ANALYSIS AND MODELING OF PASSENGER CAR EQUIVALENTS FOR HETEROGENEOUS TRAFFIC CONDITIONS

A thesis Submitted in partial fulfilment of the requirements for the award of

DOCTOR OF PHILOSOPHY

In

Civil Engineering

By

Syed Omar Ballari

(10610415)

Under the supervision of

Dr. C. Mallikarjuna

Associate Professor

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY GUWAHATI
GUWAHATI-781039, ASSAM, INDIA
SEPTEMBER 2017

Department of Civil Engineering
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Introduction

1.1 General

Road network is essential for the economic and social development of any nation. It offers accessibility, flexibility in operations, reliability, and door-to-door service for the passenger as well as the freight transport. India has the second largest road network in the world with over 3.3 million kilometres of road length spreading across the country (NHAI website: www.nhai.org). Indian roads carry over 80% of the passenger traffic and 65% of the freight traffic (NHAI). The rural roads are broadly categorized as expressways, national highways (NH), state highways (SH), major district roads (MDR), and the other district roads (ODR). Expressways are the multi-lane highways having limited access and allow high speeds. National highways are the main highways running through the length and breadth of the country connecting major ports, state capitals, large industrial areas, and tourist centres (IRC: 73-1980). They carry about 40 percent of the total road traffic. Further, national highways are divided into rural/non-urban and urban highways depending on whether they pass through villages or built-up areas, respectively (NPTEL website: http://nptel.ac.in).

National Highways are the arterials for inter-state movement of passengers and goods. Massive projects are being taken up for expanding the length of the multilane highway network in all parts of India by the year 2022. Several two-lane rural highways are upgraded to four lanes, and various multi-lane highway projects are being sanctioned to meet the demand of higher traffic volume. For a given traffic demand, the required number of lanes are calculated based on service flow rate. Service flow rate is the maximum sustainable hourly flow rate corresponding to a particular level of service during the peak 15 minute of the hour under prevailing roadway, traffic and environmental conditions (HCM, 2010).

Vehicular traffic on the multilane highways in India consists of a wide variety of vehicles such as the motorised three-wheeler (MThW), motorised two-wheeler (MTW), motorised heavy vehicles (HMV), and non-motorised vehicles besides the passenger cars. The passenger car is the dominant vehicle class on multilane highways and contribution of HMV and passenger car together amounts for 80 percent of total traffic on six-lane and eight-lane divided highways (Mehar et al. 2014). Non-motorised vehicles contribute less than 5 percent to the overall traffic in case of four-lane highway and their presence is negligible on six-lane and eight-lane highways (Velmurugan et al. 2010). For representing the demand/service flow rate of the heterogeneous traffic stream in terms of an equivalent base stream, it is necessary to use the concept of passenger car equivalent (PCE).

Passenger Car Equivalent (PCE) was first introduced in 1965 US Highway Capacity Manual (HCM) for measuring the impact of heavy vehicles on the traffic stream. HCM (1965) defined PCE as “the number of passenger cars displaced in the traffic flow by a truck or a bus, under the prevailing roadway and traffic conditions.” PCE values were used to calculate the heavy vehicle adjustment factor (f_{HV}). Along with the other adjustment factors, the f_{HV} was then used to convert the demand or service flow rate expressed in vehicles per hour to the passenger cars per hour. US HCM (2010) provided PCEs of trucks/buses and recreational vehicles (RVs) which do not vary with the level of service (LOS) and traffic composition on multilane highways passing through the level terrain. Indian Roads Congress (IRC) provided constant PCEs for different levels of service and traffic compositions in the capacity guidelines of roads in rural areas (IRC 64:1990). Previous studies (Arasan and Arkatkar, 2010; Mehar et al. 2013) found that for multilane highways in India the PCE values vary with the LOS and traffic composition. The use of dynamic PCEs with the flow rate complicates the computations of f_{HV}. The present study, therefore, analyses the dynamic variability of PCEs with flow rate and traffic composition with the aim of developing constant PCEs for the heterogeneous traffic conditions prevalent in India.

1.2 Need for the Study

In design, the number of lanes is calculated based on the future traffic demand and the expected LOS. The operational analysis involves the determination of existing LOS for the prevailing traffic conditions. Both these analyses require the conversion of demand or service flow rate expressed in vehicles per hour to the passenger cars per hour. Estimation of the PCE values is therefore essential for the design and operational analysis of roadways. Majority of the PCE studies are meant for the traffic streams consisting of cars and heavy vehicles. The presence of smaller vehicles such as the MTW and MThW complicates the vehicular interactions and restricts the applicability of such PCE values under heterogeneous traffic conditions. Studies carried out for determining the PCEs of heterogeneous traffic mainly focused on the dynamic variability of PCEs with flow rate and traffic composition. The use of such dynamic PCEs for the capacity analysis is complicated and hence it is necessary to investigate the suitability of static PCEs with varying flow rate and traffic composition. Besides, it is also necessary to check the suitability of the aggregate PCEs.

1.3 Research Objectives

The objective of the present study is to estimate the PCE values of different vehicle types for representing the heterogeneous traffic streams moving on the multilane rural highways in India. The following are the important tasks necessary to achieve this objective:

- To perform a comprehensive literature survey for understanding the concept of PCE and the challenges associated with the estimation of PCEs.
- To analyse the macroscopic relationships developed for the heterogeneous traffic streams observed on the rural highways in India.
- To conduct a comparative analysis of the different performance measures used for estimating the PCEs of the different types of vehicles using the four-lane and six-lane divided roads.
- To analyse the effect of traffic composition and traffic flow rate on the Individual and the Aggregate PCEs for four-lane and six-lane divided roads.

1.4 Scope of the Study

The present study applies to the long stretches of rural divided highways passing through the level terrain. For this research, the selected rural highways cater to only the motorized vehicles. Further, this study is not applicable to the urban highways containing a significant amount of work zone activities.

1.5 Organization of the Thesis

This thesis consists of seven chapters. **Chapter 1** presents a brief introduction to the topics and their significance in the Indian context, the need for the study, and the specific objectives of the research work. **Chapter 2** presents the state-of-the-art literature review carried out for the study. **Chapter 3** deals with the ramifications of the traditional data collection approach for developing the macroscopic relationships under heterogeneous traffic conditions. **Chapter 4** presents the comparison of different macroscopic performance measures used for the PCE estimation on rural highways. **Chapter 5** discusses the effect of flow rate and traffic composition on individual and aggregate PCE values. Finally, **Chapter 6** provides the summary and important conclusions of the present work and the future scope of research.

Chapter 2

Literature Review

2.1 General

The review presented in this chapter has been carried out to understand the different PCE estimation methods and their implications when applied to the heterogeneous traffic conditions. The review includes a detailed step by step analysis of different PCE estimation methods. It also includes the studies related to the estimation of perceived LOS on different types of roads. This chapter has six sections. Section 2.2 discusses the concept of individual and aggregate PCE applied for representing the heterogeneous traffic stream. Section 2.3 includes the detailed discussion on the various PCE estimation methods and their shortcomings, if any, suggested by the researchers. Section 2.4 summarizes the merits and demerits of various PCE estimation methods. Section 2.5 discusses the various performance measures used to define the LOS. Section 2.6 summarizes the literature review with the associated research gaps.

2.2 Individual PCE and Aggregate PCE

Individual PCE means the PCE value of a vehicle type present in the traffic stream whereas aggregate PCE means the PCE value of an entire traffic mix. For representing a heterogeneous traffic stream by an equivalent passenger car only stream, either the aggregate or the individual PCEs can be used. Majority of the studies focused on obtaining the individual PCEs while converting a heterogeneous traffic stream into homogeneous flow. Roess and Messer (1984) stated that individual PCEs may not provide accurate representation when different types of vehicles are present in the traffic stream. In such cases, aggregate PCE may be a better representation of the heterogeneous traffic flow.

2.3 PCE Estimation Methods

Different researchers have employed different methods to estimate the PCE values, and over the years a better understanding was evolved. Krammes and Crowley (1986) stated that the PCE estimation methods are basically of two types: one that relates the concept of

PCE to the LOS and the other that "emphasizes the consideration of all factors that contributes to the overall effect of trucks on the traffic stream performance." Okura and Shapit (1995) said that PCE estimation methods could be classified into macroscopic methods and microscopic methods. Microscopic methods include Walker's method and the Equivalent delay method used in 1965 US HCM. Macroscopic methods use speed-flow-density relationships to estimate the PCE values. Besides these, several other PCE estimation methods were also developed for calculating the PCEs by various researchers over the recent years. However, the discussion is still open-ended as there is no consensus among the researchers regarding the use of a particular method.

2.3.1 Methods used for Estimating the PCEs in 1965 HCM

US HCM (1965), defined the PCE as "the number of passenger cars displaced in the traffic flow by a truck or a bus, under the prevailing roadway and traffic conditions." The PCE values for two-lane two-way highways were calculated using Walker Method which is based on the criterion of the relative number of passenger cars overtaking/passing the trucks to that of passenger cars overtaking/passing the lower performance passenger cars. The rationale for selecting this criterion was that the relative number of overtaking operations is directly related to the space headways. The mathematical formulation of Walker's method is shown in the following equation.

$$PCE_T = \frac{OT_T}{VOL_T} \times \frac{VOL_{LPC}}{OT_{LPC}} \tag{2.1}$$

Where OT_T is the number of cars overtaking the trucks per mile per hour; VOL_T is the volume of trucks per hour; OT_{LPC} is the number of cars overtaking the other cars with lower performance per mile per hour; VOL_{LPC} is the volume of cars with lower performance per hour. The fundamental assumption of Walker's method was that the faster vehicles could overtake the slower vehicles without any restriction from the opposing traffic and hence can maintain their desired speeds throughout the process. This approach neglected the number of trucks overtaking the cars as well as the other slow-moving vehicles and resulted in significantly high PCE values. For multilane highways and freeways, PCE values were derived based on the relative delay to the cars due to the presence of trucks as shown in the following equation.

$$PCE_{Tj} = \frac{D_{Tj} - D_{car}}{D_{car}} \tag{2.2}$$

Where, $PCE_{T,j}$ is the PCE of the truck under condition j, $D_{T,j}$ is the delay experienced by the cars due to the presence of trucks under condition j, D_{car} is the delay for the cars due to the other slower moving cars under condition j. This method overestimated the PCE values for the roads having grades higher than two percent. Cunagin and Messer (1983) later applied Walker's method and equivalent delay method to obtain the PCEs on rural highways under free-flow conditions and the capacity conditions, respectively. For intermediate conditions, the PCEs were taken as the linear combination of the two extreme PCE values. They concluded that for multiple types of heavy vehicles such as trucks, buses and recreational vehicles, it is better to calculate the aggregate PCE of a particular traffic mix. Roess and Messer (1984) mentioned that the microscopic methods such as Walker's method and the equivalent delay method did not relate to the concept of LOS for uninterrupted facilities. Therefore, the applicability of these methods for the LOS analysis is not rational. Later, for the LOS analysis, US HCM employed the PCE estimation methods based on the macroscopic relationships.

2.3.2 PCEs Based on the Macroscopic Relationships

Huber's Method

Huber (1982) proposed that the two traffic streams would be considered equivalent if they experience similar LOS. Performance measures used to define the LOS should be used for defining the equivalence. A performance measure is defined as "any parameter that is or could be used to define some aspect of a point, a segment or a facility's performance" (Roess et al. 2010). In Huber's method, two streams: one containing only passenger cars (base stream) and the other containing passenger cars and vehicle type for which the PCE value is going to be estimated (mixed stream), will be generated as shown in Figure 2.1. At the same LOS, flow rate corresponding to base stream (q_B) can be expressed as

$$q_B = q_M \times (1 - p) + q_M \times (p) \times E \tag{2.3}$$

Where E is the PCE of vehicle type under consideration; p is the proportion of vehicle type in the actual traffic stream, and q_M is flow rate corresponding to the mixed stream at same LOS. The PCE of a vehicle type can then be calculated using equation 2.4 shown below.

$$E = \left(\frac{1}{p}\right)\left[\left(\frac{q_B}{q_M}\right) - 1\right] + 1 \tag{2.4}$$

For traffic stream containing more than two vehicle types, separate mixed streams will be generated for each vehicle type by adding their respective proportions present in the actual traffic stream. The respective PCEs will be calculated by comparing each mixed flow rate with base flow rate at the same LOS.

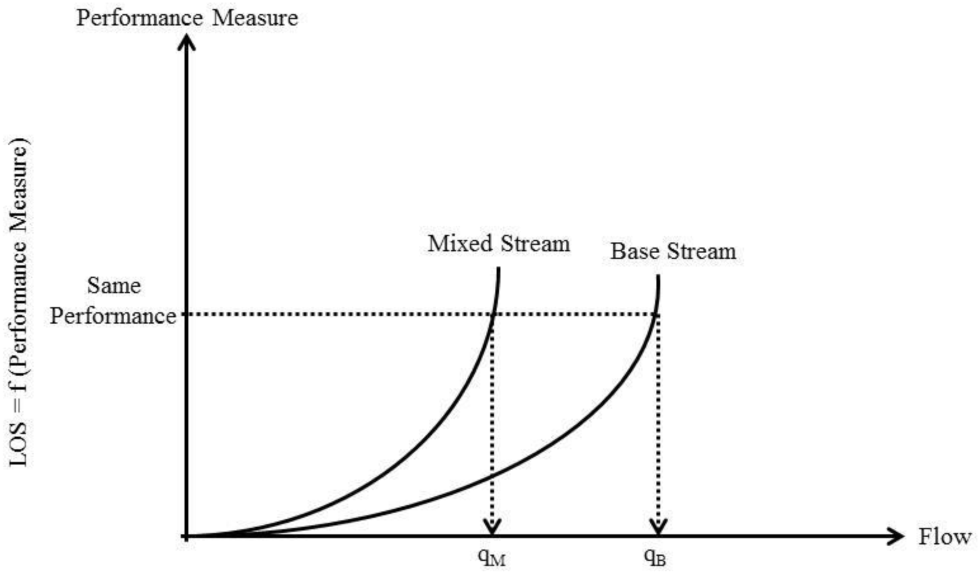


Figure 2.1 Flow-Performance Measure Curves for Base and Mixed Streams

Krammes and Crowley (1986) modified the Huber's equation using the equivalent effect on mean time headway for both the streams and estimated the PCE values. The PCE values were calculated for LOS A to LOS C, and the estimated values exhibited an increasing trend. Using this approach, Okura and Shapit (1995) estimated the PCE values of heavy vehicles at an equal proportion of capacity utilized (i.e., v/c ratio) with regards to the linear speed-density model. Equal v/c ratio provides a single set of PCE value irrespective of the traffic flow rate or LOS. They further mentioned that density could only be reasonably used for the PCE estimation under free-flow conditions whereas the speed will give appropriate results only near capacity conditions. Demarchi and Setti (2003) found that if more than two types of heavy vehicles are present in the traffic stream, Huber’s method overestimates the PCE values at an equal density. Since the Huber’s method considers only two types of vehicles, the impact felt by the passenger cars due to a particular type of heavy vehicle will be less if multiple types of heavy vehicles are present in the traffic stream.

Mallikarjuna and Rao (2006b) estimated the PCE value of the truck, bus, and the motorized two-wheeler at equal area occupancy by applying the Huber’s equation. Kerner’s three-phase traffic theory was employed for getting the flow-area occupancy relationship. The concept of area occupancy was introduced as the replacement of density in the case of heterogeneous traffic mix. Area occupancy measures the time spent by a vehicle of an area (a) over the measurement volume and can be expressed as shown below:

$$\rho_A = \frac{\sum_{i=1}^{N} t_i \times w_i \times l_i}{T \times W \times L} \tag{2.5}$$

Where W is the width; L is the length of road stretch under study; T is the measurement interval; t_i is the time spent by the i^{th} vehicle on the road stretch during time T; w_i and l_i are the width and length of the i^{th} vehicle. The PCE values are calculated for the synchronized flow conditions. Gautam et al. (2016) stated that for capacity analysis, the PCE values must correspond to the synchronized flow conditions.

Sumner et al.’s Method

Sumner et al. (1984) have modified Huber’s method to calculate the PCE values of multiple types of heavy vehicles. This approach sequentially calculates the PCE values, and in each stage, the PCE value of only one type of vehicle (subject vehicle) can be determined. After the base stream is obtained using a simulation model, a mixed stream is produced by adding *non-passenger cars* (vehicle types other than a subject vehicle type) observed on the road stretch being studied. The PCE value of mixed stream is calculated using equation 2.6. Then subject vehicles (of observed proportion) replace passenger cars to generate traffic stream being studied (subject stream). Figure 2.2 shows base stream, mixed stream, and subject stream. Base (q_B), mixed (q_M) and subject (q_S) flows can be related to each other at same performance of the traffic stream. Suppose p is the proportion of non-passenger cars in mixed stream and E_M is the equivalent of that stream, then

$$q_B = (1 - p) \times q_M + p \times q_M \times E_M \tag{2.6}$$

And suppose Δp is the proportion of base vehicles replaced by subject vehicles in the mixed stream and E_s is the passenger car equivalent of the subject vehicle type. Then,

$$q_B = (1 - p - \Delta p) \times q_s + p \times q_s \times E_M + \Delta p \times q_s \times E_s \tag{2.7}$$

From equations (2.6) and (2.7), the PCE equation is,

$$E_s = \frac{1}{\Delta p}\left(\frac{q_B}{q_s} - \frac{q_B}{q_M}\right) + 1 \tag{2.8}$$

Where, E_s is the PCE value of subject vehicle type.

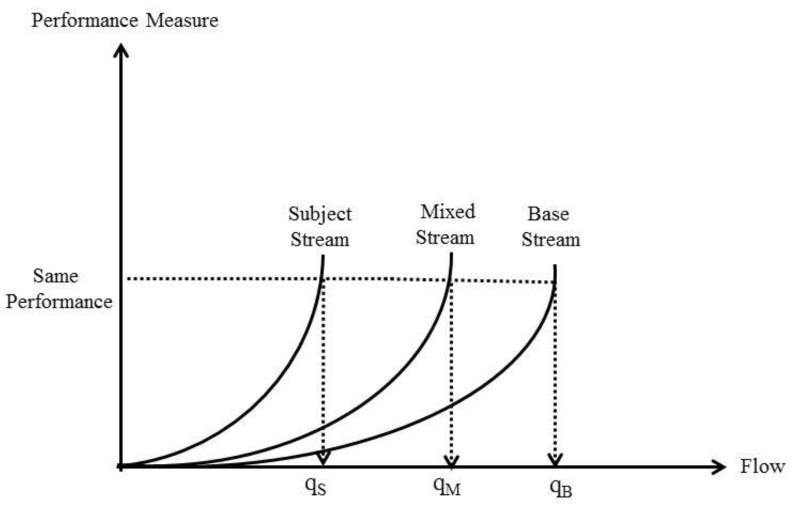


Figure 2.2 Flow-Performance Measure Curves for the Base, Mixed, and Subject Streams

Sumner et al. (1984) concluded that the accuracy of the PCE values estimated using a simulation model depends on the input parameters of the simulation model. They further mentioned that the number of simulation runs depending on the variance of the output plays a crucial role in getting the accurate PCE values. They have taken the average of *eighteen* simulation runs to get the PCE value of a particular vehicle type. They also found that the PCE values increase with the LOS.

