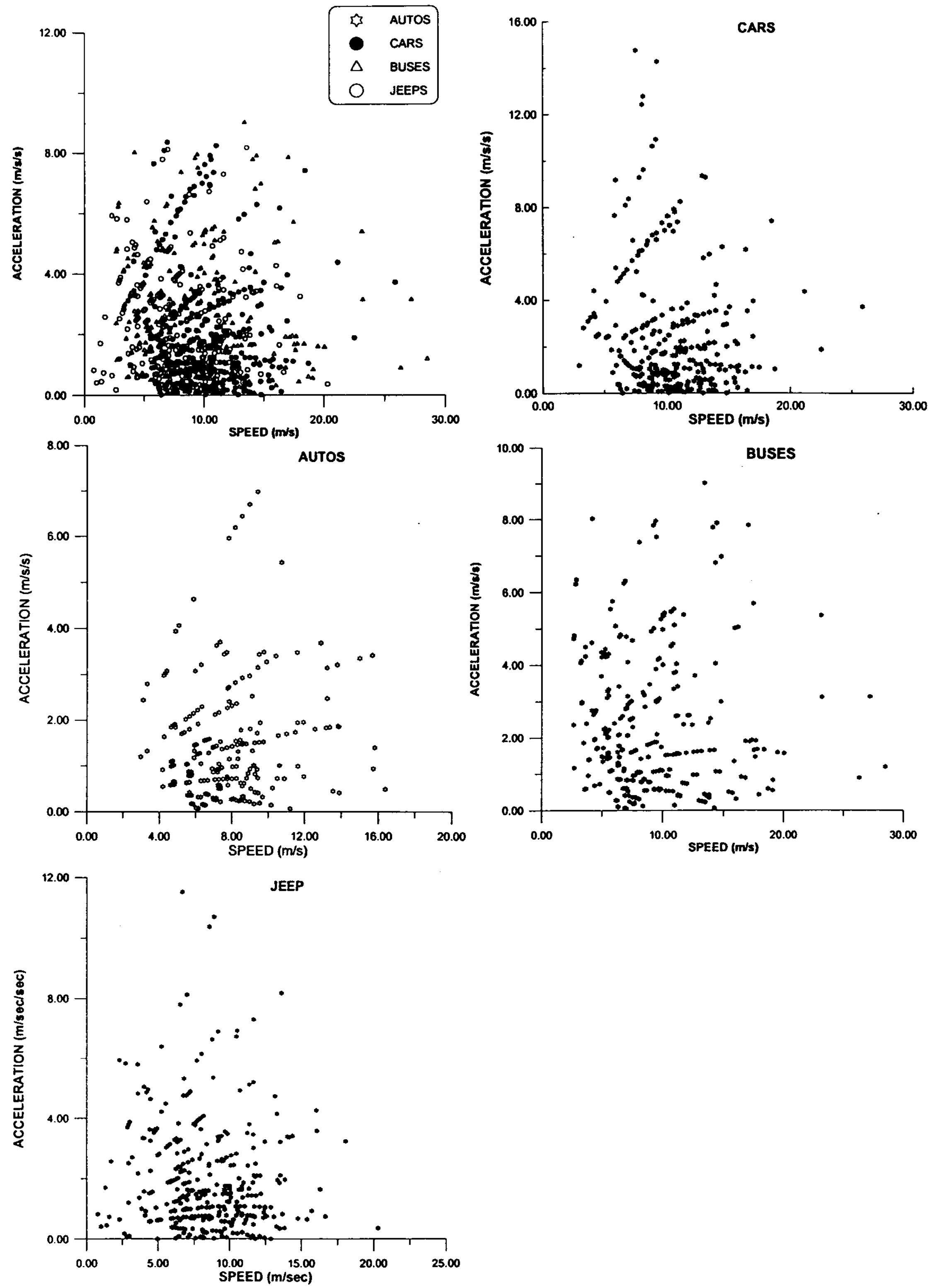

Fig 6.1 Scatter Diagram of Acceleration Values

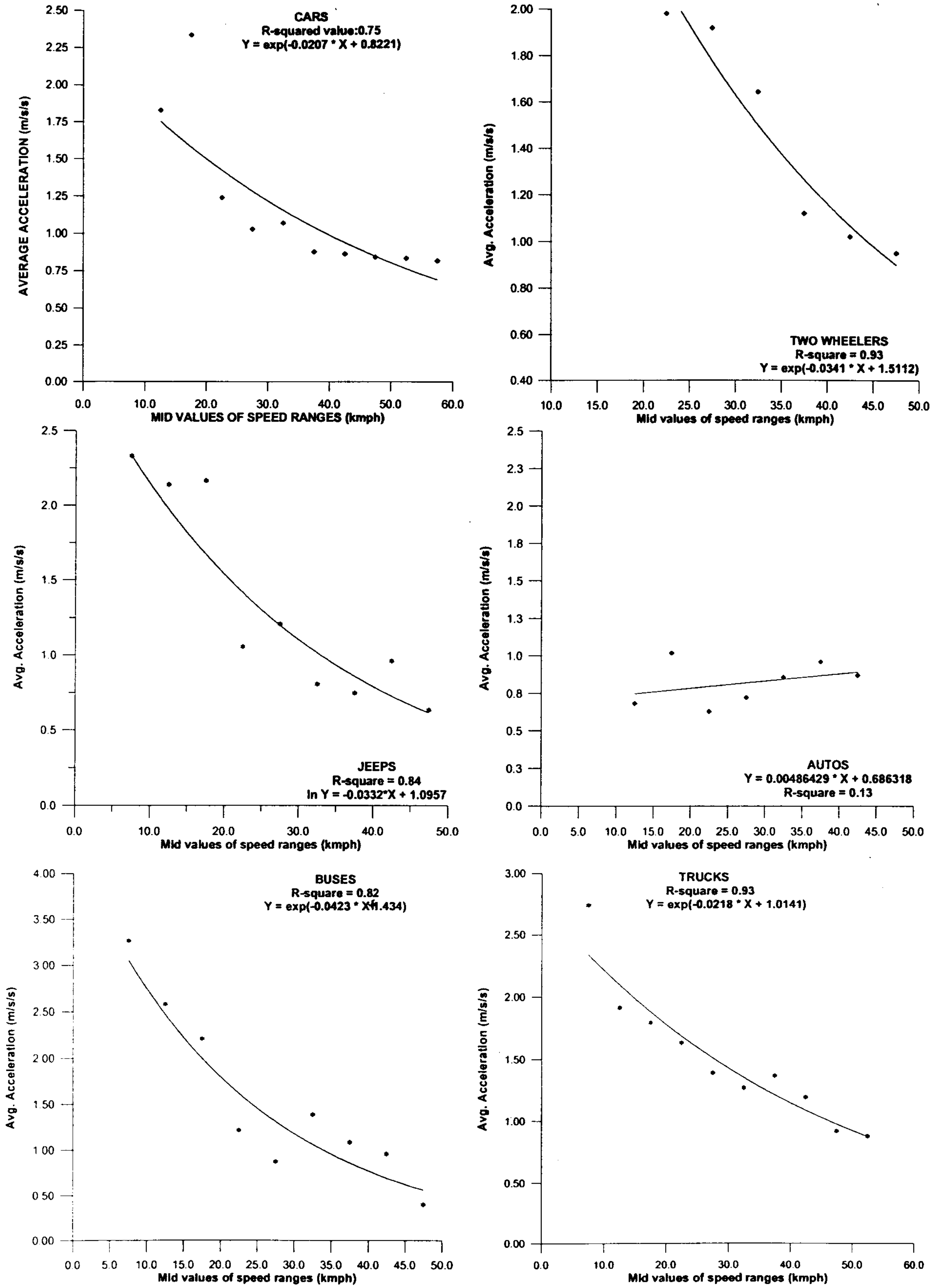

Fig 6.2 Average Acceleration Vs Speed

Table 6.6 Regression Models for Average Acceleration

Sl. No.	Type of Vehicle	Equation	R^2 -value
1	Car	Y = exp(-0.0207*X + 0.8221)	0.75
2	Jeep	Y = exp(-0.0332*X + 1.0957)	0.84
3	Two wheeler	Y = exp(-0.0341*X + 1.5112)	0.93
4	Auto	Y = 0.004864*X + 0.6863	0.13
5	Bus	Y = exp(-0.0423*X + 1.4341)	0.82
6	Truck	Y = exp(-0.0218*X + 1.0141)	0.93

Y – Average acceleration rate; X – Speed, kmph

Except for Auto-rickshaws, the R^2 values of the derived models were found to be good. This indicates that the average acceleration values can be predicted using these modes with reasonably good accuracy. The values obtained from regression models are only average values and thus do not reflect the variations as in the observed values.

6.4.1 Models to Reflect Driver Behaviour

In the earlier discussion the relationship between the acceleration rate of a particular type of vehicle and the speed of that vehicle is studied. But the acceleration rate considered is the average acceleration rate. i.e., the models may reflect the average behaviour of a particular class of vehicles, whereas the acceleration rates vary from driver to driver and time to time. The above calibrated equations do not reflect the driver behaviour. So the acceleration rate depends on the type of vehicle, speed of vehicle and the driver behaviour. While, separate models for each type of vehicle, take care of the effect of type of vehicle, the regression models in terms of speed, take care of the speed effect. So a term, which reflects the driver behaviour, needs to be added to the regression models. Thus an equation of the form given below may be able to reproduce the acceleration rates as observed in reality.

$$a_i = a_{i1} + a_{i2}$$

where a_i : the acceleration value for i^{th} class vehicle

a_{i1} : estimated average acceleration value for i^{th} class vehicle

a_{i2} : component reflecting the driver behaviour

For modelling the driver behaviour component, the values in each speed range were considered separately for each class of vehicles. Using the known values of speed and the regression models average acceleration values were calculated. The differences between the calculated and the observed acceleration values were calculated and these were found to vary quite considerably. These deviations, which are random, reflect the driver behaviour.

To know whether the deviations follow any probability distribution, the deviations, for the selected speed range were grouped into different classes with the interval of each class being calculated. Then a frequency plot was prepared as shown in the Fig. 6.3. It appeared that the deviations follow Normal distribution. So a goodness of fit test, Chi-square test, if the sample is more than 30, otherwise K-S test, was conducted to ascertain the goodness of fit of Normal distribution. The procedure was repeated for each speed range and for all the selected classes of vehicles. The results of goodness of fit tests carried out confirmed that the deviations or the values of component of acceleration rate due to driver behaviour can be well explained by Normal distribution.

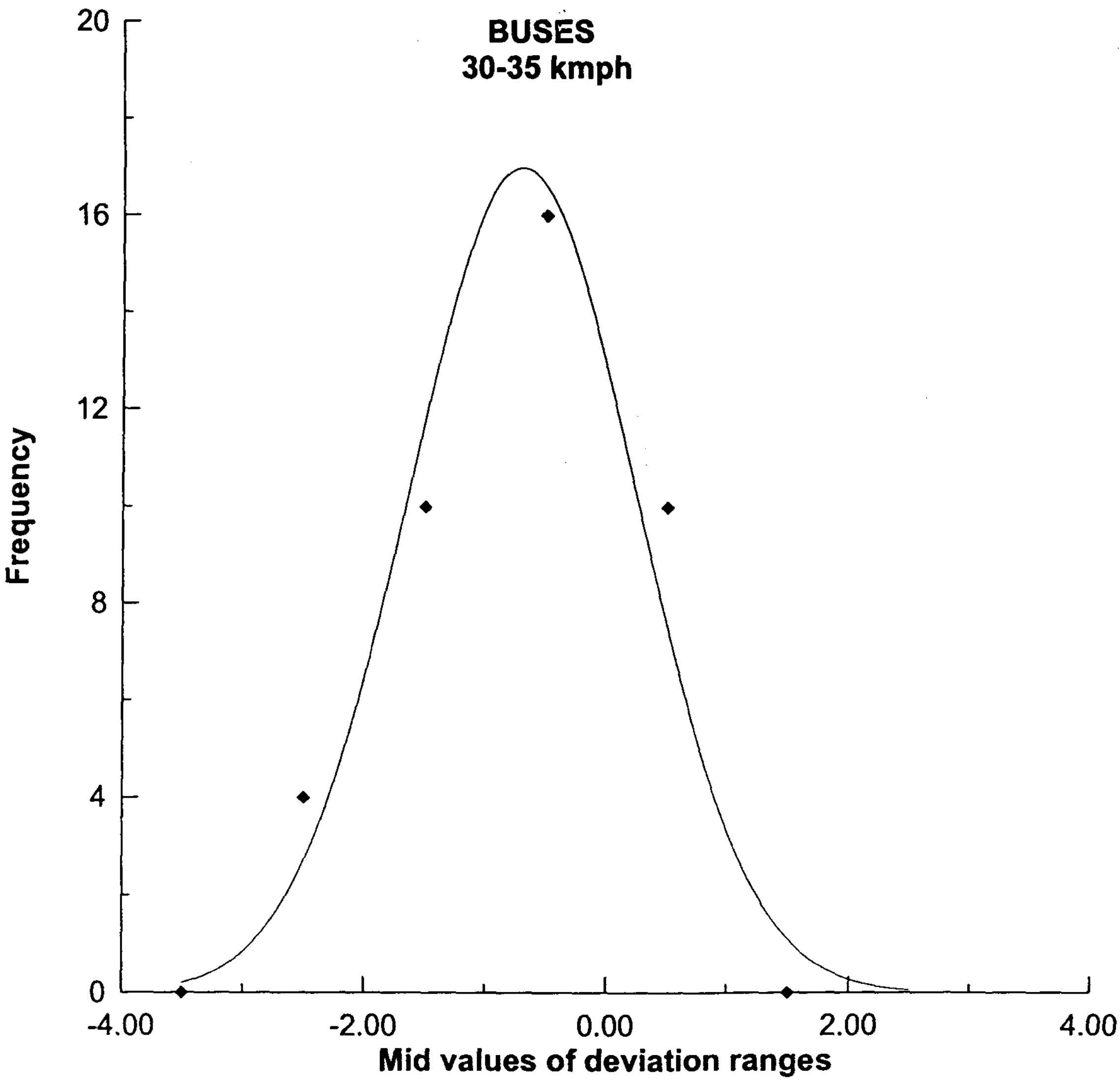

Fig 6.3 Typical Frequency Plot of Deviations of Acceleration Values from Mean Value

6.5 DEVELOPMENT OF DECELERATION MODELS

Analysis procedure in the case of deceleration was performed in the same way as it was done for acceleration. It was found that the average deceleration values when plotted against mid values of corresponding speed classes, follow an exponential pattern for all types of vehicles considered. Fig. 6.4 shows the plots of average deceleration values Vs speed for Cars, Two-wheelers, Auto-rickshaws, Jeeps, Buses and Trucks respectively. It can be observed from these figures that the deceleration rates decrease with decrease in speed values.

Regression analysis was carried out to derive models for average deceleration values in terms of speeds for each class of vehicles separately. The derived models are tabulated in Table 6.7.

Table 6.7 Regression Models for Average Deceleration

Sl. No.	Type of Vehicle	Equation	R^2 - Value
1	Car	Y = exp(0.0215*X - 0.3871)	0.84
2	Jeep	Y = exp(0.0222*X – 0.4438)	0.91
3	Two wheeler	Y = exp(0.0286*X – 0.7381)	0.90
4	Auto	Y = exp(0.0399*X – 1.5825)	0.87
5	Bus	Y = exp(0.0243*X – 0.5012)	0.85
6	Truck	Y = exp(0.0198*X + 0.5772)	0.89

Y – Average deceleration rate; X - Speed

As in the case of acceleration, the frequency distribution plots of deviations were prepared and it appeared that the deviations in the case of deceleration also follow a Normal Distribution. The results of goodness of fit tests carried for all classes of vehicles confirmed that the deviations, i.e., the values of component of deceleration rate due to driver behaviour can be well explained by Normal Distribution except in one case.

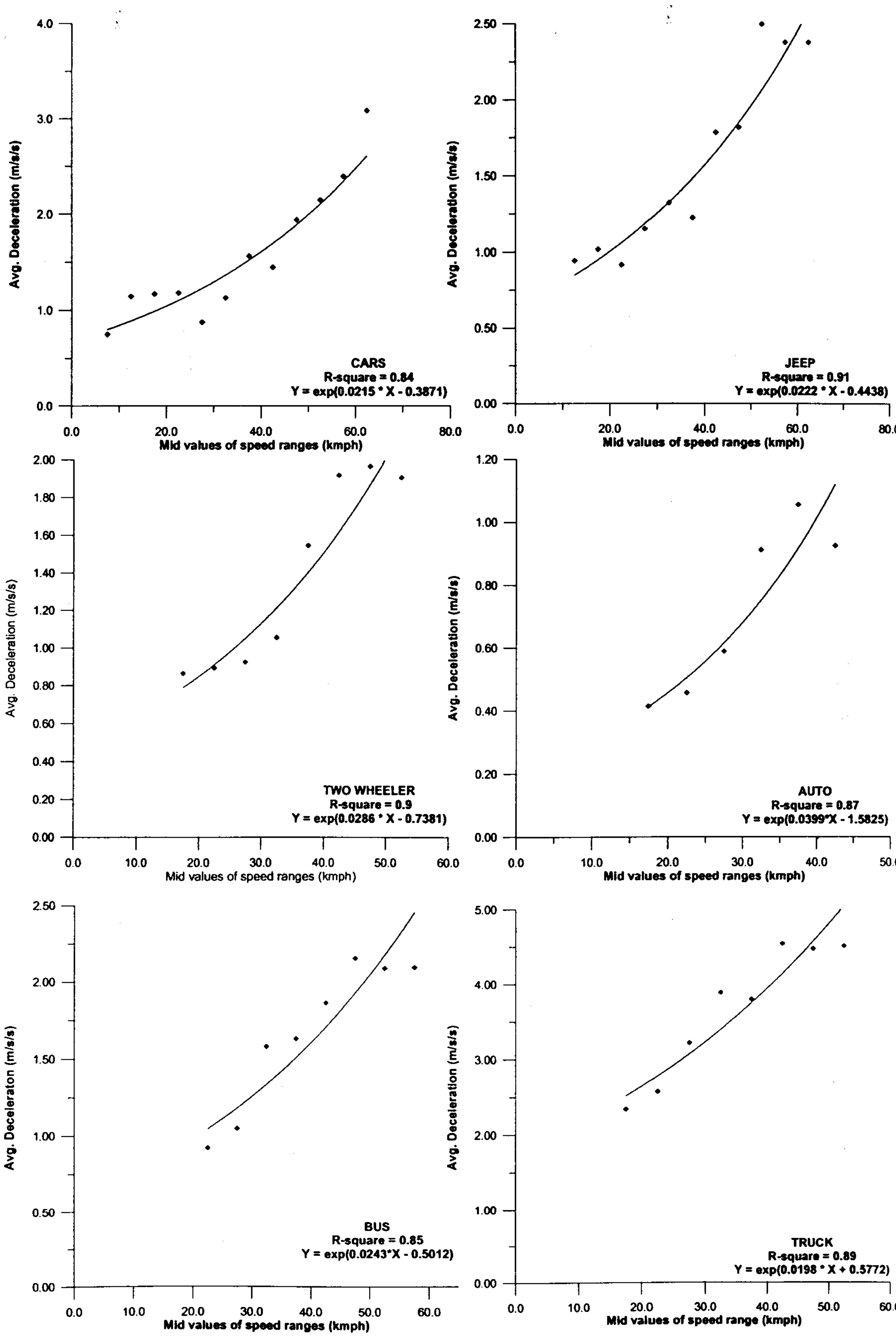

Fig 6.4 Average Deceleration Vs Speed

Considering the observed maximum deceleration rates corresponding to various speed ranges of different types of vehicles, it was found that all the types of vehicles are having the maximum observed deceleration rate in the speed range of 45 – 50 kmph, which is more common. Also, the maximum observed rate of deceleration decreases with decrease in speed for all the types of vehicles which is similar to the pattern observed in the case of average deceleration rate Vs speed.

To validate the relationship developed between the average acceleration/average deceleration and speed, acceleration values of car belonging to the speed range of 20 – 25 kmph, were considered. Using the Normal distribution parameters, the mean of the deviations and the standard deviation of the deviations, random numbers were generated. Next, using these random numbers, deviation values were calculated. Acceleration rates were estimated using these deviation values and the average acceleration value obtained from the model developed for car. The estimated and the actual acceleration values are given in Table 6.8 and percentage RMSE calculated was 8.5.

Table 6.8 Comparison of Estimated and Actual Acceleration Values

S.No.	Estimated Value	Actual Value	S.No.	Estimated Value	Actual Value
1	1.223	1.201	7	1.450	1.413
2	0.579	0.380	8	0.359	0.378
3	1.332	1.346	9	1.6935	1.713
4	1.105	1.274	10	1.211	1.309
5	1.458	1.439	11	0.2829	0.178
6	0.894	0.896	12	1.3504	1.394

6.6 CONCLUSIONS

For a given class of vehicle, the acceleration/deceleration values were found to depend on speed, with average acceleration values decreasing exponentially with increase in speed and average deceleration values increasing exponentially with increase in speed. Acceleration and deceleration models having two components, one given by the models of average acceleration/deceleration values and the other a random component, which reflects the driver behaviour, were developed. The random component, deviation of the average estimated value from the observed value, was found to follow Normal distribution in almost all cases. These models are being used to describe the acceleration and deceleration behaviour of free flowing vehicles in simulation study.

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CHAPTER-7

DEVELOPMENT OF SIMULATION MODEL

7.0 GENERAL

After the development of models for various components of the traffic system, the next stage is the integration of these building blocks into a model. A brief description of the traffic flow simulation model along with various components and validation of the model are presented in this chapter.

7.1 LANGUAGE SELECTION

The first step in the development of simulation model is the selection of language for writing the program code. In general, there are two alternatives to write computer code for simulation exercises. The computer code can be written in a general purpose language such as FORTRAN, PASCAL, C, C++ or using a special purpose simulation language such as SIMULA, GPSS, SIMSCRIPT, GASP, SIMPAC, DYNAMO etc. Even though the special purpose simulation languages have the advantages of less programming time and in-built error checking routines, the advantages such as less flexibility and increased computer running times limit the suitability of these languages for traffic flow simulation applications. Many of the traffic simulation programs were written in general purpose languages: FORTRAN language (SOVT, TWOWAF, SOFOT, TRARR), C language (DRACULA, MICSTRAN), C++ (PLANSIM-T, SITRA-B). The VTI model was written in SIMULA, a special purpose simulation language. For the present work, C has been chosen as the programming language due to its many advantages, especially the ability to handle graphics and familiarity.

7.2 INTERNAL BOOK KEEPING

To achieve efficient operation of the simulation program it is essential to represent the roadway and vehicle characteristics in a logical form so that it requires minimum computer storage and time. The different systems generally used for representation of the system are: Physical representation, Memorandum representation, Mathematical notation, Modified mathematical notation and List processing or chaining. A detailed description of these representation systems is given in Gerlough and Huber(1965) and Drew (1968). All these methods have their own advantages and limitations. The modified mathematical notation technique and list processing technique are known to be superior to other techniques.

The concept of three-dimensional circular array used in the modified mathematical notation cannot be applied to mixed traffic flow, as the vehicles do not observe lane discipline. The system that uses vehicle-based arrays was thus used in mixed traffic simulation models. In the present study, the different characteristics of a vehicle were combined to form a structure. This structure was assigned a number that indicates the position of the vehicle in queue. For continuous operation and to circumvent the problem of computer memory shortage, the Stack Data Structure was used. In this approach, once a vehicle leaves the test section, its related characteristics are popped out and the statistics of other vehicles are updated. Thus, the memory allocated to the leaving vehicle is freed and is allocated to the vehicle entering the test section. Fig 7.1 illustrates this. It should be noted that the characteristics of the leaving vehicle should not be assigned to the entering vehicle, as in circular track concept, which introduces stochastic feed back effects at higher flows (Luk 1976).

7.3 TIME FLOW MECHANISM

As it is hardly possible to monitor all the simulated vehicles simultaneously or continuously, it becomes necessary to scan the system (i.e. examine each vehicle in turn) at specified intervals of simulated time. The two basic approaches that could be adopted are time-based and event-based scanning. In mixed traffic, a large number of significant events such as changes in speeds, changes in position of vehicles, changes in state of vehicles take place simultaneously at a number of places. It is advantageous to use time based scanning technique under such situations.

The limitation of time scanning technique is that the events are assumed to take place at the end of the interval and so tends to give less accurate results. This limitation can be overcome by reducing the scan interval, but at the cost of increased simulation time. In general, one second scan interval was found satisfactory. The scanning interval is user selectable in the present model.

However, considering the complexity of the system and availability of high speed computers, a 0.3 sec scan interval was selected in this study.

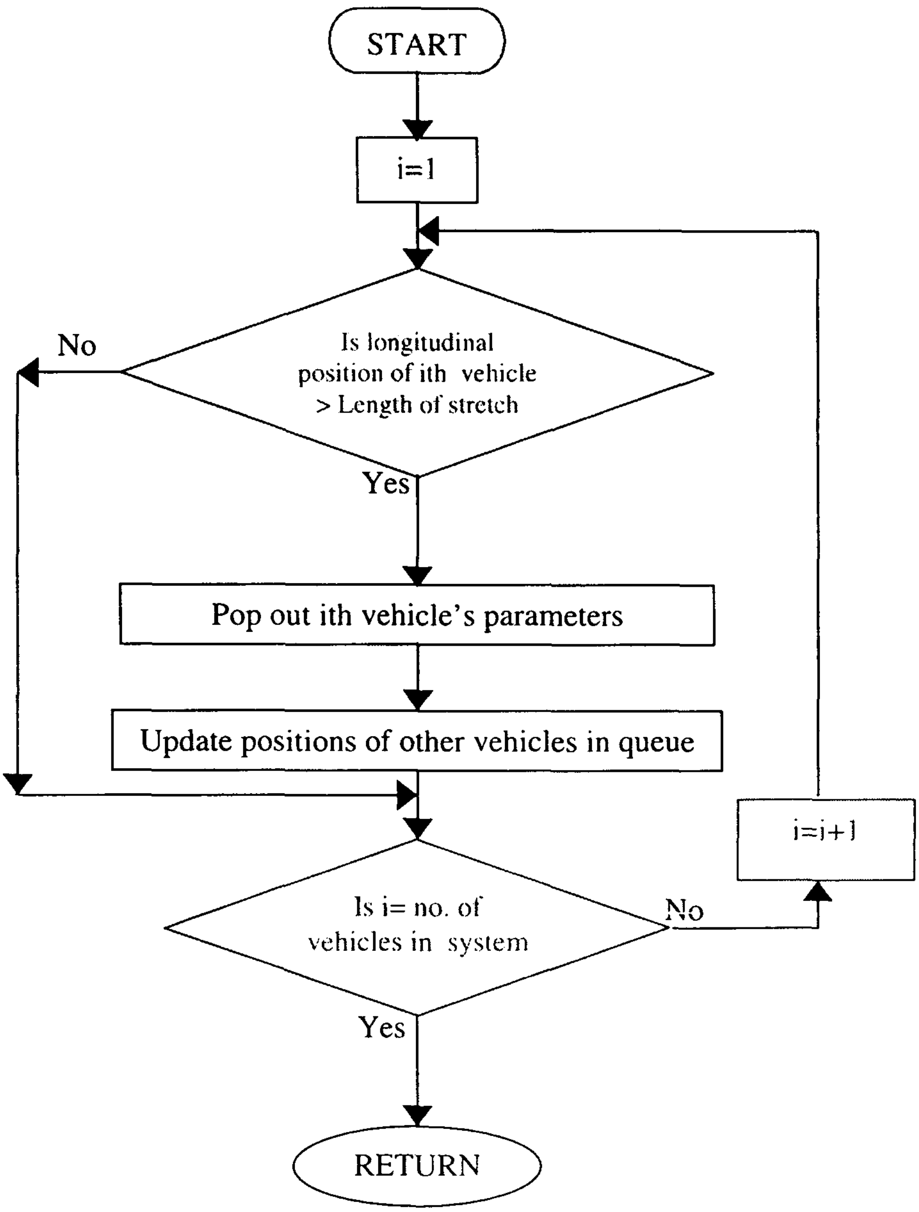

Fig 7.1 Stack Data Structure

7.4 GENERAL DESCRIPTION OF THE MODEL

The simulation model consists of a main program, which acts as master for the rest of the subroutines, which define the various component models. The sub-models include the input module, output module and processing module. The structure of the model is presented in Fig. 7.2.

