TRAFFIC FLOW MODELING AND SIMULATION OF TRAFFIC FLOW BEHAVIOR UNDER MIXED TRAFFIC CONDITIONS ON MULTILANE HIGHWAYS

Submitted in partial fulfilment of the requirements

for the award of the degree of

Doctor of Philosophy

by

S Srikanth

(Roll No: 714110)

Department of Civil Engineering

NATIONAL INSTITUTE OF TECHNOLOGY

WARANGAL

August-2019

TRAFFIC FLOW MODELING AND SIMULATION OF TRAFFIC FLOW BEHAVIOR UNDER MIXED TRAFFIC CONDITIONS ON MULTILANE HIGHWAYS

Submitted in partial fulfilment of the requirements

for the award of the degree of

Doctor of Philosophy

by

S Srikanth

(Roll No: 714110)

Supervisor

Dr. Arpan Mehar

Department of Civil Engineering

NATIONAL INSTITUTE OF TECHNOLOGY

WARANGAL

August-2019

NATIONAL INSTITUTE OF TECHNOLOGY
WARANGAL

CERTIFICATE

This is to certify that the thesis entitled "**TRAFFIC FLOW MODELING AND SIMULATION OF TRAFFIC FLOW BEHAVIOR UNDER MIXED TRAFFIC CONDITIONS ON MULTILANE HIGHWAYS"** being submitted by **Mr. S Srikanth** for the award of the degree of **DOCTOR OF PHILOSOPHY** to the Department of Civil Engineering, **NATIONAL INSTITUTE OF TECHNOLOGY, WARANGAL** is a record of bonafide research work carried out by him under my supervision and it has not been submitted elsewhere for award of any degree.

(Dr. Arpan Mehar)
Thesis Supervisor
Assistant Professor
Department of Civil Engineering
National Institute of Technology
Warangal-506004

APPROVAL SHEET

This Thesis entitled "**TRAFFIC FLOW MODELING AND SIMULATION OF TRAFFIC FLOW BEHAVIOR UNDER MIXED TRAFFIC CONDITIONS ON MULTILANE HIGHWAYS**" by **Mr. S Srikanth** is approved for the degree of Doctor of Philosophy.

Examiners

____

____

____

Supervisor

____

Chairman

____

Date: ___________

DECLARATION

This is to certify that the work presented in the thesis entitled **"TRAFFIC FLOW MODELING AND SIMULATION OF TRAFFIC FLOW BEHAVIOR UNDER MIXED TRAFFIC CONDITIONS ON MULTILANE HIGHWAYS"** is a bonafide work done by me under the supervision of **Dr. Arpan Mehar** and was not submitted elsewhere for the award of any degree. I declare that this written submission represents my ideas in my own words and where others ideas or words have been included, I have adequately cited and referenced the original sources. I also declare that I have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea / data / fact /source in my submission. I understand that any violation of the above will be a cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

(S Srikanth)

(Roll No: **714110**)

Date: __________

Dedicated to

My Beloved Family and Gurus

ACKNOWLEDGEMENTS

I express my sincere gratitude to **Dr. Arpan Mehar**, Assistant professor, Department of Civil Engineering, N.I.T, Warangal, for his expert guidance, motivation, and continuous support throughout my research work. He delivered a constant inspiration, positive thoughts and creative ideas in different stages of this research work. His unforgettable suggestions and valuable remarks gave me directional strength to accomplish my research work within required time frame. His immense knowledge has provided me an insight to accomplish research and helped me in different manners during midst of the greatest difficulty. I am greatly benefited from his teaching, scientific approach and ability to put complex ideas into simple expressions. I could not have imagined having a better guidance for my Ph.D thesis. I thankfully acknowledge him for his blessings and giving me an opportunity to work under his exalted guidance.

I would like to thank **Dr. Venkaiah Chowdary**, Associate Professor, Head of the Transportation Division, Department of Civil Engineering, for his valuable suggestions and constant encouragement throughout the course of my thesis work. I would like to thank **Dr. C.S.R.K Prasad,** Professor, Transportation Division, Department of Civil engineering, **Dr. K.V.R Ravi Shankar and Dr. S. Shankar,** Assistant Professors, Transportation Division, Department of Civil Engineering for their valuable suggestions, encouragement and continuous monitoring of my work throughout the course of my thesis.

I would like to thank **Dr. M. Chandrasekhar,** Professor and Head, Department of Civil Engineering and Chairman of my Doctoral Scrutiny Committee for his enlightening guidance and immense help rendered in bringing out this work. I would like to thank members of my Doctoral Scrutiny Committee **Dr. C.S.R.K Prasad**, Professor, Civil Engineering Department, **Dr. K.V.R. Ravi Shankar**, Assistant Professor, Department of Civil Engineering and **Dr. Srinivasacharya**, Professor, Department of Mathematics, for their support, valuable suggestions, inputs and guidance during various stages of my research work.

I would like to express deep appreciation to my co-scholars **Anil, Syed, Utsav, Peera, Abhigna, Harindar, Arjun, Aditya, Jayakrishna, Eswar, Prasanth, Yugandar, Pallavi and others** for their assistance, valuable guidance and cooperation throughout the period of my research work.

I would like to thank my friends and former PG Students of Transportation Division, **KGNV Praveen, Asish Parihar and Madhuri** for their help during the period of my research work.

I would like to thank **Mr. Md. Gaffar** and **Mr. Ramesh,** laboratory staff of Transportation Division for their constant support during my study.

I am very grateful to my parents for giving me strength, motivation and inspiration. I also express gratitude to my sisters and brother for their immense love and encouragement. I wish to express my indebted love and gratitude to my wife, **Swathi**, who supported me through her enlightened heart during my stay in N.I.T Warangal. I have no words to express my deep love and feelings to my daughter **Srithika**, her charming smile also gave me constant inspiration to make this research work possible.

I would like to express my deepest gratitude to all those who have contributed directly or indirectly in the course of my research work and throughout my life. I am thankful to them for inspiring me through their enlightened hearts, for touching me through their acts of kindness and honesty and illuminating me through their wisdom and knowledge.

(S Srikanth)

ABSTRACT

The mixed traffic behavior on multilane highways remains scarcely explored. Most of the studies on multilane traffic flow pertain to homogeneous traffic scenarios. These studies have very limited applications to mixed traffic conditions due to their incapability in satisfactorily explaining the complexities of mixed traffic behaviour. Also, the available literature on mixed traffic mainly deals with the single and two-lane roads and there is no comprehensive study for understanding traffic flow behavior on multilane highways. IRC: 64-1990 also provides detailed guidelines for capacity of single-lane, intermediate lane and two lane rural roads; leaving multi-lane highways almost unattended. Due to lack of standard codes for multilane highways in India, it is difficult for traffic engineers, policymakers and planners to take accurate decisions with respect to planning, design, and operations of these highways. The present research work aims at investigating the mixed traffic flow behaviour on highways for varying conditions of traffic volume. To understand the traffic flow behavior on four-lane divided highways under mixed traffic conditions, the arrival patterns of vehicles, time headway characteristics, speed characteristics, lateral placement of vehicles and overtaking behavior were analyzed.

The aim of the present study is to develop more appropriate models for estimating the passenger car units of different vehicle types on multilane highways, considering the limitations of available methods. Present study describes a modified methodology for estimation of PCU value of subject vehicles that includes the time headway as influencing parameter. The approach used in the present study is inspired from the method of dynamic PCU estimation where a PCU is expressed as the ratio of speed ratio and area ratio of standard cars to the subject vehicle type. Unlike dynamic PCU method, this method includes time headway factor for PCU estimation. The method was found to be more realistic and logical as it provides relatively higher values of PCUs than those obtained from dynamic PCU method.

Multiple non-linear regression (MNLR) method is proposed for estimation of equivalency units for vehicle types by developing speed models based on multiple non-linear regression approaches. The equivalency units estimated by using models are found to be realistic and logical under heterogeneous traffic flow conditions. The PCU values estimated by the multiple non-linear

regression method are compared with and found to be relatively higher values than the values obtained by the dynamic PCU method. The accuracy of the models is checked by comparing the observed values of speed with estimated speeds. The multiple non-linear regression approach is also used for estimating the equivalency units on six-lane divided highways.

The primary limitations of field data arises due to practical difficulties in conducting extensive field experiments under wide variations of traffic flow parameters, non-availability of required field conditions, difficulty in experimenting with individual components in isolation, etc. As a solution to these practical problems, computer simulation has been proved to be a powerful tool in replicating complex traffic systems which allows experimentation to the basic traffic flow system. For the simulation VISSIM microscopic simulation tool is used and data analysis is performed by considering individual parameters and performance measures like speed, volume and random seed number. Statistical tests have been performed to check the sensitivity of the different simulation parameters and calibration is done using trial and error method and optimization is performed using solver function. The maximum simulated flow rate was found with default values as 4599 veh/hr, and with calibrated values is 5147 veh/hr which is close to the target capacity 4958 veh/hr as obtained using field composition. Calibrated values of CC0 and CC1 and CC2 parameters are found as most optimised values to achieve target capacity. Finally, validation of calibrated parameter values was also performed on other section of a multilane highway which have shown satisfactory results.

Lane changing is a very complex maneuver which can be studied through microscopic and macroscopic measures. Calibrated VISSIM model was used for generating traffic flow data to obtain the essential parameters. Lane change behaviour is analysed with homogeneous vehicle type traffic on four-lane, six-lane and eight-lane divided highways sections through VISSIM simulation model. The study finds the number of lane changes depends on traffic volume as well as on number of lanes provided for a direction of travel. Lane change data was correlated with traffic volume and third degree polynomial trend was found to be fitted on each type of simulated highway sections. Maximum number of lane changes and lane change at capacity level of volume are also quantified on simulated sections of four-lane, six-lane and eight-lane divided highways. It is found that no more number of lane changes is observed in all simulated sections when traffic volume reaches to maximum capacity. The relationship between capacity per lane and number of lane changes is established which shows capacity decreases with addition of number of lanes.

Level of service concept is applied in the present study to estimate the passenger car unit (PCU) value of each vehicle type at different level of service and different percentage share. Calibrated VISSIM model was used to simulate traffic conditions for the development of PCU models. The PCU value of each vehicle type at different level of service and different percentage share was found for the development of the models. The accuracy of the models is checked by comparing the obtained PCU values with PCU values estimated by dynamic PCU method. PCUs of different vehicle types at six lane and eight lane divided highways are also estimated. The effect of number of lanes on PCUs was studied, and it was observed PCU of each vehicle type decreases with increase in the number of lanes and at a different level of service. Artifical neural networks (ANN) and Artificial neuro fuzzy interface system (ANFIS) models are also developed for estimating PCU values of subject vehicle types with respect to passenger cars. The PCU estimated from different approaches are compared statistically in order to justify the best approach with the same set of input variables.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

GLOSSARY OF TERMS

ANFIS	Artificial neuro-fuzzy interface system
ANN	Artificial neural network
ASCE	American Society of Civil Engineers
B	Binomial distribution
CA	Cellular Automata
CS	Small Car
CB	Big Car
GEV	Generalised Extreme Value
HCM	Highway Capacity Manual
HRB	Highway Research Board
HRR	Highway Research Record
HV	Heavy vehicles
HVU	Heavy vehicle unit
IHCM	Indonesian Highway Capacity Manual
IRC	Indian Roads Congress
K-S	Kolmogorov-Smirnov test
LCV	Light commercial vehicles
LOS	Level of Service
MCU	Motor Cycle Unit
MLR	Multiple Linear Regression
MNLR	Multiple Non Linear Regression
NH	National Highways

NHDP	National Highway Development Program
PCU	Passenger Car Unit
PDF	Probability density function
RUCS	Road User Cost Study
TRRL	Transportation Road Research Laboratory
TW	Motorized Two Wheeler
TwU	Two wheeler unit
VISSIM	VISualization SIMulation
3W	Motorized Three Wheeler
CC0	Standstill distance
CC1	Time headway parameter
CC2	Following distance variation

Chapter 1
INTRODUCTION

1.1 General

Study of the basic traffic flow characteristics includes analysis of the movement of different vehicles and the interactions they make with one another. For better representation and understanding of traffic flow behavior, fundamental relationships have been established among the traffic characteristics and the knowledge of which is an essential requirement in planning, design, and operation of roadway facilities. Traffic engineers assess the roadway traffic and its impact on proposed traffic flow system and identify primary locations and causes of deficiencies in order to improve the existing roadway system. All these tasks may be successfully accomplished only through a firm framework, which requires better understanding of traffic flow characteristics and their interdependences.

The analytical process consists of predicting an output as a function of specified inputs. This analytical process can vary from simple equation to a comprehensive simulation model. The important issues for analysts are their knowledge of the system being considered including its flow characteristics, and of analytical techniques and their appropriate selection for the problem at hand. Microscopic analysis may be selected for small or moderate-sized systems, where the number of transport units passing the system is relatively small and there is a need to study the behavior of individual units in the system (e.g. midblock section of a particular roadway, intersection on a particular roadway, etc.). Macroscopic analysis may be selected for larger scale systems, in which a study of the behavior of units is sufficient (e.g. city level or regional level road network).

The research work presented here is related with the study of microscopic and macroscopic traffic flow characteristics on multilane highways under heterogeneous traffic conditions. The research uses field data collected on various sections of multilane highways for analysing traffic characteristics and the same data is used as inputs for modeling traffic flow behavior on a simulated platform.

1.2 National highways in India

India has second largest road network in the world having the length of over 5.603 million kilometers as on 31st March 2016 (www.morth.nic.in). The National Highways (NH) are the primary long-distance roadways running through the length and breadth of the country, connecting all major cities and ports, state capitals, large industrial and tourist centers, etc. National Highways constituting length of over 1.01 million kilometers. Even though the National Highways represent only about 1.76% of the total road network length, they handle about 40% of the total road network (www.nhai.org). National highways in India are designated as NH, 'followed by the highway number'.

The National Highway in India usually has four or six or eight lanes, often separated with physical medians. In the category of multilane highways, National highways are not fully access controlled, as in the case of expressways. Unlike urban roads, there is no regularity of traffic signals on national highways and also, they have greater control on the number of access points per km. Also, design standards are generally higher than those used on urban arterials. Speed limits and free speeds on these highways are often higher than speeds of vehicles observed on urban arterials. Pedestrian activity, as well as parking, is considered to be minimal when compared with that on urban arterials. The vehicle composition on these highways is found to be different from that observed in urban arterials. The percentage of heavy vehicles such as buses, trucks, and, light commercial vehicles on highways is generally found to be higher side in compared to motorized and non-motorized two-wheelers and three-wheelers.

1.3 Homogeneous and heterogeneous traffic conditions

The characterization of heterogeneous and homogeneous traffic flow systems is mainly done on the basis of wide variations observed in static and dynamic characteristics of vehicles move on highways. The lane discipline behavior of drivers is also affected due to movement of vehicles with different size and speed performance. The difference between lane based and non-lane traffic conditions at midblock sections on multilane highway is shown by Figures 1.1 and Figure 1.2.

Figure 1.1 Lane-based homogeneous traffic flow condition

Figure 1.2 Non-lane based heterogeneous traffic flow condition

In homogeneous traffic conditions, vehicle follows strict lane discipline and move as traffic entity type whose characteristics do not vary much. Heterogeneous traffic comprises of vehicles with wide ranging static and dynamic characteristics such as vehicle size, engine power, acceleration/deceleration, maneuvering capabilities, etc. Due to the highly varying physical dimensions and speeds, the vehicles do not follow traffic lanes, and occupy any lateral positions based on the space availability. The vehicle types those are less capable in terms of maneuverability, cause significant level of impedance to the flow of vehicular traffic stream. Therefore, heterogeneous traffic conditions is prevailing particularly on roads in developing countries like India, poses a serious challenges to traffic planners and engineers in providing hassle free and safe traffic operations.

1.4 Measurements of traffic flow characteristics

The roadway traffic in India is highly heterogeneous which comprised of varying users characteristics and performance. For instance, roads may be occupied with the following vehicle categories such as Buses, Trucks, Light commercial vehicles comprising large vans and small trucks, Cars (Jeeps, big and small utility vans, Motorised three-wheelers (three-wheeled motorized vehicle carry passengers and small quantities of goods), Motorized two-wheelers, (motorcycles, scooters and mopeds), Bicycles, Tricycles (non-motorized) carrying passengers or small quantities of goods and Animal drawn vehicles or Bullock cart. The speeds of these vehicles vary from as small as 5 km/hr to as much as 100 km/hr. Hence, measuring traffic volume as number of vehicles passing a given section of road or traffic lane per unit time is inappropriate and provide false judgment in understanding of traffic flow demand. In order to measure traffic flow characteristics under mixed traffic conditions, the traffic flow behavior and interaction between the moving vehicles over a sufficient length of a roadway needs to be studied.

1.5 Traffic flow simulation under mixed traffic conditions

Traffic flow phenomena are highly complex under mixed traffic conditions, due to unpredictable interactions of vehicles over a road space. Moreover, vehicles do not interact simply by following the laws of mechanics, but also due to the behavior of human drivers onto impending or un-impending situations. However, the conventional approaches of assessing traffic flow behavior have some limitations, particularly, while dealing with complex situations involving considerable amount of stochastic driver behavior and attributes. An empirical approach is based on extensive field data, which is neither available nor easily collectable. The analytical approach has the limitation of underlying assumptions of homogeneity, which are far from the high variations of driver-vehicle characteristics in heterogeneous traffic conditions. In view of this, researches concentrated on appropriate modeling technique and the modeling approach through simulation that emerged as the most powerful, flexible and acceptable tool.

1.6 Need for the study

As discussed earlier, the capacity estimates under heterogeneous traffic conditions, cannot be given in terms of number of vehicles per hour and hence, it is expressed in terms of PCU per hour by converting the different type of vehicles into equivalent passenger cars using appropriate equivalency factors. Accordingly, PCU values for the different types of vehicles on Indian roads have been suggested by Indian Road Congress (IRC: 64-1990). The PCU values of different types of vehicles, proposed by IRC, are in the form of single set of constant values. Hence, it may be inferred that the PCU values are valid for a particular traffic and roadway conditions. In fact, PCU values of a vehicle type may not be a constant term, as it not only varies with vehicular factors but also with several other factors associated with roadway and traffic conditions. It may be appropriate to consider the PCU value of vehicle type as a dynamic quantity instead of considering it as static term. Hence there is a need to estimate and anlyse the PCUs of various vehicle types, through a comprehensive study of their interaction over a wide range of roadway and traffic conditions. This study is an attempt in this direction.

On Indian roads vehicles do not follow lane discipline and make their own virtual lanes instead of demarcated physical lanes. The conventional models ignores lateral and longitudinal behaviour of vehicles which affect macroscopic traffic stream behaviour. Field data always limited in describing behaviour of traffic flow stream in generalized manner as far as mixed traffic is concerned. Traffic Simulation is found to be a powerful tool to overcome such problems. Microscopic simulation is best method include complete details about the individual vehicles types having greater impact on traffic flow operation. Hence, simulation model is used in present study to generate traffic flow data that helps to analyse the mixed traffic condition which is difficult to observe in field.

1.7 Study objectives

The specific objectives of the research work are as follows:

i. To estimate the PCU values of different vehicle types under varying traffic and geometric conditions under heterogeneous traffic conditions on multilane highways.

ii. To develop the speed-flow relationship for capacity analysis on multilane divided highways through simulation.
iii. To analyze the lane-change behaviour of vehicles on multilane divided highways through simulation.

1.8 Scope of the work

The present work is on multilane divided highways with unidirectional traffic on both sides of the median. The study is restricted to straight sections of intercity highways with heterogeneous traffic flow. The study considers both paved and unpaved shoulder sections.

1.9 Organisation of the thesis

The research work is presented in nine chapters and the chapters are organized as follows

Chapter 1 gives the brief background of the topic of this study including its significance in the Indian context, the need for the study and the specific objectives of the research work.

Chapter 2, on review of literature, provides an overview of the earlier studies related to the subject matter of this research work. First, research works related to the study of traffic flow characteristics like speed, arrival pattern and headways are reviewed. Then, the research work related to the estimation of PCU values of vehicles and then, the research work related to calibration and validation of VISSIM. This is followed by a review of lane change and overtaking behaviour of vehicles on multilane highways.

Chapter 3 deals with the present study methodology and its discussion with the help of flow chart.

Chapter 4 describes the procedure and details of selection of study area and techniques and procedure adopted for field data collection for the study.

Chapter 5 provides analysis of field data like speed data analysis, headway data analysis, vehicle arrival data analysis, lateral placement of vehicles and overtaking behaviour of vehicles. The developed models for estimation of PCU's are also discussed.

Chapter 6 discussed about the input required for the simulation, analysis of simulation output and calibration of parameters.

Chapter 7 deals with the study of lane changing behaviour of vehicle type on different types of highways under mixed traffic with the help of calibrated traffic simulation model.

Chapter 8 deals with the use of the validated VISSIM model to estimate the PCU's of vehicle types at different volume to capacity ratios and proportional share and also discussed about development of Artificial neural network (ANN) models and Artificial neuro-fuzzy interface system (ANFIS) models to estimate the PCU's of vehicle types at different volume to capacity ratios and proportional share by using simulation data.

The summary and conclusions of the study, major research contribution, limitations of the study and the scope for further research are presented in **Chapter 9**.

CHAPTER 2

LITERATURE REVIEW

2.1 General

The review of literature is carried out in present study to understand the earlier research works done in the stated domain. In the light of the scope of the study, the review literature has been done under four heads, namely, (i) Studies related to the various traffic flow characteristics, (ii) Studies related to the speed-flow relationships, (iii) Studies on estimation of PCU, and (iv) Studies on lane change and overtaking behavior including lateral placement of vehicles.

2.2 Studies related to traffic flow characteristics

The vehicle speed, arrival and headway in traffic flow phenomenon are stochastic and random in nature. To represent these random variables at various flow levels, various probability distributions have been studied. The studies performed on speed, pattern of arrival of vehicles and time headway distributions are discussed as follows.

- **Studies on vehicle arrival and time headway characteristics**

Schul (1955) suggested the modifications to Poisson's distribution model for defining the rare events of traffic flow under homogeneous traffic conditions. The set of spacing between successive vehicles arrival was assumed to consist of subsets, each with a different mean and following Poisson law. The results obtained from modification of Poisson distribution showed agreement with the observed field data.

Weiss and Herman (1962), Breiman (1963), Brown (1972), Newell (1955, 1966) and Thedeen (1964) established the conclusion that the negative exponential distribution usually defines the headway distribution for traffic flows with unrestricted overtaking.

Lewis (1963) developed a model to determine the arrival time of a vehicle. The suggested model was based on the binomial distribution with two different probabilities of arrivals and it also incorporated the platooning effect of traffic stream.

Dawson and Chimni (1968) proposed a hyperlang model to describe free and constrained headways of vehicles in traffic flow stream. They suggested the distribution with a linear combination of translated exponential function (for free headways) and translated Erlang function (for constrained headways). The proposed Hyperlang distribution was fitted to the headway data at different volumes collected on sections of two-lane two ways roads.

Tolle (1971) considered lognormal distribution with three parameters for defining field headway distributions. The headway data for the study were collected on Interstate 71, Ohio in a volume range of 800-1900 vph. The graphical analysis revealed that with the increasing volume, headway frequencies tend to skew more towards left with less dispersion. At 5% level of significance, Chi-square test showed a good fit only under the volume ranges of 800 to 1200 vph, 1400 to1500vph and 1700 to 1800 vph respectively. However, Kolmogorov-Smirnov test conducted for goodness-of-fit has provided a good fit for the whole range of observed volume i.e. 800-1900vph.

Wasielewski (1974) reformulated the Semi-Poisson headway distribution model proposed by Buckley (1962, 1968) and derived an integral equation for calculating the constrained component directly from the observed headway distribution without introducing a parametric for the follower distribution.

Mahalel and Hakkert (1983) developed a model describing the sequence of vehicle arrivals in adjacent lanes of a multilane highway as a Markov renewal process. It was concluded that the presence of a vehicle in a particular lane, increases the chances of next vehicle arrival on the adjacent lane.

Ramanayya (1988) considered negative exponential distribution for highways at volumes less than 500 vph, shifted exponential distribution at 500 to 650 vph and log-normal distribution at 650 to 900 vph.

Mei and Bullen (1993) analysed headway distribution under higher traffic flow (2,500 to 2,900 vehicles/day) conditions on freeway section. The shifted lognormal distribution with a shift of 0.3 s or 0.4 s has provided a good fit for headway data.

Akcelik and Chung (1994) discussed negative exponential, shifted negative exponential and bunched exponential distributions for arrival headways. Bunched exponential distribution was found to produce more realistic results for both single lane and multi-lane traffic streams as compared to other two distributions.

Sahoo et al. (1996) studied the traffic flow characteristics on two stretches of NH-5 near Bhubaneswar city in Orissa. The negative exponential distribution was found to be in close agreement with the observed headway data.

Kumar and Rao (1998) analysed the observed headway data at locations of NH 5 and NH 6 in India. Chi-square test was performed at 5% significance level to ascertain the goodness of fit between the observed and theoretical frequencies. They found that the negative exponential distribution closely represents headway patterns for volume levels varying from low to moderate. A composite distribution which combines normal distribution (for vehicles in platoons) and shifted negative exponential distribution (for independent vehicles) was suggested for higher traffic volumes.

Chandra et al. (2001) analyzed the inter-arrival pattern of vehicles on a six-lane divided highway in New Delhi by using artificial neural network. The influence of traffic composition and volume on headway was studied. Headway between same vehicle categories found to be decreased with increase in traffic volume and their proportions in the traffic stream. A minimum headway of 1.72 s was recorded for all cars situation. Moreover, Chandra and Kumar (2001) collected the headway data on six-lane divided urban roads in New Delhi for further studies. Exponential distribution, lognormal distribution and hyperlang distribution were employed to fit the observed headway data collected on different sites. It was found that hyperlang distribution adequately fitted the data under volume range from 900 to 1600 vph.

Arasan and Koshy (2003) studied time-headway distribution of vehicles under heterogeneous traffic conditions on four-lane divided urban arterials, over a wide range of traffic flow from about 250 vph to over 6000 vph. A significant number of zero and near-zero headway were observed due to the presence of high percentage of two-wheelers in traffic stream. Negative exponential distribution was reported to describe the observed headway patterns of vehicles. Figure 2.1 shows relation between observed values of standard deviation and mean headway.

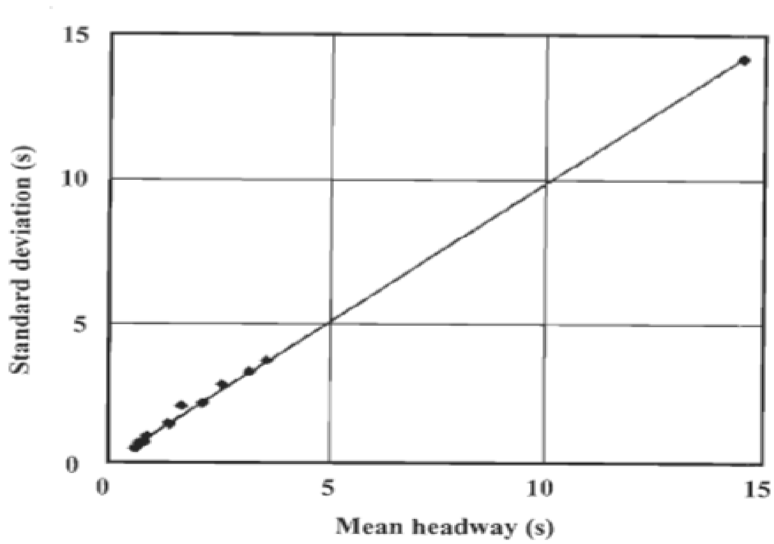


Figure 2.1 Relationship between standard deviation and mean headway (Arasan and Koshy, 2003)

Minh et al. (2005) studied motorcycle behaviour in Hanoi city of Vietnam. They collected traffic flow data on both two-lane undivided and four-lane divided roadways under varying traffic and geometrical conditions. The concept of MCU (motorcycle unit) was introduced to develop the speed-volume relationship based on observed traffic flow data. The mean speed and mean headway were compared at selected locations using statistical F-test and t-test. Authors reported average headway as 1.16s for all four locations and a standard deviation of 0.65s. It was observed that 50% of two-wheelers were found to travel in interval range of 0.5-1.0 s headways. In spite of differences in the geometric parameters, traffic composition and operations, all the four locations were reported to have same mean headway.

Xue et al. (2009) analysed time-headway distributions on expressways in Beijing, Shanghai, and Guangzhou cities, China. The time-headway data were collected on a typical weekday for this study during 6:00 a.m. and 6:00 p.m. This study identifies three types of distributions that best fit the headway data. It is found that the sections having traffic volume less than 250 vph fits negative exponential distribution to headway data. For traffic volume ranging 250 to 750 vph, data follows a sifted negative exponential distribution. And, for traffic volume ranging 750 to 1,500 vph, the time headways can be modeled with Cowan's M3 distribution model.

Panichpapiboon (2015) investigated the time-headway distributions of vehicles travel on an urban expressway in Bangkok, Thailand. Author characterized headway distribution and concluded that GEV distribution is most effective in modelling time headways. However, the exponential distribution was found to be the least effective distribution under heavy traffic volume.

- **Speed studies**

Haight et al. (1961) considered that the time speeds could be well represented by either a gamma or a log-normal distribution. These distributions offer the advantage that the same functional form is retained when the time speed distribution is transformed into a space-speed distribution and avoid the theoretical difficulty of the negative speeds given by the infinite tails of the normal distribution.

Whitby (1962) reported the results of studies conducted in 1960 on small-car speeds and their effect on highway capacity. Three locations were selected on four-lane divided expressways in the Washington Metropolitan area. No significant difference was noticed between operations of standard cars and small cars in terms of speed and spacing.

Oppenlander (1966) presented an extensive literature on factors affecting spot-speed characteristics in U.S. The study found that trip distance, vehicle type, vehicle age, traffic volume, traffic density and roadway characteristics like functional classification, curvature, gradient, length of grade, number of lanes, surface type, had significant influences on spot speed characteristics. Driver variable was observed to influence vehicle speeds to different degrees in various parts of the country. Geographic location, sight distance, lane position, lateral clearance, frequency of intersections, percentage of commercial vehicles, passing maneuvers, opposing traffic, access control, environmental variables of time and weather were among the other elements affecting vehicular spot-speeds.

Drake et al. (1967) examined various statistical hypotheses and tested them with traffic flow data of a freeway in Chicago. Speed-density relationship for all hypotheses was investigated due its transformability into linear functions. Greenshield's hypothesis predicted comparatively better results for mean free speed and three linear regimes hypothesis provided good estimates of optimum speed, maximum flow and jam density. The modified Greenberg hypothesis performed fairly with respect to all parameters examined and Underwood formula performed poorly except for highest goodness-of-fit (r^2) value. Edie formulation gave best estimates for fundamental parameters whereas the bell curve gave relatively low estimate of mean free speed for considerably higher value of actual mean free speed.

Ackroyd and Bettison (1979) reported the vehicle speeds on the motorway in Nottinghamshire during 1974-1978. They compared mean speeds and standard deviations for total traffic and for different vehicle classes. Other parameters like, mean speeds for different lanes, percentage of vehicles exceeding speed limits corresponding to each vehicle category, average traffic composition, distribution of vehicle classes between different lanes, and average traffic composition of flows in different lanes were also compared for different periods between 1974-78.

Kadiyali et al. (1981) studied free speed behaviour of vehicles at Delhi-Faridabad four-lane divided highway section of National Highway (NH-2), with 2.5 m wide earthen shoulders. Four vehicle types such as cars, buses, trucks, and 2-wheelers were observed on the highway. Platooning of vehicles and presence of slow moving vehicles were not included in the speed analysis. The speed distributions of vehicles were observed to follow the normal distribution pattern in the analysis.

Kadiyali et al. (1991, 1993) updated speed studies on Indian rural highways and compared the results with their earlier Road User Cost Study (RUCS). Free speed and speed-flow studies were carried out on single lane, two-lane, intermediate lane and four-lane (divided) rural highways. Speed data on a four-lane divided road followed normal distribution for each category of vehicles. The comparison of the findings with the earlier study showed an increase of 20-40 percent in speeds on four-lane roads and an increase of about 10 percent in speeds due to widening of two-lane roads to four-lane divided roads.

Casey and Lund (1992) studied the speed characteristics on roads connecting with four lane divided freeways at selected locations in San Luis Obispo, Ventura and Oxnard. The results were compared with a similar study conducted in 1985. The study reported travel speeds to be faster in 1988 than in 1985. Freeway drivers were observed to travel at significantly faster speeds on roads connecting with freeways than drivers approaching the freeway, due to speed adaption.

Shankar and Mannering (1998) proposed a structural model for relating mean speeds and speed deviations for each lane of a multilane roadway at macroscopic level. Data were collected using magnetic loop detectors on a three-lane divided freeway of Interstate 90, east of Seattle. Three-stage least squares (3SLS) approach was used for data analysis. The findings showed that in-lane speeds were influenced only by adjacent-lane speeds, whereas in-lane speed deviations were influenced progressively by adjacent lane speed deviations and also by in-lane and adjacent-lane speeds.

Chandra and Raj (1999) explained the role of shoulders in traffic operations. The authors concluded that for good condition of shoulders, heavy vehicles can travel up to 20 cm from pavement edge, leaving only 5.9 % pavement area unutilized. This loss may increase to 9 % for bad and 14.3 % for worst condition of shoulders. Similarly, they found that the reduction in speed varied from 2.7 % to 29.2 %, depending upon the traffic composition and condition of shoulder at site. Finally, they proposed that the combined effect might be analysed by developing speed-volume relationship with varying shoulder conditions.

Dixon et al. (1999a) applied highway capacity manual (HCM) to both 88.6 and104.7 kmph posted speed limit conditions on rural multilane highways in Georgia. Speed data were collected at 12 rural multilane stationary count locations by the Georgia Department of Transportation (GDOT). Posted speed limits of 88.6 and 104.7 km/h directly influenced free-flow speeds, and operating speeds on highways increased within the increased posted speed limit. As the HCM (year) rule-of-thumb overestimated the free-flow speeds for higher speed limit of 104.7 kmph, an alternative relationship was suggested as 91% of the 85thpercentile speed for both the 88.6 and 104.7 kmph. Dixon et al. (1999b) further reported that the free-flow speed on rural multilane highways for 89 and 105 kmph speed limits conformed to a normal distribution. The observed free flow speeds during daylight conditions were 1 kmph greater than those recorded during night and the average speed of heavy vehicles was greater than that of cars during night hours.

Abraham (2001) analyzed speed data on Ontario highways and recommended increase the speed limit from existing 100 kmph to 110-130 kmph and 105-110 kmph on two different highways respectively. Due to undisciplined driving behaviour, left lane reserved for passing operations was found to be utilized as a regular lane leading to almost same average speeds in both left and middle lanes.

Velmurugan et al. (2002) studied the change in operating speed characteristics vehicles on rural highways, based on the outcomes of Road User Cost Study (RUCS) -1982, 1992 and 2001. The comparison of results showed that there was significant increase in speeds of all vehicle categories on roads of different widths between 1982 and 2001 and also between 1982 and 1992. Basic Desired Speed (BDS) on four-lane divided highways with paved shoulders were similar to that on two-lane bi-directional roads with paved shoulders, representing insignificant impact of geographical factors on BDS. Free speed of new technology cars was observed to be 21 to 28 % higher than that of old ones on both two and four lane highways.

The study, on URUCS-2001, was undertaken by CRRI, New Delhi. Reddy et al. (2003) has presented the critical aspects of the study for Updation of Road User Cost Data (URUCS-2001). It was found that the free speeds of vehicles on four-lane divided carriage ways have increased drastically by 24 to 39 % in compared to the free speed data collected as a part of Road User Cost Study, in 1982.

Lees (2003) analysed speed characteristics of unidirectional traffic on interstate freeways including both urban and rural highways of Indiana. It was observed that the speed patterns did not follow normal approximation. A simulation model was developed in Mathematical to simulate one-way traffic on multiple lanes. A positive linear relationship between slow down traffic proportion and traffic density on two-lane and three-lane highways was observed. Passing rules also appeared to significantly influence the traffic slow down proportion and addition of a third lane improved traffic flow speed considerably.

Hunt et al. (2004) studied the short-term effects on driver speeds after increasing the posted speed limit from 100 km/hr to 110 km/hr on rural four-lane highways in Saskatchewan. Spot speed studies were conducted at 32 locations before and after the speed limit increase, out of which 23 locations were on four-lane divided highways. Data were collected on the four-lane highway system where the speed limit was raised as well as on four-lane sections and two-lane arterial highways where speed limits were not increased to identify any possible "haloeffects". The percentile distributions as well cumulative frequency plots were developed for before and after studies. No major changes in speed differential of vehicles was observed before and after studies.

Jain et al. (2005) analysed traffic operations on Mumbai-Pune expressway, a dual three lane carriageway (12.5 m wide on either side) with 7.6 m wide median and 2.5 m wide black topped shoulders. Traffic constituted mainly of cars (61%), trucks (18%) and buses (9%). For inner lanes, the average speed of traffic flow exceeded the speed limit of 80 kmph.

Jalihal et al. (2005) compared of traffic composition, volume, speed characteristics and travel patterns over a period of time, by taking case studies of some major cities in India. It was observed that vehicle composition and location of a road directly influence spot speed characteristics of vehicles. Figure 2.2 gives the cumulative speed curves for Delhi where in most cases, cars travelled with maximum speed followed by two-wheelers. Spot speeds increased from inner areas to outward areas.

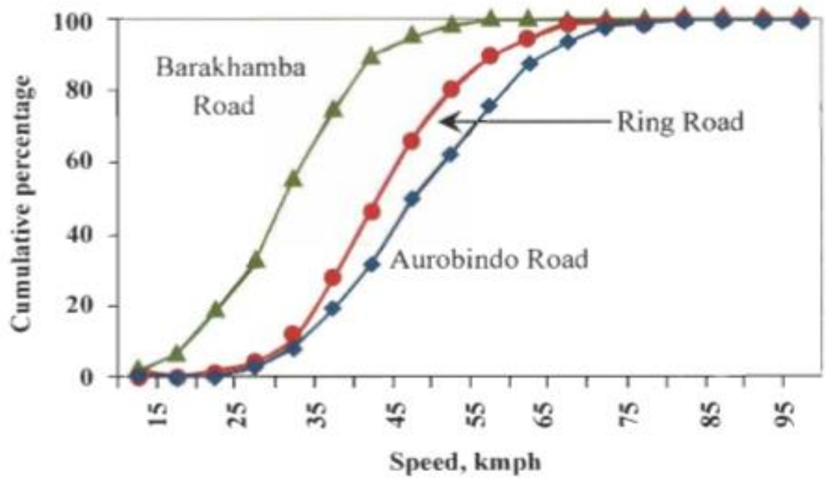


Figure 2.2 Cumulative distributions of speeds in Delhi (Jalihal et al., 2005)

2.3 Study on Speed-Volume relationship

Numerous field studies have been performed on traffic flow behavior and highway capacity was used further as the basis for developing appropriate design guidelines for highway and traffic engineers. Highway Research Board (HRB) of United State of America (USA) released the first Highway Capacity Manual (HCM) in 1965 that provided basic guidelines for capacity analysis under uninterrupted and interrupted traffic flow conditions. HCM (1965) suggested that the capacity of multilane divided highway was approximately 1800 PCU/hour/lane. Thereafter, the evolved capacity has been revised to 2000 PCU/hour/lane in US-HCM (1985). With the advent of microscopic simulation into the roadway capacity estimation, the capacity values were further refined to 2200 PCU/hour/lane and then 2400 PCUs/hour/lane in the latest version of US-HCM (2010). This clearly reflects approximately 75 years of comprehensive continual research on highway capacity and its analysis have gone into 5th edition of US-HCM (2010).

German Highway Capacity Manual (1994) provides the capacity standards for the practical application. Speed-flow diagrams were developed from simulation studies on rural highways to define capacity values for different terrain and roadway conditions. The capacity of a leveled and straight two-lane highway in Germany is 2500 PCU/hour, and for steep and curved roads it can be as low as 1000 to 1500 PCU/hour.

Indonesian Highway Capacity Manual (IHCM, 1997) provides capacity standards for Indonesian highways. The capacity on Indonesian multilane highways is significantly different from that suggested in HCM (2010) because of high proportional share of two-wheelers on Indonesian roads. The capacity has been estimated as 2300 PCU/hour/lane by considering the travel speed of light and heavy vehicles. The study performed by Bang et al. (1995) on rural roadway capacity was also incorporated for development of IHCM (1997). This manual also provides capacity reference values for different traffic composition based on the city size and population.

Danish Method of capacity analysis is the modified method of US HCM (2000) to suit traffic conditions on Denmark highways (Nielsen and Jorgensen, 2008). The adjustment factors for less ideal conditions cause a steeper capacity reduction in the Danish method than in the US-HCM (2000). The capacity under ideal conditions on a four-lane highway in Danish HCM is given as 2300 PCU/hour/lane.

Taiwan Area Highway Capacity Manual (2001) provides standard guidelines for estimating the capacity and level-of-service for Taiwanese roadways. Manual has its own methodologies for capacity analysis of highway segments in interrupted and uninterrupted traffic flow conditions. The Taiwan capacity manual also contains the relationships between traffic volumes, mean and free speed on multilane rural or suburban highways which are governed by its average space mean-speed. Tseng et al. (2005) further analyzed the traffic flow data collected at mid-blocks of seventy six sections of multilane rural and suburban highways in Taiwan aimed at revising Taiwan Area Highway Capacity Manual.

The HCM (2010) manual emphasize on two methods of assessment; one in which statistical method is used and, other is based on speed, flow and density relationships. In statistical method, the model is validated through observing the traffic volume distribution from field data (Chang and Kim, 2000). Manual describes the capacity and level-of-service analysis under uninterrupted and interrupted traffic flow conditions for multilane divided highway based on the fundamental speed-flow diagram. A typical speed-flow diagram for divided multilane highway is shown in Figure 2.3.

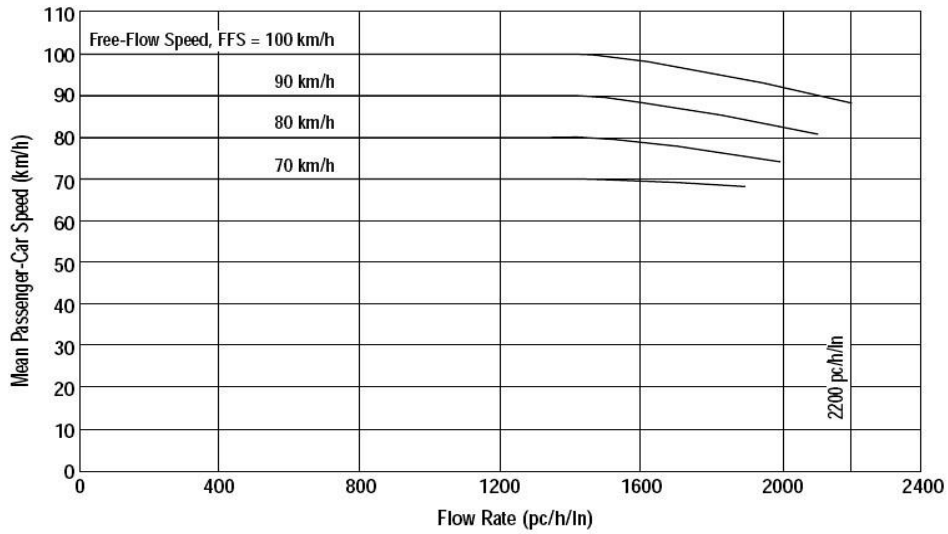


Figure 2.3 Speed-flow curve proposed for multilane highway in HCM (2010)

The speed-flow curves illustrated in HCM (2010) shows the mean free-flow speed is insensitive to traffic volume up to flow rate of 1400 PCU/hour/lane for all suggested base free-flow speed (FFS) conditions. Beyond this flow rate, significant drop (12 km/h for 100 km/h base FFS) in speed is accounted up to the point where rate of traffic flow approaches to maximum volume i.e. capacity. The procedure of capacity estimation based on the field data collected under specified traffic conditions is known as empirical method that further discriminated into direct and indirect empirical methods. Direct empirical method utilize authentic traffic flow data collected at field specific conditions and it derives conclusions based on the experiential results. Indirect empirical method follows the performance based guidelines and specifications provided in different research manuals and standard books or codes developed by different countries around the world.

HCM (2010) of US was also followed in Finland and Norway with minor modifications of design values to suit the prevailing traffic conditions. The roadway capacity estimated by modified methods is 2000 PCU/hour/lane for multi-lane highways.

In Sweden, the traffic flow analysis on multilane highways yielded a capacity of 4200 PCU/hour per direction for four-lane divided highways. The capacity values reported in different highway standards or manuals are given in Table 2.1.

Table 2.1 Roadway capacity evolved for multilane highway in different countries and conditions

Country	Highway Standard	Traffic Flow Condition	Capacity, PCU/hour/lane
USA	HCM (2010)	Uniform	2400
Denmark	Danish method	Uniform	2300
Finland and Norway	------	Uniform	2000
Sweden	Swedish HCM (1997)	Uniform	2100
Taiwan	Taiwan area HCM (2001)	Mixed	2200
Indonesia	Indonesian HCM (1997)	Mixed	2300
Korea	Korean HCM (1992)	Mixed	2200
Germany	German HCM (1994)	Uniform	2500

The road side environment and activities (side frictions) have a role in reducing the speed. The effects of these factors given extra attention in IHCM manual, in comparison to those given in Sweden, Finland/Norway manual.

The literature suggests some direct and indirect methods for estimating capacity of highway by using field data collected on various sections. Method of headway, observed volume, and speed-volume-density graphs are considered as direct methods. Many a times, these methods are quite complicated and require a complete range of traffic field survey data until the severe congestion on a roadway is encountered.

Greenshields (1934) measured traffic flow behavior by collecting the traffic flow field data on a highway using photographic technique. Data was analyzed to establish relationship between three basic traffic stream variables namely, speed, volume and density. The straight line equation was fitted between stream speed and traffic density which formed a polynomial speed-flow relationship to estimate maximum traffic flow on the section of highway. The traffic flow diagrams developed by Greenshields shows value of maximum traffic flow at its optimal density.

After a bivariate model between speed and flow introduced by Greenshields using fundamental traffic flow theory, various researchers around the world attempted to fit field collected traffic flow data into different mathematical models (Greenshields, 1934; Greenberg, 1959; Underwood, 1961). Hoban (1987) performed the similar study by collecting field traffic flow data and summarized the linear form of speed-volume relationship that can be used to determine highway capacity. The study recommended that the capacity of a highway is strongly dependent on the intercept and slope of a straight line fitting speed and volume data. The study stated that the capacity of a highway depends on several other factors such as geometric or environmental conditions etc.

Ryan and Breuning (1962) studied fundamental relationships of traffic flow on a six-lane divided urban expressway in Detroit, to get characteristics of traffic flow at high volumes. They found boundary speed of 40 mph (64 kmph) between critical and non-critical flows for both speed-volume and speed-density relationships. A comparison of range of speeds and change of average speed between successive one minute intervals reflected higher values for critical flow indicating increased internal friction as compared to non-critical flow. The fundamental relationships between speed, volume and density were found to be linear within non-critical flow. However, linearity was lost when the flow becomes unstable, which marked the boundary between non-critical and critical flow.

Drake et al. (1967) fitted generalized shape of speed-flow polynomial function for highway data in Chicago. The speed was used as independent variable and the equation for flow variable was determined that conformed the capacity value of 1800 pc/h/lane. The procedure was used to establish speed-flow curve that in fact did not actually fit the original speed-volume data as shown in Figure 2.4. Many empirical works on speed-flow relationship were summarized by different researchers who proposed generalized shape of speed-flow curves.

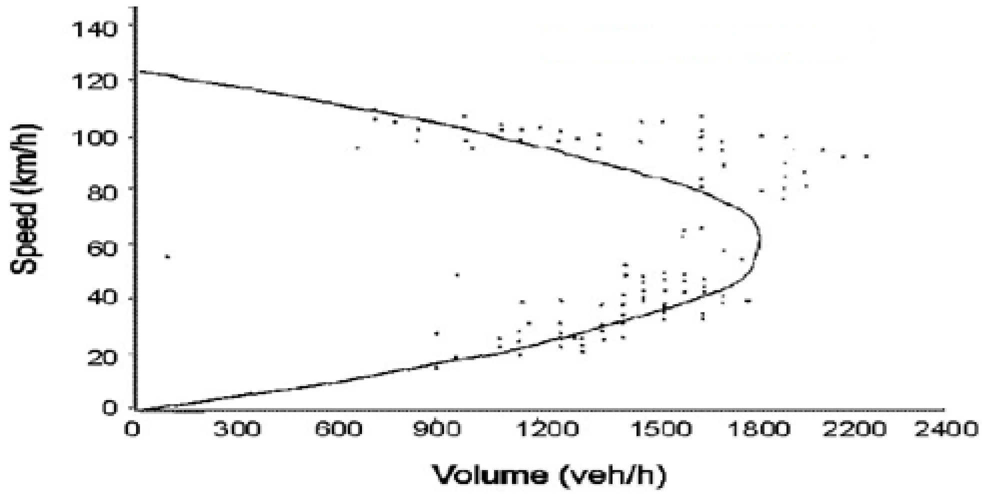


Figure 2.4 Capacity curve fitted to traffic flow data in Chicago (Darke et. al., 1967)

Duncan (1974) calculated density from speed and flow data and fitted a straight line between speed and density relationship to establish speed-volume relationship. Author also described the effect of traffic composition on speed-flow relationship and variation of slope of the curve was compared from one roadway section to another section.

Kadiyali et al. (1981) estimated the capacities and free-speed of vehicles on sections of four lane divided carriageway in India. They found speeds of vehicles are significantly affected by the factors such as pavement width, curvature, rise and fall. Kadiyali et al. (1982) developed mathematical equations for predicting speed of different vehicle categories under different conditions of road, traffic volume and composition. Data collected at 23 sites of single lane, two-lane, intermediate lane and four-lane carriageways were analysed using multiple linear regression analysis. Free speeds on four-lane divided carriageway under wet and rainy conditions were found to be lesser than on dry conditions. Equation 2.1 gives a typical speed-volume relationship for dual carriageway in plain terrain with low curvature. Figure 2.5 represents the theoretical speed-volume curve based on the linear speed-flow relationship, for four-lane divided carriageway with high curvature.

$$V_c = 51.24 - 0.00345 Q_{TV} \quad R^2 = 0.456 \qquad (2.1)$$

Where, V_c = speed of cars, kmph and Q_{TV} = volume of all vehicles, vph.