Torbic et al. (1997) and Elefteriadou et al. (1997) have also used the Sumner et al.'s method for estimating the PCE values of multiple types of heavy vehicles at an equal stream speed. They have performed three hundred simulation runs to achieve a margin of error of ± 0.5 in the output values. Webster and Elefteriadou (1999) later estimated the PCE value of multiple types of trucks using the Sumner et al.'s method at an equal density. The PCE of a truck was defined as "the number of passenger cars that would have an equivalent effect on the quality of the traffic flow." Webster and Elefteriadou (1999) have determined the number of simulation runs (n) required to achieve a margin of error (d) of ± 0.25 was estimated using the following equation:

$$n = \left(\frac{s \times 1.96}{d}\right)^2 \tag{2.9}$$

Where, s is the standard deviation of the PCE and 1.96 is the standard normal critical value corresponding to the 95% confidence level. The margin of error mentioned above was chosen due to the reason that it was the same level of precision adopted in US HCM (1997). The variables considered in their study were: traffic flow rate, percent of trucks in the traffic stream, length of grade, percent of grade, free-flow speed, truck weight-to-power ratio, truck overall length, and the number of lanes per direction. It was assumed that if the PCE values differ by 0.25 or more for a particular traffic variable, then only that variable affects the PCE values, otherwise not. They found that the traffic flow rate affects the PCE values in level terrain and exhibits an increasing trend with the increasing flow rate. Remaining variables also affect PCE value such as the proportion of various vehicle types in the traffic mix. Furthermore, the variables such as the free-flow speed and the number of lanes per direction have very less effect on the PCE value. Truck's weight-to-power ratio and overall length influence the PCE value on long and steep grades rather than the level sections. Al-Kaisy et al. (2005) have mentioned that the number of simulation runs is one of the important aspects that need to be addressed while using the simulation model for developing the parameters of practical value and significance.

Aggregate PCE Approach

Demarchi and Setti (2003) concluded that Sumner et al.'s method underestimates the PCE values. They reasoned that impact of other heavy vehicles present in the traffic stream, before the addition of subject vehicles, will be overestimated resulting in an underestimation of the PCE value of the subject vehicle. Therefore, to get an accurate estimation of the combined impact of different vehicle types present in the traffic stream, they have suggested the PCE estimate for the entire mix of vehicle types (*aggregate PCE*). For a traffic stream containing *n* vehicle types, base flow rate (q_B) can be expressed as

$$q_B = \left(1 - \sum_{i}^{n-1} p_i\right) \times q_s + \sum_{i}^{n-1} p_i \times q_s \times E_{agg} \tag{2.10}$$

Where E_{agg} represents the aggregate PCE of the traffic stream, *i* indicates vehicle type, and q_S is the flow rate of the subject stream at the same performance measure (Figure 2.3). Rearranging the terms, the equation for aggregate PCE will be

$$E_{agg} = \frac{1}{\sum_{i}^{n-1} p_i}\left[\frac{q_B}{q_S} - 1\right] + 1 \tag{2.11}$$

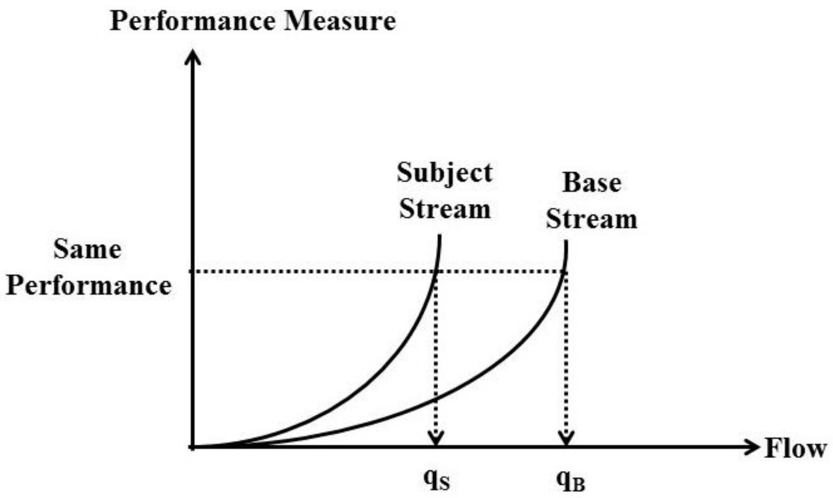


Figure 2.3 Flow-Performance Measure Curves for the Base and Subject Streams

Cunagin and Messer (1983), and Roess and Messer (1984) have already agreed upon on calculating the aggregate PCE value for the multiple types of vehicles. Roess and Messer (1984) have stated that the algebraic summation of the individual impacts used for getting the heavy vehicle adjustment factor (f_{HV}) may not be capturing the combined impact of trucks, buses and recreational vehicles. Rakha et al. (2007) stated that the concept of aggregate PCE was also adopted in the 1994, 1997, and 2000 editions of the US HCM. They proved that the aggregate PCE value of the truck corresponding to the density 12.4 pc/mi/lane (LOS C) matches with the one provided in HCM (2000) for the average weight-to-horsepower ratio of 87.5 kg/kW.

Matrix Method of Solving Simultaneous Equations

Kumar et al. (2018) developed simultaneous equations for estimating the PCE value of multiple types of vehicles, moving on the urban roads in India. They have used the following expression to estimate the PCE.

$$q_B - q_S \times P_{car} = \sum_{i=1}^{n} E_i \times P_i \times q_S \qquad (2.12)$$

Where, q_B and q_M are flow rates corresponding to base and subject stream at equal area occupancy, for the one-minute measurement period; E_i indicates the PCE value of the ith vehicle; P_i is the proportion of ith vehicle in the traffic stream during the one-minute period. For estimating the PCE values, flow-area occupancy relationships were developed using the Greenshields model in the cases of both the streams. They have solved simultaneous equations using the matrix method, and a single set of optimized PCE values were obtained for eight-lane and ten-lane divided roads. The mean absolute percentage error (MAPE) of these PCEs on eight-lane and ten-lane divided roads came out to be 8% and 9%, respectively. Based on these errors, they concluded that the estimated PCE values are accurate. Moreover, the PCE value of vehicles increases with the increasing road width. This was attributed to the higher speed difference between the passenger car and the subject vehicle, observed on the wider roads.

2.3.3 PCEs Based on Speed Reduction

Van Aerde and Yagar (1984) have estimated the PCE values for heavy vehicles of two-lane two-way highways using the relative amount of speed reductions caused by equal volumes of each vehicle type. For the entire range of practical operating volumes, the speed-volume curve was found to be linear in trend. Therefore, a linear model shown below was fitted through the data at each location.

$$\begin{aligned} &Percentile\ speed \\ &\quad = Free\ speed + C1 \times No\ of\ cars + C2 \times No\ of\ trucks + C3 \\ &\quad \times No\ of\ recreational\ vehicles + C4 \times No\ of\ other\ vehicles \\ &\quad + C5 \times No\ of\ opposing\ vehicles \end{aligned} \tag{2.13}$$

Where C1-C5 are the speed-reduction coefficients of different vehicle types. The PCE value of a vehicle type *n* is

$$PCE_n = \frac{C_n}{C_1} \tag{2.14}$$

Bang et al. (1995) estimated the PCE values of two-lane two-way roads in Indonesia by considering the reduction in the mean speed of passenger car. It was assumed that the speed-flow relationship follows a linear trend. Therefore, the PCE value was determined for low flow conditions by using the speed-flow samples corresponding to different traffic composition in the following equation:

$$V_{pc} = A - C_{pc} \times Q_{pc} - C_{MHV} \times Q_{MHV} - \cdots - C_{MHV} \times Q_{MHV} \tag{2.15}$$

Where, V_{pc} is the space mean speed of the passenger cars, in km/hr; A represents the free-flow speed; Q is the flow for each vehicle type in veh/5 min; C is the speed reduction effect caused by a specific vehicle type. The PCE value of a particular vehicle type is the ratio between the speed reduction effects of the vehicle type to that of the passenger car.

Rahman and Nakamura (2005) observed that on undivided roads, the presence of slow moving vehicles (rickshaws in their study) significantly reduces the speed of passenger cars. The PCE value was calculated using the following equation:

$$PCE = 1 + \frac{S_b - S_m}{S_b} \tag{2.16}$$

Where, S_b and S_m are the mean speed of passenger car in the base flow and the mixed flow, respectively. The mean speed of passenger car in the base flow was estimated from the one-minute interval data collected on the traffic stream with only passenger cars. They found that the PCE value of a non-motorized vehicle increases with the increasing flow rate. It also increases with the increasing proportion of non-motorized vehicle.

Basu et al. (2006) defined the PCE as “the number of passenger cars having the same impedance effect as a vehicle of a given type under a prevailing roadway, traffic, and control condition.” They calculated the PCE value of a particular type of vehicle, i, by using the ratio of reduction in stream speed caused by the subject vehicle (SR_i) to the reduction in stream speed caused by the passenger car (SR_{pc}). The PCE equation is shown below:

$$PCE_i = \frac{SR_i}{SR_{pc}} \tag{2.17}$$

The stream speed was modeled as a function of traffic volume and its composition. A linearly decreasing relationship was obtained between the stream speed and traffic volume. The PCE values of heavy vehicles and the motorized two-wheelers exhibited an increasing trend with the increasing flow rate. Moreover, the PCE value of heavy vehicle increases with the increasing proportion of heavy vehicle. But the PCE value of the motorized two-wheeler decreases with the increasing proportion of two-wheelers.

2.3.4 PCEs Based on Transport Road Research Laboratory (TRRL) Method

Transport and Road Research Laboratory (TRRL), London, U.K. in 1965 defined PCE as follows: “on any particular section of road under particular traffic conditions, if the addition of one vehicle of a particular type per hour will reduce the average speed of the remaining vehicles by the same amount as the addition of, say x cars of average size per hour then, one vehicle of this type is equivalent to x PCU”. Arasan and Arkatkar (2008, 2010) estimated the individual PCE values of different vehicle types for four-lane and six-lane divided highways using the TRRL method. For estimating the PCE value of a vehicle type (subject vehicle), few passenger cars were removed, and the subject vehicles were added to the mixed traffic stream. At a particular traffic volume, this should result in the same stream speed for the mixed traffic stream as it was before the removal of passenger cars and the addition of the subject vehicles. Then the PCE value of a subject vehicle can be defined as the ratio of the number of cars removed to the number of subject vehicles added, corresponding to that traffic volume. For obtaining the speed-flow relationship, three simulation runs were performed, and their average was taken to minimize the randomness in the output values. They found that the PCE value of a vehicle type increases with the increasing road width. The increasing trend in PCE was attributed to the larger speed difference between the passenger cars and the subject vehicles on wider roads. The PCE value of heavy vehicles decreases for low volume levels but increases near capacity conditions whereas an opposite trend was observed for the motorized two-wheelers. However, using the same approach, Brooks (2010) found that the PCE value of both the heavy vehicles and the motorized two-wheelers increases near low flow conditions and decreases under capacity conditions. This was due to the availability of larger space headways under low flow conditions which results in the larger speed difference between the passenger car and the subject vehicle. In the case of capacity conditions, available headway for the movement of passenger car reduces. This results in the reduced speed difference between the passenger car and the subject vehicle. Therefore, the PCE value decreases near capacity conditions.

2.3.5 PCEs Based On Vehicular Interaction and Space Occupied (Speed-Area Method)

Chandra and Sikdar (2000) estimated the PCE values using the inter-vehicular interaction and the physical sizes of different vehicle types present in the heterogeneous traffic stream. The PCE was defined as the ratio of the speed ratio of the car and the i^{th} vehicle to that of the space ratio of the car and the i^{th} vehicle, as shown below.

$$PCE_i = \frac{{}^{u_c}\!/_{u_i}}{{}^{A_c}\!/_{A_i}} \tag{2.18}$$

Where, u_c, u_i and A_c, A_i are the mean speeds and projected rectangular areas of the car and the i^{th} vehicle, respectively. The speed parameter was modeled using a linear speed-flow model, shown below.

$$u_i = \sum_{j=1}^{n} a_{ij}\left(q_j u_j\right) + d_i(\frac{1}{Q}) \tag{2.19}$$

Where, a_{ij}, d_i are the regression coefficients; n is the total number of vehicle types present in the traffic stream; j is the identifier for the vehicle types observed in the measurement interval; q_j is the flow rate for j^{th} type of vehicle; u_j is the mean speed of j^{th} vehicle type; and Q is the total flow rate of the road. Speed models were developed for all the vehicle types observed during the measurement interval. For a heterogeneous traffic mix, with the increase in the proportion of a particular vehicle type, the PCE value of that vehicle type decreases. Kumar et al. (2017) mentioned that this method requires extensive field data. The PCE values also vary depending on the measurement interval, and the area ratio does not vary with time.

Mehar et al. (2014) and Chandra et al. (2015) applied the expression shown in equation (2.18) for estimating the PCE values at different levels of service on multilane rural highways. For obtaining the speed-flow data, the microscopic simulation model, VISSIM, was calibrated and validated against the field data. The speed-flow curve for the mixed stream was generated with the help of the simulation model by adding the proportion of a particular subject vehicle in the traffic stream. The parabolic speed-flow relationship, estimated from the linear speed-density relationship, was fitted to the data. They found that the PCE value of the heavy vehicle and three wheelers decrease with the increasing traffic flow rate whereas two-wheelers exhibited an increasing trend. They stated that with the increasing flow rate, the speed difference between the passenger car and the larger size vehicle decreases. Therefore, the PCE value of such type of vehicle also decreases. It was further observed that the increasing proportion of a particular vehicle type (larger size) results in the decrease in the PCE value of that vehicle. The increase in the proportion of smaller sized vehicle resulted in the decreasing PCE value. The mixed stream contains only two types of vehicles such as the passenger car and the subject vehicle. Therefore, the mean speed of passenger car and the subject vehicle do not account for the interaction between the different types of vehicle present in the traffic stream.

Dhamaniya and Chandra (2016) estimated the PCE values on urban arterials using the expression shown in equation (2.18) for the heterogeneous traffic mix. The PCE was defined as "the measure of interaction between a given vehicle type and the traffic stream relative to that of the interaction between the car and the traffic stream, under a specified set of the roadway, traffic, and other conditions." The speed was modeled using the linear speed-density relationship. It was assumed that the density of all the vehicle types present in the traffic stream affects the speed of a given vehicle type for which the PCE value was being estimated. The speed-density model is shown below.

$$u_i = a_{0i} - \sum_{j=1}^{n} a_{ji} \times K_i \tag{2.20}$$

Where, u_i is the speed of the vehicle type *I*; a_{oi}, a_{ji} are the regression coefficients; *n* is the total number of vehicle types observed in the traffic stream; K_i is the density of vehicle type *i*. The density of a vehicle type was estimated using the flow and space mean speed of the corresponding vehicle during the measurement period. Therefore, the final equation for estimating the speed of a vehicle type is:

$$u_i = a_{0i} - \sum_{j=1}^{n} a_{ji} \frac{q_{ji}}{u_{ji}} \tag{2.21}$$

The PCE value of larger sized vehicles (heavy vehicles, big cars in this study) tends to increase with an increase in flow rate. On the other hand, the PCE value of smaller vehicles (three wheelers, two-wheelers in this study) decreases with increase in flow rate. At higher traffic volumes, the smaller sized vehicle tends to move through any lateral gap available in the traffic stream. This affects the flow of larger sized vehicles present in the traffic stream, consequently, leads to the greater speed difference between a passenger car and larger sized vehicle. The speed of the smaller sized vehicle does not get affected by the increase in flow rate. They also found that the PCE value of a vehicle type increases with increase in its proportion in the traffic stream. They attributed this to the greater speed difference between a passenger car and subject vehicle, once the proportion of subject vehicle increases in the traffic stream. Furthermore, the PCE value of smaller sized vehicles does not change with the change in traffic composition and traffic volume. Finally, they have provided the PCEs by varying the proportion of a subject vehicle within the practical range and also by varying the traffic volume. Biswas et al. (2016) mentioned that equation (2.17) is tedious due to the presence of speed on both the sides of the equation which makes it an iterative process. Further, the assumption of linearity between speed and density limits the applicability of the model. Therefore, to overcome, these limitations they have used kriging based approximation method for modeling the speed of different vehicle types present in the traffic stream. The PCE values of heavy vehicles, three wheelers, and big cars tend to increase with traffic flow rate whereas the PCE value of the two-wheelers tends to decrease with the increasing flow rate. This is due to the difference between the speed reduction rates of two-wheelers and the passenger cars with the changing flow rate. Gautam et al. (2016) used the expression shown in equation (2.18) for estimating the PCE values on hill roads in India. They concluded that this method overestimates the PCE value of the heavy vehicle and underestimates the PCE value of two-wheelers. Further, the PCE values of heavy vehicles in hilly terrain are on the lower side. Due to the low volume levels, the degree of maneuverability of the vehicles does not get much affected as compared to the high volume levels in level terrain.

2.3.6 PCEs Based on Vehicle Moving Space (VMS)

Zhang et al. (2006) used space occupied by a vehicle type relative to passenger car for estimating the PCE values on China highways. The PCE equation is:

$$PCE_i = \frac{s_i}{s_c} \tag{2.22}$$

Where, PCE_i is the PCE value of vehicle type i; s_i and s_c are the moving spaces of the i^{th} vehicle and car, respectively. They defined the vehicle moving space (VMS) as “the product of the occupied road space and the additional moving space for a moving vehicle among the traffic flow” and expressed as:

$$VMS_i = (width\ of\ i^{th} vehicle + lateral\ displacement\ of\ i^{th}\ vehicle) \times L \qquad (2.23)$$

Where VMS_i is the vehicle moving space of i^{th} vehicle; L is the occupied road length of the vehicle. It was found that the PCE value decreases with the increasing level of service.

2.3.7 PCEs Based on Area Density (Modified-Density Method)

Tiwari et al. (2000) and Tiwari et al. (2007) applied area density for calculating the PCE values meant for rural and suburban highways in India. For homogeneous traffic conditions, the PCE value of a vehicle type can be calculated using following equation:

$$PCE_{truck} = \frac{k_{car}}{W_L} \Big/ \frac{k_{truck}}{W_L} \qquad (2.24)$$

Where k_{car} is the density of cars in the homogeneous traffic stream; k_{truck} is the density of trucks in the homogeneous traffic stream; W_L is the width of the lane in the homogeneous traffic. Equation (2.24) compares k_{car} and k_{truck} of homogeneous car and truck traffic streams at equal space mean speed as shown in Figure 2.4.

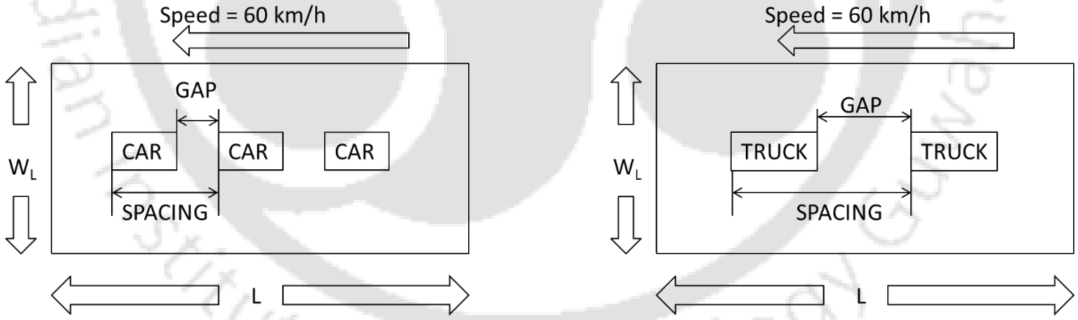


Figure 2.4 Homogeneous Traffic Streams of Car and Truck

However, in case of heterogeneous no-lane disciplined traffic conditions, vehicles do not use equal pavement width (W_L). Therefore, the authors considered 85th percentile distribution width of vehicle types for the heterogeneous traffic stream. The PCE value of a vehicle type for such kind of traffic is:

$$(PCU_j)_h = \left(\frac{k_c/W_{85c}}{q_j/u_j/W_{85j}}\right)_h \qquad (2.25)$$

Where, h is the type of highway; j is the subject vehicle for which the PCE value is estimated; k_c is the density of cars in the heterogeneous traffic stream; W_{85c} and W_{85j} is the 85th percentile distribution width of cars and the subject vehicle j in the heterogeneous traffic stream; q_j is the flow of vehicle type j; u_j is the space mean speed of vehicle type j. k_c is calculated by interpolating the density from the estimated car density-car speed graph at the speeds of other vehicle types present in the traffic stream.