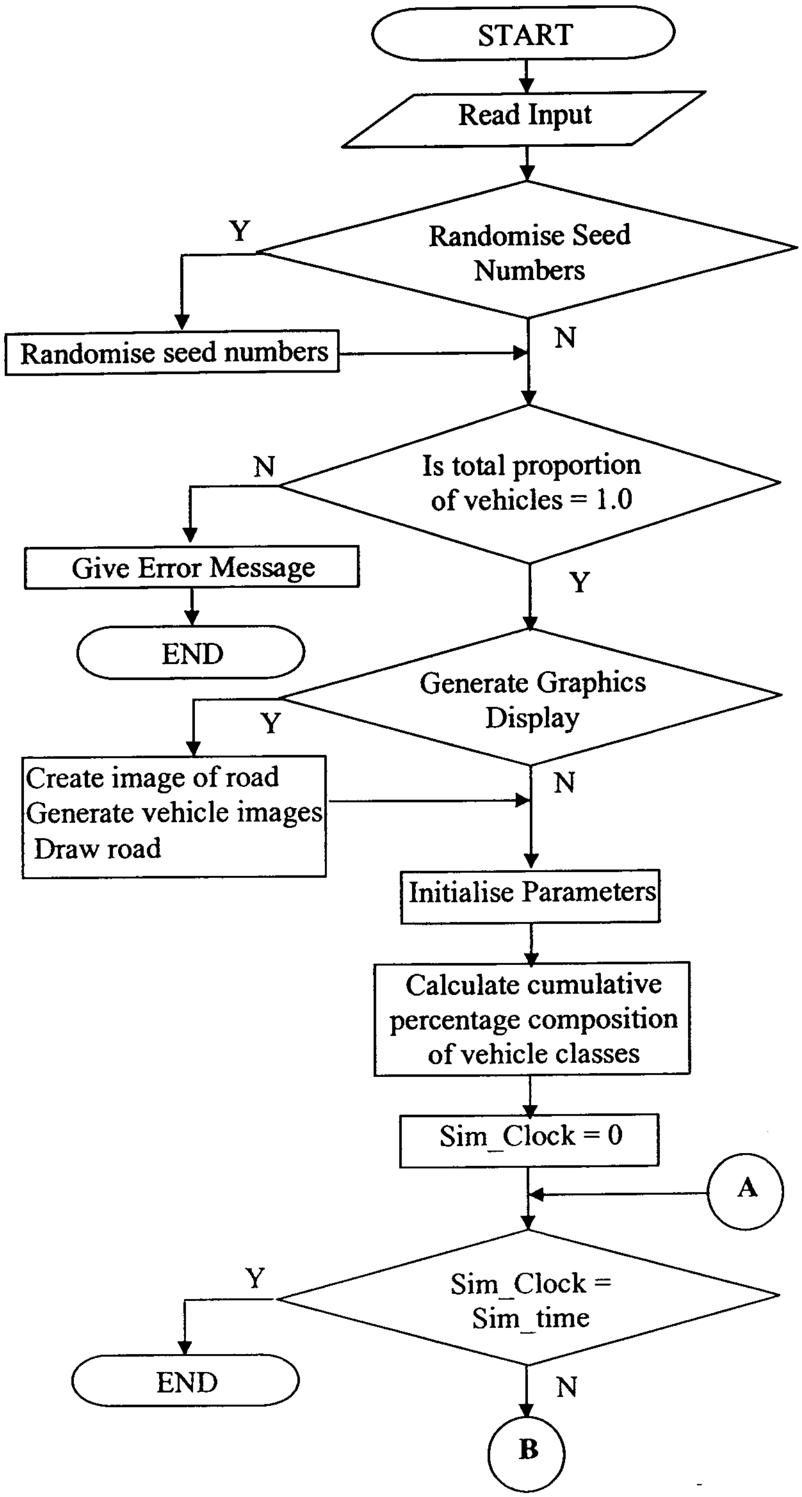

Fig 7.2 Structure of Simulation Model (contd.)

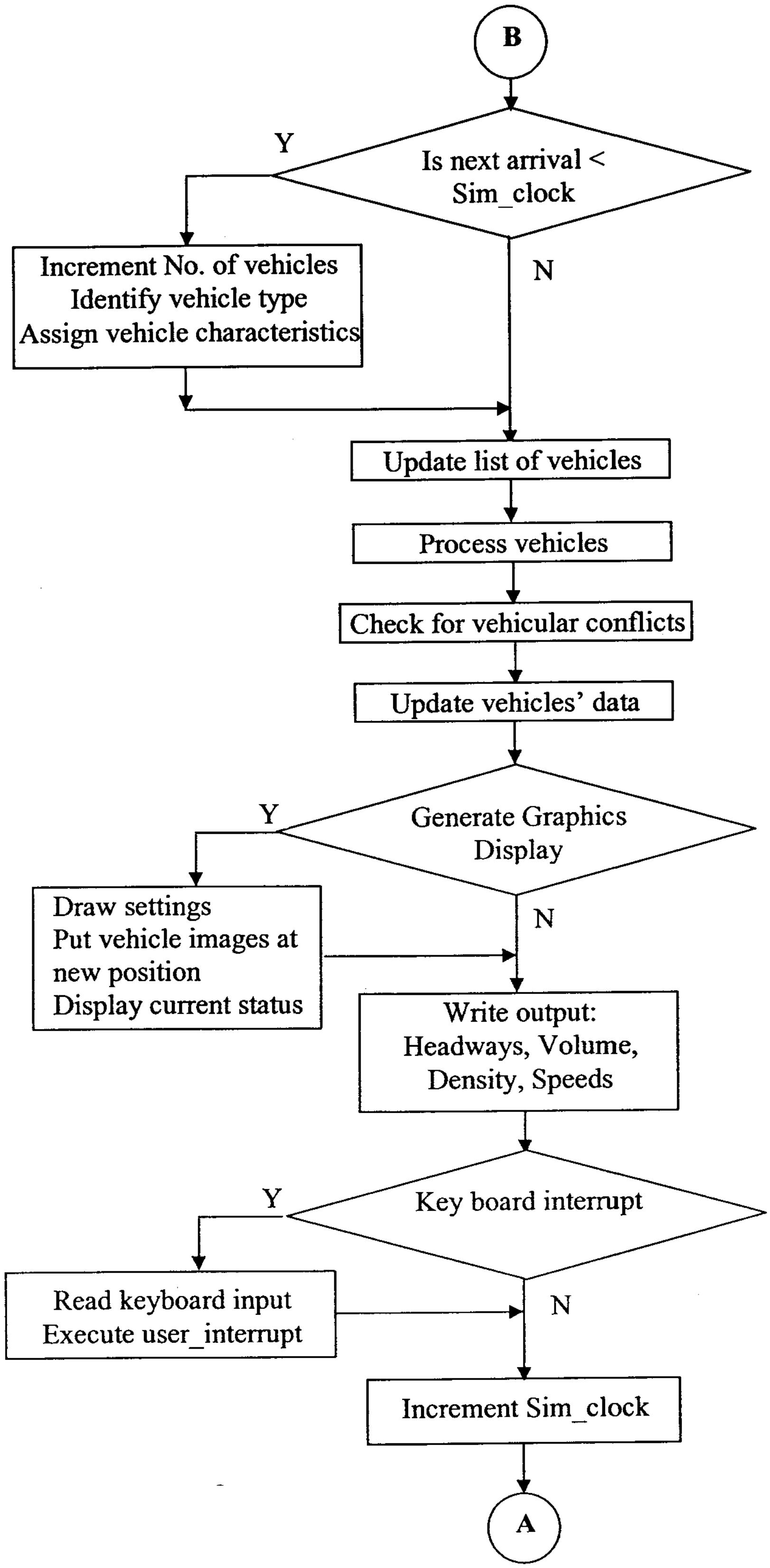

Fig 7.2 Structure of Simulation Model

7.4.1 Input Module

The input subroutine reads the needed inputs from different files. Inputs to the model can be grouped into four categories:

- Road characteristics (length and width of road, length of the road to be displayed),
- Vehicle characteristics (Dimensions, mean and standard deviation of free flow speeds, maximum acceleration/deceleration values, default path of vehicles,).
- Stream characteristics (entry flow, percentage composition of different vehicles)
- Data for various models (neural network weights/regression coefficients of desired frontal spacings, lateral clearances, car-following models, acceleration/deceleration models, free lateral placement, etc.) and
- General data (simulation time, scanning interval, duration of sampling period for recording the outputs, serial number of vehicle whose characteristics are to be stored, option to randomise seed numbers for random number generation).

The road width can be varied (need not be in terms of lanes) as per the requirement of the simulation experiment. The length of the road is also a variable in the model. However, the length of the road and the section where the observations have to be taken depend on the length of warm-up zone. The model is capable of accommodating up to ten types of vehicles.

7.4.2 Random Numbers

Road traffic systems with the driver-vehicle unit being one of the basic elements are characterised by many stochastic processes. Generation of random numbers is a fundamental element in simulation of stochastic systems. The random number generator used in this study is the linear congruential random number generator proposed by Park and Miller (1988) with Bays – Durham shuffling algorithm. The random number generator gives a uniform random deviate between 0.0 and 1.0.

In simulation of complex systems such as traffic systems, it is often necessary to deal with several independent random processes. Random numbers are required to create these random processes. If the random numbers required for generating different random processes are taken from a single series of random numbers, then the random variates generated for a particular process will lack the required characteristics. To overcome this problem, separate seed numbers were used to generate the sequences of random numbers required for different processes. For example, separate sequences of random numbers were used to assign the desired speeds of different classes of vehicles generated. The program has the capability of generating 100 sequences of random numbers.

The random number generator generates the same sequence of random numbers for a given seed number. This results in the same conditions being simulated for a given set of input values. In simulation of stochastic systems it becomes necessary to generate various possible scenarios for the specified set of conditions. At the same time, during the debugging of program, it is necessary to generate the same sequence of random numbers so that the error identification will be easy. An option was provided in the program to randomise the seed numbers so that different random numbers will be generated for a particular process on each trail.

7.4.3 Generation of Non Uniform Variates

The general requirement in simulation is for a sequence of random numbers, which follow the distribution that best describes the process under study. Uniformly distributed random numbers, generated as described in the previous article, are used for generating such numbers. Other variates needed in the present study are the exponential variates and normal variates. The inverse transformation method and the method based on central limit theorem were used to generate exponential variates and normal variates respectively.

7.4.4 Generation of Vehicle Arrivals

The arrival process consists of the generation of inter arrival times of the vehicles at the entry to the simulated stretch. The arrivals are generally simulated by sampling from specified headway distributions. It is well established that the headways in free flow conditions could be described by exponential distribution but it is not the same in platoon flow conditions. As no specific distribution will be able to describe the headways over all the ranges of flows in mixed traffic, the warm zone approach has been adopted in this study. In this approach, suitable length of hypothetical road is added before the simulated stretch. The vehicle arrival times at the entry to the warm-up zone were obtained by sampling from exponential distribution. The vehicles thus generated are moved through the warm-up zone as per the flow logic of the model

7.5 VEHICLE CHARACTERISATION

After generating the arrival pattern, it is necessary to identify the type of vehicle that would be entering the test section and assign its characteristics. The vehicle type was identified on the basis of cumulative uniform distribution for the given composition of traffic stream. For this purpose one sequence of random numbers was used. Identification of vehicle type is presented in Fig 7.3.

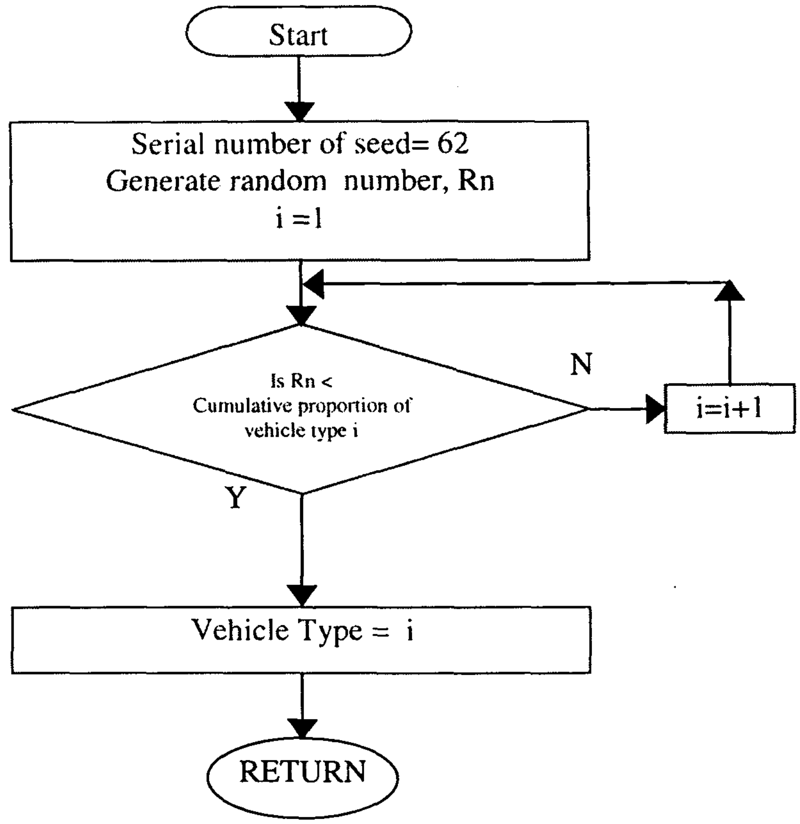

Fig 7.3 Subroutine to Identify Vehicle Type

The characteristics to be assigned to the generated vehicles include the physical dimensions, the free flow speeds, the acceleration and deceleration characteristics and the desired linear and lateral clearances.

7.5.1 Physical Dimensions of the Vehicles

The physical dimensions of the vehicles to be specified in simulation model include the length and the width of the vehicles. The physical dimensions of the design vehicle of different classes as recommended by Indian Roads Congress (IRC SP - 41, 1994) were used in this study. After identifying the type of vehicle, the dimensions of vehicles are assigned based on type of vehicle.

7.5.2 Free Flow Speeds

The next part in the vehicle characterisation is to assign the free flow speeds to the generated vehicles. Free flow speed is the speed at which a vehicle is driven when it is not under the influence of any other vehicles. No vehicle will be allowed to exceed the assigned free flow speed. Each vehicle generated was assigned the free flow speed sampled from normal distribution with the mean and standard deviations values as presented in Table. 4.14. Separate random number sequences were used for determining the free flow speeds of different classes of vehicles. Illustration of assignment of various characteristics to the generated vehicle is given in Fig. 7.4.

7.5.3 Free Lateral Placement

Each of the generated vehicles has to be assigned the position it is likely to occupy across the width of road in case it is not under the influence of any other vehicle. The free lateral position was determined using the regression models developed based on field observations and incorporating a random component to reflect the driver dependent variability.

7.5.4 Vehicle Placement

A generated vehicle is placed at the beginning of the simulation stretch after considering actual time of arrival at entry and nearest updating time. The available and required linear clearances are also being taken into consideration before fixing the longitudinal position of a vehicle at the time of entry into the simulation stretch. If the available clearance is less than the required clearance, the speed of vehicle is reduced depending on the available clearance. If the available clearance is still insufficient, the vehicle is placed before the entry point.

The lateral position of a vehicle at entry is fixed randomly and taking into consideration the available and required lateral clearances.

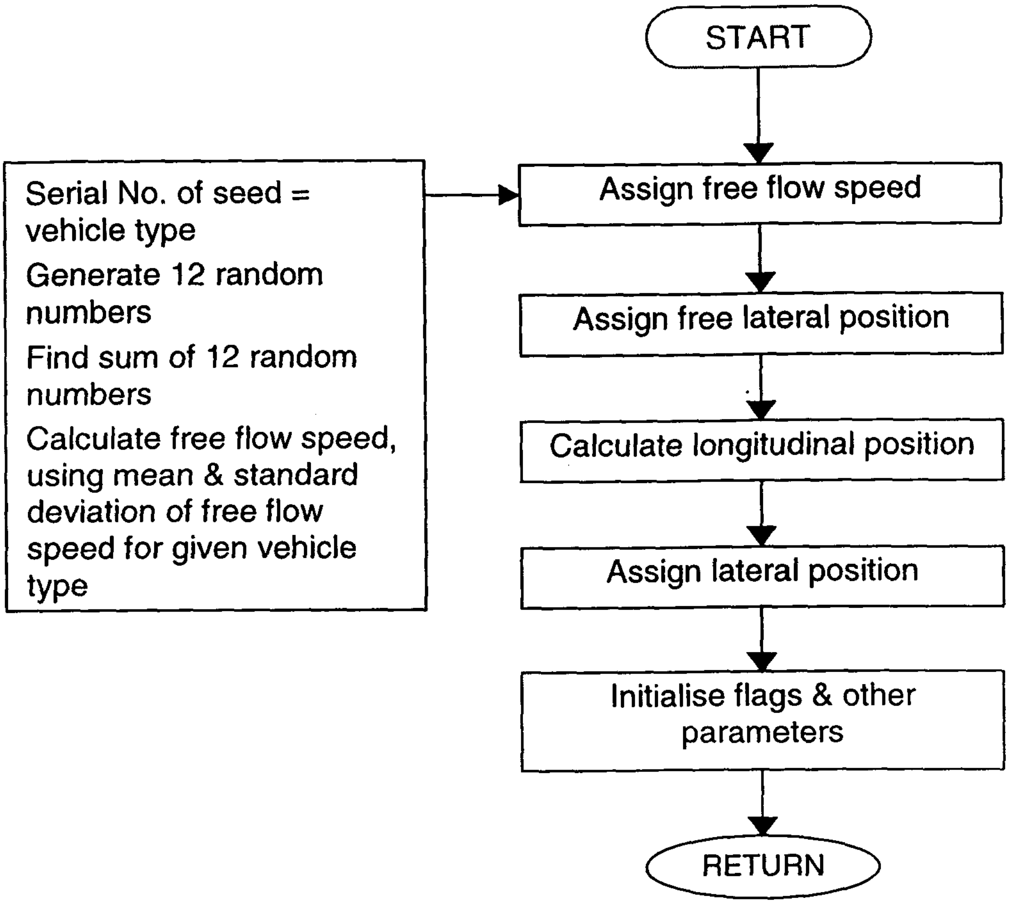

Fig 7.4 Subroutine to Assign Characteristics

As presented earlier, the simulated system is scanned and updated after each increment of simulated time. Processing of all the vehicles, from the first vehicle to the last vehicle, in the simulated system is carried out at every scanning of the system. If a vehicle's longitudinal position is greater than the length of simulated stretch then that vehicle will be removed from the list and all other vehicles are updated in the list. Thus, as and when a vehicle leaves the simulated stretch, the memory allocated for that vehicle is freed and is allocated to the next vehicle in the list. The first vehicle is moved with its desired speed and its lateral position is determined based on flow logic. All subsequent vehicles are moved through the simulated system as described in the vehicle manoeuvring logic.

7.6 VEHICLE MANOEUVRING LOGIC

The processing of the vehicles through the simulated traffic system forms the core of the simulation model. For each vehicle, the vehicles in the surrounding are determined. In case of lane disciplined traffic it is done on the basis of lane occupied. In case of heterogeneous traffic consisting of vehicles of different dimensions, it is a tough task to find the vehicles in the surroundings. This is done considering the lateral space (sum of required clearance on left, width of vehicle and required right clearance) required for the vehicle to continue at the present speed. For each vehicle, the vehicle on the front, the vehicle on front left, the vehicle on the front right, the vehicle on the left, the vehicle on the right, the vehicle at the back, the vehicle at back left and the vehicle at back right are determined. Consider the vehicles shown in Fig. 7.5. Let R be the subject vehicle. B is the vehicle in front, C is the vehicle on front left, D is the vehicle on front right, E is the vehicle on the left, F is the vehicle on the right, G is the vehicle on the back left, H is the vehicle on the back, I is the vehicle on the back right.

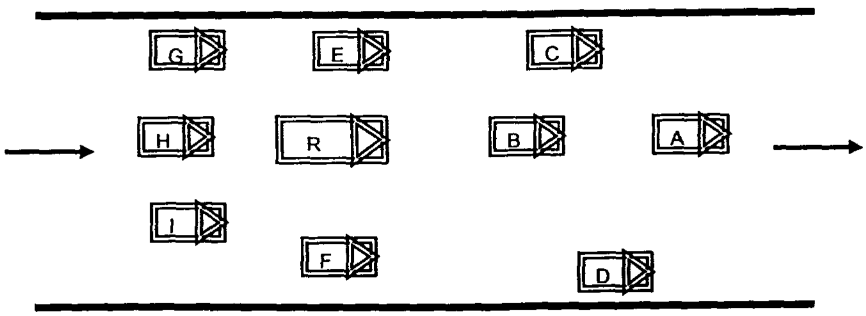

Fig 7.5 Schematic Representation of Positions of Vehicles

The headway with respect to the nearest front vehicle is determined and if it is greater than the threshold headway the vehicle is considered to be in the free flowing regime. In this case, if the vehicle's current speed is lower than its desired speed, it accelerates at the normal acceleration rate to achieve its desired speed; if the current speed is higher than the desired speed, the vehicle decelerates with the normal deceleration rate to slow down. Also, a free flowing vehicle moves towards its free lateral position (default path) at the rate of lateral movement, if it need not move to the side to give way to a fast vehicle to overtake.

If a vehicle is not able to move under free flow conditions, the vehicle will move to the right side and overtake the leading slow vehicle using the space available adjacent to the vehicle in front. This can be called as 'Fly-over taking' condition. Some times it may so happen that for a vehicle under consideration there may two vehicles in the front. Then the vehicle on the front right will be identified and will be taken as the reference vehicle.

If it is not possible to overtake the leading slow vehicle, then the vehicle will follow the lead vehicle at a reduced speed, till overtaking opportunity is available. This can be called as 'Following' condition. This is preceded by 'Decelerating' state and followed by 'Accelerating' state.

So a vehicle can be in any one of these five states at a given point of time. The logic used for movement of vehicles through the simulated stretch was developed taking into account the available and required linear and lateral clearances for a vehicle to continue in a particular state or change to a different state.

Logic

- If the frontal headway is greater than or equal to the threshold headway, the vehicle will accelerate/decelerate to its desired speed or continue at its desired speed. (Free Flowing Condition)
- If the available frontal headway is less than the threshold headway and the available total lateral clearance is greater than or equal to the required total lateral clearance, the vehicle will initiate overtaking.
- If the clearances with respect to the vehicles in the target path are greater than required clearances, then the vehicle continues to overtake. Let the vehicles A & C are in the target path of the subject vehicle (R). The lead gap, gap with respect to the lead vehicle B in the target path and lag gap, gap with respect to the lag vehicle C in the target path will be determined based on the positions and speeds of the vehicles and if they are greater than the required lead and lag gaps then the subject vehicle will move laterally to the target path, as shown in Fig 7.6. The required gaps are determined taking into consideration the type and speed of the vehicles in target path.
- If moving to the target path and overtaking is not possible, the vehicle will follow the vehicle in the front. For this purpose, the car-following subroutine is invoked.

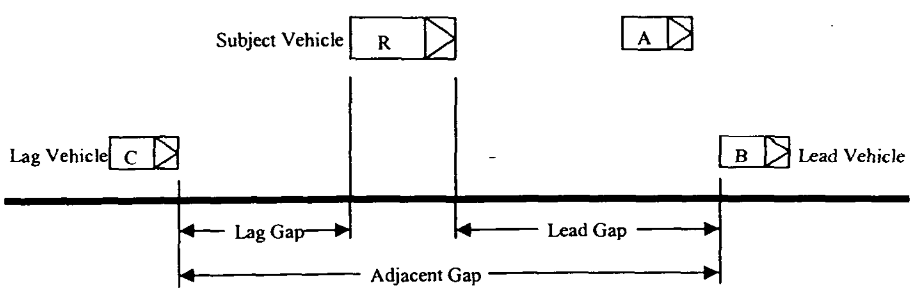

Fig 7.6 Lead & Lag Gaps

- A vehicle in the *car-following* mode continuously looks for an overtaking opportunity and will initiate the overtaking as and when an opportunity is available.

The various subroutines that are part of the vehicle manoeuvring logic are presented in Fig 7.7 to 7.10.

7.7 OUTPUT MODULE

During the scanning and updating process at the end of each scan interval, each and every vehicle is examined whether it had passed any of the control points. In case it had passed, the information of the vehicle was recorded in a separate file. At the end of simulation, the recorded information were analysed and the required output were obtained. The simulation model output included the stream characteristics viz., the flow, the density and the stream speed and the speeds of individual vehicles.

Provision has been made in the program to obtain the number of overtakings performed and the headways also. It is also possible to define any number of control points, at which the information of the vehicles has to be recorded.

7.7.1 Graphics Output Module

A graphics display module that facilitates the visual observation of vehicles as they move through the simulated system was specially incorporated in the simulation program. The graphics module include various functions for drawing of a blank road image, drawing of windows for display of simulation status, display of the simulation status, generation of vehicle images, putting the vehicle images at proper positions on the screen, etc. Components and working of graphics module are shown in Fig.7.11.