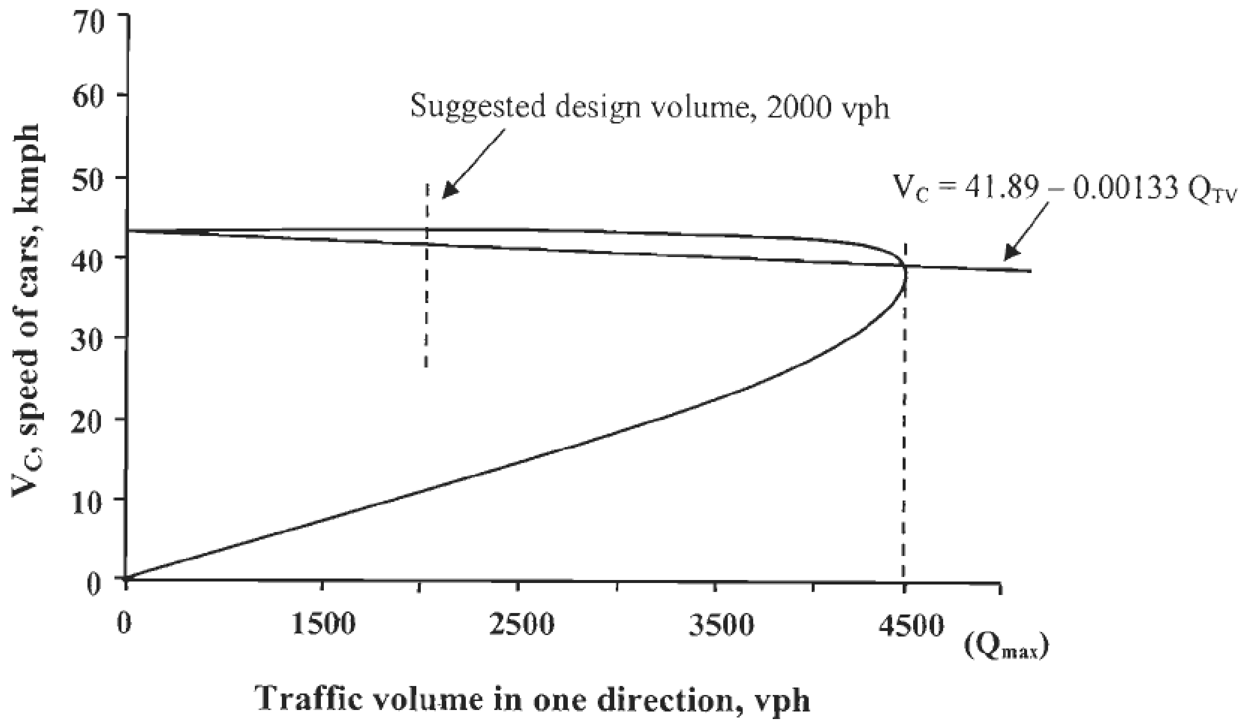


Figure 2.5 Speed flow relationship for four-lane divided road in plain terrain with high curvature (Kadiyali et al. 1982)

As a part of Road User Cost Study conducted by CRRI (1982), the simple linear regression equations were plotted, using the total traffic volume as independent variable and average speed of a particular vehicle category as dependent variable with intercepts agreeing with the free speed values appropriate to the type of road. Further, the theoretical parabolic curve of speed-flow relationship was superimposed over these dropping straight lines for each of the road categories. The point of intersection made by a dropping line and a theoretical parabolic curve was considered as the point of capacity. The design-service-volume may be derived using a Level-of-service (LOS) of B, which is about 45-50 % of the maximum capacity.

Huber (1982) proposed a methodology for estimating passenger car equivalency factors for vehicles under free-flowing conditions on multilane highways. Measure of impedance used as a function of traffic flow that related with two types of traffic streams; proportion of truck mixed in passenger cars traffic stream and passenger cars only. PCE values are related to the ratio of two streams volumes at some common level-of-service. Proposed model is based on the concept that a truck covers more space than a passenger car, which reduces the lane capacity. This model is used to estimate PCU of trucks to develop speed-flow relationship. The author suggested that the PCU value vary with speed, length and, percent share of the subject vehicles in the traffic stream.

Hyde and Wright (1986) studied temporal variation of traffic volume for determining highway capacity by using direct probability method and asymptotic value method. The methods applied in this study require the extreme value of observed flow or flow rate at capacity. The prediction of the largest possible value capacity was made by deploying direct probability method which was based on the assumption that the observed traffic volume must conform to theoretical model volume. The results derived were finally accepted as the largest flow value as the predicted traffic flow matched with the field observed values and the constraints for capacity was apparent when asymptotic method was applied to the similar study.

Ramanayya (1988) has emphasized on the fact the capacity standards adopted in western countries do not take into account the mixed traffic characteristics prevalent in India. The author, through simulation technique, estimated service volumes of 600, 1000 and 1500 western passenger cars, respectively, for levels of service of A, B and C on two-lane roads. Tentatively, a value of 35000 PCU/day has been proposed by Indian Roads Congress (IRC: 64-1990) for four-lane divided carriageways, located in plain terrain for level- of- service (LOS) of B i.e. volume to capacity (V/C) ratio of 0.5.

Tanaboriboon and Aryal (1990) studied the effect of vehicle size on highway capacity in Thailand. All vehicles observed on field were classified into small, medium and large size to make analysis more convenient and their respective time headways were calculated to derive the basic capacity value. Capacity of highway was taken as the reciprocal of time headway as one of the effective parameter to understand car-following behavior of leading and following vehicles of the same size. It was found that medium size vehicles do not affect the lane capacity. Roadway capacity was estimated to be 2100 PCU/hour/lane which was 9% more than the basic capacity adopted on highways in Thailand.

Kadiyali and Vishwanathan (1991) has presented the critical aspects of the study on Road-User-Cost Data Updation (URUCS-1991). The capacity of a four-lane divided carriageway, with paved shoulders on plain terrain, was found to be in the range of 90,000 to 1, 20,000 PCU/day. It was also proposed that if the paved shoulders are omitted, the maximum flow, a four-lane road in plain terrain can accommodate, is in the range of 80,000 to 1, 00,000 PCU/day. The authors also stressed the need to carry out further studies to arrive at more accurate values.

Nakamura (1994) introduced the new research methodology to assess capacity of multilane highway in Japan. He revised the method of deciding the number of lanes and analyzed the adjustment factors for converting basic capacity as maximum service flow rate to determine the possible capacity flow rate. Road side adjustment factors in Japan were divided into three different areas including undeveloped area, slightly developed area and developed area.

Vaziri (1995) studied speed and traffic flow data of six-lane and eight-lane basic freeway sections in Iran. Six models namely Greenshields, Greenberg, Underwood, Drake, Pipe & Munjal and Drew, were evaluated for the observed traffic data. No significant difference was observed in the sum of square of errors of all the calibrated models.

Rao and Mookerjee (1997) used Highway Capacity Manual (1994) of USA to estimate the service-flow rates and traffic capacities for a range traffic variables such as terrain type, the percentage of truck traffic, directional distribution, the peak hour factor and the K-factor (peak hour traffic as percentage of AADT). The authors presented the estimated traffic capacities as traffic volume matrices, which may be helpful for planners in the determination of number of traffic lanes for the projected traffic of given composition and nature.

Kumar and Rao (1998) studied the speed, density and flow characteristics on some stretches of NH-5 and NH-6 to establish their relationships. The authors concluded that speed decreases with increase in traffic density, indicating a linear relationship between them. The speed-flow relationships indicate that, with the increase of traffic volume, the speed decreases. Moreover, the data was found to be not adequate for estimating the capacity values, as the traffic conditions on the observed stretches have not reached the level, where it could be possible to estimate the capacity values.

Zhou and Hall (1999) studied the speed-flow relationship under congested conditions on a freeway. Through the analysis, the authors pointed out that it is important to have data from a full range of flows in order to properly fit a curve that is meant to represent the congested part of the speed-flow curve. Further, it was pointed out that the data collected from several sites need to be combined in order to establish speed-flow relationships within congestion.

Chang and Kim (2000) presented a quantitative method for the estimation of highway capacity and formulated an alternative approach of deriving capacity from statistical distribution of observed traffic volume. The study used headway method to determine the capacity statistically. The time headway is considered as the reciprocal of traffic volume on highway. The variance in the confidence interval obtained from this method greatly affected the estimated capacity.

Jian et al. (2000) developed the new vehicle classification methodology for determining expressway capacity in China. Authors described the limitations in the analysis by using traditional vehicle classification, as roads in China, consists of different physical and operating characteristics of vehicle types. The free-flow speed was estimated based on new vehicle configuration and wheel base frequency by proposing three new vehicle classification schemes based on performance characteristics of different vehicles. After analysis of the performance characteristics of new vehicle-types, validation was performed by using another set of local data. The new vehicle classification was proven and applied in China successfully for estimating capacity of expressways.

Fellendorf and Vortisch (2001) performed on analysis for validation of microscopic traffic simulation model VISSIM on German and US roads in different real world conditions. Authors calibrated the microscopic traffic simulation model VISSIM by setting the distribution of desired speeds from the study conducted on the German highways. They simulated the German freeway data at different volumes and proportions. The simulated speed-flow curves of German (left) and US (right) freeways are shown in Figure 2.6. The simulated capacity found for German two-lane freeway around 3500 vehicle/hour, which is the typical value for German non-commuter traffic. For the US roads, the simulated capacity of five-lane freeway obtained as12000 vehicle/hour.

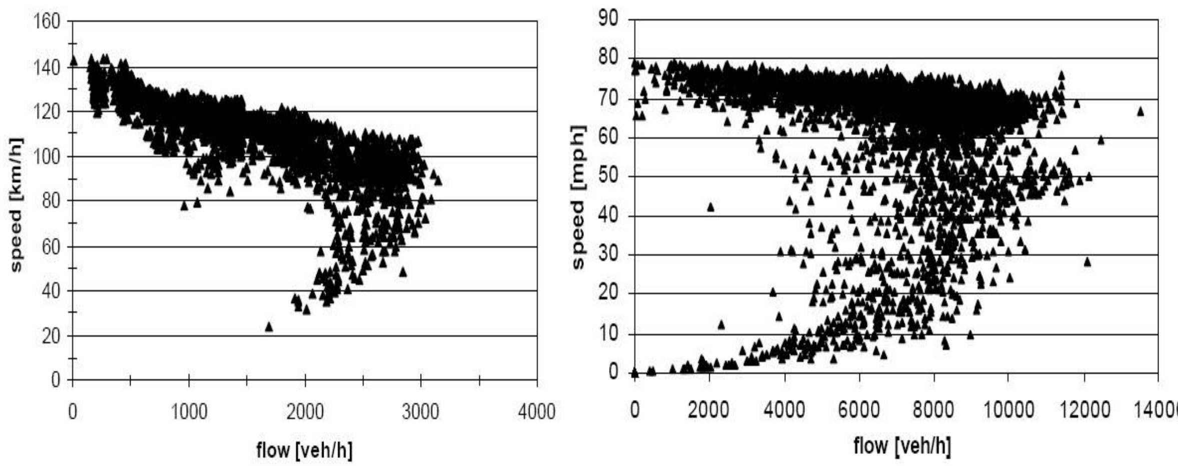


Figure 2.6 Speed-flow curves on German and US freeways

Minh et al. (2005) developed speed-flow relationships for motorcycles on four-lane divided roads in Hanoi, Vietnam. MCU (motorcycle unit) was developed for each vehicle category to study the parameters of heterogeneous traffic. F-test and t-test were used to compare the speed variances of two locations. Maximum speed, minimum speed and mean speed values showed a reduction of 10 kmph at sections 3 & 4 with respect to sections 1 & 2. Frequency distribution of motorcycle speed corresponded to normal function at 5% level of significance.

Yang and Zhang (2005) established the fundamental flow relationships based on extensive field traffic flow survey data collected on multilane highways in Beijing. Statistical test is used to investigate the effect of number of lanes on highway capacity. The analysis of variance and t-test was applied to check the differences in estimated capacity values. The average capacity per lane was found to reduce with addition of extra lane to a highway. They suggested ideal capacity values for four-lane, six-lane and eight-lane divided highways as 2250, 2100 and 2000 PCU/hour/lane.

Lownes and Machemehl (2006) investigated the effect of individual driver behavior parameter on capacity of a freeway in the Dallas. Author investigated the values of driver behavior parameters are significantly influenced capacity of a roadway individually. Further analysis was performed by Lownes and Machemehl (2006) focused specifically on the relationship between CC0 and CC1 and with other driver behavior parameters. Location of bottleneck was used as study section near interchange. The individual impacts of CC2 and CC4/CC5 were found effective at 0.05 level of significance. The evidence suggested that the effect on capacity by increasing or decreasing CC4/CC5 is the same and regardless with change in the value of CC2. CC4 and CC5 are negative and following thresholds in driving which governs sensitivity to the acceleration of preceding driver. Further, the authors suggested that CC7 have no significant influence on capacity due to its interactions with CC4/CC5. They found the impact on capacity with change in parameter CC8 and the parameters CC4 and CC5 are primary dependent on the values of CC0 and CC1, respectively. To summarize, they stated that the capacity of roadway highly dependent upon CC0 and CC1 parameters during the process of calibration of the simulation model for roadway capacity as minor error alter the capacity drastically.

Menneni et al. (2007) demonstrated the applicability of a microscopic simulation model by matching speed-flow graphs for a highway segment. A small scale test network simulation model was developed to implement that for a large scale simulation model. The study was performed with help of evolutionary algorithm. The results suggested the developed evolutionary algorithm based on objective of matching speed-flow graphs was even close to the default values.

Laufer (2007) estimated the capacities on freeways and investigated the applicability of the VISSIM microscopic simulation software to replicate the flow. It was noted that the changes in capacities appeared to be based on the observed throughput rather than clearly defined flow rates. This study examined the refinement of the car-following algorithm to adjust the modeled throughput. It was found that the calibration parameters of the Wiedemann-99 car-following algorithm can represent capacities presented in 2000 edition of the Highway Capacity Manual.

Chitturi and Benekohalin (2008) described a procedure for calibration of VISSIM for freeways to obtain the desired capacity and queue length. The calibration procedure has been developed for freeway work zones application. In this study the speeds of free flowing vehicles in work zones on interstate highways in Illinois with 55 mph speed limit were used as desired speed distribution input to VISSIM. The range of capacity was evaluated with different CC0 and CC1 parameters. The effects of CC0 and CC1 on capacity were quantified and the relationship is shown in Figure 2.7. It is observed that the range of capacity tends to decreases as the value of CC1 is increasing.

At CC1 values below 0.8 seconds, the variance in the capacity was ordered with magnitude more than the variance when the CC1 value is above 0.8 seconds. Authors stated that the defining parameter of CC1 below 0.8 seconds in VISSIM is not recommended.

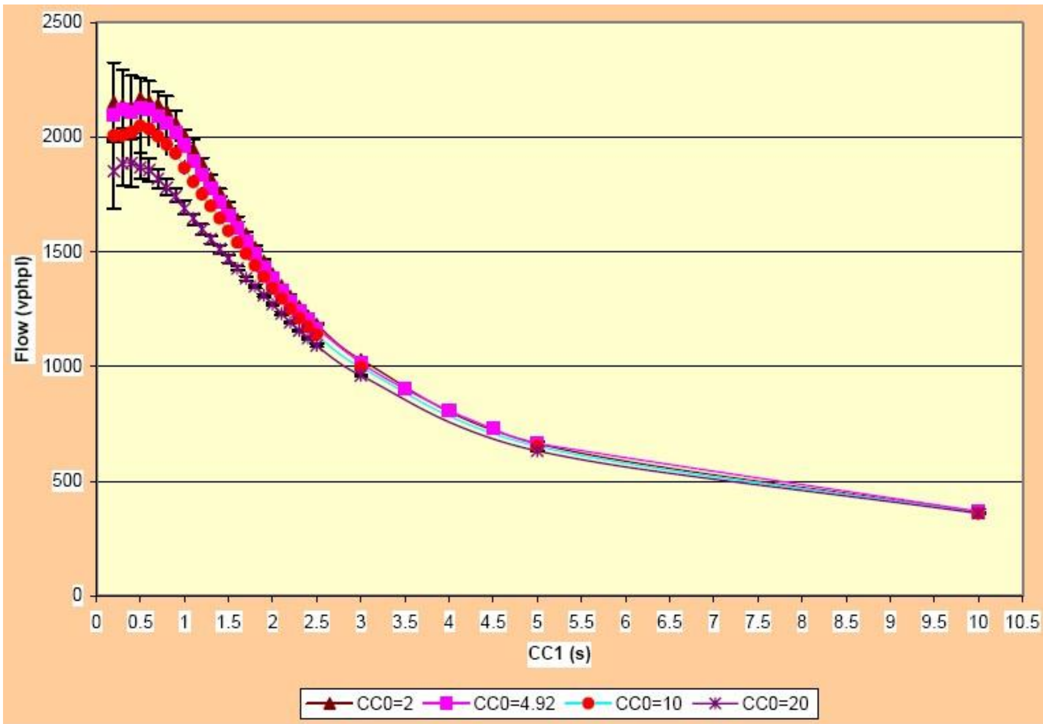


Figure 2.7 Effect of CC0 and CC1 on capacity (Chitturi and Benekohalin, 2008)

Rotwannasin and Choocharukul (2008) examined the applicability of HCM (2000) procedure of capacity analysis to highways in Bangkok. Traffic flow characteristics such as vehicular speed, flow and density were collected and extracted by the image processing unit. Existing traffic flow models were investigated by fitting field traffic data into single-regime model (Greenshields, 1935; Greenberg, 1959; and Underwood, 1961). They found, capacity reduced substantially due to higher number of access points along the multilane highway, especially on shoulder lanes and results do not conform to the HCM (2000) guidelines.

Shukla (2008) studied the traffic flow behavior on four-lane divided highways with varying traffic volume and shoulders conditions. Author described the arrival pattern of vehicles, speed characteristics, lateral placement of vehicles and overtaking behavior through field data. It was observed that the provision of paved shoulders significantly influences the placement behavior of vehicles on four-lane divided highways. The rate of acceleration during overtaking and lane changing behavior was also found to be influenced by shoulder conditions (paved and unpaved) and inverse correlation was found between the two.

Edara and Chaterjee (2010) calibrated and validated a few driver behavior parameters in microscopic simulation model VISSIM affecting the capacity of multilane highways in work zone. Data were collected on various sections in US highways having significant truck percentage in traffic composition. The default values related to static and dynamic characteristics (size, speed, and acceleration/deceleration etc.) of truck were modified as per field data. The capacity of work zone was determined from field data and VISSIM was simulated to estimate the capacity based on the default values of all parameters. The field and simulated capacity were compared on the basis of percentage share of HV mixed in the traffic stream. To represent critical driving parameters affecting capacity the multivariate regression models was developed. The parameters CC1 and CC2 in VISSIM were found as most influential on work zone capacity. The relationship was developed between the influencing parameters and lane configuration, percentage of trucks and work zone capacity. Validation of model was also performed using field data from work zone on different sections. Authors suggested that the calibrated values of parameters are capable to produce better estimate of capacity and queue lengths.

Fellendorf and Vortisch (2010) discussed calibration of VISSIM based on microscopic data and speed-flow is compared with the field data. Techniques to calibrate the core traffic models like driver behaviour model in VISSIM is discussed by the author and the interfaces of VISSIM like COM interface is discussed which describe that how VISSIM models can be regenerated using other programming languages.

Velmurugan et al. (2010) explicitly studied the speed - flow relationship on 21 multi-lane highways encompassing four-lane, six-lane and eight-lane divided carriageways in plain terrain. Speed profiles and speed - flow equations for different vehicle types for varying widths of multi-lane highways in the country has been developed based on traditional and microscopic simulation models and subsequently roadway capacity has been estimated. Figure 2.8 shows the impact of lane change behaviour on roadway capacity of four-lane, six-lane and eight-lane divided carriageways.

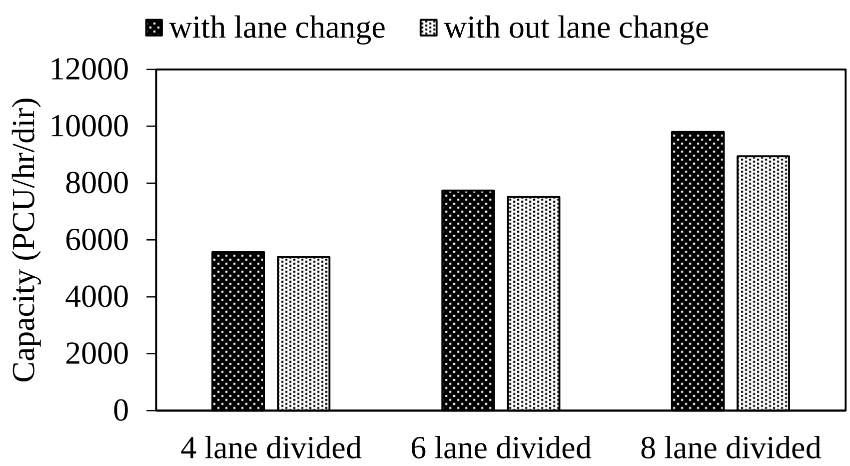


Figure. 2.8 Impact of Lane Change Behaviour on Roadway Capacity on multilane highways (Velmurugan et al., 2010)

Arasan and Arkatkar (2011) estimated the capacity values on multilane intercity highways in India under heterogeneous traffic conditions. They used a simulation model to investigate the effect of influencing factors on PCU of a vehicle type. Speed-volume curves were developed using simulation model and the estimate roadway capacity values were estimated to be 4600 PCU/hour in case of four-lane divided carriageway and 7200 PCU/hour in case of six-lane divided carriageway.

Madhu et al. (2011) also developed free-flow speed equations for high-speed multilane highways in plain terrain on both straight as well as curved sections. They found that speed limits observed for different vehicles on eight-lane sections were significantly higher than those observed on four-lane and six-lane sections. Authors have also noted that free-flow speed of different vehicles is significantly increased from four-lane to six-lane divided carriageway except for small cars, 2W and LCV.

Shukla and Chandra (2011) developed a simulation model for simulating multilane highways. Overtaking and lane changing behavior of different vehicles observed in the field to incorporate them in simulation model. The simulation run performed to develop speed-volume curves for determining capacity of four-lane divided highway having earthen and paved type of shoulders under mixed traffic conditions. General relations were proposed to calculate the capacity of mixed traffic under a given traffic composition. The Equations proposed in this study for determining capacity of four-lane divided highway having earthen and paved type of shoulders under mixed traffic conditions are shown in Equations (2.2) and (2.3).

$$\frac{100}{C_{mix}} = \frac{P_{car}}{4770} + \frac{P_{HV}}{1300} + \frac{P_{tractor}}{877} + \frac{P_{3W}}{2970} + \frac{P_{2W}}{13520} \tag{2.2}$$

$$\frac{100}{C_{mix}} = \frac{P_{car}}{5290} + \frac{P_{HV}}{1349} + \frac{P_{tractor}}{892} + \frac{P_{3W}}{3097} + \frac{P_{2W}}{15088} \tag{2.3}$$

Putranto and Setyarini (2011) analyzed the impact of vehicle composition and lane distributions phenomenon on the capacity of Indonesian multilane highways. They observed that the percentage of motorcycles in traffic stream is higher than the default values provided in the IHCM (1997). The analysis was performed for four-lane and six-lane divided highways and vehicle distribution coefficients were determined for each category of vehicles.

Madhu et al. (2011) used a simulation model PARAMICS to study the speed-flow equation of different vehicles on the eight-lane expressway. The study finds the capacity of eight-lane expressway under no virtual lane condition reduced by 15 per cent. Modeling of traffic flow is currently of great interest in too many transportation professionals due to a rapid increase in vehicle population at the highway.

Sinha et al. (2012) performed a comparative study on effect of motorcycle proportions on capacity of four-lane urban roads in India and Thailand. Traffic flow data was collected on four-lane divided urban road at six different sections in Thailand and ten different sections in India. Traffic volume in Thailand consisted of high percentage of motorcycles and, therefore, dynamic PCU were estimated to convert heterogeneous traffic volume and speed-volume relationship was developed to determine capacity. Linear regression was carried out to propose the relationship between capacity and motorcycle percent share for the highways in both countries.

Ah-Semeida (2013) analyzed the roadway capacity and level-of-service by considering roadway geometry and percent share of heavy vehicles as two important parameters for study. Statistical modeling was done in preliminary stage of analysis and in final stage Artificial Neural Network was modeled. Field traffic flow and geometric data were collected at 45 different locations of desert and agriculture sections of multilane rural highways in Egypt. ANN provided better results than the regression models for predicting capacity and density for highways. Increase in 5 percent share of HV resulted to increase in density by 2.3 veh/km/lane and increase in pavement width by 2 m leads to decrease in density by 1.5 veh/km/lane. Increase in lane width from 3.6 to 3.7 m increases the capacity of section from 1940 to 2115 veh/hr/lane for the sections located in desert.

Mehar et al., (2013) performed calibration of simulation model and developed speed flow curves, for 4-lane divided highway. The speed flow curves from both field data and simulated data were compared. For the simulation only two driver behaviour parameters are taken into consideration i.e. CC0 (standstill distance) and CC1 (Time Headway in seconds) as these two parameters are most effective parameters for calibration. The simulation is first run for homogeneous traffic conditions having only one out of different categories of vehicles and then results are aggregated to get value of parameters for mixed traffic conditions. Due to aggressive nature of drivers a slightly higher value of parameters like headway and standstill distance will better reflect mixed traffic behaviour. Methodology is also suggested for aggregation of traffic data to mixed traffic data.

$$CC0_{mixed} = CC0_{car} \times P_{car} + CC0_{TW} \times P_{TW} + CC0_{3W} \times P_{3W} + CC0_{HV} \times P_{HV} \qquad (2.4)$$

$$CC1_{mixed} = CC1_{car} \times P_{car} + CC1_{TW} \times P_{TW} + CC1_{3W} \times P_{3W} + CC1_{HV} \times P_{HV} \qquad (2.5)$$

Where, $CC0_{car}$ and $CC1_{car}$ are the CC0 and CC1 values for ‘all cars’ situation and P_{car} is the proportional share (in fraction) of car in traffic stream.

For the validation the same test is done on different section of 4 lane highway and simulated capacity found is 5329 pcu/hr which is comparable with field capacity of 5277 pcu/hr.

Sharma et al. (2014) collected field data on two National Highways having four and six lane divided carriageways. The videography technique was used to capture the speed-flow data for a period of 12 hour. Simulation study was also conducted to assess the behavior of traffic flow. Further capacity analysis was worked out for four and six lane divided carriageway using static and dynamic PCUs. The value of capacity with respect to static and dynamic PCU for four and six lanes divided National highway was 2200 and 2250 PCU/lane/hour/direction and 2166 and 2233 PCU/lane/hour/direction respectively.

Mehar et al. (2015) studied the effect of traffic composition on capacity of multilane highways. They considered the four-lane and six-lane divided highways in India. The vehicles on a highway were divided into five categories namely standard car, big car, heavy vehicle, motorized three-wheelers and motorized two-wheelers. VISSIM software was calibrated and speed-flow curves were developed to find simulated capacity values for different combinations of standard car and one of the remaining four types of vehicles in traffic stream. Finally they proposed the generalized equations to determine capacity.

2.4 Studies related to estimation of PCU

The problem of quantification of volume under heterogeneous traffic has been addressed by converting the different types of vehicles into equivalent passenger cars and expressing the volume or capacity of roads in terms of PCU per hour. The traffic movement under heterogeneous traffic condition differs, however, significantly from that of homogeneous traffic in respect of the pattern of occupancy of road space by vehicles. The HCM (1965) introduced concept of PCE or PCU, for first time, to express the traffic volume or capacity in terms of passenger cars per hour per lane (Pcphpl) by considering the effect of heavy vehicles in the traffic stream. Though PCU values recommended by these organizations are widely applied in many countries, their applicability is higher under debate due to the varying local traffic stream conditions prevailing in different parts of the world and within a country. HCM, (2000) provides different sets of PCE to be used for different types of highway facilities; i.e. two-lane highways, multilane highways, and freeways. The traffic under fairly homogeneous traffic conditions, with cars constituting about 80 % or more of the vehicles, has lane discipline, and the volume or capacity under such situations may be expressed in terms of PCU per hour per lane. Even, the effects of the smaller percentage of other vehicles (heavy vehicles) present in the traffic stream has been addressed by incorporating appropriate adjustment factors in the formulation for capacity estimation.

Guinn et al. (1970) proposed the headway method for the PCU estimation at highways and intersections considering a traffic stream in which only two categories of vehicles are present which are cars and trucks. With the introduction of the computer simulation technique the effect of mixed traffic could be evaluated for any road and traffic condition and PCU values could be calculated.

St. John (1976) developed a microscopic simulation model to derive nonlinear truck factor for two-lane highways including all important factors those can affect the flows. Results from the simulation model indicated that the truck factor should be nonlinear. This paper presented a brief description of the simulation, the evidence for a nonlinear truck factor, and the derivation and testing of the nonlinear factor.

Craus et al. (1980) reviewed and discussed the HCM methods to determine the PCE, and then suggested and evaluated a revised method which he has developed for the determination of passenger car equivalencies (PCE) for large sized vehicle like truck and bus. In the HCM 1965, the PCE was evaluated by the ratio of the theoretical number of passings of one truck to the average theoretical number of passings of one passenger car, which was not directly related to the opposite-traffic volume. The HCM overestimated the deterrent factor of trucks to the traffic flow which did not establish the specific nature of the effect of opposing downgrades, it showed only an average picture of truck performance where the real-world variation was completely disguised. Thus, doubt arise regarding the validity of the PCE for trucks through this approach.

Joseph et al. (1980) proposed a model for determination of PCE value by considering actual traffic delays caused and volume of opposing traffic. The PCE values which they reported were significantly lower as compared to the HCM values, but followed same variations.

Hu and Johnson (1981) described how to use 1965 HCM to find PCEs based on speed. They used equation developed by John and Glauz (1976) to calculate PCE. Operating speeds were based on design charts obtained by research performance by the Mid-west Research Institute (MRI). The PCEs were calculated based on extended freeway segments.

Roess and Messer (1984) identified three approaches in their research for estimating PCU values i) the constant volume to capacity approach, ii) the equal density approach and iii) The spatial headway approach. They have performed a study which had been directed toward the calibration of passenger car equivalent (PCE) values for trucks. Opportunity had been provided by these studies to review the PCE values for uninterrupted flow contained in Transportation Research Board Circular 212, "Interim Materials on Highway Capacity." The results of these efforts and their implications for highway capacity analysis were reviewed. Specific recommendations for revisions of the PCE values of Circular 212 have been made.

Ramanayya (1988) considered the Western car as the Design Vehicle Unit DVU and estimated the PCU factors for different vehicle types at different levels of services. He also used the simulation technique to develop Design Vehicle Unit (DUV) for mixed traffic which indicated that Indian car and bus have almost the same equivalency factors in terms of a western car at all LOS. Further equivalency factor for a truck is more than that for a bus or passenger car at different LOS. Bus and truck because of large size and poor acceleration and deceleration capabilities, have more damaging effect in traffic flow than a passenger car, and hence, their DVU values should be much higher than for a car.

Tanaboriboon and Roshan (1990) estimated the PCE values to measure the effect of vehicle size on highway capacity in Thailand. The primary data used in this study were the average minimum headways adopted by various types of vehicles collected in the form of time headways i.e. the time in seconds for the rear bumper of two successive vehicles in the same lane to pass a common reference point. The authors quantified the headways for PCE estimation and capacity analysis and measured the effect of vehicle size on overall performance. At the time of this study, the PCEs for Thailand had not been determined. The department of highways was using a value of 2.0 for heavy vehicles as given in the HCM (1965).

Chandra et al. (1995) proposed a methodology to derive dynamic PCU values based on the relative space requirement of a vehicle type compared to that of a passenger car as the basis of measure. They developed a mathematic model for PCU estimation as the ratio of the speed and projected area ratio of car and subject vehicle. The model was calibrated with field data and used to calculate PCU factor for a vehicle type. The variation in PCU for each category of vehicle with various parameters like composition of traffic stream, volume levels, and roadway width has been studied.

Chandra et al. (1997) compared the PCU values calculated using five different methods adopted in the earlier studies under heterogeneous traffic condition. The important methods discussed in this include; Homogenisation coefficient method, Walker's method, Headway method, Simultaneous equation method, and Multiple linear regression technique. It was found that the PCU values of different vehicle types obtained by using these under heterogeneous traffic condition showed a wide variation.

Chandra and Sikdar (2000) through an experiential study found that for a certain road width, an increase in volume level of heterogeneous traffic causes more density on the road resulting in reduced uniform speed of vehicles. The lower speed difference between cars and subject vehicles yield smaller PCU value for the vehicle type. In a heterogeneous traffic flow, different types of vehicles move in a same roadway space without any physical segregation where the amount of interaction among the vehicles varies with the mix characteristics. The interaction is maximum during a peak period on urban roads. In this situation the common practice to analyze the mixed traffic flow is to convert all vehicles into equivalent numbers of passenger car units (PCUs). In this paper a mathematical equation has been suggested relating PCU for a vehicle. The variation of PCU with traffic and geometric condition is explained graphically in this research.

Chandra and Kumar (2003)studied the effect of lane width on PCU values and hence upon the capacity of a two-lane road under mixed traffic conditions when vehicles move alongside each other instead of following each other. It is a new concept to estimate the passenger car unit (PCU) of different types of vehicles under mixed traffic conditions utilizing the area, as opposed to only the length, and speed of a vehicle. Data's were collected from different parts of India. They choose 10 sections of 2-lane roads having a carriageway width ranged from 5.5 - 8.8 m. All vehicles of the study area were divided into 9 different categories and their PCU's were estimated at each road section. It was found that the PCU for a vehicle type increases linearly with the width of carriageway. Reason is that, wider roads lead to the greater freedom of movement which results in a greater speed differential between a car and a vehicle type and the PCU is increased.

Rongviriyapanich and Suppattrakul (2005) studied the effect of motorcycles on traffic operations at signalized intersections and mid-block sections of an urban road. The data relevant to discharge headway and queue length at intersections and time headway of vehicles on mid-block sections were collected through video recording. The PCE values of motorcycle were estimated for various combinations of passenger car and motorcycle positions on the road space. The PCE of motorcycle obtained from this study was in the range of 0.3 to 0.7. It was found that the PCE of motorcycle increases when flow rate increases from 10-15 to 15-20 PCU/minute. However beyond that level of flow rate PCE of motorcycle decreased with the flow rate.

Tiwari et al. (2007) in their study have highlighted on the non-homogenous traffic condition of India which includes significant percentages of motorized, three-wheelers, motorized two-wheelers, and non-motorized traffic entities. They have developed methods to verify the Highway Capacity Manual (HCM) 2000 density method to obtain passenger car equivalencies (PCEs) or units (PCUs) for heavy vehicles and recreational vehicles. In their study modification of the density method has been needed so that more accurate passenger car unit (PCU) values can be derived for accurate capacity, safety, and operational analysis of highways carrying non-homogeneous traffic.

Aggarwal (2008) in his study has identified a number of factors affecting PCU value and focused on developing a fuzzy MATLAB based model for the estimation of PCU. The results were compared thereafter with the quoted PCU values of bus by different researches and high degree of correlation was found to be existed between the modeled and the quoted results.

Arasan and Krishnamurthy (2008) used simulation technique for four-lane divided road to quantify the impedance caused to traffic flow by the different categories of vehicles in heterogeneous traffic in terms of PCU, over a wide roadway and traffic conditions using simulation technique and also to study the effect of road width and traffic volume on PCU values of vehicles. From the results obtained it was found that the PCU value of a vehicle significantly changes with change in traffic volume. At low volume levels, the PCU value of vehicles increases with increase in traffic volume, whereas under higher volume conditions the PCU values decrease with increase in traffic volume.

Praveen (2008) developed Artificial Neural Network Model for Passenger Car Unit. He considered the effect of various factors on PCU such as pavement width, shoulder condition, directional split and slowing moving traffic. He mainly aimed at developing Artificial Neural Network (ANN) based model for the estimation of PCU values for bus. Finally he compared the results obtained by Artificial Neural Network (ANN) model with the quoted results and found that a high degree of correlation exists.

Arasan and Arkatkar (2010) conducted a micro-simulation Study to find the Effect of Volume and Road Width on PCU of Vehicles under Heterogeneous Traffic condition. The traffic volume and roadway capacity which are the important basic inputs required for planning, analysis, and operation of roadway systems cannot be expressed without converting different types of vehicles into equivalent passenger in terms of passenger car unit (PCU). This study is concerned with the estimation of PCU values of vehicles in different traffic conditions, using a microscopic simulation model named HETEROSIM. The PCU values obtained for different types of vehicles, for a wide range of traffic volume and roadway conditions, indicate that the PCU value of a vehicle significantly changes with change in traffic volume and width of roadway. The study showed that at low volume levels the PCU value of vehicles, larger than passenger cars decreases with increase in traffic volume but increases with increase in traffic volume and at high volume levels whereas, in the case of vehicles, smaller than passenger cars, the trend of variation of the PCU is totally opposite.

Cao and Sano et al. (2012) presented an investigation of the accurate methodology of motorcycle equivalent units (MEUs) in mixed traffic flow by considering the characteristics of moving vehicles such as velocity, physical size of the subject vehicle and the surrounding motorcycles in the proposed methodology. The field data were collected in Hanoi, Vietnam. The proposed equation is shown below,

$$MEU_k = (V_{mc}/V_k)*(S_k/S_{mc}) \quad (2.6)$$

Where MEU_k= MEU of type k vehicle, V_{mc},V_k= mean speed of motorcycles and type k vehicle respectively (m/s) and S_{mc},S_k= mean effective space for motorcycles and type k vehicle respectively (m^2).

The results indicated that the MEU values of cars, buses, minibuses and bicycles are 3.4, 10.5, 8.3 and 1.4 respectively.

Mehar et al. (2013) estimated PCU values for vehicle types on interurban multi-lane highways at different LOS and traffic composition. Dynamic PCU method was used to estimate PCUs. Field data for their study were collected on two sections of interurban highways; one with four-lane divided and another with six-lane divided roadway. Authors used the traffic simulation model VISSIM for generating data under controlled traffic conditions. Finally, PCU values for different vehicle types were suggested at different LOS on 4-lane and 6-lane divided highways.

Mardani et al. (2016) determined the PCU on two-lane intercity highways under heterogeneous traffic conditions. They estimated the PCU factors based on speed and size of vehicle type in traffic stream. Simultaneous equations were formulated to calculate the speeds of different vehicle types for a given traffic composition and volume. From many plots they felt that it was not enough to explain the complete variation. So a concept of a stream equivalency factor was suggested to convert a heterogeneous traffic stream into homogeneous stream.

Mishra et al. (2017) developed a novel area-occupancy based methodology for estimation of PCU values for different category of vehicles under heterogeneous traffic conditions on multi-lane urban roads for a wide range of traffic flow levels. Authors found that the PCU values suggested by IRC and dynamic PCU concept using speed-area ratio, underestimates and overestimates the flows, respectively at different traffic volumes. However, the values obtained using area occupancy concept is found to be consistent with the traffic flow in cars-only traffic situation at different flow conditions.

Biswas et al. (2017) proposed a methodology based on the kriging approach, which is a good alternative to conventional regression techniques, to arrive at a realistic prediction of speed. Authors also proposed a novel algorithm for selecting the optimal correlation function. The vehicular speeds obtained using the proposed approach based on the kriging surrogate are found to be in good agreement with the observed field data. Authors concluded that traffic volume and traffic composition have a significant influence on PCU of different vehicle types. For the same traffic stream composition, PCU of small-sized vehicles decreases with increases in traffic volume, whereas, the PCU of larger-sized vehicles increases with increases in traffic volume on the road.

2.5 Studies based on Lane change and Overtaking behavior and Lateral placement of vehicle

Worrall et al. (1969) defined lane-changing on multilane freeway sections as a random process. Lane-changing maneuvers were considered as an isolated, independent event within the traffic stream and were estimated using Poisson process. A finite Markov process was also used to model lane changing patterns. The average number of lane changes was found dependent on traffic speed and traffic volume, along with proximity to entrance and exit ramps. The randomness of lane changes was lost for medium heavy flow periods.

Sparmann (1979) studied the difference between passing and overtaking operations on multi-lane one-way carriageways. Lane changing frequency reduced at traffic volumes higher than 2000 vph. A very slight relation between lane-changing frequency and the proportion of trucks was observed, but no relation was found with speeds. The frequency decreased with increased percentage of trucks.

Huang (2002) studied lane-changing behavior on multilane highways within a model system using cellular automata approach. The study analyzed the effect of speed limit and stochastic noise on lane-changing. Lane-changing was done on the basis of headways on current and target lanes. The study also considered aggressive vehicles, which opted for frequent lane-changes whenever the incentive and safety criteria were fulfilled. In the absence of stochastic noise, frequent lane changes were reported even in the absence of slow moving traffic.

Yang et al. (2004) presented lane changing behavior for normal lane change (NLC). The lane changing process was divided into four phases: turning-angle phase, approaching phase, counterturning-angle phase, and adjustment phase. The reliability of the model was tested by comparing it with lane changing operations observed on roads of Xi'an, China. The analysis also suggested improvements in exiting two-phase and three-phase lane change models.

Laval (2007) studied the effects of overtaking ban on trucks for uphill bottlenecks on multilane freeways. Two categories of trucks, light and heavy, were used for modeling, and each truck was assumed to cause disturbance to traffic as per Newell's kinematic wave theory of moving bottlenecks. The results showed that overtaking ban increased bottleneck capacity and reduced delays for whole traffic including trucks. The findings of the study were in good agreement with the empirical data.

Kesting et al. (2007) developed a lane change model where a lane change is valued according to the acceleration of the lane changer, but also of the surrounding vehicles. A lane change maneuver can influence the acceleration of the old and the new leader. In the model, the utilities of the others are considered as well before performing a lane change, but generally valued less than one's own benefit.

Moridpour et al. (2010) developed a lane change model to estimate the lane changing behaviour of heavy vehicle drivers. The lane changing manoeuvre was characterized as a sequence of two stages: the decision to change lanes and the execution of the lane change. Hence, separate models were considered for those two stages of the heavy vehicle drivers' lane changing behaviour in their study. The estimated number of heavy vehicle lane changing manoeuvres by the fuzzy logic model and the VISSIM default model were compared to the observed lane changing manoeuvres of heavy vehicles in the field data. The number of heavy vehicle lane changing manoeuvres estimated by the fuzzy logic model was found to be more accurate than the estimates from default lane changing model in VISSIM, even when the latter model was recalibrated to reflect conditions in the test dataset.

Knoop et al. (2012) determined the number of LCs by splash over effects in loop detector counts. They collected the data from a three-lane freeway in the United Kingdom where the loop detectors are densely placed, obtained the relationship between the LC rates and roadway density, and determined the ramp's impact on LC. Collection of data from loop detectors is significantly easier than the data collection from video images. However, large errors in this type of data are inevitable because the LC is indirectly determined. Knoop et al. estimated that the error of their method is approximately 10%.

Hill et al., (2015) studied the lane changing behavior of drivers in instrumented vehicles driving on I-4 Freeway in Orlando, Florida, and I-95 Freeway in Jacksonville, Florida. The time for a lane changing maneuver, desired speed, front gap (in the target lane) and rear gap (in the target lane) were recorded for 321 discretionary lane changes. They found that the Gamma distribution provided the best fit for the rear gap. However, the Johnson SI distribution provided the best fit for the front gap.

Gowri and shravani (2017) studied the overtaking characteristics of vehicles on undivided roads under mixed traffic conditions. For this purpose, details of overtaking data were collected on a two-lane two-way undivided road using moving car observer method and registration plate method. Authors found negative correlation between the speed differential and total overtaking time for all categories of vehicles and the number of overtaking increases with increase in the flow rate in the on-going direction and decreases with increase in flow in the opposite direction.

- **Studies on Lateral placement of vehicles**

Nagaraj et al. (1990) studied the lateral placement of mixed traffic on a two-lane two-way road section of 8 m width and shoulders of 0.5 m on each side. The concepts of space coefficient and influence area were used to develop speed-flow-density relationships. The average lateral placement was observed to be a function of stream speed.

Reddy and Pandey (1995) studied lateral placement of commercial vehicles on single-lane and two-lane two way highways in Eastern India. The study suggested that for estimating the pavement thickness of single lane highways, all the commercial vehicles should be considered for computing standard axle loads as more than 95% of wheel path lies within 0.5m on either side of the maximum used path. In case of two-lane highways, about 50% of the inner wheel paths lies within lm on both sides of center line of pavement which receives maximum number of wheel load repetitions.

Dey et al. (2006a, b) found that the placement data on a two-lane road may follow a unimodal or a bimodal distribution depending on the traffic characteristics and composition. The data were observed to follow a unimodal distribution if the placement factor (PF) and skewness range (SR) were less than 1.3 and 0.54 respectively; otherwise the data followed a bimodal distribution.

Mahapatra and Maurya (2015) studied the impact of different lane positions, i.e. the Median Lane (ML) and Shoulder Lane (SL) has been analyzed for 4 lanes, 6 lanes and 8 lanes divided highways in India on average travel speed. They found that the variation in lateral gap for all mixed vehicle type of ML is more or less constant for all the three types of highways. The variation in lateral gap in SL shows an increasing trend for the six lane and 8 lane highway. Whereas it shows a decreasing trend in case of 4 lane highway. For 6 lane highway, the variation in lateral gap is more or less constant for ML and SL. The decreasing trend of car for ML of 6 lane highway is may be due to the presence of crash barrier at the median side. Hence vehicles feel safe to drive closer to the barrier.

2.6 Summary of literature review

From the review of literature regarding speed-volume relationship, it may be concluded that the selection of location for data collection is a very important step in developing speed-flow relationships, since the speed range is a function of location and the prevailing roadway and traffic conditions at that time. Further, it might be very difficult to obtain the empirical data covering all the parts of speed-flow fundamental curve from one single location. Regarding, the function, which can be used for depicting the speed-volume relationship correctly, there is no common consensus among the researchers. Based on the data points collected from field, in general, researchers all over the world have used linear, parabolic, and second degree polynomial functions, to depict speed-volume relationships.

From the review of literature regarding capacity estimation, it may be concluded that there is substantial variation in the capacity values estimated by various researchers, by virtue of the variations in the roadway and traffic conditions and the uncertainties associated with the traffic and its characteristics. In the past, researchers have used both empirical (observations in the field) as well as semi-empirical (micro-simulation) approaches to determine the capacity values. The capacity standards adopted in developed countries are not suitable for the heterogeneous traffic conditions prevalent in developing countries like India. Hence, there is a need to derive capacity guidelines for heterogeneous traffic conditions prevailing in India, for a wide range of roadway and traffic conditions using an appropriate technique. Microscopic simulation technique has been used by various researchers, extensively and effectively, due to its ability to carry out traffic flow related analysis over wide range of roadway and traffic conditions.

The review on traffic flow simulation models reveals that, the traffic flow simulation models and its application into modeling traffic flow operations have been tremendous from last few decades. The choice of simulation models greatly depends upon the area of application and logics or algorithm used for simulation. The microscopic simulation models are now popular and very effective for resolving many practical problems. Based on the exhaustive literature review conducted in this study, it is evident that microscopic simulation tool VISSIM possesses batter capabilities and features for modeling heterogeneous traffic flow operations than other commercially available simulation packages. Large number of parameters given in VISSIM provides flexibility in modeling complex operational networks. Various methods suggested in literature for calibration and validation of model, are sophisticated and follow different complex algorithms for automatic optimization of parameters. Hence, the VISSIM has been deployed for modeling multilane highway capacity and traffic flow behavior under mixed traffic conditions in this study.

From the review of literature regarding PCU estimation, it is understood that most of the studies have been conducted under more or less homogeneous traffic conditions. There are some studies on PCU under highly heterogeneous conditions and are not sufficient to provide exactly the value to be used for a particular vehicle type. It is also clear that the PCUs varies with the different types of facilities and with vehicle population. Out of various methods of PCU estimation, simulation models are considered very reliable. Literature concludes that microscopic simulation models are able to provide more precise results under mixed traffic conditions. The microscopic simulation model VISSIM has vast applications in PCU estimation and has greater flexibility to simulate traffic with the large number of input parameters.

From the review of literature regarding overtaking and lane-changing behaviour shows a good amount of work done mainly on overtaking and lane-changing operations on multilane freeways for homogenous traffic with disciplined traffic behaviour. However, very few studies are reported on understanding of overtaking operations on multilane highways especially for mixed traffic conditions with undisciplined traffic conditions.

Chapter 3

STUDY METHODOLOGY

3.1 General

This chapter deals with the methodology adopted for the study.

3.2 Methodology Flow Chart

Present study was performed in different stages and the flow chart is as shown in Figure 3.1.

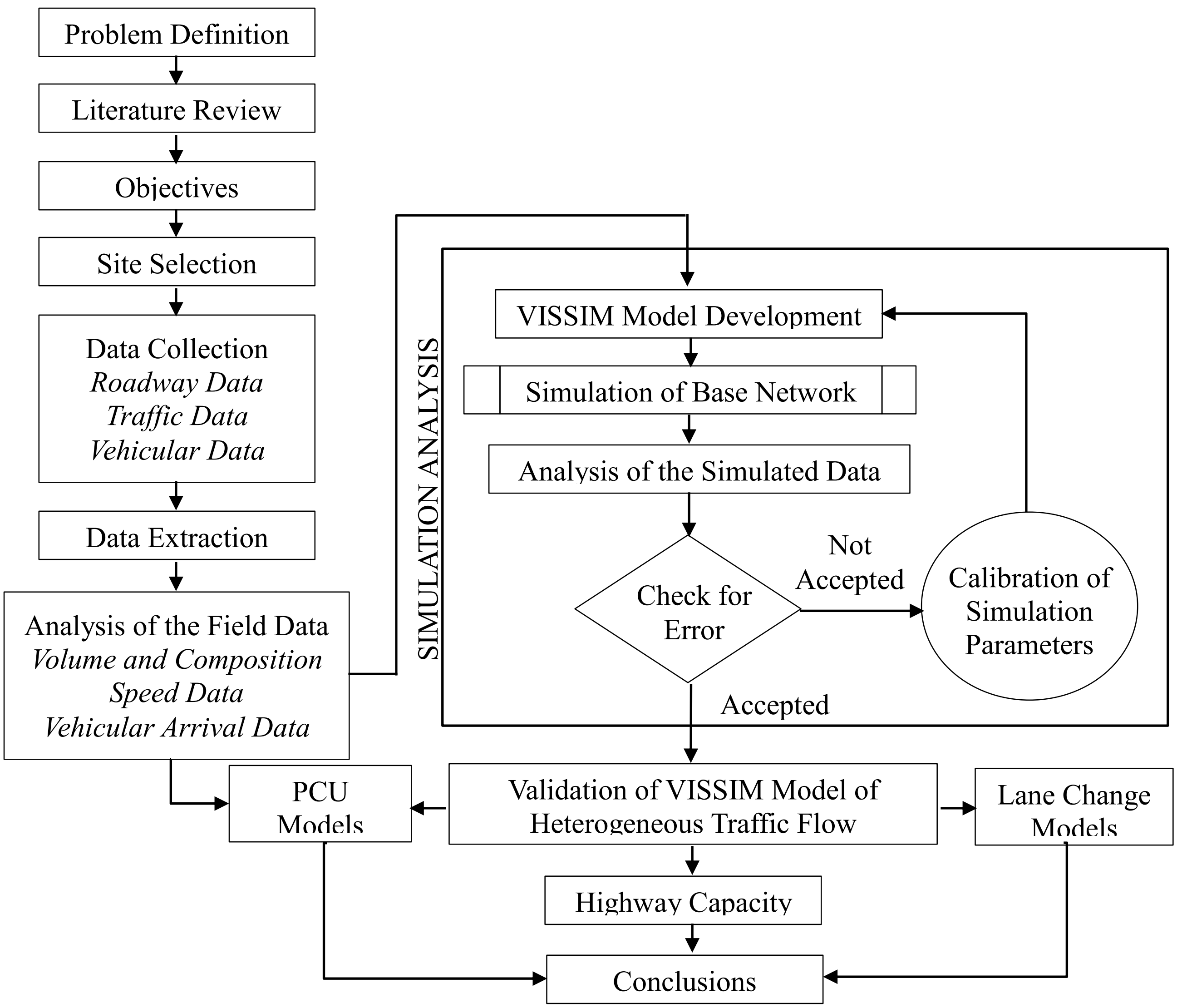


Figure 3.1 Proposed flow chart of study methodology

3.3 Methodology in detail

Various steps in the methodology are described in detail:

3.3.1 Site selection

It includes mid-block sections of intercity multi-lane divided highways with four-lanes, six and eight lanes. The sections also include different types of shoulders such as bituminous paved shoulder and earthen shoulder.