2.3.8 PCEs Based On Variation in Capacity (Stochastic Capacity Variability Method)

Geistefeldt (2009) used a capacity distribution function to calculate the PCE values for the LOS analysis. Capacity is “the maximum flow rate up to which acceptable traffic performance of the facility is achieved and beyond which, in case of greater demand, unacceptable traffic conditions arise” (Brilon *et al.*, 2007). This is similar to the definition of “practical capacity” provided in HCM (1950). This was later termed as service volume in HCM (1965). The transition between the acceptable and unacceptable flow conditions on uninterrupted flow facilities is called a breakdown” (Brilon et al. 2007). On a roadway, such breakdowns are characterized by a sudden reduction in the average travel speed from an acceptable value of speed. Therefore, the capacity distribution functions were estimated for a particular roadway using the breakdown concept. The cumulative distribution function of the capacity (c) was found to be Weibull distributed and is shown below.

$$F_c(q) = p(c \leq q) = 1 - e^{-(\frac{q}{\beta})^{\alpha}} \tag{2.26}$$

Where, $F_c(q)$ is the capacity distribution function; q is the traffic volume in veh/hour; α is the shape parameter and ranges between 10 and 22; β is the scale parameter which depends on the factors affecting the freeway capacity such as the number of lanes, grade, and the driver population. It was assumed that the variation in the capacity values is due to the presence of heavy vehicles. Therefore, the stream of only the passenger cars should have the minimum variation in the capacity values. The coefficient of variation (c_v) of the capacity distribution function was calculated for the range of the PCE values lying between

0 and 4. The minimum value of c_v corresponds to the PCE value of the truck. It was found that the PCE values decrease with the increasing number of lanes. They stated that for analyzing the effect of different variables on PCE, simulation approaches are more appropriate.

2.4 Summary of PCE Estimation Methods

After the advent of PCE in 1965 HCM, several studies have been conducted for estimating the PCE values. Those studies have analyzed the concept of PCE using either empirical data or simulations. Each PCE estimation method involves a different set of tasks to be accomplished while estimating the PCE values. The selection of a particular PCE estimation method is a challenging task particularly for the heterogeneous traffic conditions that include different types of vehicles. US HCM uses the method based on macroscopic relationships for estimating the PCE values of uninterrupted facilities. This method involves two tasks: selection of the performance measure and the development of macroscopic relationships. For estimating the PCEs of different types of trucks, Sumner et al.'s method is used in the developed economies. But, this method was not used for the heterogeneous traffic conditions prevalent in the developing economies.

The speed-area method is widely used for the PCE estimation in India (Kumar et al. 2017). This method involves the modeling of the speed of individual vehicle type present in the stream and also incorporates the physical size of the individual type of vehicle. Biswas et al. (2016) mentioned that modeling the speed of individual vehicle type using linear macroscopic models as done in previous studies (Chandra and Sikdar, 2000; Dhamaniya and Chandra, 2016) limits the applicability of such speeds for the entire range of macroscopic parameters. They, therefore, suggested the use of kriging based approximation approach to overcome this limitation. Moreover, Kumar et al. (2017) stated that the area parameter in speed-area method does not vary with time. Therefore, the dynamic space occupancy such as the area occupancy can be incorporated into this method to check whether more accurate PCEs can be estimated or not.

Modified-density method involves the determination of the 85th percentile width distribution of different vehicle types and the densities of individual vehicle types in the traffic stream. The accuracy of the PCE values estimated by using this method has not been ascertained in any of the studies. Moreover, the area density can be replaced by area occupancy, and the accuracy of the estimated PCEs can be checked. Stochastic capacity variability method involves determining the number of traffic breakdowns at a site for a range of flow rate. This method will result in the PCE values only corresponding to the design service volume (practical capacity) for a particular roadway. Extensive data requirement is a major drawback of this method. A comparative analysis of the Huber's method, Sumner et al.'s method, speed-area method and modified density method is absent in the literature. It should be mentioned here that Gautam et al. (2016) compared Huber's method, speed-area method, and modified-density method for the heterogeneous traffic stream in hilly terrains. However, the comparison in their study lacked the quantitative aspect such as the error analysis of those methods.

The variables which influence the PCEs include the traffic composition, traffic volume, length of grade, and percent of grade. The PCE value of the heavy vehicles increases with the increasing flow rate or LOS (Huber, 1982; Sumner et al. 1984; Krammes and Crowley, 1986; Webster and Elefteriadou, 1999). However, in the case of the heterogeneous traffic stream, there is variability amongst the trends of the PCEs with the flow rate. Arasan and Arkatkar (2008, 2010) found that the PCE value of the heavy vehicles first increases and then decreases for a heavy vehicle whereas opposite trend was observed for two-wheelers. Mehar et al. (2014) found that the PCE value of heavy vehicles and two-wheelers decreases and increases with the increase in traffic flow rate, respectively. The effect of traffic flow rate on PCEs, therefore, needs to be further studied for the heterogeneous traffic stream. Limited studies have been conducted in India for measuring the effect of grade on the PCEs of highways.

Getting the field data corresponding to the combination of all the variables mentioned above is a very tedious task and practically nonexecutable. Various researchers

(Webster and Elefteriadou, 1999; Geistefeldt, 2009; Arasan and Arkatkar, 2010) pointed out that a simulation model is more apt for analyzing the effect of different variables on the PCE values. Simulation helps in analyzing the various situations which are very difficult to obtain empirically. Most of the PCE studies in the developed economies involve the use of the simulation model. However, few simulation studies (Arasan and Arkatkar, 2010; Mehar et al. 2013) have been conducted in India for analyzing the PCEs. Sumner et al. (1984), Webster and Elefteriadou (1999), Al-Kaisy et al. (2005) stated that the most important aspect of using simulation model is to estimate the number of simulation runs that can minimize the randomness in the output values. Further, the simulation models should be extensively validated. Therefore, these things should be considered while applying the simulation model for estimating the PCEs of the heterogeneous traffic stream.

2.5 Review of the Performance Measures

PCE values are obtained for the different levels of service which corresponds to the different traffic states present in the traffic stream. In order to distinguish between different levels of service, one must need to identify the performance measure that can be used to define the LOS for a particular roadway. This is not a straightforward task as LOS is an indicator of traffic flow quality as perceived by the users of a roadway. LOS measure is decided based on two criteria: first "the chosen measure should represent travel speed, freedom to manoeuvre, traffic interruptions, comfort and convenience in a manner most appropriate to characterise the quality of a service for a particular traffic facility being analysed." Secondly, "it should be sensitive to traffic flow rates so that service measure characterises the degree of congestion of the facility" (Seager 2004; Washburn and Kirschner 2006). US HCM used different performance measures over the years for defining the LOS. HCM (1965) used the performance measures such as operating speed, and v/c (demand/capacity or service volume/ capacity) ratio to define the LOS criteria for freeways, multilane highways, and two-lane two-way highways. However, the LOS criteria for both the performance measures were independently set in 1965 HCM irrespective of the speed-flow relationships that correspond to the actual traffic behaviour. Linzer et al.

(1979) found that such independent values in HCM (1965) inherently makes v/c ratio as the limiting criteria in attaining the LOS thresholds rather than the operating speed. This is contradictory to the LOS definition as volume is not perceivable by the users of a roadway moving within the traffic stream. HCM (1985) used separate performance measures for multilane highways and two-lane two-way highways. This is due to the fact that the interactions between vehicles are completely different in two-lane two-way highways as opposed to the multilane highways and freeways. For multilane highways and freeways, density was used as the LOS measure whereas, for two-lane two-way highways, percent time delay and average travel speed were used as the LOS measures. HCM (2000) and (2010) applied the same performance measures from the previous edition to define the LOS thresholds. However, the LOS measures in HCM has been decided based on the professional judgment of the Highway capacity and quality of service (HCQSC) members rather than the perceptions of drivers and passengers moving on the highway (Pfefer, 1999). Kittleson (2000) explained in his paper the way five levels of service from A-E were formed based on the discussions held among the task members of highway capacity committee. Further, Washburn et al. (2004) and Washburn and Kirschner (2006) raised concerns stating that the use of performance measures should vary depending on the area type such as urban and rural. On rural highways, drivers expect free-flow conditions and do not want their speed to drop below the free-flow speed as opposed to the urban highways where the major concern for the drivers is their travel time (Washburn and Kirschner, 2006). Sufficient knowledge of such expectancy is required before selecting a performance measure as LOS measure. Flannery et al. (2006) and Choocharukul et al. (2004) mentioned that there are other characteristics such as heavy vehicle presence, speed variance, traveler information, better geometry, and flow rate which affect users' perceptions rather than only density and should be incorporated in defining LOS. Washburn and Kirschner (2006) found that density is highly influential in determining the trip quality experienced by the users' on rural highways. Mallikarjuna and Rao (2006a) stated that density is not a suitable performance measure for the developing countries like India due to the fact that it is estimated by occupancy which only includes the length of the vehicle. In the traffic stream with the disorderly movement of vehicles, occupancy (a surrogate measure of density) is better represented by the area of the vehicle. They thereafter introduced a new performance measure called area occupancy which is defined as the fraction of time for which the set of observed vehicles occupies the selected road stretch in the time interval under consideration. Jensen (2017) found that car driver's LOS is greatly affected by the average speed of vehicles than flow or density. When speed drops significantly, drivers shift from being satisfied to be dissatisfied with most of the cases.

2.6 Summary of Literature Review

Several studies have been carried out in India for estimating the PCE values and they have been summarized in Table 2.1. They have either used simulation model or empirical data for estimating the individual PCEs. Sumner et al.'s method is widely used for estimating the individual PCEs of different vehicle types present in developed countries (HCM, 2010). Lack of reliable simulation model for Indian traffic conditions, restricts the use of this method and led to the development of various other PCE estimation methods. The present study therefore investigates the applicability of Sumner et al.'s method for Indian traffic conditions. The present study will be further considering area occupancy as performance measure into Sumner et al.'s method for incorporating the seepage of smaller sized vehicles through the heterogeneous traffic stream.

For heterogeneous traffic mix, getting the individual PCE value for a specified proportion of vehicle type may not be useful due to the fact that PCE value of a vehicle type for a given proportion will change depending on the proportion of other vehicle types present in the traffic stream. Hence, the aggregate PCE can be estimated for a particular traffic mix rather than estimating individual PCEs of different vehicle types. The present study will stand out in terms of reducing error to zero in the flow conversion by incorporating the aggregate PCE.

Table 2.1 PCE Studies on Indian Sub-Continent

Author (publication year)	Type of Road	Type of Study	Method Used	PCE Estimated
Chandra & Sikdar (2000)	Urban Road	Empirical	Speed-Area Method	Individual
Tiwari et al. (2000)	Rural Highways	Empirical	Modified-Density Method	Individual
Basu et al. (2006)	Urban Road	Empirical	Speed-Reduction Method	Individual
Mallikarjuna & Rao (2006)	Urban Road	Simulation	Huber's Method	Individual
Arasan & Arkatkar (2010)	Multilane Highway	Simulation	TRRL Method	Individual
Mehar et al. (2013)	Multilane Highway	Simulation	Speed-Area Method	Individual
Dhamaniya & Chandra (2016)	Multilane Highway	Empirical	Speed-Area Method	Individual
Kumar et al. (2017)	Urban Arterial	Empirical	Matrix Method of Solving Simultaneous Equations	Individual

Chapter 3

Data Collection and Analysis of Macroscopic Relations for No-Lane Based Heterogeneous Traffic Stream

3.1 General

Majority of the passenger car equivalent (PCE) studies do not perform a detailed analysis of the macroscopic relationships. Most of these studies employ well-established simulation models to produce the macroscopic relations. In India, most of the PCE studies are based on the empirical data and the macroscopic relations are estimated based on these data. Studies carried out in India use data sets related to free-flow and partly congested conditions and fit the Greenshields model of linear speed-density relation to further estimate the speed-flow relation. The present chapter explains the issues in developing the macroscopic relationships with a limited data. It further discusses an alternative approach of identifying the stationary conditions and compares the results of two approaches thereafter. After analysing the implications of using empirical data sets, this chapter elaborates the simulation model which will be used to generate the macroscopic relations. The chapter also includes the calibration technique to obtain an appropriate fit for the speed-density model and subsequently to generate the speed-flow, and flow-density relations.

The chapter is organized as follows: Section 3.2 includes the field data collection and extraction process carried out in this study. Section 3.3 describes the traditional data extraction approach for obtaining the macroscopic traffic characteristics from the field data. Section 3.4 includes a brief review of the literature on the analysis of macroscopic relations. Section 3.5 provides the detailed analysis of the macroscopic relations with different amount of data sets. Section 3.6 explains the Cassidy's method of identifying the stationary traffic conditions for the development of macroscopic relations. Section 3.7 discusses the simulation model framework and its validation with field data. Section 3.8 represents the calibration technique adopted in the study for fitting an appropriate speed-density model through the simulated data. Finally, section 3.9 incorporates the summary of the chapter along with the important concluding remarks

3.2 Data Collection and Extraction Methodology

Macroscopic relations describe traffic stream behaviour in all its possible states. It is necessary to observe traffic stream behaviour and collect data on all such possible states for developing macroscopic relations. Given the difficulties in field data collection, it is quite difficult to collect information related to all the states of traffic stream moving on a rural highway. Most of the past studies (conducted in India) that have tried to collect the data from rural roads ended up in getting the data related to only free flow conditions. This is mainly due to the absence of automated data collection techniques. Development of the macroscopic relations based on such data leads to the misrepresentation of traffic stream behaviour. To know the related problems, in the remainder of this section, a data set representing all the possible traffic states collected from an urban arterial road is analysed. Video films were processed for the extraction of traffic data using Traffic Analyser and Enumerator (TRAZER).

The field data inputs required for the Image processing software TRAZER were collected at the study location using a digital video camera. TRAZER was used for the data extraction as it better suits the heterogeneous traffic conditions. Mallikarjuna et al. (2009) have provided a detailed methodology for using the TRAZER. The important things to be considered for the data extraction using TRAZER are: a) Video should be collected with the camera facing the traffic and a minimum of 30 to 50 m road stretch being visible b) Accurate homography data necessary for correlating the real world and the image coordinates.

Conventional data collection methodologies adopted for extracting the macroscopic variables of heterogeneous traffic stream involves the collection of video film on the traffic stream being studied. Flow and speed data are collected while replaying the video film. Flow data is obtained at a suitable road section covered in the video film. Spot speeds are calculated using a trap length of 30 - 50 m. It is ensured that the flow data is calculated over the selected trap length. The length of the observation period used for data collection is another aspect that affects the macroscopic data. Researchers have used 1 minute to 15 minute observation periods and found that as the length of the observation period increases, scatter of the data is reduced.

The field observations used in the present study were collected at Jubilee Hills, on the traffic stream moving from Panjagutta to Madhapur, Hyderabad. The camera was placed over a foot-over bridge to get the midblock section data. The basic consideration in the selection of the road stretch was that it should be fairly straight and level for at least 100 m, free from the effect of the nearby intersection. The study stretch is a three-lane, 10 m wide road. Traffic data have been collected on Thursday (working day) for two hours, from 2.40 pm to 4.40 pm. The majority of the vehicles present in the traffic stream were cars, motorized two-wheelers, and three-wheelers. Traffic volumes varied from 3900 to 4560 vehicles/hour during the observation period and the traffic composition details are shown in Table 3.1.

Table 3.1: Observed Traffic Composition at Jubilee Hills, Hyderabad

Time of day	Composition in no. of Vehicles					Composition in %			
	LMV	MThW	HMV	MTW	Total	LMV	MThW	HMV	MTW
Jubilee 2.42 pm to 3.42 pm	1860	435	64	2196	4555	41	10	1	48
Jubilee 3.44 pm to 4.44 pm	1743	417	65	1703	3928	44	11	2	43

Image processing software TRAZER classifies all the vehicles into four categories, namely, Light motorized vehicles (LMV), Motorized-two-wheeler (MTW), Motorized-three-wheeler (MThW), and Heavy motorized vehicle (HMV). It gives output in the form of vehicle trajectories tracked over most of the road section visible in the video film. The trajectory data obtained from TRAZER has some problems, and the corresponding corrections are to be done before extracting the data on macroscopic variables. Mallikarjuna (2007) have listed various problems and the corresponding corrections in detail. It is to be noted that the vehicles that are not detected by the software can be manually added and tracked for the required road length. After correcting the position and speed data collected from TRAZER, speed and flow values were calculated.

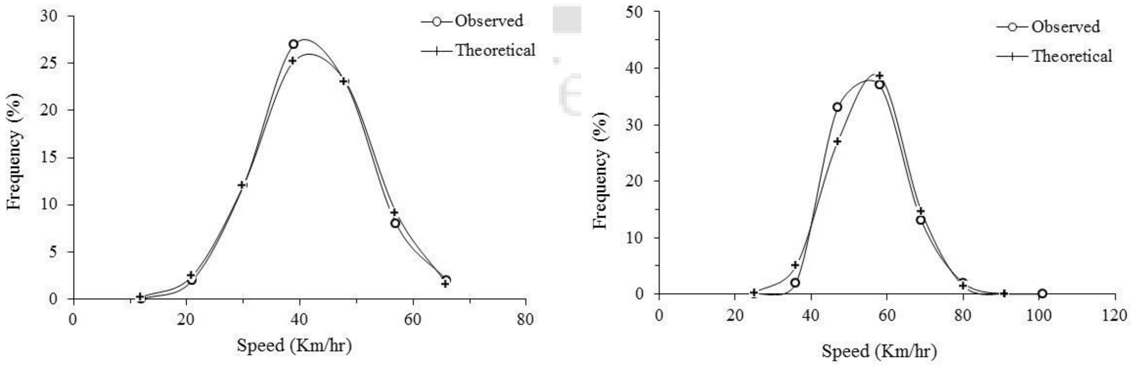
3.3 Macroscopic Traffic characteristics

Flow, speed, and density are the macroscopic characteristics used for describing the traffic stream behaviour. In this study, flow and density are expressed in terms of vehicles and speed is the harmonic mean of the spot speeds of vehicles, observed at a section. Mehar et al. (2014) have taken the arithmetic mean of the spot speeds as the average stream speed. Anand et al. (2011) have estimated the space mean speed of each vehicle type by taking the harmonic mean of the spot speeds. They have taken a weighted average of the space mean speeds of various vehicle types to get the stream speed. The weighting factors are the flows corresponding to different vehicle types observed in a road section.