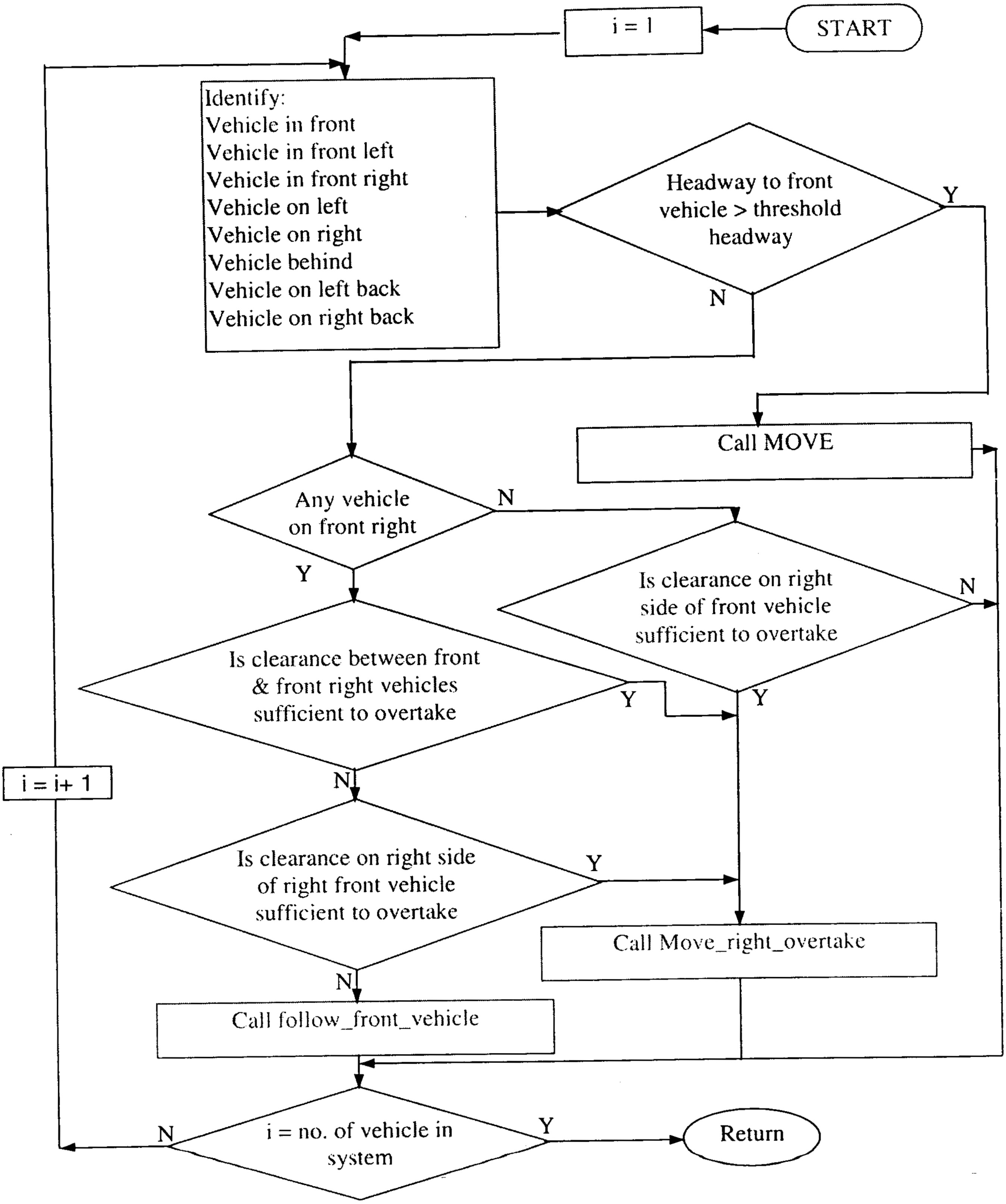

Fig 7.7 Subroutine update_vehicle_data

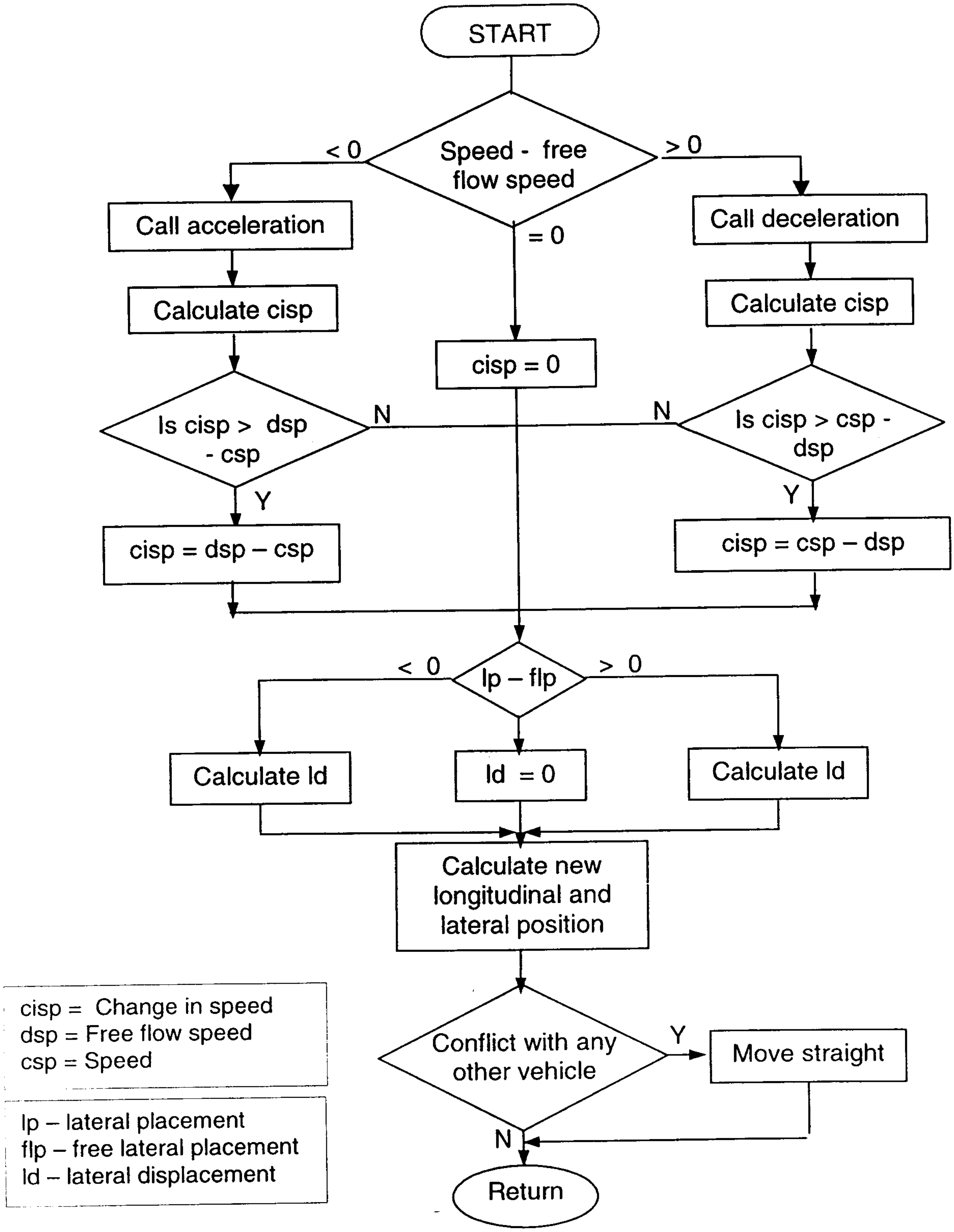

Fig 7.8 Function MOVE

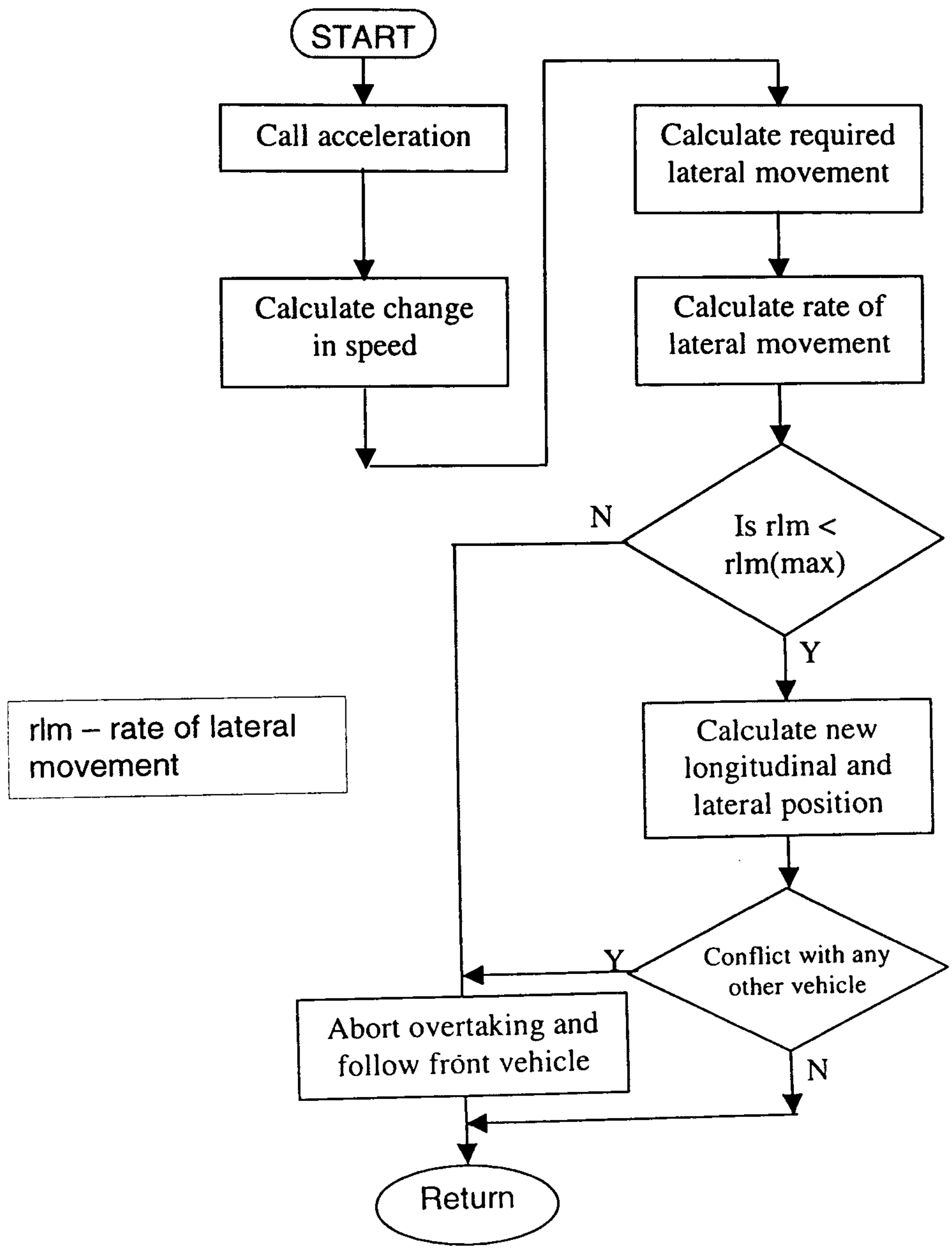

Fig 7.9 Function move_right_overtake

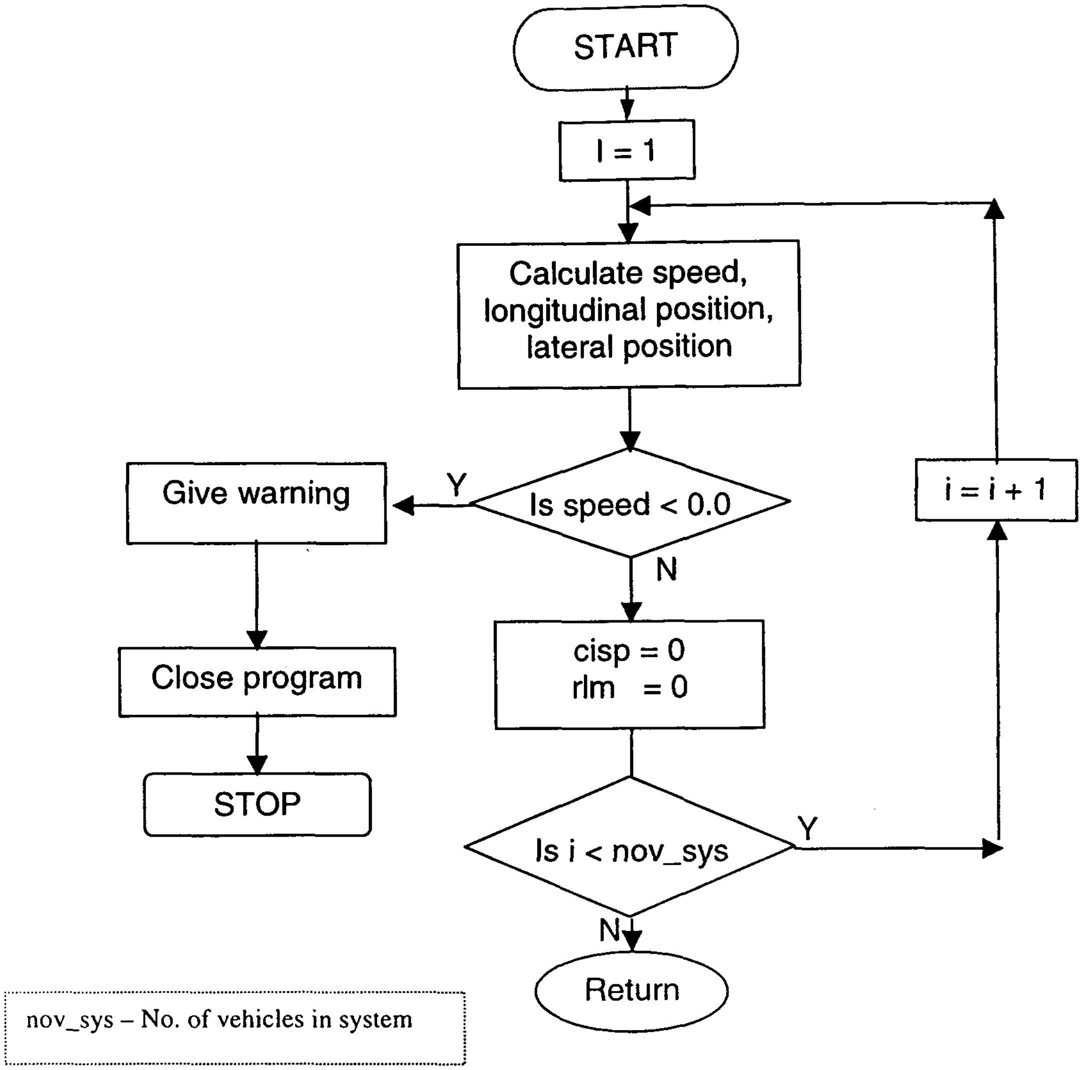

Fig 7.10 Function update_position

A realistic display of vehicle movement was achieved by erasing and drawing images of vehicles at the end of each scanning interval. The graphics display provided an opportunity to check for any inconsistencies, such as two vehicles occupying the same position at same time, and to apply suitable corrections. As the vehicles have to be erased and redrawn at the end of each scan period, running of simulation model with graphical display takes lot of computer time. An option has been provided to run the simulation model without the graphics display so that experimental runs can be conducted in less time. Fig 7.12 & 7.13 show screen shots of Graphic Display of simulation model.

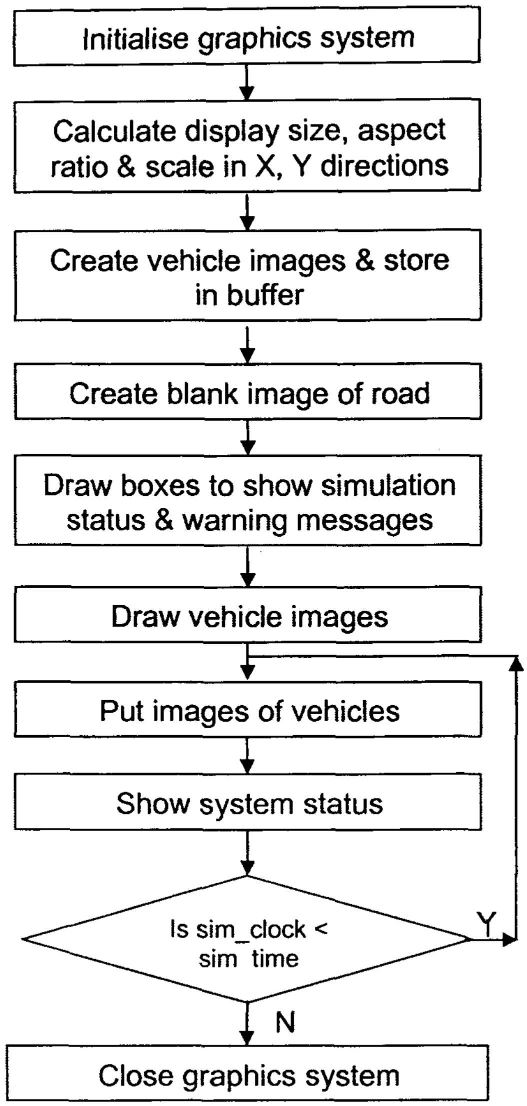

Fig 7.11 Components of Graphics Module

Another feature of the model is that the user can interact with the program through key board. The program will check if any key is pressed during each scan interval and if any key is pressed it will call the function 'user_interrupt'. The working of this function is presented in Fig. 7.14.

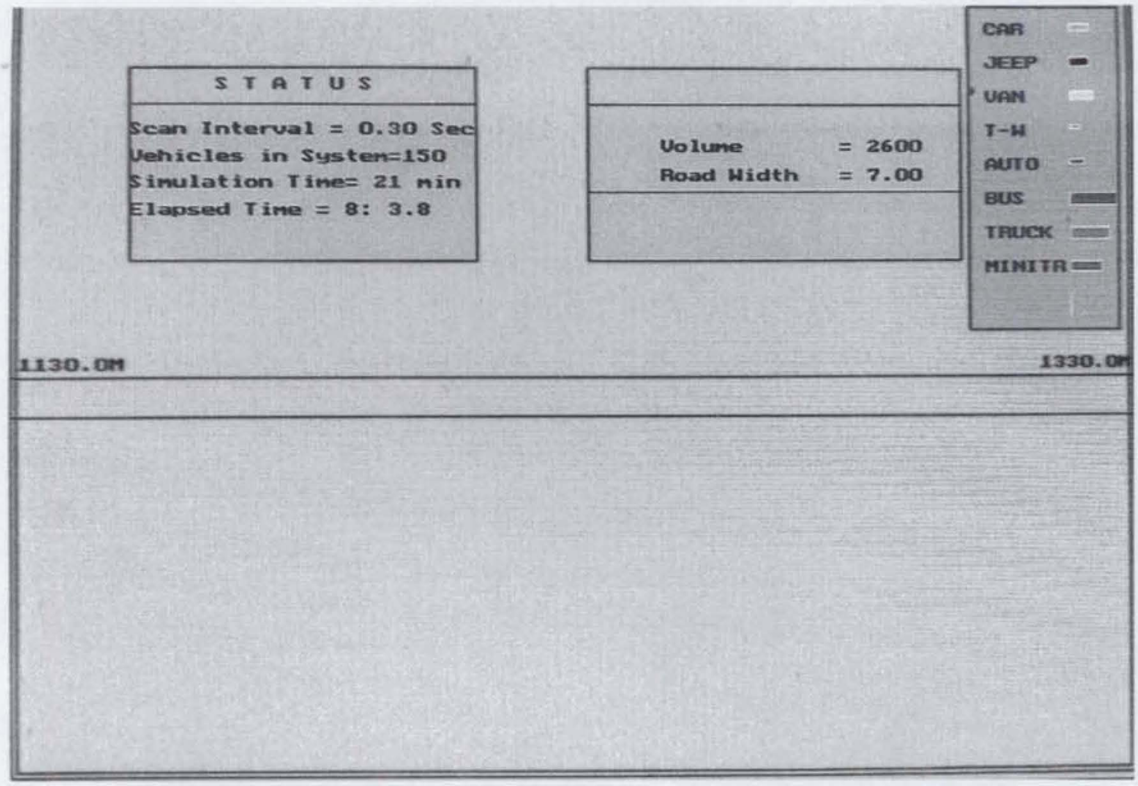

Fig 7.12 Graphic Display of the Model with Empty Roadway

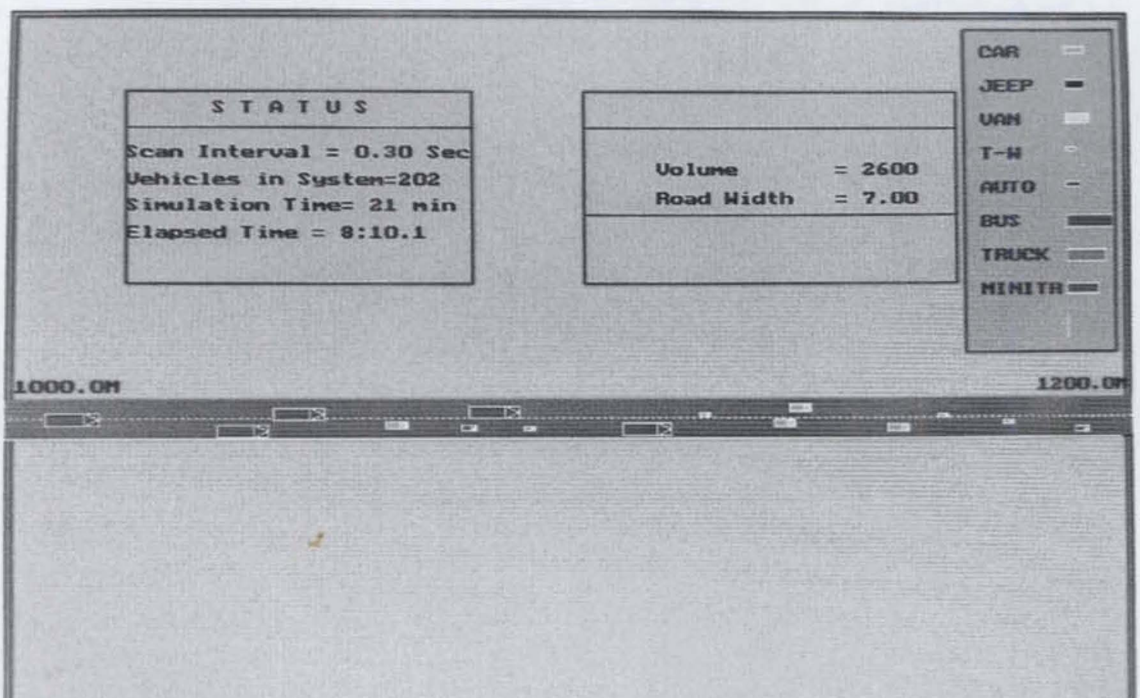

Fig 7.13 Graphic Display of the Model with Vehicles

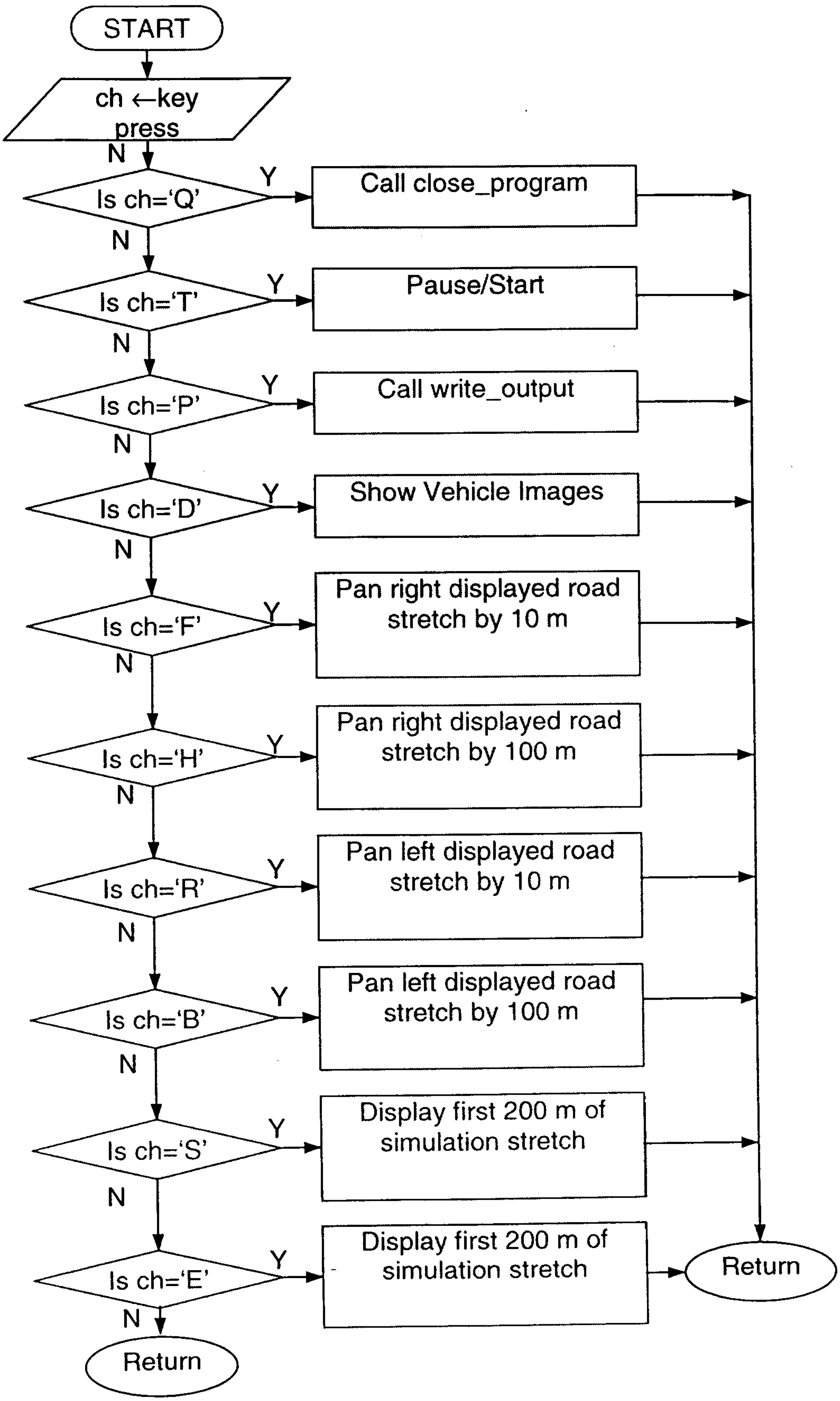

Fig 7.14 Function User_Interrupt

One of the features is to display or not to display the images of vehicles. By default, the images will not be displayed and if required key 'D' has to be pressed.

Another feature is to pan the view of the simulated stretch either forward or backward. By default the section at the beginning of the simulated stretch will be displayed. The view can be moved forward by 100 m by pressing the key 'H'. This feature enables to follow a selected vehicle as it moves through the simulated stretch.

7.8 DETERMINATION OF WARM-UP ZONE LENGTH

In the warm-up zone approach, it is assumed that the traffic would reach a state of equilibrium by the time the vehicles reach the entry to the observation stretch. Hence, the system will begin to give true results only after the traffic stream achieves spatial and temporal stability. To arrive at the length of the warm-up zone and the time-taken for the system to stabilise, the following experiments were conducted on the model.

Experiments were conducted with 2 km as the length of simulation stretch and a simulation time of 60 minutes for various volume levels. The headways were recorded at every 0.100 km sections. Headways at successive sections were compared using the percent frequency distribution diagrams prepared. The frequency diagrams of headways for the volumes 500, 1500 and 2500 vph are presented in Fig.7.15, 7.16 & 7.17 respectively. The headways were also compared by calculating the sum of normalised squared deviations of percent frequencies of headways at successive sections, which are given in Table 7.1. It was observed from these frequency distribution diagrams and the values of sum

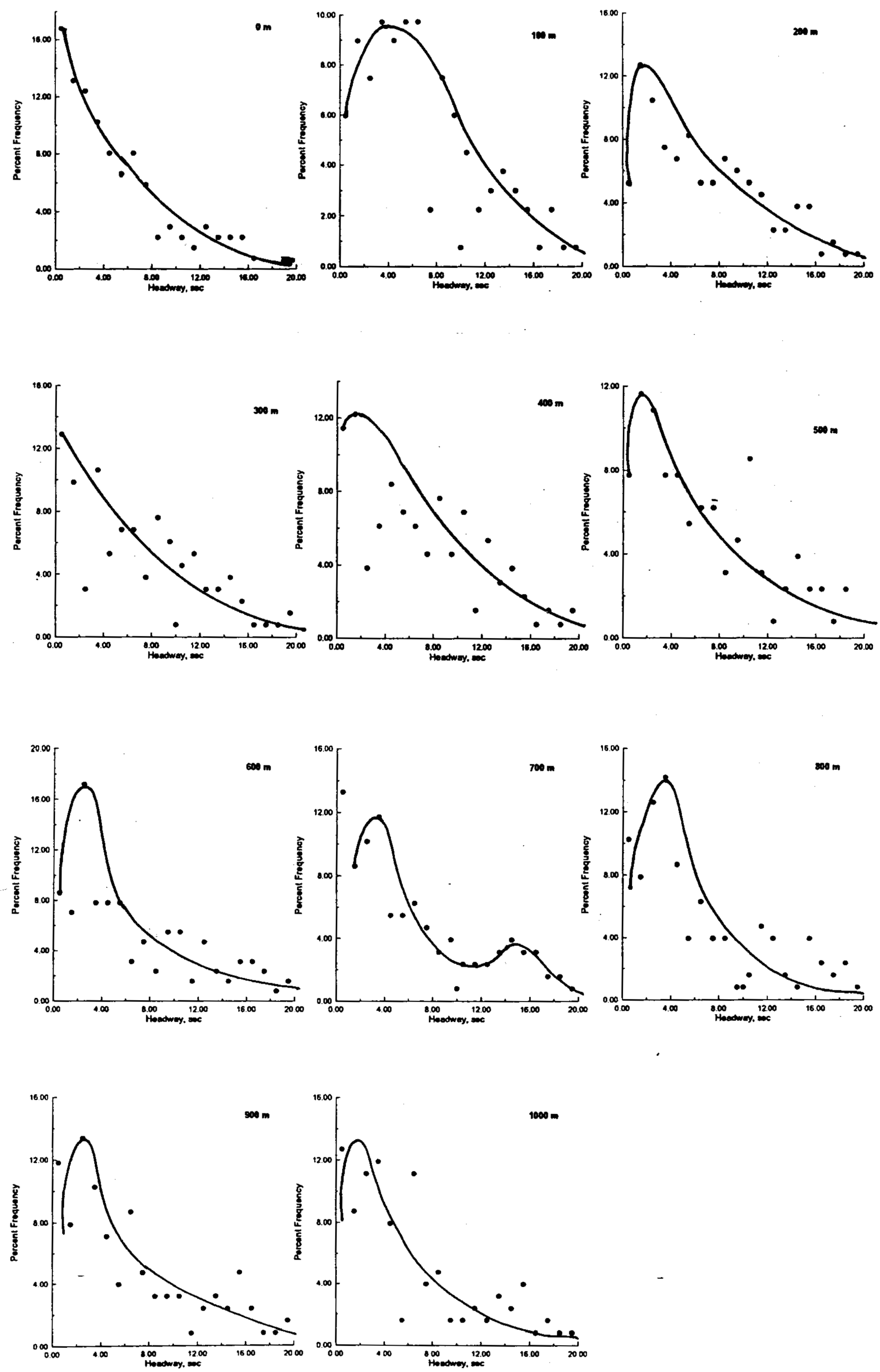

Fig. 7. 15 Percentage Frequency Distribution of Headway – 500 vph

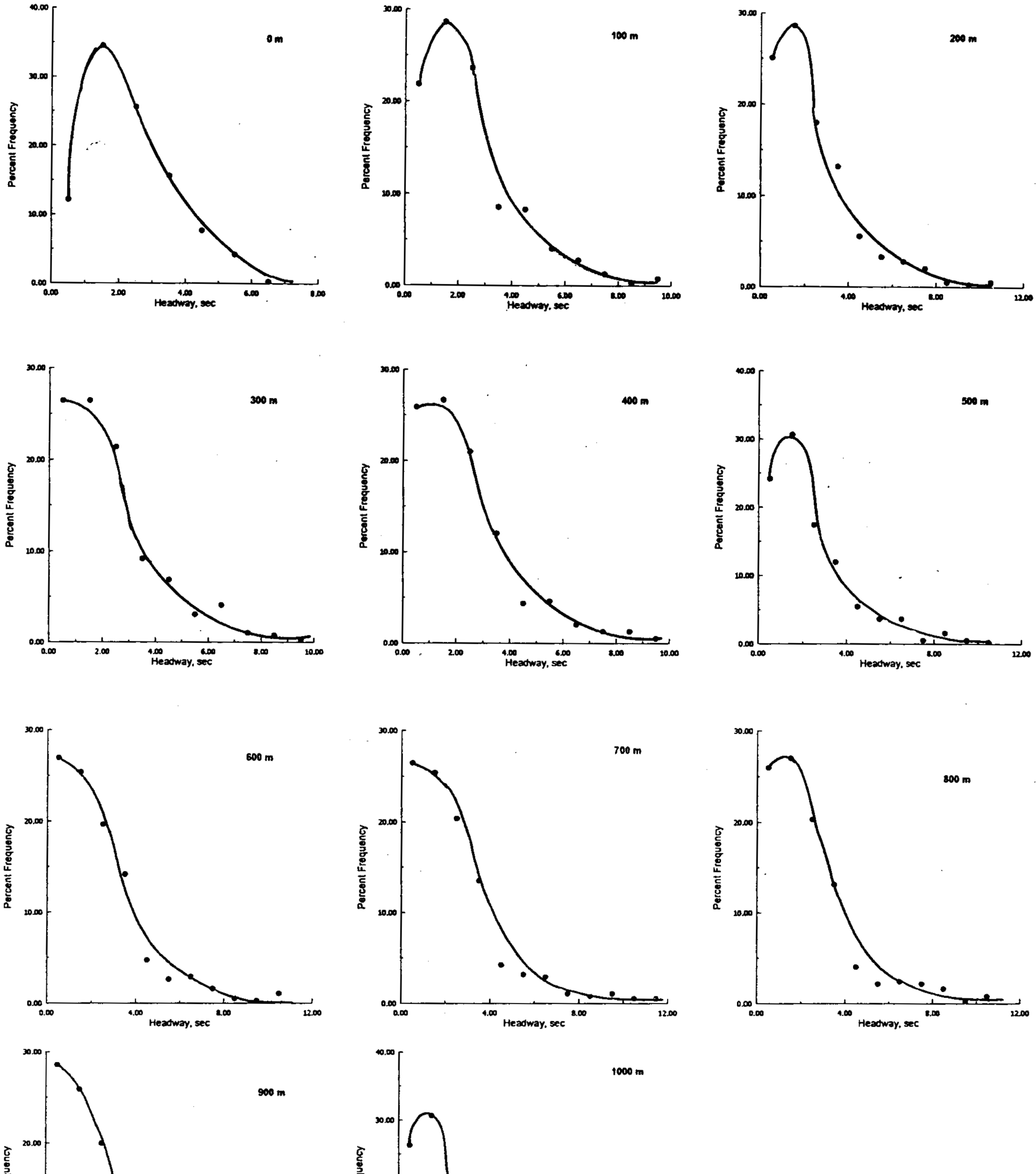

Fig. 7. 16 Percentage Frequency Distribution of Headway – 1500 vph

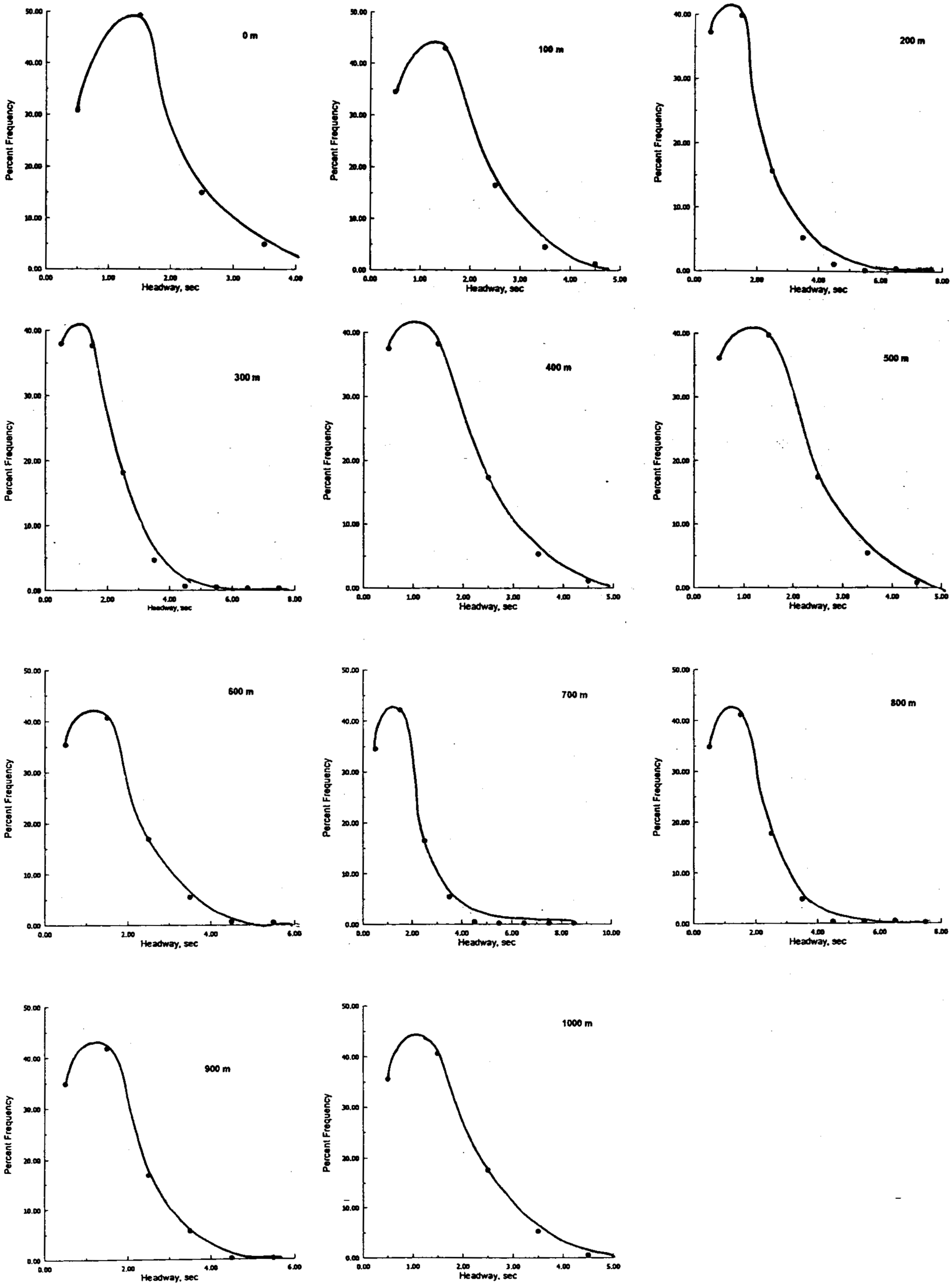

Fig. 7. 17 Percentage Frequency Distribution of Headway – 2500 vph

of normalised squared deviations that flow stabilises at 500 m from the entry point of the simulated stretch. Hence, the warm-up zone length was taken as 500 m and observations for the stream characteristics were taken over 500 – 1500 m stretch. The time required to traverse 1500 m length is approximately 3 minutes, hence, the warm up time was selected as 5 minutes.