3.3.2 Data collection

Data collection is performed using video graphic survey. The camera is mounted on the specific selected location so that it can cover stretch length which is considered as an effective length for the purpose of analysis.

3.3.3 Data extraction

The data extraction is done using MPC-HC Player and classified in categories to understand the problem easily; data is extracted in following four types

- Vehicular characteristics
- Speed of the vehicles
- Vehicle inter-arrival data
- Traffic volume and composition of the vehicles

3.3.3.1 Vehicular characteristics

Vehicular characteristics include the categories of vehicles and dimension of the different vehicles. Dimension of vehicles refers to the width, length and height of vehicles which is more of less affecting effective parameters like speed, density and flow.

3.3.3.2 Speed data

In mixed traffic condition, different vehicles are moving with different speed. Speed is extracted by taking start time and end time of vehicle in the particular stretch.

3.3.3.3 Vehicle inter-arrival data

Vehicle Inter-arrival data is nothing but the arrival time of the vehicles on the start line of the stretch. This data will be helpful to calculate the Time Headway for the stream.

3.3.3.4 Traffic volume and composition

Number of vehicles flowing in particular direction for a given interval of time is the traffic volume for the given stretch, and the percentage of different categories of vehicles moving on the given section is the traffic volume composition.

3.3.4 Field data analysis

Field analysis is the analysis of the data obtained from the field; the data may be speed data, traffic volume data and Time headway data. These data used for estimation of PCU using available methods, development of PCU models and simulation analysis.

3.3.5 Simulation Analysis

Base network is developed by using field data and simulate the base network to get the required data. Comparison made between field and simulated data and estimate error. If error is within permissible limits, model can be used for further analysis otherwise calibration required.

3.3.5.1 Calibration Methodology

Particular flow chart is showing calibration methodology for calibration as well as for validation of the model parameters. Microscopic traffic simulation software VISSIM will be used for the simulation work. To develop microscopic simulation model initial steps will be taken for input data which is extracted from the field studies. The input data will be including Speed distribution of each vehicle type, observed volume, number of lanes and lane width, vehicular composition, dimension of each type of vehicles. The driver behavior such as car-following, lane-change and overtaking will also be considered during calibration process. The calibration will be done on the basis of field data and analysis to replicate the mixed traffic condition.

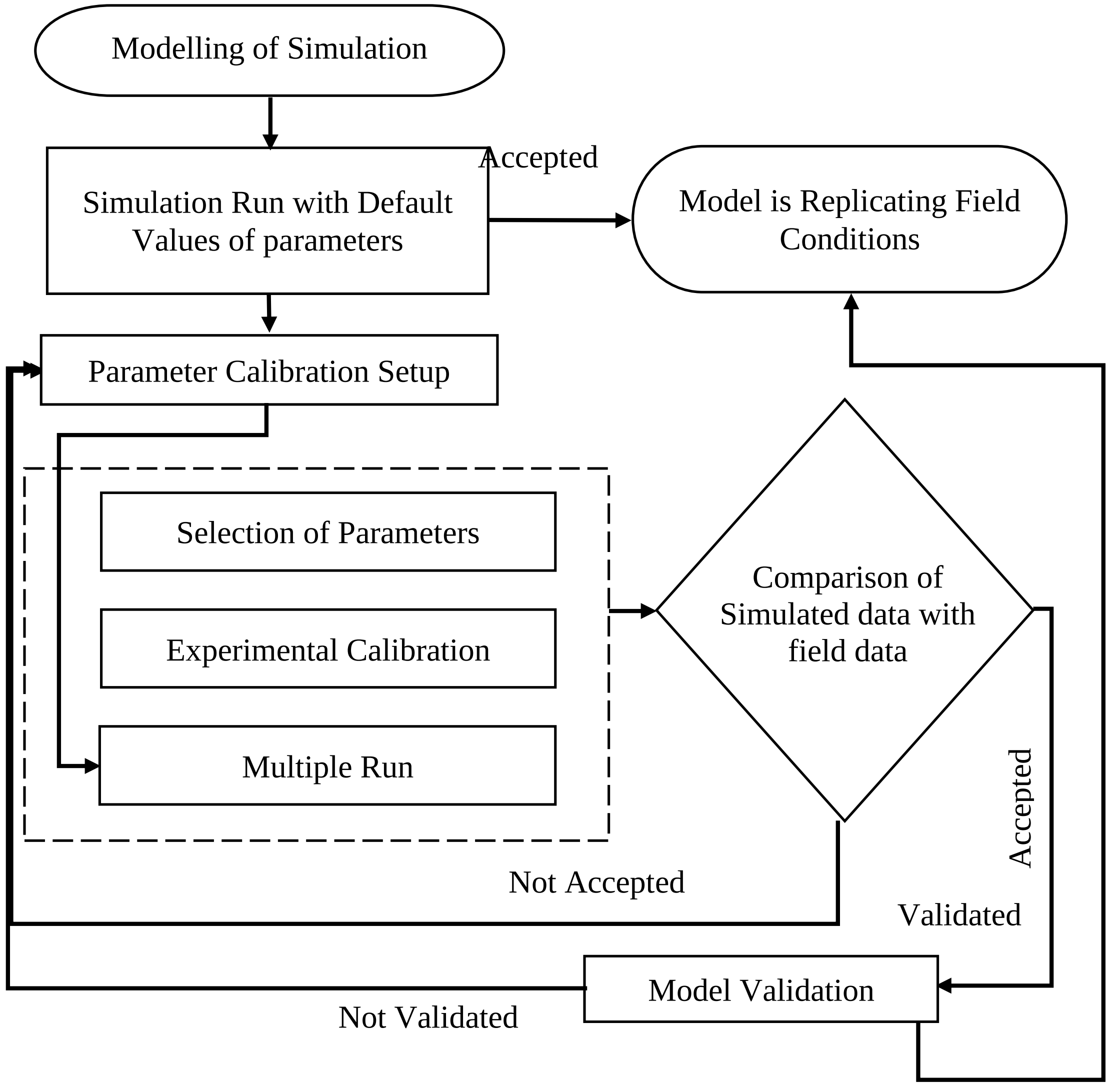


Figure 3.2 Calibration Methodology

3.3.5.2 Validated VISSIM model

The simulated data obtained from validated VISSM model used for to develop the PCU models, lane change models and the estimation of capacity for multilane highways under mixed traffic conditions.

3.4 Summary

In this chapter methodology is presented in the form of a flowchart. Effect of various parameters like headway, speed, composition on PCUs and calibration methodology of VISSIM are done as per the procedure mentioned in this chapter. Succeeding chapter deals with the site selection and data collection methodology.

Chapter 4

FIELD SURVEYS AND TRAFFIC DATA COLLECTION

4.1 General

Traffic flow data were collected at different locations of multilane divided interurban highways in India. The sites for collection of field data were selected based on the study needs and requirement of type of highway having typical mixed traffic flow. Field data were collected with respect to both microscopic and macroscopic traffic flow behavior. The data related to vehicle arrival, time headway, speed, volume and placement of vehicles were collected by using the videography method and overtaking data was collected by using floating car method. All sections are devoid of any influence of bus stop, parking, vicinity of intersections, pedestrians or other side frictions and curvature or gradients. Site selection and data collection methodology are presented in this Chapter.

4.2 Selection of study locations

Field data for study was collected at different mid-block sections of multilane divided intercity highways. Location of highways where data was collected are parts of National Highway (NH) exists on plain terrain with straight alignment, some sections are access control and some are partially access controlled in both the directions of travel. Details of the study sections have been given in the Table 4.1. Section I, Section II and Section III are located on NH 163 near Madikonda village, Bibinagar village and Ghanpur village respectively. However, these sections are differs from the type of access control. Section-I has no access control and Section-II has fully control of access whereas Section-III is partially access controlled. Section IV is NH45A (NH332, as per new numbering) connecting Chennai to Nagapatinam, near Viluppuram district, in Tamilnadu State. Section-V is a part of NH 58 located in between Delhi and Meerut city near Modinagar. Section VI is located on NH 24 connecting Delhi and Harpur cities. Section-VII and Section-XI are located on NH 16 between Guntur and Ongole cities respectively, which is a six-lane divided intercity highway having 1.8 m paved shoulders. Section-VIII is selected from NH 8 near Delhi, which is an eight-lane divided intercity highway having 1.8 m paved shoulders. The snapshot of study sections are shown in Figure 4.1 to Figure 4.3.

Table 4.1 Details of the study sections

Sections	Highway No.	Location	Type of highway	Type of Shoulder	Properties	Posted speed limit (Kmph)
I	NH 163	Near Madikonda (Telangana)	Four lane Divided	Paved	CW: 7.0 m SW: 1.5 m	80
II	NH 163	Near Bibinagar (Telangana)	Four lane Divided	Paved	CW: 7.0 SW: 1.5 m	80
III	NH 163	Near Ghanpur (Telangana)	Four lane Divided	Paved	CW: 7.0 m SW: 1.5 m	80
IV	NH 332	Near Vilupparam (Tamilnadu)	Four lane Divided	Paved	CW: 7.0 m SW: 1.5 m	80
V	NH 58	Meerut(Uttar Pradesh)	Four lane Divided	Un paved	CW: 7.0 m	80
VI	NH 24	Delhi-Hapur (Uttar Pradesh)	Four lane Divided	Unpaved	CW: 7.0 m	80
VII	NH 16	Near Guntur (Andhra Pradesh)	Six lane Divided	Paved	CW: 10.5 m SW: 1.8 m	90
VIII	NH 8	Delhi-Gurgaon(Delhi)	Eight lane Divided	Paved	CW: 14.0 m SW: 1.8 m	120
IX	NH 16	Ongole (Andhra Pradesh)	Six lane Divided	Paved	CW: 10.5 m SW: 1.8 m	90

*CW-Carriageway width, SW-Shoulder width

Figure 4.1 Section of four-lane divided highway with paved shoulders (Section-II)

Figure 4.2 Section of six-lane divided highway with paved shoulder (section-VII)

Figure 4.3 Section of eight-lane divided highway with paved shoulder (section-VIII)

4.3 Vehicle Type and their static characteristics

The traffic flow was observed to be heterogeneous in terms of physical size and operating characteristics of vehicles. Even within the same category of car, several new models are observed on Indian roads. To make the data more amenable, vehicles with similar physical dimensions and operating characteristics were grouped together and all vehicles were divided into eight categories as Standard car (CS), Big utility car (CB), Light commercial vehicles (LCV), Heavy vehicle (HV), Multi axle vehicles (MAV), Bus (B), Motorized three-wheelers (3W) and Motorized two-wheelers (TW). The standard car in the present study is defined as a vehicle having 3.6 m length and 1.5 m width, and engine power up to 1400 cc. The Big utility car is the one having a length of 4.6 m and width of 1.7 m, and the engine power up to 2500 cc. The physical sizes of the different types of vehicles are given in Table 4.2.

Table 4.2 Dimensions of vehicles

Vehicle Type	Length(m)	Width(m)	Projected Area(m^2)
Standard Car(CS)	3.60	1.50	5.40
Big utility Car (CB)	4.60	1.70	7.82
Light Commercial Vehicles(LCV)	4.30	1.56	6.71
High Vehicles(HV)	6.70	2.30	15.41
Multi Axle Vehicles(MAV)	11.50	2.42	27.83
Two-Wheeler(TW)	1.97	0.74	1.46
Three Wheeler(3W)	3.20	1.30	4.16
Bus(B)	10.60	2.40	25.44

4.4 Field data collection

4.4.1 Speed and volume data

The videography technique was used to record the movements of vehicular traffic in one direction of travel. Two thick white lines with clear visibility were marked on the pavement to make a longitudinal trap on the highway section by using a self-adhesive cloth tape. A distance of 60-100 m was selected for the trap (depending upon the posted speed limit of highway) which would eventually act as reference lines for measurement of speed. However, the traffic volume count was made based on the one of reference lines. A high definition video camera was placed on a tripod at suitable vantage point and recording was done continuously for 6.00 to 8.00 hrs on typical weekdays in clear weather conditions.

4.4.2 Vehicle arrival and time headway data

The video recording technique was used to record the traffic flow operations to obtain vehicle arrival data as well. A thick white line with clear visibility was marked across the road for providing a reference line to measure vehicle arrival frequency. The time of arrival of each vehicle from the recorded videos were extracted by playing it on a large screen monitor in Traffic Engineering Laboratory. The time of arrival of the first vehicle and each consecutive vehicle crossing the reference line was recorded as arrival time, irrespective of the travelling lanes. The time difference between the two consecutive vehicles (front bumper) was calculated to obtain the time headway. The type of vehicle was also noted for determining time headway of each vehicle type.

4.4.3 Lateral placement characteristics

The lateral position of vehicles across the whole carriageway width was also noted by videography technique. The pavement width was divided into sections of 25 cm width using self-adhesive tape and these were numbered from 0 to 28 starting from pavement edge to the highway median. The positions of vehicle types were recorded by noting down the strip number over which vehicles placed its left wheel while traversing the highway section.

4.4.4 Overtaking characteristics

Floating car method was used for collecting the data. The data were collected on typical weekdays with normal weather conditions. The process of overtaking operation was divided into 5 events and recorded the time taken for each event (Chandra and Shukla, 2012). The description of events recorded during data collection is described as under.

Event 1: When the overtaking vehicle deflects to the adjacent lane for starting the maneuver.

Event 2: When the front bumper of the overtaking vehicle is in line with the rear bumper of the test vehicle.

Event 3: When the front bumper of the overtaking vehicle is in line with the front bumper of the test vehicle.

Event 4: When the rear bumper of the overtaking vehicle is in line with the front bumper of the test vehicle.

Event 5: When the overtaking vehicle either returns back to its lane or continues to travel in the adjacent lane after overtaking the test vehicle.

A Tata Indigo car (Diesel) was used as a test vehicle for collection of data. The vehicle was driven at different constant speed on a stretch of 2 km for recording of above events and maneuvers. A team of three persons were appointed in the car to record the data. The first person was sitting on the back seat of test vehicle and recorded the type of overtaking vehicle and the speed of test vehicle during each overtaking maneuver. Figure 4.4 gives the details of the events considered for data collection during overtaking or lane-changing maneuver.

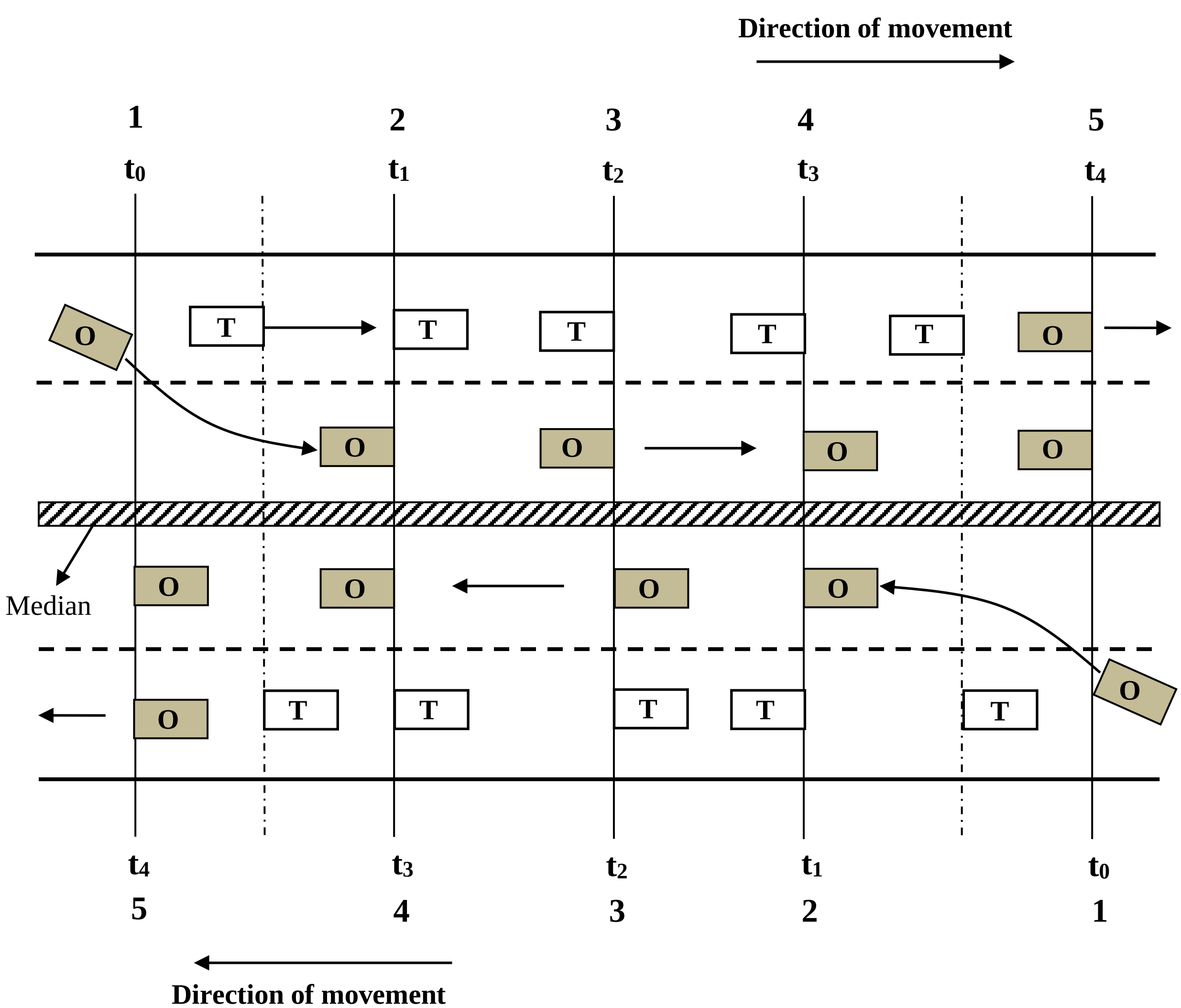


Figure 4.4 Details of events during overtaking/lane-changing operation

4.5 Summary

The details of study sections and methodology for collection of data such as speed, volume, vehicle arrival, time headway, lateral placement and overtaking characteristics of vehicles are described. The type of vehicles along with their static characteristics was also described.

Chapter 5
FIELD DATA ANALYSIS

5.1 General

The modeling of traffic flow is required detailed analysis of field data. The statistical analysis of traffic flow data also helps in in-depth understanding of traffic flow behavior and traffic system performance. In this chapter, data collected from the field has been analysed by statistical testing and probabilistic analysis to observe the behavior of the traffic flow stream. The analysis of data involves microscopic and macroscopic variables those are found to be influential on modeling traffic flow behavior.

5.2 Vehicle arrival characteristics

Knowledge of distribution of the vehicle arrival pattern or inter-arrival pattern (headways) of vehicles is very essential in order to understand the general traffic flow behavior on multilane highways. The arrival pattern of vehicles at a point (or) line on roadway defines the longitudinal distribution of vehicles in a traffic stream. The distribution of arrival time of vehicles enables the traffic engineers and planners to estimate the availability and magnitude of gaps and headways in traffic stream, which are the direct measure of the density and volume on the highway. Vehicle arrival is also used as an essential input in the simulation of traffic behaviour. However, most of the researchers have studied the arrival characteristics of vehicles through headway distributions.

5.2.1 Statistical distributions for vehicle arrival

Poisson distribution is considered to be an appropriate distribution for describing random occurrence of discrete events like arrival pattern of vehicles. However, it was observed that mean and variance are not equal in all the cases and hence Poisson does not always give a good fit for vehicle arrival pattern. In these cases, negative binomial distribution may be more suitable. In some cases, Binomial distribution also models vehicle arrivals, since Poisson distribution is a limiting form of Binomial distribution (Johnson and Kotz, 1969). A brief description of three discrete distributions is given below.

- **Binomial Distribution**

The binomial distribution is defined as

$$P(X = x) = \binom{n}{x} p^x (1 - p)^{n-x} \qquad x = 0,1,\ldots \tag{5.1}$$

Where, $P(X = x)$ = probability of x successes in n trials; p = probability of success on any given trial; q = (1-p) = probability of failure on any given trial; n = number of independent trials; x = number of successes.

The mean and variance of the binomial distribution are “np” and “npq” respectively.

- **Poisson distribution**

The Poisson distribution is defined as

$$P(X = x) = \frac{e^{-\lambda} \lambda^x}{x!} \qquad x = 0,1,\ldots \tag{5.2}$$

Where, $P(X = x)$ = probability of x successes during a time t; λ = np = mean number of successes in time t; n = number of trials; p = probability of success.

The mean and variance of Poisson distribution are equal and denoted by λ.

5.2.2 Analysis of vehicle arrival pattern

Vehicle arrival data collected at three different mid-block sections of multilane highway was extracted in the laboratory. Time interval of extraction of data was chosen as 20s. The frequency tables were prepared as per the observed number of arrivals in a time interval of 20s for more than one hour of observation. These frequency tables were used to evaluate values of mean and variance of vehicle arrival rate. Then statistical analysis of data was performed. The statistical distributions are analysed to fit the observed vehicle arrival data on highway locations. Chi-square test of goodness of fit was applied for testing the hypothesis. Table 5.1 gives the data for fitting of Poisson distribution to vehicle arrival at Section-I. The calculated value of chi-square is obtained as 9.83, which found to be less than the tabulated value of Chi-Square as 12.59 obtained (at 6 degrees of freedom) at 5% level of significance. Hence, the null hypothesis (H_0) is accepted stating that the observed arrival pattern follows Poisson distribution on Section-I. Figure 5.1 shows the histogram of vehicles arrival pattern at Section-I. Similarly, Poisson distribution was also tried with the data obtained at other sections. Table 5.2 and Table 5.3 provide the field and expected frequencies fitted to Poisson distribution for Section-V and Section-VII. Figure 5.2 and Figure 5.3 show the histogram of vehicle arrivals at Section-V and Section-VII.

Table 5.1 Fitting of Poisson distribution to arrival data at Section-I

Number of vehicles in 20 sec interval (x)	Observed frequency (O_f)	($x \times O_f$)	($(x-\mu)^2 \times O_f$)	Estimated frequency by Poisson distribution (E_f)	E_f (after Pooling)	O_f (after Pooling)	$\chi^2 = \left(\frac{(O_f-E_f)^2}{E_f}\right)$
0	3	0	52.78	3	--	--	--
1	14	14	142.86	11	14	17	0.64
2	24	48	115.57	24	24	24	0.00
3	39	117	55.64	33	33	39	1.09
4	23	92	0.87	35	35	23	4.11
5	33	165	21.41	30	30	33	0.30
6	16	96	52.16	21	21	16	1.19
7	13	91	102.32	13	13	13	0.00
8	8	64	115.86	6	10	15	2.50
9	3	27	69.28	3	--	--	--
10	3	30	101.11	1	--	--	--
11	1	11	46.32	0	--	--	--
	180	755	876.19	180			9.83

Null hypothesis H_0: Arrival pattern observed at Section-I follow Poisson distribution.

Alternative hypothesis H_1: Arrival pattern observed at Section-I does not follow Poisson distribution.

Mean rate of arrival (μ)$= \frac{\sum(x \times O_f)}{\sum O_f} =$ 4.194 sec

Variance of arrivals (σ^2) from mean $= \frac{\sum((x-\mu)^2) \times O_f}{\sum O_f - 1} =$ 4.895 sec

Degree of Freedom (υ) = 8-2 = 6

At, $\upsilon = 6$ and $\alpha = 5\%$ $\chi^2(tabulated) = 12.59$

$\chi^2(calculated) < \chi^2(tabulated)$

Hence, null hypothesis H_0 is accepted.

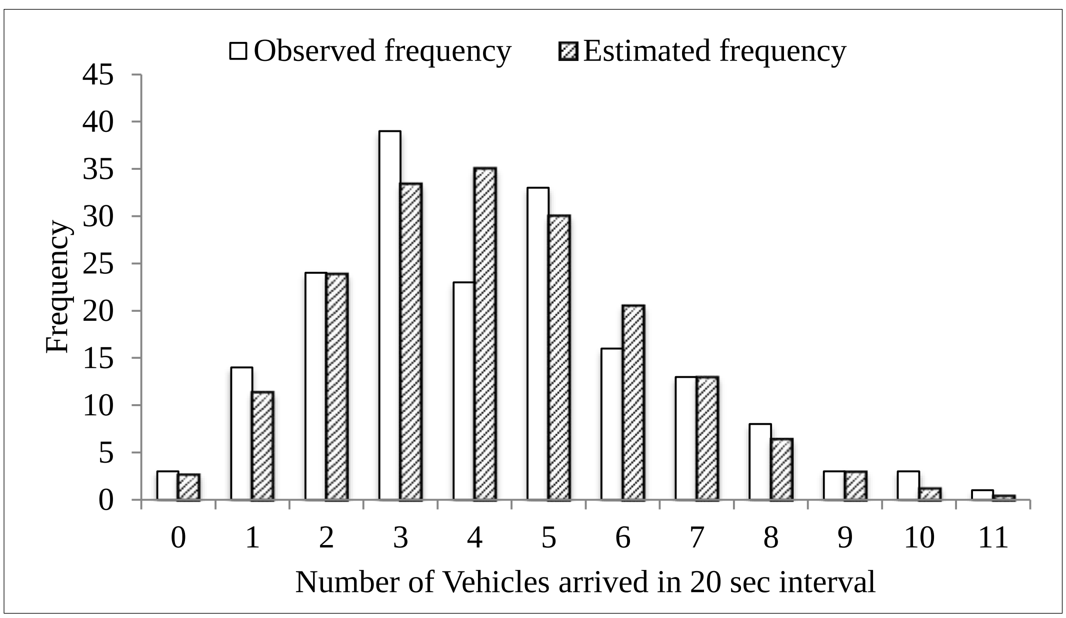


Figure 5.1 Comparison of Histograms of vehicle arrivals at Section-I

Table 5.2 Fitting of Poisson distribution to arrival data at Section-V

Number of vehicles in 20 sec interval (x)	Observed frequency (O_f)	($x \times O_f$)	($(x-\mu)^2 \times O_f$)	Estimated frequency by Poisson distribution (E_f)	E_f (after Pooling)	O_f (after Pooling)	$\chi^2=\left(\frac{(O_f-E_f)^2}{E_f}\right)$
0	0	0	0.00	0	--	--	--
1	2	2	158.42	0	--	--	--
2	1	2	62.41	0	--	--	--
3	0	0	0.00	1	--	--	--
4	4	16	139.24	4	--	--	--
5	7	35	168.07	7	12	14	0.33
6	20	120	304.20	12	12	20	5.33
7	12	84	100.92	16	16	12	1.00
8	22	176	79.42	20	20	22	0.20
9	18	162	14.58	23	23	18	1.09
10	25	250	0.25	23	23	25	0.17
11	17	187	20.57	20	20	17	0.45
12	13	156	57.33	17	17	13	0.94
13	14	182	134.54	13	13	14	0.08
14	6	84	100.86	9	9	6	1.00
15	4	60	104.04	6	6	4	0.67
16	6	96	223.26	4	9	15	4.00
17	3	51	151.23	3	--	--	--
18	1	18	65.61	1	--	--	--
19	4	76	331.24	1	--	--	--
25	1	25	228.01	0	--	--	--
	180	1782	2444.20	180			15.26

Null hypothesis H_0: The arrival pattern at Section-V follows Poisson distribution.

Alternative hypothesis H_1: The arrival pattern at Section-V does not follow Poisson distribution

Mean rate of arrival (μ)$= \frac{\sum(x \times O_f)}{\sum O_f} =$ 9.90 sec

Variance of arrivals (σ^2) from mean $= \frac{\sum((x-\mu)^2) \times O_f}{\sum O_f - 1} =$ 13.65 sec

Degree of Freedom (υ) = 12-2 = 10

At, υ = 10 and α = 5% $\quad \chi^2(tabulated)$ = 18.31

$$\chi^2(calculated) < \chi^2(tabulated)$$

Therefore, null hypothesis H_0 is accepted.

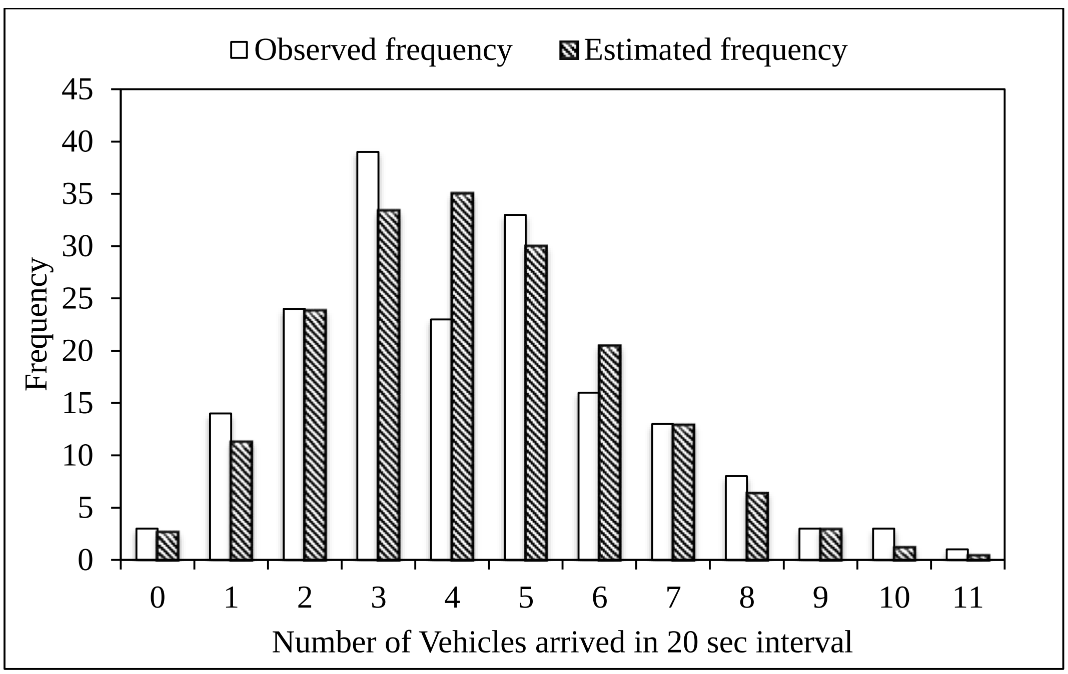


Figure 5.2 Comparison of Histograms of vehicle arrivals at section-V

Table 5.3 Fitting of Poisson distribution to vehicle arrival data at Section-VII

Number of vehicles in 20 sec interval (x)	Observed frequency (O_f)	$(x \times O_f)$	$((x-\mu)^2 \times O_f)$	Estimated frequency by Poisson distribution (E_f)	E_f (after Pooling)	O_f (after Pooling)	$\chi^2 = \left(\frac{(O_f-E_f)^2}{E_f}\right)$
0	0	0	0.00	0	--	--	--
1	1	1	42.61	1	--	--	--
2	4	8	122.23	3	--	--	--
3	5	15	102.50	7	11	10	0.09
4	23	92	286.24	14	14	23	5.79
5	19	95	121.40	19	19	19	0.00
6	15	90	35.01	24	24	15	3.38
7	21	147	5.85	26	26	21	0.96
8	29	232	6.47	25	25	29	0.64
9	19	171	41.18	21	21	19	0.19
10	16	160	97.79	15	15	16	0.07
11	7	77	84.39	11	11	7	1.45
12	10	120	200.01	7	7	10	1.29
13	8	104	239.56	4	7	11	2.29
14	2	28	83.78	2	--	--	--
15	1	15	55.83	1	--	--	--
	180	1355	1524.86	180			16.14

Null hypothesis H_0: The arrival pattern at Section-VII follows Poisson distribution.

Alternative hypothesis H_1: The arrival pattern at Section-VII does not follow Poisson distribution.

Mean rate of arrival (μ)$= \frac{\sum(x \times O_f)}{\sum O_f} =$ 7.53 sec

Variance of arrivals (σ^2) from mean $= \frac{\sum((x-\mu)^2) \times O_f}{\sum O_f - 1} =$ 8.52 sec

Degree of Freedom (υ) = 11-2 = 9

At, υ = 6 and α = 5% $\chi^2(tabulated) = 16.92$

$$\chi^2(calculated) < \chi^2(tabulated)$$

Therefore, null hypothesis H_0 is accepted.

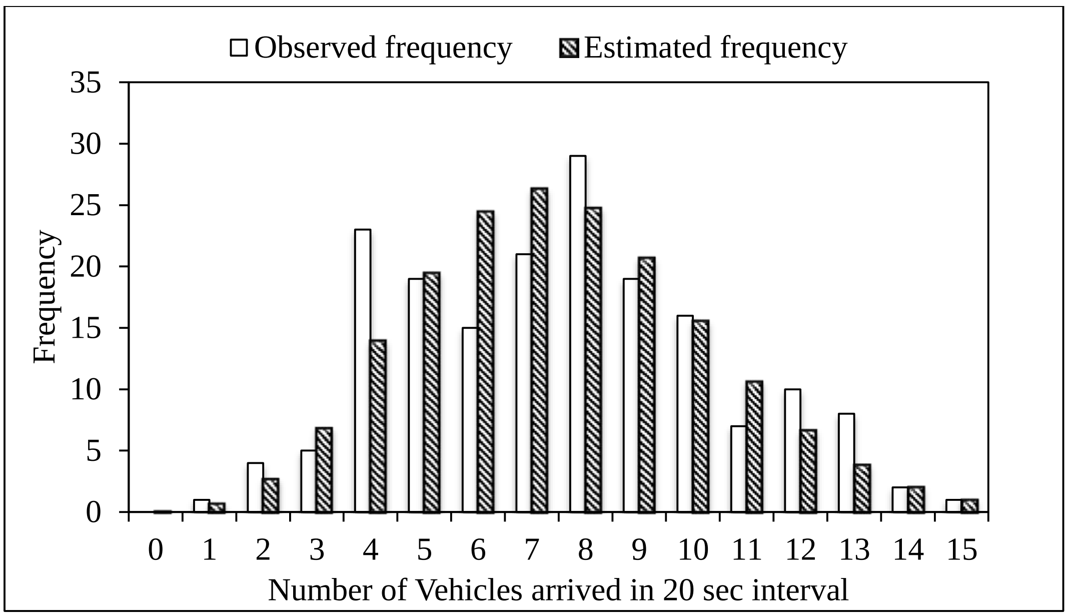


Figure 5.3 Comparison of Histograms of vehicle arrivals at section-VII

5.3 Time headway characteristics

Time headway is defined as the measurement of time between occurrences of two consecutive vehicles (front bumpers) at a specific location or point. In developed countries, the traffic flow is generally homogeneous and disciplined in nature. Vehicles move in their respective lanes depending upon the speed. Headway distribution of vehicles for such conditions may be easily recorded and analysed for each lane. However, traffic in most of the developing countries like India is heterogeneous in nature. The mix of different types of vehicles combined with lack of lane discipline makes traffic flow analysis very complex. In the case of multilane roads, vehicles do not move in their assigned lanes and small sized vehicles move abreast resulting in situations where headways between vehicles cannot be measured. Therefore, total width of the roadway in one direction is considered rather than each lane for determining the headways. The analysis of headway data carried out in present study is described in details as follows.

5.3.1 Statistical distributions for time headway

Brief descriptions of statistical distributions tried to fit the observed time headway data are given in this section.

- **Generalized Extreme Value (GEV)**

The Generalized Extreme Value (GEV) distribution is a flexible three-parameter model that combines the Gumbel, Frechet, and Weibull maximum extreme value distributions. It has the following probability density function (PDF).

$$f(x) = \begin{cases} \dfrac{1}{\sigma} \exp\left(-(1+kz)^{-1/k}\right)(1+kz)^{-1-1/k} & k \neq 0 \\ \dfrac{1}{\sigma} \exp(-z - \exp(-z)) & k = 0 \end{cases} \tag{5.3}$$

Where $z=(x-\mu)/\sigma$, and k, σ, μ are the shape, scale, and location parameters respectively. The scale must be positive, the shape and location can take on any real value.

A shape parameter, as the name suggests, affects the general shape of a distribution. The Location parameter tells about where the distribution is centered on the horizontal axis. The Scale parameter gives an idea of the scale on the horizontal axis.

- **Weibull distribution**

The Weibull distribution is one of the most commonly used distributions in reliability engineering. There are two versions of this distribution: two-parameter Weibull and three-parameter Weibull distributions. It has the following probability density function (PDF).

$$f(x) = \frac{\beta}{\alpha}\left(\frac{\beta}{x-\gamma}\right)_{\alpha-1} exp\left(-\left(\frac{\beta}{x-\gamma}\right)\right)_{\alpha} \tag{5.4}$$

Where, α,β and γ are the shape, scale, and location parameters respectively. γ=0 yields the two-parameter Weibull distribution. The scale, shape must be positive, and location can take on any real value.

- **Pearson (6) Distribution**

There are two versions of this distribution: two-parameter Pearson 6 and three-parameter Pearson 6 distributions. It has the following probability density function.

$$f(x) = \frac{((x-\gamma)/\beta)^{\alpha_1 - 1}}{\beta\, B(\alpha_1,\alpha_2)\, (1 + (x-\gamma)/\beta)^{\alpha_1+\alpha_2}} \tag{5.5}$$

Where, α_1,α_2 are shape parameters and β,γ are the scale, and location parameters respectively. γ=0 yields the two-parameter Pearson 6 distribution. The scale, shape parameters must be positive, and location can take on any real value.

- **Gamma distribution**

There are two versions of this distribution: two-parameter Gamma and three-parameter Gamma distributions. It has the following probability density function.

$$f(x) = \frac{(x-\gamma)^{\alpha-1}}{\beta^{\alpha}\,\Gamma(\alpha)} \exp(-(x-\gamma)/\beta) \tag{5.6}$$

Where, α,β and γ are the shape, scale, and location parameters respectively. γ=0 yields the two-parameter Gamma distribution. The scale, shape must be positive, and location can take on any real value.

5.3.2 Statistical analysis of time headway

The time headway data of each vehicle observed in recorded videos were extracted in 20 sec. interval. The descriptive analysis was performed with extracted data to understand its basic characteristics. The parameters those describe the basic characteristics such as mean and variance of data are given in the Table 5.4 for Section-I, Section-V and Section-VII respectively.

Table 5.4 Descriptive parameters of time headway data

	Section-I	Section-V	Section-VII
Mean (sec)	4.48	2.41	2.98
Median (sec)	3.71	1.71	2.21
Standard deviation (sec)	3.31	2.09	2.49
Sample size (N)	590	1400	1100

It is known that the time headway of vehicles is affected by the traffic volume observed on highway section. It has also been observed that the mean time headways, median values and standard deviation found to be decreased with increase in traffic volume ranges. The decreasing trend clearly indicates that the proportion of free-flowing vehicles is lesser in high volume a range which is resulted in smaller time headways. However, in all cases the median values of time headways are found to be smaller than the mean, infers more than 50% of drivers chose time headways lesser than their mean values.

Table 5.5 Average time headway (sec) of vehicle types

Vehicle Type	Section-I	Section-V	Section-VII
CS	3.96	1.83	2.69
CB	4.04	1.73	2.72
LCV	4.10	1.83	2.78
HV	4.16	2.17	2.76
MAV	4.66	3.63	2.77
TW	4.48	2.07	2.68
3W	4.43	1.64	2.76
B	4.58	2.64	2.72

In order to fit different probability distribution functions to the time headway data, 5% of long time headways may be neglected and statistical results for different flow levels will be evaluated by considering 95% time headway values. In the present study, goodness of fit for each probability density function is tested by performing Kolmogorov-Smirnov (K-S) test at 5% significance level. The results of time headway distributions analysis for different study sections based on K-S test are given in Table 5.6.

Table 5.6 Estimated parameters of the best fitted distributions for Time headway data at different study sections

Sections	Best fit	Parameters	K-S Test Value	K-S Test Critical Value
I	Pearson6	α_1=1.53 α_2=89.8 β=26158.0	0.04166	0.05581
	Gamma	α=1.46 β=3.057	0.04429	
	Weibull	α=1.1238 β=5.0426	0.04470	
V	GEV	k=0.258 σ=1.096 μ=1.402	0.03587	0.03618
VII	GEV	k=0.159 σ=1.59 μ=1.76	0.03495	0.04096

From Table 5.6, it is observed that Pearson 6 is found to be the best fit for headway data on Section-I whereas, Generalised Extreme Value (GEV) distribution is fitted best to the time headway data observed on Section-V and Section-VII. In addition, Gamma and Weibull distributions are also found suitable to fit time headway distribution at Section-I. Figure 5.4 to 5.6, provides the PDF profile of fitted distribution to respective highway sections.

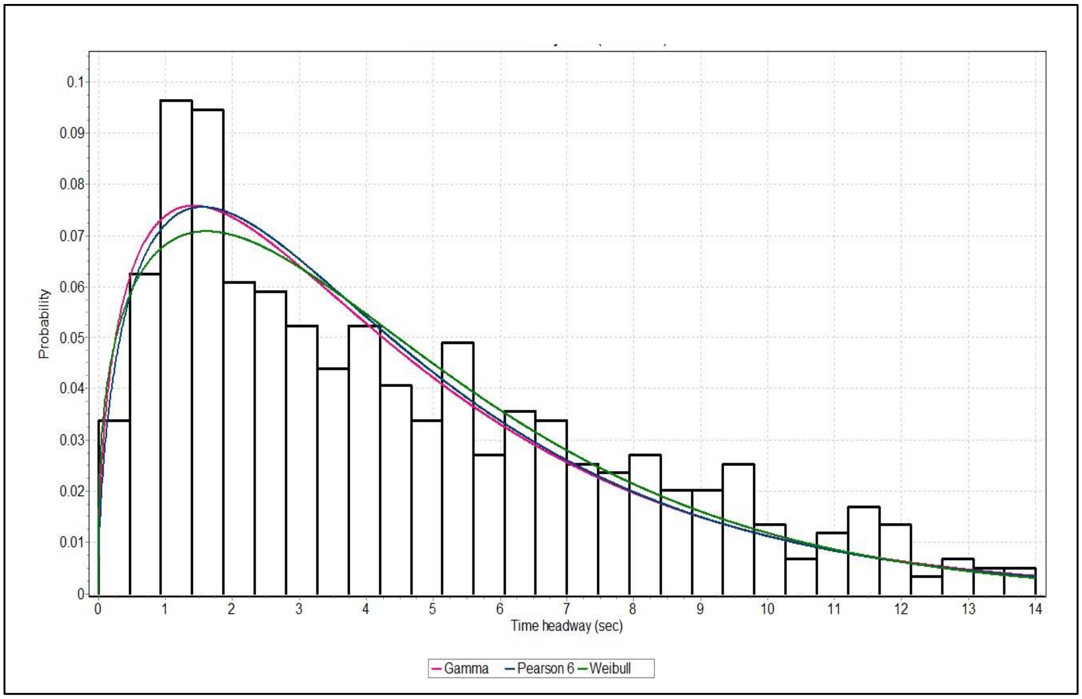


Figure 5.4 Time headway distribution profile at Section-I

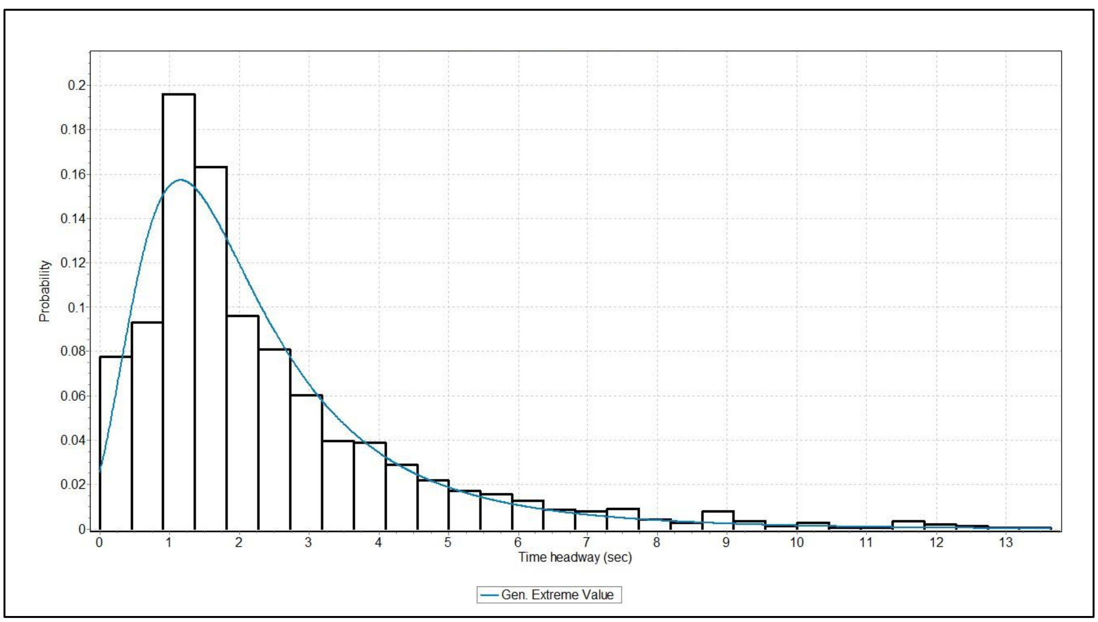


Figure 5.5 Time headway distribution profile at Section-V

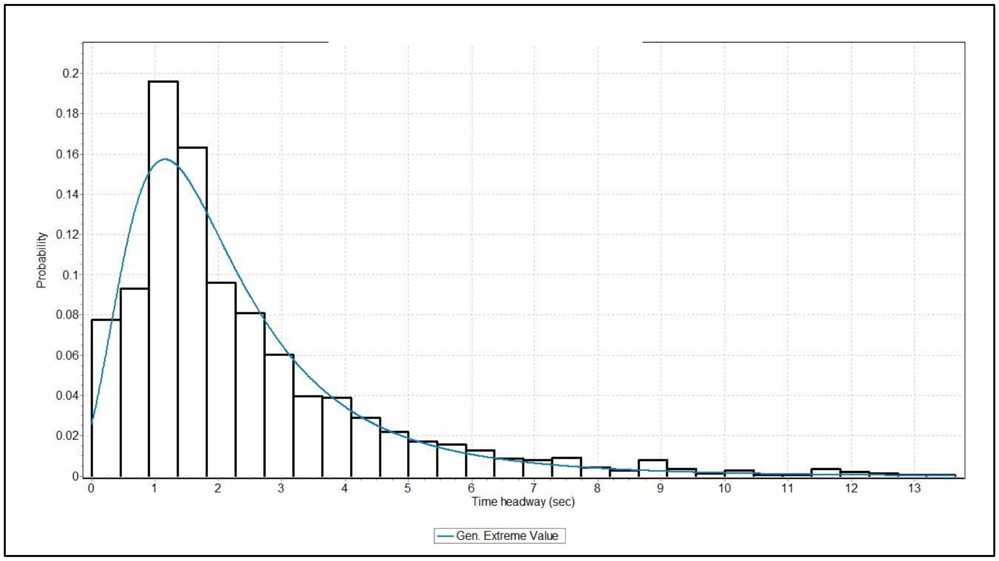


Figure 5.6 Time headway distribution profile at Section-VII

5.4 Speed characteristics

An understanding of vehicular speed characteristics is an important requirement in the field of traffic engineering. IRC: 64 (1990) defines speed as the rate of motion of individual vehicles or of a traffic stream measured in meters per second (m/s), or more generally in kilometres per hour (km/hr). Speed indicates the quality of service experienced by the traffic stream and used as performance criteria to evaluate traffic flow system. The knowledge of speed is an essential component of traffic engineering projects related to geometric design of roads, regulation and control of traffic operations, accident analysis, before and after studies of road improvement schemes, assessing journey times, and congestion on roads and in correlating capacity with speeds (Kadiyali, 2002). It is one of the components of the fundamental relationships of traffic flow theory other than density and volume. The speed characteristics of a traffic facility serve as an essential input in simulating the traffic behavior.

5.4.1 Speed data analysis

Speed data was collected using video recording method at 8 different roadway sections on multi-lane divided highways in India. Two thick white lines were marked across the road width to act as reference lines for measuring the speed of vehicle types. Speed data was extracted from the recorded videos and played on a wide screen computer system. Speed of individual type of vehicles was extracted by noting down the time taken by a vehicle to cross a longitudinal trap length using electronic stop watch with 0.001s accuracy. Speed related parameters such as 15th, 50th and 85th percentile speed were estimated and analysed by developing box plots. Figure 5.7 presents the box plot developed using speed percentiles estimated at each section of selected highways.

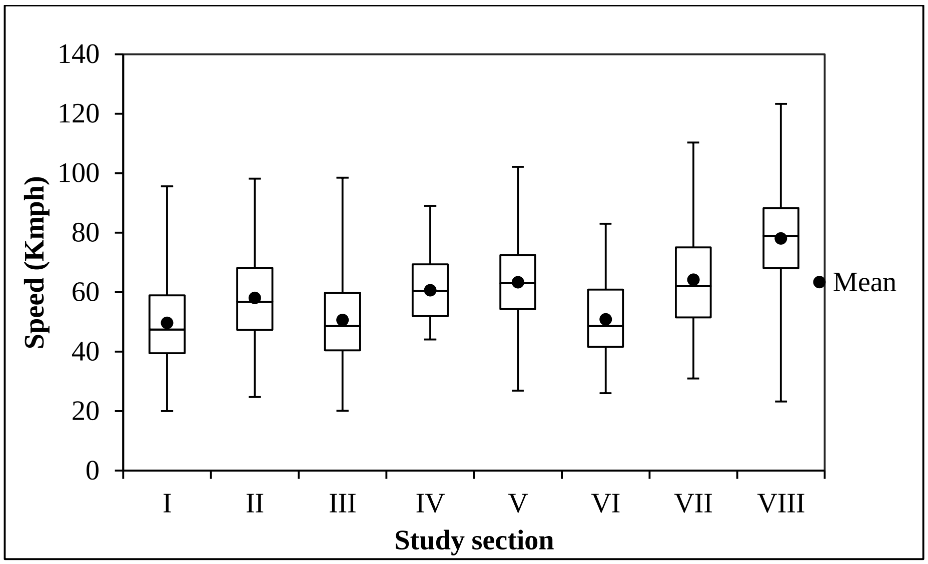


Figure 5.7 Box plot of speed percentile of vehicles at study sections

The percentage speeds were estimated by considering all the vehicles observed in mixed traffic stream. At Section-I, Section-II, Section-III, Section-VI and Section-VII, mean speed is found to be greater than median speed which reflects the shape of the speed data profile is positively skewed. In Section-IV and Section-V, mean speed is almost equal to median speed which depicts that the speed data profile is symmetric. At Section-VIII the shape of the data is slightly left skewed because the speed of the vehicles are relatively higher which may be inferred from the estimated mean speed which is observed to be smaller than the median speed. Speed raw data of each vehicle type was used to find mean and other related parameters such as 15th, 50th and 85th percentile speed of each vehicle type at 8 study locations and are given in Table 5.7.