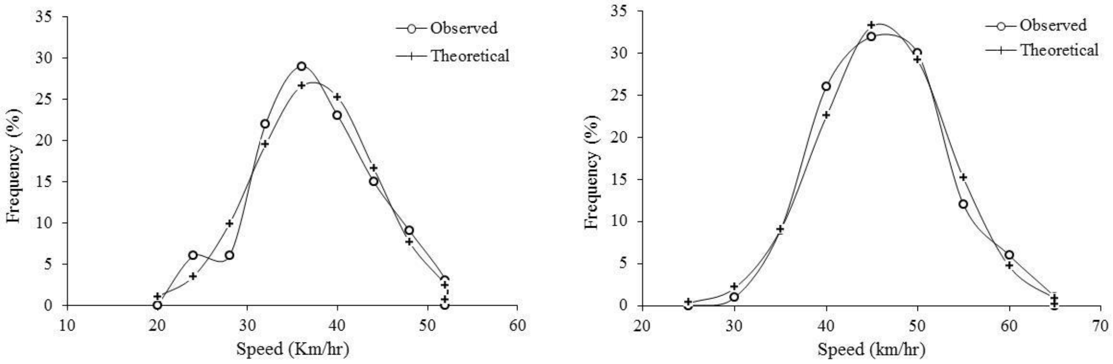
Free flow speed and jam density are the characteristics of the road stretch and the traffic stream being studied, and any speed-density model meant for studying the traffic stream moving on that road segment should reflect these characteristics. In this context, it is necessary to get the data on these two parameters. Free flow speed refers to the average speed of the vehicles moving on the road stretch when the vehicles are moving freely. For the homogeneous traffic conditions, where all the vehicles are of similar type, the mean speed of the free moving passenger cars is taken as the free flow speed. In the case of heterogeneous traffic, this parameter should consider the presence of all the types of vehicles in the traffic stream. For this purpose, average free flow speeds of various types of vehicles have to be obtained, and a weighted average of such speeds gives the average free flow speed. For each vehicle category, the space mean speed is calculated by taking the ratio of total distance travelled to the total time spent by all the vehicles within the same category. The space mean speed of the traffic stream is obtained by weighing the space mean speeds of individual vehicle category with their corresponding densities. For jam density calculation, it is necessary to know the average vehicular composition of the traffic stream. Once the average composition is known, the jam density can be estimated. In case of the present study, the jam density values were observed based on several photographs similar to the one shown in Figure 3.1.

Figure 3.1 Snapshot Showing the Congested Traffic Conditions

Average free flow speed was taken as the weighted average of the mean speeds of the free-flowing vehicles corresponding to four vehicle types. Free flow densities corresponding to the four vehicle types are taken as the weights. Free flow densities are estimated based on the average traffic flow composition observed on this road stretch. From each category, approximately 100 freely moving vehicles were identified for this purpose. It was observed that the free speeds of all the four vehicle types follow a normal distribution. The distributions of observed and the theoretical speeds (arising from the normality assumption) of different types of vehicles are compared and are shown in Figure 3.2. Cumulative distributions of the speeds are shown in Figure 3.3 for different types of vehicles.



(a) (b)



(c) (d)

Figure 3.2 Comparison of Observed and Theoretical Normal Distribution of Speeds for (a) HMV (b) LMV (c) MThW and (d) MTW

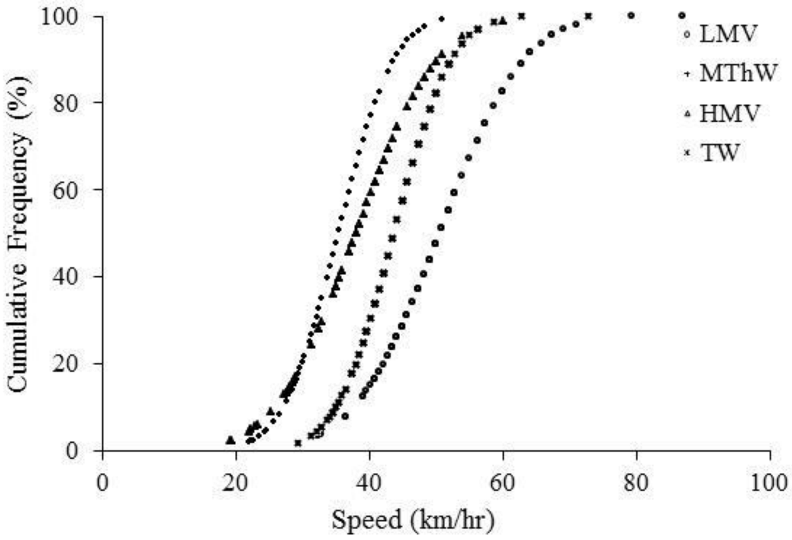


Figure 3.3 Cumulative Distribution of the Speeds for (a) HMV (b) LMV (c) MThW and (d) MTW

The statistics corresponding to the free flow speeds observed at the study location are shown in Table 3.2.

Table 3.2: Free-flow Speed Parameters Observed from the Field data

Vehicle type	Mean (μ) (km/hr)	Standard deviation (σ) (km/hr)
LMV	47	9
HMV	33	7
Auto	30	5
TW	42	6

For determining the jam density of the road, different frames having jammed conditions were identified (Figure 3.1). Frames having an approximately similar composition were used to obtain the jam density.

3.4 Review of the Studies on Macroscopic Relations

Analysis and modeling of the traffic stream behavior are necessary for traffic flow modeling. The traffic stream behavior is traditionally described using the macroscopic and microscopic characteristics/models. The functional relationship between the macroscopic parameters, flow, and density is known as the fundamental diagram, and it plays an important role in traffic flow theory and traffic engineering. A great deal of research has been done over the past several decades to establish the relationship between the traffic characteristics. The results of these researches yielded many mathematical and behavioural models. Behavioural models are derived based on the car-following behavior, and the mathematical models are estimated using the curve fitting approaches (Del Castillo and Benitez, 1995). A macroscopic model needs to satisfy several important observed traffic stream and roadway characteristics, and most of the models fail to do so (Del Castillo and Benitez, 1995).

Many researchers have used the speed-density relation as the basis for obtaining speed-flow and flow-density models (Lum et al. 1998). Greenshields (1935) developed a macroscopic model, in which the density and speed are linearly related. This is one of the widely used speed-density models though it has limitations in reflecting some of the observed traffic stream characteristics. Drake et al. (1967), Duncan (1974), Gerlough and Huber (1975), Duncan (1976), Duncan (1979), Chandra and Kumar (2003), Joshi et al. (2011), Kumar et al. (2011), Anand et al. (2011), Dhamaniya and Chandra (2014), and Mehar et al. (2014) have used this relation as the basis for getting the other macroscopic relations.

Problems related to the estimation of speed-density models can be divided into two parts. The first problem is due to the data used for model estimation. In case of homogeneous traffic conditions, density and speed values are always estimated from the observed flow and occupancy data. In case of heterogeneous traffic conditions, flow and speed observations are used to estimate the density. In this process, the density values corresponding to the queued traffic states are underestimated (Treiber and Kesting, 2013). The underestimation is due to the temporal nature of the flow data used for estimating the density. The second problem is related to the field observations on either flow-occupancy (in case of homogeneous traffic stream) or flow-speed (in case of heterogeneous traffic stream). Most of the times the data are collected without bothering about the variations in the traffic conditions within an observation period (Cassidy 1998). Besides these two issues, Van Aerde (1995) and Qu et al. (2015) have pointed out that the calibration methodologies used in parameter estimation also influence the model. Duncan (1976, 1979) has shown that calculating density from speed and flow, fitting a line to the speed-density data, and then converting that line into a speed-flow function, gives a biased result relative to the direct estimation of the speed-flow function. This is a consequence of three things: the non-linear transformations involved in both the directions, the stochastic nature of the observations, and the inability to match the time and space measurement frames exactly. Irrespective of the functional form considered for modelling the speed-density relation, the above issues are common.

In case of no lane-disciplined heterogeneous traffic conditions, speed and flow data are collected using the video films collected on the traffic stream. Speed-flow relations are estimated based on these data. Most of the times the field data are limited to free flow conditions and very few researchers have captured the queued traffic states (that too only on urban traffic). Speed-density relations are used to get the congested branch of the speed-flow relation (Dhamaniya and Chandra 2014). Density values are estimated using the speed, flow data and the fundamental relation among the macroscopic variables. For modelling the speed-density relation Green Shield’s model is widely used even in heterogeneous traffic conditions (Kumar et al., 2011; Dhamaniya and Chandra 2014; Mehar et al., 2014). In the majority of these studies, the modelled speed-density relation was used for predicting the congested branch of either the speed-flow or the flow-density diagram.

In this context, it is necessary to understand the implications of the field data collected on limited traffic states.

3.5 Analysis of the Macroscopic Traffic characteristics

3.5.1 Background

Many researchers have estimated the parameters of the Greenshields model using the observed speed and the estimated density data. Most of the times, field observations cover free-flow conditions and rarely the capacity flow conditions. This section explains the impact of using such data in estimating the parameters of the Greenshields model. Three sets of data were used for this purpose. The first set of data contains only the data corresponding to free-flow conditions. The second dataset contains the data on free-flow and capacity conditions. The third data set contains the data on free-flow, capacity, and the congested conditions. Another dataset, containing the observed speed data on free, capacity and congested traffic conditions and the estimated density data on free, capacity but the observed density data on congested conditions, has also been used to estimate the model parameters.

3.5.2 Development of Macroscopic Models for the Different Traffic States

Speed-Flow relation, corresponding to the data averaged over the 1 minute observation period, is shown in Figure 3.4. From this figure, it can be seen that the field observations cover the free, capacity, and queued traffic states. Clustering of data might be resulted due to the smaller observation period and the other inherent variability of the data. Density data were estimated using the flow and speed data and the fundamental relation of the macroscopic variables. Figure 3.5 shows the resulting flow-density relationship. From this figure, it can be seen that the density values corresponding to some of the queued traffic states (shown with the triangular markers) are less than the density corresponding to the capacity states. This is evident from the comparison of both the speed-flow and flow-density diagrams. Triangular (fully congested) and diamond markers (partly congested) represent the queued state. Density corresponding to the triangular markers is underestimated, and the reasons behind the underestimation are explained in Treiber and Kesting (2012). They have also explained the ways to overcome this problem. However, they used the microscopic quantities such as the speed of the individual vehicle, and time headway. For the heterogeneous traffic stream, getting the time headway for the different pairs of vehicles is a very complicated task, and therefore it limits the use of this approach for such traffic conditions.

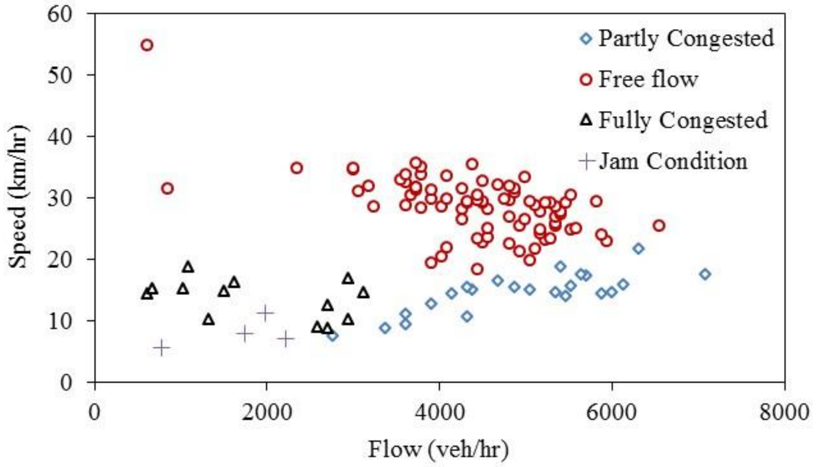


Figure 3.4 Speed-Flow relationships corresponding to one-minute averages

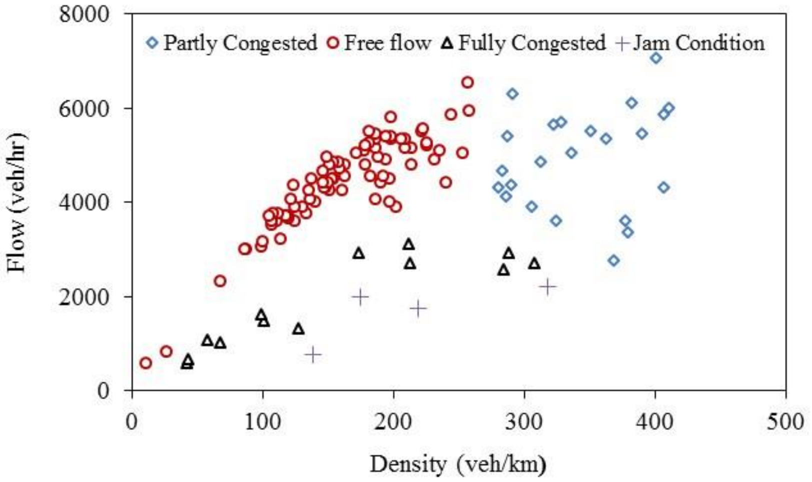


Figure 3.5 Flow-Density relationships corresponding to one-minute averages

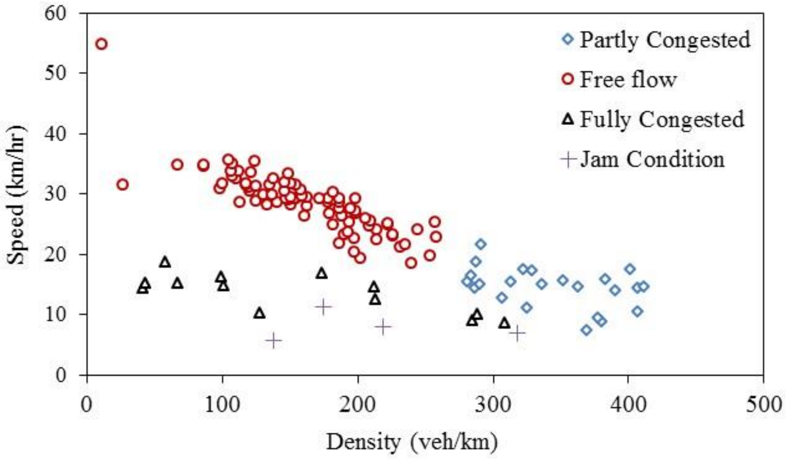


Figure 3.6 Speed-Density relationships corresponding to one-minute averages

Figure 3.6 shows the speed-density relationship, and from this figure, it is also clear that the density values corresponding to some of the queued states are underestimated. When such data are used in estimating the speed-density model it can be expected that the resulting parameters may not represent the roadway characteristics.

Conventional data collection approach considers selected data sets containing mostly the free-flow traffic states. Figure 3.7 represents the linear speed-density relation based on such type of data set. Based on the empirical observations, the field jam density and free-flow speed turn out to be 769 vehicles/km and 39 km/hr, respectively. But due to the limited data used for parameter estimation, the Greenshields model results in the jam density and free-flow speed values of 508 vehicles/km and 42 km/hr.

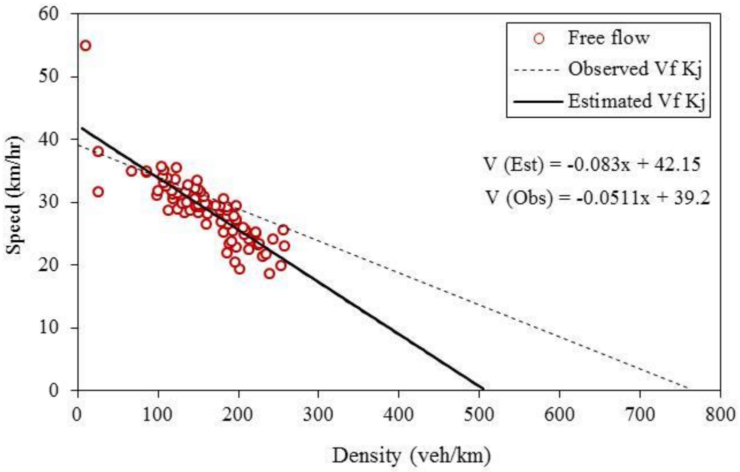


Figure 3.7 Linear Speed-density relationships corresponding to only the free flow traffic states

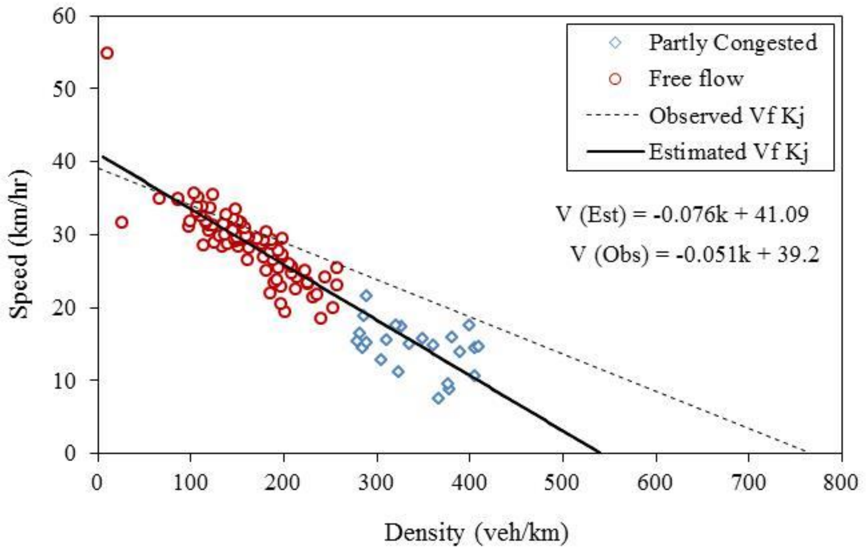


Figure 3.8 Speed-density relationship corresponding to free flow and a part of the queued traffic states

Figure 3.8 shows two speed-density relations: One is corresponding to the observed parameters, and the other is corresponding to the speed-density data on free flow and a part of the queued traffic states. From this figure, it can be seen that there is a slight improvement in the parameters estimated based on the speed-density data. The jam density value has slightly increased compared to the parameter estimated based on the data corresponding to only free flow traffic states. The estimated speed and densities were 41 km/hr and 549 veh/km, whereas the observed speed and densities were 39 km/hr and 769 veh/km. Figure 3.9 also shows the estimated speed-density model. This model corresponds to the field data covering all the possible traffic states. Very few studies (rarely in case of rural roads) consider such dataset in estimating the model parameters. Underestimated densities corresponding to some of the queued traffic states affect the model, and the estimated parameters are different from the observed roadway parameters.

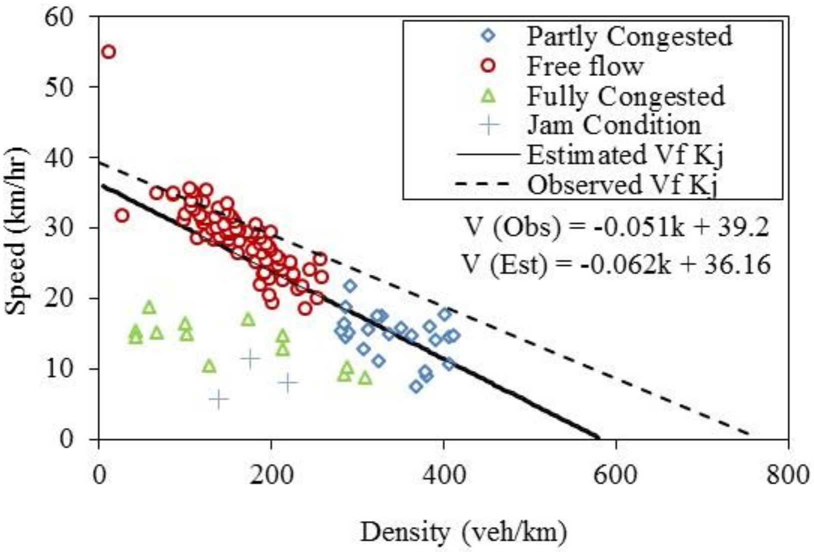


Figure 3.9 Speed-density relationship for free-flow and all the queued states

Relationships among the macroscopic variables shall represent the equilibrium or stationary traffic states and uniform one minute observation periods used in getting the previously used data do not guarantee this (Cassidy, 1998). Equilibrium traffic states refer to the similar traffic conditions for a considerably longer observation period. Cassidy (1998) has proposed a methodology to identify such traffic states. Though the data used in this study are limited to only two hours, an attempt has been made to segregate the data corresponding to stationary traffic conditions using the methodology provided by Cassidy.