Table 7.1 Sum of Normalised Squared Deviations

Section at, m	Volume, vph					
	500	1000	1500	2000	2500	4000
100	45.12	20.49	16.14	6.26	2.69	0.76
200	21.02	16.84	7.23	4.54	1.53	1.01
300	29.39	10.88	4.29	3.16	1.17	0.83
400	20.41	11.66	5.08	3.67	0.34	0.73
500	21.03	6.83	3.74	1.25	0.22	0.36
600	21.19	6.36	4.49	1.66	0.25	0.27
700	21.00	8.56	2.19	2.26	0.29	0.22
800	21.73	7.10	4.09	2.00	0.34	0.29
900	16.78	7.13	6.32	2.13	0.34	0.03
1000	13.13	8.82	3.60	2.05	0.34	0.07

7.9 VALIDATION OF THE MODEL

Validation is the process of checking the reality of the model in representing the system under study. The three board types of validity generally applied to traffic flow models are the predictive, the structural and the elemental validity.

Predictive validity means that the model is capable of predicting outcomes with an acceptable degree of accuracy, i.e. there should be statistical concordance between predicted and real values. Structural validity means that the model structure is a valid representation of the traffic flow process. The ability of the model to predict outcomes with reasonable accuracy is generally taken as evidence of structural validity. Elemental validity applies to specific elements or components of the model. It is particularly relevant to the theoretical aspects of a model's formulation, or to simplifying assumptions made. Elemental validity helps furthering the state-of-art at the theoretical level. In simulation modelling, elemental validity and predictive validity are the two main things with which the developer has to be concerned.

In the present work, the elemental validation was done by comparing the outcomes of the component models with corresponding field values. The field data collected for development of various component models was split into two parts, one part for calibration of the model and the other part for validation. In many cases, it was found that the model values were in close agreement with actual values.

The vehicle movement logic was validated making use of the display of the simulated vehicles provided by the graphics output module. In the absence of such a device, the modeller will be unaware of the unwanted things, like, more than one vehicle occupying the same position, vehicles coming very close to each other, etc., taking place in the simulated system. With the help of graphics display, it was possible to observe each and every vehicle as it passed through the study section for any inconsistencies. Many hours of time were spent in front of the monitor watching the display and modifications were made to the model as and when found necessary.

Predictive validation was carried out in two stages. Firstly, it was checked whether the model is reproducing the general trends observed in real life situation. For instance, it is generally accepted that the average stream speed reduces with increase in vehicular volume or density. For this purpose, the model was run by varying the input volume from 100 vph to 5000 vph for cars only stream. The model outputs in the form of speed-density, speed-flow and flow-density plots were prepared and are given in Fig. 7.18. It could be observed that the model could reproduce the general trend as expected in reality.

The model was also validated by comparing the model outputs with the corresponding field data collected at two locations in Calicut city. The model outputs and the field values are presented in Table 7.2. It could be observed that the predicted values are not much different from actual values. The literature survey has revealed that there are no clear guidelines in regard to the allowable percentage error. Hence, the model was considered to be reasonably good in replicating the field conditions and hence accepted for further experimentation.

Table 7.2 Results of Validation

S.No.	Simulated Volume	Simulated Speed	Observed Volume	Observed Speed	Percentage difference in Speed
Location 1					
1	624	43.50	620	42.86	-1.49
2	720	41.73	723	41.82	0.22
3	984	37.56	983	38.28	1.88
4	1320	36.65	1339	37.2	1.48
5	856	41.20	864	40.39	-2.01
Location 2					
	912	39.52	910	40.12	1.51
	1248	39.23	1254	38.82	-1.03
	2124	36.39	2120	35.67	-1.98
	3084	31.74	3080	30.96	-2.47
	2484	35.61	2486	35.89	0.79
	2724	34.71	2729	33.98	-2.09

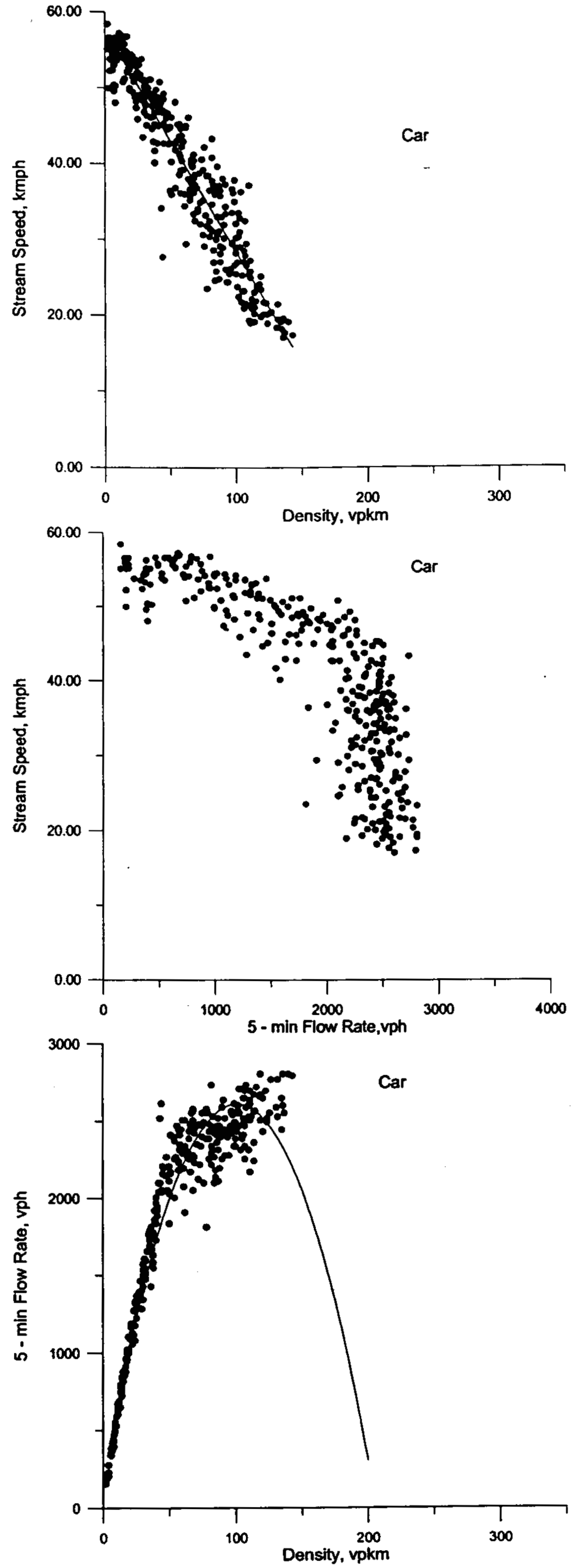

Fig 7.18 Speed – Density - Flow Plots

7.10 CONCLUSIONS

Development of the neural integrated simulation model by integrating the models for various components has been described in this chapter. The simulation model developed in 'C' can simulate the unidirectional traffic flow with options to vary the traffic flow composition, width of road, length of simulated stretch. The model gives steam speed, desired speed of stream, average flow rate, average density, percentage composition, headways, etc as the output. The model provides an visual display of the vehicles as they move through the simulated stretch and this feature was found to be very helpful in observing the working of the simulation model for any inconsistencies. The simulation model was validated by comparing the simulation outputs with values from field observation. The model thus developed is intended to be used for conducting experiments with the aim of understanding the behaviour of mixed traffic and the details of the experiments and analysis of results are presented in the coming chapter.

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CHAPTER-8

EXPERIMENTS USING SIMULATION MODEL

8.0 GENERAL

Development of Neural network integrated model for simulating mixed traffic flow on a unidirectional road section was presented in the previous chapters. In order to understand the behaviour of the mixed traffic under different flow conditions and compositions and thus to characterise the mixed traffic it is necessary to conduct experiments using the developed simulation model. This chapter describes the experiments conducted with simulation models, analysis of simulation output and the results.

8.1 EXPERIMENTAL FRAMEWORK

The stream characteristics depend on the traffic volume and composition of traffic stream. In order to understand the behaviour of mixed traffic a series of experiments were conducted on the simulation model by varying the volume and the composition. For a given input volume repeated experiments were conducted by varying the random number sequences used for each of the stochastic processes. This procedure has been adopted to simulate various possible scenarios for the specified set of conditions. The characteristics observed included the speed, the density and the flow. The width of road was kept constant at 7.0 m. The observations for the stream speed, the flow rate and the density were taken after a warm up time of five minutes and considering a warm up zone of length 500 m. The sampling period was taken as 5 minutes. The density values were calculated as average of counts of the vehicles present in the stretch from 500 m to 1500m taken at every 10 sec interval during the sampling period. Flow rate was calculated by counting the number of vehicles passing 1000 m point during the sampling period. Space mean speed was taken as the stream speed. Experiments were conducted under two categories.

(i) Experiments with only one class of vehicles in the stream.

(ii) Experiments with varying mix of various classes of vehicles.

8.2 CHARACTERISTIC STUDY OF SINGLE VEHICLE STREAMS

Experiments were conducted using the simulation model with only one class of vehicles at a time and by varying input volume. Details of experiments conducted with one class of vehicles in the stream are presented in Table 8.1.

Table 8.1 Details of Experiments with One Class of Vehicles in Stream

S. No.	Class of Vehicles in the Stream	Input Flow range
1	Car	0 - 6000
2	Bus	0 - 4000
3	Two-Wheeler	0 -10000
4	Auto-Rickshaw	0 - 8000

The speed – density, the speed – flow and the flow – density plots were prepared for car, two wheeler, auto-rickshaw and bus streams and are presented in Fig.

8.1 – 8.4 respectively. It can be observed from these plots that the speed decreases with increase in density. In case of cars only stream, the maximum flow rate observed was around 2800, even though, the input volume was varied up to 6000. In case of other vehicles also, similar observations were made, which indicates that even if large number of vehicles are generated, the number of vehicle that could pass through the observation section will not cross some maximum value. This is because the vehicles, as they pass through the warm up zone, get adjusted and reach stable flow condition. Hence, it is not possible to observe very high density or flow values in simulation modelling.

Speed – density relationship has an important role in the study of traffic stream characteristics. For a homogeneous flow condition, the speed is likely to be inversely proportional to density with a linear relationship. With a view to identify the relationship between the speed and density values, regression analysis was carried out. Various forms of models, as given in Table 8.2, were tried by regression analysis and the R^2-values for different model forms are given in Table 8.3. Of the various forms of models tried, the performance of linear model was fund to be good for most of the cases. The increase in R^2-value for 2nd and 3rd order polynomial models is very marginal. Hence, linear speed – density model has been selected for further study. The regression parameters and the goodness of fit statistics are presented in Table 8.4.

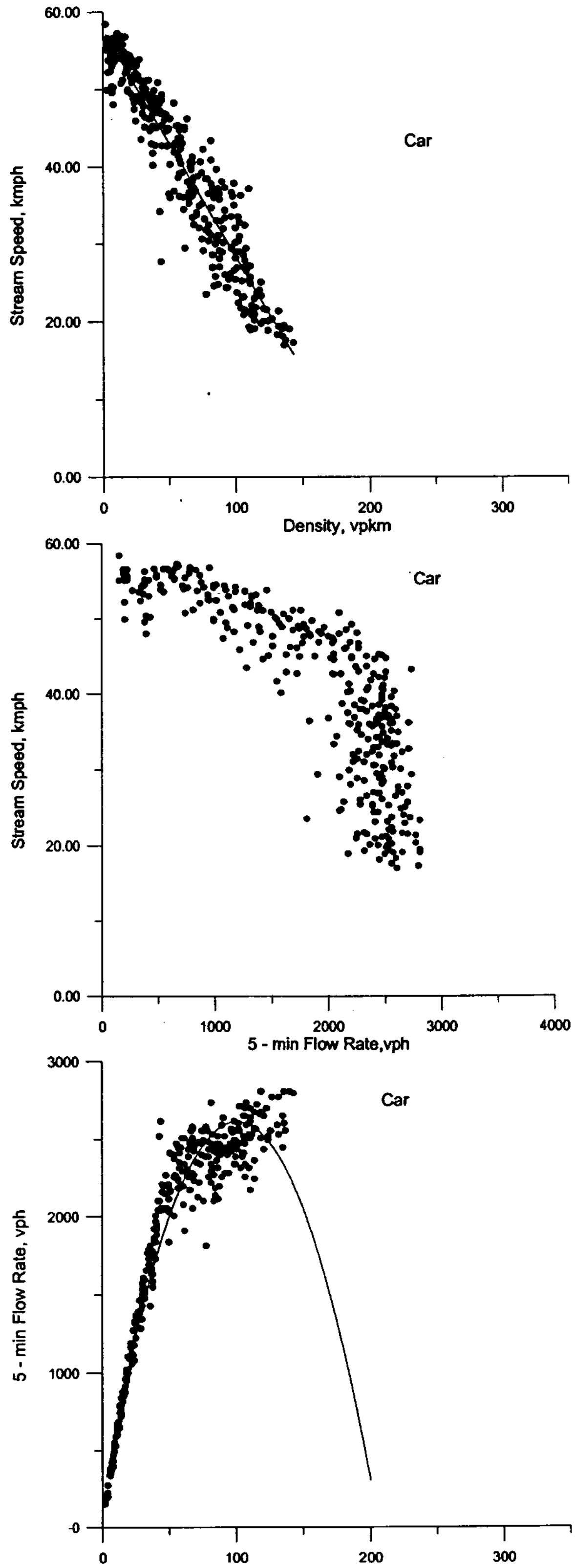


Fig 8.1 Speed – Density – Flow Plots for Car only Stream

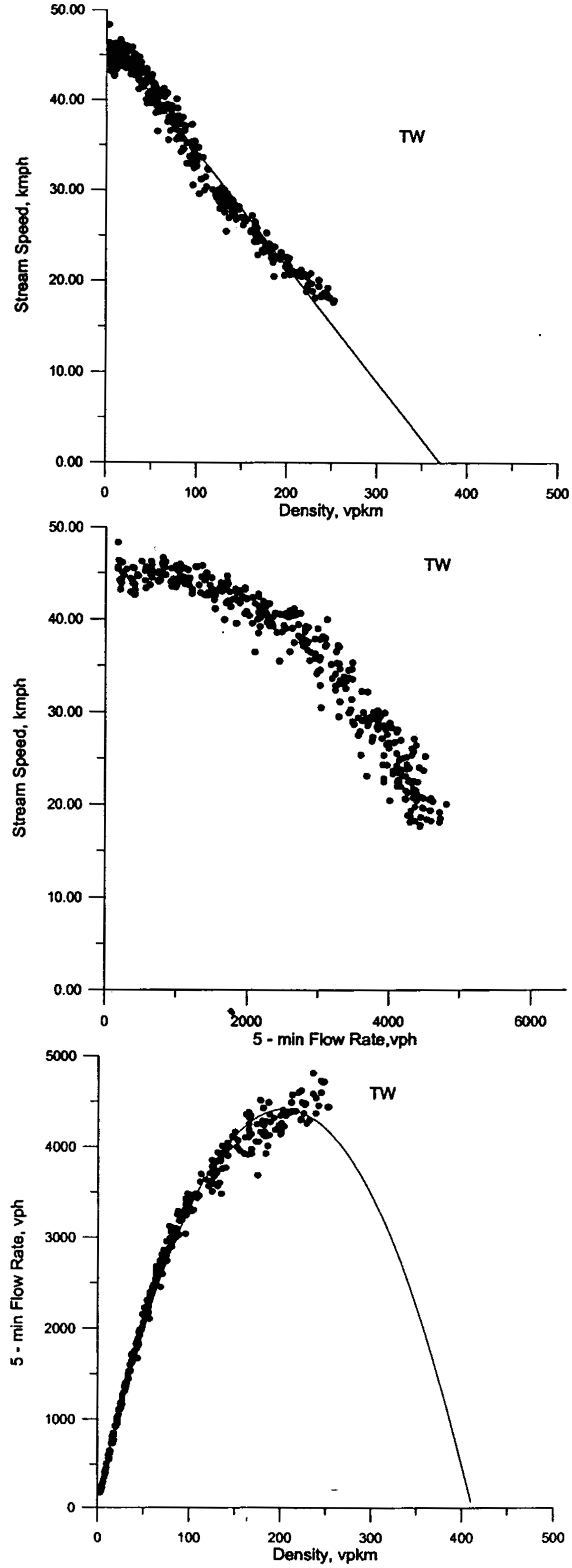


Fig 8.2 Speed – Density – Flow Plots For Two-Wheeler Only Stream

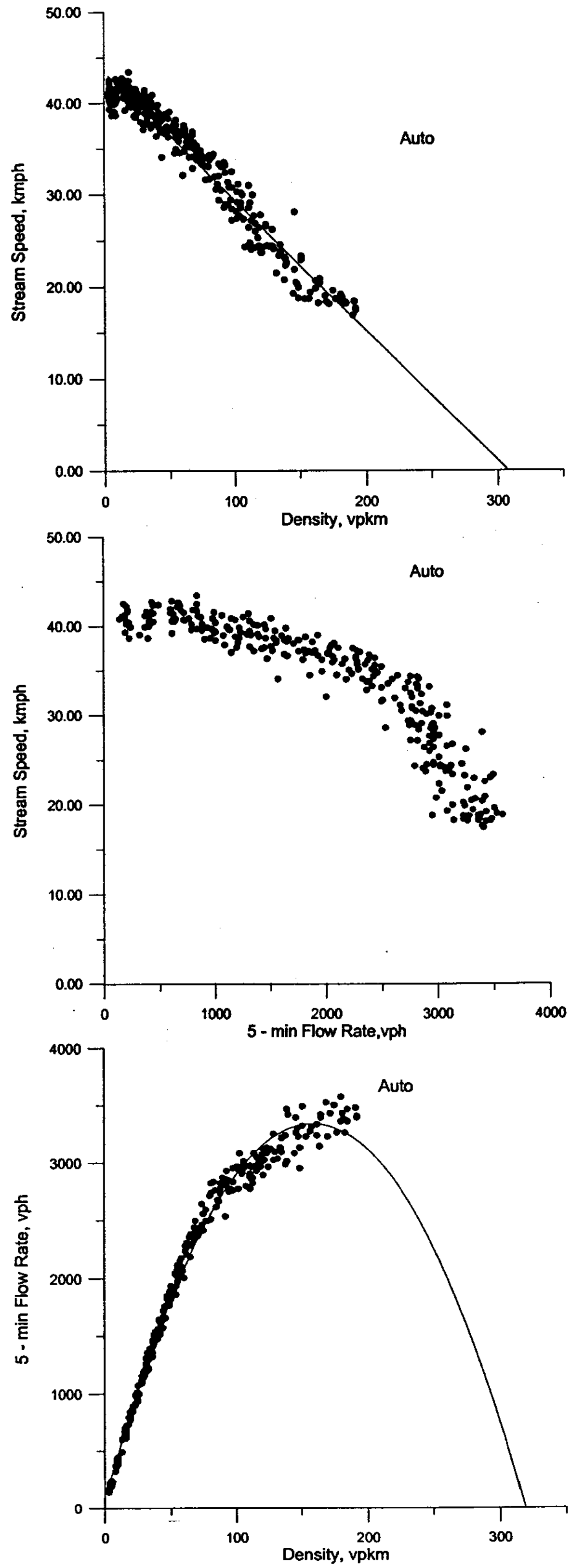


Fig 8.3 Speed – Density – Flow Plots for Auto-rickshaw only Stream

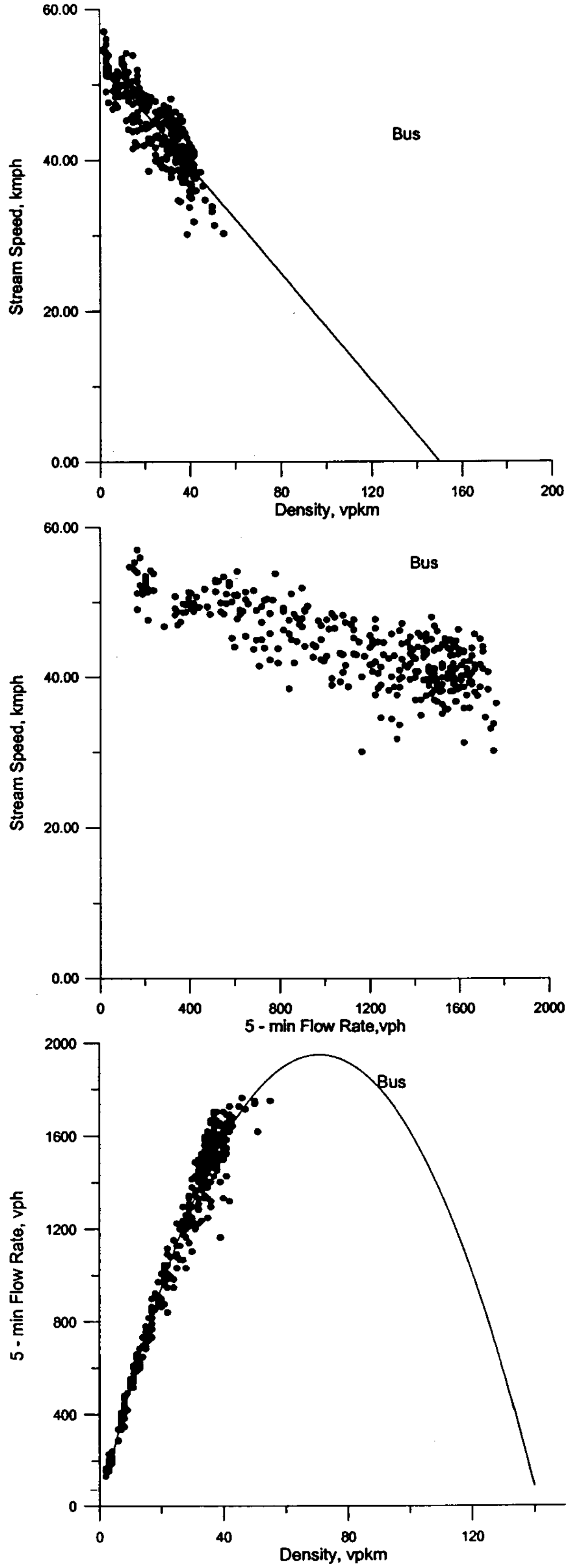


Fig 8.4 Speed – Density – Flow Plots for Bus only Stream

Table 8.2 Forms of Models Tried

Model No.	Model Name	Model Form
1	Linear	$u = a_0 + a_1 k$
2	Polynomial – 2nd order	$u = a_0 + a_1 k + a_2 k^2$
3	Polynomial – 3rd order	$u = a_0 + a_1 k + a_2 k^2 + a_3 k^3$
4	Logarithmic	$u = a_0 + a_1 \ln(k)$
5	Power	$u = a_0 k^{a_1}$
6	Exponential	$u = a_0 e^{(a_1 k)}$

Table 8.3 R^2- values for Different Forms of Models Tried

S.No.	Class of Vehicles	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
1	Car	0.908	0.908	0.909	0.727	0.640	0.877
2	Two-Wheeler	0.973	0.975	0.983	0.760	0.706	0.977
3	Auto-Rickshaw	0.954	0.956	0.967	0.701	0.640	0.938
4	Bus	0.770	0.773	0.779	0.686	0.645	0.749

8.4 Results of Regression Analysis

S.No.	Class of Vehicles	Speed (u)– Density (k) Relationship	R^2 - value	F – Test value	t- test value
1	Car	u = 57.66 – 0.2931 * k	0.908	3405.06	-58.35
2	Two-Wheeler	u = 46.87 – 0.1265 * k	0.973	13302.62	-115.34
3	Auto-Rickshaw	u = 43.45 – 0.1413 * k	0.953	6128.66	-78.29
4	Bus	u = 53.63 – 0.3804 * k	0.770	1135.58	-33.70

This indicates that the linear speed – density relationship is valid for single vehicle streams. The linear speed – density model is of the form as suggested by Greenshields and is given below.

$$u = u_f \left(1 - \frac{k}{k_j}\right) \qquad \text{Eqn. 8.1}$$

where u_f - Free flow speed, kmph

k_j – Jam density, vpkm

k – Density, vpkm

The linear speed – density models derived for various classes were used to obtain the stream characteristics such as free flow speed, jam density and capacity flow.

The capacity flow was calculated using the equation 8.2.

$$q_c = \frac{u_f k_j}{4} \qquad \text{Eqn. 8.2}$$

The values of free flow speed, jam density and capacity flow for various single vehicle streams are given in Table 8.5. The Passenger Car Unit (PCU) values for different classes of vehicles at capacity flow are calculated as the ratio of capacity flow for cars only stream to the capacity flow for class of vehicles under consideration. The estimated PCU values for different classes of vehicles are given in Table 8.5.

Table 8.5 Characteristics of Single Vehicle Streams

S.No.	Class of Vehicles	Free Flow Speed, u_f	Jam Density, k_j	Capacity Flow, q_c	PCU Value
1	Car	57.66	196	2825	1.00
2	Bus	53.63	141	1890	1.49
3	Two-Wheeler	46.87	370	4335	0.65
4	Auto-Rickshaw	43.45	307	3335	0.85

Even though there are wide variations in PCU values reported by different researchers, the trend of PCU values derived in this study closely match with that of the study reported by Nagaraj, et al (1990). It is observed in this study that the capacity of two lane roads for cars only stream is 2825. This appears to be less than the value that one can infer if they were to be influenced by HCM value of 2000 passenger cars per lane. But things are quite different between these two comparisons. The speed Vs flow comparisons in these situations are very much different. The free flow speed in the ideal car flow situation reported by HCM is almost double the free flow speed observed in our conditions. The jam density on the other hand is very much nearly the same. Thus, the slope of the speed – density relationship in HCM studies is very much higher and capacity flow occurs at much higher speed. Therefore, the HCM capacity values will be more compared to capacity flow in our conditions.

The speed – flow plots superimposed with speed-flow curve obtained using derived speed – density model and the fundamental equation of traffic flow were prepared. These are presented in Fig. 8.5 to 8.8 for car, two-wheeler, auto-rickshaw and bus streams respectively. It could be observed from these plots that most of the data points obtained from simulation experiments lie within a reasonable band about the speed-flow curve. The various Levels – of –Service indicated are as per SP-165.