Table 5.7 Speed parameters for vehicle types at different sections

Sections	Vehicle type	Mean speed (km/h)	Maximum speed (km/h)	Minimum speed (km/h)	Standard Deviation (km/h)	V_{15} (km/h)	V_{50} (km/h)	V_{85} (km/h)
I	CS	64.50	95.54	28.88	12.12	52.44	65.06	77.03
	CB	67.00	92.07	34.97	11.73	55.59	67.35	78.46
	TW	45.10	82.72	21.27	10.70	34.29	44.36	55.65
	3W	40.80	56.41	20.00	7.11	33.95	41.47	48.11
	LCV	47.56	83.06	21.63	12.88	35.00	46.12	61.12
	HV	42.10	62.39	21.47	9.90	30.18	42.49	51.94
	MAV	39.10	63.22	25.57	8.26	29.85	38.72	48.48
	B	45.20	64.15	22.81	8.08	37.93	44.61	52.30
II	CS	66.59	91.42	36.20	10.25	57.99	68.29	79.33
	CB	69.78	98.09	41.82	10.20	59.27	69.15	80.47
	TW	50.02	84.77	29.81	9.80	41.49	49.96	61.31
	3W	39.49	71.05	24.75	6.59	36.45	42.01	47.73
	LCV	49.84	85.63	31.00	11.05	40.12	49.84	61.94
	HV	46.64	70.75	30.65	8.44	37.78	45.91	57.00
	B	50.47	82.99	38.81	7.75	46.36	51.63	59.49
III	CS	65.30	90.00	30.28	13.40	48.56	64.29	78.94
	CB	66.00	94.74	39.25	12.92	55.84	67.10	81.82
	TW	52.10	98.50	22.27	13.13	39.04	51.34	65.35
	3W	39.50	62.07	20.13	8.47	29.81	39.94	48.10
	LCV	52.03	84.59	24.35	13.86	37.50	46.23	62.07
	HV	47.20	71.14	24.82	8.55	38.70	46.50	56.25
IV	CS	65.30	78.24	46.46	7.88	55.92	67.09	73.65
	CB	65.84	89.02	44.32	8.76	55.86	66.81	72.58
	TW	50.97	69.09	33.00	5.16	46.38	49.76	54.86
	3W	44.03	55.41	25.00	5.74	38.87	42.70	49.78
	LCV	48.74	72.59	33.00	6.48	42.74	47.61	53.56
	HV	49.11	64.07	26.55	6.46	44.25	47.80	54.81

V	**CS**	68.42	99.89	30.00	11.70	57.04	69.23	81.63
	CB	71.46	102.10	32.81	14.08	55.64	71.49	84.48
	TW	57.28	96.77	26.91	12.37	46.24	57.93	70.75
	3W	46.90	66.06	33.34	7.73	41.83	46.92	55.90
	LCV	58.35	81.01	36.32	8.80	50.96	58.58	67.77
	HV	54.90	89.91	18.37	10.45	45.12	55.74	65.32
	MAV	47.80	67.16	31.65	7.85	41.61	48.24	55.07
	B	65.20	82.53	38.73	9.20	54.90	65.54	73.13
VI	**CS**	63.08	71.77	48.61	7.02	56.04	64.05	69.56
	CB	60.71	83.00	47.74	9.43	51.76	63.59	69.74
	TW	58.05	73.92	39.85	8.47	49.66	60.83	64.79
	3W	42.92	60.32	26.00	7.70	37.96	40.96	47.16
	LCV	46.50	53.03	32.91	5.76	41.00	47.90	52.61
	HV	43.32	49.37	33.01	4.87	38.60	44.35	48.09
	MAV	36.19	42.86	28.66	4.37	31.67	36.43	41.36
VII	**CS**	75.10	106.70	51.44	12.49	63.44	76.34	88.95
	CB	83.30	110.29	50.01	12.15	70.21	82.83	95.06
	TW	56.50	96.57	32.16	11.20	45.23	55.44	68.25
	3W	49.40	66.00	35.00	8.29	42.19	47.75	60.22
	LCV	60.10	91.09	34.79	13.36	43.73	59.31	73.34
	HV	51.90	77.92	30.97	10.92	41.42	52.20	63.83
	MAV	50.90	73.53	38.68	7.38	45.27	49.59	55.47
	B	66.10	80.25	39.84	7.39	59.29	66.57	74.04
VIII	**CS**	83.14	123.29	50.00	11.83	72.00	82.57	94.74
	CB	84.42	118.42	47.62	11.11	73.77	84.11	95.74
	TW	67.66	111.11	23.23	13.66	55.15	65.83	83.03
	HV	67.76	105.88	38.96	14.91	52.08	64.29	83.33

The speed difference between vehicle type TW and car is more in the case of six lane divided highway section than four lane divided highway section. As the lanes increases, the cars have more freedom to increase their speeds. It has been observed that the 85th percentile speed of vehicle type CB at section II, III, V and VI found to be more than the posted speed limit defined at those sections. And also, 85th percentile speed of vehicle type CS at Section V is more than the post speed limit.

5.4.2 Analysis of speed distribution

Speed of vehicles on a traffic facility is expected to follow a normal distribution. Under the set of circumstances, where a normal distribution fails to provides a better fit to the speed data, gamma distribution or lognormal distribution are also used. Many researchers claim that the speed data on a section of highway follow the normal or gamma or log normal distribution (Donald and Daniel (1951), Dixon et al., (1999), Christopher (1994), Filippo and Marinella (2011), Khairi et al., (2011), Hastim and Ramli (2013)). The probability density function of these distributions are discussed as follows.

- **Normal distribution**

The normal probability curve with mean μ and standard deviation σ is given as

$$f(x;\mu,\sigma)=\frac{1}{\sqrt{2\pi\sigma^2}}\exp\left\{-\frac{(x-\mu)^2}{2\sigma^2}\right\} \qquad -\infty<x<\infty \quad -\infty<\mu<\infty \quad \sigma>0$$

The normal distribution generally known as "normal curve of errors" is bell-shaped and is symmetrical about the line x = μ. The mean, median and mode of the normal distribution coincide with each other.

- **Gamma distribution**

The probability density function of gamma distribution is given as,

$$f(x)=\frac{1}{\Gamma(\alpha)\beta^{\alpha}}x^{\alpha-1}e^{-x/\beta} \qquad x\geq 0 \quad \alpha>0 \quad \beta>0$$

α and β = parameters, Γ(α) =(α-1)Γ(α–1), mean of gamma distribution (μ)= αβ, and variance of gamma distribution (σ^2) = $\alpha\beta^2$.

- **Log-Normal distribution**

The probability density function of Log-Normal distribution is given as,

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma x} e^{-(\ln(x)-\mu)^2/2\sigma^2} \qquad x > 0, \quad -\infty < \mu < \infty, \quad \sigma > 0$$

where, μ is the mean of lognormal distribution which is expressed as $\mu = e^{\alpha+\beta^2/2}$, and σ^2 is the variance of lognormal distribution expressed as $\sigma^2 = e^{2\alpha+\beta^2}(e^{\beta^2} - 1)$.

For present study, the profiles of observed speed frequencies were developed and compared them with the above stated distributions for all the study sections. Chi-square test, K-S test and Anderson Darling test were applied at 5% level of significance and test of hypothesis was performed. The results from the goodness of fit tests conducted on speed data are summarized in Table 5.8.

Table 5.8 Goodness of fit tests of the fitted distributions for different study sections

Distribution	Goodness of fit	Section I	Section II	Section III	Section IV	Section V	Section VI	Section VII	Section VIII
Normal	Chi-squared test	Not follow	Not follow	Not follow	Not follow	Follow	Not follow	Not follow	Not follow
	K -S test	Not follow	Not follow	Not follow	Not Follow	Follow	Follow	Not follow	Follow
	Anderson Darling test	Not follow	Not follow	Not follow	Not Follow	Follow	Follow	Not follow	Follow
Log-Normal	Chi-squared test	Not follow	Not follow	Follow	Not follow	Not follow	Follow	Not follow	Not follow
	K -S test	Follow	Not follow	Follow	Not Follow	Not Follow	Follow	Follow	Not follow
	Anderson Darling test	Follow	Not follow	Follow	Not follow	Not follow	Follow	Follow	Not follow

Gamma	**Chi-squared test**	Not follow	Not follow	Follow	Not follow	Follow	Follow	Not follow	Not follow
	K- S test	Not follow	Not follow	Follow	Not follow	Follow	Follow	Not follow	Not follow
	Anderson Darling test	Not follow	Not follow	Follow	Not follow	Not Follow	Follow	Not follow	Not follow

It is confirmed by the results obtained from different tests of goodness of fit that the observed speed frequencies at Section-I, Section-III, Section-VI and Section-VII follows lognormal distribution and at Section-V, Section-VI and Section-VIII it follows normal distribution. Moreover, the speed frequencies are observed to be followed gamma distribution at Section-III, Section-V and Section-VI. However, Speed data collected at Section-II and Section-VI does not follow any of these three distribution types.

Generally, the observed speed data expected to follow normal distribution. But, it is observed as except study Section-V other sections have not followed the normal distribution. This is because of when the traffic mix become heterogeneous, both congestion and free flow situation exit, the speed distribution deviates from the normal distribution. However at Section-V, most of vehicles are travelling with less variation in the speed, even with the good mix of traffic.

Table 5.9 presents stated probabilistic distributions fitted to the speed data along with their estimated parameters. The results obtained from the various tests conducted to confirm the goodness of fit are also given in this table.

Table 5.9 Estimated parameters of the fitted distributions for speed data at different study sections

Section	Distribution	Parameters	K-S Test Value	Critical K-S Test Value
I	Log-Normal	σ=0.28418 μ=3.8661	0.02883	0.03223
III	Log-Normal	σ=0.28477 μ=3.8853	0.02512	0.04519
	Gamma	α=12.773 β=3.9654	0.0327	
V	Normal	σ=13.316 μ=63.266	0.07281	0.03123
	Gamma	α=22.572 β=2.8028	0.02633	
VI	Normal	σ=12.146 μ=50.818	0.01897	0.1182
	Log-Normal	σ=0.24165 μ=3.8995	0.08143	
	Gamma	α=17.505 β=2.903	0.07793	
VII	Log-Normal	σ=0.253 μ=4.1303	0.03101	0.03736
VIII	Normal	σ=14.616 μ=77.989	0.03441	0.03714

The frequency distribution profiles for all the sections are plotted and PDF of Section-I and Section-VIII are shown in Figure 5.8 and Figure 5.9.

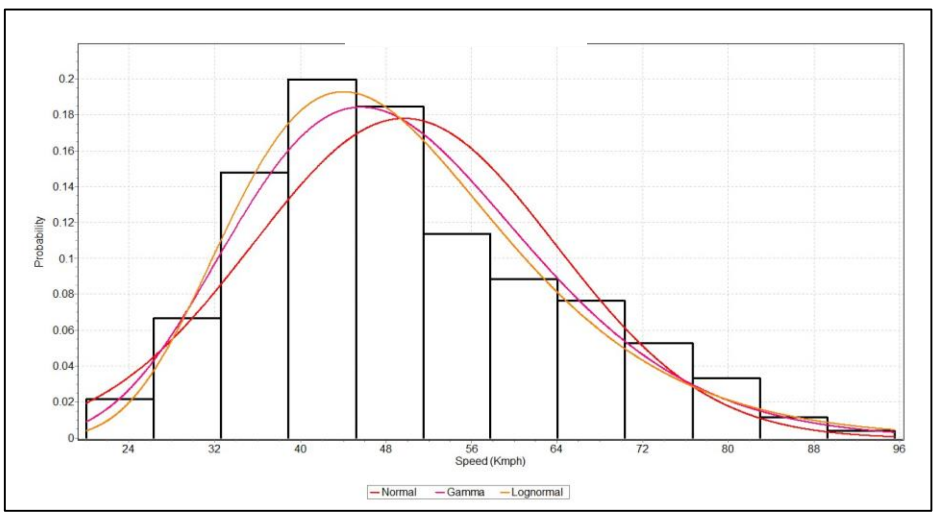


Figure 5.8 Distribution fit for speed data collected at Section-I

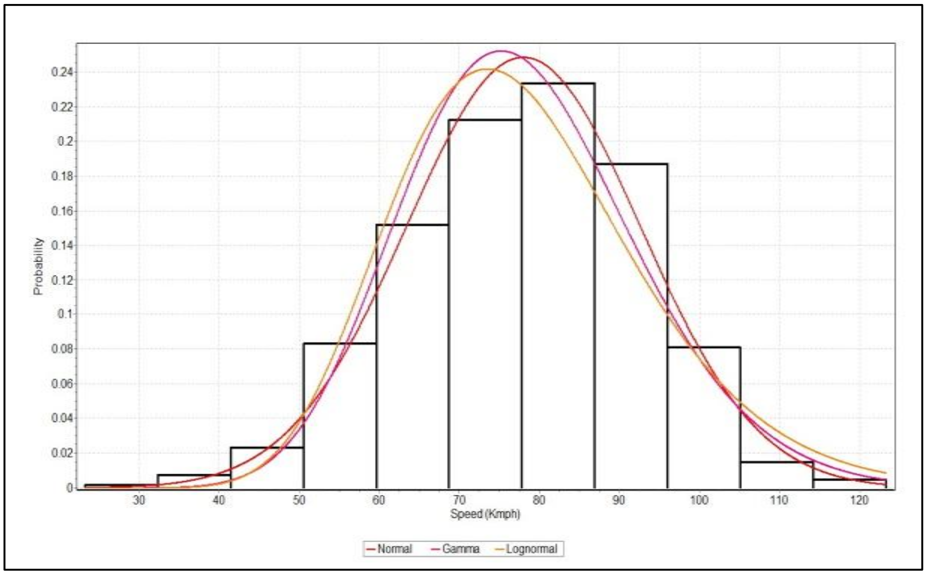


Figure 5.9 Distribution fit for speed data collected at Section-VIII

5.5 Traffic volume

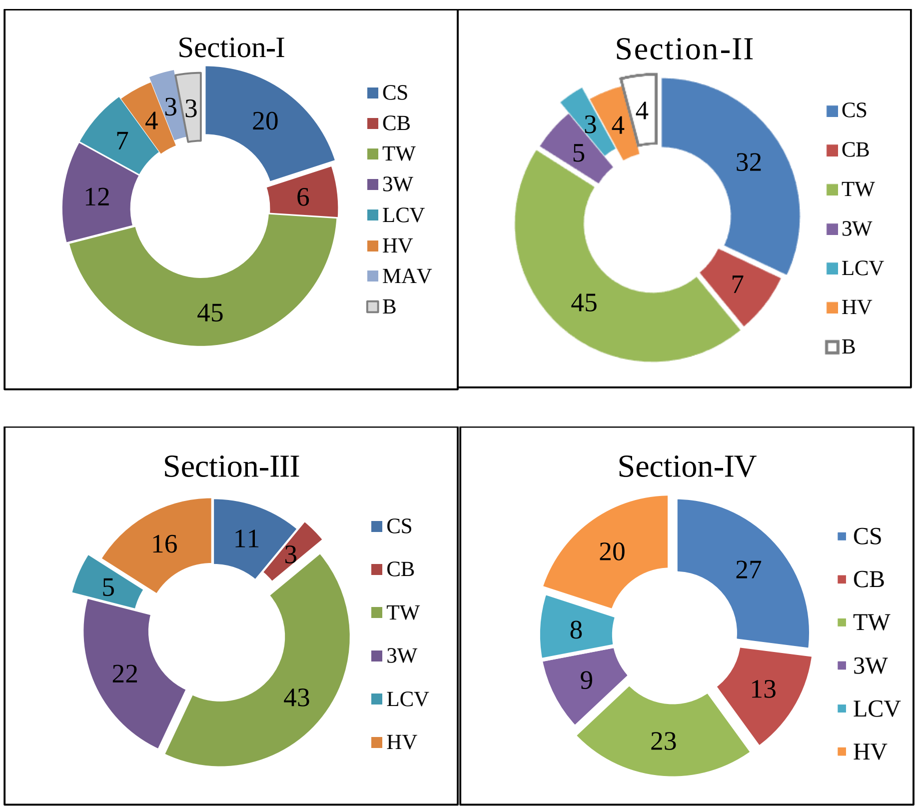
Classified traffic volume count was extracted from the recorded video based on the reference line marked on highway section. Details of data collection duration, volume and speed limits of study sections are given in Table 5.10.

Table 5.10 Details of duration and traffic volume of different sections

Section	Duration	Traffic Volume(Veh/hr)	
		Maximum	Minimum
Section-I	9.00 AM to 12.00 PM and 3.00 PM to 6.00 PM	1400	600
Section-II	9.00 AM to 12.00 PM and 3.00 PM to 6.00 PM	1512	576
Section-III	7.00 AM to 11.00 AM	551	414

Section-IV	9.00 AM to 12.00 PM and 3.00 PM to 6.00 PM	1630	445
Section-V	9.00 AM to 12.00 PM	1679	1200
Section-VI	8.00 AM to 12.00 PM	1656	960
Section-VII	9.00 AM to 12.00 PM and 3.00 PM to 6.00 PM	1776	900
Section-VIII	10.00 AM to 1.00 AM	1496	1044

The traffic composition of study sections was measured from observed volume data and is given for all highway sections in Figure 5.10. Traffic composition of vehicle type TW is found to be more than all other vehicle types at Section I, II, III and VII whereas traffic composition of vehicle type CS is observed as higher in rest of the highways sections.



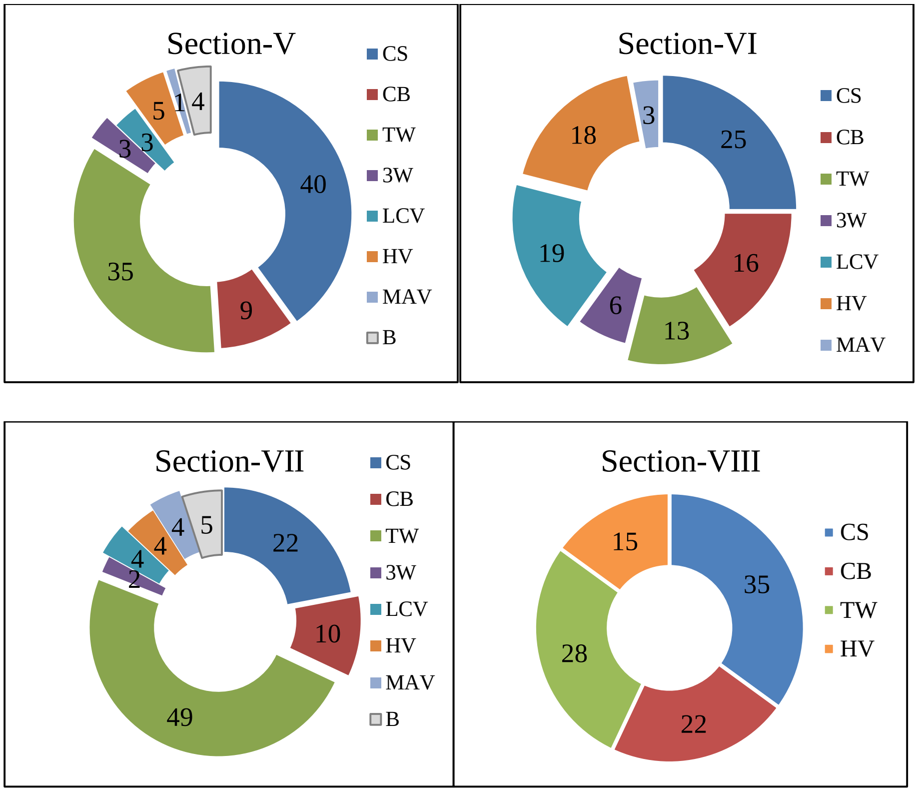


Figure 5.10 Vehicle compositions (%) at study sections

5.6 Estimation of following distance

While travelling on road, if driver is following a vehicle, he/she will be maintaining a certain distance with a lead vehicle which is called as following distance. Following distance is depending on the behaviour (speed change) of the leading vehicle. Some drivers are maintaining larger spacing between vehicles even though speed of leading vehicle is low. Optimum velocity model suggests that a driver tries to achieve an optimal velocity based on speed difference between both vehicles. The equation 5.7 gives the speed of nth vehicle.

$$v^t_{n\ desired} = v^{opt}(\Delta x^t_n) \qquad (5.7)$$

where, v^{opt} is the optimal velocity function which is a function of instantaneous distance headway Δx^t_n. Therefore a^t_n is given by

$$a^t_n = \left[\frac{1}{\tau}\right] \left[\ v^{opt}(\Delta x^t_n) - v^t_n\right] \qquad (5.8)$$

where, $\frac{1}{\tau}$is called as sensitivity coefficient. In short, the driving strategy of n^{th} vehicle is that, it tries to maintain a safe speed which in turn depends on the relative position, rather than relative speed.

Above equation implies that velocity is proportional to distance between the vehicles. At higher speed, spacing between the vehicles will also be higher. In the action point model drivers are able to perceive relative speed by detecting changes in the apparent size of downstream vehicles.

Action point model is used to calculate safety distance between the vehicles at Section-V. Relationship shown in Figure 5.11 is indicating as volume on the road is increasing following distance between vehicles is decreasing exponentially.

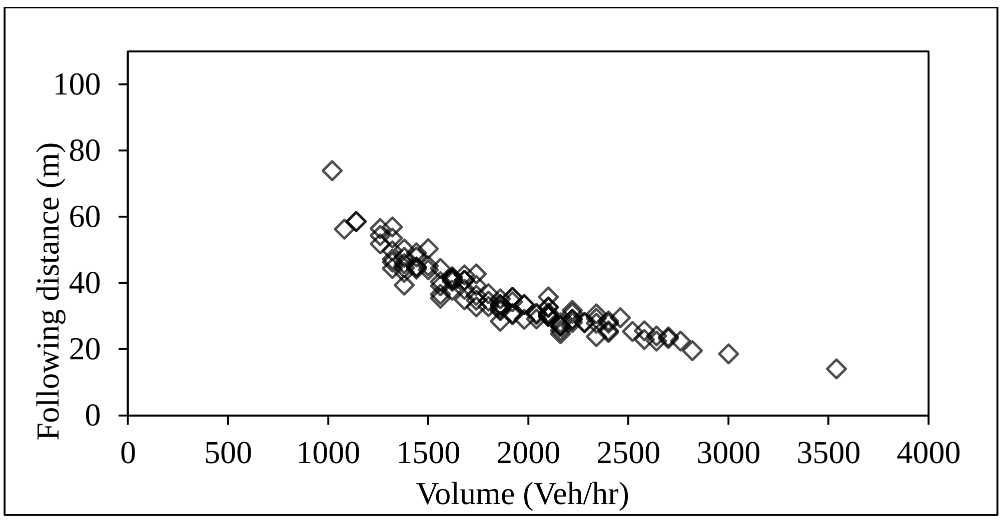


Figure 5.11Variation of following distance with traffic volume

5.7 Lateral distribution of vehicles

Lateral distribution or lateral placement characteristics of vehicles on multilane divided highways refer to the distribution of traffic across the pavement width or lanes. It is an important operational measure of lateral traffic behavior which may be influenced by the vehicle dimensions, speed, traffic volume, traffic composition, roadway features, and origin-destination, travel patterns, geometric conditions, weather, driver behavior, lane width, lane type, location of entry points, and other road characteristics. (Shukla and Chandra, 2008).

5.7.1 Lateral distribution Analysis

Placement data for lateral distribution characteristics of vehicles was collected at Section-II. Video-graphic method was used to collect the data. The placement data from the recorded videos were extracted by playing the videos on a large screen monitors in Transportation Engineering Lab. The placements of vehicles were recorded by noting down the strip number over which vehicle placed its left wheel while traversing the highway section. The vehicles category was also recorded along with the strip number. The lateral position of the left wheel of Cars and Two wheelers from the median edge along the width of the pavement is shown in Figure 5.12. From Figure 5.12, it is observed that higher percentage of cars is travelling in median lane where as higher percentage of two wheelers is travelling in shoulder lane.

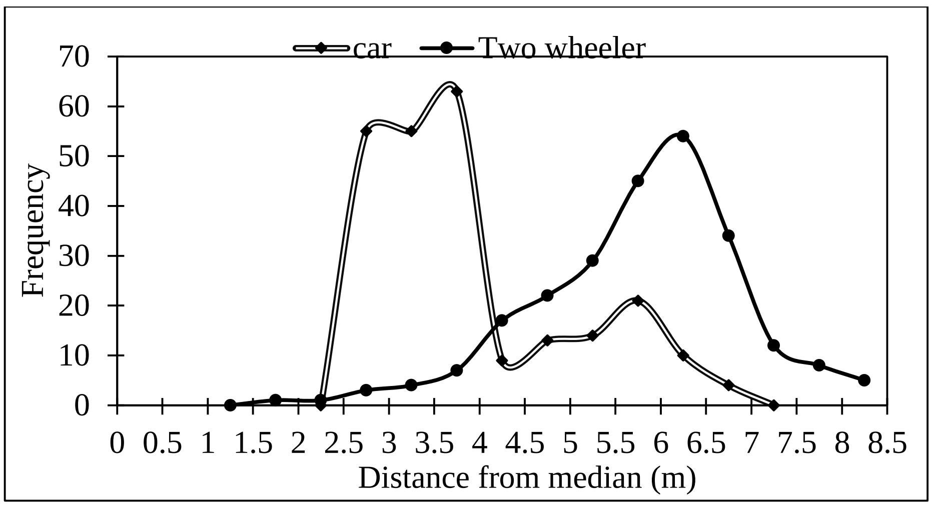


Figure 5.12 Lateral placement of Cars and two wheelers at section II

Literature review indicates that the frequency distribution of lane-wise lateral placement data of vehicles generally follows a normal distribution. Therefore, a normal distribution was tried to fit the observed data. The frequency distribution of vehicles observed for total traffic at Section II is shown in Figure 5.13. The calculated Chi-square value at Section II was obtained as 136.30 against the tabulated value as 21.02 at 12 degrees of freedom, at 5% level of significance. The observed distribution shows many fluctuations in the data throughout the pavement width, along with fluctuations existing within each lane. Thus, a normal distribution is failed to explain the observed lateral distribution of vehicles across the carriageway width.

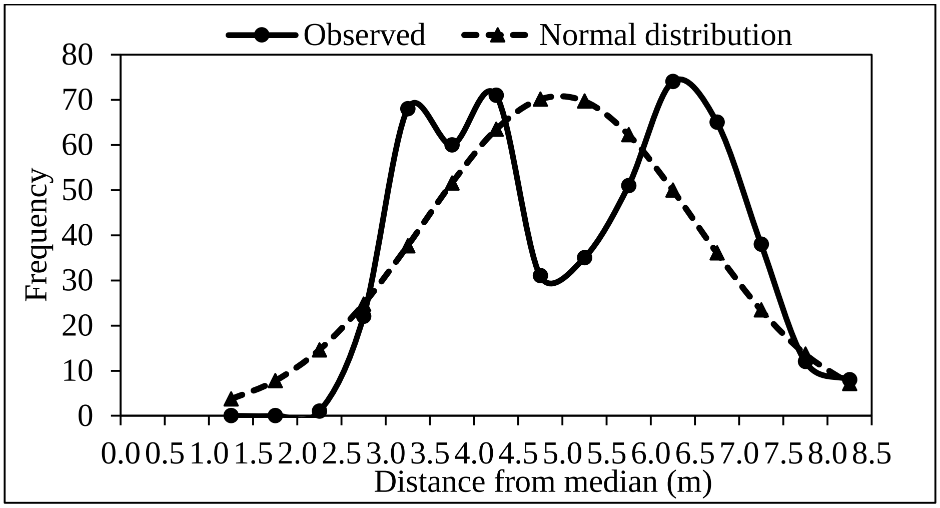


Figure 5.13 Lateral placement of mixed traffic at section II fitted to normal distribution

5.8 Overtaking behavior

Overtaking and lane-changing maneuvers are undesirable traffic movements with potential for causing road accidents, due to increase in number of conflicting points. These operations may produce hazardous situations which consequently reduces traffic safety. The present section describes the overtaking behavior of vehicles on multilane divided highways on a selected highway section.

5.8.1 Analysis of overtaking data

The data to analyze overtaking vehicles was collected by floating car (test vehicle) method on highway section and determined the relevant parameters such as acceleration rates, overtaking speed, overtaking distance and time of overtaking vehicles. The methodology of data collection has been discussed in the Chapter 4 in detailed manner. Table 5.11 gives a sample set of overtaking data.

Table 5.11 Set of overtaking data for truck

S. No.	Speed of test vehicle, kmph	Type of overtaking vehicle	Event 1 (t_0)	Event 2 (t_1)	Event 3 (t_2)	Event 4 (t_3)	Event 5 (t_4)
1.	40	Truck	0	5.27	6.95	10.00	-

Distance travelled between events 2 and 3:

By test vehicle = $u_t*(t_2-t_1)$

By overtaking vehicle = $u_t*(t_2-t_1) + L_t$

Distance travelled between events 2 and 4:

By test vehicle = $u_t*(t_3-t_1)$

By overtaking vehicle = $u_t*(t_3-t_1) + L_t + L_o$

Where, u_t= speed of test vehicle, m/s

u_o = speed of overtaking vehicle, m/s

t_1 = time elapsed between events 1 and 2, s

t_2 = time elapsed between events 1 and 3, s

t_3 = time elapsed between events 1 and 4, s

L_t = Length of test vehicle, m

L_o= Length of overtaking vehicle, m

The distance travelled by a vehicle with initial speed 'u' and acceleration 'a' in time 't' is given as,

$$S = ut + \frac{1}{2}at^2 \tag{5.9}$$

A comparison of equations for distance travelled by overtaking vehicle between events 2 and 3 with the distance calculated from equation (5.10) gives,

$$S_{2-3} = u_t(t_2 - t_1) + L_t = u_o(t_2 - t_1) + \frac{1}{2}a(t_2 - t_1)^2 \tag{5.10}$$

Similarly, a comparison of equations for distance travelled by overtaking vehicle between events 2 and 4 with equation 5.6 gives,

$$S_{2-4} = u_t(t_3 - t_1) + L_t + L_o = u_o(t_3 - t_1) + \frac{1}{2}a(t_3 - t_1)^2 \tag{5.11}$$

Equations (5.10) and (5.11) were solved for the values of u_o and a for each data set collected in field. For the sample data set given in Table 5.3, t_1 = 5.27 s, t_2 = 6.95 s, t_3 = 10.00 s, L_t = 4.15 m, L_o = 10 m and u_t= 40 kmph ≈ 11.11 m/s. Substituting these values in equations 5.10 and 5.11 and solving for u_o and a provides, $a =$ 0.342 m/s^2 ≈ 1.23 km/h/s and $u_o =$ 13.29 m/s ≈ 47.8 kmph.

Similarly, acceleration rates were determined for each data set for different types of overtaking vehicles. The average, minimum and maximum rates of acceleration for different types of vehicles are given in Table 5.12. The overtaking study was done separately and only default acceleration are used for simulation.

Table 5.12 Acceleration rates on highway section with paved shoulders

Type of overtaking vehicle	Minimum (km/h/s)	Maximum (km/h/s)	Average (km/h/s)	Standard deviation (km/h/s)
CS	0.03	2.80	1.12	0.97
CB	0.04	3.75	1.22	1.18
TW	0.18	1.71	1.05	0.55

The acceleration rates of overtaking vehicles were analysed with varying overtaking speeds. Figure 5.14 depicts the influence of overtaking speed on acceleration rate for different vehicle types. A negative correlation is observed to exist between the two parameters. Higher overtaking speeds are generally associated with lower rates of acceleration and therefore this trend is justified.

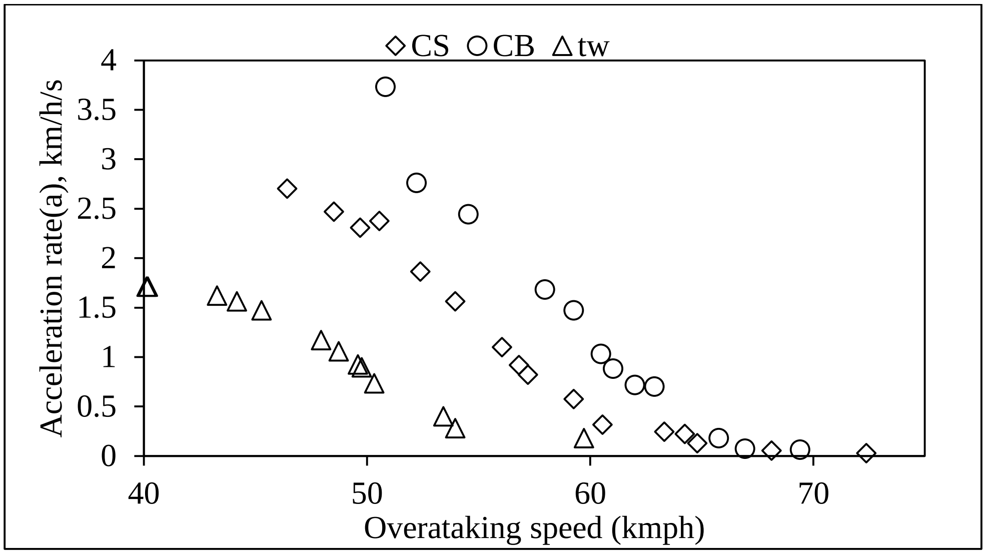


Figure 5.14 Effect of overtaking speed on acceleration rate

5.8.2 Estimation of overtaking distance and time

In the case of flying overtaking, the overtaking vehicle does not follow the test vehicle and deflects to the adjacent lane much earlier and accomplishes the overtaking maneuver. After completing the overtaking, it may return to its original lane at a farther distance or it may continue to move in the adjacent lane. For example, a vehicle A overtakes vehicle B then considering the case when overtaking vehicle A returns to its original lane, the total time for overtaking vehicle A to complete the maneuver may be calculated as

The total distance travelled by vehicle A during overtaking maneuver,

$$S = S_A + L_B + S_B + L_A \quad (5.12)$$

$$S = (0.278 * V_A * t) + L_B + (0.278 * V_B * t) + L_A \quad (5.13)$$

Where, $t = 2$ seconds, minimum time between vehicles *A* and *B* before and after overtaking

S_A and S_B = Distance (in meters) travelled by overtaking vehicle *A* and overtaken vehicle *B* respectively, in time *t* seconds

L_A and L_B = Length of overtaking vehicle *A* and overtaken vehicle *B* respectively, in m

V_A and V_B = Speed of overtaking vehicle *A* and overtaken vehicle *B* respectively, kmph

Equation 5.13 is used to calculate the total distance travelled by the each category of overtaking vehicle for various categories of overtaken vehicles during flying overtaking operations. This calculated distance is then used to determine the total overtaking time T, by using the relationship,

$$\text{Time taken, T (s)} = \frac{\text{Overtaking distance, S (m)}}{0.278 * (V_A - V_B)}, \text{ where, } V_A \text{ and } V_B \text{ in kmph} \qquad (5.14)$$

Figure 5.15 show the plot between the difference in speeds of overtaking and overtaken vehicle and total overtaking time required for overtaking. A negative correlation observed between difference in speeds of overtaking and overtaken vehicle and total overtaking time required for overtaking.

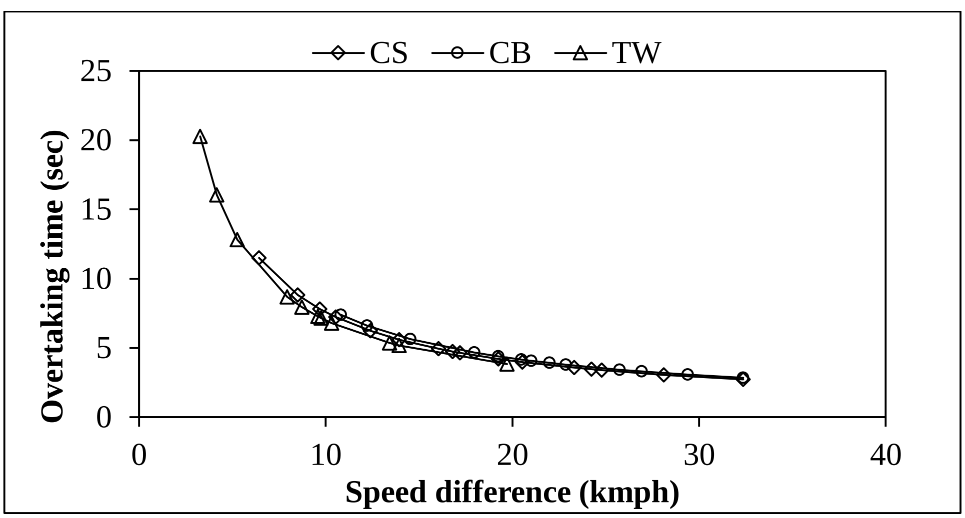


Figure 5.15 Plot of overtaking time versus speed difference

5.9 Estimation of PCUs

In developing countries, several methods were developed for determining PCUs like homogenization coefficient method, method based on relative delay, walker's method, headway method, simultaneous equations, multiple linear regression method, simulation method (Chandra et al., 1997). Most of these methods are commonly used under homogeneous traffic conditions but failed in to justify the converted volumes under heterogeneous traffic conditions. Methods developed for PCU estimation under mixed traffic conditions are also not unerring in all the aspects of degrees or levels of heterogeneity as the estimated PCUs are also tend to vary with the change in the traffic and roadway factors. Since, PCU is expected to be different for different vehicle types therefore, it is considered to be a very complex parameter for the measurement of traffic flow. Due to arising complexities in volume measurement, selection of appropriate parameters and methods for determining PCUs needs a better and comparative study. In this section, models developed for estimation of PCUs of different vehicle types by using field data under mixed traffic conditions.

5.9.1 PCU estimated by available methods

The field data collected at Section-I and Section-VII used for estimation of PCUs of different vehicle types by Homogenisation Coefficient Method, Headway Method, Simultaneous Equation Method, Multiple Linear Regression Method and Dynamic PCU Method. Details of these methods were explained in APPENDIX-A. PCU values of different vehicle types on Section-I and Section-VII estimated by various methods are shown in Table 5.13 and Table 5.14.

Table 5.13 PCU values of different vehicle types on Section-I

Methods	Passenger Car Unit(PCU)						
	CB	3W	HCV	MAV	LCV	TW	B
Homogenisation Coefficient Method	1.24	1.39	2.92	5.74	1.63	0.77	4.31
Headway Method	1	1	11.05	11.05	1	1	11.05
Simultaneous Equation Method	4.02	6.03	-5.03	2.02	6.27	-0.04	-8.11
Multiple Linear Regression Method	-10.19	13.7	34.67	-12.95	30.72	21.91	34.72
Dynamic PCU Method	1.26	1.05	3.96	8.21	1.51	0.34	5.00

Table 5.14 PCU values of different vehicle types on Section-VII

Methods	PCU of subject vehicle type						
	CB	3W	HCV	MAV	LCV	TW	B
Homogenisation Coefficient Method	1.36	1.50	3.01	5.3	1.67	0.79	3.74
Headway Method	1	1	5.72	5.72	1	1	5.72
Simultaneous Equation Method	29.91	-8.51	-81.19	-17.89	-69.07	9.17	35.98
Multiple Linear Regression Method	2.69	-12.77	-1.67	-0.88	-5.49	1.94	-0.06
Dynamic PCU Method	1.38	1.12	4.08	7.58	1.54	0.35	5.29

It can be observed that the values obtained for both the sections by simultaneous equation method and multiple linear regression are negative which seems meaningless as per the PCU definition is concerned. Headway method results are also found unrealistic beacuse it devides the vehicle type into two categories which is underestimating the different vehicle classes. The results obtained from homogenisation coefficient method and dynamic PCU method are found realistic and suitable for converting heterogeneous traffic flow into homogeneous traffic flow.

In homogenization coefficient method, PCU values are estimated by using speed and length of vehicle types. However, Indian traffic follows lane disordered system and using only length of vehicle type alone with speed may not be sufficient for PCU estimation. In dynamic PCU method, rectangular projected area is used instead of using length of vehicle types. Hence, the dynamic method of estimating PCUs is considered to be more effective under such traffic conditions.

5.9.1.1 Comparison of Dynamic PCUs on unpaved and paved shoulder section

PCU values were estimated for subject vehicle type using dynamic PCU method on two sections of four-lane divided highways, one with paved shoulder (Section-I) and another with unpaved shoulder (Section-IV). The average PCU values estimated for each vehicle type on two different sections were compared by performing z-test (with equal variance) at 5% level of significance level to check whether the type of shoulder affect the PCU values of subject vehicle types on four-lane highway sections (Table 5.15).

Table 5.15 PCU value comparison for paved and unpaved shoulder section

Vehicle type	PCU (Unpaved)	PCU (Paved)	z	Remark
CB	1.35	1.38	2.85	Significant
TW	0.26	0.34	-8.59	Significant
3W	1.02	1.05	-1.22	Insignificant
LCV	1.49	1.51	-0.34	Insignificant
HV	3.69	3.96	-1.84	Insignificant
MAV	7.9	8.21	-0.37	Insignificant

* z critical = 1.96

The test results confirmed that there is significant difference was observed between the PCU values of vehicle types CB and TW due to change in shoulder type. However, no difference in PCU was found in case of vehicle types LCV, HCV, MAV and 3W. It may be due the vehicle type CB and TW experience more freedom to increase their speeds in presence of bituminous paved shoulders.

5.9.2 Modified dynamic PCU method

By considering factors influencing PCU and available methodologies, modification to the dynamic PCU method has been proposed. Unlike dynamic PCU method, this method includes time headway factor for PCU estimation. The methodology for estimation of PCU using modified method is explained below.

5.9.2.1 Development of Method

The factors considered in present study are average speed, average time headway and rectangular projected area of vehicle types as observed in field. Details of factor considered in development of modified PCU method are given as under.

- **Speed factor (F_v)**

Time spent by a vehicle in the traffic stream may be decreased with increase in average speed of vehicle. When other factors remain constant, PCU of subject vehicle type will be inversely proportional to average speed of same vehicle type. Speed factor is expressed as the ratio of average speed of standard car to the average speed of subject vehicle type.

$$F_v = \frac{V_c}{V_i} \qquad (5.15)$$

Where

F_v = Speed factor of subject vehicle type; V_c = Average speed of standard car (kmph); V_i = Average speed of subject vehicle type (kmph)

- **Headway factor (F_t)**

Longitudinal space available for vehicle in the traffic stream increases with the increase in the mean time headway maintained by the vehicle (Anand et al., 1999). Longitudinal space available for vehicle may also affect its maneuverability in the stream. Hence, time headway may also affect the PCU of a subject vehicle type. Therefore, Headway factor is calculated based on the average time headway of different vehicle type as ratio of average lower time headway of subject vehicle type (T_i) to the average lower time headway of standard car (T_c). The relation to find headway factor is given in equation 5.16.

$$F_t = \frac{T_i}{T_c} \qquad (5.16)$$

Where F_t = Headway factor of subject vehicle type; T_c = Average lower time headway of standard car (sec); T_i = Average lower time headway of subject vehicle type (sec).

- **Area factor (F_a)**

PCU of a vehicle type depends on vehicular dimensions. PCU is inversely proportional to area of vehicle. Area factor is the ratio of rectangular projected area of standard car (A_c) to the area of subject vehicle type (A_i).

$$F_a = \frac{A_i}{A_c} \tag{5.17}$$

Where F_a = Area factor of subject vehicle type; A_c = Rectangular projected area of standard car (m^2); A_i = Rectangular projected area of subject vehicle type (m^2).

- **PCU of subject vehicle type**

The new equation for calculating PCU_i is proposed in present study which includes speed factor, headway factor and area factor.

$$PCU_i = F_v * F_t * F_a \tag{5.18}$$

Where PCU_i = PCU value of subject vehicle type; F_v = Speed factor of subject vehicle type; F_t = Headway factor of subject vehicle type; F_a = Area factor of subject vehicle type.

5.9.2.2 Estimation of PCU

The PCU value of a subject vehicle type is estimated as the product of speed factor, headway factor and area factor. The average speed data and average time headway data of vehicles are used to estimate PCUs. The data used for PCU estimation has been provided in the previous sections. Correlation analysis was performed between speed and headway data. The correlation coefficient values obtained between speed and headway data obtained from the Section-I, Section-V and Section-VII are -0.12, -0.09 and -0.15 respectively. Statistically weak correlation was found between the speed and headway data hence there is no dependency found between speed and headway of vehicles. PCU values of different types of vehicles are estimated by proposed method using data obtained at Section-I, Section-V and Section-VII are given in Table 5.16 to Table 5.18.

Table 5.16 PCU values of different vehicle types by modified method on Section I

Vehicle Type	Speed factor	Headway factor	Area factor	PCU
CB	0.963	1.021	1.299	1.28
LCV	1.355	1.034	1.100	1.54
HV	1.532	1.049	2.525	4.06
MAV	1.650	1.175	4.566	8.85
TW	1.430	1.131	0.239	0.39
3W	1.581	1.118	0.667	1.18
B	1.427	1.156	4.184	6.90

Table 5.17 PCU values of different vehicle types by modified method on Section V

Vehicle Type	Speed factor	Headway factor	Area factor	PCU
CB	0.958	0.947	1.299	1.18
LCV	1.174	1.000	1.100	1.29
HV	1.244	1.185	2.525	3.72
MAV	1.130	1.443	4.566	7.95
TW	1.192	1.130	0.239	0.32
3W	1.458	0.896	0.667	0.87
B	1.048	1.446	4.184	6.34

Table 5.18 PCU values of different vehicle types by modified method on Section VII

Vehicle Type	Speed factor	Headway factor	Area factor	PCU
CB	1.107	1.012	1.299	1.45
LCV	1.383	1.035	1.100	1.57
HV	1.601	1.027	2.525	4.15
MAV	1.633	1.029	4.566	7.67
TW	1.471	0.997	0.239	0.35
3W	1.682	1.025	0.667	1.15
B	1.257	1.012	4.184	5.32

It can be observed that the PCUs of different vehicle types at Section-I are higher than Section-V i.e. PCUs of different vehicle types observed on paved shoulder section are higher than unpaved shoulder section. It is due average headway of vehicle types higher at Section-I than Section-V. PCUs of different vehicle types at Section-VII (Six-lane) are higher than Section-I (Four-lane) except vehicle types B and MAV. It is due to average speeds of B and MAV vehicle types are very low values at Section-I.

5.9.2.3 Comparison of estimate PCUs

The PCU values estimated using proposed method are compared with the values determined by using dynamic PCU method for all subject vehicle types. The estimated PCU values estimated from both the methods are shown in Figure 5.16 and Figure 5.17.

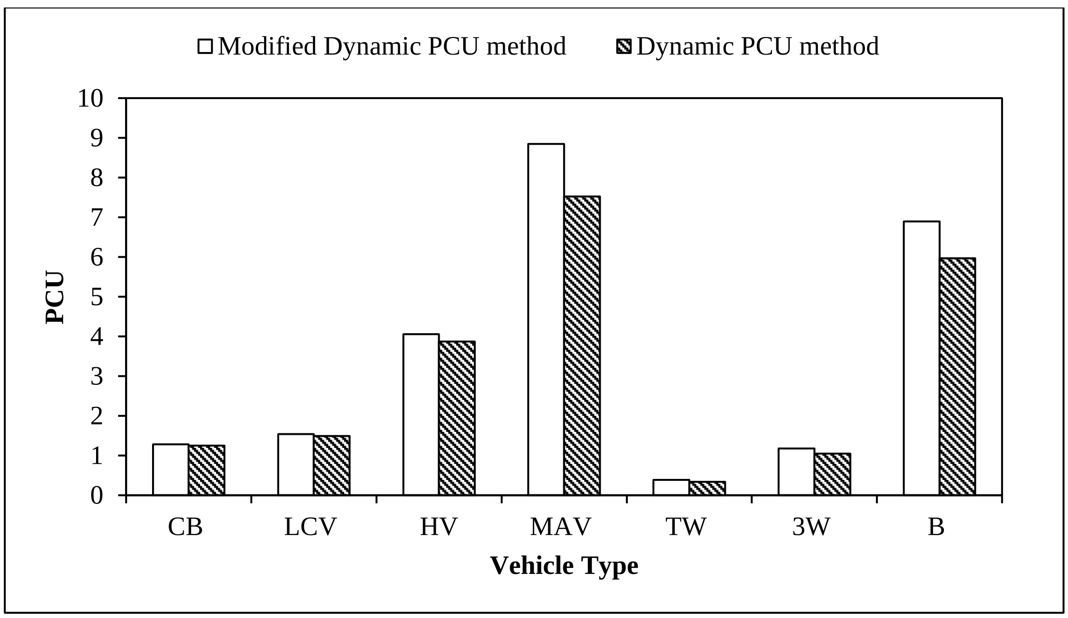


Figure 5.16 PCU estimated using modified method and dynamic method on Section-I

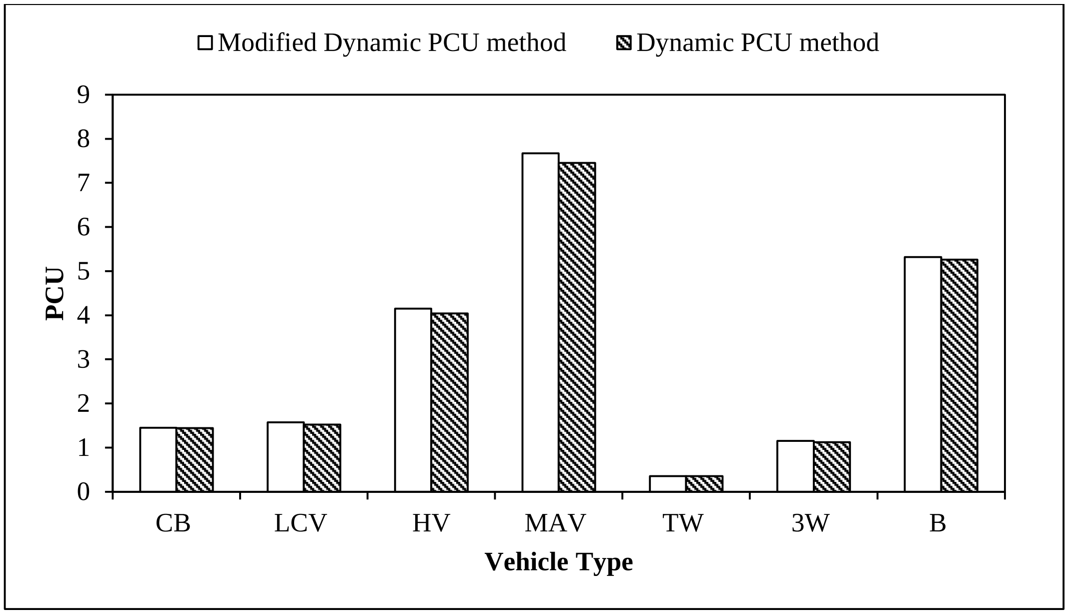


Figure 5.17 PCU estimated using modified method and dynamic method on Section-VII

The comparative analysis confirmed that the PCU values estimated from modified approach are relatively higher than the Dynamic PCU values. However, the difference is more in case of heavy vehicle type such as HV, MAV and B.

5.9.3 PCUs by multiple non-linear regression (MNLR method)

The methodology for estimation of PCU is based on the non-linear regression method in which a speed model will be developed to estimate PCU values. The method includes the variables like proportion of each vehicle type, average speed of vehicle types and area ratios of standard vehicle to all other subject vehicle types. There are two components in the model; one is additive component and another one is multiplicative component. The product of the area ratio of vehicle type Small car (CS) to subject vehicle type, proportion share of subject vehicle type, and average speed of subject vehicle type are used as multiplicative component, whereas a proportional share of CS is used as an additive component in the proposed method. The proposed model was developed to predict the speed of standard vehicle type such as CS, whose coefficients are estimated as PCUs of all subject vehicle types. Equation (5.19) is provided as general form for predicting the average speed of standard car.

$$V_{CS} = \sum_{j=1}^{k} a_j \left(\frac{A_{CS}}{A_j} * n_j * V_j\right) + a_i * n_{CS} \tag{5.19}$$

Where V_{CS} is the average speed of small car (km/h), a_j and a_i are the regression coefficients, k is total number of vehicle types in the traffic stream, V_j is average speed of vehicle type j (km/h), n_j is the proportion of vehicle type j, n_{CS} is proportion of small cars, A_j is the rectangular projected area of subject vehicle type j, and A_{CS} is the rectangular projected area of a CS (m^2).