3.6 Selection of stationary periods using the Cassidy's Method

Traffic flow on a road stretch is stationary for a measurement period if one is not able to identify what time it is or where he is by inspecting a time-space diagram through a small window in a template (Daganzo, 1997). It is characterized by the trajectories of the same slope which are equidistant, similar to the Figure 3.10. In other words, traffic flow is stationary if the average values of flow and speed do not change with time (Cassidy, 1998).

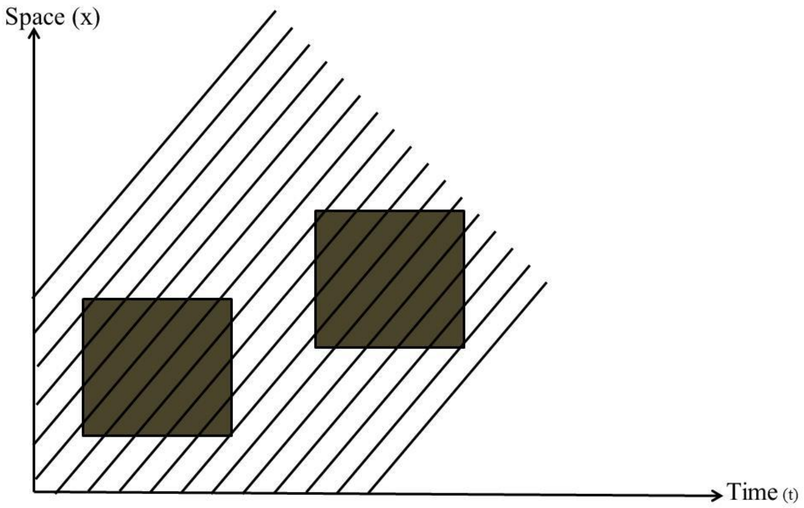


Figure 3.10 Hypothetical Time-space diagram for stationary traffic

Stationary conditions ensure that a smooth uniform relationship exists between the macroscopic variables such as the speed, density, and flow. The determination of the stationary periods, therefore, plays a crucial role in the calibration of the macroscopic relationships. A fixed period is considered for the measurement of the macroscopic variables. Mostly, smaller measurement periods (such as 30 seconds, 1 min, etc.) are used for obtaining the data on macroscopic variables. However, for smaller measurement periods, the flows may not be stable. Cassidy (1998) proposed a method to identify the near-stationary periods with the help of cumulative plots.

To select the stationary periods, Cassidy (1998) used the cumulative curves of vehicle arrivals and occupancy over time. First, a cumulative arrival curve $N\ (x,\ t)$ (also known as Moskowitz function) is constructed based on the data collected at a location x over time t. The cumulative arrival curve, $N\ (x,\ t)$ will then be enlarged to observe the variation in arrival rates and is achieved by subtracting a factor $q_o t$ from each $N\ (x,\ t)$. The scaling factor ($q_o t$) will be selected in such a manner that the difference between the two curves $N\ (x,\ t)$ and $N\ (x,\ t) - q_o t$ is 70% or more. Then the time periods corresponding to the almost linear slope were identified visually from the $N\ (x,\ t) - q_o t$ curve (Figure 3.11). The almost linear slope indicates the constant arrival rate within the selected period. At any t, if the slope (i.e., the tangent line) has a deviation of more than ten vehicles, (with respect to the average arrival) arrivals will be considered non-stationary. After obtaining the periods with almost constant arrival rate, the next step is to check whether the vehicle speeds remain constant during these periods. As explained by Cassidy (1998), for constant vehicle speeds and lengths (i.e., stationary conditions) the flow becomes equal to the occupancy multiplied by a scaling factor. Therefore, to fulfil this criterion, cumulative occupancy curve T (x, t) is superimposed over the cumulative arrival curve N (x, t) for the previously selected periods. For a given period (t_s, t_e), arrival number and occupancy were set equal to zero at $t = t_s$ and scale factors α and β were applied to N and T so that they had the same numerical value at $t = t_e$ as shown in Figure 3.12. For the purpose of visually identifying short-term fluctuations, a function of $b_o t$ was then subtracted from both the scaled curves. The curves $\alpha N(x, t) - b_o t$ and $\beta T(x, t) - b_o t$ were plotted together for each (t_s, t_e) and is shown in Figure 3.13. The periods within (t_s, t_e) when the curves $\alpha N(x, t) - b_o t$ and $\beta T(x, t) - b_o t$ almost superimpose represent the stationary conditions.

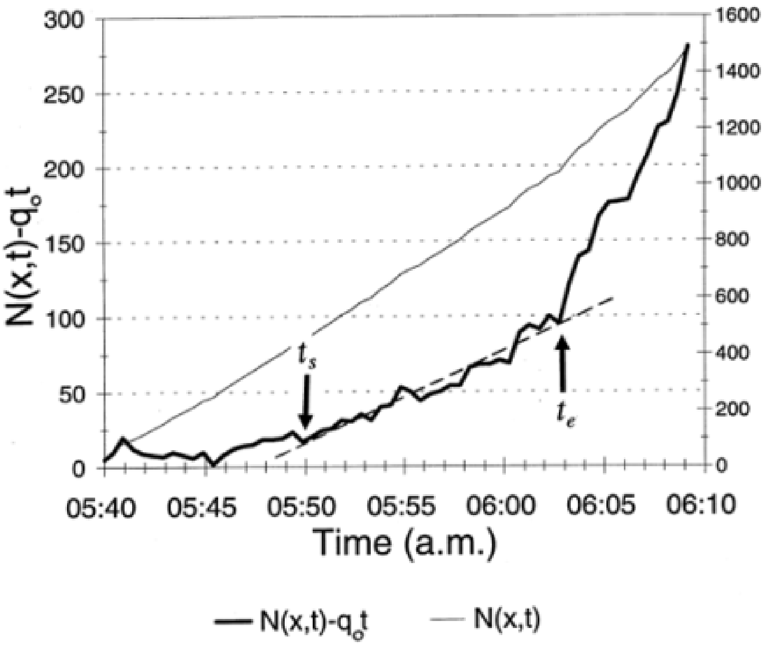


Figure 3.11 Curves of *N(x,t)* and *N(x,t)*-$q_o t$ (Source: Cassidy, 1998)

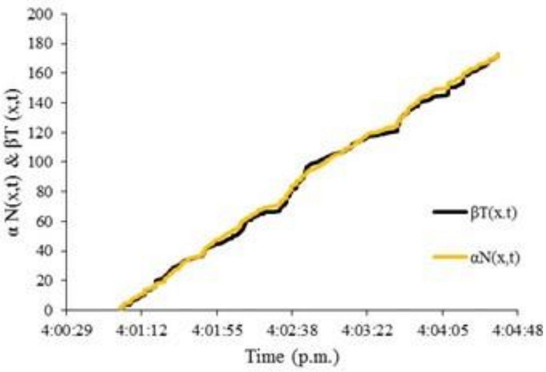


Figure 3.12 Curves of α$N(x, t)$ Vs. β$T(x, t)$

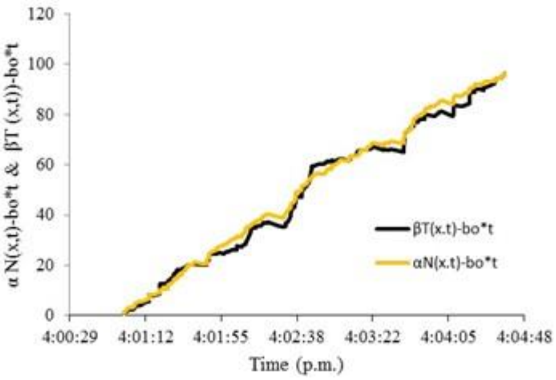


Figure 3.13 Curves of α$N(x, t)$-$b_o t$ Vs. β$T(x, t)$ - $b_o t$

Using this method, 37 stationary periods were identified from the two hours' data collected for the study. Figure 3.14 represent the speed-density relation corresponding to the various traffic states obtained using Cassidy's approach and the estimated jam density value is found to be much higher than that of the observed value. As shown in Figure 3.14, the density values corresponding to some of the queued states are underestimated, and these density values have been observed by replaying the video film. The speed-density model was estimated using the revised density data and is shown in Figure 3.15.

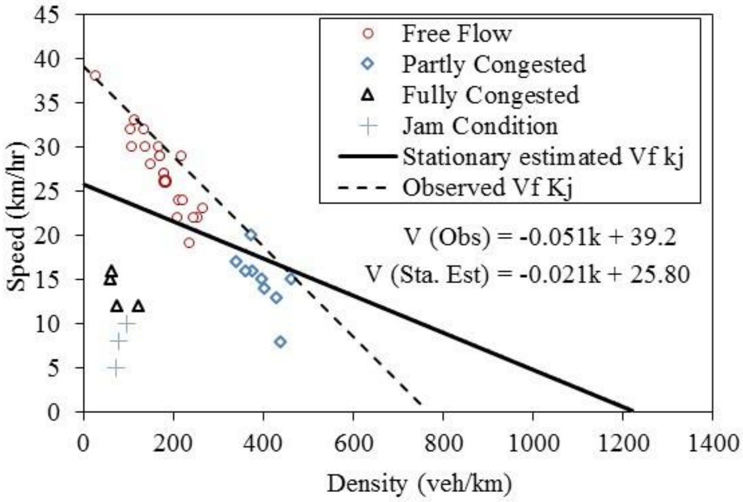


Figure 3.14 Speed-density relation obtained using Cassidy's method (free-flow and all the queued states)

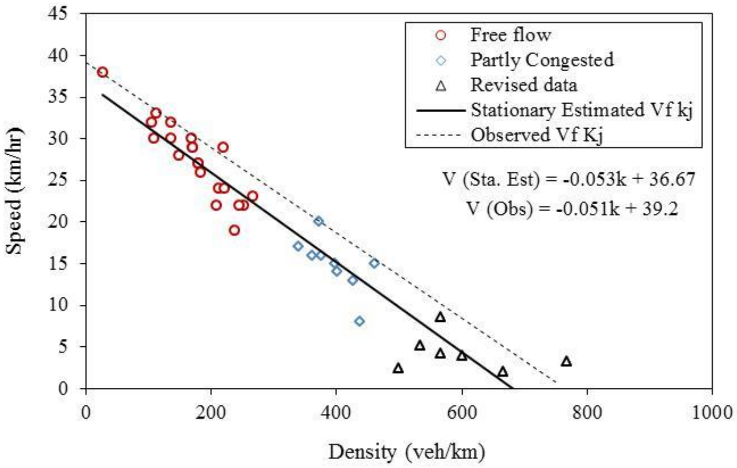


Figure 3.15 Stationary Speed-density relation for free-flow and all the queued states

The estimated parameters corresponding to the various models are shown in Table 3.3. The notation ND refers to the parameters estimated using the commonly used data collection approaches and the notation SDC refers to the parameters estimated using the stationary data collected using Cassidy's approach. From Table 3.3, it can be seen that the road parameters corresponding to the stationary data, with the revised density values for the congested traffic states, are matching with that of the actual values.

Table 3.3: Road parameters corresponding to linear speed-density relation

Road Parameters	Actual	Data corresponding to free flow conditions	Data corresponding to free flow and capacity conditions	Data corresponding to all the traffic conditions		
		ND	ND	ND	SDC	SDC with revised density
Jam density (vehicles/km), K_{jam}	769	502	534	581	1229	680
Free-flow speed (km/hr), V_f	39	42	41	36	26	37

3.7 Cellular Automata (CA) Model

CA model involves the discretization of the road stretch into a number of cells of similar sizes. All cell states are updated simultaneously in each time step. The new state of a cell depends on its previous state and the neighboring cells. However, for updating cell states, a finite set of local interaction rules need to be provided. This local interaction rules capture the micro-level dynamics and transform it to the macro-level behavior. For traffic flow modeling, CA model is an excellent tool due to its efficient and fast performance (Maerivoet and De Moor 2005). It can simulate large traffic systems in an extremely efficient manner. CA models can adequately capture the complexity of real traffic, by allowing different vehicles to possess different driving behaviors (Benjaafar et al., 1997). Furthermore, for the heterogeneous traffic conditions, smaller moving vehicles such as MTW can also be incorporated using smaller cell sizes. The working principle of CA model is described in the following sections.

3.7.1 CA Model Framework

CA model incorporates cells of smaller sizes which in turn provide more flexibility for a better representation of lateral movements in the heterogeneous no-lane disciplined traffic conditions. A cell length of 0.5 m was considered in the model due to the inferior acceleration capabilities of slow-moving vehicles found in the heterogeneous traffic stream. The cell width of 0.3 m was taken based on the physical dimensions and the lateral gaps maintained by different vehicle types in varying traffic conditions. In each time step, movement of the subject vehicle (SV) can be divided into lateral and longitudinal movements. The SV will identify its lateral and longitudinal influencing vehicles and will make a movement based on the available lateral and longitudinal gaps. These gaps are calculated based on the lateral and longitudinal interaction rules suggested by Mallikarjuna (2009) and Pal and Mallikarjuna (2017). Vehicular inputs include acceleration, deceleration, mean, and standard deviation of the maximum speed, length, width and maximum lateral gap of each vehicle type present in the traffic stream. Inputs related to the traffic stream are vehicular compositions and their free-flow speeds. The parameters of CA model, which were calibrated to get the desired output, are discussed below.

(1) Randomization parameter

Randomization parameter (p) indicates the random delay in the acceleration of the different vehicles, associated with some probability. It affects the maximum flow, i.e., the capacity point of the free-flow branch in the fundamental diagram (Mallikarjuna, 2007). It is calculated based on the slow-to-start probability (p_0), brake light probability (p_{bl}), and slow-down probability (p_{dec}). Vehicles which are at rest, take some time to accelerate with probability (p_0) and this is different for different types of vehicles. For example, HMV takes more time to start as compared to MTW. At moderate traffic conditions, a driver reacts according to the speed changes of the leading vehicle, i.e., to the "brake light" of the leading vehicle. When the brake light of the leading vehicle is on, the following driver will adjust his/her speed depending on the available headway. Slow-down probability represents the random fluctuations in the speeds of the different vehicles when they are forced to decelerate.

$$p = p(v_i(t), b_{i+1}(t), t_i^t, t^s) = \begin{cases} p_{bl,} & if\ b_{i+1}(t) = 1\ and\ t_i^t < t^s \\ p_{0,} & if\ v_i(t) = 0 \\ p_{dec}, & \text{in all other cases} \end{cases} \tag{3.1}$$

$$t_i^t = d_i^{eff} / v_i(t)\ ;\ d_i^{eff} = \min(d_{flv}^{eff},\ d_{lflv}^{eff}, d_{rflv}^{eff}) \tag{3.2}$$

Where, i and i+1 refer to the SV and effective leading vehicle, respectively; $v_i(t)$ is the speed of the SV at time step t; $b_{i+1}(t)$ is the binary variable denoting the brake light's status of the effective leading vehicle (if brake light is on, equal to 1; otherwise 0); t_i^t is the available time headway for the SV, t^s is the interaction headway (i.e., threshold) within which the effect of the leading vehicle is felt by the SV. d_{flv}^{eff}, d_{lflv}^{eff}, and d_{rflv}^{eff} are the effective gaps available to the SV with the front, left and right leading vehicles respectively; d_i^{eff} is the effective gap available for the SV after considering the anticipated movement of the effective leading vehicle.

(2) Acceleration

In this stage, the SV will accelerate based on its brake light status and that of the effective leading vehicle, and also the time headway with the effective leading vehicle.

If ($b_{i+1}(t) = 0$ and $b_i(t) = 0$) or ($t_i^t \geq t^s$) then,

$$v_i(t + 1/3) = \min(v_i(t) + a_i(v_i, l_i), v_{max}) \tag{3.3}$$

Else

$$v_i(t+1/3) = v_i(t) \tag{3.4}$$

Where l_i is the length of the SV, a_i (v_i, l_i) is the acceleration of the SV and v_{max} is the maximum allowable speed.

(3) Braking rule

The braking action is decided based on the effective longitudinal gap, and the available route width.

If ($rW_{i,t} \geq W_i + lg_i^l$) then,

$$v_i(t+2/3) = \min\left(v_i(t+1/3), d_{flv}^{eff}\right) \tag{3.5}$$

Else

If $\left(v_i(t+1/3) > min(d_{lflv}^{eff}, d_{rflv}^{eff})\right)$

$$v_i(t+2/3) = \min(\max(v_i^{re}), v_{max}), \quad v_i^{re} \ni \{W_i + lg_i^l < rW_{i,t}\} \tag{3.6}$$

Else

$$v_i(t+2/3) = \min\left(v(t+1/3), \min\left(d_{lflv}^{eff}, d_{rflv}^{eff}\right)\right) \tag{3.7}$$

Where, $rW_{i,t}$ is the effective route width available ahead to the SV at time t; v_i^{re} is the revised speed of the SV which is re-evaluated based on the available $rW_{i,t}$. W_i is the width of the SV, lg_i^l is the lateral gap required for the SV; (t + 1/3) and (t + 2/3) denotes various stages at which speed values are updated within each updating step.

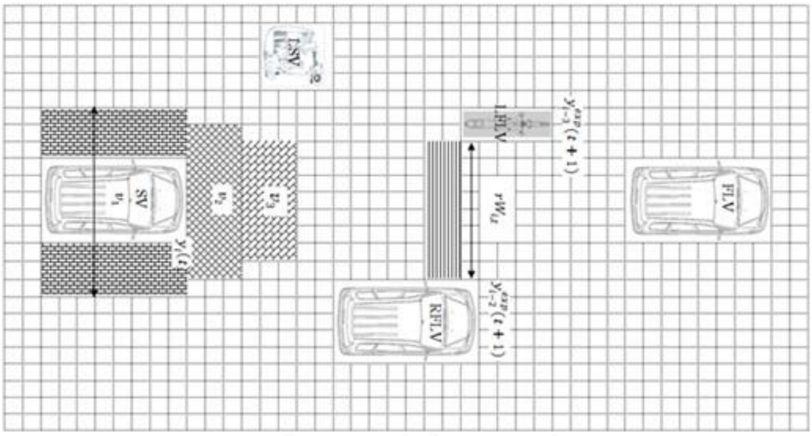


Figure 3.16 Effective route width available for the SV at various speeds (Source: Pal, 2016)

Figure 3.16 shows the total lateral gap required by the SV for different speeds. For higher speeds, the lateral gap requirement of the SV is more. To move through the width available ($rW_{i,t}$) as shown in the figure, the SV has to lower its speed to v_3. This has been incorporated in this step, and the speed has been modified accordingly. If the modified speed of the SV at this stage is lower than the speed of the SV at time step t, then

$$b_i(t+1) = 1 \tag{3.8}$$

(4) Deceleration

Based on the randomization parameter p, calculated in the first step, the vehicles will decelerate accordingly.

If $(rand() < p)$ then,

If $(p = p_{bl} or\ p_0)$

$$v_i(t+1) = \max(v_i(t+2/3) - \gamma_i(l_i), 0) \tag{3.9}$$

If $(p = p_{dec})$

$$v_i(t+1) = \max(v_i(t+2/3) - 1{,}0) \tag{3.10}$$

If $(p = p_{bl})$

$$b_i(t+1) = 1 \tag{3.11}$$

Where, $\gamma_i(l_i)$ is the deceleration of the SV.

(5) Updating the vehicle position

After going through the sequence of steps stated earlier, finally, the position of SV will be updated.

$$y_i(t+1) = y_i(t) + v_i(t+1). \tag{3.12}$$

Where, $y_i(t)$ and $y_i(t+1)$ are the positions of SV at time t and $t+1$ and $v_i(t+1)$ is the expected speed of the SV at time $t+1$. Based on all the rules mentioned above, the vehicles will be moving on the simulated road stretch.