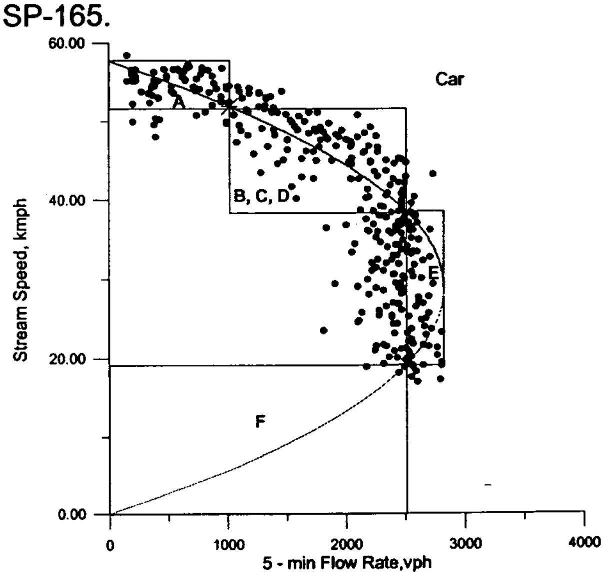


Fig 8.5 Speed Vs Flow Diagram – Car only

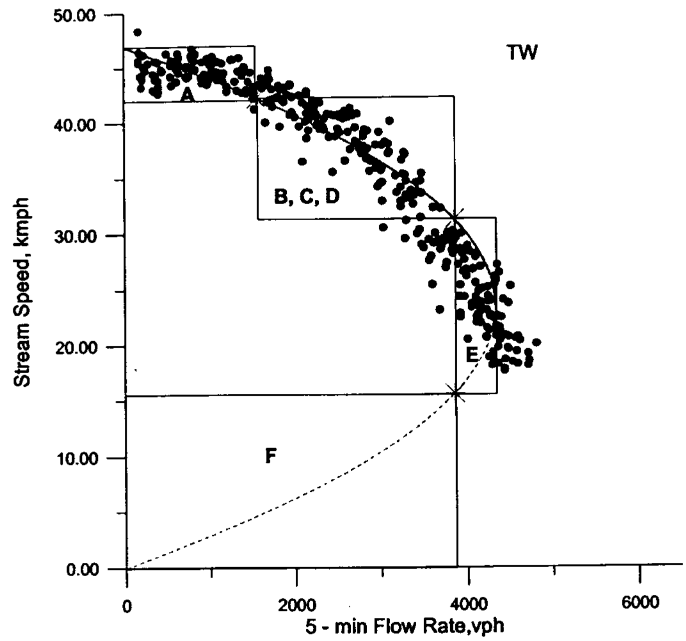


Fig 8.6 Speed Vs Flow Diagram – Two-wheeler only

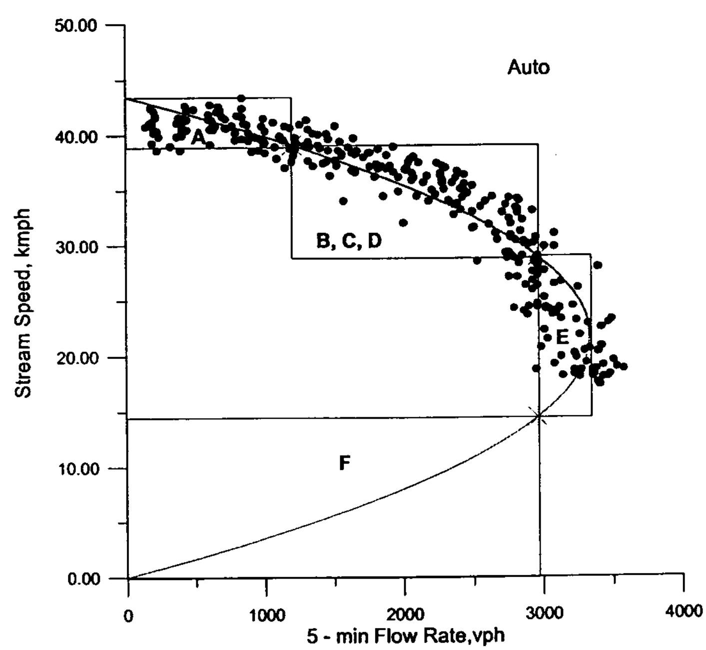


Fig 8.7 Speed Vs Flow Diagram – Auto-rickshaw only

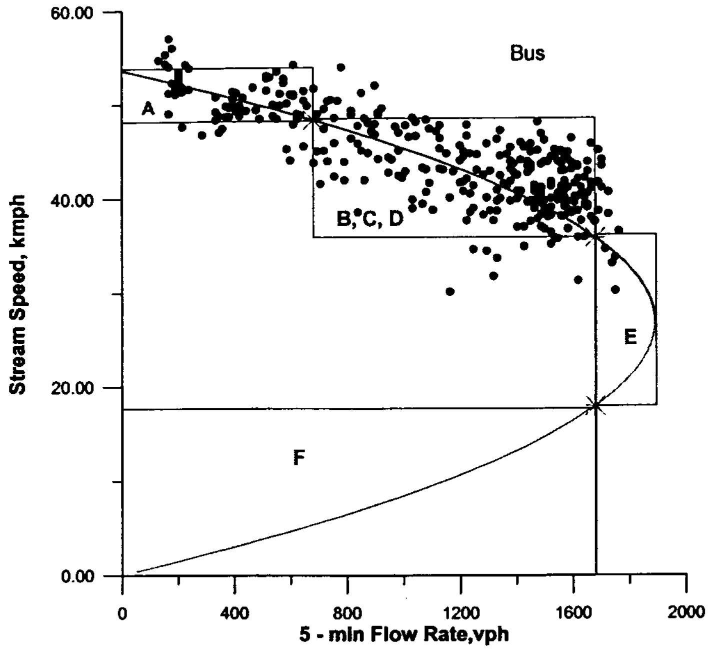


Fig 8.8 Speed Vs Flow Diagram – Bus only

8.3 EXPERIMENTS WITH DIFFERENT MIX PROPORTIONS

To study the behaviour of mixed traffic under a variety of traffic volumes and composition of various classes of vehicles, experiments were conducted using the simulation model developed. The proportions of four classes of vehicles, viz., car, two-wheeler, bus and auto-rickshaw were varied. As the proportions of different classes of vehicles vary depending on the area type, i.e., urban, semi urban and rural, various mix proportions were tried. Totally, experiments were conducted with 51 different mix proportions as indicated in the structure shown in Fig. 8.9. The various mix proportions considered for conducting experiments using simulation model are given Table 8.6. As it can be seen from Table 8.6, percentage of cars, two-wheelers, buses and auto-rickshaws was varied from 10-40, 0-50, 10-40 and 0-50 respectively. The input volume was varied from 0 to 8000. The output of simulation runs included the proportions of different classes of vehicles, flow, stream speed, desired speed of the stream, density.

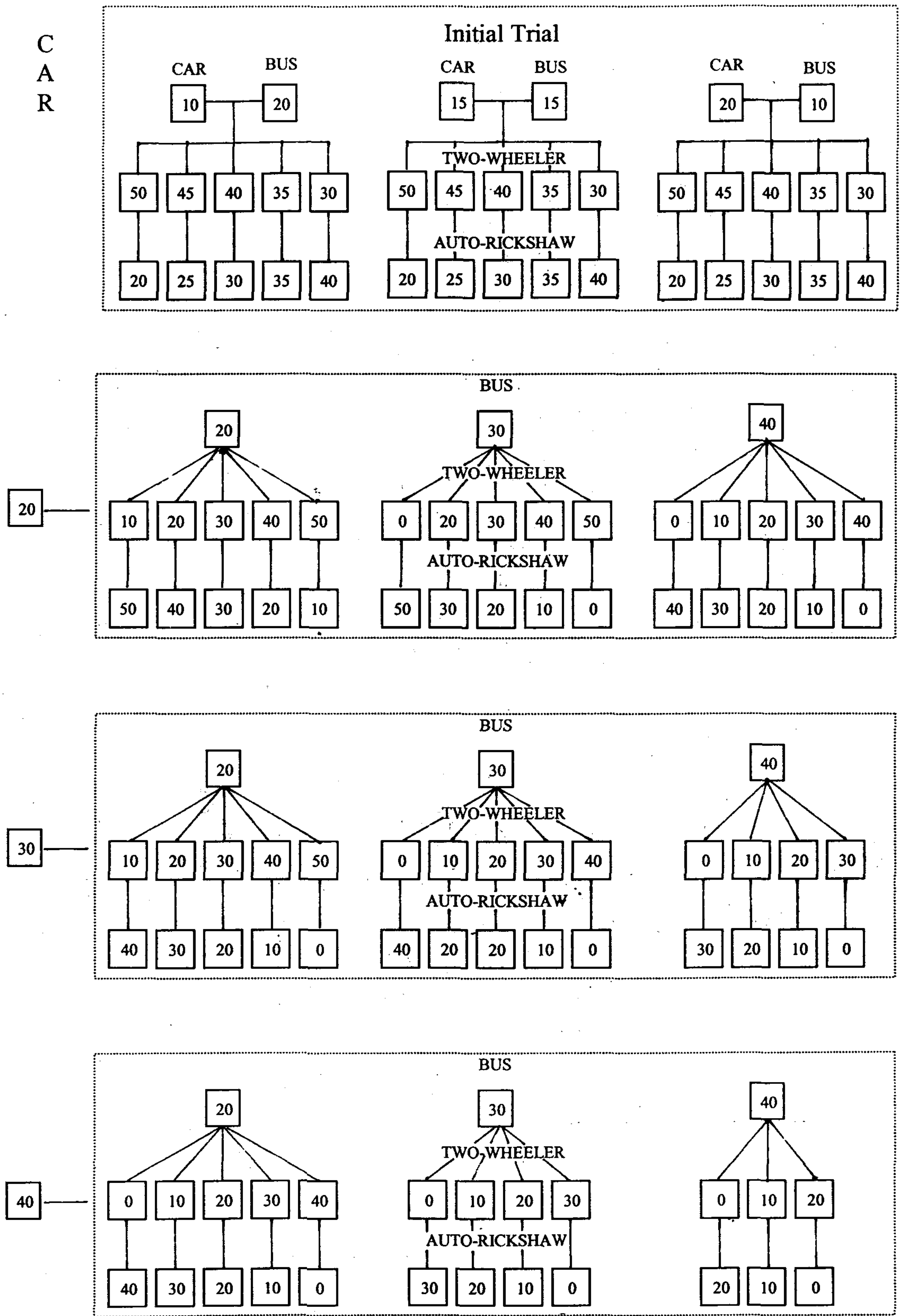


Fig 8.9 Mix Combinations Experimented

Table 8.6 Different Mix Proportions Considered for Experimentation

S. No.	Mix Code	Percentage of			
		Car	Two-wheeler	Auto-rickshaw	Bus
1	1 – 1	10	50	20	20
2	1 – 2	10	45	25	20
3	1 – 3	10	40	30	20
4	1 – 4	10	35	35	20
5	1 – 5	10	30	40	20
6	3 – 1	20	50	20	10
7	3 – 2	20	45	25	10
8	3 – 3	20	40	30	10
9	3 – 4	20	35	35	10
10	3 – 5	20	30	40	10
11	4 – 1	20	10	50	20
12	4 - 2	20	20	40	20
13	4 - 3	20	30	30	20
14	4 - 4	20	40	20	20
15	4 - 5	20	50	10	20
16	5 - 1	20	0	50	30
17	5 - 2	20	20	30	30
18	5 - 3	20	30	20	30
19	5 - 4	20	40	10	30
20	5 - 5	20	50	0	30
21	6 - 1	20	0	40	40
22	6 - 2	20	10	30	40
23	6 - 3	20	20	20	40
24	6 - 4	20	30	10	40
25	6 - 5	20	40	0	40
26	7 - 1	30	10	40	20
27	7 - 2	30	20	30	20
28	7 - 3	30	30	20	20
29	7 - 4	30	40	10	20

30	7 - 5	30	50	0	20
31	8 - 1	30	0	40	30
32	8 - 2	30	10	30	30
33	8 - 3	30	20	20	30
34	8 - 4	30	30	10	30
35	8 - 5	30	40	0	30
36	9 - 1	30	0	30	40
37	9 - 2	30	10	20	40
38	9 - 3	30	20	10	40
39	9 - 4	30	30	0	40
40	10 - 1	40	0	40	20
41	10 - 2	40	10	30	20
42	10 - 3	40	20	20	20
43	10 - 4	40	30	10	20
44	10 - 5	40	40	0	20
45	11 - 1	40	0	30	30
46	11 - 2	40	10	20	30
47	11 - 3	40	20	10	30
48	11 - 4	40	30	0	30
49	12 - 1	40	0	20	40
50	12 - 2	40	10	10	40
51	12 - 3	40	20	0	40

Experiments with Mix Codes 2-1, 2-2, 2-3, 2-4 and 2-5 were also completed but not reported in the Table.

The speed – density, the speed – flow and the flow – density plots were prepared for all the experimented mixes of traffic streams. The plots given in Fig. 8.10, 8.11 and 8.12 correspond to the mixes 4-3, 8-3 and 12-3 respectively. Mix 4-3, with percentage of cars, two-wheelers, auto-rickshaws and buses being 20, 30, 30 and 20 respectively, represent typical urban conditions, Mix 8-3, with percentage of cars, two-wheelers, auto-rickshaws and buses being 30, 20, 20 and 30 respectively, represents typical semi urban conditions and Mix 12-3, with

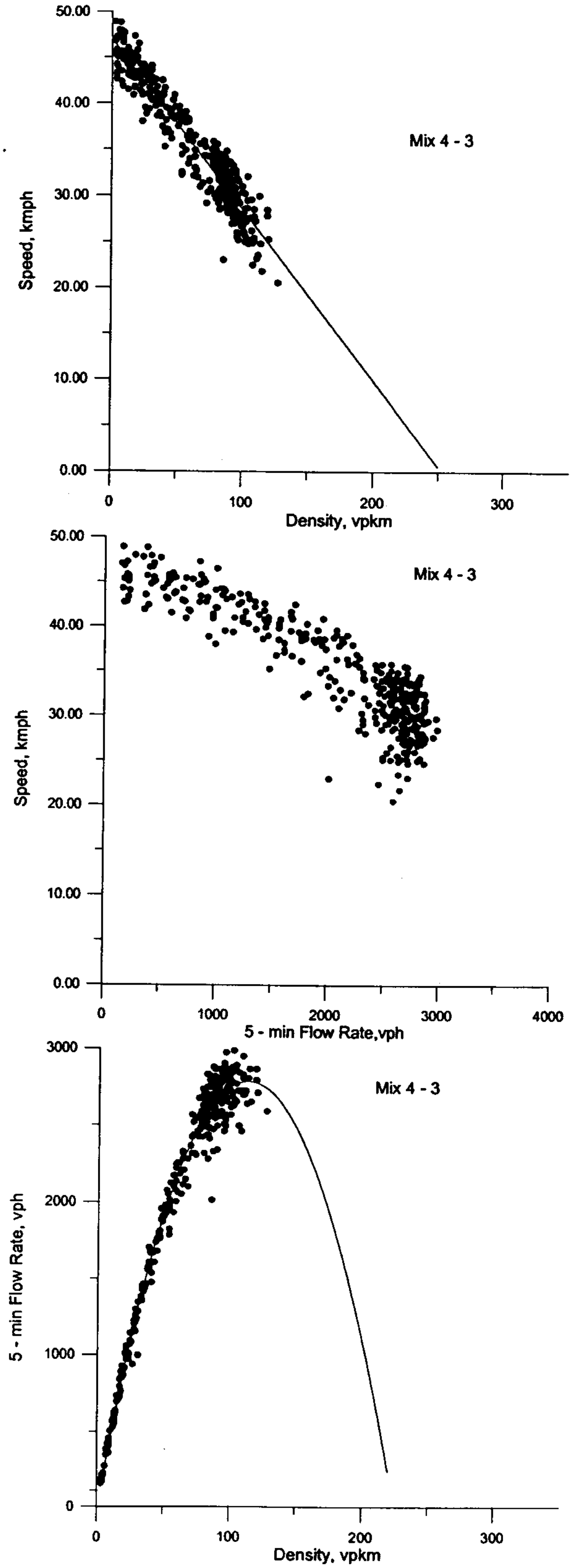


Fig 8.10 Speed – Density – Flow Plots for Car 20% - TW 30% - Auto 30% - Bus 20% Mix

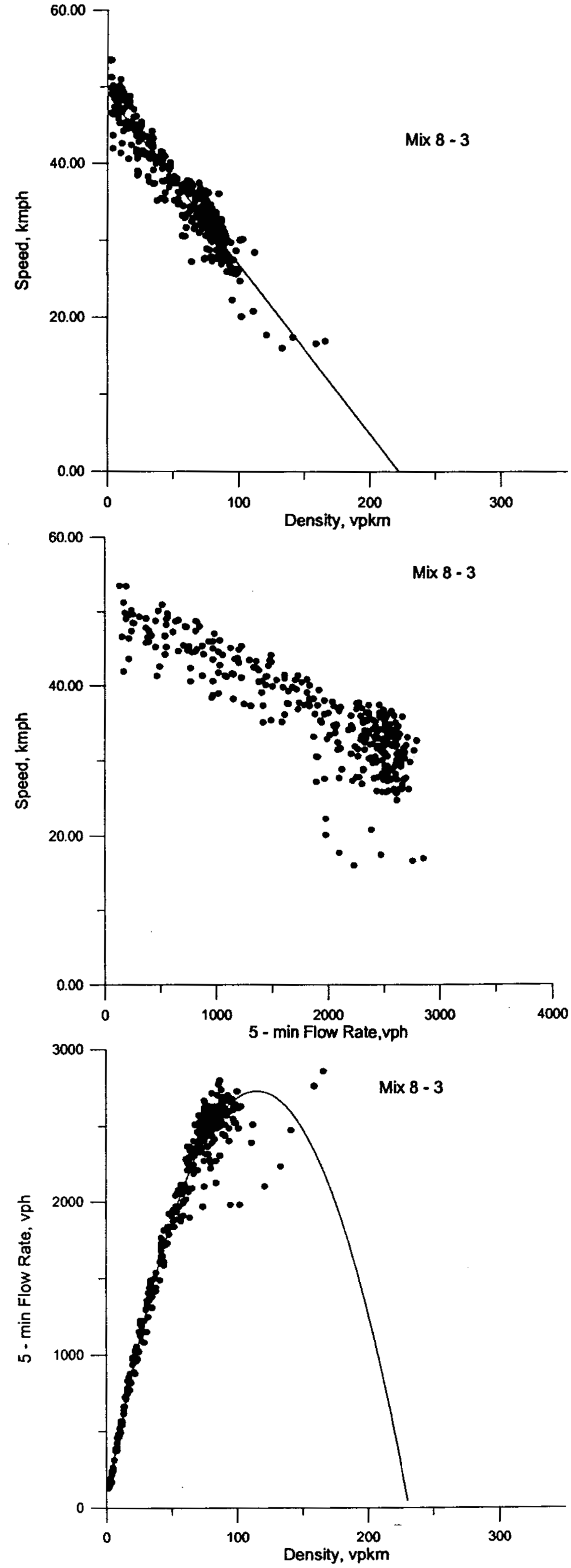
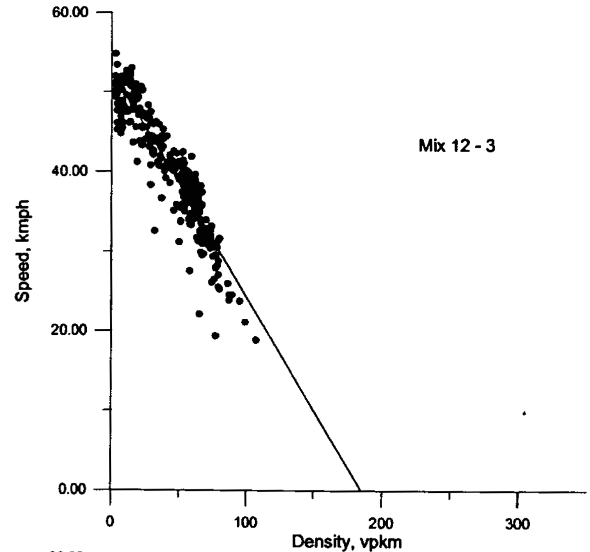
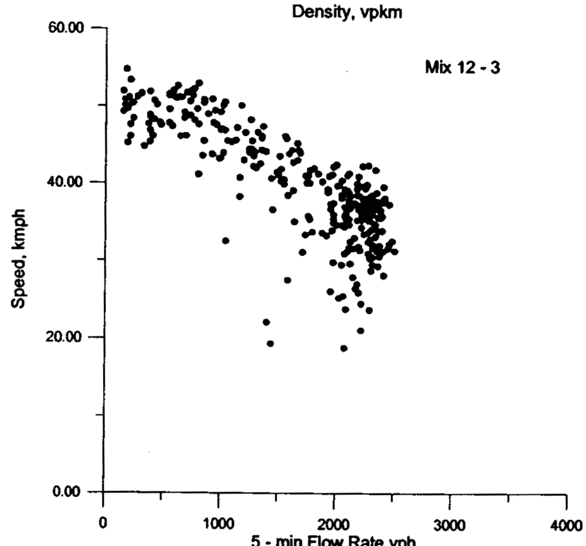


Fig 8.11 Speed – Density – Flow Plots for Car 30% - TW 20% - Auto 20% - Bus 30% Mix





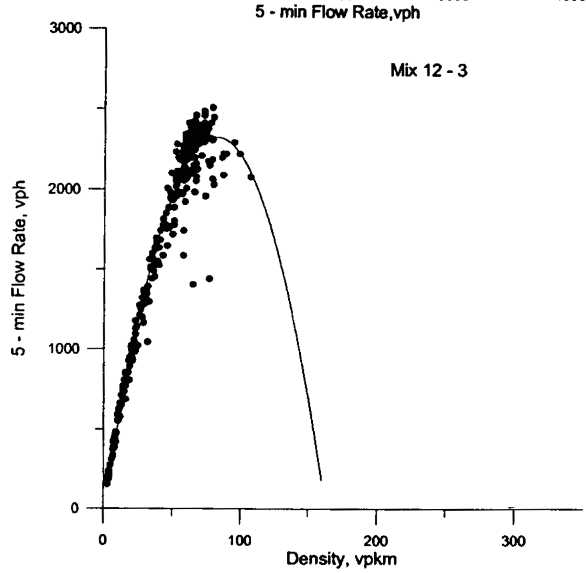


Fig 8.12 Speed – Density – Flow Plots for Car 40% - TW 20% - Auto 0% - Bus 40% Mix

percentage of cars, two-wheelers, auto-rickshaws and buses being 40, 20, 0 and 40 respectively, represents typical rural conditions. Regression analysis was carried to identify the relationship between the stream speed and density for each of the mixes separately. Models of various forms, as presented in Table 8.2 were tried. In almost all the cases the relationship between speed and density was found to be linear. This indicates that even in mixed traffic flow, the speed – density relationships are linear and hence, the speed – flow relationships are parabolic. However, there is a band within which the data points get scattered due to several reasons. Also, that the simulation approach does not support multi-regime models. The values of regression parameters and R^2 are given in Table 8.7.

Table 8.7 Results of Regression Analysis

S.No.	Mix Code	%Car-%TW-%Auto-%Bus	Intercept Constant	Regression Coefficient	R^2-value
1	1 – 1	10-50-20-20	46.50	-0.183	0.940
2	1 – 2	10-45-25-20	46.28	-0.180	0.959
3	1 – 3	10-40-30-20	45.50	-0.176	0.936
4	1 – 4	10-35-35-20	45.45	-0.188	0.944
5	1 – 5	10-30-40-20	45.46	-0.183	0.948
6	3 – 1	20-50-20-10	46.50	-0.173	0.960
7	3 – 2	10-45-25-20	45.59	-0.162	0.958
8	3 - 3	20-40-30-10	45.50	-0.165	0.960
9	3 - 4	20-35-35-10	45.15	-0.163	0.959
10	3 – 5	20-30-40-10	45.22	-0.168	0.937
11	4 – 1	20-10-50-20	46.02	-0.184	0.913
12	4 - 2	20-20-40-20	46.68	-0.186	0.908
13	4 - 3	20-30-30-20	47.03	-0.186	0.911
14	4 - 4	20-40-20-20	47.97	-0.193	0.914
15	4 - 5	20-50-10-20	48.73	-0.196	0.927
16	5 - 1	20-0-50-30	47.06	-0.222	0.914

17	5 - 2	20-20-30-30	47.40	-0.211	0.896
18	5 - 3	20-30-20-30	48.65	-0.221	0.906
19	5 - 4	20-40-10-30	49.44	-0.225	0.899
20	5 - 5	20-50-0-30	50.58	-0.230	0.931
21	6 - 1	20-0-40-40	47.53	-0.241	0.864
22	6 - 2	20-10-30-40	47.77	-0.238	0.866
23	6 - 3	20-20-20-40	49.48	-0.251	0.913
24	6 - 4	20-30-10-40	50.10	-0.253	0.907
25	6 - 5	20-40-0-40	51.32	-0.258	0.892
26	7 - 1	30-10-40-20	47.31	-0.205	0.901
27	7 - 2	30-20-30-20	47.65	-0.198	0.906
28	7 - 3	30-30-20-20	49.02	-0.215	0.931
29	7 - 4	30-40-10-20	49.68	-0.213	0.912
30	7 - 5	30-50-0-20	50.58	-0.216	0.931
31	8 - 1	30-0-40-30	47.76	-0.236	0.894
32	8 - 2	30-10-30-30	47.96	-0.224	0.863
33	8 - 3	30-20-20-30	48.91	-0.221	0.903
34	8 - 4	30-30-10-30	50.42	-0.226	0.921
35	8 - 5	30-40-0-30	51.43	-0.235	0.924
36	9 - 1	30-0-30-40	49.38	-0.268	0.891
37	9 - 2	30-10-20-40	49.68	-0.262	0.855
38	9 - 3	30-20-10-40	50.77	-0.266	0.888
39	9 - 4	30-30-0-40	51.27	-0.255	0.879
40	10 - 1	40-0-40-20	47.79	-0.220	0.901
41	10 - 2	40-10-30-20	48.58	-0.222	0.896
42	10 - 3	40-20-20-20	49.26	-0.229	0.908
43	10 - 4	40-30-10-20	50.26	-0.229	0.917
44	10 - 5	40-40-0-20	51.96	-0.240	0.929
45	11 - 1	40-0-30-30	49.21	-0.254	0.906
46	11 - 2	40-10-20-30	49.99	-0.255	0.901
47	11 - 3	40-20-10-30	51.56	-0.264	0.902
48	11 - 4	40-30-0-30	52.15	-0.260	0.897

49	12 - 1	40-0-20-40	51.18	-0.298	0.888
50	12 - 2	40-10-10-40	51.85	-0.292	0.897
51	12 - 3	40-20-0-40	52.62	-0.285	0.861

The stream parameters such as free flow speed (kmph), jam density (vpkm) and capacity flow (vph) for the various mixes were calculated using the regression models relating speed and density. These values are presented in Table 8.8.

Table 8.8 Traffic Flow Characteristics for various Stream compositions

S.No.	Mix Code	%Car-%TW-%Auto-%Bus	Free Flow Speed, u_f	Jam Density, k_j	Capacity Flow, q_c
1	1 – 1	10-50-20-20	46.50	254	2952
2	1 – 2	10-45-25-20	46.28	257	2973
3	1 – 3	10-40-30-20	45.50	258	2930
4	1 – 4	10-35-35-20	45.45	241	2738
5	1 – 5	10-30-40-20	45.46	249	3052
6	3 – 1	20-50-20-10	46.50	269	3127
7	3 – 2	10-45-25-20	45.59	281	3223
8	3 - 3	20-40-30-10	45.50	276	3140
9	3 - 4	20-35-35-10	45.15	277	3126
10	3 – 5	20-30-40-10	45.22	270	3052
11	4 – 1	20-10-50-20	46.02	250	2878
12	4 - 2	20-20-40-20	46.68	250	2918
13	4 - 3	20-30-30-20	47.03	253	2975
14	4 - 4	20-40-20-20	47.97	240	2975
15	4 - 5	20-50-10-20	48.73	248	3021
16	5 - 1	20-0-50-30	47.06	212	2494
17	5 - 2	20-20-30-30	47.40	225	2666
18	5 - 3	20-30-20-30	48.65	220	2675
19	5 - 4	20-40-10-30	49.44	220	2719
20	5 - 5	20-50-0-30	50.58	220	2782
21	6 - 1	20-0-40-40	47.53	197	2340
22	6 - 2	20-10-30-40	47.77	200	2388

23	6 - 3	20-20-20-40	49.48	197	2436
24	6 - 4	20-30-10-40	50.10	198	2480
25	6 - 5	20-40-0-40	51.32	199	2553
26	7 - 1	30-10-40-20	47.31	230	2720
27	7 - 2	30-20-30-20	47.65	240	2859
28	7 - 3	30-30-20-20	49.02	228	2794
29	7 - 4	30-40-10-20	49.68	233	2893
30	7 - 5	30-50-0-20	50.58	234	2958
31	8 - 1	30-0-40-30	47.76	202	2412
32	8 - 2	30-10-30-30	47.96	214	2565
33	8 - 3	30-20-20-30	48.91	221	2702
34	8 - 4	30-30-10-30	50.42	223	2810
35	8 - 5	30-40-0-30	51.43	218	2802
36	9 - 1	30-0-30-40	49.38	184	2271
37	9 - 2	30-10-20-40	49.68	189	2347
38	9 - 3	30-20-10-40	50.77	191	2424
39	9 - 4	30-30-0-40	51.27	201	2576
40	10 - 1	40-0-40-20	47.79	217	2592
41	10 - 2	40-10-30-20	48.58	218	2647
42	10 - 3	40-20-20-20	49.26	215	2647
43	10 - 4	40-30-10-20	50.26	219	2751
44	10 - 5	40-40-0-20	51.96	216	2805
45	11 - 1	40-0-30-30	49.21	193	2374
46	11 - 2	40-10-20-30	49.99	196	2450
47	11 - 3	40-20-10-30	51.56	195	2513
48	11 - 4	40-30-0-30	52.15	200	2607
49	12 - 1	40-0-20-40	51.18	171	2188
50	12 - 2	40-10-10-40	51.85	176	2281
51	12 - 3	40-20-0-40	52.62	185	2434

The speed – flow plots superimposed with speed-flow curve obtained using derived speed – density model and the fundamental equation of traffic flow were prepared and are shown in Fig 8.13, 8.14 and 8.15 for mixes 4-3, 8-3 and 12-respectively.