The intercept term in the equation was not kept because the speed of the small car type must be fully explained by the chosen variables. The PCU value of vehicle type j is the regression coefficient (a_j) of corresponding subject vehicle type.

Similarly, the equations (5.20) and (5.21) are also proposed in case of two-wheeler (TW) and heavy commercial (HV) vehicle types, in order to obtain their average speeds and to estimate equivalency units in their respective term.

$$V_{TW} = \sum_{j=1}^{k} b_j \left(\frac{A_{TW}}{A_j} * n_j * V_j\right) + b_i * n_{TW} \tag{5.20}$$

Where V_{TW} is the average speed of two-wheelers (km/h), b_j and b_i are the regression coefficients, k is total number of vehicle types in the traffic stream, V_j is average speed of vehicle type j in km/h, n_j is the proportion of vehicle type j, n_{TW} is the proportion of two-wheelers, $A_j$is the rectangular projected area of subject vehicle type j, and A_{TW} is the rectangular projected area of the two-wheelers.

$$V_{HCV} = \sum_{j=1}^{k} c_j \left(\frac{A_{HCV}}{A_j} * n_j * V_j\right) + c_i * n_{HCV} \tag{5.21}$$

Where V_{HV} is the average speed of HV (km/h), c_j and c_i are the regression coefficients, k is the total number of vehicle types in the traffic stream, V_j is the average speed of vehicle type j (Km/h), n_j is the proportion of vehicle category j, n_{HCV} is proportion of HV, A_j is the rectangular projected area of subject vehicle type j, and A_{HCV} is the rectangular projected area of HV.

5.9.3.1 Estimation of equivalency units

Field data collected at Section-I was used for the development of speed models and Section-II data was used for the validation of the developed model. The traffic composition and average speed of all vehicle types on these sections are given in pervious sections. Proportional share of MAV vehicle type on these sections are very less. Therefore, MAV vehicle type is merged with HV vehicle type.

The MNLR equation (5.22) predicts the speed of a standard vehicle type within a heterogeneous traffic stream. Initially, the speed of vehicle types and their proportional shares are aggregated in 5 minute intervals, establishing a relationship to estimate average speed of the CS. The PCU values of subject vehicle types are identified as regression coefficients of the proposed regression model, as given in Table 5.19. The coefficient a_1 was estimated as the average speed of CS, which is also affected by its own proportional share. The value of coefficient a_1 was 63 km/h. The value of R^2 for the model is 0.77. The high R^2 value indicates the strength of the model in predicting the speed of CS.

$$\begin{aligned} V_{CS} = a_1 * n_{CS} + a_2 * \left[\frac{A_{CS}}{A_{CB}} * n_{CB} * V_{CB} \right] + a_3 * \left[\frac{A_{CS}}{A_{LCV}} * n_{LCV} * V_{LCV} \right] + a_4 * \left[\frac{A_{CS}}{A_{HV}} * n_{HV} * V_{HV} \right] \\ + a_5 * \left[\frac{A_{CS}}{A_{TW}} * n_{TW} * V_{TW} \right] + a_6 * \left[\frac{A_{CS}}{A_{3W}} * n_{3W} * V_{3W} \right] + a_7 * \left[\frac{A_{CS}}{A_{B}} * n_{B} * V_{B} \right] \end{aligned} \tag{5.22}$$

Where, a_2=PCU of CB, a_3=PCU of LCV, a_4=PCU of HV, a_5=PCU of TW, a_6=PCU of 3W, a_7=PCU of B.

Table 5.19 Regression coefficient as PCU value of subject vehicle types

Vehicle Type	Coefficients	PCU Values	Standard Error
CB	a_2	1.56	0.15
LCV	a_3	2.69	0.42
HCV	a_4	3.83	0.65
TW	a_5	0.28	0.02
3W	a_6	0.85	0.12
B	a_7	6.80	1.12

Similarly, Speed equations (5.23) and (5.24) for vehicle types TW and HV assuming as standard vehicles types were established by using the same set of field data. High R^2 values indicate the strength of the models in predicting the speed.

$$V_{TW} = 50 * n_{TW} + 2.82 * \left[\frac{A_{TW}}{A_{CS}} * n_{CS} * V_{CS} \right] + 3.82 * \left[\frac{A_{TW}}{A_{CB}} * n_{CB} * V_{CB} \right] + 3.43 * \left[\frac{A_{TW}}{A_{LCV}} * n_{LCV} * V_{LCV} \right]$$
$$+ 7.72 * \left[\frac{A_{TW}}{A_{HV}} * n_{HV} * V_{HV} \right] + 2.39 * \left[\frac{A_{TW}}{A_{3W}} * n_{3W} * V_{3W} \right] + 10.54 * \left[\frac{A_{TW}}{A_{B}} * n_{B} * V_{B} \right] \qquad \boxed{R^2=0.78} \tag{5.23}$$

$$V_{HV} = 46 * n_{HV} + 0.17 * \left[\frac{A_{HV}}{A_{CS}} * n_{CS} * V_{CS} \right] + 0.22 * \left[\frac{A_{HV}}{A_{CB}} * n_{CB} * V_{CB} \right] + 0.38 * \left[\frac{A_{HV}}{A_{LCV}} * n_{LCV} * V_{LCV} \right]$$
$$+ 0.02 * \left[\frac{A_{HV}}{A_{TW}} * n_{TW} * V_{TW} \right] + 0.25 * \left[\frac{A_{HV}}{A_{3W}} * n_{3W} * V_{3W} \right] + 1.44 * \left[\frac{A_{HV}}{A_{B}} * n_{B} * V_{B} \right] \qquad \boxed{R^2=0.76} \tag{5.24}$$

5.9.3.2 Validation of MNLR speed models

Validation of the MNLR speed models was performed using another set of field data obtained from Section-II. The average speed of vehicle types observed on the field section was used validate the values obtained from the models. First, estimated average speeds of CS were compared with the field observed values at varying compositions and volume levels. The two average speeds were plotted against the 45° line chart, as shown in Figure 5.18. The test of significance was performed for comparison and the p-value was obtained which confirms that there is no difference between observed and estimated speeds.

Similarly, Equation 5.20 and Equation 5.21 were also validated using the same set of field data. The two average speeds of TW and HV, when considered as standard vehicle types were plotted against the 45° line chart as shown in Figure 5.19 and Figure 5.20, respectively. The test of significance was also performed which also confirms that there is no significant difference between observed and estimated speeds at 5 % level of significance.

From the Figure 5.18 to 5.20, it is observed that P-value shown for Vehicle type TW is less than CS and HV. This is because of Altman and Bland validation method was used to calculate p-value based on mean differences of points. In validation plots of CS and HV, the points are equally distributed with respect to mean, whereas in the validation plot of TW the points are skewed. Therefore, the P-value for vehicle type TW is found to be very less relatively.

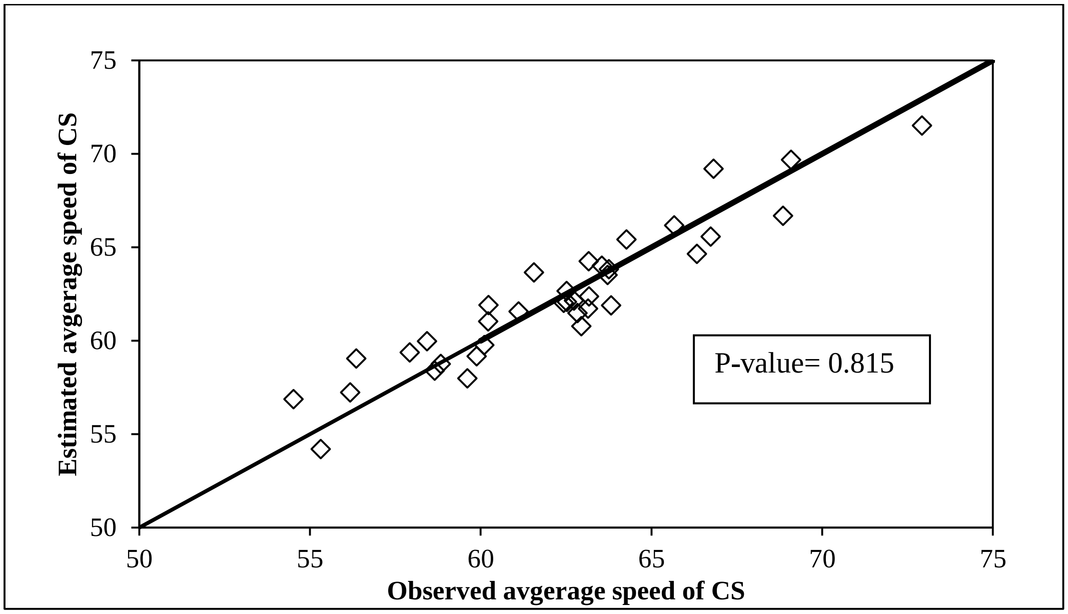


Figure 5.18 Validation of average speed of vehicle type CS

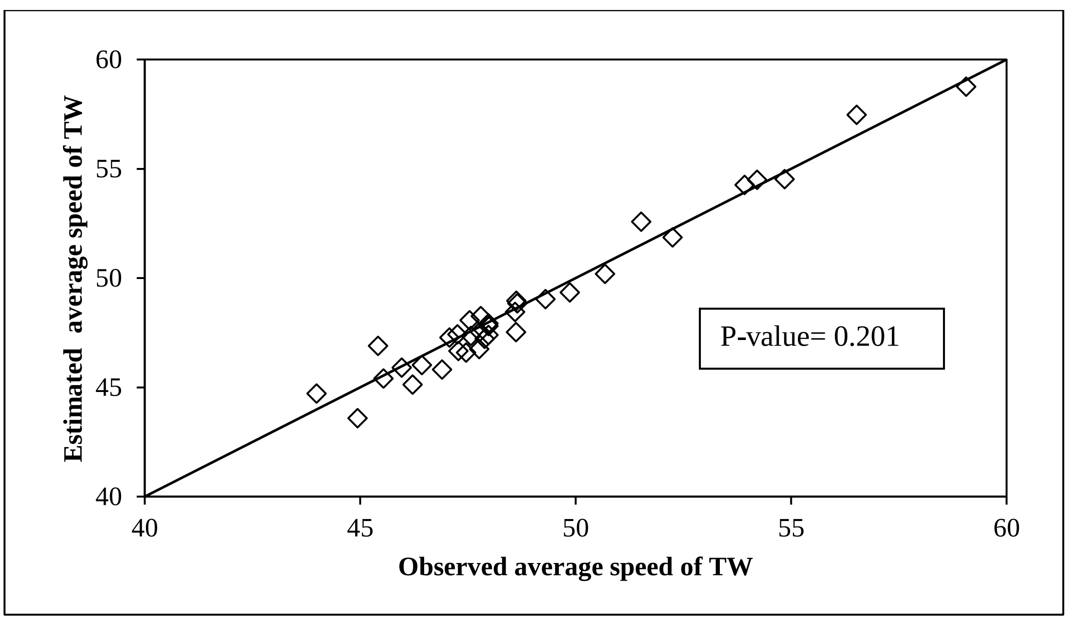


Figure 5.19 Validation of average speed of vehicle type TW

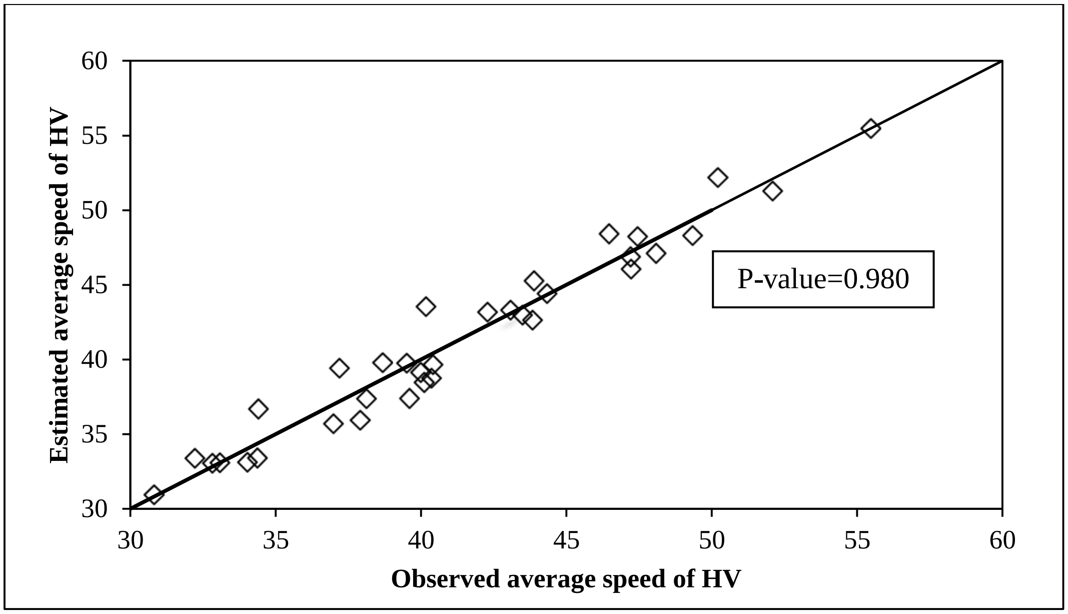


Figure 5.20 Validation of average speed of vehicle type HV

5.9.3.3 Comparison of estimated PCUs with dynamic PCUs

The PCU values estimated by the MNLR method were compared with the values obtained by the dynamic PCU method. The data collected at Section-I was used to obtain PCU values by the dynamic PCU method as well as by MNLR method. The comparison of mean PCU values from two different method is shown in Figure 5.21. The estimated PCUs from two different methods were also used to convert the traffic volume obtained at the field section at 5 min interval and results are shown in Figure 5.22. It may be seen that the traffic volume converted using MNLR method is relatively higher than the dynamic PCU values during the interval when the proportion of vehicle types LCV, HCV and B are higher in traffic composition. The total volume obtained by the MNLR method is almost on par with the total volume estimated using PCU suggested by IRC 64-1990.

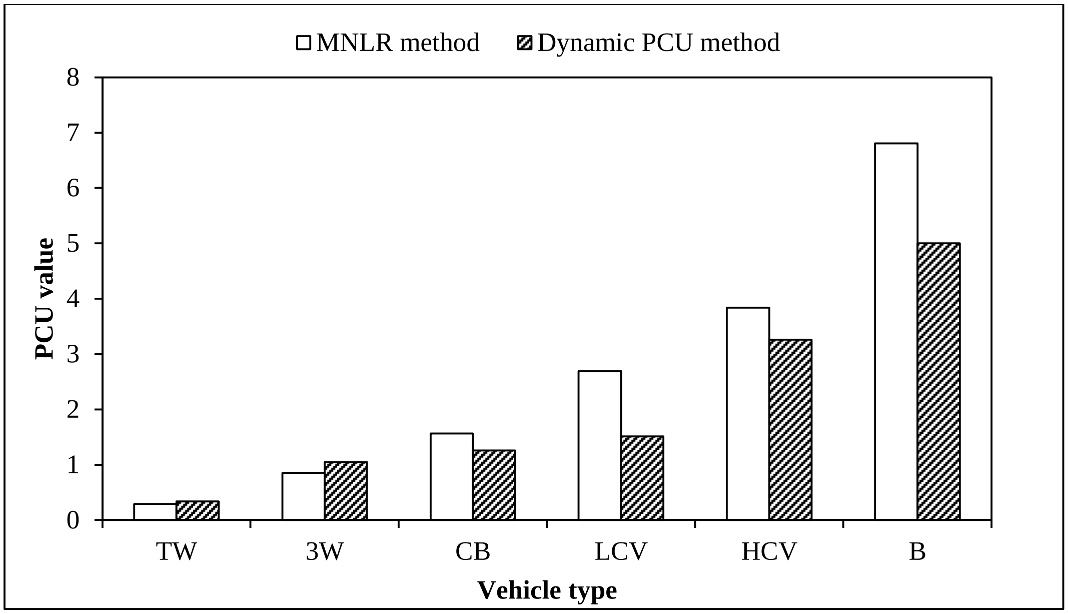


Figure 5.21 PCU estimated using MNLR method and dynamic method for subject vehicle types

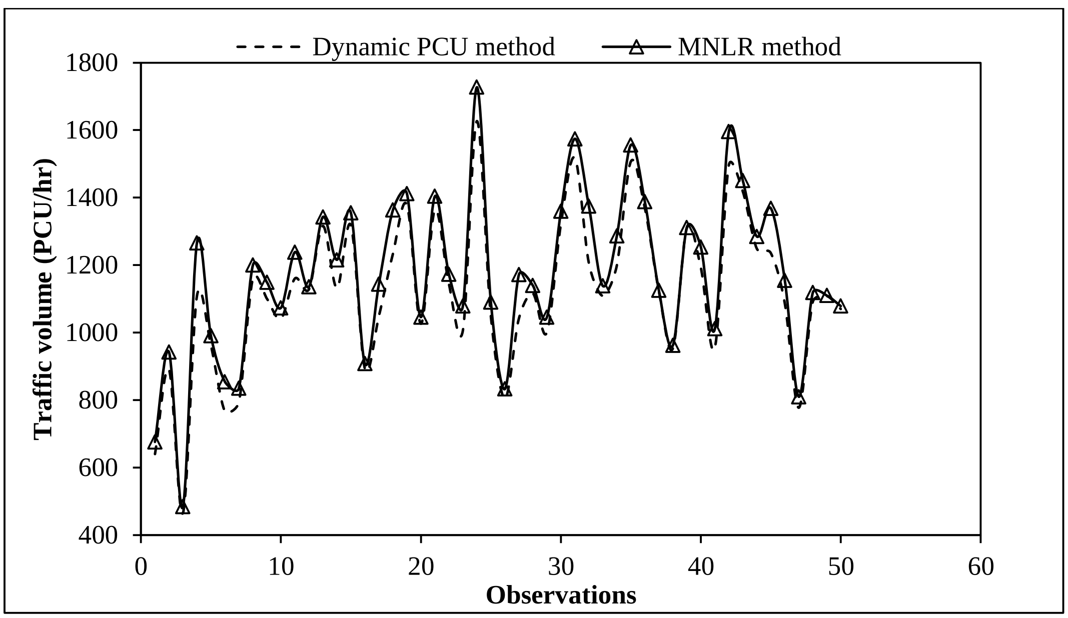


Figure 5.22 Comparison of converted traffic volume using two methods

5.9.3.4 Estimation of PCUs on six-lane divided highway

The multiple non-linear regression approach can be used for estimating the equivalency units on six-lane and eight-lane divided highways as well. The MNLR method to find PCU of vehicle types was also applied with the field data collected on six-lane divided highway section. The equivalency units of different vehicle types at Section-VII are shown in Table 5.20. The obtained equivalency units are realistic and logical values.

Table 5.20 Equivalency units of different vehicles at Section-VII

Vehicle Type	PCU	TwU	HVU
CS	1.000	2.624	0.205
CB	1.660	3.246	0.504
TW	0.342	1.000	0.086
LCV	2.239	3.245	0.908
3W	1.221	2.126	0.227
HV	3.828	7.413	1.000
BUS	6.181	10.246	2.088
MAV	7.862	15.346	2.362

* TwU-Two wheeler unit, HVU- Heavy vehicle unit

5.10 Speed-flow relationship

One of the important aspects in traffic flow research is establishment of speed-flow relationships. On highways, traffic flow is varied with the time of the day and vehicles can move with free speeds without hindrance when traffic volume is less. As flow increases, vehicles cannot sustain their free speeds due to interaction from other vehicles in the traffic stream. This phenomenon leads to reduction in average speed of the vehicles and it further leads to congestion when traffic demand exceeds capacity (Velmurugan et al., 2010).

The phenomenon is well understood by developing speed volume relationship. Field data collected at Sections II and IX was used to developed speed-volume relationship to understand the traffic flow behavior. A total of 6 hours of video recording covering peak and lean traffic flows was done under fine and dry weather conditions. Traffic volume and speed data extracted at 5 min interval were used to establish relationship. The classified traffic volume was converted into PCU/hr by dynamic PCU method and hourly volume was obtained. The stream speed was also obtained as weighted mean speed by considering the space mean speed of vehicle types observed during the same set of interval. Finally, relationship was establish between traffic volume and mean stream speed and is shown in Figure 5.23 and 5.24 for Sections II and Section IX respectively. .

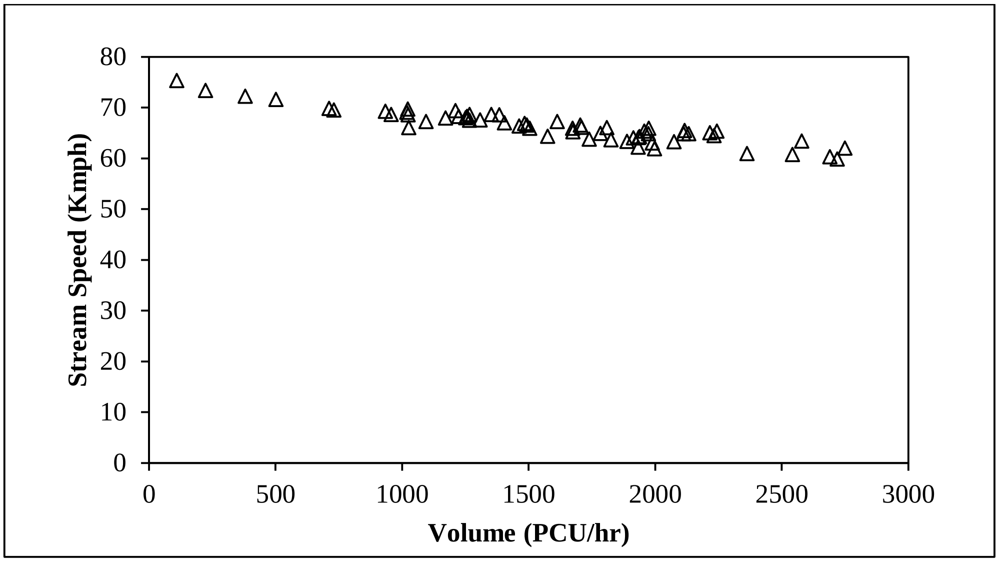


Figure 5.23 Speed - flow curve on section-II

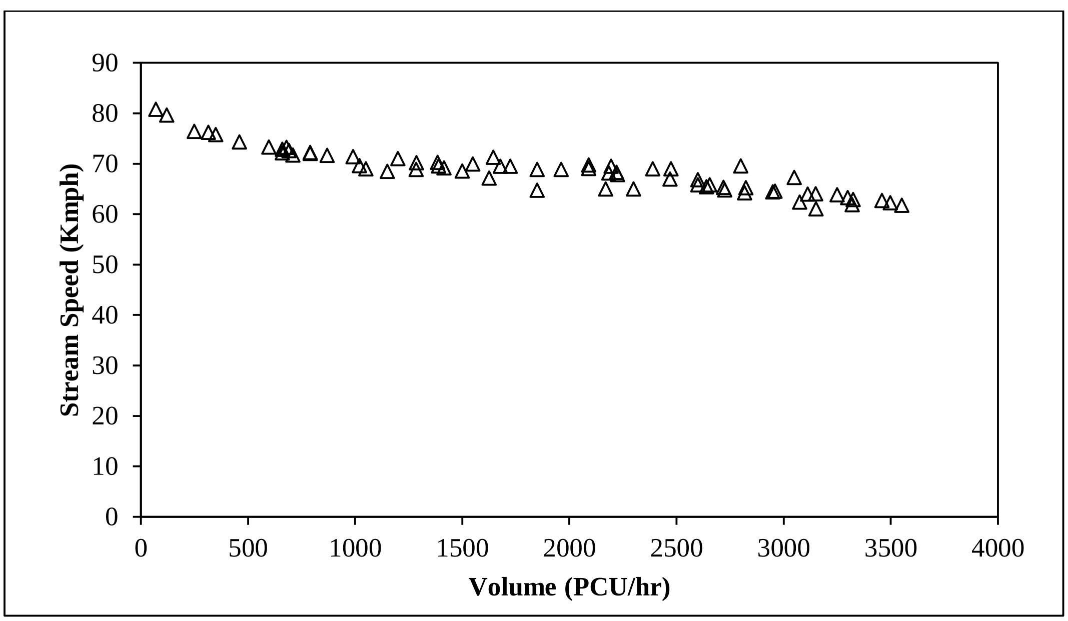


Figure 5.24 Speed - flow curve on section-IX

5.11 Summary

Traffic flow characteristics have been discussed for different sections and values of performance measures have been evaluated in this chapter which will be required as an input for development PCU models and simulation analysis.

The PCUs of each vehicle type as estimated in the study by applying different methods available in the literature showed variation among them. The results obtained by dynamic PCU method are found to be logical and more realistic. Comparison of PCU of different vehicle types on highway sections with paved and earthen shoulders reveals that PCUs are changing with the type of shoulders as higher value obtained on paved shoulder than earthen shoulder. Modification to Dynamic PCU method was attempted by adding the time headway factor which found that the PCU based on modified method may also be used under low to medium traffic volumes. However, modified approach used for PCU estimation in present study suggests relatively higher values than those obtained from dynamic PCU method mainly for large size vehicle types.

The new approach i.e. multiple non-linear regression method was developed in the study to estimate more accurate values of equivalency units of standard vehicle types especially under highly heterogeneous traffic conditions. It is confirmed from the results, MNLR method is more realistic to estimate PCU values in compared to any other method available in the literature. The accuracy of the proposed method was also checked by validating it on other highways sections. The validation of speed model used for finding PCU has confirmed that the there was no significant difference between the estimated and observed speeds when compared at 5% level of significance. Hence, developed models may be used for estimating the equivalency units of subject vehicles at a given volume and vehicle composition.

Chapter 6
SIMULATION OF TRAFFIC FLOW BEHAVIOUR

6.1 General

Microscopic traffic simulation models are very useful tools for performing various kinds of operational and behavioral analysis. Now-a-days, application of traffic simulation models in the area of transportation engineering and research is very popular and being utilized for modeling the complex traffic flow operations worldwide. VISSIM is multi-purpose traffic simulation model which can be used at both microscopic and macroscopic levels. VISSIM is based on car-following behavioral model which contains number of driving behavior parameters. Any change in parameter values may cause a substantial change in simulated output. The present study employs VISSIM software to generate mixed traffic flow data for analysis of complex traffic flow behavior. The chapter describes the important driving behavior model parameters as incorporated in VISSIM and the procedure to perform calibration and validation based on the data obtained from field. Further, simulation of mixed traffic flow was performed under different scenarios for analysis of traffic flow behavior which could not be observed from field data.

6.2 Driver behaviour in VISSIM

Microscopic traffic flow simulation model VISSIM contains two modified models developed by Wiedemann on the basis of different car-following algorithms. In car-following behavior, longitudinal movement of vehicles is influenced by the vehicles within a lane or in adjacent lanes which can be examined or modeled mathematically. The VISSIM incorporates the psycho-physical behavior based model for drivers, developed by the continuous work conducted by Wiedemann (1974-1992). This is also known as Action Point (AP) model and is based on the assumption that a driver of a vehicle performs an action when a threshold is reached to its boundary, expressed as the function of speed differences and distances between the vehicles. The basic premise behind this model is that the driver of a fast moving vehicle will start to decelerate, as or when he/she observes the individual perception threshold with respect to leading moving vehicle. The fundamental concept of the model is formed on assumptions that the driver of a vehicle in VISSIM can be under one of the four driving modes; (1) Free driving, (2) Approaching, (3) Following and, (4) Breaking. These four driving behavior mode represent a psycho-physical car-following model that incorporates changes in conditions of driver from free-flow to congested flow conditions.

6.3 Behaviour models and default parameters

Traffic simulation models contain numerous variables to define and replicate traffic control operations, traffic flow characteristics, and driver behaviour. VISSIM simulation model contains default values for each parameter, but also allows a range of user-applied values for each variable. Any change to these parameters should be justified through proper calibration and validation based on observed data. In other words, a change in the variables should be justified and defensible (Ahmed, 2005).

6.3.1 Car-following models

VISSIM incorporates the two different sets of psycho-physical behavior based car-following models. The models follow separate rules and algorithms to execute the behavior of drivers generated in the network. The car following driver behavior models provided in VISSIM consists of multiple numbers of parameters related to the speed and acceleration control of a vehicle. The car-following models used in VISSIM with their default values of parameters are briefly described below.

- **Wiedemann 74 Model**

The initial algorithm was developed by Wiedemann (1974) to present a typical car-following behavior of vehicles on urban roads. This model consists of three desirable parameters used to calculate safe distance between the vehicles; Average standstill distance (ax); Additive part of safety distance (bx_add) and Multiplicative part of safety distance (bx_mult). The model computes the safety distance (d) between the two vehicles using the equation (6.1).

$$d = ax + bx \qquad (6.1)$$

The equation provides safety distance (m) between vehicles as the sum of average standstill distance ax (m) and the distance d (m) maintained by the driver while following any vehicle at speed *v* (m/s). The bx (m) is calculated by using the equation (6.2).

$$bx = (bx\_add + bx\_multi \times z) \times v \qquad (6.2)$$

Where, z is a value of range [0, 1] which is normally distributed around 0.5 with standard deviation of 0.15. The parameters bx_add and bx_multi are the parts of desired safety distance that are observed to affect the simulation outputs. These parameters values are unit less, and the model has default values of these parameters as 2.0 and 3.0 respectively.

- **Wiedemann 99 Model**

The Wiedemann 99 model is the more complex representation of car-following behavior of drivers as it contains ten different parameters related to action or reaction of driver according to perceived traffic situations. This model is a modified version of Wiedemann 74 model, and was developed to represent behavior of drivers on freeways, interurban highways and expressways. CC0 and CC1 are two parameters used in calculation of safety distance dx (m) maintained by drivers at speed V (m/s). It can be expressed by equation (6.3).

$$dx = CC0 + CC1 \times V \qquad (6.3)$$

Where, CC0 is standstill distance (m); CC1 is time headway (s). The distance dx represents the headway a driver wishes to maintain at the significant volume level. It is the reciprocal of roadway capacity and it reduces with the increase in desired safety distance of vehicles. Moreover, parameters that govern safety distance are found to have influence on traffic flow behavior and affect the individual characteristics of drivers as well. Wiedemann 99 model contain ten different car-following behavior parameters termed as CC-parameters (CC0 to CC9). Each parameter is related to the behavior of a driver and its interaction among the vehicle population. The parameters defined in car-following model are able to describe four modes of driving behavior. The CC parameters given in car-following model are very intrinsic to static and dynamic characteristics of vehicles plying on a roadway. The parameters used to present car-following behavior of drivers are provided with their default values as shown in Table 6.1.

Table 6.1 Wiedemann 99 model parameters listed with default values in VISSIM

S. no.	Notation	Name of parameter	Default value
1.	CC0	Standstill distance	1.5 m
2.	CC1	Time headway	0.9 s
3.	CC2	Following variation	4 m
4.	CC3	Threshold for entering 'following'	-8*
5.	CC4	Negative following threshold	-0.35*
6.	CC5	Positive following threshold	0.35*
7.	CC6	Speed dependency of oscillation	11.44*
8.	CC7	Oscillation acceleration	0.25 m/s^2
9.	CC8	Standstill acceleration	3.5 m/s^2
10.	CC9	Acceleration at 80 km/h	1.5 m/s^2

* Values are unit less

6.3.2 Lane change model

The lane change behavior of vehicles has different operational aspects from car-following behavior. The lane change behavior in VISSIM can be defined for two conditions, necessary lane change condition and free-lane selection. If the driver of a vehicle needs to approach any link or connector within a network in order to follow pre-decided routing, then necessary lane change is required. The desired safety distance depends on vehicle speed moving ahead and the speed of vehicle that wants to change the lane. The analyst can select any one lane change behavior depending upon the objectives of analysis. The following parameters control lane-changing behavior of a vehicle.

- Maximum deceleration and adopted deceleration of own and trailing vehicle
- Waiting time before diffusion
- Minimum headway (front/rear)
- safety distance reduction factor
- Maximum deceleration for cooperative braking

6.3.3 Lateral behaviour model

VISSIM model incorporates parameters for controlling the lateral distance between the vehicles. If enough lateral distance is available then a faster vehicle can overtake other vehicles. In VISSIM, single vehicle can occupy the entire lane width along a highway length by default. Therefore, lateral behavior of vehicles can be modified in VISSIM by checking the parameter settings for different vehicle types. The model contains the following parameters as given below.

- Lateral position at free-flow speed
- Minimum lateral distance of vehicle (at the speed of 0 km/h and 50 km/h)
- Observe vehicles on next or adjacent lane for taking desired lateral position
- Overtaking on same lane and type of vehicle to be overtaken either from right side or from left

6.4 Other simulation parameters

There are other important parameters in VISSIM to run simulation process more accurately and efficiently. The following simulation parameters are discussed below.

(a) **Traffic regulations**- Traffic regulation must be specific before running simulation; VISSIM has both right hand side and left hand side traffic rule.

(b) **Period of simulation**- This is the time for which simulation need to run for specific purpose. It may me one hour or more based on the traffic measurement and the geometry of the location.

(c) **Simulation resolution**- It controls how many number of times the position of vehicle will be updated. It ranges from 1 to 10 (time steps /simulation seconds). The simulation speed is inversely proportional to the number of time steps and therefore, use of an appropriate value for simulation resolution is very important.

(d) **Simulation speed**- The number of simulation seconds are required to complete a real time second. However, change in the simulation speed will not affect the overall result of the simulation.

(e) **Simulation random seed number**- VISSIM is stochastic simulation model; it generates vehicles based on random seed numbers. These numbers are assigned to follow a certain vehicle arrival distribution. Therefore, multiple simulations runs using different seed numbers may be required to increase the accuracy of the simulation results.

6.5 Network coding and verification

6.5.1 Link properties and link behaviour type

The basic element of a VISSIM network is called as 'Link'. A link represents a single or multiple-lane roadway segment that has one specified direction of flow. Therefore, a two-way road section can be constructed using two links with opposite direction of flow. Assigning the link attributes is an important task for specifying roadway and geometric conditions to VISSIM. The properties about the link such as, name, number of lanes, lane width, link length etc., can be given to the created link. The link behavior type is required to depict the nature of a link on which the traffic is to be generated. Moreover, for the particular type of link behavior, specific driver behavior models and parameters can be chosen. VISSIM also provides some default link behavior types for simulating different traffic flow situations such as urban (motorized), freeway, pedestrian area and bicycle track. In the present study, custom link behavior type has been created and named as Indian condition which comprises the driving behavior signifying the Indian mixed traffic condition. The link behavior type for Indian condition is shown in Figure 6.1**.**

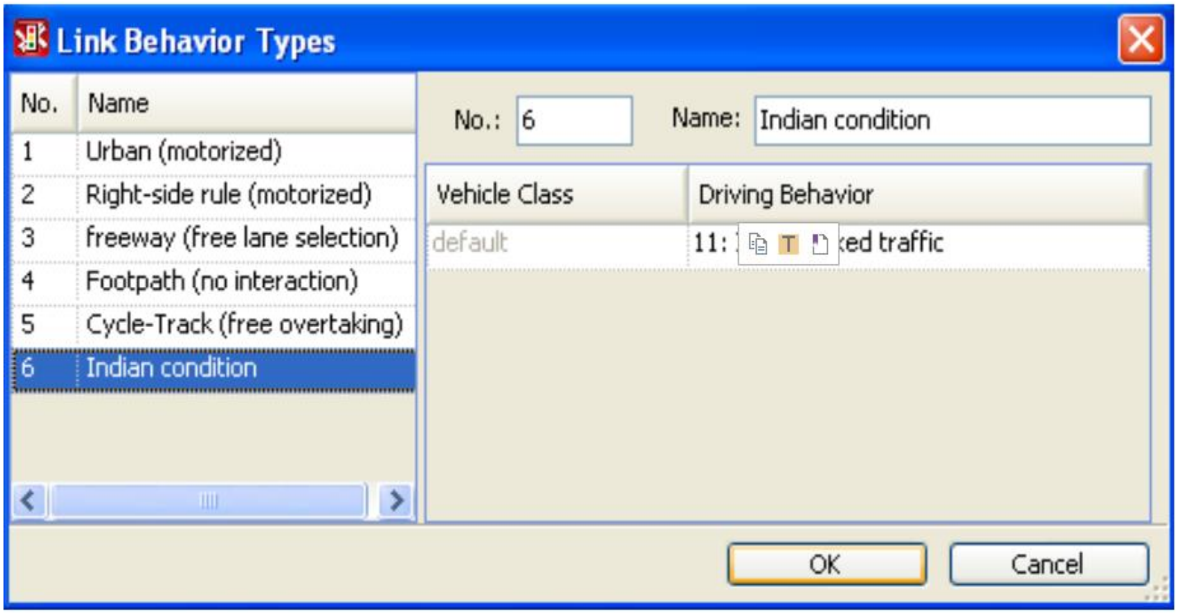


Figure 6.1 Link behavior assigned to VISSIM

6.5.2 Data input to VISSIM

To prepare the base model network, field data collected at the Section-V is given as input to VISSIM. Following are the inputs required to run simulation.

(a) Vehicle data

Vehicle type and vehicle class data as observed from the field is given as an input for the simulation run. Vehicle types includes group of vehicles with similar static and dynamic characteristics. Vehicle class represents a logical container for one or more previously defined vehicle types. As per field data, six vehicle types were identified and provided as input for simulation run. The procedure to assign vehicle type in VISSIM is shown in the Figure 6.2.

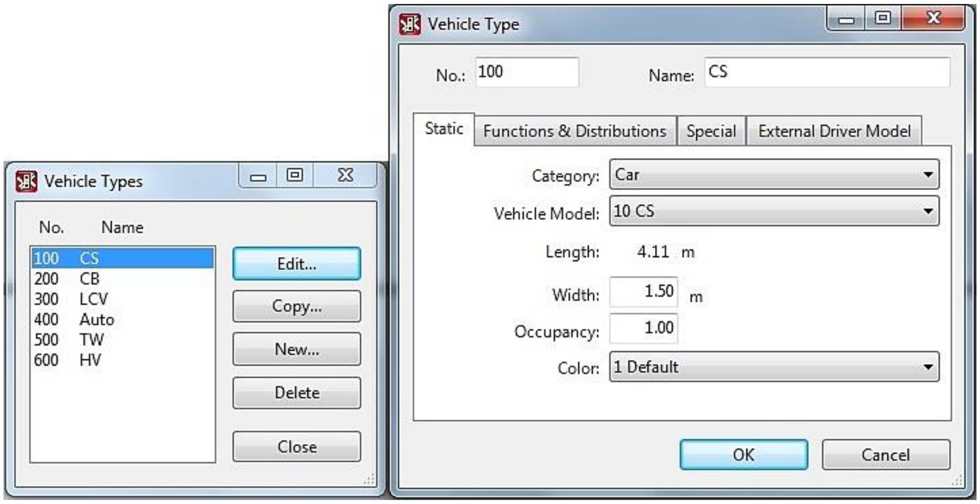


Figure 6.2 Vehicle type data input in VISSIM

(b) Desired Speed distribution

Speed distribution profile is a very important input to be given to run simulation model.. Speed profile is created in VISSIM using percentile speed, such as 15th, 50th and 85th percentile values as estimated from the field data. Speed distribution profile for each vehicle type is given individually. The procedure for desired speed distribution profile created in VISSIM for vehicle type CS is shown in Figure 6.3.

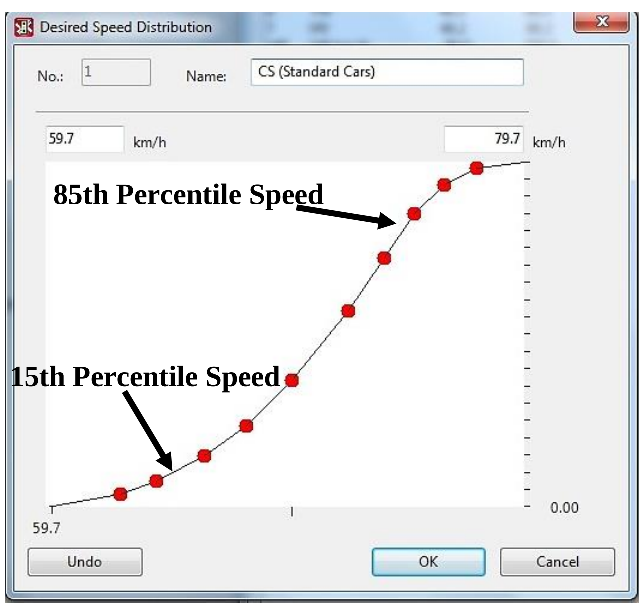


Figure 6.3 Desired speed distribution profile of vehicle type CS in VISSIM

(c) Traffic volume and vehicle composition

Traffic volume is required as essential input to VISSIM model as it simulates the network based on vehicle input provided to assigned link in terms of number of vehicle per hour. The generation of vehicles on specified link is random and follows Poisson distribution. The time period (in sec.) is also need to be specified for an assigned volume. The volumes assigned to a particular link must contain vehicle composition along with their desired speed distribution which is assigned as relative flow. The relative flow in fraction defines the proportional share of a vehicle type in the traffic stream. Vehicle composition assigned to VISSIM for a given volume is shown in Figure 6.4.

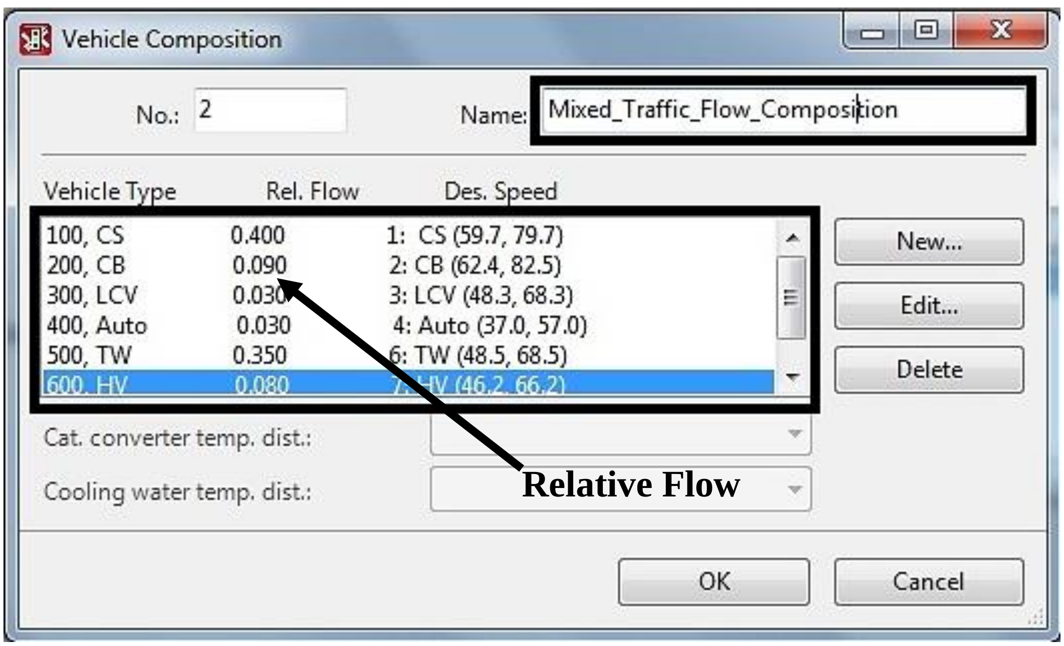


Figure 6.4 Vehicle compositions in VISSIM

6.5.3 Simulation output and its analysis

VISSIM simulation runs were performed for 3 hr extracted from the output files. The traffic volume and speed was measured from the travel time output file saved while creation of base simulation network. Travel time out file records the time taken by each vehicle type on the travel time section (60 m) assigned at appropriate distance away from the point of vehicle generation (about 500 m). Simulated data such as vehicle count, vehicle type, travel time and simulation time etc was obtained from the travel time output file. The analysis of result obtained from the base simulation run is described below.

(a) Speed data

Speed was measured for individual vehicle type by using travel time data. Various speed parameters were estimated from descriptive statistics analysis and their values are given in Table 6.2. Travel time section of same length which is given in the field is assigned to given link to collect speed of individual vehicle types.

Table 6.2 Speed parameters

Speed parameter	CS	CB	3W	LCV	HV	TW
Mean (Kmph)	62.61	62.13	43.76	47.52	48.64	50.17
Standard Deviation (Kmph)	5.75	7.07	2.71	3.08	2.59	2.77
Sample size	4701	992	320	347	942	4109
Minimum (kmph)	47.31	45.27	29.79	35.08	34.83	35.42
Maximum (kmph)	72.80	74.58	48.25	54.25	54.15	55.29
15th Percentile(Kmph)	56.40	52.63	41.18	44.54	45.88	47.37
50th Percentile(Kmph)	63.15	63.30	43.96	47.52	48.44	50.16
85th Percentile(Kmph)	68.29	69.47	46.61	50.14	51.72	53.43

The simulated space mean speed estimated for each vehicle types were compared with the field observed mean speed. Figure 6.5 shows the comparison of the average speed of field and simulated data. It is found that the field and simulation average speed of all vehicle types is almost same.

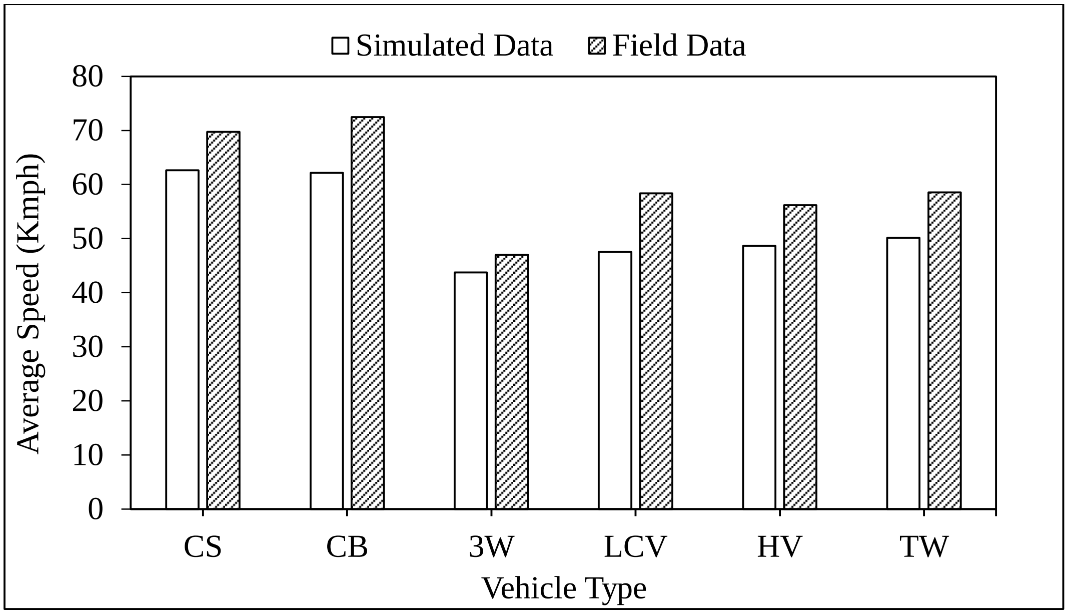


Figure 6.5 Comparison of field and simulation average speed of vehicle types

(b) Vehicle arrival pattern

Vehicle arrival data was extracted from simulation output file and their frequencies of arrival were obtained at 20 second interval. From the simulated frequency data, probabilities of vehicle arrived in given intervals were estimated based on Poisson distribution function to find expected values of frequency. The simulated frequencies are shown with field observed frequencies in the Figure 6.6. Further, simulated and field observed frequencies were compared by applying Chi-square test to confirm whether simulated and field observed frequencies are same or different. The procedure and result of statistical test is described below.

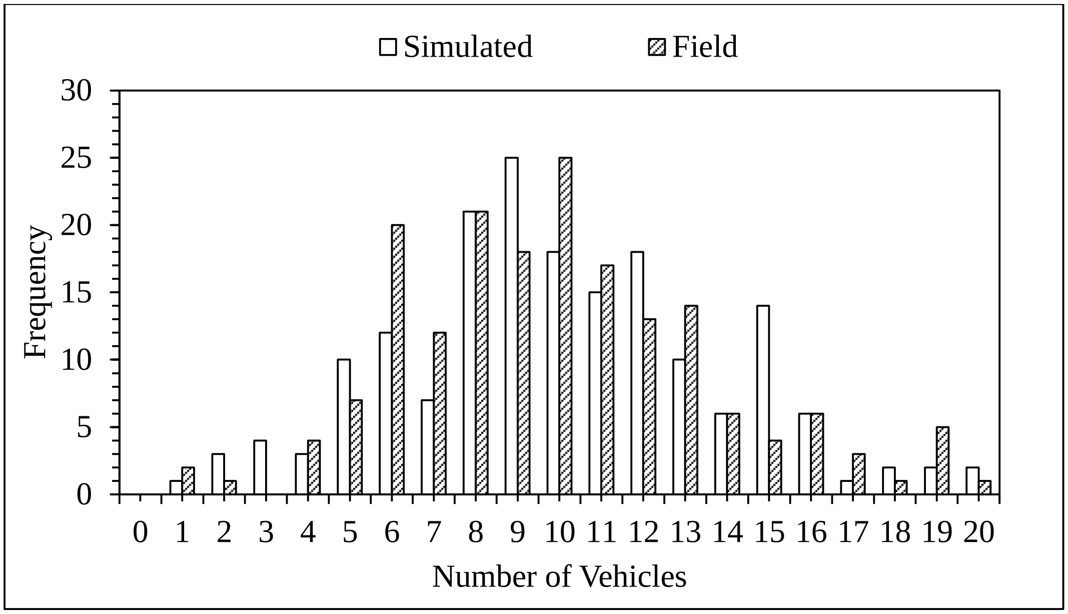


Figure 6.6 Comparison of simulated and field arrival frequencies

For the simulated data, the value $\chi^2(calculated)$ is 32.2388

Degree of Freedom (υ) = 12 – 2 = 10.

At, υ = 10 and α = 5% $\chi^2(tabulated) = 18.307$.

$\chi^2(calculated) > \chi^2(tabulated)$, therefore, null hypothesis H_0 is rejected. Hence, it is confirmed that the simulated frequencies done not match with field observed frequency of vehicle arrival when tested at 5% significance level.

6.5.4 Traffic volume at different RSN

VISSIM consists of some default values of parameter such as Random seed number which directly affect the vehicle arrival characteristics. Default seed number in VISSIM is 42, for this study random seed numbers taken as 40, 41, 42, 43, 44 and 45. Check for volume is performed for seed number 40, 41, 42 43, 44, 45. The volume obtained after each simulation is compared with the field volume and RMSE (Root mean square error) is calculated for each seed number is shown in Figure 6.7. The least error obtained for RSN is selected for further study.