3.7.2 Extraction of Macroscopic Data

The CA model shown in the previous sections was used to simulate the traffic stream moving over a road length of 2 km. The macroscopic parameters such as the density (k), flow (q) and space mean speed (v_s) are then obtained from CA model based on the following relationships.

$$k = \sum_{j=1}^{n} k_j \Rightarrow \sum_{j=1}^{n} N_j/P \Rightarrow 1/P \sum_{j=1}^{n} N_j \tag{3.13}$$

$$q = \frac{1}{T * P} \sum_{t=1}^{T} \sum_{j=1}^{n} \sum_{i=1}^{N} v_{j,i}(t) \tag{3.14}$$

$$v_s = \frac{1}{T} \sum_{i=1}^{T} \mathrm{v}_s(t) \tag{3.15}$$

Where, k is the global density in terms of cells/unit cell length; N_j is the number of cells in the j^{th} sub-lane that are filled with the vehicles; n is the number of lanes (sub-lanes); P is the number of cells in all lanes (sub-lanes); q is the global flow in cells/sec, $v_{j,i}(t)$ is the speed of the i^{th} vehicle in the j^{th} sub-lane; v_s is the mean speed during the measurement period T; $v_s(t)$ is the mean speed, in cell length/sec at time step t and can be calculated based on the following equation.

$$v_s(t) = \sum_{j=1}^{n} \sum_{i=1}^{N} v_{j,i} \Bigg/ \sum_{j=1}^{n} N_j \tag{3.16}$$

3.7.3 Validation of CA Model

The important inputs of CA model are warm-up time, data extraction time, boundary conditions, and the length of the simulated road stretch. Warm up time is the time required by the model actually to start mimicking the real traffic stream. That is why during this period, data will not be collected. Data extraction time is the duration for which the data will be collected from the model. The boundary conditions can be periodic or open. The periodic boundary conditions provide the stationary macroscopic data for the different traffic scenarios such as the free flow and the congested flow. Therefore, periodic boundary conditions have been used in the model.

3.7.3.1 Validation of the Four-lane CA Model

For validating the CA model, macroscopic data such as flow and space mean speed have been collected from the four-lane divided road in Kodihalli, Bangalore, India for two hours. Table 3.4 shows the observed traffic composition, vehicular characteristics, and road characteristics used in the CA model. Table 3.5 shows the coefficients and thresholds of total lateral gap model used in the model. The calibrated parameters of CA model for each type of vehicle present in the traffic stream are presented in Table 3.6.

Table 3.4: Parameters and vehicle characteristics used in Four-lane divided road CA model

Road Characteristics					
Cell width (m)		0.3			
Cell length (m)		0.5			
Road Width (m)		7.0			
Road Width (Cells)		24			
Road length (Cells) (Periodic boundary)		4000			
Warm up Time (Sec)		480			
Data collection time (Sec)		60			
Total run time (Sec)		540			
Vehicular Characteristics and Composition					
Parameter		LMV	HMV	MThW	MTW
Composition (%)		33.62	3.9	12.65	49.83
Length (Cell)		9	21	6	4
Width (Cell)		6	8	5	2
Mean maximum speed (Cell/sec)		26	21	21	24
Standard deviation of the maximum speed (Cell/sec)		5	3	4	4
Acceleration (Cell/sec^2)	$\vartheta_i \le 5.5$	4	2	2	5
	$5.5 < \vartheta_i < 11$	3	1	2	4
	$\vartheta_i \ge 11$	2	1	1	3
Deceleration (Cell/sec^2)		4	3	3	2
Maximum lateral gap (Cells)		7	7	5	1
Lateral gap **	Variable based on speed and type of subject vehicle as well as adjacent vehicles				

Note: ϑ_i is the SV's speed in Cells/sec.

The total lateral gap for different vehicle types is estimated using the approach proposed by Pal and Mallikarjuna (2017) and are shown below.

Table 3.5: Coefficients and threshold total lateral gap for Four-lane divided road

Vehicle type	Coefficients				L_g^{max}	Threshold speed values for speed		Threshold speed values for size	
	α_0	α_1	α_2	α_3		(km/hr)	(km/hr)	(km/hr)	(km/hr)
LMV	1.179	-0.022	-0.469	---	3.6	50.50	22.75	---	---
MTW	1.252	-0.031	-0.295	-0.88	3.6	39.97	15.02	49.94	20.01
MThW	1.003	-0.039	-0.588	---	3.06	20.2	10.45	---	---
HMV	0.829	-0.043	-0.394	---	3.48	20	12.26	---	---

Table 3.6: CA model parameters corresponding to the calibrated model (Four-lane divided road)

Parameters corresponding to Updating Model				
	LMV	HMV	MThW	MTW
Slow down probability (P_{dec})	0.3	0.1	0.3	0.1
Slow-to-start probability (P_o)	0.1	0.5	0.5	0.1
Brake light probability (P_{bl})	0.94	0.94	0.94	0.94
Minimum Gap (Cells)	4	4	4	4
Interaction Headway (t^{th}) (Sec)	2	3	3	2
Security distance ($d_{security}$)	10	12	12	10
Parameters corresponding to Lane change Model				
Probability in lane change (P)	0.95	0.6	0.6	0.95
Multiplication parameter (α)	1.0	1.1	1.2	1.0
Back gap ($\beta * \vartheta_t^b + \Delta$) factor ($\beta$)	1.0	1.0	1.0	1.0

Note: v_t^b is back vehicle speed.

For the given traffic and vehicular characteristics (Table 3.4), the model was simulated over a measurement region of length two kilometers. For four-lane divided road, lack of field data related to congested traffic states (Figure 3.19) restricts the development of macroscopic relations while using Cassidy's method. Thus, the comparison of simulated and field data in terms of their capacity values cannot be carried out for the time being in this study. Therefore, the macroscopic relations between speed, flow, and density developed using simulation model and field data are presented in Figures 3.17 to Figure 3.19.

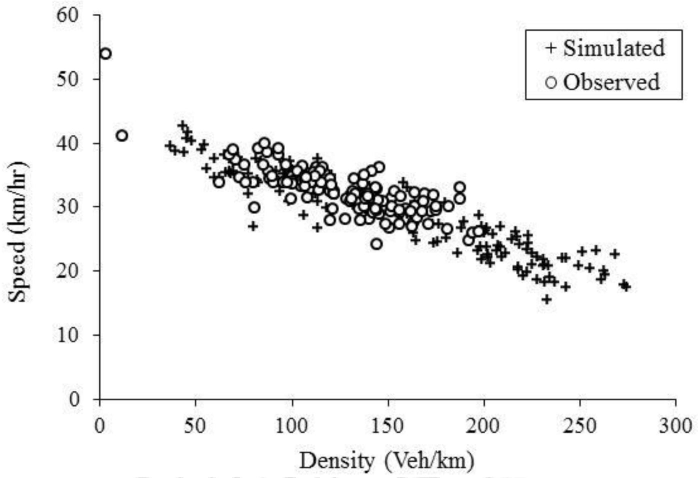


Figure 3.17 Observed and simulated Speed-Density relationships for four-lane divided road

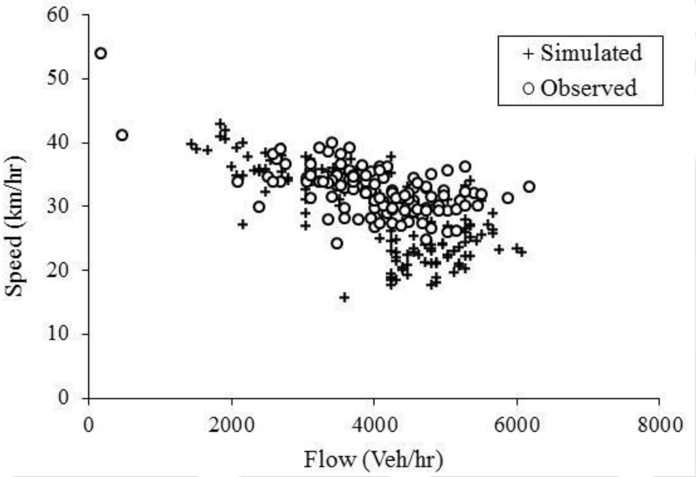


Figure 3.18 Observed and simulated Speed-Flow relationships for four-lane divided Road

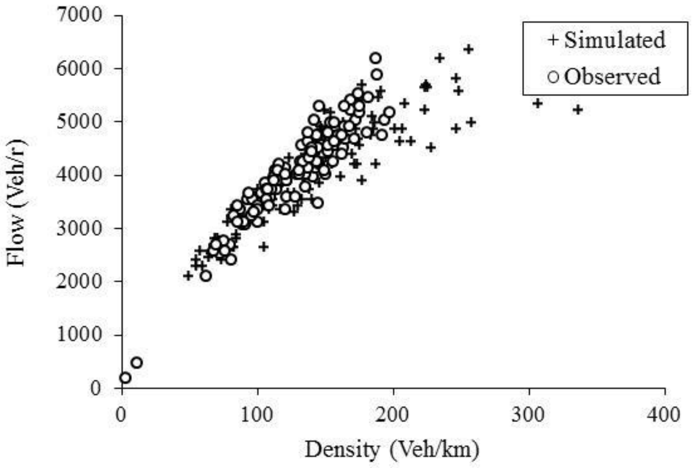


Figure 3.19 Observed and simulated Flow-Density relationships for four-lane divided Road

3.7.3.2 Validation of the Six-lane CA Model

Observed traffic composition and free flow speeds are the key inputs to the simulation model. Another important aspect is the road width to be used in the simulation model. The CA model for the six-lane divided road is validated using the two hours' data collected from Jubilee Hills, Hyderabad, India. Majority of the vehicles plying on this road section were LMV, and MTW and around 10% MThWs and 1% HMVs were also observed on the road stretch. Table 3.7 presents the details of traffic composition, vehicular characteristics, and the road characteristics. Table 3.8 shows the coefficients and thresholds of total lateral gap model. Table 3.9 shows the calibrated parameters of CA model for each type of vehicle present in the traffic stream.

Table 3.7: Parameters and vehicle characteristics used in six-lane divided road CA model

<table>
<tr><td colspan="6">Road Characteristics</td></tr>
<tr><td colspan="2">Cell width (m)</td><td colspan="4">0.3</td></tr>
<tr><td colspan="2">Cell length (m)</td><td colspan="4">0.5</td></tr>
<tr><td colspan="2">Road Width (m)</td><td colspan="4">10.5</td></tr>
<tr><td colspan="2">Road Width (Cells)</td><td colspan="4">35</td></tr>
<tr><td colspan="2">Road length (Cells) (Periodic boundary)</td><td colspan="4">4000</td></tr>
<tr><td colspan="2">Warm up Time (Sec)</td><td colspan="4">480</td></tr>
<tr><td colspan="2">Data collection time (Sec)</td><td colspan="4">60</td></tr>
<tr><td colspan="2">Total run time (Sec)</td><td colspan="4">540</td></tr>
<tr><td colspan="6">Vehicular Characteristics and Composition</td></tr>
<tr><td colspan="2">Parameter</td><td>LMV</td><td>HMV</td><td>MThW</td><td>MTW</td></tr>
<tr><td colspan="2">Composition (%)</td><td>44.3</td><td>1.2</td><td>10</td><td>44.5</td></tr>
<tr><td colspan="2">Length (Cell)</td><td>9</td><td>21</td><td>6</td><td>4</td></tr>
<tr><td colspan="2">Width (Cell)</td><td>6</td><td>8</td><td>5</td><td>2</td></tr>
<tr><td colspan="2">Mean maximum speed (Cell/sec)</td><td>26</td><td>18</td><td>17</td><td>23</td></tr>
<tr><td colspan="2">Standard deviation of the maximum speed (Cell/sec)</td><td>5</td><td>4</td><td>4</td><td>4</td></tr>
<tr><td rowspan="3">Acceleration (Cell/sec^2)</td><td>$v_i \le 5.5$</td><td>4</td><td>2</td><td>2</td><td>5</td></tr>
<tr><td>$5.5 < v_i < 11$</td><td>3</td><td>1</td><td>2</td><td>4</td></tr>
<tr><td>$v_i \ge 11$</td><td>2</td><td>1</td><td>1</td><td>3</td></tr>
<tr><td colspan="2">Deceleration (Cell/sec^2)</td><td>4</td><td>3</td><td>3</td><td>2</td></tr>
<tr><td colspan="2">Maximum lateral gap (Cells)</td><td>7</td><td>7</td><td>5</td><td>1</td></tr>
<tr><td colspan="2">Lateral gap **</td><td colspan="4">Variable based on speed and type of subject vehicle as well as adjacent vehicles</td></tr>
</table>

Note: ϑ_i is the SV's speed in Cells/sec.

The total lateral gap for different vehicle types are estimated using the approach proposed by Pal and Mallikarjuna (2017) and are shown below.

Table 3.8: Coefficients and threshold total lateral gap for six-lane divided road

Vehicle type	Coefficients				L_g^{max}	Threshold speed values for speed		Threshold speed values for size	
	α_0	α_1	α_2	α_3		(km/hr)	(km/hr)	(km/hr)	(km/hr)
LMV	0.997	-0.032	-0.379	---	3.47	40.98	15.67	---	---
MTW	1.739	-0.034	-0.571	-0.388	3.48	29.97	15.03	38.58	15.03
MThW	1.003	-0.039	-0.588	---	3.06	20.2	10.45	---	---
HMV	0.829	-0.043	-0.394	---	3.48	20	12.26	---	---

Table 3.9: CA model parameters corresponding to the calibrated model (six-lane divided road)

Parameters corresponding to Updating Model				
	LMV	HMV	MThW	MTW
Slow down probability (P_{dec})	0.3	0.1	0.3	0.1
Slow-to-start probability (P_o)	0.2	0.5	0.5	0.1
Brake light probability (P_{bl})	0.94	0.94	0.94	0.94
Minimum Gap (Cells)	4	4	4	4
Interaction Headway (t^{th}) (Sec)	4	5	4	3
Security distance ($d_{security}$)	10	12	12	10
Parameters corresponding to Lane change Model				
Probability in lane change (P)	0.95	0.6	0.7	0.95
Multiplication parameter (α)	1.0	1.1	1.2	1.0
Back gap ($\beta * v_t^b + \Delta$) factor (β)	1.0	1.0	1.0	1.0

Note: v_t^b is back vehicle speed.

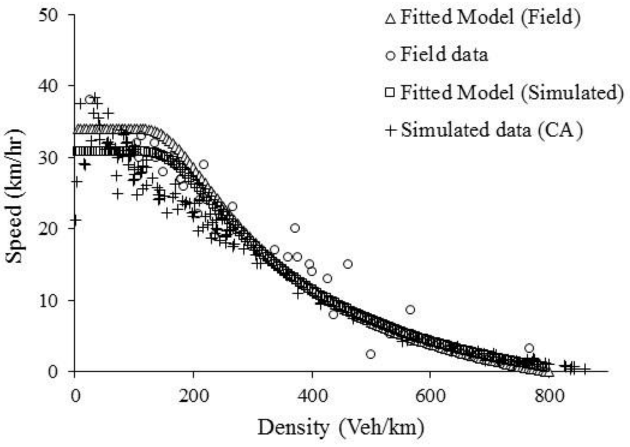


Figure 3.20 Comparison of Speed-density relations for simulated and field data for six-lane divided road

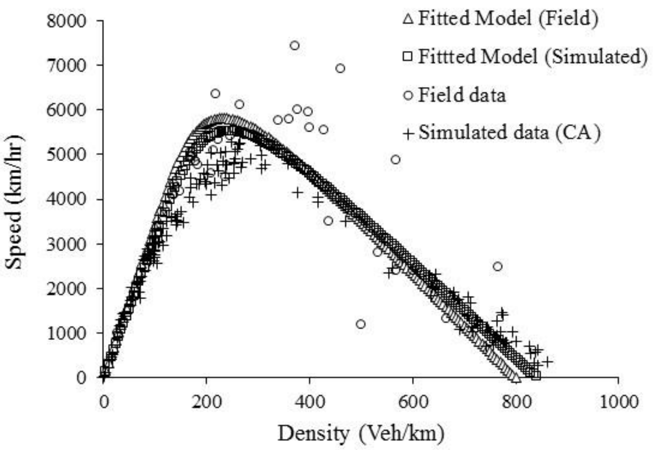


Figure 3.21 Comparison of Flow-density relations for simulated and field data for six-lane divided road

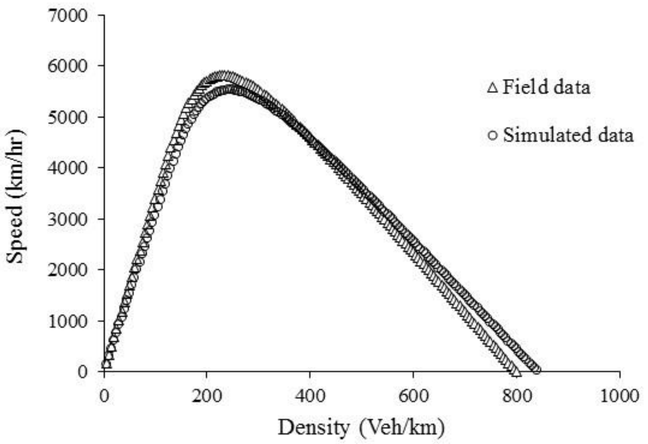


Figure 3.22 Comparison of capacity values for six-lane divided road

By fitting the speed-density model of Del Castillo and Benitez through the field and simulated data, the flow-density relationships have been obtained as shown in Figure 3.22. For the six-lane divided road, the capacity values came out to be 5540 veh/hr (field) and 5595 veh/hr (simulation). Hence, CA model can be used to replicate the field conditions for the six-lane divided road.

3.8 Calibration of Speed-Density Models

Researchers over the years have employed both single regime and multi regime models for developing the macroscopic relationships. Multi regime models fit the data better in different traffic states, but the calibration of such models is cumbersome. Van Aerde and

Rakha (1995) and Qu et al. (2015) stated that single regime models fail in representing the congested traffic states not because of their functional forms but due to the presence of less data related to those states. Van Aerde and Rakha (1995) further stated that minimum twenty data points (as a thumb rule) should be there for the congested traffic state. It is obvious that a single regime model with several parameters will provide a better fitting to the data. However, the majority of the studies on single regime models focused on getting empirically accurate models with less number of meaningful parameters (Qu et al. 2015). Speed-density model is the basic relationship used to derive the other macroscopic relationships such as the speed-flow and flow-density. The shape of the speed-density model is highly influenced by the parameters such as free-flow speed (v_f), jam density (k_j), and characteristic wave speed (c_j). Further, the number of data points corresponding to the congested traffic states affects the parameter estimation (Van Aerde and Rakha, 1995). Hence it is important to consider these aspects while choosing and calibrating the speed-density model. Prior determination of the range of the traffic flow parameters can be a better approach to provide certain constraints while fitting various models to the empirical data.

3.8.1 Regression Method for the Speed-Density Model

The present study selects single regime models with parameters v_f, C_j, and k_j in their functional forms. For calibrating such models, the values of the parameters v_f, C_j, and k_j are to be determined from the field. Free-flow speed and jam density have been observed from the field by replaying the video, and the range of C_j has been taken from the literature. Moreover, for calibrating the speed-density model, the selection of either speed or density as the dependent or the independent variable is not a straightforward task. This is because there is no clear-cut evidence that speed depends on density or vice-versa.