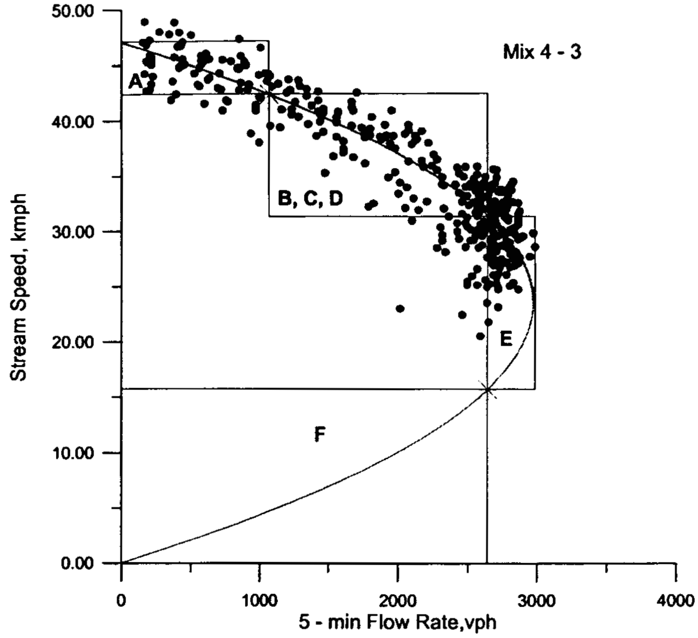


Fig 8.13 Speed Vs Flow Diagram – Mix 4-3

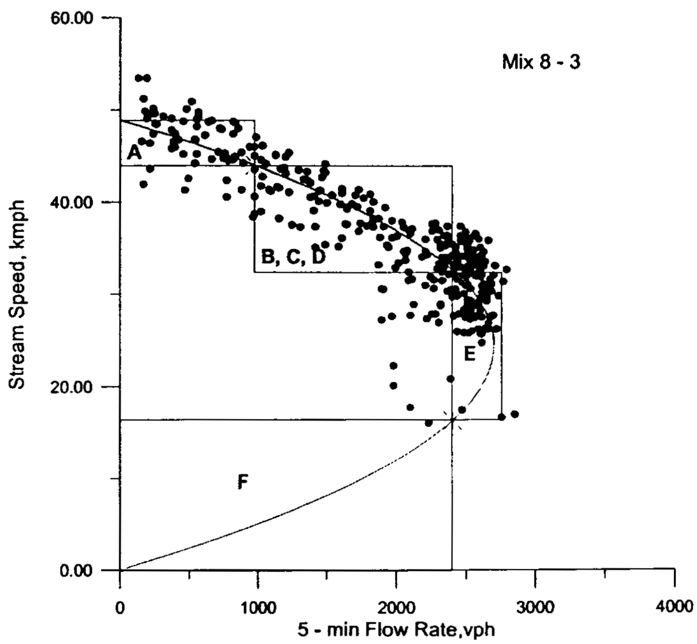


Fig 8.14 Speed Vs Flow Diagram – Mix 8-3

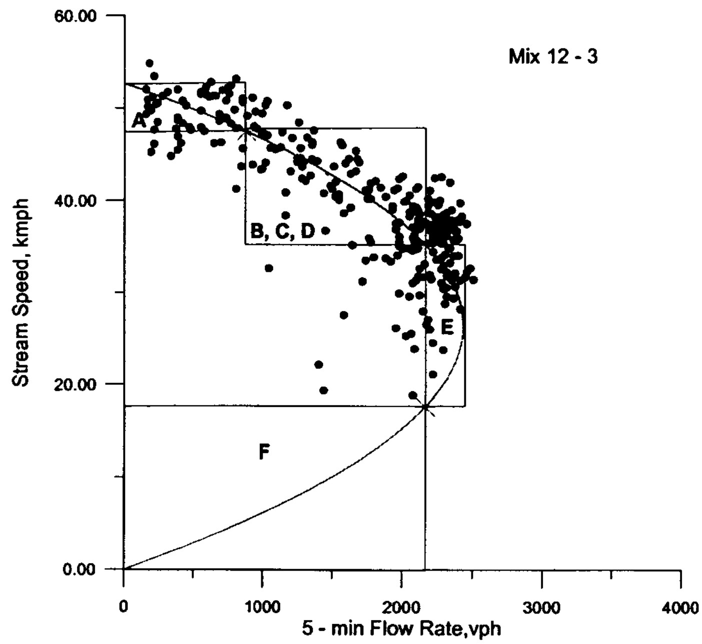


Fig 8.15 Speed Vs Flow Diagram – Mix 12-3

From Table 8.8, values for a few mixes are extracted and presented in Table 8.9. In all the mixes reported in this table, the total of percentages of two-wheelers and auto-rickshaws is maintained at 40. From this table it appears that the trend of free flow speed, jam density and capacity flow remains more or less identical for two-wheeler and auto-rickshaw combinations, except at higher percentage of two-wheelers in the stream.

Table 8.9 Variation in Stream Characteristics with changing proportions of Cars & Buses

%Car	%Bus	%TW	%Auto	Free flow speed	Jam Density	Capacity flow
20	40	0	40	47.53	197	2340
30	30	0	40	47.76	202	2412
40	20	0	40	47.79	217	2592
20	40	40	0	51.32	199	2553
30	30	40	0	51.43	218	2802
40	20	40	0	51.96	216	2805
20	40	10	30	47.77	200	2388

30	30	10	30	47.96	214	2565
40	20	10	30	48.58	218	2647
20	40	20	20	49.48	197	2436
30	30	20	20	48.91	221	2702
40	20	20	20	49.26	215	2647
20	40	30	10	50.10	198	2480
30	30	30	10	50.42	223	2810
40	20	30	10	50.26	219	2751

8.4 EFFECT OF AUTO-RICKSHAW ON STREAM CHARACTERISTICS

The effect of auto-rickshaws on stream characteristics was studied by comparing the results obtained through simulation experiments. For this purpose the graphs were prepared between the capacity flow and percent of auto for a given percent of two-wheelers. As experiments were conducted with percentage of two-wheelers being varied from 10 to 50, there are five graphs. These plots are shown in Fig. 8.16. Plots a, b, c, d and e show the capacity flow for 10%, 20%, 30%, 40% and 50% of two-wheelers respectively. Each curve in a graph corresponds to a particular percentage of cars in the stream. It can be seen from these plots that as the percentage of auto-rickshaws increase the capacity flow increases.

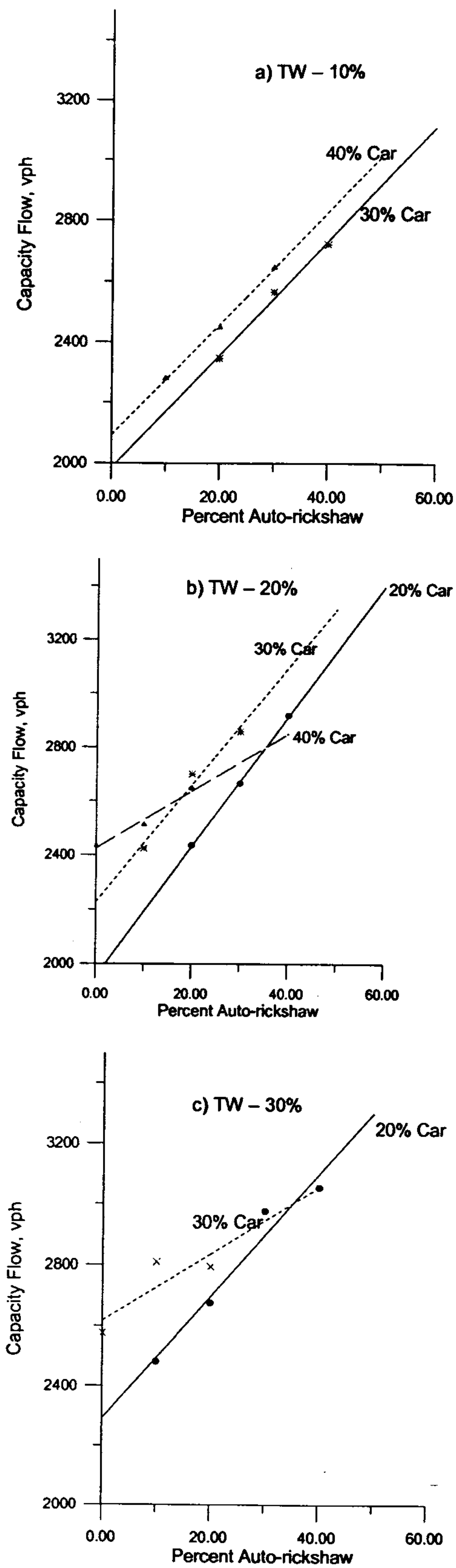
However, the graphs for capacity flow for different percentages of car and bus combinations are not parallel to each other. Fig 8.16(b) shows that 40% car capacity flow curve intersects both the 30% and 20% car capacity flow curves. This means identical capacities can be obtained for different car-bus combinations. For example, capacity flow of 2800 vph can be obtained at 20% TW, 35% Auto, 20% Car and 25% Bus combination, as well as 20% TW, 35% Auto, 40% Car and 5% Bus combination. This is equivalent to saying that 20%

increase in car percentage and 20% decrease in bus percentage will not alter the capacity.

Here lies the challenge in the mixed traffic flow research. At lower percentages of two-wheeler, cars and buses are able to independently influence the behaviour of stream. However, as the percentage of auto-rickshaw and two-wheelers increase, the vehicles like cars and buses loose their freedom and the potential effect of the combination of these vehicles may not be different. Similar trend is observed in the case of jam density characteristics also, as shown in Fig. 8.17.

Fig 8.18 (a) and (b) show the increase of capacity flow and jam density with increase in auto-rickshaw percentage, but keeping the car percentage identical and varying the two-wheeler percentage. There could be large differences in capacity, expressed in vehicles per hour, thus demonstrating the effect of two-wheeler in mixed traffic.

Regression equations were developed relating capacity flow and jam density to percentage of auto-rickshaws. Making use of the regression equations developed for capacity flow and jam density in terms of percentage of auto-rickshaws, capacity flow values were estimated for various mixes. These values are given in Table 8.10. It could be observed that for a given percentage of cars, the values along a row indicate the change in capacity flow with increase of percentage of auto-rickshaws. Similarly, reading the values down a column show that capacity flow increases with increase in percentage of two-wheelers (or decrease in percentage of buses), for a given percentage of cars. The values in the cells from right to left diagonally indicate the capacity flows for a given percentage of buses. It can be observed that the capacity flows decrease and



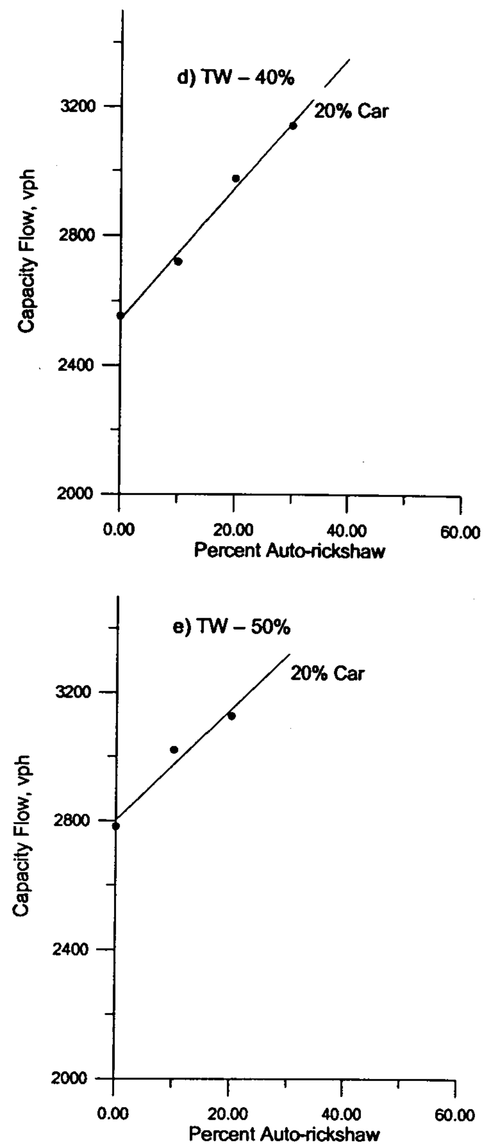


Fig 8.16 Capacity Flow Vs Percentage Auto-rickshaw

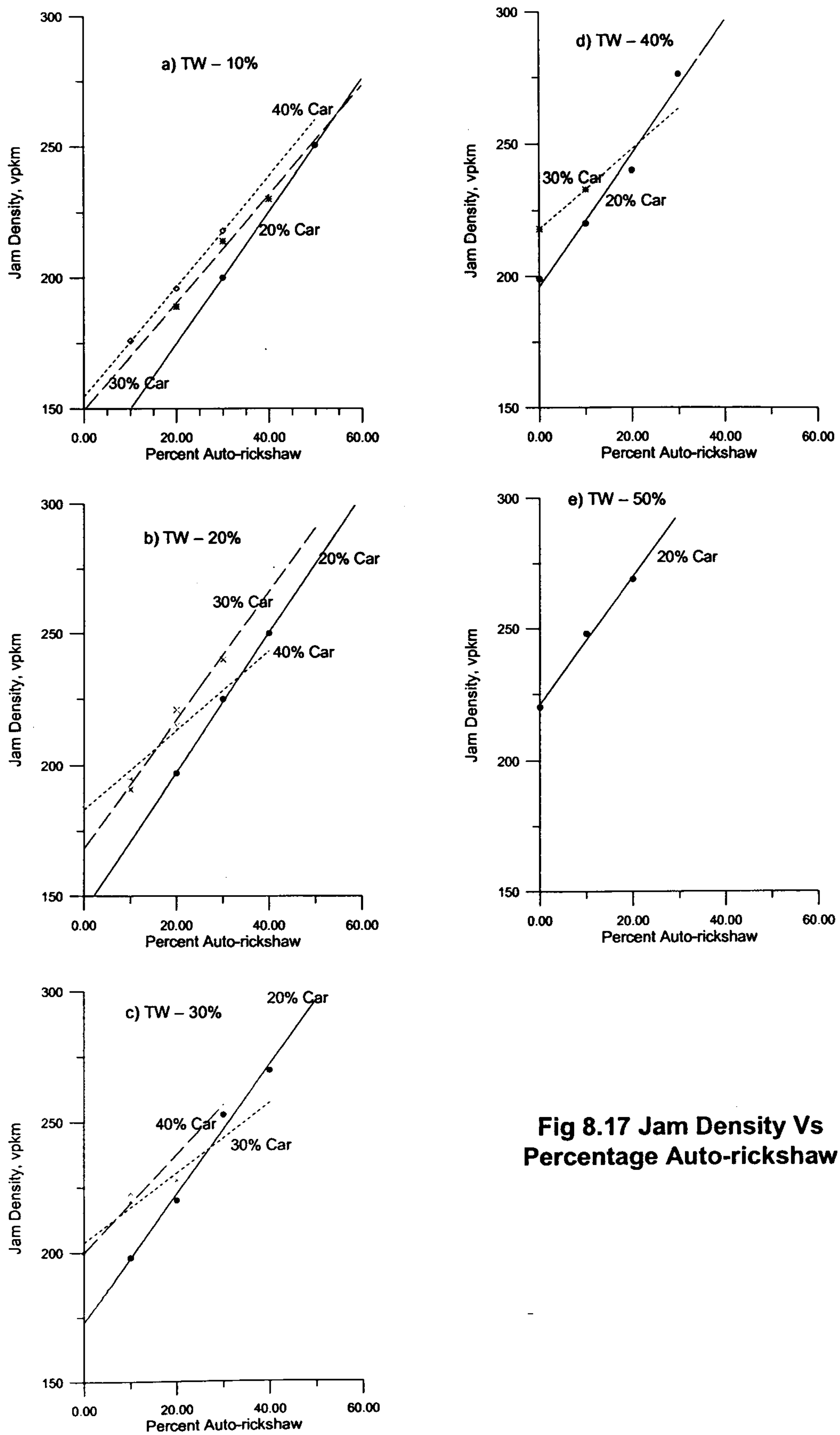
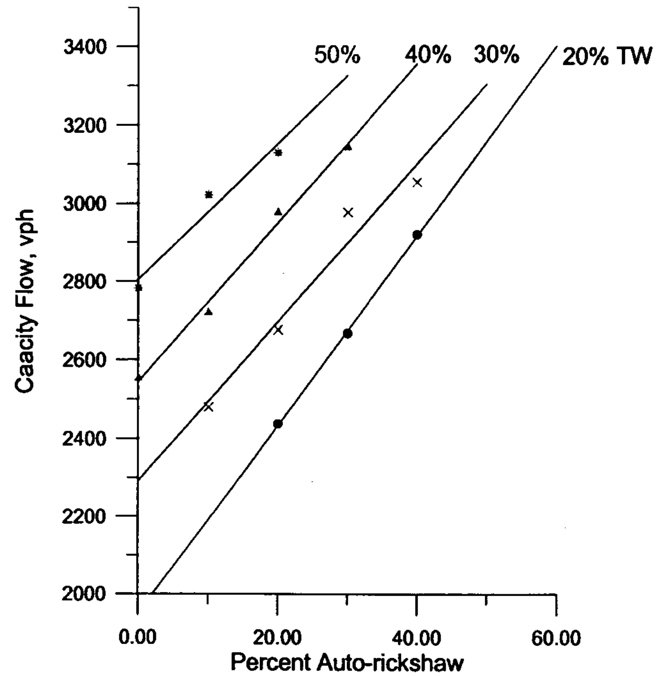
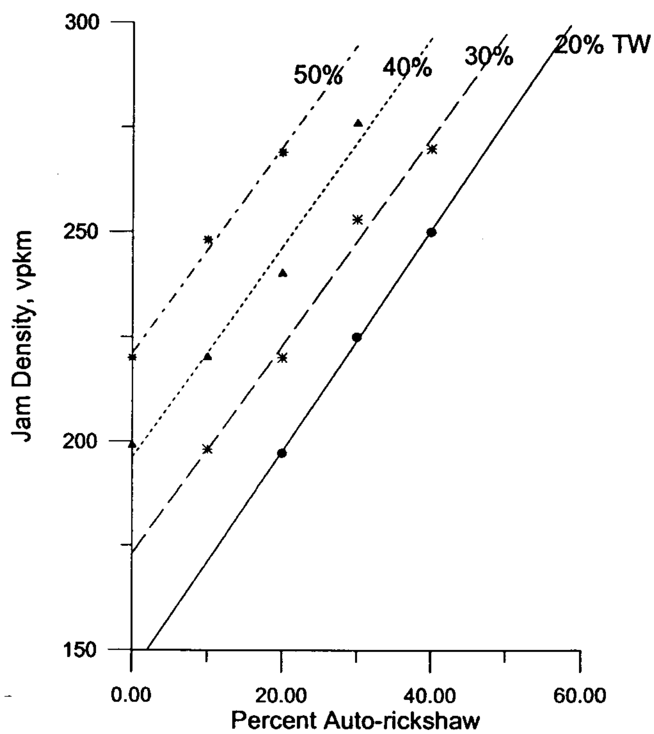


Fig 8.17 Jam Density Vs Percentage Auto-rickshaw



a) Capacity Flow



b) Jam Density

Fig 8.18 Variation with respect to change in Two-wheeler Percentage (Car – 20%)

then increase along the diagonal. This indicates that for a given percentage of cars and buses, two-wheelers and auto-rickshaws have same effect in the stream characteristics. For example, for 20% cars and 10% buses, the same capacity flow can be obtained with 30% auto-rickshaw and 40% two-wheeler or 20% auto-rickshaw and 50% two-wheeler.

Table 8.10 Capacity Flows for Varying Proportions of Vehicles

% TW	% Auto					
	10	20	30	40	50	60
Car – 20%						
20	2191 (50)	2432 (40)	2673 (30)	2914 (20)	3155 (10)	3396 (0)
30	2493 (40)	2694 (30)	2896 (20)	3097 (10)	3299 (0)	-
40	2746 (30)	2947 (20)	3149 (10)	3351 (0)	-	-
50	2977 (20)	3149 (10)	3322 (0)	-	-	-
Car – 30%						
10	2171 (50)	2357 (40)	2544 (30)	2730 (20)	2917 (10)	3103 (0)
20	2444 (40)	2661 (30)	2879 (20)	3096 (10)	3314 (0)	-
30	2726 (30)	2835 (20)	2944 (10)	3053 (0)	-	-
40	2893 (20)	2984 (10)	3075 (0)	-	-	-
Car – 40%						
10	2276 (40)	2459 (30)	2642 (20)	2825 (10)	3008 (0)	-
20	2532 (30)	2638 (20)	2745 (10)	2851 (0)	-	-
Numbers in brackets indicate the percentage of buses						

8.5 CORRELATION ANALYSIS BETWEEN STREAM CHARACTERISTICS & PROPORTIONS OF DIFFERENT VEHICLES

Correlation analysis of the capacity flow/ jam density with proportions of various classes of vehicles was carried to understand the influence of each class of the vehicle on stream characteristics. The correlation coefficients are given in Table 8.11.

At capacity flow condition, the effect of percentage of buses and two-wheelers is highly significant, with of course in opposite direction. This is followed by the car percentage in the stream. However, the percentage of auto-rickshaw has little influence at the capacity (insignificant statistical contribution). But on the other hand auto-rickshaw percentage has significant in jam density determination.

Table 8.11 Results of Correlation Analysis

Variable		% Bus	% Car	% Auto	% TW
Capacity Flow	Pearson Correlation	-0.864**	-0.471**	0.028	0.750**
	Sig. (2-tailed)	0.000	0.001	0.849	0.000
	N	48	48	48	48
Jam Density	Pearson Correlation	-0.878**	-0.600**	0.310*	0.581**
	Sig. (2-tailed)	0.000	0.000	0.032	0.000
	N	48	48	48	48

** Correlation is significant at the 0.01 level (2-tailed).

* Correlation is significant at the 0.05 level (2-tailed).

8.6 STUDY ON INTERACTION EFFECT OF DIFFERENT CLASSES OF VEHICLES IN THE STREAM

The capacity flow values obtained, in vehicles per hour were converted into passenger cars per hour using the PCU values estimated earlier. In most of the cases the capacity value in pcuph was found to be less than that of the cars only stream (2825). This indicates that influence of a class of vehicle on stream characteristics varies depending on the percentage of that class of vehicles and hence, the composition of traffic stream. A possible solution to this problem is to use variable PCU values depending on the proportion of class of vehicles under consideration. Determination of variable PCU values with two classes or three classes of vehicles can be done. However, the degree of complexity increases as the number of classes of vehicles in the stream increase. Even with use of variable PCU values also, the capacity values are likely to differ from that of cars only stream. Model of the form given below was developed relating the ratio of capacity flow for the mix to the capacity of cars only stream and the proportions of various classes of vehicles.

$$Q_{c_{Mix}} = Q_{c_{Car}} * (0.95 p_c + 1.33 p_{tw} + 1.11 p_{auto} + 0.48 p_{bus}) \qquad \text{Eqn. 8.3}$$

Where: $Q_{c_{Mix}}$ - Capacity Flow for mix

$Q_{c_{Car}}$ - Capacity flow for car only stream

p_c, p_{tw}, p_{auto}, p_{bus} – Proportion of car, two-wheeler, auto-rickshaw and bus respectively.

R^2-value – 0.93; F-test value - 147.70

t-test value: p_c - 31.42, p_{tw}-76.75, p_{auto} - 55.20, p_{bus} - 14.99

In deriving the above equation, the coefficients of the percentages of different vehicles were modified taking into account the PCU of the corresponding vehicles at capacity. In this equation, however, the order of significance is different. The proportion of two-wheelers has the highest significance followed by auto-rickshaw, car and bus respectively. The modified capacity values using the above equation for various experimental mixes were estimated and are given in Table 8.12.

Table 8.12 Capacity Values for Various Mixes using the Proposed Equation

Proportion of				@Capacity Flow, vph	#Capacity Flow, pcuph	PDC	*Modified Capacity Flow, pcuph	PDC
Car	Bus	TW	Auto					
0.3	0.3	0.3	0.1	2810	2894	-2.45	2993	-5.93
0.2	0.2	0.1	0.5	2878	2849	-0.86	2955	-4.60
0.3	0.3	0.2	0.2	2702	2837	-0.43	2947	-4.30
0.2	0.2	0.2	0.4	2918	2830	-0.19	2930	-3.71
0.3	0.3	0.4	0	2802	2830	-0.18	2916	-3.21

0.2	0.2	0.3	0.3	2975	2826	-0.04	2922	-3.45
0.3	0.4	0.3	0	2576	2821	0.15	2941	-4.09
0.3	0.2	0.2	0.3	2859	2816	0.31	2917	-3.27
0.2	0.2	0.4	0.2	2975	2767	2.06	2861	-1.26
0.2	0.3	0.2	0.3	2666	2759	2.33	2857	-1.15
0.4	0.4	0.2	0	2434	2750	2.64	2905	-2.82
0.2	0.2	0.5	0.1	3021	2749	2.69	2845	-0.69
0.3	0.3	0.1	0.3	2565	2745	2.85	2866	-1.45
0.3	0.2	0.5	0	2958	2736	3.15	2828	-0.10
0.3	0.2	0.4	0.1	2893	2734	3.23	2825	-0.01
0.3	0.2	0.1	0.4	2720	2734	3.24	2839	-0.50
0.4	0.3	0.3	0	2607	2724	3.56	2824	0.02
0.2	0.1	0.4	0.3	3140	2716	3.85	2847	-0.77
0.2	0.3	0.3	0.2	2675	2715	3.89	2801	0.85
0.2	0.3	0.5	0	2782	2712	3.98	2785	1.42
0.2	0.4	0.4	0	2553	2706	4.21	2793	1.13
0.2	0.3	0.4	0.1	2719	2705	4.23	2783	1.49
0.3	0.4	0.2	0.1	2424	2703	4.33	2838	-0.47
0.2	0.1	0.3	0.4	3052	2701	4.39	2823	0.06
0.4	0.2	0.1	0.3	2647	2700	4.43	2810	0.53
0.3	0.2	0.3	0.2	2794	2696	4.56	2788	1.29
0.4	0.2	0.3	0.1	2751	2696	4.57	2790	1.24
0.4	0.2	0	0.4	2592	2696	4.58	2817	0.27
0.4	0.2	0.4	0	2805	2693	4.68	2783	1.50
0.1	0.2	0.45	0.25	2973	2691	4.76	2786	1.37
0.2	0.3	0	0.5	2494	2681	5.10	2805	0.69
0.1	0.2	0.4	0.3	2930	2681	5.10	2775	1.78
0.2	0.4	0.2	0.2	2436	2680	5.15	2800	0.88
0.2	0.4	0.3	0.1	2480	2678	5.19	2780	1.58
0.4	0.3	0.2	0.1	2513	2676	5.26	2789	1.27
0.2	0.4	0.1	0.3	2388	2675	5.33	2816	0.32
0.2	0.4	0	0.4	2340	2668	5.57	2833	-0.28

0.3	0.4	0.1	0.2	2347	2664	5.70	2821	0.14
0.4	0.3	0.1	0.2	2450	2658	5.90	2787	1.34
0.4	0.2	0.2	0.2	2647	2647	6.30	2746	2.80
0.1	0.2	0.3	0.4	2830	2646	6.33	2737	3.12
0.2	0.1	0.5	0.2	3127	2642	6.47	2780	1.61
0.1	0.2	0.5	0.2	2952	2642	6.48	2738	3.07
0.3	0.3	0	0.4	2412	2629	6.94	2763	2.20
0.4	0.3	0	0.3	2374	2623	7.14	2770	1.94
0.4	0.4	0.1	0.1	2281	2623	7.15	2795	1.05
0.3	0.4	0	0.3	2271	2623	7.15	2804	0.75
0.4	0.4	0	0.2	2188	2560	9.38	2756	2.45
						-2.45		-5.93
						9.38		3.12
No. of values within ± 2% error						8		34

@ - Capacity Values obtained from simulation

- Capacity values in pcuph, estimated using the suggested PCU values.

* - Capacity values pcuph, estimated using proposed equation

PDC – Percentage difference with respect to capacity for car only stream

8.7 DETERMINATION OF SERVICE VOLUMES FOR DIFFERENT LEVELS-OF-SERVICE (LOS)

Highway Capacity Manual (HCM) gives the criteria for demarcation of different levels of service using density as the criterion. The indicated demarcation limits are as given in Table 8.13. Using the speed-density relationships and LOS criteria as specified in HCM the service volumes were computed for different levels of service for single vehicle streams and other experimental mixes. These values are show in Table 8.14. The speed-flow curves with various levels of service demarcated are presented in Fig 8.19 to 8.25 for various single vehicle streams and selected mixes.