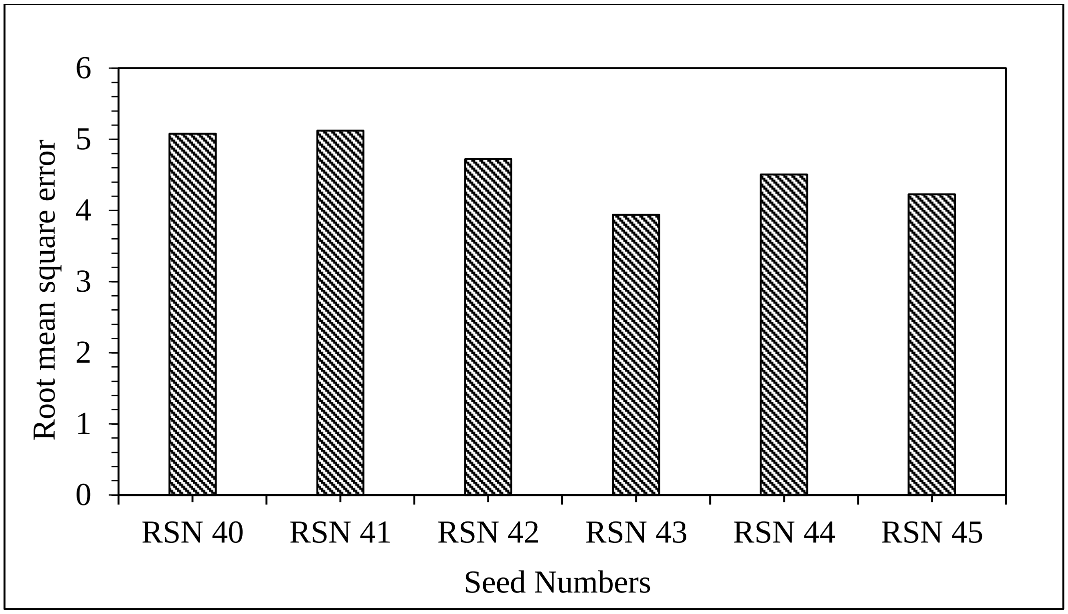


Figure 6.7 Root mean square error for total volume

6.5.5 Comparison of speeds

The spot speed data obtained from field is used to create the desired speed distribution profile in VISSIM. As with spot speed data, the speed distribution profile must be developed based on the off peak (free flow) speed. The simulated speed is then compared with the field speed (shown in Figure 6.8). The calculated error (given in Table 6.3) is which is showed that the VISSIM's simulated speed has relatively no difference from field speed. RMSE value at different seed numbers are estimated which provides clear idea that the RSN number have no effect on simulated average speed.

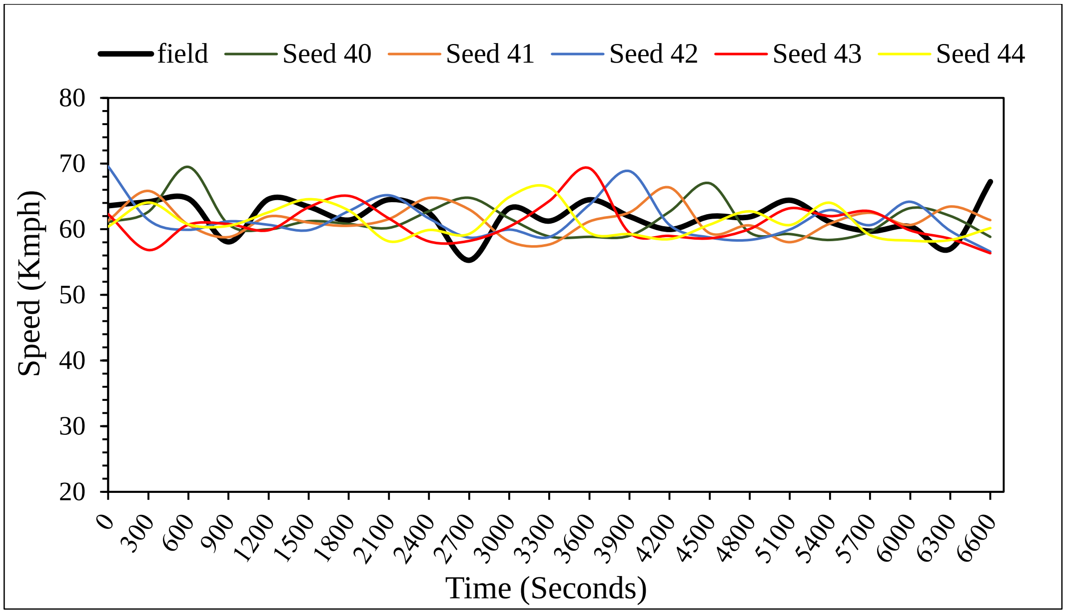


Figure 6.8 Variation in speed for different seed numbers

Table 6.3 RMSE and MAPE calculation for comparing speed data

RSN	RMSE	MAPE
40	4.16	5.43
41	3.81	4.94
42	3.98	4.83
43	3.86	4.83
44	3.27	4.17
45	4.03	4.98

6.5.6 Comparison of volume

The simulated traffic volume was also compared at different RSN to check whether RSN has any effect on simulated volume. The traffic volume data as observed in field was given as input from lower to higher range and simulation run was performed. Simulated volume obtained at different RSN were compared with each other as well as with field observed volume. The result of the simulated output at varying RSN is shown with the field observed values in the Figure 6.9.

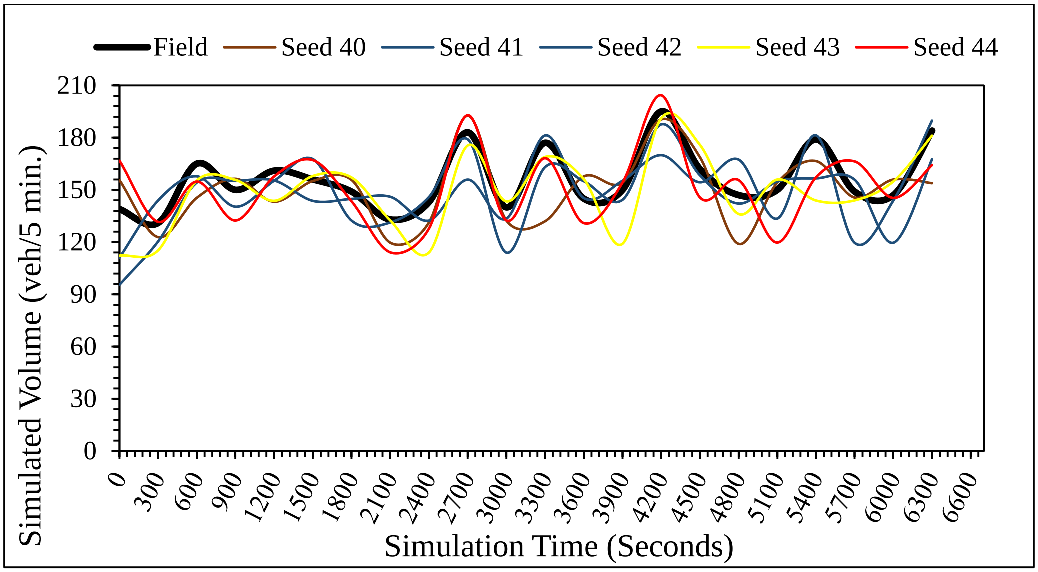


Figure 6.9 Variation in traffic volume for different seed numbers

Root mean square error and mean absolute percentage error were calculated and the result is shown in Table 6.4. The error estimated between field and simulated values at different RSN values is found between 7% to 8%. Hence, it indicates the difference between values is acceptable and no calibration is required related to traffic volume.

Table 6.4 MAPE and RMSE calculation for comparing volume

Seed no.	MAPE	RMSE
40	7.66	6.46
41	7.07	6.12
42	7.59	6.35
43	7.40	6.21
44	7.87	6.58
45	6.83	5.42

6.6 Test for calibration of driver behaviour model

The calibration test was performed on the driver behavior model parameters to check whether with the present setting of parameter values are able to replicate the field observed traffic flow behavior. The speed-volume relationship as obtained from field data was used for testing the need for calibration of driver behavior model parameters. The field data such as traffic volume, speed profile for each vehicle types and vehicle composition etc. were given as inputs and simulation run was performed for more than 7200 simulation sec. with lower to higher traffic volume (100 vph to 6000 vph). The driver behavior parameter values as given in the VISSIM model were remained default for the test run. The speed-volume relationship was developed after extracting speed and volume data at 5 min interval from the simulated outputs. Further, simulated speed-volume relation was compared with one which obtained from the field data. Figure 6.10 shows the comparison of two speed-volume relationships. It may be seen that the speed-volume relation developed from field data is clearly above and follows different trend than that obtained from simulation data. Hence, calibration is required to VISSIM model parameters to replicate same traffic flow behavior as it observed in the field data under the same range of traffic volume. The model parameters affecting traffic flow behavior are identified by altering their parameter values to match speed-volume relationships. Care is taken while changing parameter values based on the literature (Mehar et al, 2014). Literature confirms that some of the parameters are not affecting traffic flow behavior and maximum flow. However, those parameters were also taken into considerations to check their interdependency.

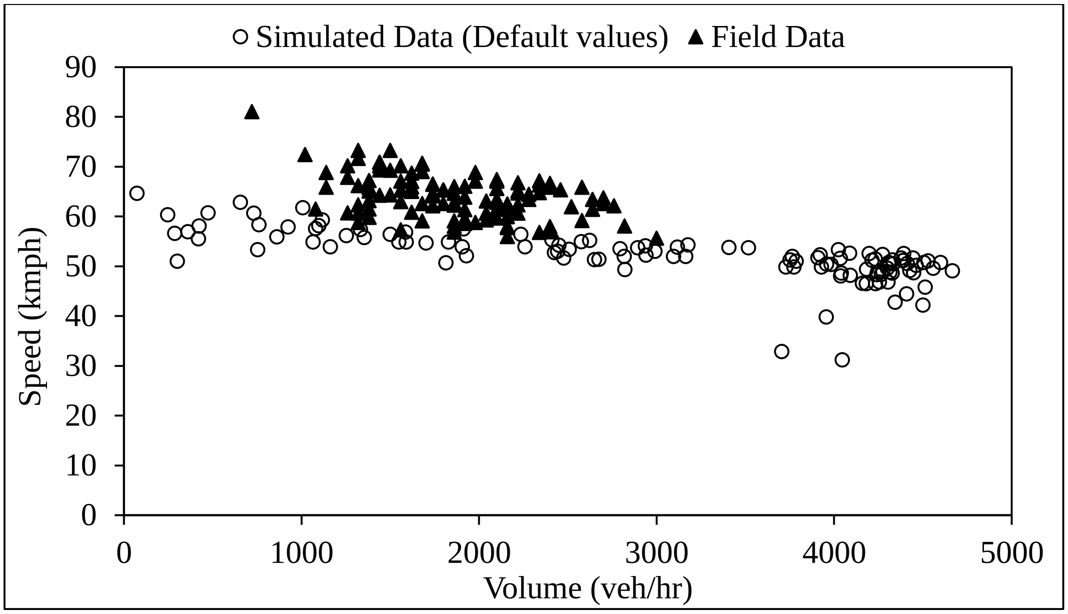


Figure 6.10 Comparison of speed-flow curves on four lane divided highway

6.7 Effect of model parameters on capacity

The driving behavior in VISSIM is based on the car following model and for present study Wiedemann 99 model was selected to perform simulation analysis. This model contains 10 different parameters called as CC parameters varying from CC1 to CC9 as described in earlier section. To calibrate the model parameters, simulation runs were performed to check the sensitivity of these parameters by making them in individual groups. The model parameter values were changed to observe the difference between field and simulated speed-volume relationships. The parameters CC0, CC1 and CC2 are varied independently and in combination to observe their sensitivity on capacity and on the shape of the speed-volume relation. The speed flow relationship developed for each simulation run with different pairs of CC1 and CC0 parameters is shown in Figure 6.11. Other parameters were remained constant during the simulation runs.

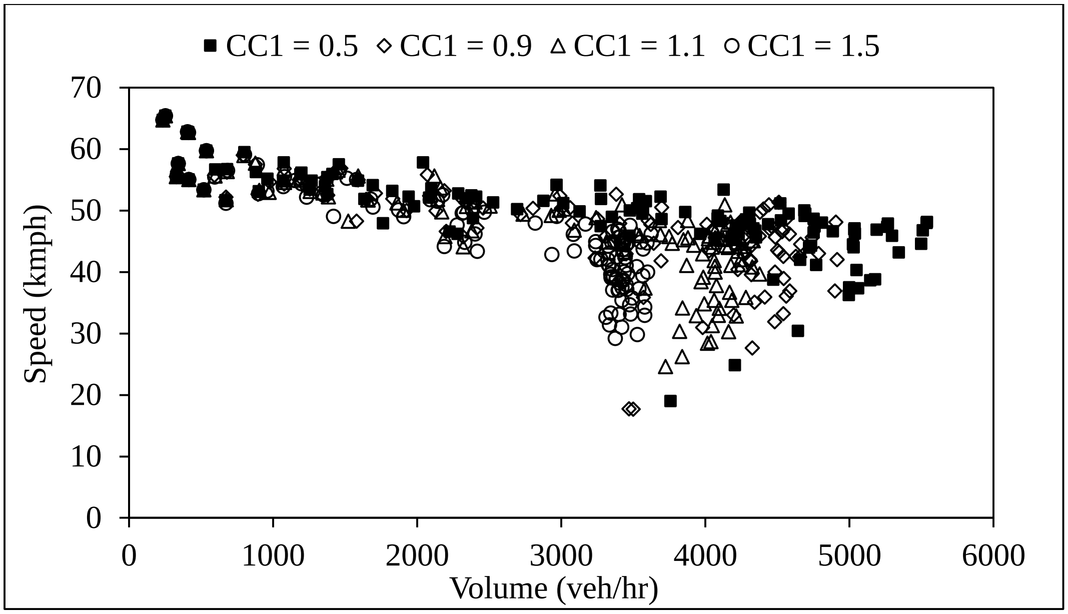


Figure 6.11 Effect of CC1 on speed-flow curve

The simulated capacity values were determined by observing the data cluster and the values are provided in the Table 6.5 for different combination of CC1 and CC0 parameters pairs.

Table 6.5 Simulated capacity (veh/hr) values due to CC0 and CC1

	Simulated capacity (veh/hr)			
	CC0 (m)			
CC1 (s)		0.5	1	1.5
	0.5	5647	5541	5170
	0.7	5374	5231	5025
	0.9	4894	4743	4628
	1.1	4468	4333	4241
	1.3	4034	3893	3825
	1.5	3639	3596	3516

The result shows that the change in capacity values was observed significantly at different values of CC1. However, change in capacity is found to be insignificant at different CC0 parameter values. Figure 6.12 and Figure 6.13 depict the variation in capacity due to CC0 and CC1 parameters.

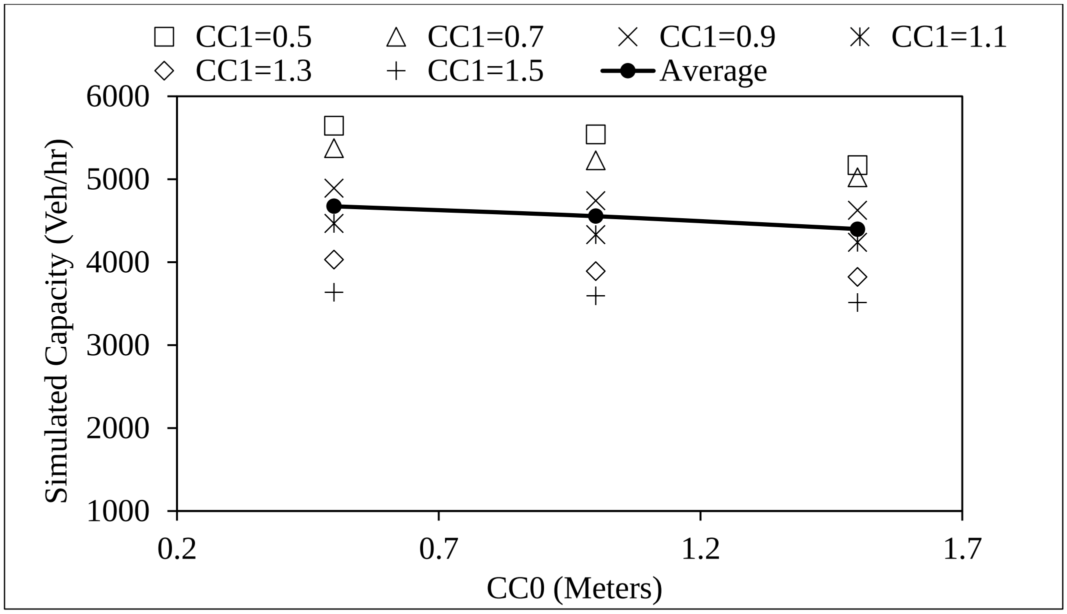


Figure 6.12 Simulated capacity at varying standstill distance

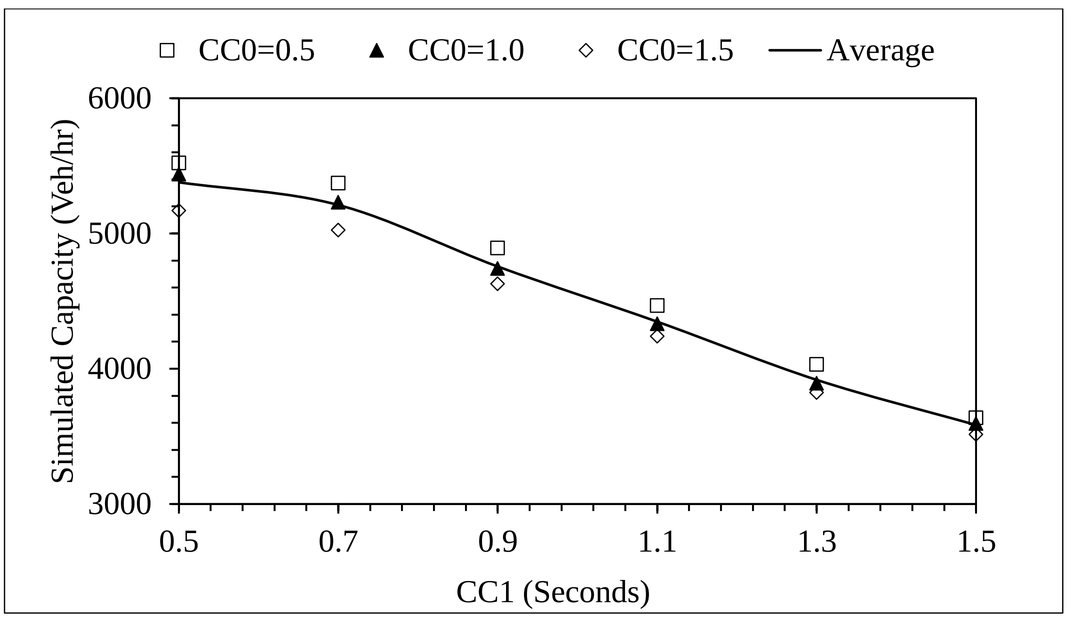


Figure 6.13 Simulated capacity values at varying time headway

The literature suggests the variation in following distance parameters has an effect on capacity. Therefore, the following distance variation i.e. CC2 parameter was also varied from 2 m to 10 m and simulation runs were performed under lower to higher volume conditions. The value of CC0 parameters was kept default during simulation runs while CC1 parameter was varied from lower to higher. The speed volume relations developed at varying CC2 parameter is shown in Figure 6.14. The capacity value determined from speed-volume relations were found to be varied at different CC2 parameter values. The simulated capacity values at varying CC2 are shown in Figure 6.15.

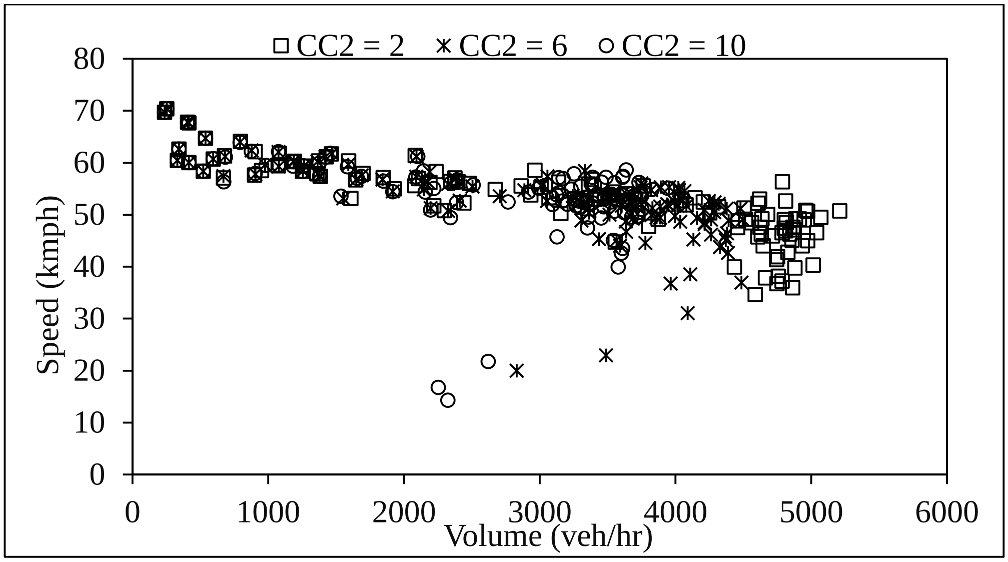


Figure 6.14 Simulated speed-flow relations at different CC2 values

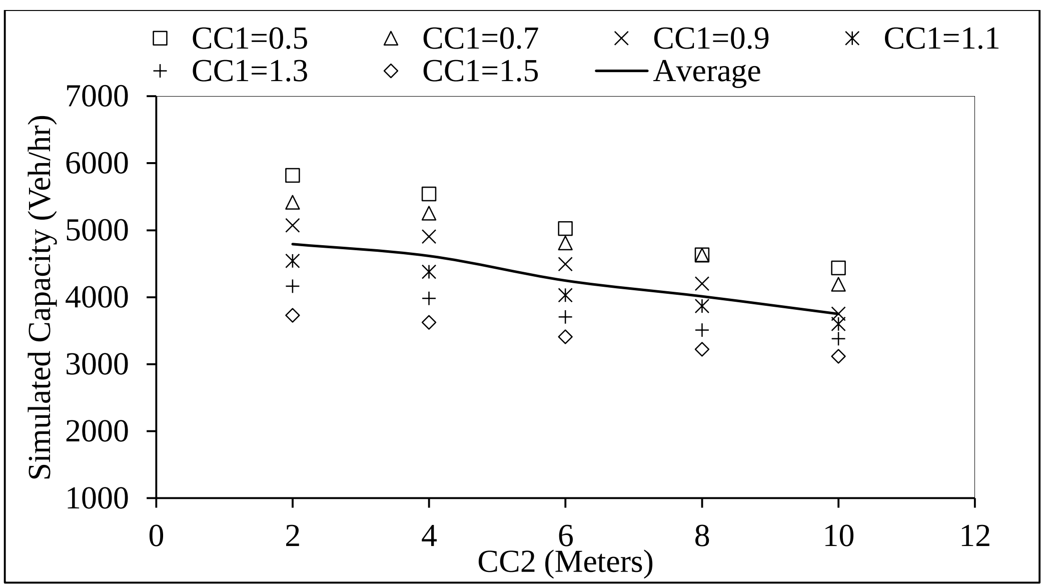


Figure 6.15 Simulated capacity at varying CC2

Figure 6.16 showing a consistent drop in capacity values due to change in CC1 values for different CC2 combinations.

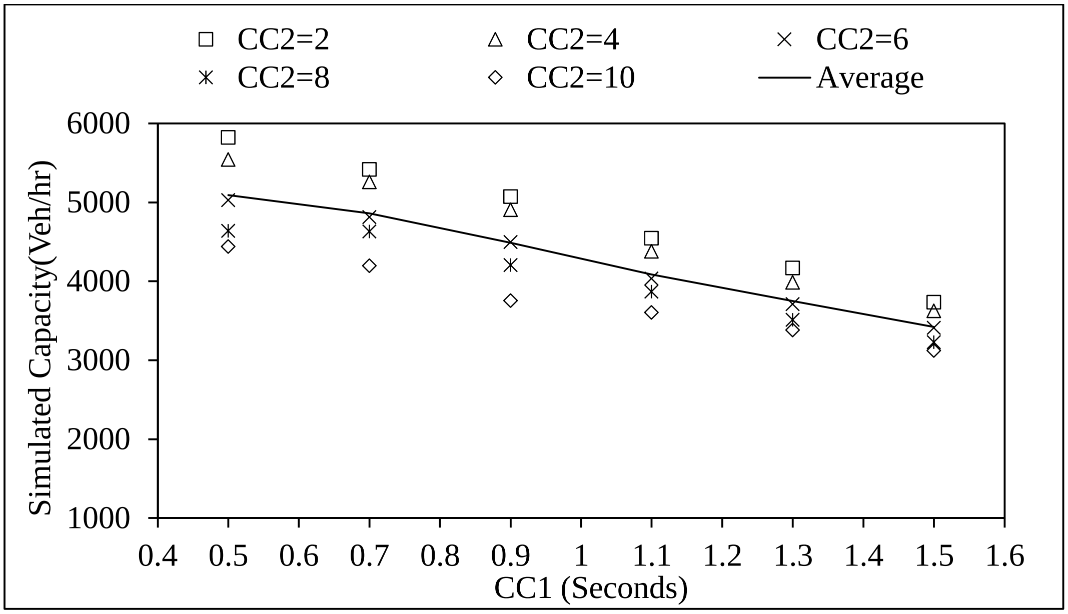


Figure 6.16 Variation in capacity due to CC1 **(**sec**.)** on different CC2 values

The parameters CC0, CC1 and CC2 are found to be influencing on traffic flow behavior and capacity of simulated section. Higher values of these parameters increase the spacing between the vehicles and making drivers more safety conscious. Hence, resultant capacity of simulated section reduces due to change in their driving behavior. It has been noticed that parameters are affecting capacity in combinations, therefore, a model has been proposed in Equation (6.4) to find capacity which includes the effect of these three parameters.

$$\text{Capacity} = 7034.2 - 183.6 \times CC0 - 1801.4 \times CC1 - 130.4 \times CC2 \qquad (6.4)$$

Equation 6.4 can be used as an objective function for the purpose of the optimization as left side variable i.e. capacity is measure of effectiveness. At default simulation run capacity may or may not be same but it can be our desired capacity value based on different parameter values. The upper bound and lower bound values of capacity may be estimated using Equation 6.4 by taking extreme values of these parameters as given in Table 6.6.

Table 6.6 Range and Interval of driver behavior parameters

Parameters	Range	Interval
CC0 (m)	0.5-1.5	0.5
CC1 (Sec.)	0.5-1.5	0.2
CC2 (m)	2-10	2

The capacity boundaries are defined to find optimized values of CC0, CC1 and CC2 parameters which can provide the target capacity or field capacity. The proposed equation (6.4) is having 3 unknown parameters which make it difficult to solve using conventional methods. To get the optimum value of these three parameters, constraints equations has been formed using the simulated data and ranges for the decision variables has been fixed for the case. The following constrained equations has been formed

$$6326.23 - 183.67 * CC0 - 1801.49 * CC1 <= 5333 \quad (6.5)$$

$$6850.51 - 1801.49 * CC1 - 130.41 * CC2 <= 5689 \quad (6.6)$$

$$5232.69 - 183.67 * CC0 - 130.41 * CC2 <= 4788 \quad (6.7)$$

The above equations and constrained functions can be written by simplex method of optimization, in which capacity function is objective function which is subjected to constrained equations and the type of the decision variables. Basic syntax of simplex form is given as follows

Objective = *f* (decision variables)

Subjected to constraints $(X_1, X_2, \ldots\ldots\ldots X_n) \left\{ \begin{matrix} \leq \\ = \\ \geq \end{matrix} \right\} d_1$

Where, $X_i \geq 0$

X_i = unrestricted in sign

$X_i \leq 0$

For the above syntax, decision variables are CC0, CC1 and CC2 and their ranges are also given in Table 6.6.

6.8 Estimation of model parameters

To solve the optimization problem, solver function is used in Microsoft (MS) excel and template is prepared for different values of decision variables such as CC0, CC1 and CC2. Solver is part of a suite of commands which is commonly known as what-if analysis tools. With Solver, it is easy to find an optimal (maximum or minimum) value for a formula in one cell called the objective cell subject to constraints, or limits, on the values of other formula cells on a worksheet. Template showing the solution to optimization problem is shown in Figure 6.17.

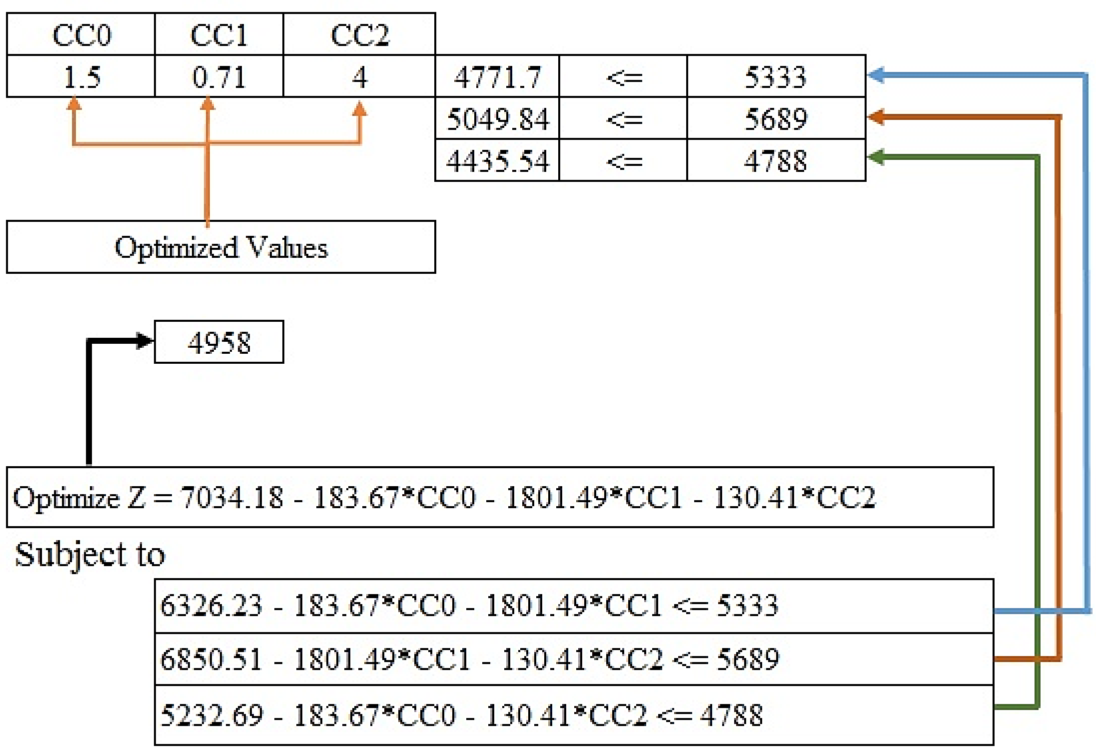


Figure 6.17 Process solving optimization problem using solver function in MS Excel

The optimized values are generating based on the target value and the given constrained equations. The same problem is modelled in solver function to solve for the optimum values of the decision variable i.e. parameters. Figure 6.18 is the user interface for the solver plug-in in excel, every equation has been defined accordingly. Solving method is used as Simplex Linear Programming problem because objective function is a linear constrained problem.

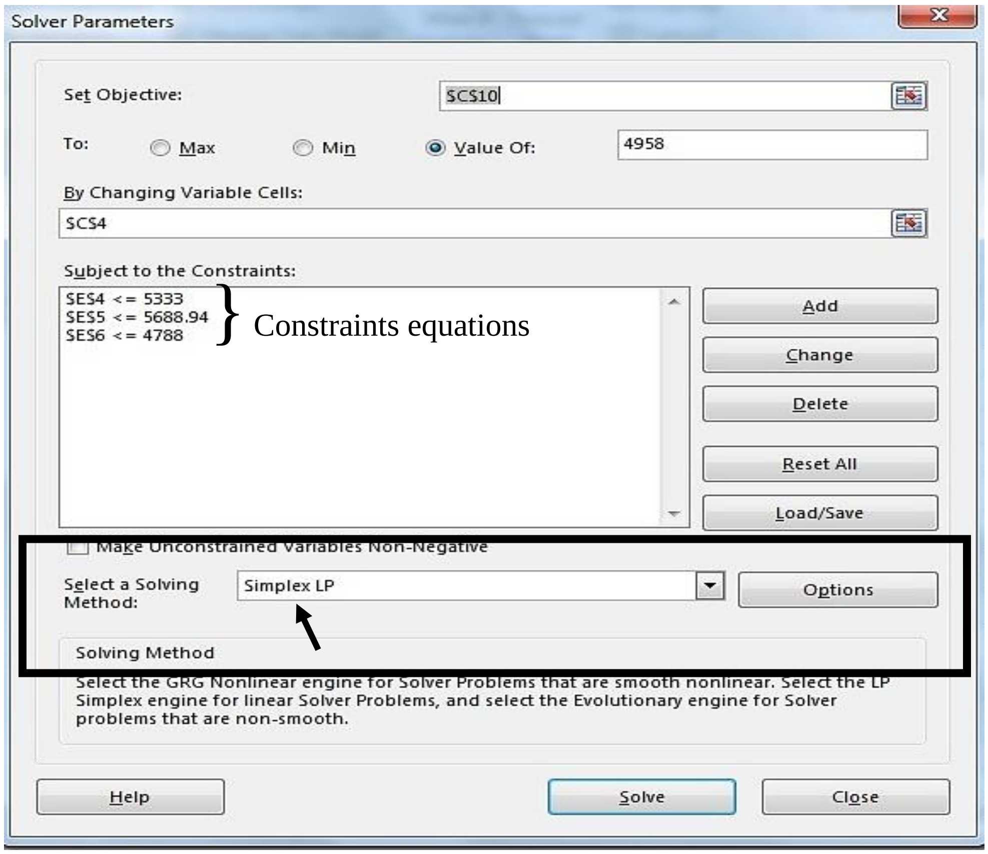


Figure 6.18 Solver function in MS Excel

Parameter values have been calculated using trial and error method and found that each parameter is sensitive to each other. For any fixed value of CC2, variation in CC0 and CC1 is plotted for different CC2 values. Similarly, for the fix value of CC0, variation in CC1 and CC2 is plotted. All the obtained plots are then merged to get the optimal point; the plot has been made on the 3-axis graph using excel. The plotted graphs are shown in Figure 6.19. X-axis represents CC1 (Standstill distance), Y-axis represents CC0 (headway time) and X-secondary axis representing CC2 (following distance variation). Data has been plotted considering 2-axis at a time by keeping 3rd one as a constant, and the intersection points of the lines have been identified as optimal points.

Two optimal points (i.e. 1.8, 0.53, 6 and 1.2, 0.74, 4) have been found and simulation run was performed with these inputs.

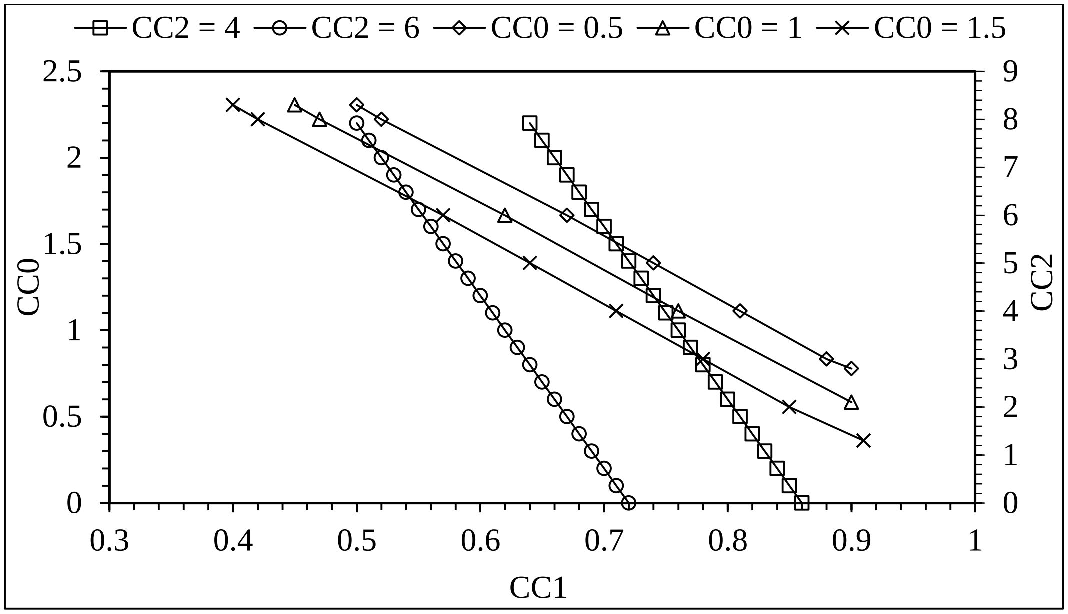


Figure 6.19 Variation of parameters for equal capacity line

6.9 Effect of calibration on speed-flow curve

Speed-volume curve developed from simulated data based on the optimal input parameter values was compared with the speed-volume curve obtained from field data as well as from default setting of model parameters. The Speed flow curves based on three different data sets are shown in the Figure 6.20.

It may be observed that the shape and pattern of speed-volume curve is different for different cases. It is due to the change in traffic flow behavior which was affected by changing the default values of model parameters. However, no difference was observed between speed-volume curve developed from field data and simulated data after calibrating the parameters of VISISM model. The simulated capacity after calibrating the VISISM model parameters was obtained as 5147 veh/hr which is close to the target capacity of 4958 veh/hr, as estimated from field data and 10.65 % higher than the capacity determined based on the default values of VISSIM parameters.

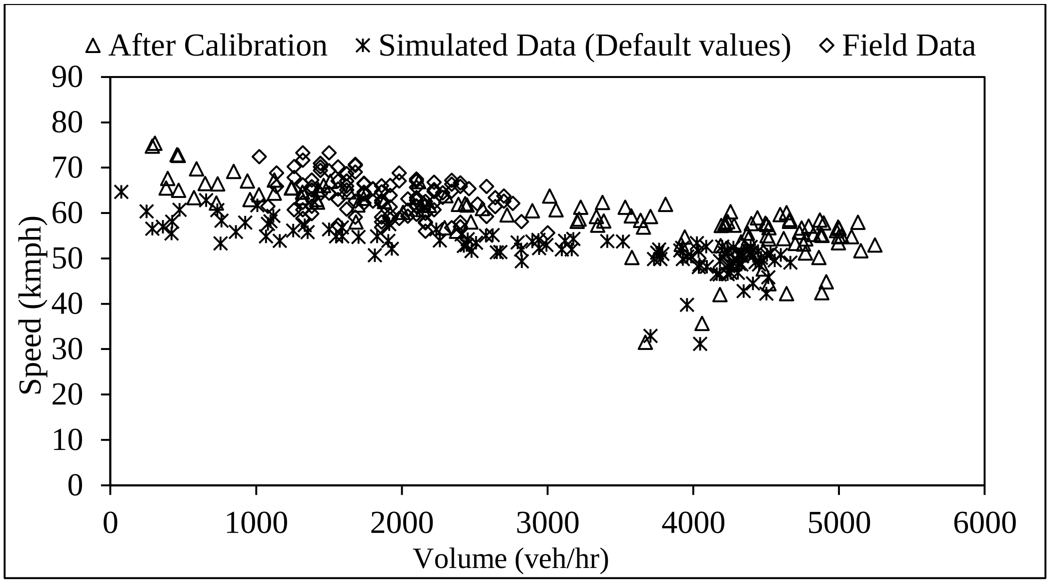


Figure 6.20 Speed-volume relations based on field and simulated data

6.10 Validation of Model parameters

For the purpose of the validation, field data collected on a six-lane divided highway section was used. The parameters estimated from field observed data are given in Table 6.7. The basic input parameters such as speed parameters, volume and vehicle composition etc. are given to VISSIM as per the values observed in the field. The values of driver behavior model parameters as calibrated in the study were provided as inputs to run the simulation for validation.

Table 6.7 Descriptive statistics for field data

	CS	CB	LCV	3W	TW	HV
Mean (km/h)	84.04	81.61	58.99	48.93	56.20	56.27
Standard Deviation (km/h)	13.32	18.63	13.34	5.82	11.87	11.37
Total Number	571	252	92	38	1191	311
Composition (%)	23.3	10.3	3.7	1.5	48.5	12.7
Minimum (km/h)	40.01	33.18	34.79	35.58	12.92	21.90
Maximum (km/h)	129.50	143.88	91.09	64.47	112.22	91.93
15th Percentile (km/h)	70.41	62.74	42.82	43.13	44.68	45.41
50th Percentile (km/h)	84.03	81.89	59.41	47.87	55.37	56.64
85th Percentile (km/h)	96.96	100.88	72.68	54.78	67.99	67.62

The simulation was run for 2 hr under lower to higher traffic volume (100 vph to 7000 vph) and output was obtained. Simulated speed and volume as obtained from output were used to develop speed-volume relation. The speed–volume relation developed from simulated data was compared with one which was obtained from field. The speed –volume relations developed from two methods is shown in Figure 6.21. It may be seen that the simulated speed and volume data is closely follows the field data which confirms the successful validation of model parameters with user defined values.

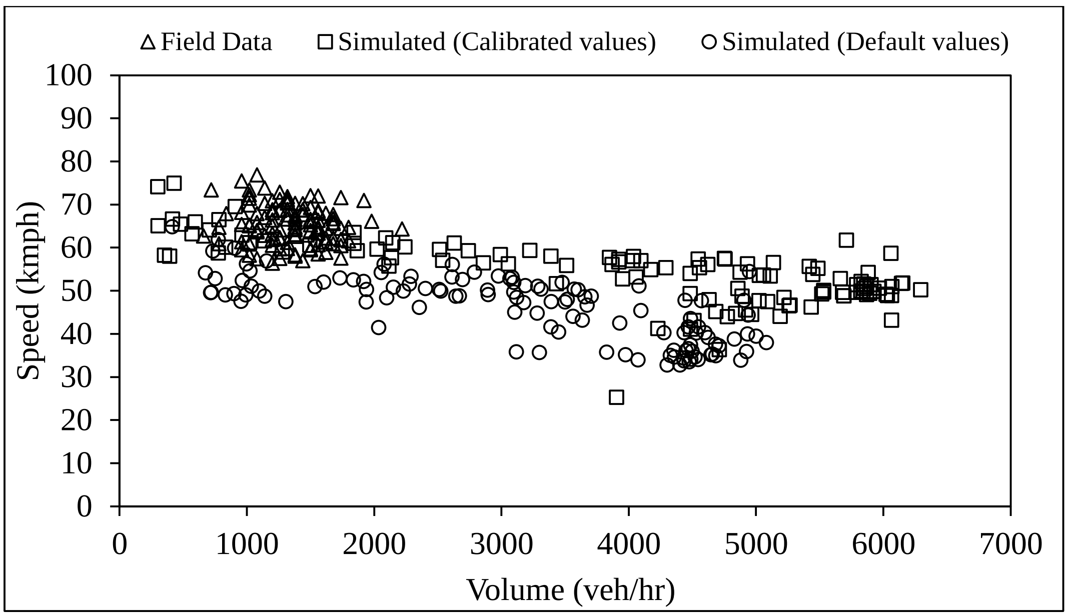


Figure 6.21 Speed flow comparison before and after calibration of parameters

6.11 Summary

In the chapter the simulation data analysis is performed considering individual parameters and performance measures like speed, volume and seed number. Statistical tests have been performed to check the sensitivity of the parameters. Calibration is done using trial and error method and optimization is performed using solver function. The maximum flow value found after simulation at default value was found to be 4599 veh/hr, and after simulating at calibrated value is 5147 veh/hr.

Chapter 7

ANALYSIS OF LANE CHANGING BEHAVIOUR AND HIGHWAY CAPACITY

7.1 General

Lane changing behaviour and capacity on multilane highways are analysed in this Chapter. This chapter uses the calibrated microscopic simulation model VISSIM to generate traffic flow data for the analysis.

7.2 Lane changing behaviour

Lane change behaviour of vehicles over a section represents the macroscopic traffic flow behaviour that influences the operational characteristics on highway very substantially. Lane changing is some time necessary for overtaking and passing another vehicle moving the same lane. After lane changing a vehicle creates voids or spaces within traffic streams which tend to increase the stream. The vehicles in India do not follow lanes due to heterogeneity even if the lane markings are provided on the highways. However, the study was performed to evaluate the possibilities for improving traffic performance with an assumption if vehicle follows lane discipline strictly on high speed corridors such as expressways. In this study the lane changes are quantified as number of lane changes per kilometre over a short interval of time.

7.2.1 Analysis under homogeneous traffic conditions

VISSIM simulation run was performed under homogeneous type traffic situations on four-lane divided section. Field data collected at Section-II was given as VISSIM input. ‘All CS’ traffic stream was simulated in VISSIM for approximately 2 hr with calibrated values of CC0, CC1 and CC2 parameters. Traffic volume inputs were given to VISSIM from lower to higher levels. After completion of simulation run, output file was generated to extract traffic volume and lane change data at 10 minutes interval. The frequency of lane change was obtained at given interval on simulated section with traffic observed volume. The relationship between number of lane changes and traffic volume was established. Similar exercise is done in case of six-lane and eight-lane divided simulated sections as well. Traffic volume and lane changes plot for four lane, six lane and eight lane divided roads is shown in Figure 7.1. It has been observed that the number of lane changes increases slowly under low to moderate volume, but it decreases as traffic volume reaches to the higher level. It is also inferred that number of lane changes stagnated at or before the level when traffic volume reaches to the maximum. No further increase or decrease in lane change was found at maximum volume level.

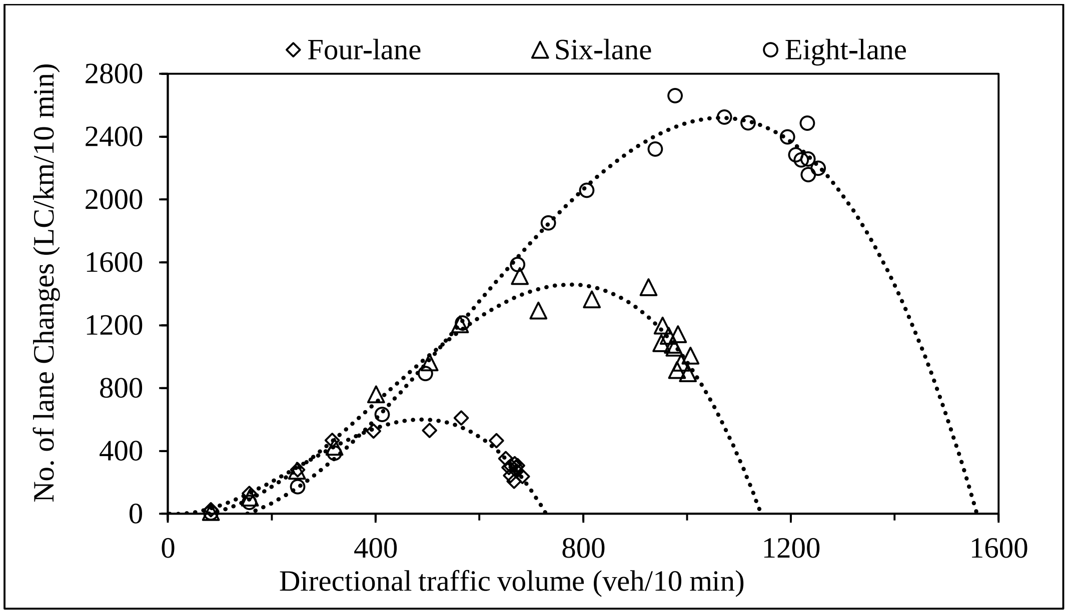


Figure 7.1 Numbers of lane changes under all CS traffic situation

Third degree polynomial curve was found to be best for fitting lane change data because the frequency of lane changes has gradually increased with traffic volume. The relationships presented in Figure 7.1 provide the value of finding maximum number of lane changes on each type of highway. Maximum numbers of lane change in one direction of travel at given time interval were identified from the lane change profile created for different simulated sections. Maximum number of lane changes was found from the trend line fitted in each case of simulated multilane section. The highest value of lane changes pertains to the curve was measured as maximum lane changes. Traffic volume and speed data was also extracted from the simulation output at 10 minutes interval and speed-flow curve was developed. The number of lane changes presented along with the speed-flow curve is shown in Figure 7.2.

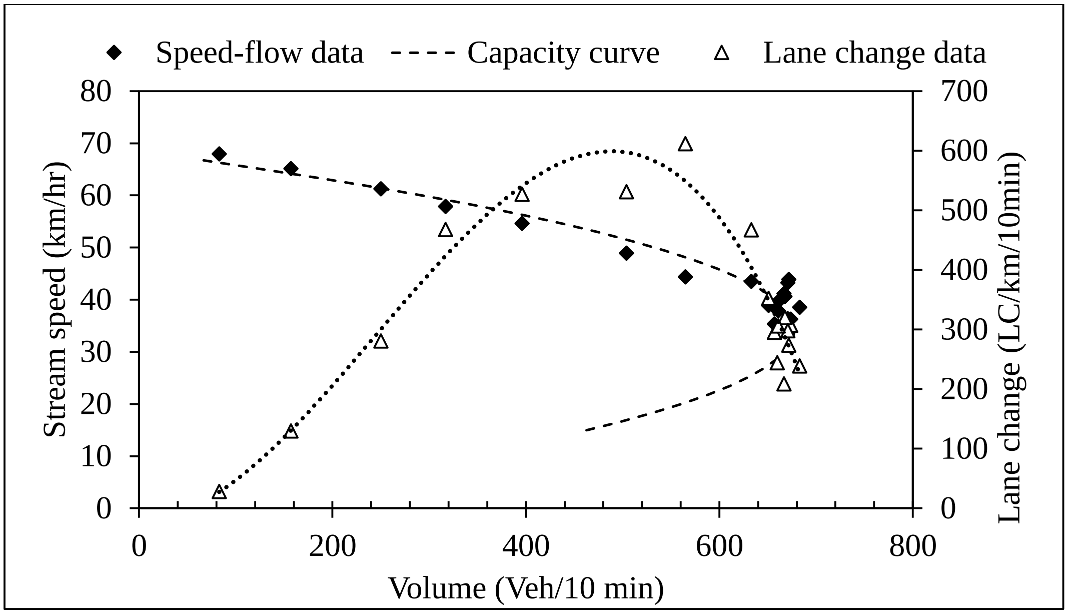


Figure 7.2 Lane change and speed-flow curve on four lane highway

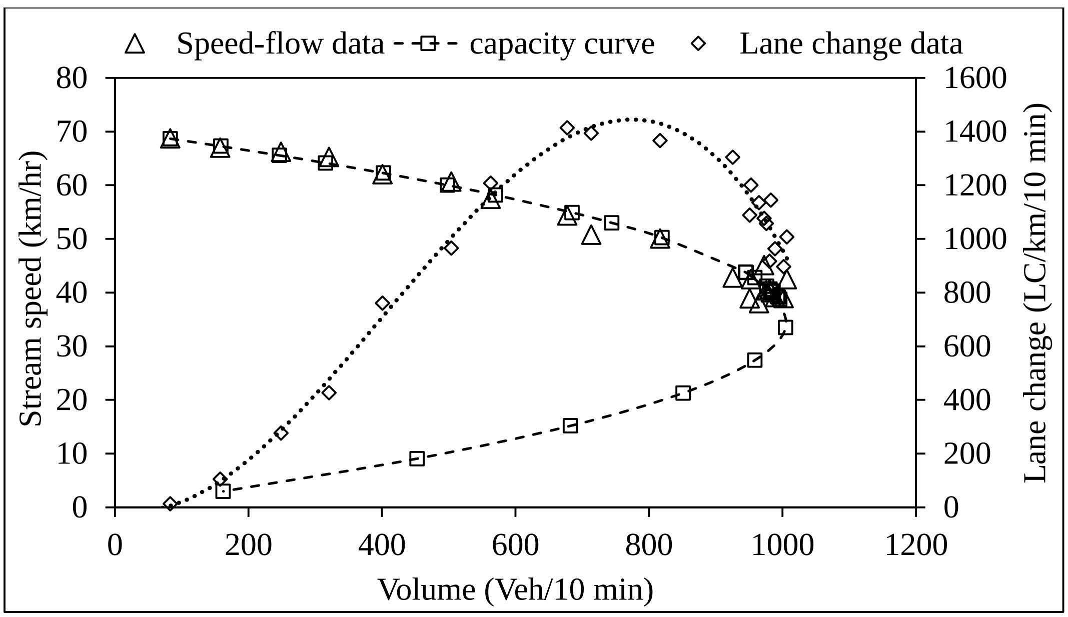


Figure 7.3 Lane change and speed-flow curve on six-lane highway

Similarly, for simulated six-lane and eight-lane divided sections speed-flow curves were developed and examined with lane change curves. Speed flow curve with lane change data on simulated six lane section is shown in Figure 7.3.