It is known that regression of Y as a function of X, $f_1(X)$, will not yield the same relationship as the regression of $X=f_2(Y)$. It is therefore important, in any curve fitting exercise, to establish which variables are independent and which are dependent variables. It is evident from Figure 3.23 that the regression of Y on X (REG 1) results in a different line from the regression of X on Y (REG 2). Therefore, it is not clear whether “REG 1” or

"REG 2" is the most appropriate fit to the data where the dependency between the two variables is not certain. To overcome the problem of identifying the dependent and independent variables, Van Aerde and Rakha (1995) suggested that both the variables should be considered as independent variables. This results in minimizing the orthogonal error (length c, shown in Figure 3.24) of the observed data about the regression line and can be represented as,

$$\text{Min} \quad E = \sum_i((x_i - \hat{x}_i)^2 + (y_i - \hat{y}_i)^2)$$

$$\text{S.T. } \hat{y}_i = a + b\hat{x}_i$$

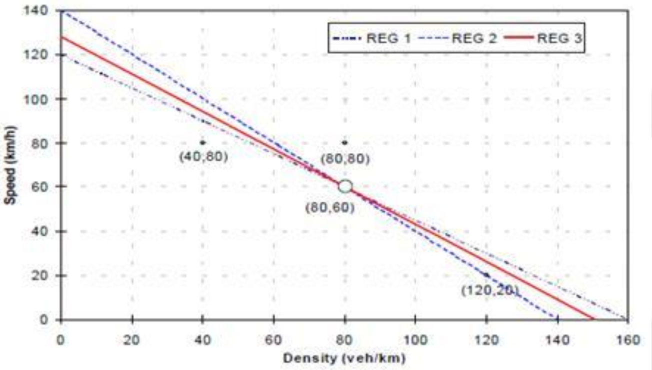


Figure 3.23 Regression line fits to three points (Source: Van Aerde and Rakha, 1995)

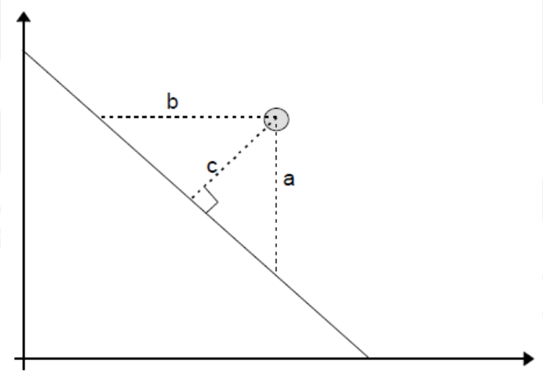


Figure 3.24 Orthogonal error estimates about the regression line (Source: Van Aerde and Rakha, 1995)

However, this can result in a biased fit as the sum of squared errors for one variable might be much larger than that for another variable and also depends on the unit of the variable. So, to estimate an unbiased sum of squared errors, the individual errors were normalized against their mean values as shown below:

$$\text{Min} \quad E = \sum_i \left(\left(\frac{x_i - \widehat{x_\iota}}{\bar{x}} \right)^2 + \left(\frac{y_i - \widehat{y_\iota}}{\bar{y}} \right)^2 \right)$$

$$\text{S.T.} \; \widehat{y_\iota} = a + b\widehat{x_\iota}$$

Where, x_i, y_i are the observed values, the variables with (^) are the estimated values, the variables with (−) are the mean values of the observed values. Now, the minimization of the speed-flow-density model in three-dimensions can be expressed as:

$$\text{Min} \quad E = \sum_i \left(\left(\frac{v_i - \widehat{v_\iota}}{\bar{v}} \right)^2 + \left(\frac{q_i - \widehat{q_\iota}}{\bar{q}} \right)^2 + \left(\frac{k_i - \widehat{k_\iota}}{\bar{k}} \right)^2 \right)$$

$$\text{S.T.} \qquad \widehat{v_\iota},\; \widehat{q_\iota},\; \widehat{k_\iota} > 0$$

$$\widehat{k_\iota} < k_j \; ; \; \widehat{v_\iota} < v_f$$

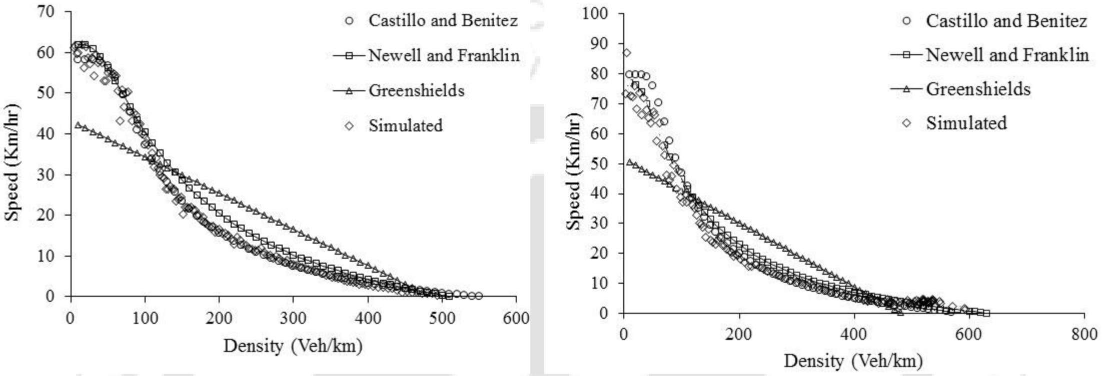
$$\widehat{q_\iota} = \widehat{k_\iota}\widehat{v_\iota}$$

Where, v_i, q_i, k_i are the traffic flow characteristics corresponding to the stationary traffic states, $\widehat{v_\iota}$, $\widehat{q_\iota}$, $\widehat{k_\iota}$ are the traffic flow values estimated by the model, $\overline{v_\iota}, \overline{q_\iota}, \overline{k_i}$ are the average values of traffic flow characteristics of stationary states. A MATLAB code was developed to solve the non-linear optimization problem. The optimization toolbox function ‘fmincon’ was used to minimize the sum of least squared errors *E*. The minimization was iterated for a range of feasible values of free-flow speed v_f, characteristic wave speed C_j, and jam density k_j. The search space of the optimization problem was reduced with each iteration to obtain the parameter values as close to absolute minima as possible. The speed-density models considered for the calibration are given in Table 3.10.

Table 3.10: Speed-density models for calibration

Models	Functional Form	Parameters
Del Castillo and Benitez (1995)	$v_f \left[1 - \exp\left(1 - \exp\left(\frac{c_j}{v_f} \left(\frac{k_j}{k} - 1 \right) \right) \right) \right]$	v_f, k_j, c_j
Newell (1961) and Franklin (1961)	$v_f \left[1 - \exp\left(\frac{c_j}{v_f} * \left(1 - \frac{k_j}{k} \right) \right) \right]$	v_f, k_j, c_j
Greenshields (1935)	$v_f \left[1 - \frac{k}{k_j} \right]$	v_f, k_j

The above-mentioned empirical models have been calibrated and tested against the data extracted from simulation model (CA). The calibrated speed-density models are shown in Figure 3.25. Various constraints to the traffic parameters have been applied while fitting these models. It was assumed that the characteristic wave would propagate within the speed range of -15 kmph to -20 kmph (Chiabaut et al., 2009: Kerner and Rehborn, 1996; Windover and Cassidy, 2001).



Four-lane　　　　　　　　　　Six-lane

Figure 3.25 Calibration of Speed-Density Models

From Table 3.11, it can be observed that both the parameters of Del Castillo and Benitez's model came out to be accurate. Therefore, it is better to use this approach for the calibration of the speed-density model under heterogeneous traffic conditions.

Table 3.11: Estimated Road parameters for simulated data

Models	Six-lane		*Four-lane*	
	K_j	V_f	K_j	V_f
Del Castillo and Benitez (1995)	600	79.74	483	58.53
Newell (1961) and Franklin (1961)	640	76.89	537	61.92
Greenshields (1935)	482	52	48	43.21
Simulated	588	81	460	62.0

3.9 Summary

Macroscopic relations play a vital role in the PCE estimation. Speed-density relation is widely used as the basic relation to estimate the other macroscopic relations. Greenshields linear speed-density model is used for modelling the heterogeneous traffic stream. Initially, the data extracted using the traditional approach (i.e., fixed and smaller measurement period) has been employed to obtain the speed-density model. The accuracy of the model parameters such as the free-flow speed and jam density was verified with the observed values. Three models, corresponding to the free traffic states, free and a part of the queued (mostly capacity states) traffic states, and data on all the traffic states, have been used to investigate the effect of limiting the data only to certain traffic states. Among the three models, the one estimated with the data on all traffic states provided the jam density closest to the actual value. So, the lack of data corresponding to various traffic states leads to the biased estimates of the model parameters. Further, most of the times, the presence of wide scatter in the speed-density relationship complicates the calibration process. The wide scatter is due to the consideration of non-stationary data points in the data. Cassidy's approach has been used to identify the stationary periods, and the speed and flow values are calculated for each stationary period. When the Greenshields model has been estimated using the data related to all the traffic states jam density was found to be much higher than the actual value. The reason was that some of the congested traffic states were underestimated. To overcome this, density values corresponding to those traffic states (congested states) have been observed from the video films collected from the field. When the Greenshields model has been estimated based on the revised density values, the estimated parameters were found to be close to the actual values. Getting the data related to all the traffic states is a complicated endeavour and therefore the present study uses CA model to generate the macroscopic relationships. The present study used the calibration method described by Van Aerde and Rakha (1995) for the speed-density models. The speed-density model of Del Castillo and Benitez provided the best fit to the data.

Chapter 4

Selection of Equivalency Criterion for PCE Estimation

4.1 Background

The PCE values are estimated for a particular level of service which represents specific operating conditions as perceived by the users of a roadway. Therefore, one needs to distinguish between different levels of service for estimating the PCE values. Equivalency criterion refers to the performance measure which defines the level of service for a particular roadway. The use of the performance measure varies depending on the type of facility. Stream speed, the speed of passenger car, density, area are the commonly used performance measures for estimating the PCE values of uninterrupted facilities. The present study uses the method of Sumner et al. for estimating the PCE values of multiple types of vehicles. All the results shown in this chapter are estimated based on the Sumner et al.'s method. Each of the above-mentioned performance measures will be evaluated for their suitability against the heterogeneous traffic stream on rural highways. Mean absolute percentage error (MAPE) in estimating the heavy vehicle adjustment factor (f_{HV}) was used for comparing the different performance measures. The chapter consists of seven sections. Section 4.2 presents the different performance measures used for the PCE estimation in this study. It further elaborates the various criteria that can be used while choosing a certain performance measure. Section 4.3 discusses the various steps involved in Sumner's method for getting the individual PCEs using different performance measures. Section 4.4 deals with the PCE values estimated using stream speed as the equivalency criterion. Section 4.5 includes the PCE values obtained using the speed of passenger car as the equivalency criterion. Section 4.6 deals with the PCE values using density as the equivalency criterion. Section 4.7 deals with the PCE values estimated using speed drop as the equivalency criterion. Section 4.8 presents the PCE values estimated using the area occupancy as the equivalency criterion. Finally, section 4.9 incorporates the summary of the chapter along with some important observations.

4.2 Performance Measures and Selection Criteria

Average stream speed, the speed of passenger car, density, number of vehicle hours, v/c ratio, headway, delay, number of passing maneuvers, area occupancy are the commonly considered performance measures for the PCE estimation. Microscopic parameters such as the delay, the number of passing maneuvers, headway are difficult to measure from the field. However, one of the bases of the selection of the performance measure is that it should be easily measurable from the field (Mallikarjuna and Rao, 2006b). Further, it should give increasing trend of the PCEs with traffic flow rate or LOS (Huber, 1982). Most importantly, the performance measure should be relatable to the concept of LOS for a particular roadway (Roess and Messer, 1984; Krammes and Crowley, 1986). Keeping these things in mind, the performance measures such as the stream speed, the speed of passenger car, density, v/c ratio, and area occupancy have been considered in the present study for the analysis of the PCE values using the Sumner et al.'s method. Most of these performance measures are quite well known in the literature, except area occupancy and therefore it is briefly explained here. Area occupancy, defined by Mallikarjuna and Rao (2006a), is the time occupancy of a vehicular area (a_i) divided by the area of the measurement region and is shown in equation (4.1).

$$Area\ Occupancy\ (AO) = \frac{\left(\frac{\sum_{i=1}^{n} t_i \times a_i}{T}\right)}{(W \times L)} \tag{4.1}$$

Where, W is the width and L is the length of the measurement region; t_i is the time spent by the ith vehicle of area a_i; T is the observation period; and n is the number of vehicle types crossing a stationary observer during T. Therefore, out of the available measures, area occupancy is considered in this study for the PCE estimation.

Drivers on rural highways expect free-flow conditions and do not want their speeds to drop below free-flow speed. This is mainly because on rural highways drivers seldom experience congestion and therefore their expectation of the travel speed is always close to the free-flow speed. To account such expectations of the drivers, in this study, speed drop

(SD) was considered as the performance measure for rural highways. The speed drop can be defined as the difference between the average free-flow speed (FFS), and space mean speed (SMS) of the traffic stream at a particular traffic condition relative to that of the average free-flow speed, expressed in percentage.

$$\text{Speed Drop (\%)} = \left\{\left[\frac{(\text{Average FFS of the stream}) - (\text{SMS of the stream})}{(\text{Average FFS of the stream})}\right] \times 100\right\} \quad (4.2)$$

At equal speed drop, it was assumed that both the streams would experience a similar level of service (LOS). This is quite logical as the drivers will experience the same LOS in both the streams if their speed drops are same (Figure 5.1). Different drop in speeds can be related to the different levels of service for the rural highways.

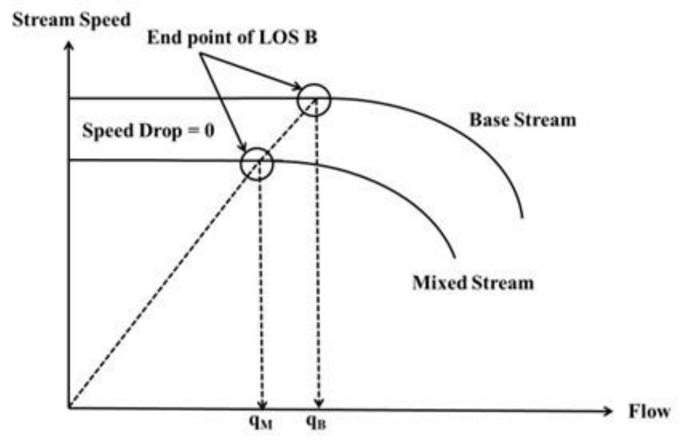


Figure 4.1 Speed-Flow Diagram with Speed Drop as the performance measure

4.3 PCE estimation approach

The steps involved in Sumner et al.’s approach are given below.

1. The flow-performance measure relations have to be generated with the help of the simulation model for the passenger car only traffic stream, called base stream.
2. For this, traffic on the selected road stretch has to be simulated for a range of density values starting from 0 to the density point beyond which vehicles will move no further, i.e., the jammed conditions. The relationships mentioned above were obtained by fitting the speed-density model through the simulated data.

3. Utilising the typical vehicle mix that contains the passenger cars, heavy vehicles, MThWs, and MTWs, simulation runs are to be made for the densities up to the jam level excluding the vehicle type for which the PCE value is estimated (subject vehicle). The proportion of the subject vehicles is added to the passenger cars, and the resulting stream is called the mixed stream in this study. The corresponding flow-performance measure relationships have to be obtained by fitting the appropriate model to the data.
4. Finally, the traffic stream with all the vehicle types present in the traffic mix (called subject stream in the present study) has to be simulated. At an equal performance of the traffic streams, three flow values have to be selected, as already shown in Figure 2.2. The PCE value of a subject vehicle type was then calculated using the equation 2.8.

4.4 PCE Values Using Stream Speed as the Equivalency Criterion

Speed is a performance measure immediately experienced by all the users on a roadway. It also provides a clear indication of how smoothly a facility is operating. Various researchers such as Huber (1982), Okura and Shapit (1995), Elefteriadou et al. (1997), Tiwari et al. (2000) have used stream speed as the equivalency criterion for the homogeneous traffic. The present study checks the suitability of this performance measure for the heterogeneous no lane-disciplined traffic. For estimating the PCE values at an equal value of the stream speed, the macroscopic variables such as the stream speed and flow data are obtained from the simulation model. The heavy vehicle adjustment factor, f_{HV} can be estimated using the equation shown below:

$$f'_{HV} = {}^{1}\!/_{(1 + p_1(E_1 - 1) + p_2(E_2 - 1) + p_3(E_3 - 1))} \tag{4.3}$$

Where, E_1, E_2, E_3 are the PCE values of HMV, MThW, and MTW, respectively. The actual f_{HV} can be calculated using the following equation:

$$f_{HV} = \frac{Flow\ in\ subject\ stream\ (q_s)}{Flow\ in\ base\ stream\ (q_B)} \tag{4.4}$$

The percentage error will be equal to,

$$Percentage\ error\ in\ f_{HV}, \in = \frac{f'_{HV} - f_{HV}}{f_{HV}} \times 100 \tag{4.5}$$

Table 4.1: PCE and Error values of four-lane divided road using stream speed

Speed (km/h)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
57	2.88	1.54	0.43	0.68	0.57	20.46
53	2.27	1.49	0.45	0.78	0.71	8.73
49	1.98	1.41	0.49	0.83	0.75	10.00
45	1.90	1.38	0.57	0.83	0.77	7.69
41	1.87	1.30	0.61	0.84	0.79	6.33
37	1.86	1.26	0.66	0.83	0.80	4.46

Table 4.1 presents the PCE values of HMV, MThW, and MTW estimated at different stream speeds observed on a four-lane highway. The average traffic composition of the stream is 45% passenger cars, 28% HMV, 19% MTW, and 8% MThW. The absolute values of the errors were averaged across different stream speed values to get the mean absolute percentage error (MAPE) for a particular traffic mix. The MAPE for a four-lane road came out to be 9.61% using stream speed as the equivalency criterion. In the case of a six-lane divided road, the average traffic composition is 72% passenger cars, 15% HMV, 3% MThW, and 10% MTW. Table 4.2 shows the PCE and the corresponding error values at different stream speeds. The range of stream speed is selected based on the minimum of the free-flow speed and speed at capacity among the three curves such as base, mixed and subject streams generated for each vehicle type using the selected PCE estimation approach in this study. The MAPE for the six-lane divided road came out to be 11.07% using stream speed as the equivalency criterion.

Table 4.2: PCE and Error values of six-lane divided road using stream speed

Speed (km/h)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
74	2.13	1.25	0.41	0.89	0.77	16.23
69	2.00	1.22	0.43	0.91	0.80	13.00
64	1.96	1.21	0.47	0.91	0.82	10.91
59	1.92	1.17	0.52	0.91	0.83	9.54
54	1.91	1.15	0.54	0.91	0.84	8.72
49	1.89	1.09	0.59	0.91	0.85	8.02

The decrease in stream speed corresponds to the increase in traffic flow rate and results due to the increased interaction between the different types of vehicle present in the traffic stream. From the error analysis similar to that shown in Figure 4.2, it was found that the stream speed as equivalency criterion overestimates the PCE values near free-flow conditions. This is in agreement with the statement made by Okura and Sthapit (1995) saying that stream speed can be only used as equivalency criterion near capacity conditions. As the traffic condition reaches capacity, the error comes out to be less, and therefore this validates the notion that stream speed produces reasonable results only close to capacity.