Table 8.13 Criteria for various Levels- Of- Service

LOS	LOS Criteria	Corresponding Percentage Reduction in Speed from Free Flow Speed
A	$k_j/15$	6.67
B	$k_j/7.5$	13.33
C	$k_j/4.5$	22.22
D	$k_j/3$	33.33
E	$k_j/2$	50

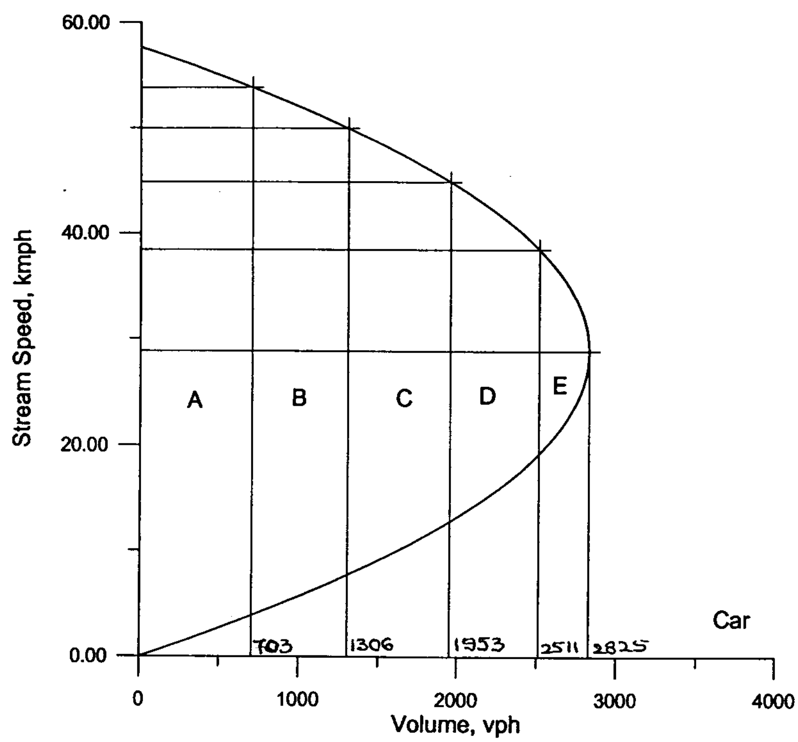


Fig 8.19 Service Volume at Various Levels-Of-Service for Car Stream

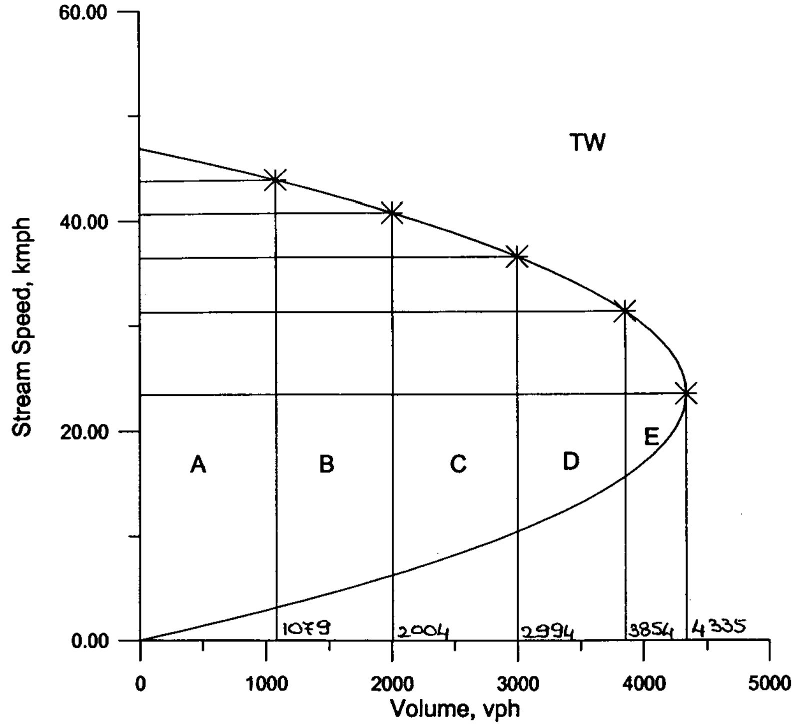


Fig 8.20 Service Volume at Various Levels-Of-Service for Two-wheeler Stream

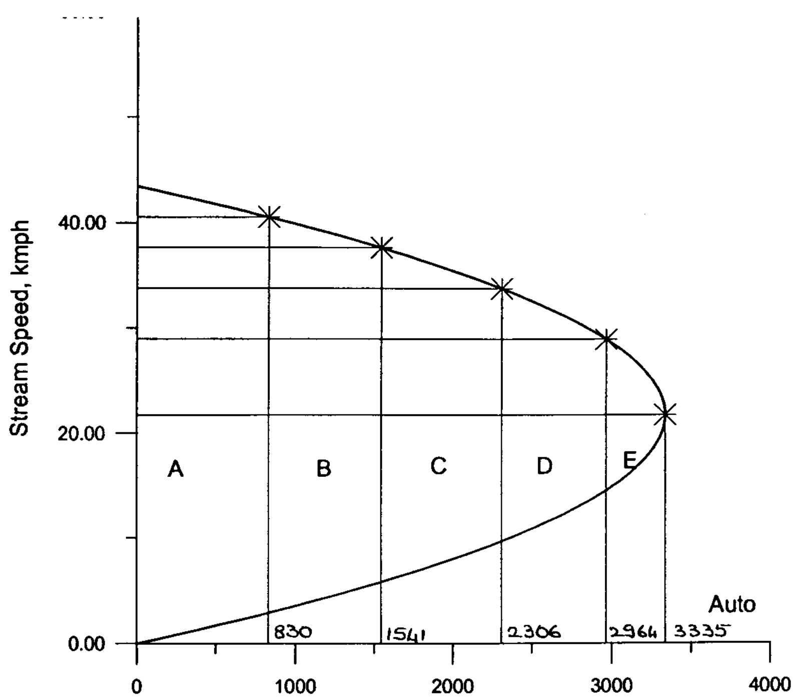


Fig 8.21 Service Volume at Various Levels-Of-Service for Auto-rickshaw Stream

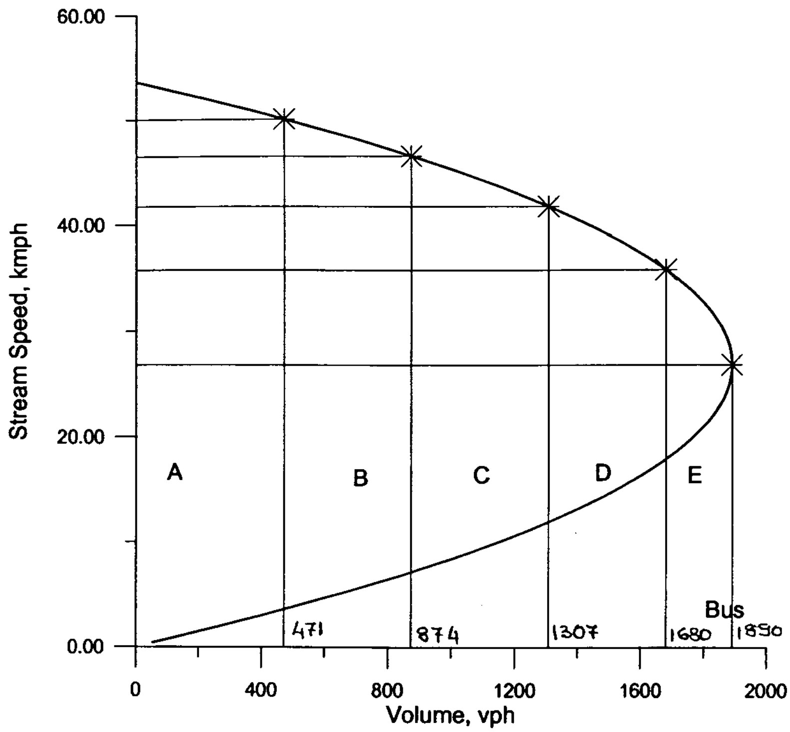


Fig 8.22 Service Volume at Various Levels-Of-Service for Bus Stream

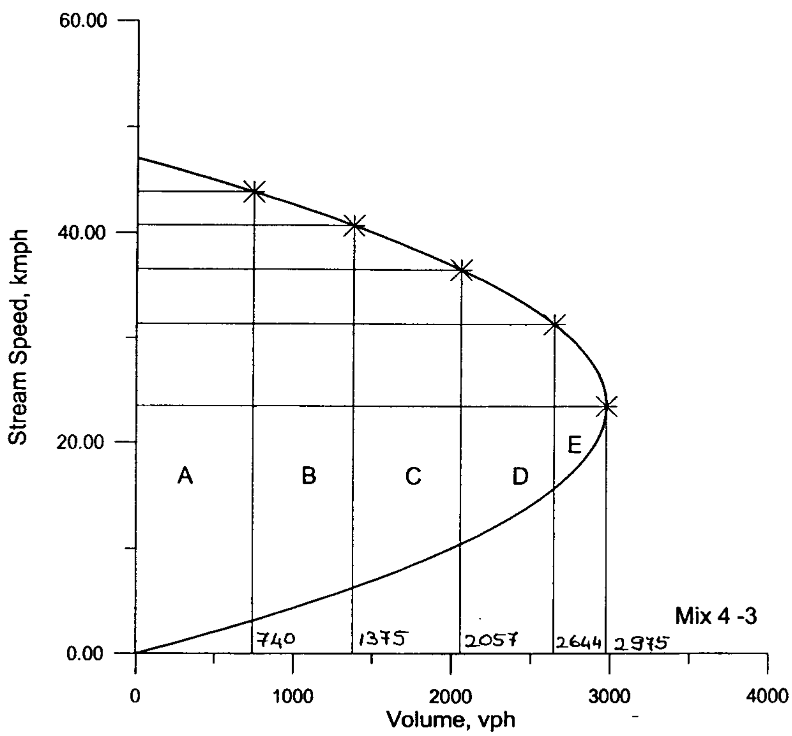


Fig 8.23 Service Volume at Various Levels-Of-Service for Car 20% - TW 30% - Auto 30% - Bus 20% Stream

Table 8.14 Service Volumes for Different Levels-Of-Service

Stream Composition	U_f	K_j	Service Volume for LOS				
			A	B	C	D	E
Car	57.66	196	703	1306	1953	2511	2825
Two-wheeler	46.87	370	1079	2004	2997	3854	4335
Auto-rickshaw	43.45	307	830	1541	2306	2964	3335
Bus	53.63	141	471	874	1307	1680	1890
Mix 4-3	47.03	253	740	1375	2057	2644	2975
Mix 8-3	48.91	221	673	1249	1868	2402	2702
Mix12-3	52.62	185	606	1125	1683	2163	2434

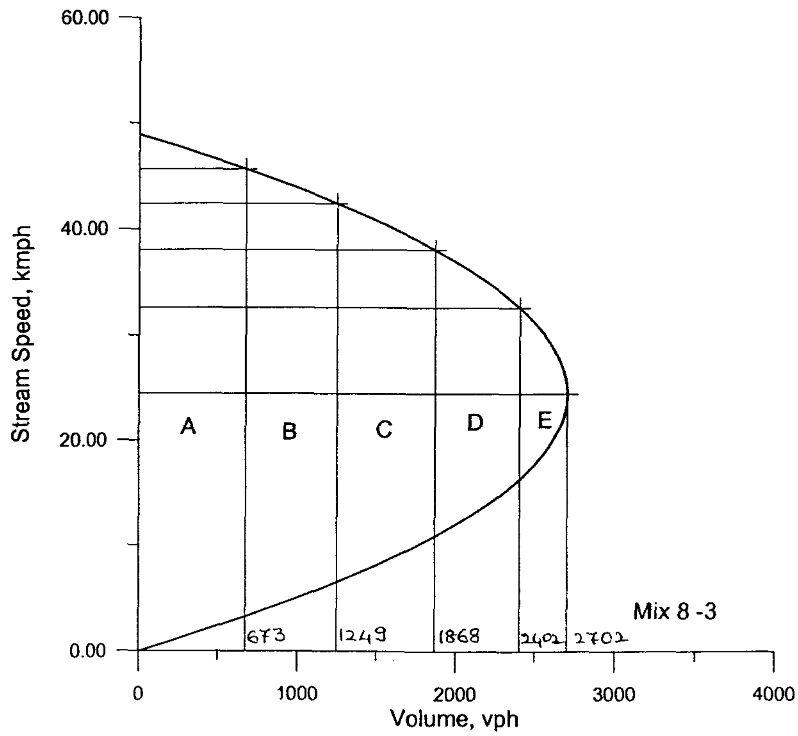


Fig 8.24 Service Volume at Various Levels-Of-Service for Car 30% - TW 20% - Auto 20% - Bus 30% Stream

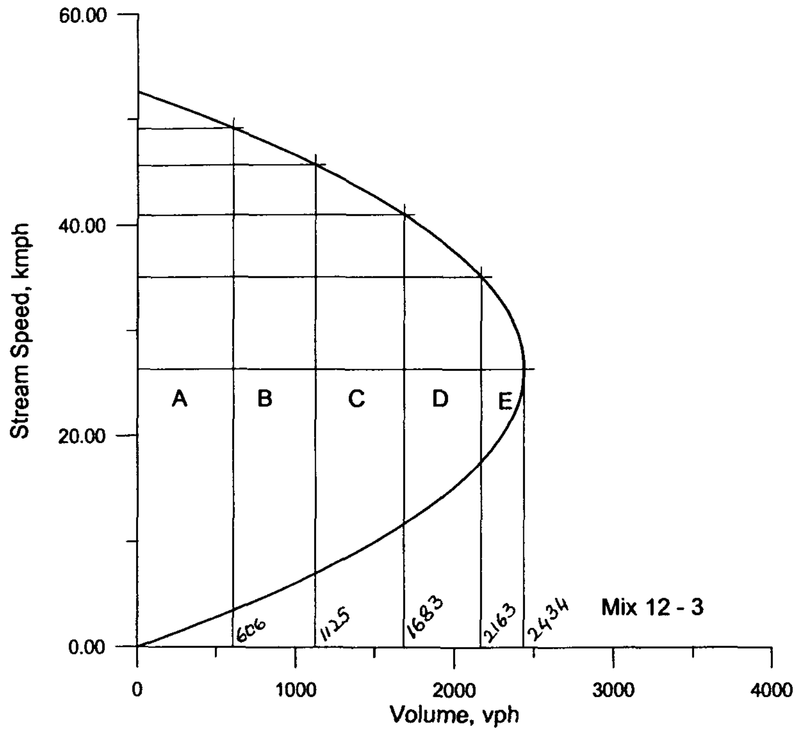


Fig 8.25 Service Volume at Various Levels-Of-Service for Car 40% - TW 20% - Auto 0% - Bus 40% Stream

8.8 COMPUTATION OF PASSENGER CAR UNITS AT VARIOUS LEVELS OF SERVICE

Graphs were prepared for single vehicle streams between percentage reduction in speed Vs volume using the output of simulation experiments. Points on the graph correspond to the average values of clusters of points within each 100 vph range. These graphs are shown in Fig. 8.26 to 8.29. Superposed on these graphs are the points corresponding to the values estimated using the curves presented in Fig. 8.19 – 8.22 It can be seen from these figures that those points corresponding different levels of service do not fall on the curves drawn. For car only stream, around LOS B the superposed point falls on the curve. But at other levels of service, the points deviate from the curve. At LOS C, D and E the predicted volumes are less than the superposed values and the predicted value at LOS A is more than the superposed value. This only means that the full service volumes as predicted from Greenshields model will not be able to be realised in actual conditions at LOS C, D and E. But more cars than what is predicted can pass through at LOS A.

Similar interpretations are possible for other vehicles also. But it is worthwhile to mention that the predicted service volumes for two-wheeler and auto-rickshaw only streams coincide with predictions from Greenshields model at LOS D and E. The behaviour of bus is very much similar to that of car.

Making use of these graphs, the ratio between the service volumes from the curve and the service volumes as per Greenshields model were calculated and are tabulated in Table. 8.15. Normalising the values for different levels of service to unity for car, the equivalent values for different vehicles at various levels of service were calculated and are as shown in Table 8.16.

Table 8.15 Service Volume Ratios

Vehicle Type	Service Volume Ratio at Level Of Service				
	A	B	C	D	E
Car	1.24	1.03	0.84	0.85	0.88
Two-wheeler	0.41	0.48	0.48	0.57	0.59
Auto-rickshaw	0.60	0.61	0.63	0.73	0.75
Bus	1.89	1.59	1.35	1.42	1.40

Table 8.16 Variable PCU Values

Vehicle Type	PCU Values at Level Of Service				
	A	B	C	D	E
Car	1	1	1	1	1
Two-wheeler	0.33	0.47	0.57	0.67	0.67
Auto-rickshaw	0.48	0.59	0.75	0.86	0.86
Bus	1.52	1.54	1.61	1.67	1.59

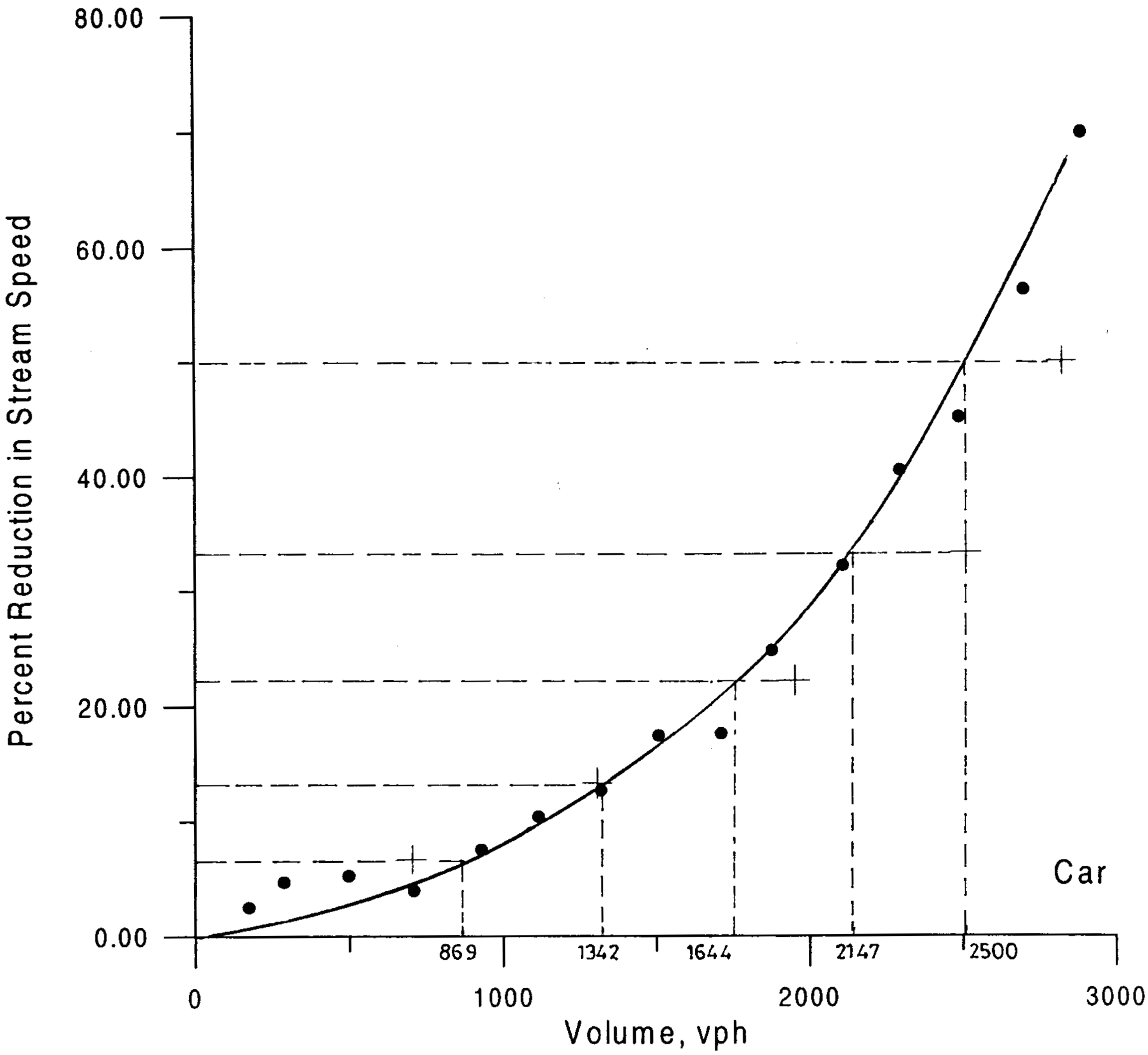


Fig 8.26 Percent Reduction in Speed Vs Volume - Car

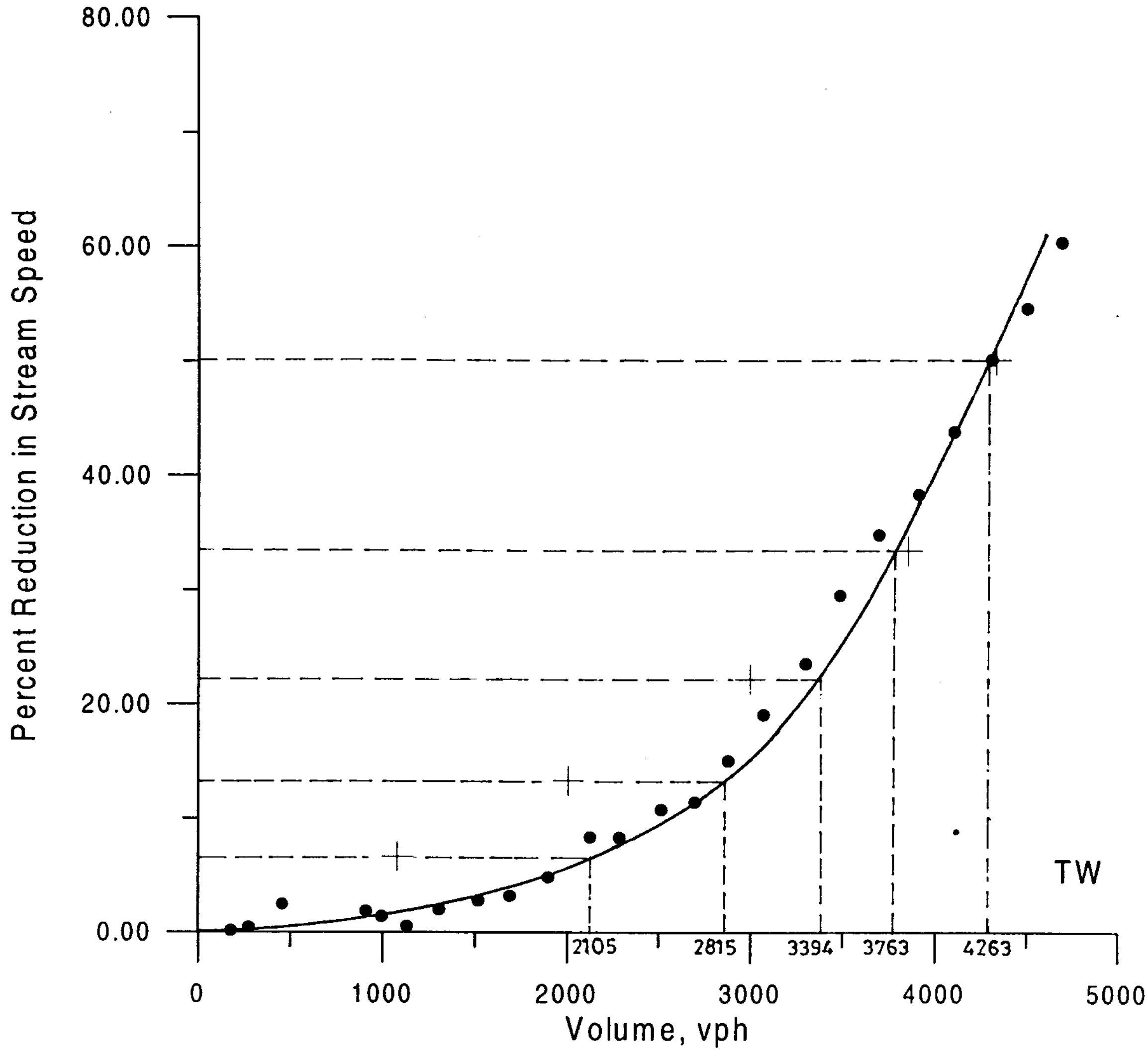


Fig 8.27 Percent Reduction in Speed Vs Volume – Two-wheeler

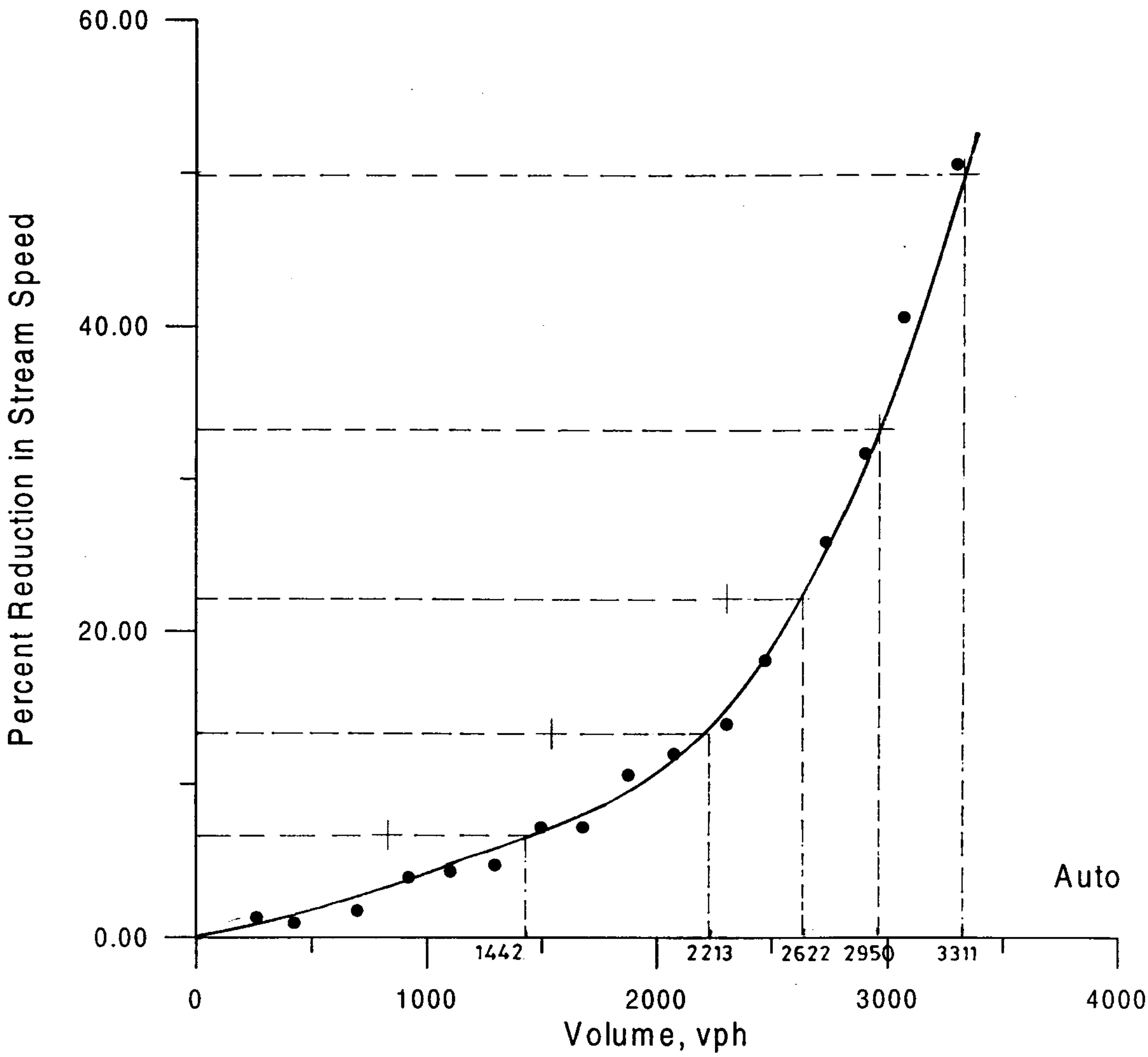


Fig 8.28 Percent Reduction in Speed Vs Volume – Auto-rickshaw

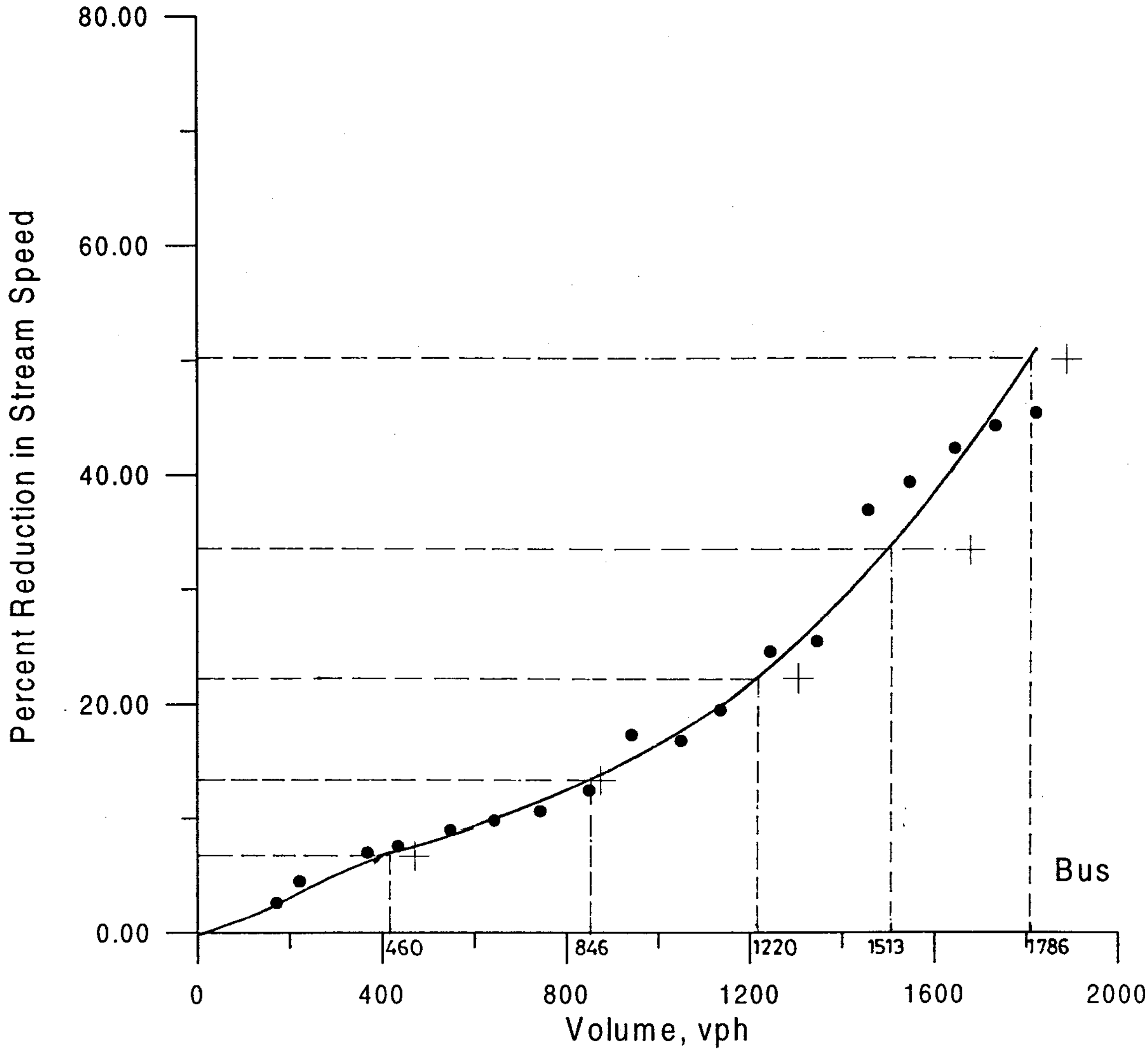


Fig 8.29 Percent Reduction in Speed Vs Volume -Bus

From this table it can be seen that these equivalent values, which may be equivalent passenger car units, vary widely for two-wheeler and auto-rickshaw, whereas for bus, the variation is not much. Two-wheelers and auto-rickshaws enjoy high level of service at low traffic volume and as the traffic volume increases these vehicles attain stable conditions as demonstrated by constant PCU values at LOS D & E. These values can be referred to as Dynamic PCU Values. These equivalent passenger car values are meant for single vehicle flow conditions only. Extensive research is necessary to find these dynamic PCU values for different mix combinations. However, the equivalent PCU values tend towards the PCUs derived based on Greenshields model at capacity conditions.