Simulated capacity (maximum volume) on four-lane, six-lane and eight-lane divided highways was determined under 'All CS' type traffic situation. The numbers of lane change value on each highway section at maximum volume level were also estimated. The numbers of lane change at capacity and Maximum lane change for different simulated sections are given in the Table 7.1. It may be observed that the frequency of lane changes reduces after reaching its maximum level and stops at capacity level of volume. The phenomenon happens due to the fact that drivers feels extremely difficult to find empty lane and tend to follow vehicle on the same lane when volume level reaches to capacity.

Table 7.1 Lane change on simulated multilane highway sections

Simulated section	Capacity (PCU/hr/lane)	Lane change at capacity (LC/km/hr)	Maximum lane change (LC/km/hr)
Four-lane	2393	1860	3600
Six-lane	2114	6013	8484
Eight-lane	2012	12960	15156

7.2.2 Analysis under mixed traffic conditions

This section deals with the analysis of lane changing behaviour under mixed traffic conditions. It is believed that the size of vehicle type affects the lane change behaviour of overall traffic stream. Therefore, this analysis was performed to examine whether proportion of one vehicle type affects the overall lane change behaviour when two types of vehicle simulated in the traffic stream.

7.2.2.1 Effect of two wheelers on lane change behaviour

The effect of two wheelers on lane changes on multilane highway sections was examined through VISSIM. To study the effect on lane changes, the proportions of two wheelers were added to the 'All cars' traffic flow stream by 10%, 20%, 30%, 40%, 50% and 100%. This incremental proportion of two wheelers was simulated in VISSIM under varying traffic volume conditions. Traffic volume was also varied from lower to higher levels and simulation runs was performed for at least 2 hours. Simulated lane changes were extracted from output at 10 minutes interval.

The traffic volume data was also extracted from the simulation output at same interval. Similar exercise is done for six lane and eight lane simulated sections and lane change behaviour of two vehicles was examined. Further, maximum lane change frequency under each incremental proportion of two wheeler mixed with “All car” traffic stream was determined for four lane, six lane and eight lane divided highway sections. Figure 7.4 shows the variation of maximum number of lane changes with different proportions of two wheelers mixed in the traffic stream when simulated on four lane, six lane and eight lane divided highway sections.

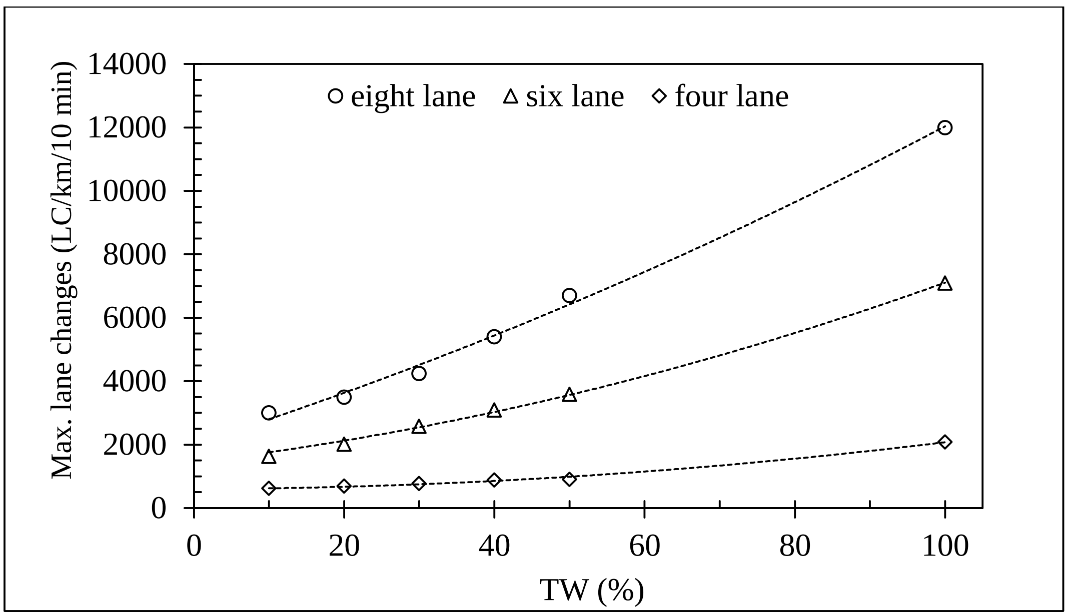


Figure 7.4 Variation of maximum lane changes with varying proportions of TW

It is obvious that the increase in proportion share of 2W in the traffic stream will increase the total number of lane changes proportionally. The maximum lane change with different proportional share was obtained and their percentage increase was estimated. Table 7.2, 7.3 and 7.4 gives the percentage increase in the maximum number of lane changes with different proportions of two wheelers on four lane, six lane and eight lane divided highways respectively. From Tables 7.2, 7.3 and 7.4 it can be inferred that the maximum number of lane changes observed per kilometre per 10 minutes increases in the same amount with increase in proportion of two wheelers in the traffic stream and the addition of a lane further increases the lane changes frequency linearly.

Table 7.2 Maximum lane changes on four lane divided section for varying TW proportion

Proportion of TW (%)	Maximum no. of lane changes (LC/km/10min)	Percent increase in Maximum lane changes
0	600	0
10	621	4
20	700	17
30	780	30
40	891	49
50	913	52
100	2085	248

Table 7.3 Maximum lane changes for six lane divided highways for varying TW proportion

Proportion of TW (%)	Maximum No. of lane changes (LC/km/10 min)	Percent increase in Max lane changes
0	1450	0
10	1636	13
20	2020	39
30	2578	78
40	3100	114
50	3585	147
100	7095	389

Table 7.4 Maximum lane changes for eight lane divided highways for varying TW proportion

Proportion of TW (%)	Maximum No. of lane changes (LC/km/10 min)	Percent increase in Max lane changes
0	2500	0
10	3000	20
20	3500	40
30	4245	70
40	5400	116
50	6700	168
100	12000	380

7.2.2.2 Effect of Heavy vehicles on Lane change behaviour

Similar exercise was also performed with varying proportions of HVs in the “All car” traffic stream by 10%, 20%, 30%, 40% 50% and 100%. It is observed that as the proportion of HVs increases the frequency of lane changes also increases. However, the lane change frequency starts to decrease after attaining maximum number of lane change level. The values of maximum number of lane changes were obtained with increase in proportion of HVs obtained by simulating traffic stream on four-lane, six-lane and eight-lane divided highway sections. It is found to be interesting that the maximum number of lane changes are reduced significantly as the percentage of heavy vehicle are added proportionately in a stream of “All cars” which is contrary to the results obtained in case of 2Ws.

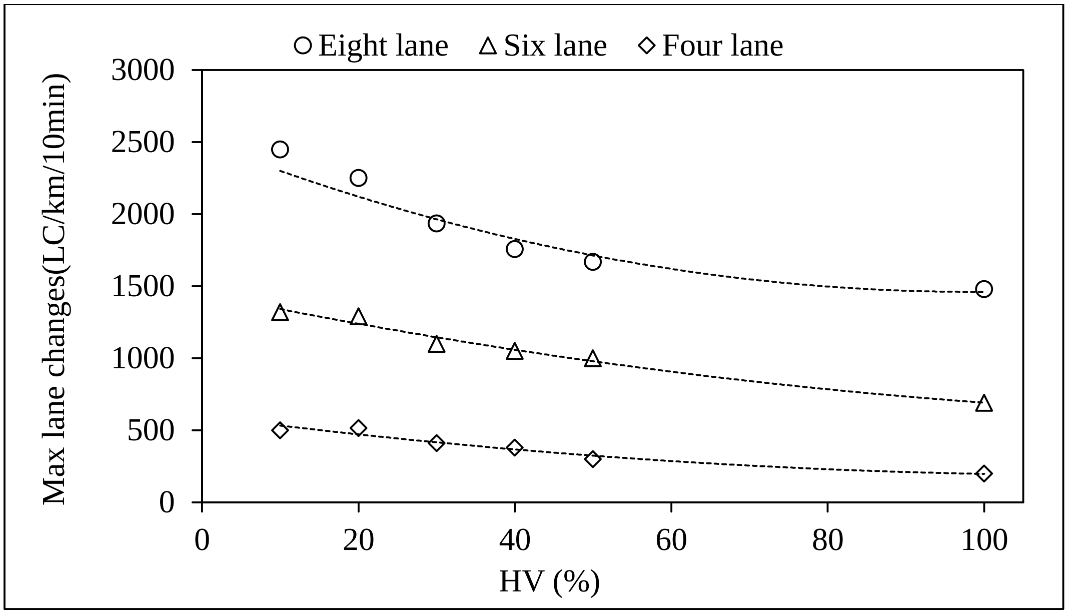


Figure 7.5. Maximum number of lane changes at varying proportions of HV

Table 7.5, 7.6 and 7.7 gives the percentage decrease in the maximum number of lane changes at different proportions of HVs on simulated four-lane, six-lane and eight-lane divided highway sections. It can be inferred that the maximum number of lane changes decreases with increase in proportion of heavy vehicle in 'All CS' traffic stream with addition of extra lane in direction of travel.

Table 7.5 Maximum lane changes on simulated four lane divided section under varying HVs proportion

Proportion of HV (%)	Maximum no. of lane changes (LC/km/10 min)	Percent decrease in Max lane changes
0	600	0
10	500	17
20	516	14
30	410	32
40	380	37
50	300	50
100	200	67

Table 7.6 Maximum lane changes on simulated six lane divided section under varying HVs proportion

Proportion of HV (%)	Maximum No. of lane changes (LC/km/10 min)	Percent decrease in Max lane changes
0	1450	0
10	1320	9
20	1290	11
30	1100	24
40	1050	28
50	1000	31
100	690	52

Table 7.7 Maximum lane changes on simulated eight lane divided section under varying HVs proportion

Proportion of HV (%)	Maximum No. of lane changes (LC/km/10 min)	Percent decrease in Max lane changes
0	2500	0
10	2448	2
20	2251	10
30	1936	23
40	1757	30
50	1668	33
100	1479	41

7.2.3 Maximum lane change model

Maximum lane change behaviour with respect to four different vehicle types was analysed independently through simulation. Traffic flow stream was simulated with only homogeneous vehicle type traffic stream such as ‘All 2W’, ‘All Car’, ‘All 3W’ and ‘All HV’ and maximum lane changes were obtained through simulation. It is believed that the composition of vehicle types affects the lane change behaviour. Therefore, linear models were proposed for four-lane, six-lane and eight-lane divided highway sections to estimate maximum lane change for the condition when all the above types of vehicles are present in the traffic stream. The expressions to estimate maximum number of lane changes for the known traffic composition on simulated highway sections are as shown in Equation 7.1, 7.2 and 7.3.

For Four lane divided highway

$$\frac{100}{LC_{max}} = \frac{P_{car}}{600} + \frac{P_{TW}}{2085} + \frac{P_{HV}}{200} + \frac{P_{3W}}{489} \tag{7.1}$$

For Six lane divided highway

$$\frac{100}{LC_{max}} = \frac{P_{car}}{1480} + \frac{P_{TW}}{7095} + \frac{P_{HV}}{690} \tag{7.2}$$

For Eight lane divided highway

$$\frac{100}{LC_{max}} = \frac{P_{car}}{2526} + \frac{P_{TW}}{12000} + \frac{P_{HV}}{1479} \tag{7.3}$$

Where, LCmax is the maximum number of lane changes (LC/km/10min) and P_{car}, P_{HV}, P_{3W} and P_{TW} are the percentage of car, heavy vehicle, three wheeler and two-wheeler percentage. The numerical values in denominators are the maximum lane changes/km/10min under homogeneous type of traffic.

7.3 Estimation of capacity on multilane highways

In this section the capacity of four lane, six lane and eight lane divided highway sections are determined using VISSIM model calibrated for mixed traffic conditions. The simulation runs were performed with the 1.4 km of highway sections having 7 m, 10.5 m, and 14 m carriageway width in one direction of traffic movement under left sided regulation. Link attributes were assigned as per the field data observed on section-V. Simulation run was performed for about 3 hr and data was extracted to develop speed-volume curve. The capacity of simulated four-lane, six -lane and eight-lane divided highway sections are determined as 4786, 6341 and 8046

PCU/hr/dir respectively. The speed volume relationship for four-lane, six-lane and eight-lane simulated sections is shown in Figure 7.6, Figure 7.7, and Figure 7.8 respectively.

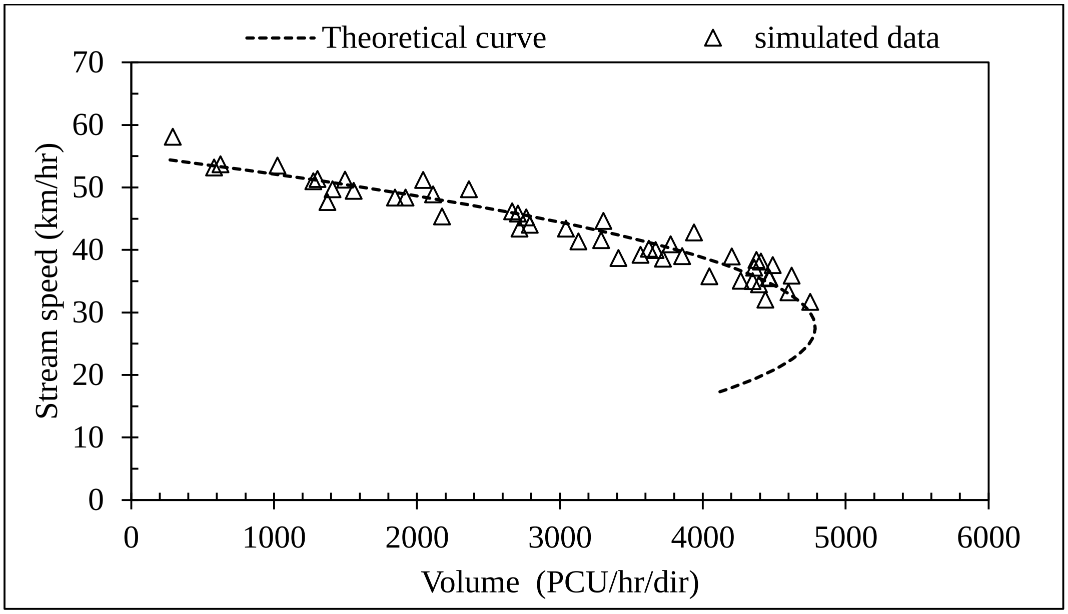


Figure 7.6 Capacity of four lane simulated section

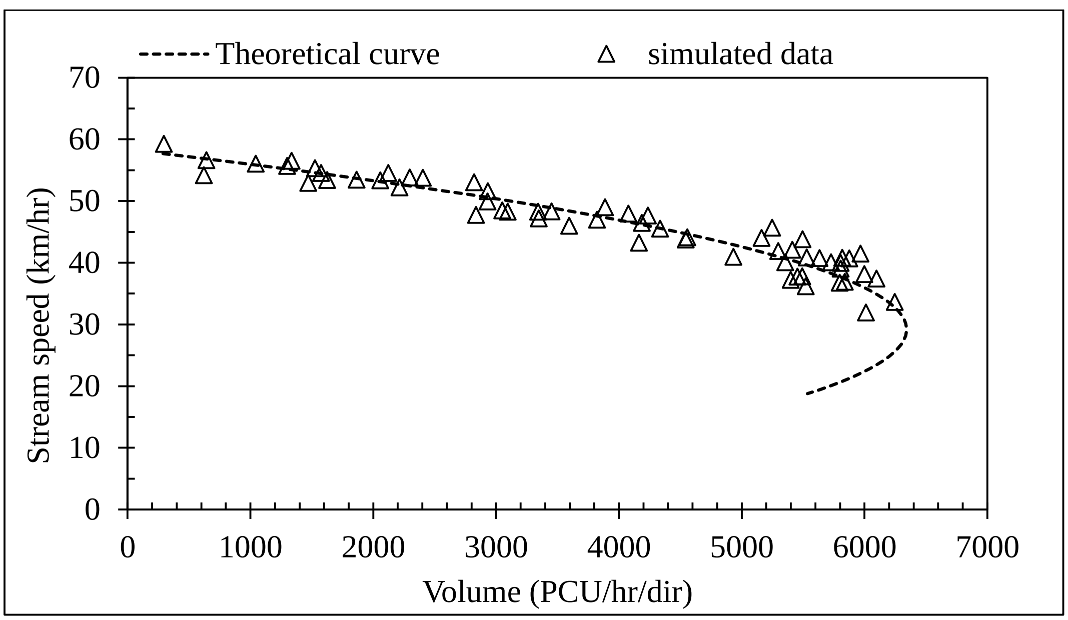


Figure 7.7 Capacity of six lane section through VISSIM

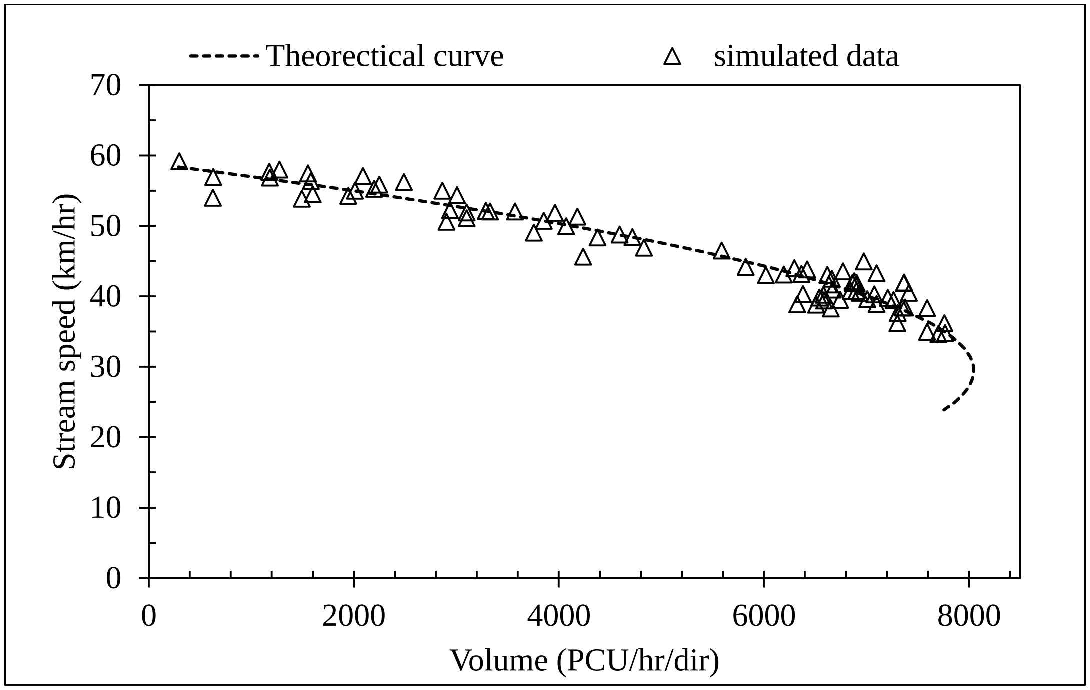


Figure 7.8 Capacity of Eight lane section through VISSIM

The capacity values determined for all simulated sections are given in Table 7.8. It may also be observed that the capacity per lane is decreasing with the addition of number of lanes to the section. This marginal decrease in capacity per lane with increase in number of lanes is due to increase in lane changing opportunities (Yang and Zhang, 2007).

Table 7.8 Capacity of different types of highways

Highway type	Capacity (PCU/hr/dir)	Capacity per lane (PCU/hr/dir)
Four lane divided	4786	2393
Six lane divided	6341	2114
Eight lane divided	8046	2012

7.4 Summary

In this Chapter the macroscopic parameters lane changing and capacity are analysed using the calibrated simulation model. Lane changing for homogeneous type of traffic on four lane, six lane and eight lane are analysed. Effect of two wheelers and heavy vehicles on lane changing are discussed and a general relation is proposed to estimate the max number of lane changes for a given mixed traffic proportion. Capacity on four lane, six lane and eight lane sections are estimated using VISSIM.

Chapter 8

ESTIMATION OF PASSENGER CAR UNITS THROUGH MICROSCOPIC TRAFFIC SIMULATION

8.1 General

This chapter presents a methodology for estimating passenger car units (PCUs) through microscopic traffic simulation model VISSIM. The output generated through VISSIM simulation was used to estimate PCU under controlled traffic and roadway conditions. The estimated PCU were further examined under different levels of service and using soft computing techniques such as artificial neural network.

8.2 VISSIM simulation for PCU estimation

VISSIM microscopic traffic simulation model as calibrated for mixed traffic condition is used for estimation of PCU values. Field data collected on Section I was used as inputs to the base model as prepared in VISSIM. The detail about the field section and collected data are given in Chapter 5. The base model created in VISSIM has following characteristic: link length 1.4 kms (200 m used as buffer length on both the ends); four-lane divided section with 1.5 m of paved shoulder on both the sides of travel directions. The details of the base section created in VISSIM and testing of base section was explained in Chapter 6.

Calibrated values of driver behaviour model parameters such as CC0, CC1 and CC2 are assigned as 1.8 m, 0.53 sec. and 6 m respectively. Driver behavior model as modified for mixed traffic conditions are used and simulation run was performed for 2 hrs. After completion of simulation run, simulated output was extracted at 5 min interval to estimated speed and traffic volume. For present analysis, dynamic PCU method (Chandra and Kumar, 2000) was used for estimating PCU of simulated vehicle types. The PCU values of vehicle types estimated through simulation were compared with the values obtained on Section I using field data. The comparison of simulated and field estimated PCUs and the percent error between the PCUs was calculated for each vehicle type, and the results are shown in Table 8.1. The estimated percentage errors between simulated and field values varies from 0.79% to 3.78% which show that the VISSIM is able to replicate field PCUs and may be used for developing PCU model.

Table 8.1 Percentage error between simulated and field PCU values

Vehicle Type	Simulated PCU	Field estimated PCU	Percentage error (%)
CB	1.27	1.26	0.79
LCV	1.48	1.51	1.99
HV	3.84	3.96	3.03
MAV	8.05	8.21	1.95
TW	0.33	0.34	2.94
3W	1.05	1.04	0.96
B	5.86	6.09	3.78

8.3 Development of PCU model

Present study attempt for developing a PCU model for easy conversion of traffic volume observed under mixed traffic conditions. The proposed PCU model does not require enormous field data and complex methodology to estimate PCUs and thereby to convert traffic volume. To develop PCU model, VISSIM simulation runs were performed to generate traffic flow data under controlled or predetermined type of traffic conditions. The variables to be incorporated for development of the models are composition of vehicle type and volume to capacity ratio. The analysis uses the assumption that the PCU of a vehicle type is a measure of impedance caused by a subject vehicle type to the vehicle type car. The impedance will be more if their proportion is more in the traffic stream. Therefore, it is decided first to evaluate the effect of individual subject vehicle type on stream of cars. Traffic simulation runs were performed with adding different proportional share of subject vehicle types into All CS type traffic stream. The proportion of a vehicle type for which PCU need to be estimated was varies from lower to higher. The traffic volume was also increased from lower level to higher level (100 vph to 6000 vph) to generate the congestion on simulated link section. After completion of simulation run, speed-volume curve was developed for each case and capacity was estimated in veh/hr. The speed-volume curve for heavy vehicles (HV) at different proportional share is shown in Figure 8.1. Similar exercise is done for all the other subject vehicle types and speed-volume curve was developed to find maximum volume. The maximum flow values at varying proportion of vehicle types are given in Table 8.2.

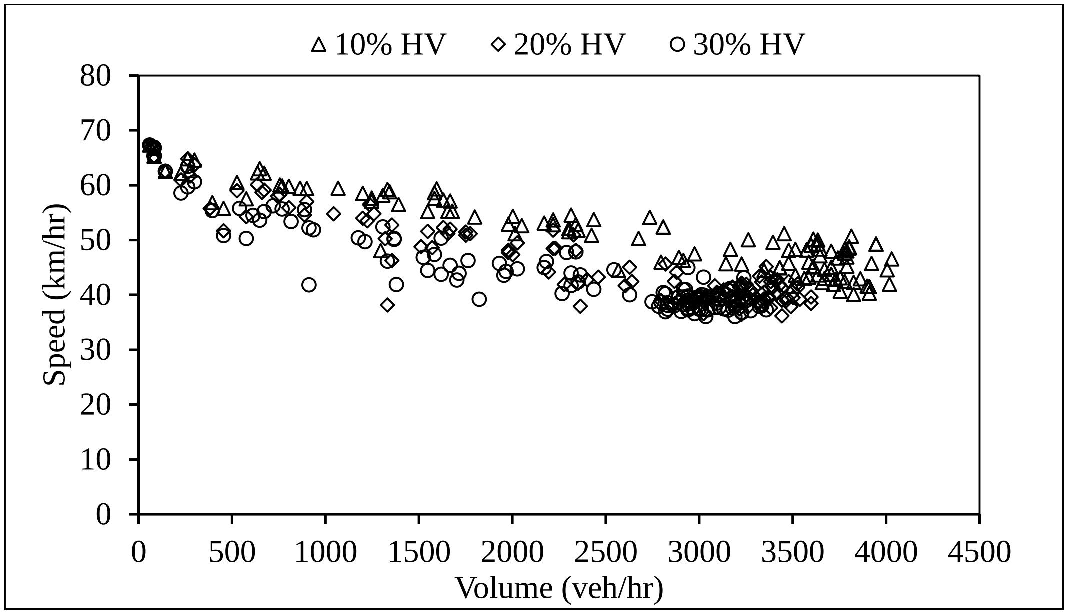


Figure 8.1 Speed-flow curves for Heavy vehicles (HV) at varying proportion

Table 8.2 Simulated capacity values on four-lane divided section and share of second vehicle type

% share	Maximum Volume (vph)						
	CB	TW	3W	LCV	HV	MAV	BUS
10	4512	5208	4320	4452	3900	3936	3984
20	4236	5388	4224	4248	3600	3312	3600
30	4260	5544	4224	4152	3360	2988	3264

The maximum flow under mixed traffic stream found to be reduced as the percentage share of second vehicle types like CB, HV, MAV, LCV, BUS, 3W increased. It is due to their large size in comparison to standard car (CS). However, the addition of same amount of TW tends to increase the capacity of section due to its less space occupancy.

8.3.1 Estimation of volume to capacity ratio

For the estimation of volume to capacity ratio, the speed-volume curve developed under varying proportional share of subject vehicle types are used. The volumes corresponding to change in the speed on speed-volume curve were identified to demarcate the boundaries of highest volume to capacity ratio. The boundary of highest volume to capacity ratios are marked on speed-volume curve for a vehicle type LCV is shown in the Figure 8.2.

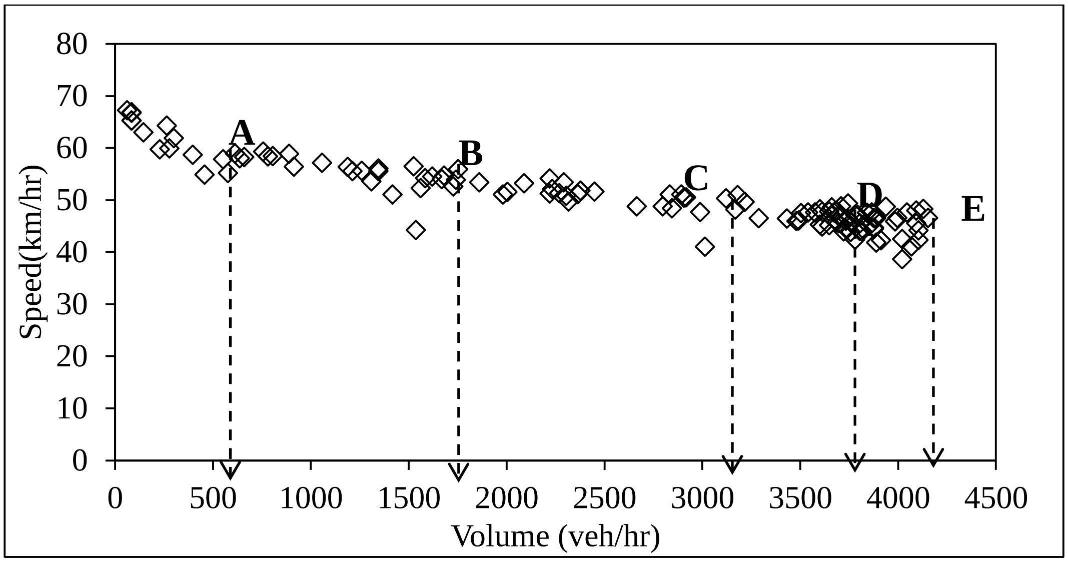


Figure 8.2 Determination of v/c ratio corresponding to different Levels of Service for LCV at 30% of its own proportion

Similarly, the boundaries for volume to capacity ratios were found for all other subject vehicle types on simulated speed volume curve developed under their varying proportional share in the traffic stream. It is observed that the capacity values decreases with increase in proportional share of vehicle type CB, 3W, HV, and LCV. However, capacity values are found to be increased with the addition of TWs in the traffic stream. The traffic volume and volume to capacity ratios as estimation through all the scenarios are given in Table 8.3.

Table 8.3 Volumes corresponding to different v/c ratio for LCV at different percentage share

% share	v/c ratio	Volume (veh/hr)
10	0.14	636
	0.50	2220
	0.77	3432
	0.91	4068
	1.00	4452
20	0.21	888
	0.52	2220
	0.78	3324
	0.92	3912
	1.00	4248
30	0.18	756
	0.42	1752
	0.77	3180
	0.90	3744
	1.00	4152

8.3.2 PCU values with volume to capacity ratios

The PCU values of all subject vehicle types were estimated at 5 min interval by using Dynamic PCU method. The PCU value were obtained at various levels of volume to capacity ratios for a given proportional share of each subject vehicle type. The variation of PCU with different volume to capacity ratios for 10% HVs is shown in the Figure 8.3. Similar, plot was also shown in case of 3W in Figure 8.4.

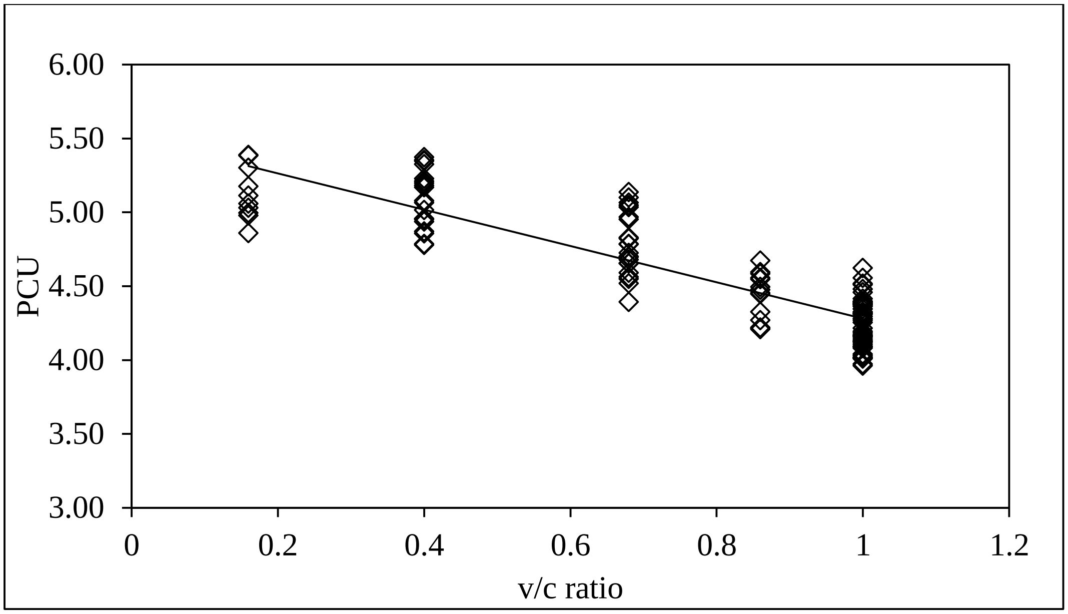


Figure 8.3 Variation of PCU at different v/c ratio for 10% HV

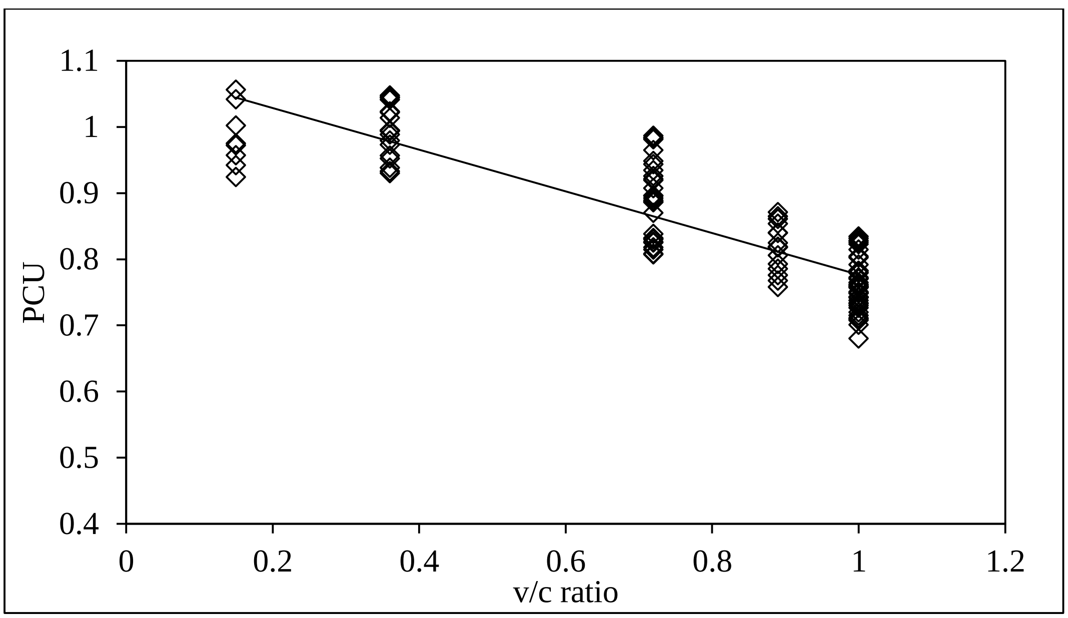


Figure 8.4 Variation of PCU at different v/c ratio for 10% 3W

The correlation analysis between volume to capacity ratios, proportional share of subject vehicle types and their PCUs was performed in order to develop a multiple linear regression model. The result indicates that there is no correlation was observed between volume to capacity ratio and the percentage share of subject vehicle types. Hence, these variables were selected as independent variables for the development of regression model. The separate multiple linear regression (MLR) model was proposed to estimate PCU for each subject vehicle type. The following equations are developed to estimate PCUs as given below.

$$PCU_i = a + b*(v/c) + c*P_i \tag{8.1}$$

$$PCU_{HV} = 5.82 - 1.398*(v/c) - 0.017*P_{HV} \tag{8.2}$$

$$PCU_{TW} = 0.357 - 0.063*(v/c) - 0.001*P_{TW} \tag{8.3}$$

$$PCU_{CB} = 1.20 + 0.078*(v/c) \tag{8.4}$$

$$PCU_{MAV} = 8.061 - 2.162*(v/c) - 0.023*P_{MAV} \tag{8.5}$$

$$PCU_{3W} = 1.096 - 0.316*(v/c) - 0.002*P_{3W} \tag{8.6}$$

$$PCU_{BUS} = 6.069 - 1.082*(v/c) - 0.013*P_{BUS} \tag{8.7}$$

$$PCU_{LCV} = 1.518 - 0.223*(v/c) - 0.003*P_{LCV} \tag{8.8}$$

It was observed that the 'p' value representing the effect of volume to capacity ratio and the 'p' value representing the effect of percentage share (P) of vehicle type were significant as the values were less than 0.025. It means that v/c ratio and % share have significant effect on PCU. But for CB, the 'p' value corresponding to % share was 0.344 which is greater than 0.025 which means that it has no significant effect on the PCU value of CB. Hence, only the v/c ratio was used in the equation for CB.

8.3.3 Application of PCU model

The model developed for estimation of PCU was applied to convert the traffic volume observed on four-lane divided highway section. Field data as observed on Section I was given as input to VISSIM and simulation of mixed traffic was performed for 2 hrs. Simulated data were extracted from the output file and developed speed-volume relationship to determined capacity. The boundaries of maximum volumes were identified at different stages of drop in average speed on speed-volume curve. Volume to capacity ratios were estimated at five different levels and used as input along with the vehicle composition for applying PCU model. The speed-volume curve developed through simulation with demarked boundaries is shown in Figure 8.5. The capacity value is determined as 4560 veh/hr from speed-volume curve for estimation of volume to capacity ratios (V/C). . The volume to capacity ratios were also assigned as higher to lower levels of service (LOS) from A to E. The lower V/C ratio is assigned as A which denotes higher level of service and higher V/C ratio assigned as E which denotes lower or poor level of service. The volume and the speed corresponding to particular LOS are given with V/C ratios in Table 8.4. The PCU model was applied to estimate PCU of all subject vehicle types using five different V/C ratios and its own proportion in the traffic stream. The PCU estimated for each vehicle types were used to convert traffic volume as obtained from simulated data.

From the Figure 8.5, it is observed that average speed of vehicles at low volume level is also very less. This is because of the total composition of vehicle type TW and 3W is about 60%. These vehicles have relatively low speed than passenger cars.

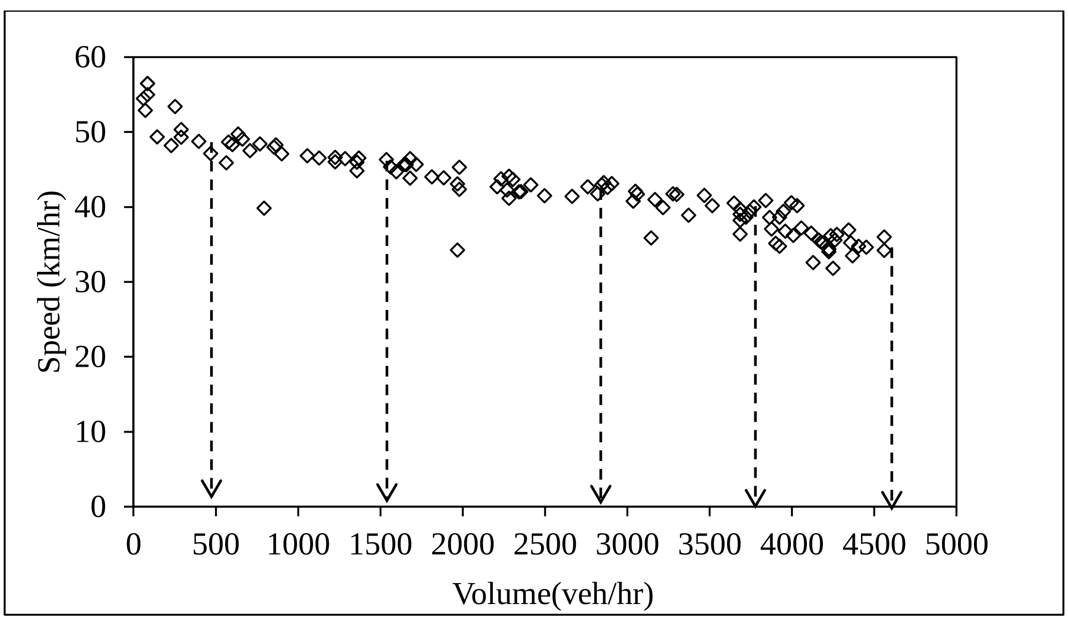


Figure 8.5 Speed - flow curve of mixed traffic showing the volume boundaries at different Level of Service

Table 8.4 V/C ratio determined at different LOS

LOS	A	B	C	D	E
v/c ratio	0.14	0.37	0.63	0.88	1.00
Maximum volume (veh/hr)	636	1680	2856	3996	4560
Speed at LOS (kmph)	49.7	43.8	43.2	40.5	34.2

It is observed that, a speed reduction from 43.8 kmph to 43.2 kmph from LOS B to LOS C with volume difference of 1176 veh/hr. This is because of from LOS B to LOS C, there is not much difference in the average speeds. However, a significant difference was observed in density values as it has more potential to add more vehicles.

8.4 Verification of PCU model

The models developed to estimate PCU has been validated by using field data collected at Section-II. The PCU values of different vehicle types as estimated by using Dynamic PCU method were compared with the PCUs estimated from models. The variation in PCU of HV and TW estimated by Dynamic PCU method and PCU equations is shown in Figure 8.6 and Figure 8.7 respectively.

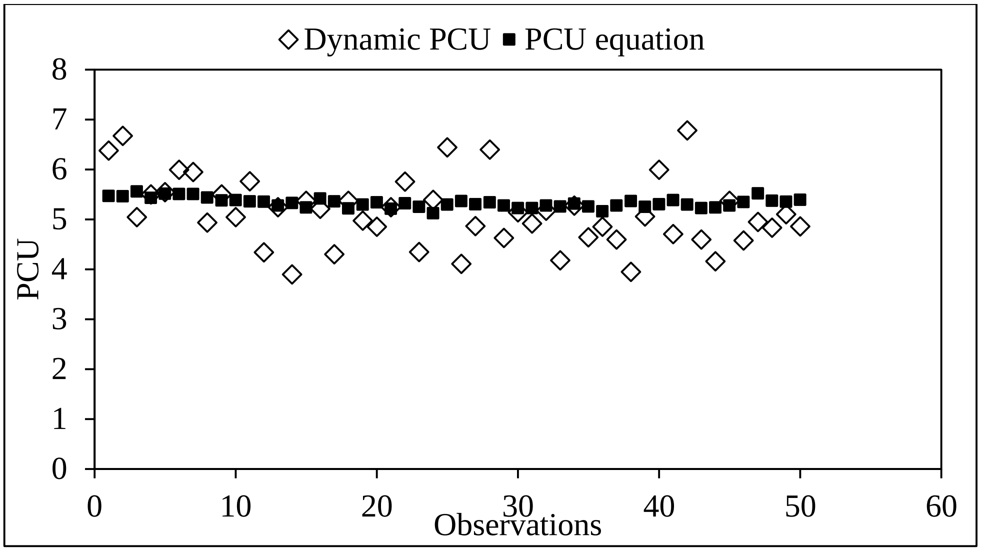


Figure 8.6 PCU of HV from the two methods

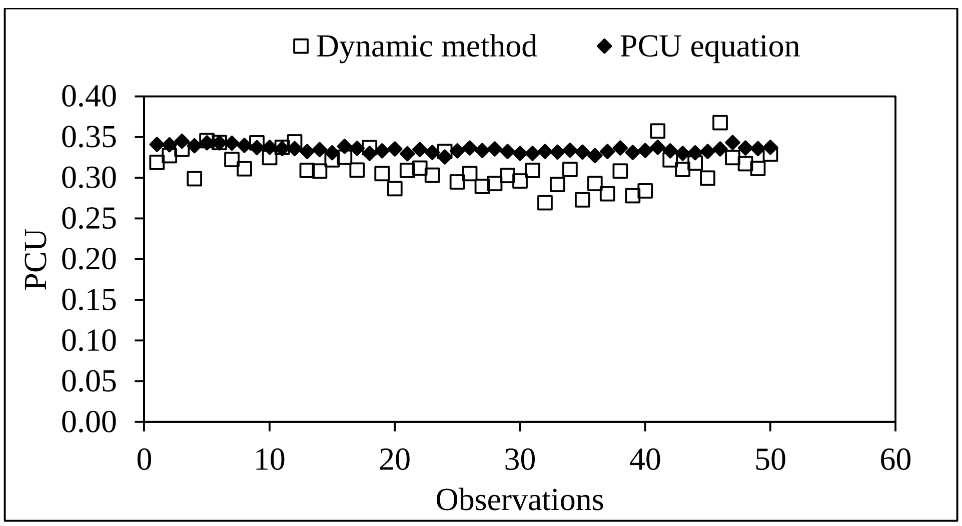


Figure 8.7 PCU of TW from the two methods

8.5 PCU on Simulated Six-lane and Eight-lane divided highway sections

The PCU value of subject vehicle types on six-lane and eight divided sections were estimated through simulation. Field data collected at Section-VII and Section-VIII was used to estimate PCU using proposed model. The PCU of all subject vehicle types estimated for this section are shown in Table 8.5. It is observed that as the v/c ratio increases the PCU of all vehicle types decreases except for CB. This may be because of less difference in the speed between CB and CS.

Table 8.5 PCU of each vehicle type on six-lane section obtained by proposed PCU equations

v/c ratio	PCUs of subject vehicle types						
	CB	LCV	BUS	MAV	HV	TW	3W
0.24	1.22	1.45	5.74	7.45	5.41	0.31	1.00
0.52	1.24	1.39	5.44	6.84	5.02	0.30	0.91
0.74	1.26	1.34	5.20	6.36	4.71	0.28	0.85
0.93	1.27	1.30	5.00	5.96	4.45	0.27	0.79
1.00	1.28	1.29	4.92	5.81	4.35	0.27	0.76

Similarly, PCU of subject vehicle types were determined by using the proposed equations on simulated eight-lane divided highways section. It may be seen that as the v/c ratio increases the PCU of all vehicle types decreases except for CB. This might be because of higher speed of CB when compared to CS. The PCU of different vehicle types are shown in Table 8.6.

Table 8.6 PCU of different vehicle types on eight-lane section

v/c ratio	PCU values		
	CB	HV	TW
0.14	1.21	5.37	0.33
0.36	1.23	5.06	0.32
0.66	1.25	4.64	0.3
0.89	1.27	4.31	0.29
1	1.28	4.17	0.28

The PCU values of all subject vehicle types on six-lane and eight-lane sections were compared. It is found that as the numbers of lanes increases the PCU of subject vehicle types decreases. This change in PCU values of vehicle types HV and CB at different LOS are shown in Figure 8.8 and Figure 8.9 respectively. On an average there with the increase in number of lanes decrease in the PCU value of HV and CB have been observed as 1.8% and 3.5% respectively. Similarly, the same trend was observed for other subject vehicle types also.

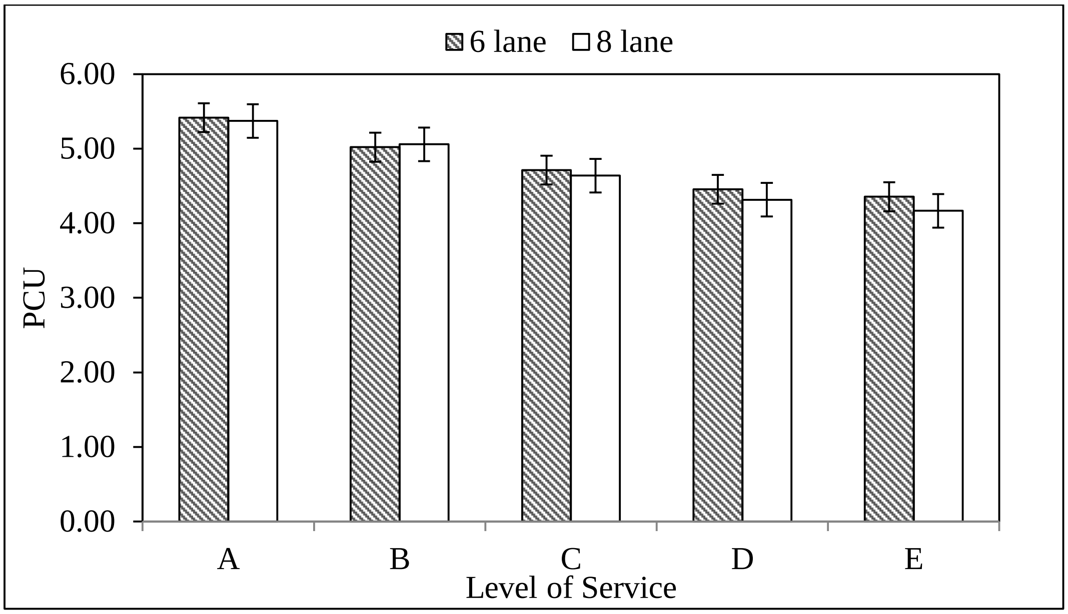


Figure 8.8 Effect of number of lanes on PCU of HV

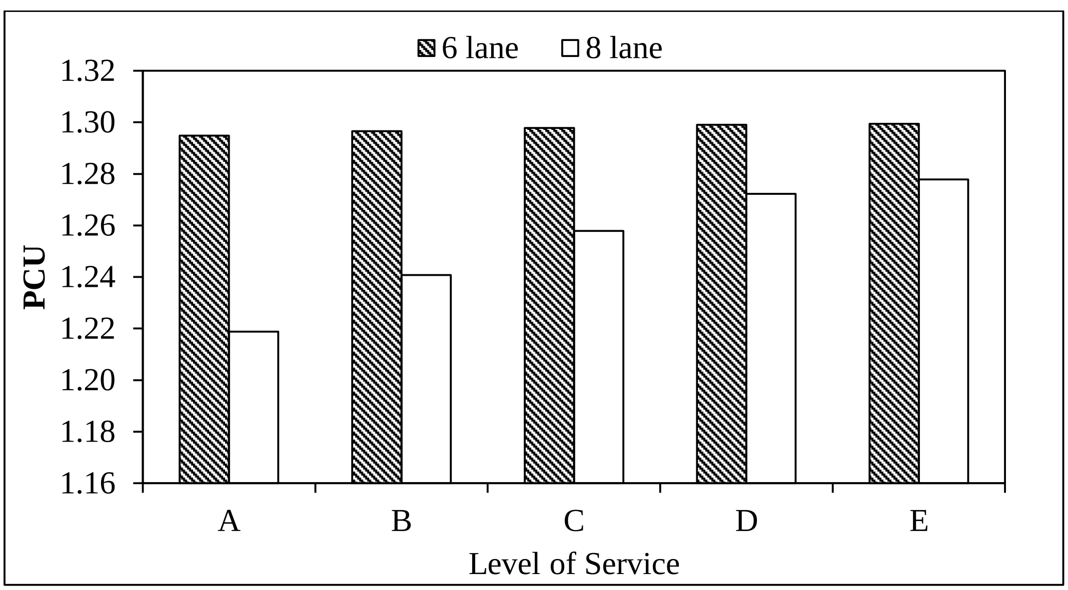


Figure 8.9 Effect of number of lanes on PCU of CB

Similarly, the comparison of PCU of different vehicle types was made on four-lane, six-lane and eight-lane simulated sections. The PCU of vehicle types are found to be decreased with increase in number of lanes. There was a decrease in PCU of HV about 2.3% with the increase in number of lanes from four-lane to six-lane and a decrease of 1.8% was observed with the increase in number of lanes from six-lane to eight-lane. The PCU values estimated for vehicle type HV on different simulated sections are shown in Figure 8.10.

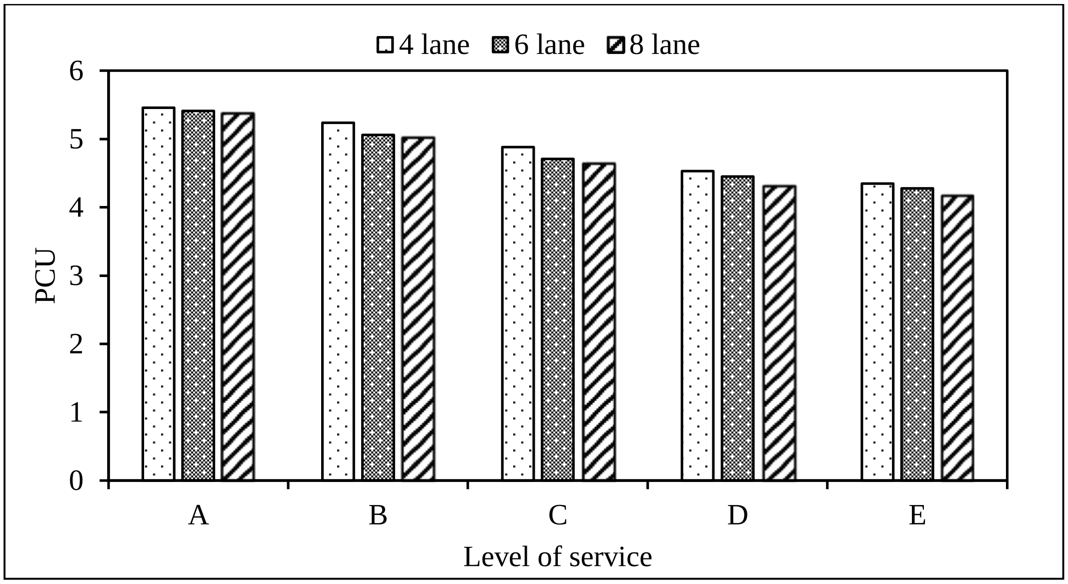


Figure 8.10 Decrease in PCU of HV with increase in number of lanes

8.6 Development of ANN and ANFIS models

Some of the pervious studies indicated that PCU is non linear relation with the V/C ratio. So, Artifical neural networks (ANN) and Artificial neuro fuzzy interface system (ANFIS) models are also developed for estimating PCU values of subject vehicle types with respect to passenger cars. The PCU estimated from different approaches are compared statistically in order to justify the best approach with the same set of input variables.

8.6.1 ANN

Neural networks represent simplified methods of a human brain and can be replaced with the customary computations which finds the problems difficult to solve. The artificial neural network obtains knowledge through learning. The same way as the human brain, ANN utilizes examples to learn. Artificial neural networks have been used broadly in the various engineering applications because of their ability to offer a worldwide practical method for real-valued, discrete-valued, and vector valued functions (Khademi et al., 2017). The general structure of ANN is shown in Figure 8.11. The network contains three different layers namely input layer, hidden layer, and an output layer.

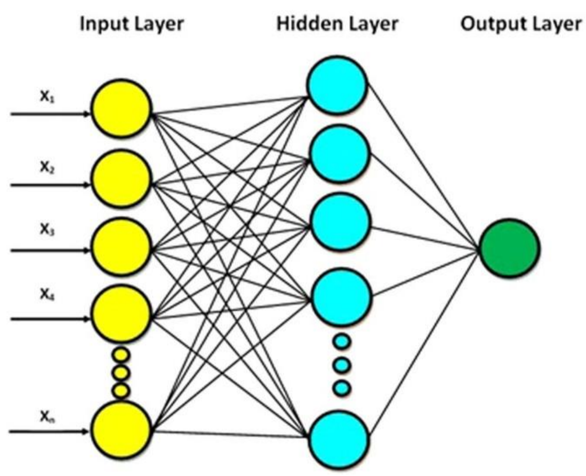


Figure 8.11 Structure of ANN model

In this study, the Alyuda Neuro-Intelligence software was used to develop the ANN models. The procedure of development of ANN model of this study is shown in Figure 8.12.

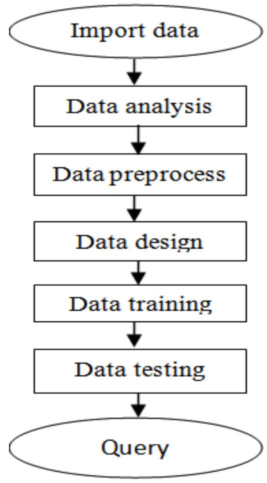


Figure 8.12 Flowchart for development of ANN model

To produce the best results by the network, several architectures including different number of hidden layers, distinct activation functions as well as different combination of neurons in each hidden layer was utilized in training of all ANN models. Tables 8.7 summarize the best architectures, activation functions and number of iterations used in the network to obtain the best results for all types of vehicles.

Table 8.7 Characteristics of the best structure of ANN architecture

Vehicle type	Best architecture	Best algorithm	Training error	Iterations
TW	4-5-1	Conjugate Gradient Descent	0.011494	10000
3W	4-4-1	Conjugate Gradient Descent	0.035779	10000
CB	1-7-1	Quick Propagation	0.017115	10000
LCV	4-4-1	Quick propagation	0.030412	10000
HV	4-7-1	Conjugate Gradient Descent	0.157045	10000
BUS	4-8-1	quick propagation	0.15271	10000
MAV	4-10-1	Quasi-Netwon	0.21338	10000

8.6.2 ANFIS

ANFIS is identified as a solution for different complex problems. ANFIS is a class of adaptive, multi-layer and feed-forward networks which is comprised of input–output variables and a fuzzy rule base of the Takagi–Sugeno type (Khademi et al., 2016). The structure of ANFIS is shown in

Figure 8.13. The structure of ANFIS has contained five different layers. Layer 1 takes the responsibility for fuzzification of input feature values in the range of 0 to 1. Any node in the Layer 2 multiplies the incoming signals and sends the results out. The membership values are getting normalized in the Layer 3. Layer 4 can establish the relationship between the input and output values, and Layer 5 is also called the de-fuzzification layer consists of one single node which generates the summation of all incoming signals from previous node and results in a single value. In this layer, each rule output is added to the output layer. (Khademi et al., 2016, 2017).

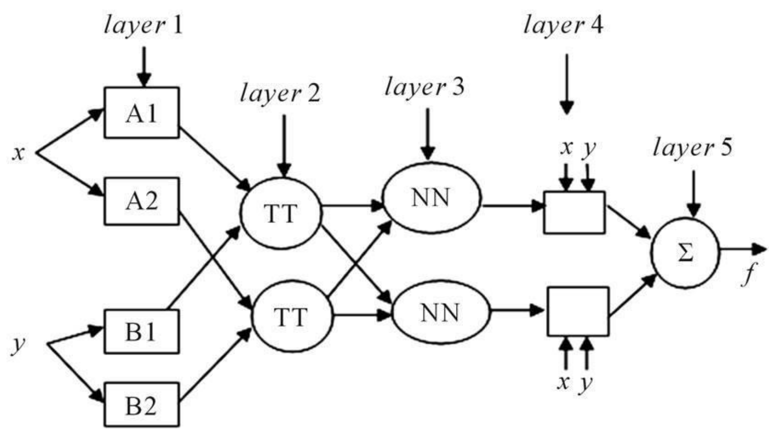
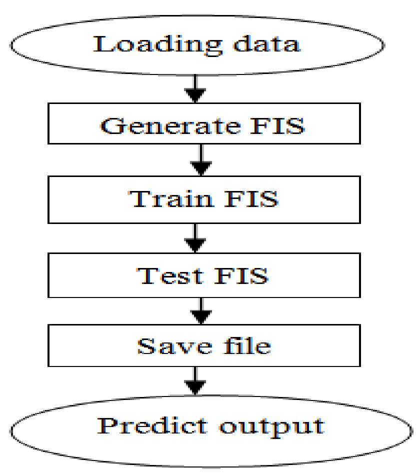


Figure 8.13 Structure of ANFIS model

In this study, the MATLAB was used to develop the ANN models. The procedure of development of ANFIS model of this study is shown in Figure 8.14.



8.14 Flowchart for development of ANFIS model

In a preliminary analysis, evaluated a command genfis1 with different types of membership functions (including gbellmf, gaussmf, gauss2mf, psigmf, dsigmf, pimf, trapmf, and trimf) and different numbers of epochs to get the best training performance with minimum squared error. The command genfis1 generates a Sugeno-typeFIS structure as initial conditions (initialization of the membership function parameters) for ANFIS training. Hybrid learning algorithm was also employed to optimize the learning procedure of the ANFIS models in each trial. The hybrid learning algorithm is a combination of the least-squares method and the back-propagation gradient descent method for training FIS membership function parameters in emulating a training data set. Finally, the trimf with 3 numbers of membership functions was used for the adaptive system analysis.

Table 8.8 indicated the results of statistically performance and optimal architecture of ANFIS networks. The combination of Trimf and constant MFs for input and output layers, respectively, and hybrid as learning method produced the better consequences rather than the application of other combinations for all types of vehicles. The 50 epochs was used to train the model for lower RMSE training error for all vehicle types except TW. The 70 epochs was used to train the TW type vehicle model.

Table 8.8 Characteristics of the best structure of ANFIS architecture

Vehicle type	Optimization algorithm	Training error	Epochs
TW	Hybrid	0.022568	70
3W	Hybrid	0.081484	50
CB	Hybrid	0.069125	50
LCV	Hybrid	0.084698	50
HCV	Hybrid	0.20302	50
BUS	Hybrid	0.35374	50
MAV	Hybrid	0.51569	50

8.6.3 Comparison of MLR, ANN and ANFIS models

The RMSE and MAPE statistical tools were used to compare the accuracy of the ANFIS, ANN and MLR models in estimating the PCU. Comparison of the performances of ANFIS, ANN and MLR models for estimation of PCU of different vehicle types in training period are presented in Table 8.9.

Table 8.9 Comparison of performances of the MLR, ANN and ANFIS models

	Linear regression		ANN		ANFIS	
	RMSE	MAPE	RMSE	MAPE	RMSE	MAPE
TW	0.44	0.49	0.38	0.48	0.22	0.10
3W	1.74	0.72	1.48	1.07	0.19	0.09
CB	0.91	0.60	0.94	0.80	0.86	0.34
LCV	1.32	0.71	1.17	0.82	0.04	0.00
HCV	3.45	0.88	1.19	1.85	0.24	0.07
BUS	5.32	0.65	3.91	1.77	0.35	0.07
MAV	12.51	31.75	6.07	3.70	0.59	0.55

ANFIS showed the best estimation performance; namely, the lowest RMSE values were obtained when the data was modelled using ANFIS. When the calculated MAPE values were taken into consideration; however, ANN was observed to exhibit the best prediction performance because the lowest MAE values were obtained. On the other hand, the MLR had the lowest accuracy regarding the RMSE and MAE statistical accuracy testing tools. These results meant the behaviour of the inputs and output was non-linear. ANFIS model provides the better accuracy then MLR and ANN model values for all the subject vehicle types as the PCU values estimated from the ANFIS model are closer to the simulated PCU values. It is also concluded that the ANFIS model showed a greater potential in predicting PCUs using volume-to-capacity ratio and proportional share for vehicle type than the conventional methods**.**

8.7 Summary

MLR models were developed for estimating the PCU of each vehicle type considering the effect of volume to capacity (v/c) ratio and the percentage share of subject vehicle. VISSIM was used for simulating the field conditions. The model developed was applied to the field data to obtain the capacity in pcu/hr. The capacity obtained from the proposed model and from the conventional method were compared. Also, the effect of no. of lanes on PCU was also found. ANN and ANFIS models were developed for estimating the PCU.

Chapter 9

SUMMARY AND CONCLUSIONS

9.1 Summary

Present study is performed to analyse the traffic flow behavior by developing the fundamental relations between traffic characteristics based on field data as well as through traffic simulation. Field data was collected by video-graphic method on different midblock sections of multilane highways. The speed and volume data was analysed by classifying the vehicles into eight different categories such as Standard Cars (CS), Big Cars (CB), Two-wheelers (TW), Light Commercial Vehicles (LCV), three wheelers (3W), Heavy Vehicles (HV), Multiaxle Vehicles (MAV) and Bus (B). Speed data collected on sections are used to develop statistical distribution profiles for different vehicle types observed in the field. Headway data and lateral positions of vehicles were observed from field. Normal and Beta distribution (with 4 parameters) was observed to fit lateral placement data on four-lane highway. Classified traffic volume obtained from the field were converted in terms of PCU by applying various available methods. Modified methods were proposed for estimation of PCUs of all subject vehicle types by using field data. Speed-volume realtionship was developed using field data to analyse traffic flow behaior. VISSIM microscopic simulation tool is used to generate data and analysis was performed to test simulation parameters and model parameters based on performance measures like speed and volume etc. Statistical tests have been performed to check the sensitivity of the different simulation parameters. Calibration is done using trial and error method by altering the driver behavior model parameters those are significantly affecting traffic flow behavior and capacity. The simulated maximum volume on a four-lane divided section with 1.5 m paved shoulders was found as 5147 veh/hr with the calibrated model parameters which is found as close as 4958 veh/hr which is the target capacity of the section. Finally, validation of calibrated model parameter was performed using field data collected on other section of a multilane highway which has given satisfactory results.

The analysis on lane changes behaviour and capacity was performed through VISSIM traffic simulation model as calibrated and validated in present study. The number of lane changes on four lane, six lane, and eight lane divided sections were quantified and its relationship with traffic volume was analysed. Maximum number of lane changes and number of lane changes at capacity level of volume was observed on different sections of multilane highways. Further, effect of different proportional share of vehicle types mixed in all cars type traffic stream was analysed on number of lane changes. Three general lane changing models are proposed to estimate the maximum number of lane changes for four lane, six lane and eight lane divided highways. Capacity of four-lane, six-lane and eight-lane divided sections were also determined through simulated data. Finally, a multiple linear models were developed to estimate PCU value of each vehicle type by considering volume to capacity ratio (v/c) and the percentage share of subject vehicle type as independent variables. ANN and ANFIS models are also developed for PCU estimation and performed comparative analysis among MLR ,ANN and ANFIS models.

9.2 Conclusions

The following conclusions are drawn from the present study.

1. Different methods given in the literature to calculate PCU value of vehicle types are not found realistic under the traffic flow conditions as observed in field. However, homogenisation method and dynamic PCU method have provided better results.
2. Modification to dynamic PCU method done by adding the time headway factor has given realistic results. The modified dynamic PCU method provides relatively higher values for large vehicle types than those obtained from dynamic PCU method.
3. A new multiple non-linear regression (MNLR) method has been developed to estimate more accurate equivalency units of a subject vehicle type under highly heterogeneous traffic conditions.

4. Normal distribution failed to fit the lateral placement data observed in the field as the placements of vehicles was found more on the inner lanes because fluctuations observed along width of road in field data.

5. While testing simulation parameters, random seed numbers (RSN) which has potential to alter the simulation results are found to be insignificant to influence volume outputs when tested at 5% level of significance. However, the RSN value giving the least percentage error between the field and simulated volume was used in the study.
6. Sensitivity analysis performed by altering driving behavior model parameters namely CC0, CC1 and CC2 are found to be influential on simulated capacity at 5% level of significance. As CC1 parameter value increases the simulated capacity reduces consistently.
7. A capacity equation formed based on multiple linear regression model to find value of target capacity by providing CC0, CC1 and CC2 parameters as inputs. The study suggests the range of CC0, CC1 and CC2 parameters as 0.50 to 4.9, 0.45 to 1.63 and 2.1 to 8.3 respectively, to be given as inputs for simulating multilane highway section to achieve target capacity.
8. Lane changing of vehicles found as an important parameter which has a significant effect on capacity of multilane highways. Frequency of lane changes are found to be increased gradually as traffic volume increased on simulated section up to some extent. Further, decrease in lane changes were observed as traffic volume reached to higher level. However, no further increase or decrease was observed in lane changes of vehicles at capacity level of volume. It is concluded that if traffic volume level reaches to capacity, lane changing opportunity of vehicles reduces due to unavailability of vacant spaces on a roadway.

9. The value of maximum lane changes was estimated separately on simulated four-lane, six-lane and eight-lane sections. It increases with increase in proportional share of 2Ws and reduces with the increase in proportional share of heavy vehicle (HV) in under mixed traffic. Maximum lane changes found to increase with increase in addition of extra lanes to the directional traffic stream. Moreover, maximum lane changes are found inversely proportional to the proportional share of HV and it increases linearly from four-lane to eight-lane divided sections.

10. Maximum numbers of lane change on a highway is found dependent on vehicle types sharing their respective proportions in the mixed traffic stream. Present study proposed as general model to quantify the maximum lane change capacity of mixed traffic stream over 1 km length of highway.

11. Capacity on simulated four lane, six lane and eight lane divided sections having width of 7.0 m, 10.5 m and 14.0 m are determined as 4786, 6341 and 8046 PCU/hr/dir respectively by developing speed-volume relationships. It may be inferred that the capacity of 7.0 m carriageway section increases by 22.0% with the addition of extra lane to one direction of travel.

12. Capacity of per lane section was also estimated by considering width of each directional lane as 3.5 m. Per lane capacity of divided four-lane section was reduced by 11.65% with the addition of one extra lane in directions of travel. The reduction has been found to be only 4.8% in case of six-lane divided section. Addition of every single lane, increases the opportunity of lane changing but reduces the per lane capacity of a multilane divided highway section.

13. Calibrated VISSIM generated the field traffic conditions for development of PCU equations. The PCU equations are established by using dynamic PCU expression (Chandra, et al. 1997). Volume to capacity ratio and traffic mix both were found as significant variables for PCU estimation. However, the effect of percentage share is not significantly observed on PCU values of vehicle type CB under lower to higher traffic volume conditions.

14. The effect of number of lanes on PCU was studied and it was observed that the PCU of each vehicle type decreases with increase in number of lanes and at a different volume to capacity ratios.

15. From the comparison of MLR, ANN and ANFIS models, it may be concluded that the ANFIS model showed a greater potential in predicting PCUs of subject vehicles type than the MLR and ANN methods**.**

9.3 Limitations of the study

- Modified dynamic PCU method developed under relatively lower to medium traffic flow level provides no evidence about PCUs under higher to maximum traffic flow level based on field data.
- Lane changing and overtaking of vehicles are not considered for calibration and validation.

9.4 Scope for further research

- The study can be continued in future to observe the variation in PCU based on MNLR method for different composition of vehicle types using a simulation technique under lower to higher volume conditions.
- Field lane change data will be collected and it verify the capacity of road section.
- Validation of lanechange models may be performed through simulation by collecting field data.

APPENDIX-A

POPULAR METHODS OF PCU ESTIMATION

A.1 General

There are different methods adopted for the estimation of PCU by various researches and organizations in various parts of world. The important methods are: (i) Homogenization Coefficient method; (ii) Walker’s method; (iii) Headway method; (iv) Simultaneous equations method; (v) Multiple linear regression method; (vi) Simultaneous equation method; and (vii) Dynamic PCU method.

A.2 Homogenization Coefficient Method

As the name imply, different types of traffic stream are expressed by the term homogenization coefficient. It is the earliest method of PCU estimation and is based on the methodology that the PCU value of a vehicle type is calculated by taking the ratio of the theoretical maximum capacity of the subject vehicle type to passenger car only. Basically this method compares the traffic stream which contains all vehicles as passenger cars and all vehicles as other than passenger car. The estimate PCU values of subject vehicle type using this method following equation is proposed.

$$PCU_i = (L_i/V_i)/(L_c/V_c) \tag{A.1}$$

Where, L and V are the length (m) and speeds (km/hr) of vehicle type, subscript i indicate the subject vehicle type and subscript c indicate the passenger car.

A.3 Walker’s Method

This method is proposed by Walker (1957) and the concept behind this method is based on the numbers of overtaking performed in one kilometer length of highway if each vehicle continued at its normal speed. The method pointed the difference in operating capabilities between the heavy vehicles and the passenger cars. The passenger car unit is calculated as the ratio of numbers of overtaking when traffic stream has one slow moving vehicle per hour to the number of overtaking when there are passenger car of equal volume.

$$N = \sum_{i=1}^{n} \sum_{i=1}^{n} x_i y_i [\frac{1}{s_{2i}} - \frac{1}{s_{1i}}] \tag{A.2}$$

Where, N is the sum of overtaking in terms of vehicles travelling at speed S_1, that will overtake x vehicle per hour travelling at speed S_2 within one km of highway. If N_1, and N_2 are given as desired number of overtaking in a stream of 100 passenger cars per hour than the PCU of truck is calculated by equation (A.3).

$$\text{PCU of Truck} = \frac{100N_2}{N_1} \tag{A.3}$$

A.4 Headway Method

Headways have been used in many of the popular methods of PCU estimation to account for the primary effect of heavy vehicles in the traffic stream as they take more space than single passenger car. The equation (A.4) is given below for PCU estimation by headway method.

$$PCU = [(\frac{h_m}{h_c}) - c] \big/ t \tag{A.4}$$

Where h_m, and h_c are the time headway of mixed vehicle and passenger cars; c is the proportion of cars in the traffic stream; t is the proportion of commercial vehicles in traffic stream. This method is suitable for high density conditions but in mix traffic adaptation of method is very limited.

A.5 Simultaneous Equation Method

PCU of a vehicle can be calculated by solving simultaneous equations using the concept of headway. In this approach the equivalency factor of a subject vehicle type is the ratio between the headways maintained by the subject vehicle to the standard vehicle. Flow of car traffic and flow of mixed traffic is calculated from the average headways of the car traffic stream and mixed traffic stream.

A.6 Multiple Linear Regression Method

In statistics, regression analysis is a technique for modelling and analyzing variables. It focuses on relationships between a dependent variable and one or more independent variables. Regression analysis is divided into two categories, one is asynchronous regression and other one is synchronous regression. The speed of the car is regress against volume of different types of vehicle where speed is considered as dependent variable. The generalized form of multiple linear regression equation (A.5) is given as

$$V_1 = A_0 + A_1Q_1 + A_2Q_2 + \ldots.. + A_nQ_n \qquad \text{(A.5)}$$

Where V_1 and Q_1 are the speed and the volume of cars; Q_2, Q_3…...Q_n are the volume of vehicle type 2, 3 … n; A_1 and A_2 are the regression coefficient; A_0 is a constant. Therefore, PCU value of vehicle type n is given as

$$PCU_n = A_n/A_1 \qquad \text{(A.6)}$$

A.7 Method of Simulation

In general, simulation is defined as dynamic representation of some part of real world achieved by building a computer model and moving it through time. The use of computer simulation started when Gerlough (1955) published his dissertation: "Simulation of freeway traffic on a general-purpose discrete variable computer" at university of California. Thereafter, methods of simulation become popular for various kinds of traffic related analysis. Method of simulation is an alternative approach employed with various input parameters required for design and analysis. By considering the heterogeneous traffic conditions and week lane discipline behaviour microscopic simulation model VISSIM is also popularly used for estimation of PCU.

A.8 Dynamic PCU method

Chandra et al. (1995) proposed a method for estimation of PCU value for different vehicles under mixed traffic situation. The basic concept used in this method is that the PCU value is directly proportional to the speed ratio and inversely proportional to the space occupancy ratio with respect to the standard design vehicle that is passenger car. Equation (2.18) for estimating PCU values of any subject vehicle type.

$$PCU_i = (V_c/V_i)/(Ac/A_i) \tag{A.7}$$

Where,

PCU_i = Passenger car unit value of the i^{th} vehicle.

V_c/V_i = Speed ratio of the car to the i^{th} vehicle.

Ac/A_i = Space ratio of the car to the i^{th} vehicle.

The variable speed ratio in the above equation is a function of roadway and traffic conditions. Any change in these conditions will affect the speed of vehicles, which is duly reflected by the changes in the speed ratio. The speed of any vehicle type will be true representation of overall interaction of vehicle type due to presence of other vehicles of its own category and other category. The second variable of space ratio represents roadway occupancy which is a surrogate measure of density.

REFERENCES

1. Abraham, J. M. (2001). *Analysis of highway speed limits.* Bachelor of Applied Science Thesis, Department of Civil Engineering, University of Toronto.
2. Ackroyd, L. W., and Bettison, M. (1979). "Vehicle speeds on the Ml motorway in Nottinghamshire-1974-78." *Traffic Engineering and Control*, 20(5), 254-255.
3. Aggarwal, P. (2008)."Artificial Neural Network Model for Passenger Car Unit." *Applied Computing Conference.*
4. Akcelik, R., and Chung, E. (1994). "Calibration of the bunched exponential distribution of arrival headways." *Road and Transport Research*, 3(1), 42-59.
5. Ahmed, S.A. (2005). *Calibration of VISSIM to the traffic conditions of Khobar and Dammam, Saudi Arabia.* Undergraduate thesis, KING FAHD UNIVERSITY OF PETROLEUM & MINERALS, SAUDI ARABIA.
6. Ah-Semeida, M. (2013). "New models to evaluate the level-of-service and capacity on multilane highways in Egypt." *Alexandria Engineering Journal*, *52, 3,* 455–466.
7. Anand, S., Sekhar, S.V.C., and Karim, M. R. (1999). "Development of Passenger Car Unit values for Malaysia." *Journal of the Eastern Asia Society for Transportation Studies*, 3(3), 1-9.
8. Arasan, V. T. and Arkatkar, S.S. (2010). "Micro-simulation Study of Effect of Volume and Road Width on PCU of Vehicles under Heterogeneous Traffic." *Journal of Transportation Engineering*, 136 (12), 1110–1119.
9. Arasan, V.T and Arkatkar, S.S. (2011). "Derivation of capacity standards for intercity roads carrying heterogeneous traffic using computer simulation." *6^{th} International Symposium on Highway Capacity and Quality of Service, Procedia Social and Behavioral Sciences*, Elsevier, 16, 218–229.
10. Arasan, V.T., and Koshy, R. Z. (2003). "Headway distribution of heterogeneous traffic on urban arterials." *Institution of Engineers (India),* 84, 210-215.
11. Arasan, V. T. and Krishnamurthy, K. (2008)."Effect of traffic volume on PCU of vehicles under heterogeneous traffic conditions." *Road and Transport Research Journal, Australian Road Research Board*, 17(1), 32-48.

12. Biswas, S., Chakraborty, S., Chandra, S., and Ghosh, I. (2017). "Kriging-Based Approach for Estimation of Vehicular Speed and Passenger Car Units on an Urban Arterial." *Journal of Transportation Engineering, Part A: Systems,* 143 (3), 04016013.
13. Breiman, L. (1963). "The Poisson tendency in traffic distribution." *Ann. Math. Statist.,* 34, 308-311.
14. Brown, M. (1972). "Low-density traffic streams."*Adv. Appl. Prob*., 4, 177- 192.
15. Cao, N.Y and Sano, K. (2012). "Estimating Capacity and Motorcycle Equivalent Units on Urban Roads in Hanoi, Vietnam." *Journal of Transportation Engineering,* 138(6), 776-785.
16. Casey, S. M., and Lund, A. K. (1992). "Changes in speed and speed adaptation following increase in national maximum speed limit." *Journal of Safety Research*, 23,135-146.
17. Central Road Research Institute (CRRI) (1982). *Road user cost study in India*, Final report, New Delhi.
18. Chandra, S., Arora, M. K, and Sahu, V. K. (2001). "Headway modelling on urban roads using artificial neural network." *Highway Research Bulletin*, 65, Highway Research Board, Indian Roads Congress, New Delhi, 127-137.
19. Chandra, S. and Kumar, U. (2003). "Effect of lane width on capacity under mixed traffic conditions in India." *Journal of Transportation Engineering,* ASCE, 129 (2), 155-160.
20. Chandra, S., Kumar, V. and Sikdar, P. K. (1995). "Dynamic PCU and Estimation of Capacity of Urban Roads." *Indian Highways,* Indian Roads Congress, New Delhi, 23(4).
21. Chandra, S., and Raj, D. (1999). "Role of shoulders in traffic operation." *Indian Highways*, 27(11), 31-38.
22. Chandra, S., and Sikdar, P. K. (2000). "Factors affecting PCU in mixed traffic on urban roads." *Road and Transport Research,* ARRB, 9(3), 40-50.
23. Chandra, S., Zala, L.B. and Kumar, V. (1997). "Comparing the methods of passenger car unit estimation." *Journal of Institution of Engineer*, 78, 13-16.
24. Chang, M. and Kim, Y. (2000). "Development of capacity estimation method from statistical distribution of observed traffic flow." *Transportation Research Circular E-C018, Fourth International Symposium on Highways Capacity,* 299-308.
25. Chitturi, M.V. and Benekohal, R.F. (2008). "Calibration of VISSIM for freeway." *87th TRB Annual Meeting Compendium of paper DVD*, Washington DC.
26. Christopher, R. (1994). “A speed prediction model for rural two-lane highways.” Bennett.

27. Craus, J., Polus, A. and Grinberg, I. (1980). "Revised Method for the Determination of Passenger Car Equivalencies." *Transportation Research-A*, 14, 241–246.
28. Dawson, R. F. and Chimni, L. A. (1968). "The hyperlang probability distribution – a generalized traffic headway model." *Highway Research Record*, 230, Highway Research Board, National Research Council, Washington, D.C, 1-14.
29. Dey, P. P., Chandra, S., and Gangopadhaya, S. (2006a). "Lateral distribution of mixed traffic on two-lane roads." *Journal of Transportation Engineering*, ASCE, 132(7), 597-600.
30. Dey, P. P., Chandra, S., and Gangopadhaya, S. (2006b). "Lateral placement of vehicles under mixed traffic conditions." *Indian Highways*, *Indian Roads Congress*, New Delhi, 34(9), 9-17.
31. Dixon, K. K, Wu, C-H, Sarasua, W., and Daniel, J. (1999a). "Posted and free-flow, speeds for rural multilane highways in Georgia." *Journal of Transportation Engineering*, ASCE, 125 (6), 487-494.
32. Dixon, K. K, Wu, C-H, Sarasua, W., and Daniel, J. (1999b). "Estimating free-flow speeds for rural multilane highways." *Transportation Research Record*, 1678, Transportation Research Board, National Research Council, Washington DC, 73-82.
33. Donald, B., and Daniel, M. B. (1951). “Distribution of Vehicle Speeds and Travel Times.”
34. Drake, J.S., Schofer, J.L. and May, A. D. (1967). "A Statistical analysis of speed density hypotheses." *Highway Research Record, National Research Council*, Washington DC, 154, 53-87.
35. Duncan, N.C. (1974). "Rural speed-flow relations." *Transport and Road Research Laboratory, TRRL 651*, Crowthorne, Berkshire, England
36. Edara, P. and Chaterjee, I. (2010). "Multivariate regression models to estimate driving behavior parameters in work zone capacity simulation." *Transportation Letter*, 2, 3, 175-186.
37. Fellendorf, M. and Vortisch, P. (2001). "Validation of the microscopic traffic simulation model VISSIM in different real-world situations." *80th Annual Meeting of the TRB Compendium of Papers DVD, Transportation Research Board,* Washington DC.
38. Fellendorf, M. and Vortisch, P. (2010). "Microscopic Traffic Flow Simulator VISSIM. In: Barceló." *J., Ed., Fundamentals of Traffic Simulation, International Series in Operations Research & Management Science,* Springer, New York, 145, 63-93.

39. Filippo, G.P., and Marinella, G. (2011). "Speed Distribution on Low-Volume Roads in Italy: From Inferences to Rehabilitation Design Criteria." *Transportation Research Record*, 2203 (1), 79-84.
40. Gowri, A. and Shravani, G. (2017). "Overtaking behaviour of vehicles on undivided roads in non-lane based mixed traffic conditions." *Journal of Traffic and Transportation Engineering (English Edition)*, 4(3), 252-26.
41. Greenshields, B.D. (1934). "A study of traffic capacity." *Proceeding of Highway Research Board*, 14, 448-477.
42. Greenberg, H. (1959). "An analysis of traffic flow." *Operations Research*, 7, 78-85.
43. Guinn, D.W., Reilly, E.F. and Seifert, J. (1970). "Truck Equivalency." Final Report, *Bureau of Safety on Traffic*, *New Jersey Department of Transportation*, New Jersey.
44. Haight, F. A., Whisler, B. and Mosher,W. W. Jr. (1961). "New statistical method for describing highway distribution of cars." *Proceedings*, *Highway Research Board, 40th Annual Meeting*, Washington, D. C, 557-564.
45. Highway Capacity Manual (1965). *Transportation Research Record*, National Research Council, (First Edition), Washington DC.
46. Highway Capacity Manual (2010). *Transportation Research Board of the National Academies*, (5th Edition, *Vol. 2)*, Washington DC
47. Hill, C., Elefteriadou, L., and Kondyli, A. (2015). "Exploratory Analysis of Lane Changing on Freeways Based on Driver Behavior." *J. Transp. Eng.*, 141(4), 04014090.
48. Hoban, C.J. (1987). "Evaluating traffic capacity and improvements to road geometry." *The World Bank Technical Papers, 74*, Washington DC.
49. Hu, Y. and Johnson, R. (1981). *Passenger Car Equivalents of Trucks in Composite Traffic*. Report DTFH-61-80-c-00127, FHWA, U.S. Department of Transportation.
50. Huang, D.W. (2002). "Lane-changing behavior on highways." *Physical Review E*, 66(1), 1-5.
51. Huber, M.J. (1982). "Estimating of passenger cars equivalent of truck in traffic stream." *Transportation Research Record, 869, TRB*, National Research Council, Washington DC, 60-70.

52. Hunt, P., Larocque, B. and Gienow, W. (2004). "Analysis of 110 km/h speed limit: Implementation on Sabkatchewan divided rural highway." *Proceeding of Transportation Association of Canada (TAC) Annual Conference*, Quebec City, Quebec.
53. Hustim, M., and Ramli, M. I. (2013). "The Vehicle Speed Distribution on Heterogeneous Traffic: Space Mean Speed Analysis of Light Vehicles and Motorcycles in Makassar – Indonesia." *Proceedings of the Eastern Asia Society for Transportation Studies*, 9.
54. Hyde, T. and Wright, C. (1986). "Extreme Value Methods for Estimating Road Traffic Capacity." *Transportation Research B*, 20(2), 125-138.
55. Indonesian Highway Capacity Manual (IHCM, 1996), Directorate General, Bina-Marga, Republic Indonesia.
56. IRC: 64 (1990). Guidelines for capacity of roads in rural areas, *Indian Roads Congress*, New Delhi, 1990.
57. Jain, S. S., Naqvi, H. M., Patil, S. N., and Parida, M. (2005). "Analysis of traffic operation on Mumbai-Pune expressway." *Proceedings of seminar on High Speed Road Corridors*, 23-24 September, Vigyan Bhawan, NewDelhi.
58. Jalihal, S. A., Kayitha, R. and Reddy, T. S. (2005). "Traffic characteristics of India." *Proceedings of the Eastern Asia Society for Transportation Studies*, 5, 1009 - 1024.
59. Jian, R., Xiamaming, L. and Futian, R. (2000). "Vehicle classification for expressway capacity analysis in China." *Transportation Research Circular E-C018, Fourth International Symposium on Highways Capacity*, National Research Council, Washington DC, 130-141.
60. John, A., and Glauz, W. (1976). "Speed and Service on Multilane Upgrades." *Transportation Research Record*, 61, 4-9.
61. Johnson, N. L., and Kotz, S. (1969). "Distributions in statistics discrete distributions." Houghton Mifflin Company, USA.
62. Joseph, C., Polus, A. and Grinberg, I. (1980). "A revised method of the determination of PCE." *Transportation Research*-A, *Record* 869, TRB, National Research Council, Washington DC, 14, 241-246.
63. Kadiyali, L.R., Vishwanathan, E. and Gupta, R. K. (1981). "Free-speed of vehicles on Indian Roads." *Journal of Indian Roads Congress*, 343, 387-458.

64. Kadiyali, L.R., Viswanathan, E., Gupta, R. K. and Unmat, S. K. (1982). "Speed-volume studies on rural highways in India." *Journal of Indian Roads Congress*, 35, 43(3), 359-435.
65. Kadiyali, L.R. and Vishwanathan, E. (1991). "Study for updating road users cost data." *Journal of Indian Roads Congress*, New Delhi, 165, 645-731.
66. Kadiyali, L. R. and Viswanathan, E. (1993). "Study for updating road user cost data." *Journal of Indian Roads Congress*, 54(3), 645-731.
67. Kadiyali, L.R. (2002). *Traffic Engineering and Transport Planning*. Khanna Publishers, New Delhi, India.
68. Kesting, A., Treiber, M., and Helbing, D. (2007). "General lane-changing model MOBIL for car-following models." *Transportation Research Record: Journal of the Transportation Research Board*, 1999, 86-94.
69. Khademi, F., Akbari, M., Jamal, S.M., and Nikoo, M. (2017). "Multiple linear regression, artificial neural network, and fuzzy logic prediction of 28 days compressive strength of concrete." *Frontiers of Structural and Civil Engineering*, 11(1), 90-99.
70. Khademi, F., Jamal, S.M., Deshpande, N., and Londhe, S. (2016). "Predicting strength of recycled aggregate concrete using artificial neural network, adaptive neuro-fuzzy inference system and multiple linear regression." *International Journal of Sustainable Built Environment*, 5(2), 355-369.
71. Khairi, S., Muhammad, A., and Intan, E. (2011). "Exploration of 85th Percentile Operating Speed Model on Horizontal Curve: A Case Study for Two-Lane Rural Highways." *Procedia - Social and Behavioral Sciences*, 16.
72. Knoop, V.L., Hoogendoorn, S.P., Shiomi, Y., and Buisson, C. (2012). "Quantifying the number of lane changes in traffic empirical analysis." *Transport Research Record*, 31–41.
73. Kumar, V. M., and Rao, S. K. (1998). "Headway and speed studies on two-lane highways." *Indian Highways*, 26 (5), 23-36.
74. Laufer, J. (2007). "Freeway capacity, saturation flow and the car following behavioral algorithm of the VISSIM micro-simulation software." *30th Australasian Transport Research Forum*, Australia.
75. Laval, J. A. (2007). "Delays caused by trucks on multilane freeways: impacts of overtaking bans for trucks." *Proceeding, Transportation Research Board, 86th Annual Meeting*, Washington D.C., Compendium of Papers CD-ROM.

76. Lees, J. (2003). "A simulation of traffic flow on highways." *Senior Research Paper*, Stetson University.
77. Lewis, R. (1963). "A proposed headway distribution for traffic simulation studies." *Traffic Engineering*, 33, 16-19.
78. Lownes N.E. and Machemehl, R.B. (2006). "Sensitivity of simulated capacity to modification of VISSIM driver behavior parameters." *TRR 1988, Transportation Research Board, National Research Council,* Washington DC, 102-110.
79. Lownes N.E. and Machemehl, R.B. (2006). "VISSIM: A multi-parameter sensitivity analysis." *Proceedings of 2006 Winter Simulation Conference*, Monterey, California.
80. Madhu, E., Velmurugan, S., Ravinder, K. and Natraju, J. (2011). "Development of free-speed speed equation for assessment of road-user cost on high-speed multi-lane carriageways of India on plain terrain." *Journal of Current Science*, 100 (9), 1362-1372.
81. Mahalel, D. and Hakkert, A.S. (1983). "Traffic arrival patterns on a cross section of multilane highway." *Transportation research-A*, 17,263-270.
82. Mahapatra, G. and Maurya, A.K. (2015). "Study on Lateral Placement and Speed of Vehicles under Mixed Traffic Condition." *Eastern Asia Society for transportation Studies*, Cebu City, Philippines.
83. Mardani, N. M., Ghosh, I., and Chandra, S. (2016). "Determination of Passenger Car Units on Two Lane Intercity Highways under Heterogeneous Traffic Conditions." *Journal of Transportation Engineering*, 142(2), 548–556.
84. Mehar, A., Chandra, S., Velmurgan, S. (2014)."PCU at different levels of Service for capacity analysis of multilane Interurban highways in India." *Journal of Transportation Engineering*, 140, 81-88.
85. Mehar, A., Chandra, S., Velmurugan, S. (2013). "Highway Capacity Through Vissim Calibrated for Mixed Traffic Conditions." *KSCE Journal of Civil Engineering*, 18(2), 639-645.
86. Mehar, A., Chandra, S., Velmurugan, S. (2015). "Effect of Traffic Composition on Capacity of Multilane Highways." *KSCE Journal of Civil Engineering*, 1-8.
87. Mei, M., and Bullen A. G. R. (1993). "The log-normal distribution for high traffic flow." *Transportation Research Record*, 1398, Transportation Research Board, National Research Council, Washington, D. C, 125-128.

88. Menneni, S., Carlos, S. and Vortisch, P. (2007). "Micro-simulation calibration using speed flow relationships." *TRR 1981, Transportation Research Board, National Research Council,* Washington DC, 1-9.

89. Menneni, S., Sun, C., and Vortisch, P. (2008). "Micro-simulation Calibration Using Speed-Flow Relationships." *Transportation Research Record: Journal of the Transportation Research Board*, 2088, 1-9.

90. Mishra, R., Kumar, P., Arkatkar, S.S., Sarkar, A.K., and Joshi, G.J. (2017). "Novel Area Occupancy–Based Method for Passenger Car Unit Estimation on Multilane Urban Roads Under Heterogeneous Traffic Scenario." *Transportation Research Record: Journal of the Transportation Research Board*, 2615(1), 82-94.

91. Moridpour, S., Sarvi, M., and Rose, G. (2010). "Lane Changing Models: A Critical Review." *Transportation Letters: The International Journal of Transportation Research*, 2(3), 157- 173.

92. Nagaraj. B. N., George, K. J., and John, P. K. (1990). "A study on linear and lateral placement of vehicles in mixed traffic environment through video-recording." *Highway Research Bulletin*, 42, Highway Research Board, Indian Roads Congress, New Delhi, 105-136.

93. Nakamura, M (1994). "Research and application of highway capacity in Japan." *Proceedings of 2nd International Symposium on Highway Capacity*, Sydney, Australia, 104–111.

94. Newell, G. F. (1955). "Mathematical models for free flowing highway traffic." *Opns. Res*., 3, 176-186.

95. Newell, G. F. (1966). "Equilibrium probability distributions for low density highway traffic." *J. Appl. Prob*., 3, 247-260.

96. Nielsen, O.A. and Jorgensen, R.M. (2008). "Estimation of speed-flow and flow-density relations on the motorway network in the Greater-Copenhagen-Region." *Journal of IET Intelligent Transportation Systems*, 2, 120-131.

97. Oppenlander, J. C. (1966)."Variables influencing spot-speed characteristics." Special Report 89, Highway Research Board, NAS-NRC Publication 1332, Washington, D.C, 1-39.

98. Panichpapiboon, S. (2015). "Time-Headway Distributions on an Expressway: Case of Bangkok." *Journal of transportation engineering*, 141(1), 05014007.

99. Praveen, A. (2008). "Artificial Neural Network Model for Passenger Car Unit". *Applied Computing Conference.*

100. Putranto, L.S. and Setyarini, N.S. (2011). "Vehicle composition and lane distribution in multilane highways in Indonesian cities." *Procedia Social and Behavioral Sciences*, Elsevier, 16, 374-381.

101. Ramanayya, T. V. (1980). "Simulation studies on traffic capacity of road system for Indian condition." Ph. D. Thesis, Department of Civil Engineering, R. E. C. Warangal, India. (Unpublished).

102. Ramanayya, T. V. (1988). "Highway capacity under mixed traffic conditions." *Traffic Engineering and Control*, 29(5), 284-287.

103. Rao, S. K. and A. K. Mookerjee (1997). "Traffic capacity, LOS and lane requirement analysis for expressway planning in India." *Indian Road Congress Journal, 58, 509-527.*

104. Reddy, K. S., and Pandey, B. B. (1995). "Lateral placement of commercial vehicles on highways." *Highway Research Bulletin*, 53, Highway Research Board, Indian Roads Congress, New Delhi, 63-84.

105. Roess, R. P. and Messer, C. J. (1984). "Passenger car equivalents for uninterrupted flow: Revision of circular 212 values." Transportation Research Record: *Journal of the Transportation Research Board, No. 971*, National Research Council, Washington DC, 7–13.

106. Rongviriyapanich, T. and Suppattrakul, C. (2005). "Effects of motorcycles on traffic operations on arterial streets." *Journal of the Eastern Asia Society for Transportation Studies*, 6, 137 – 146.

107. Rotwannasin, P. and Choocharukul, K. (2005). "Transferability of HCM to Asian countries: An exploratory evidence from Bangkok’s multilane highways." *Proceedings of the 3rd International SIIV Congress.*

108. Ryan, D. P., and Breuning, S. M. (1962). "Some fundamental relationships of traffic flow on a freeway." *Proceedings, Highway Research Board, 41st Annual Meeting*, Washington, D.C., 73-84.

109. Sahoo, P. K., Rao, S. K., and Kumar, V. M. (1996). "A study of traffic flow characteristics on two stretches of National Highway No.5." *Indian Highways*, 24 (4), 11-17.

110. Schul, A. (1955). "The probability theory applied to distribution of vehicles on two lane highways." *The Eno foundation for highway traffic control*, Saugatuck.

111. Shankar, V., and Mannering, F. (1998). "Modeling the endogeneity of lane-mean speeds and lane-speed deviations: a structural equations approach." *Transportation Research Part A*, 32(5), 311-322.

112. Sharma, N., Sarkar, P.K., Kathuria, A., and Velmurugan, S. (2014). "Estimation of capacity for multilane divided national highways in India." *Journal of Indian Roads Congress*, 75(3), 271–277.

113. Sinha, S., Taneerananon, P., and Chandra, S. (2012). "A comparative study of effect of motorcycle volume on capacity of four-lane urban roads in India and Thailand." *Journal of Eastern Society of Transportation and Traffic Studies (JSTS)*, 3(1), 52-62.

114. Shukla, S. (2008). *Traffic flow modeling on multilane divided highways.* PhD Thesis, Indian Institute of Technology Roorkee, India.

115. Shukla, S. and Chandra, S. (2011). "Simulation of mixed traffic flow on four-lane divided highways." *Journal of Indian Roads Congress*, 567, 55-69.

116. Sparmann, U. (1979). "The importance of lane-changing on motorways." *Traffic Engineering and Control*, 20(6), 320-323.

117. St. John, A. D. (1976). "Nonlinear Truck Factor for Two-Lane Highways." Transportation Research Record: *Journal of the Transportation Research Board, No. 615*, National Research Council, Washington, D.C., 49–53.

118. Tanaboriboon Y. and R. Aryal (1990). "Effect of vehicle size on highway capacity in Thailand." *Journal of Transportation Engineering, ASCE*, 116 (5), 658-666.

119. Thedeen, T. A. (1964). "A note on the Poisson tendency in traffic distribution." *Ann. Math. Statist*, 35, 1823-4.

120. Tiwari, G., Fazio, J. and Gaurav, S. (2007). "Traffic planning for non- homogeneous traffic." *Sadhana*, 32(4), 309–328.

121. Tolle, J. E. (1971). "The lognormal headway distribution model." *Traffic Engineering and Control*, 12(5), 22-44.

122. Tseng, P. Lin, F. and Sheng, S. (2005). "Estimation of free-flow speed for multilane rural and suburban highways." *Journals of Eastern Asia Society for Transportation Studies*, 6, 1484-1495.

123. Underwood, R.T. (1961). "Speed, volume and density relationships." *Quality and Theory of Traffic Flow, Yale Bureau of Highway Traffic*, Yale University, 141-188.

124. Updating Road User Cost Data (2001). Final Report, Central Road Research Institute. vol.1, New Delhi.

125. Vaziri, M. (1995). "Traffic flow characteristics for Iranian freeways." *Proceedings, International Conference on New Horizons in Roads and Road Transport (ICORT 95)*, December 11-14, Indian Institute of Technology-Roorkee, India, 1398-1403.

126. Velmurugan, S., Errampalli, M., Ravinder, K., and Sitaramanjaneyulu, K. (2010). "Critical evaluation of roadway capacity of multilane high speed corridors under heterogeneous traffic condition through traditional and microscopic simulation model." *Journal of Indian Roads Congress*, 566, 253-264.

127. Velmurugan, S., Madhu, E.,and Reddy, T. S. (2002). "Changing speeds of operation on rural highways." *Proceedings of National Seminar on Road Transportation in India: Emerging Trends and Techniques*, September 12-13, IIT Kharagpur, India, 5.29-5.40.

128. VISSIM: Users Manual (2012). Version 5.4, *Planning Transport Verkehr (PTV) AG*, Karlsruhe, Germany.

129. Wasielewski, P. (1974). "An integral equation for the Semi-Poisson headway distribution model." *Transportation Science*, 8 (3), 237-247.

130. Weiss, G., and Herman, R. (1962). "Statistical properties of low-density traffic." *Quart. Appl. Math.*, 20, 121-130.

131. Whitby, W. D. (1962). "Small-car speeds and spacings on urban expressways." *Proceedings, Highway Research Board, 41st Annual Meeting*, Washington, D.C, 18-23.

132. Worrall, R. D., Bullen, A. G., and Gur, Y. (1969). "Lane-changing in multilane freeway traffic." *Highway Research Record*, 299, Highway Research Board, National Research Council, Washington, D. C, No 279, P 160.

133. Xue, C., Zhao, J., and Pei, Y. (2009). "Research on the characteristics of time headway distribution in expressway weaving section of type A." *Proc., Int. Conf. Chinese Transportation Professionals (ICCTP)*, ASCE, Reston, VA, 1–7.

134. Yang, J., Wang, J., Li, Q., and Wang, Z. (2004). "The behavioral analysis and modeling of lane changes in traffic micro-simulation." *Proceedings, 8th International Conference on Applications of Advanced Technologies in Transportation Engineering*, May 26-28, Beijing, China.

135. Yang, X. and Zhang, N. (2005). "The marginal decrease of lane capacity with the number of lanes on highway." *Proceeding of International Conference of Eastern Asia Society for Transportation Studies (EASTS)*, 5, 739-749.

136. Zhou, M. and Hall, F. L. (1999). "Investigation of speed-flow relationship under congested conditions on a freeway." *CD ROM Proceedings of the 78th Annual Meeting of Transportation Research Board,* Washington, D.C.

LIST OF PUBLICATIONS

[1]. Srikanth, S. and Mehar, A., 2017. A modified approach for estimation of passenger car units of different vehicle types on intercity multilane highways. *Archives of transport*, 42(2), 65-74.

[2]. Srikanth, S. and Mehar, A., 2017. Estimation of equivalency units for vehicle types under mixed traffic conditions: multiple non-linear regression approach. *International journal of technology-civil engineering*, 8(5), 820-829.

[3]. Srikanth, S., Mehar, A. and Praveen, KGNV., 2017. Simulation of traffic flow to analyze lane change behavior on Multi-lane Highways. *Europian Transport*, 66, paper no. 2.

[4]. Srikanth, S., Mehar, A. and Parihar, A., 2017. Calibration of VISSIM model for multilane highways using speed flow curves. *Civil Engineering Journal*, 3, 303-314.

[5]. Srikanth, S. and Mehar, A., 2018. Development of MLR, ANN and ANFIS models for estimation of PCUs at different level of service. *Journal of Soft Computing in Civil Engineering* , 2(1), 18-35.

[6]. Srikanth, S. and Mehar, A., 2017. Estimation of PCU at different level of service and proportional share. *SUT Engineering journal*, 24(3), 247-261.

[7]. Srikanth, S., Mehar, A. and Praveen, KGNV., 2017. "Simulation of traffic flow to analyze lane change behavior on Multi-lane Highways under non-lane discipline." *Perodica polytechnica transportation engineering.* DOI: 10.3311/PPtr.10150