8.9 COMPARISON OF THE RESULTS OF THE STUDY WITH REPORTED RESULTS OF OTHER RESEARCHERS

It is of interest to compare the results of this study to those reported from the literature. One to one comparison may not be possible always because of the differences in the environment in which those studies have been performed. Even the important figures of merit like the density at capacity, the density at jam, etc., have not been reported in all the studies. In this background a critical comparison of the works may not be possible at all.

Unlike the works of Ramanayya (1980), Badarinath (1993) and Kuncheria (1995), this study has altogether dispensed from assuming any a priori headway distribution models for generation of entry process. Instead, the study has recommended a warm-up zone approach so that the vehicles in the observation section could reach both spatial and temporal stability in the movement. This warm-up approach has demonstrated that, even when large flow levels as high as 10000 vph are generated, the flow logic in the experimental stretch would not permit such high flows. The maximum capacity that could be achieved on the two-lane unidirectional test section for the two-wheeler only traffic was of the order of 4335. This was possible to be achieved at a density of 185 vpkm per lane. A typical graph of speed-density relationship of this particular stream consisting of only two-wheelers shows an extended tail beyond the capacity value as shown in Fig. 8.30. Similar tail is seen in the speed-flow plot exhibited in the same figure. The scatter near the capacity flow is very much perceptible in the flow-density graph. Beyond the flow level of 4335 two-wheelers per hour, there was near breakdown of flow logic, some times the vehicles coming dangerously close to each other, requiring unacceptable levels of acceleration and deceleration rates.

Similar behaviour has been noticed in the case of cars and auto-rickshaws also. The graphical display exclusively incorporated in the simulation model developed was able to closely monitor this loss of rhythm or the breakdown in the vehicular movements. All these suggest to the fact that the capacity definition has to be redefined. Perhaps it could be stated as the maximum number of vehicles that can pass a section without requiring the acceleration/deceleration values of vehicles crossing the threshold values for safe riding. Without such a rider, it is quite possible that the researchers could have observed large flows of particularly two-wheeler and auto-rickshaw movements near capacity.

The suggested capacity of two-lane one-way cars only stream of 2825 car per hour closely matches with that of the Indian Roads Congress recommended value of 2400 pcu.

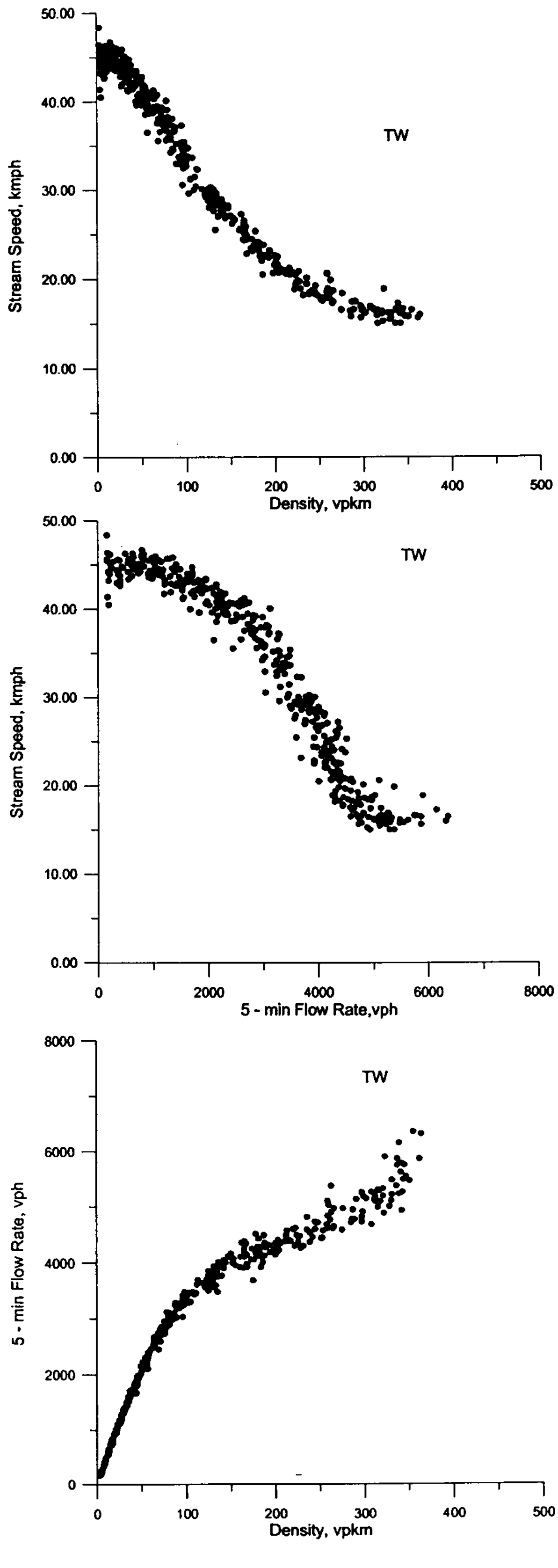


Fig 8.30 Speed – Density – Flow Plots for Two-wheeler only Stream

The primary emphasis in this study is on the use of density as the most critical variable for development of characteristic curves. But in most of the other studies, flow has been used as the reference figure of merit for development of the other traffic flow relationships. Quite often, the perception of the drivers in the assessment of the driving environment is based on the presence/absence of vehicles and their proximity, which are reflected in the density. Hence, development of fundamental relationship based on density as a predictor is most logical. In this study, speed-density and flow-density models have been developed based on the simulation results of nearly 30 experiments carried under nearly identical density conditions. In view of the fact that there are so many replication experiments conducted at nearly similar conditions, the confidence with which the behaviour can be hypothesised is much greater in the present study. Even though considerable time was spent on generation of such closely monitored data sets, it was thought essential in order to over come certain judgemental errors. Based on this kind of experimentation only, it has been concluded in this study that the speed-density, flow-density and speed-flow relationships in mixed traffic are quite similar to that of homogeneous traffic stream, only difference being, the variance could be some what higher in former case. This is one particular aspect in which the present study differs from that of studies carried out by other researchers in mixed traffic environment. According to the present study, the speed-density relationship within the capacity limits could be described by a single regime model.

One other aspect of the present study is that all the variables, which are likely to vary, like for example linear and later placement of vehicles, acceleration/deceleration characteristics of vehicles etc, have been modelled combining both the deterministic and the stochastic components. But such elaborate care is not generally seen in the reported literature. It is for this reason there is an overall control in the simulation outputs as observed from the various traffic stream plots even in mixed traffic conditions.

Speed-flow-density models have been developed in this work for more than fifty mix combinations and the equivalent passenger car units derived in this study are based on the outcome of all these experimentation. It is for this reason, a general model of the form $y=b_1x_1 + b_2x_2 + b_3x_3+ b_4x_4$ could be developed for the prediction of capacity flow for different mix combinations in terms of capacity for single vehicle streams. If there were considerable variability, such a simple regression analysis could have resulted an unusually high intercept constant virtually robbing the explanatory power of the independent variables.

The variable PCU values derived in this study fairly reflect the interactions between vehicles that take place in different mix proportions. The PCU values of different vehicles will increase with the decrease in quality of service up to certain level of service. Then on, PCU value may remain nearly constant for those vehicles which has the freedom of mobility though on a much restricted scale. However, the vehicle which controls the whole traffic behaviour, will have greater mobility and hence there can be a decrease in PCU value of that vehicle. The dynamic PCU values derived in this study very nearly coincide with the PCU equivalents derived by Nagaraj et. al. (1990) based on the concept of influence area. They had indicated in their study that because of higher manoeuvrability of public transport buses and auto-rickshaws in Calicut city, the equivalent values of these vehicles are much less than those observed by Ramanayya (1980).

8.10 CONCLUSIONS

The following are the major conclusions resulting out of simulation experiments:

i. There is no much justification in using models other than single regime linear model for modelling speed-density relationship in mixed traffic conditions. Consequently the speed-flow and the flow-density models could be adequately described by parabolic relationships.

ii. The PCU values at capacity flow conditions for two-wheeler, auto-rickshaw and bus have been worked out to be 0.65, 0.80 and 1.49 respectively in this study. These values very nearly match the observations of Nagaraj, et al.(1990).

iii. The capacity values for single vehicle streams of two-wheeler, auto-rickshaw and bus have been found to be 4335, 3334 and 1890 vph respectively in this study.

iv. A regression equation developed in this study based on 51 speed-density relationships for mixed vehicle combinations is able to predict the capacity of the mixed traffic very close to the reality. However, it is to be used only in situations of mixed traffic and not for streams with only single type of vehicle.

v. Experiments conducted by varying the percentage of auto-rickshaws in the traffic stream while keeping the two-wheeler and car percentages constant, have indicated that the capacity flow Vs percentage auto-rickshaw is linear, but those linear graphs for varying percentages of cars may intersect with one another, suggesting that there may be more than one mix combinations for which the capacity flows may be identical. Hence, vehicles do interact to modify the capacity flow.

vi. Capacity flow is highly correlated with bus and two-wheeler percentage, but in opposite sense. This means that larger the percentage of buses the lesser the capacity flow. However, larger the percentage of two-wheelers greater the capacity flow. Similar inference, as in above, was possible for jam density also.

vii. Equivalent passenger car units calculated in this study for different types of vehicles at various levels of service have indicated that the PCU values for two-wheelers and auto-rickshaws vary quite considerably over different levels of service. On the other hand, change in equivalent PCU values for bus was found to be not very significant.

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CHAPTER - 9

SUMMARY, CONCLUSIONS AND SCOPE FOR FURTHER WORK

9.1 SUMMARY AND CONCLUSIONS

9.1.1 Many traffic simulation studies that have been completed in mixed mode environment on Indian Roads have raised considerable doubts in the applicability of conventional speed-flow-density relationships. Some of the researchers have even pointed out that the speed-flow model appears more bell-shaped than being parabolic. Inspite of these doubts and conjectures, research work is continuing in India, as though, those observations are accidental and hence cannot be accepted as proven.

9.1.2 In this confused research background, the objective of the present work is to introduce certain orderliness in the experimentation and model building, so that consistency could be achieved in the results.

9.1.3 Exhaustive literature review in the present topic of interest has revealed that Simulation approach has been the most widely used for modelling of traffic flow due to its many advantages over other approaches. Even those models that have been built using simulation approach have fallen short of expectations due to very many simplifying assumptions made in building up of component models.

9.1.4 Literature review has also revealed that neural networks, which mimic the human behaviour, have been successfully used for many traffic engineering applications. However, the neural networks need prior exposure in the form of training before application. Hence, it was felt that neural networks might not be able to provide total solution to the mixed traffic flow. Rather, they will be useful for representing some of the sub-systems.

9.1.5 Models of linear and lateral clearances, which form the most important component of the simulation model, were derived based on exhaustive data collected using video graphic technique.

9.1.6 The linear and lateral clearances were found to be dependent mainly on speed of subject vehicle for a given combination pair of vehicles. The relationship was observed to be linear.

9.1.7 The deviations of individual values of clearances from mean value for a pair of vehicles and for a speed range were found to follow normal distribution. The regression models of linear and lateral clearances were modified to reflect driver behaviour by adding a component, whose values follow normal distribution.

9.1.8 The placement of a vehicle with respect to surrounding vehicles in a traffic stream depends on a number of factors. The effect of many of these influencing factors is difficult to be directly observed and quantified in addition to being non-linear. Hence, Neural networks, which identify the input-output relations without any preconception or bias through training, were used for modelling linear spacings. The feed forward neural network with back propagation algorithm and with two layers (one hidden layer) was used for modelling these spacings. The optimum number of neurons in hidden layer was found to be ten. The performance of the neural network model was found to be superior to that of regression models both at calibration and validation stages.

9.1.9 The position of a vehicle across the width of the road, while being not under the influence of any other vehicle, need to be given separately and hence the models that describe the lateral placement of vehicles under free flow conditions form another component of simulation model. Analysis of the data collected at four locations of varying road width revealed that the lateral placement of vehicles depend on the type of vehicle, speed of vehicle and width of road. The variance of the distances maintained by any class of vehicles from the edge of the road was found to increase with increase in road width.

9.1.10 In simulation modelling, the generated vehicles are assigned the free flow speeds by sampling from speed distributions. Field surveys were conducted at four typical urban mid blocks to obtain the free speeds of different types of vehicles. The free flow speeds were found to follow normal distribution.

9.1.11 The threshold headway is generally used to distinguish free flowing and following vehicles. The threshold headway was determined by analysis of speed and headway data of vehicles, which were moving one behind the other, and it was found to be 3 sec.

9.1.12 Drivers steer their vehicles across the width to overtake another vehicle or to take the desired position across the width of the road and in such a way that no collision takes place. An attempt has been made in this study to determine the rate of lateral movement for different classes of vehicles. The rate of lateral movement was found to vary depending on the type of vehicle.

9.1.13 Car following models, which describe the behaviour of drivers reacting to the behaviour of their leaders, is another important component of traffic flow simulation model. In this study, the car-following behaviour of constrained drivers was modelled using both conventional response-stimuli method and neural network.

9.1.14 The parameters of conventional response-stimuli models were obtained by regression analysis. The optimum structure of the neural network was determined through trial and error process. The performance of neural and conventional stimulus-response models in predicting the speeds of each vehicle type was also compared. It was found that neural models performed better than car-following models during both training and validation stages.

9.1.15 Separate models were built to describe the acceleration characteristics of unconstrained vehicles. The data required for modelling of acceleration/deceleration characteristics of vehicles was collected by a special method developed for this purpose. The average acceleration/deceleration values were found to be exponentially related with the speed of vehicle. These models were modified to reflect the driver behaviour by adding a term to the value obtained by regression model. Validation of the models was done by comparing the predicted acceleration/deceleration values with the actual values and the predicted values were found to be in close agreement with the actual values.

9.1.16 A computer model was developed by integrating the various component blocks for simulation of unidirectional traffic flow on a mid block section. The code for the simulation model was developed in 'C' language. Stack Data Structure was used for internal book keeping. The time scanning technique was used for scanning and updating the system.

9.1.17 Some of the salient features of the model are ability to simulate up to ten classes of vehicles, flexibility to change the length and width of simulated road, etc. The model has the capability of generating 100 sequences of random numbers so that each component process of simulated system can be described by a separate sequence of random numbers. An option was provided to randomise the seeds for generation of random numbers so that different sequences of random numbers can be used in different trials for a particular process

9.1.18 The inter arrival times at the entry to simulated stretch were generated by sampling from exponential distribution. The model generates the free flow speeds of vehicles based on normal distribution.

9.1.19 The generated vehicles were moved through the simulated system as per the vehicle manoeuvring logic. A vehicle was classified being in one of five conditions based on the speed, positions of other vehicles in vicinity and available clearances. Separate functions were used to move a vehicle depending on its condition.

9.1.20 The output of the simulation model includes the stream characteristics such as the flow, the speed and the density at user selected points or stretches. The speeds of individual vehicles can also be obtained as output. The simulation model provides graphic display of the vehicles as they move through the simulated stretch. The model provides the user, the freedom to interject the program as per his/her requirement. The length of the warm up zone was determined by comparing the headways at various sections along the length of the simulated road. The length of warm up zone was obtained as 500 m based on frequency distribution analysis of headways. The warm up time was taken as 5 min.

9.1.21 Validation of the model was carried out in two stages. Firstly, it was checked whether the model is reproducing the general trends observed in real life situation. The model was found to be able to reproduce the general trend as expected in reality. The model was also validated by comparing the model outputs with the corresponding field data collected at two locations in Calicut city. The model was found to be reasonably good in replicating the field conditions.

9.1.22 In order to understand the behaviour of mixed traffic, a series of experiments were conducted on the simulation model by varying the volume and the composition. Experiments were conducted for single vehicle streams and for various mix proportions. Regression analysis was carried out to derive the speed-density relationships for each of the experimental mixes. The speed-density relationship was found to be linear in all the cases. Using the derived speed-density models, stream parameters such as capacity flow and jam density were computed. Using the capacity flow values for single vehicle streams, the PCU values for two-wheeler, auto-rickshaw and bus were estimated.

9.1.23 Analysis of results of simulation experiments confirmed that the capacity flow increases with increase in percentage of auto-rickshaws in the stream for a given total percentage of cars and two-wheelers. Service volumes were determined for various levels of service using density as the criterion variable and the demarcation limits suggested by the HCM. Using the percentage reduction in speed as the criterion, the equivalent PCU values were obtained for different vehicles at various levels of service.

9.1.24 The major conclusions resulting from the experiments carried out on simulation model developed are:

i. There is no much justification in using models other than single regime linear model for modelling speed-density relationship in mixed traffic conditions. Consequently, the speed-flow and the flow-density models could be adequately described by parabolic relationships.

ii. The PCU values at capacity flow conditions for two-wheeler, auto-rickshaw and bus have been worked out to be 0.65, 0.85 and 1.49 respectively in this study.

iii. The capacity values for single vehicle streams of two-wheeler, auto-rickshaw and bus have been found to be 4335, 3334 and 1890 vph respectively in this study.

iv. A regression equation developed in this study based on 51 speed-density relationships for mixed vehicle combinations is able to predict the capacity of the mixed traffic very close to the reality. However, it is to be used only in situations of mixed traffic and not for streams with only single type of vehicle.

v. Experiments conducted by varying the percentage of auto-rickshaws in the traffic stream while keeping the two-wheeler and car percentages constant, have indicated that the capacity flow Vs percentage auto-rickshaw is linear, but those linear graphs for varying percentages of cars may intersect with one another, suggesting that there may be more than one mix combination for which the capacity flows may be identical. Hence, vehicles do interact to modify the capacity flow.

vi. Capacity flows in traffic stream is highly correlated with bus and two-wheeler percentage, but in opposite sense. This means that larger the percentage of buses the lesser the capacity flow. However, larger the percentage of two-wheelers greater the capacity flow. At or near capacity, the influence of auto-rickshaw percentage is very marginal. Similar inference, as in above, was possible for jam density also. But the contribution of both the car and the auto-rickshaw percentages were found to be statistically significant in jam density calculations.

vii. Equivalent passenger car units calculated in this study for different types of vehicles at various levels of service have indicated that the PCU values for two-wheelers and auto-rickshaws vary quite considerably over different levels of service. On the other hand, change in equivalent PCU values for bus was found to be not very significant. Because of this reason, it was found that the auto-rickshaws and two-wheelers had significant contribution to make in the determination of capacity of traffic stream expressed in PCUs.

9.2 CONTRIBUTIONS OF THE STUDY

The major contributions of the study are

i. Development of a Neural Integrated Simulation model for experimentation related to vehicular movements in mixed traffic environment.

ii. Further refinement of basic simulation models by incorporation of linear and lateral placement of vehicles, acceleration/deceleration characteristics of vehicles, vehicle manoeuvring logic, etc, through neural network approach.

iii. Introduction of warm up zone approach in simulation modelling of traffic flow.

iv. Simulation of vast possible scenarios for a given set of input situation by randomising the seeds for generation of random number sequences.

v. Graphic display of simulated movements of vehicles and introduction of appropriate corrections to the flow logic when found necessary.

vi. Determination of capacity and jam density values for single and multiple vehicle streams by the development of a simple linear regression equation.

vii. Demonstration of the suitability of linear speed-density and parabolic speed-flow and flow-density models even in mixed traffic situation.

viii. Study of sensitivity of capacity values and jam densities to composition of the vehicles in the stream.

ix. Development of equivalent passenger car units and service volumes at different levels of service and study of variable PCU values.

9.3 LIMITATIONS OF THE STUDY

The developed simulation mode is useful only for one directional uninterrupted traffic movements under ideal geometric conditions. The effect of other influencing factors is not the subject matter of the present study. Separate models have to be developed for bi-directional movement and for the study of influence of geometrical factors.

9.4 SCOPE FOR FURTHER WORK

The limitations of the study are the food for thought for further research in the field. Apart from this, it has been noticed from the results of simulation experiments, that the scatter of points in speed-density, speed-flow and flow-density plots are quite extensive, particularly near the capacity and free flow conditions. However, within the middle ranges, the scatter appears to be within reasonable limits.

These scatters particularly give rise to conjecture whether the idealised model of q=ku is valid in mixed traffic conditions. Fig 9.1 shows the scatter of points in the q/ku Vs density graph for car, two-wheeler, auto-rickshaw and bus only traffic streams, whereas, Fig 9.2 shows the scatter for three different mix combinations.

It is worthwhile investigating whether a modified equation of the form:

$$q = cku$$

where c is a dimensionless parameter, either independent or dependent on density could be built, so that the scatter of points in traffic stream interrelationships could be explained. 'c' appears to be a parameter related to the way in which the vehicles within the stream orient themselves, giving rise to series of gaps, similar to void ratio or porosity in soil structure.

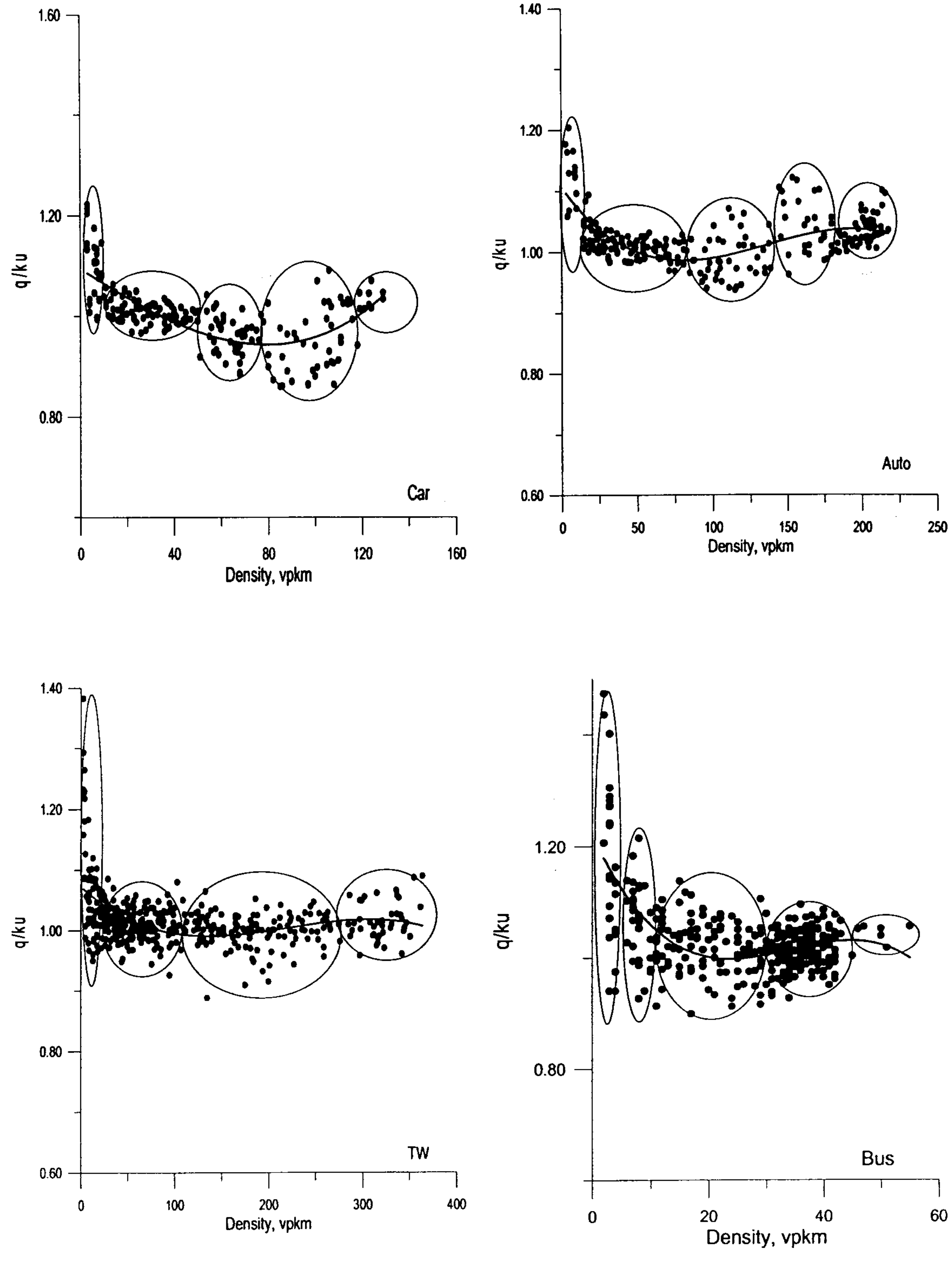
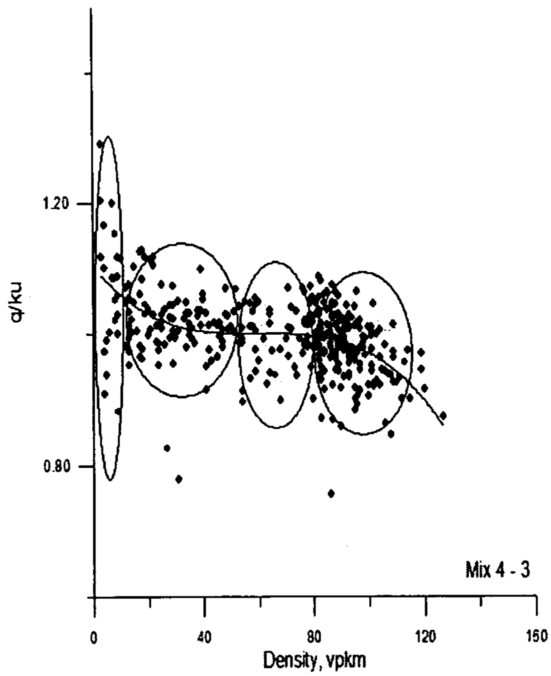
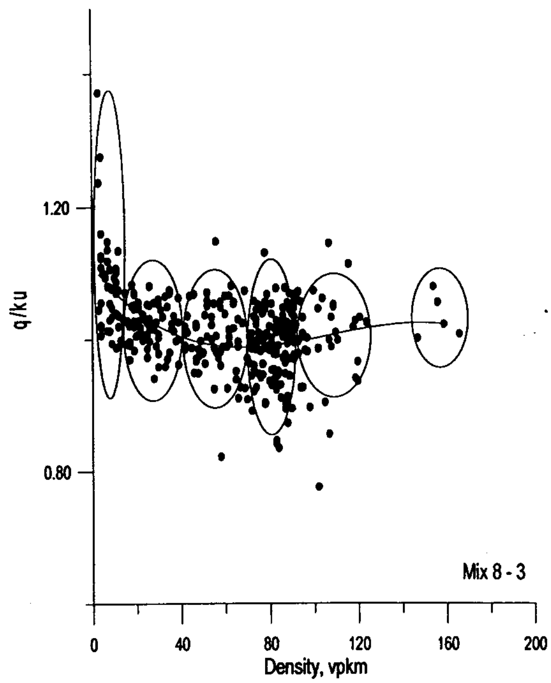


Fig 9.1 q/ku Vs Density Plots for Single Vehicle Streams





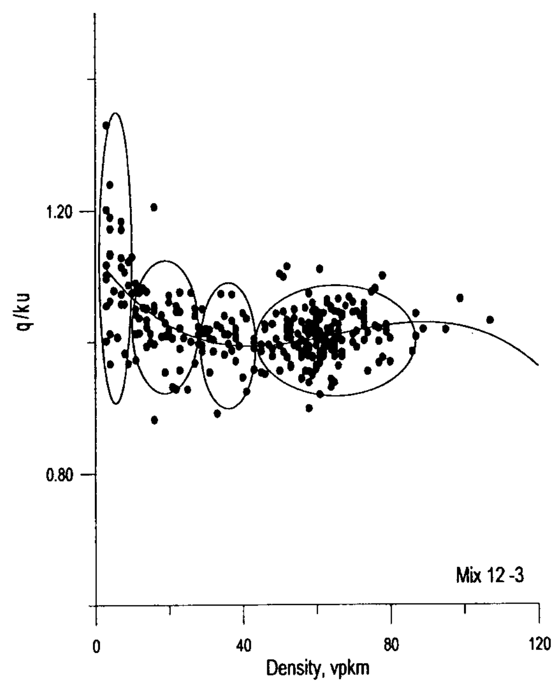


Fig 9.2 q/ku Vs Density Plots for Selected Mixes

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