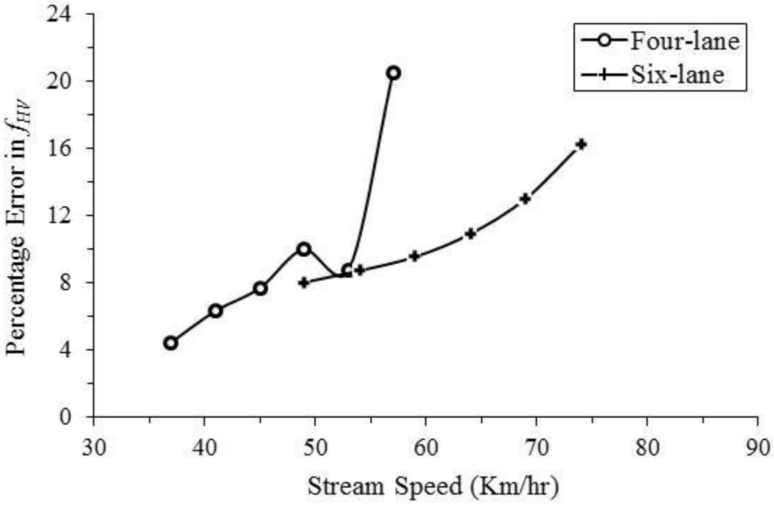


Figure 4.2 Variation of Error values with stream speed for four-lane and six-lane divided roads

Furthermore, the PCE values of larger sized HMV and MThW exhibit a decreasing trend with the increasing traffic flow. This can be observed from the Figure 4.3, showing the variation of PCEs with the stream speed. For HMV and MThW, at higher speeds (lesser flows) the PCEs are higher and lower at the lower speeds (higher flows). Huber (1982) made similar observations on the PCE of trucks for multilane rural highways, moving under free-flow conditions.

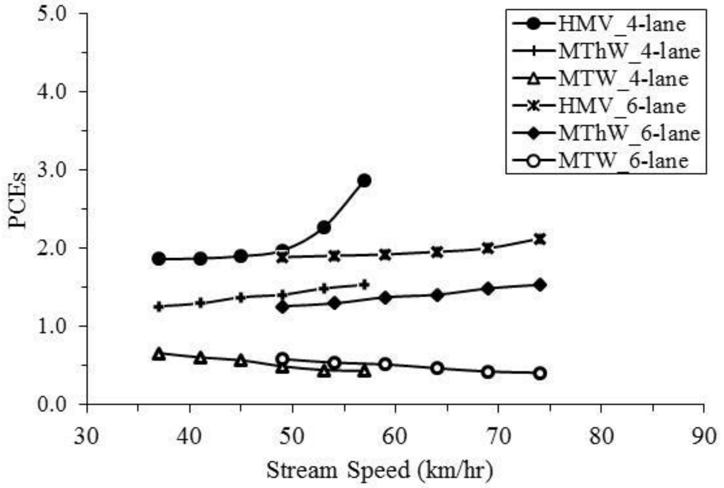


Figure 4.3 Variation of PCEs with stream speed in the case of four-lane and six-lane divided roads

4.5 PCE Values Using Speed of Passenger Car as the Equivalency Criterion

The speed of passenger car was used as the equivalency criterion by St. John (1976), Huber (1982), and Bang et al. (1995) for estimating the PCE values. Indonesian Highway Capacity Manual (IHCM) uses speed of passenger car as the LOS measure for rural highways (IHCM, 1997). IHCM estimated the PCE values of HMV, MThW, and MTW using the speed of passenger car as the equivalency criterion. Table 4.3 shows the PCE values for a heterogeneous traffic stream with 45% passenger cars, 28% HMV, 19% MTW, and 8% MThW, moving on a four-lane divided road. It may be seen that the MAPE in estimating f_{HV} is very large (28.59%), in case of the four-lane divided road using this equivalency criterion.

Table 4.3: PCE and Error values of four-lane divided roadway using speed of passenger car

Speed of Passenger Car (km/h)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
58	3.23	2.02	0.71	0.61	0.47	27.83
54	3.00	1.96	0.63	0.64	0.50	28.59
50	2.89	1.95	0.59	0.66	0.51	28.68
46	2.81	1.94	0.52	0.67	0.52	29.12
42	2.75	1.92	0.52	0.68	0.53	28.74
35	2.66	1.87	0.48	0.70	0.54	28.58

For the six-lane divided road, the variation in the PCE values is very high as compared to the four-lane divided road corresponding to the average traffic stream composition.

Table 4.4: PCE and Error Values for six-lane divided road using speed of passenger car

Speed of Passenger Car (km/h)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
79	4.77	2.08	0.75	0.64	0.48	32.86
72	3.47	2.03	0.71	0.73	0.56	30.96
65	3.40	1.97	0.66	0.74	0.58	27.73
58	3.28	1.86	0.56	0.76	0.59	28.01
51	2.27	1.82	0.49	0.86	0.60	43.37
44	2.05	1.75	0.43	0.89	0.61	46.64

The PCE values tend to decrease with the increasing flow rate. The same can be seen in Figure 4.4 which shows the variation of PCEs with the speed of passenger car (corresponds to the increase in flow rate). The PCE Values and error values obtained using the speed of passenger car as the equivalency criterion are presented in Table 4.4.

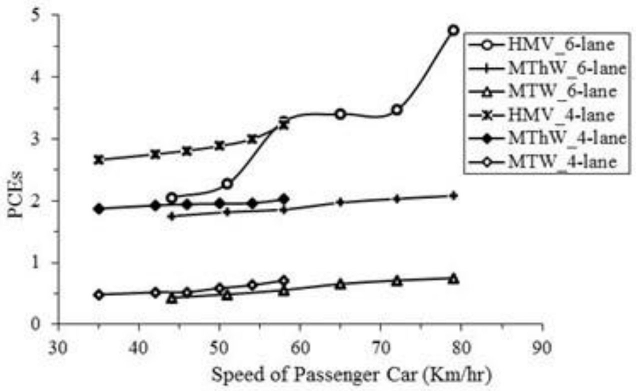


Figure 4.4 Variation of PCEs with speed of passenger car for four-lane & six-lane roads

This is also evident from the variation of percentage error in f_{HV} as shown in Figure 4.5. The MAPE came out to be 34.93% in the case of the six-lane divided road using the speed of passenger cars as the equivalency criterion.

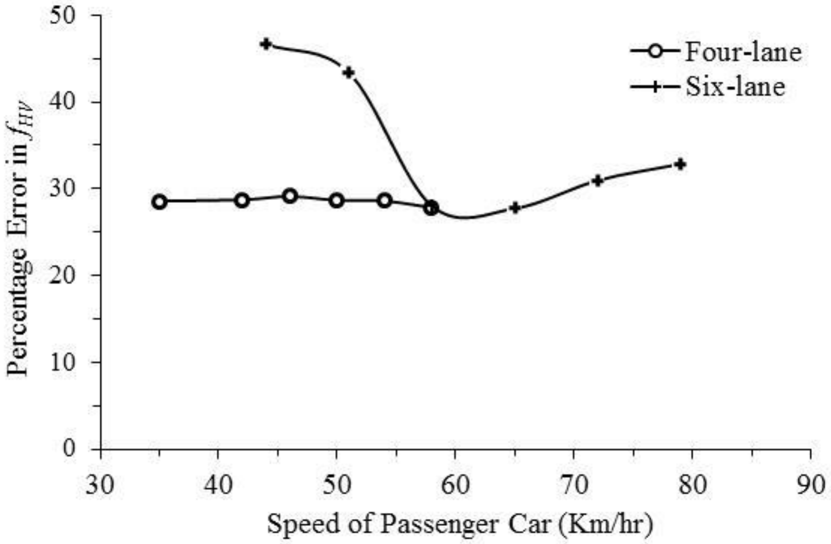


Figure 4.5 Variation of Error values with the speed of passenger car in the case of four-lane and six-lane divided roads

4.6 PCE Values Using Density as the Equivalency Criterion

Density is a surrogate measure of the proximity of the vehicles to each other and can be indirectly related to the freedom of maneuver within the traffic stream. Various researchers such as Huber (1982), Okura and Shapit (1995), Webster and Elefteriadou (1999), Demarchi and Setti (2003) have used density as the equivalency criterion for estimating the PCE value of heavy vehicles. US HCM (2000) and HCM (2010) have used density as the LOS measure and the equivalency criterion for multilane highways and freeways. The present study also analyses the suitability of density as the equivalency criterion for the heterogeneous traffic. Employing the speed-flow relationships developed for the base, mixed and subject streams, the PCE values of HMV, MThW and MTW are estimated one by one for a given traffic mix. Table 4.5 shows the PCE values and the associated errors in estimating the heavy vehicle adjustment factor (f_{HV}) for the four-lane divided road. The MAPE came out to be 4.31% for four-lane divided road corresponding to the given traffic mix.

Table 4.5: PCE and Error values for four-lane divided road using density

Density (veh/h)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
14	1.55	1.16	0.75	0.89	0.86	3.86
28	1.57	1.21	0.69	0.89	0.86	4.00
32	1.58	1.24	0.66	0.90	0.86	4.26
44	1.59	1.31	0.62	0.89	0.86	4.26
56	1.62	1.33	0.59	0.89	0.85	5.18

The PCE values for the six-lane divided road do not change much with the change in the density. The MAPE came out to be 3.86% for this road related to the given traffic mix.

Table 4.6: PCE and Error values for six-lane divided road using density

Density (veh/h)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
21	1.56	1.11	0.73	0.94	0.90	4.75
33	1.59	1.18	0.70	0.94	0.90	4.46
48	1.65	1.23	0.64	0.94	0.90	3.85
66	1.68	1.27	0.62	0.93	0.90	3.20
84	1.70	1.34	0.57	0.93	0.90	3.02

The PCE values of the larger sized HMV and MThW tend to increase with the increasing traffic flow rate. The similar observations were made by Huber (1982), Webster and Elefteriadou (1999) while estimating the PCE value of the trucks. The Variation of PCEs with density for four-lane and six-lane divided roads is shown in Figure 4.6.

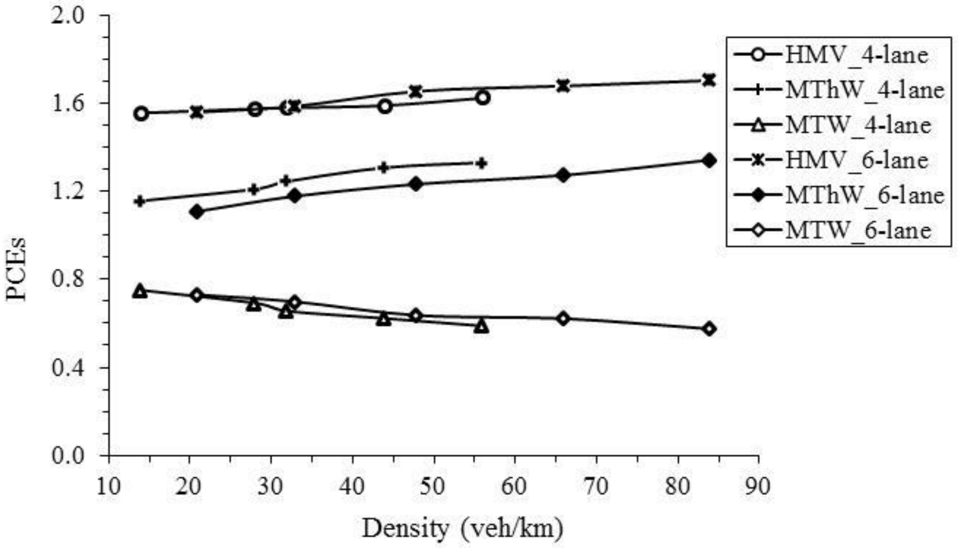


Figure 4.6 Variation of PCEs with density for four-lane and six-lane divided roads

For the four-lane divided road, the error value tends to increase with the increasing flow rate whereas opposite trend is observed in the case of six-lane divided road and is shown in Figure 4.7.

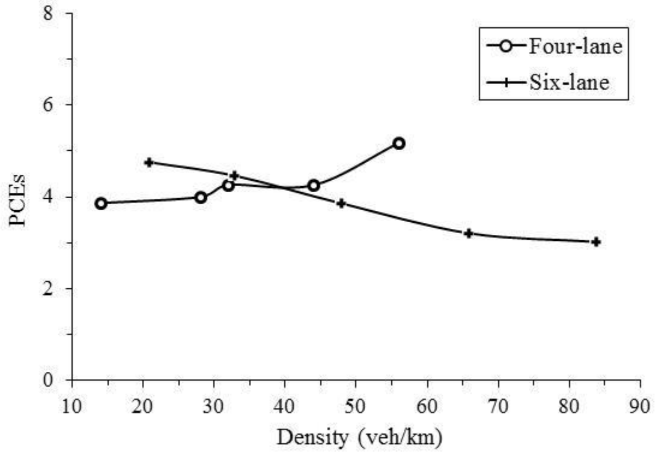


Figure 4.7 Variation of Error values with density for four-lane and six-lane divided roads

4.7 PCE values using speed drop as the equivalency criterion

On rural highways, drivers expect the free-flow conditions and do not want the speed to drop below the free-flow speed. Also, the driver's expectation on the speed-drop may not be the same across the geographical locations. On the rural highways drivers seldom experience congestion and therefore their expectation of the travel speed is always close to the free-flow speed. To account for such expectations of the drivers, speed drop was considered as the performance measure for rural highways in this research. The speed drop can be defined as the difference between the average free-flow speed (FFS) and the space-mean-speed (SMS) of the traffic stream at a particular traffic condition relative to that of the average free-flow speed, expressed in percentage. At equal speed drop, it was assumed that both the streams would experience the similar level of service (LOS). This is quite logical as the drivers will experience the same LOS in both the streams if their speed drops are same. Different drop in speeds can be related to the different levels of service for the rural highways, e.g., zero percent speed drop means free-flow conditions. The PCE values of HMV, MThW, and MTW are estimated for a particular traffic mix on the four-lane divided road using the Sumner et al.'s method. The PCE and the error (in f_{HV} estimation) values are shown in Table 4.7. The MAPE came out to be 3.40% using speed drop as the equivalency criterion.

Table 4.7: PCE and Error values for four-lane divided road using speed drop

Speed Drop (%)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
0	1.66	1.11	0.58	0.90	0.88	2.24
10	1.75	1.15	0.52	0.88	0.86	3.37
20	1.87	1.27	0.47	0.86	0.82	4.58
30	1.94	1.36	0.46	0.84	0.81	4.20
40	2.03	1.41	0.44	0.82	0.80	2.60

The PCEs and Error values for four-lane and Six-lane divided road using speed drop as the equivalency criterion are presented in Table 4.7 and Table 4.8 respectively. The PCE values of the six-lane divided road do not vary significantly with the speed-drop, similar to that happened in the case of density. The MAPE came out to be 4.28% using the speed drop as the equivalency criterion.

Table 4.8: PCE and Error values for six-lane divided road using speed drop

Speed Drop (%)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
0	1.56	1.03	0.48	0.97	0.95	1.59
10	1.56	1.05	0.45	0.97	0.94	3.43
20	1.59	1.10	0.41	0.97	0.92	4.91
30	1.65	1.13	0.41	0.96	0.91	5.60
40	1.66	1.15	0.40	0.96	0.91	5.87

The PCE value of larger sized HMV and MThW increases with the increase in the speed drop whereas the PCE value of two-wheelers decreases with the increase in the speed drop. The variation of Error values with speed drop for the four-lane and six-lane divided roads are shown in Figure 4.9.

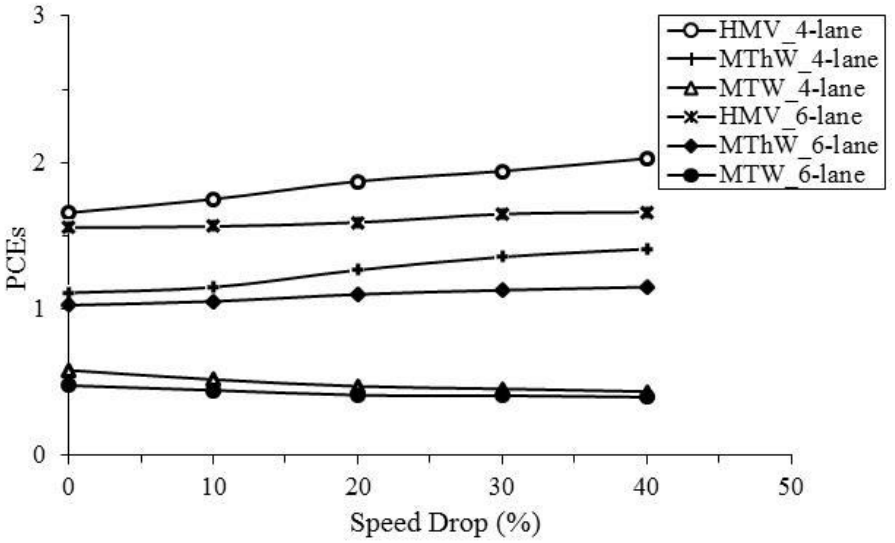


Figure 4.8 Variation of PCEs with speed drop for four-lane and six-lane divided roads

For the six-lane divided road, the percentage error in f_{HV} increases with the increasing speed drop. In the case of the four-lane divided road, the error value first increases up to the speed drop of 20% and then decreases.

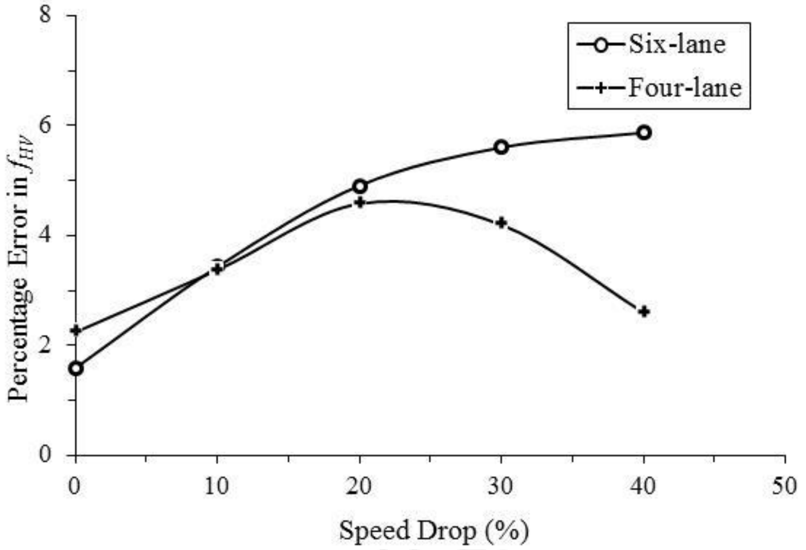


Figure 4.9 Variation of Error values with speed drop for four-lane and six-lane divided roads

4.8 PCE values using area occupancy as the equivalency criterion

Occupancy represents percent of time a detector is occupied by a particular length of vehicle and is used for getting the density of vehicles present in the homogeneous traffic stream. However, in the case of heterogeneous traffic, vehicles with varying static and dynamic characteristics occupy the road space. Therefore, occupancy which considers only vehicle's length cannot be used as a measure of space occupancy of the vehicles present in the traffic stream. Mallikarjuna and Rao (2006b) proposed a new performance measure termed area occupancy for characterising the heterogeneous traffic stream. Using area occupancy as the equivalency criterion, the present study estimates the PCE values of HMV, MThW, and MTW for the four-lane divided road. The error values corresponding to the different area occupancy values are shown in Table 4.9, and the MAPE for the four-lane divided road is 3.88%.

Table 4.9: PCE and Error values for four-lane divided road using area occupancy

Area Occupancy (%)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
1	1.45	1.05	0.63	0.94	0.88	7.3
2	1.54	1.18	0.57	0.92	0.88	4.9
3	1.61	1.20	0.51	0.91	0.88	4.0
4	1.68	1.23	0.49	0.90	0.88	2.3
5	1.73	1.28	0.46	0.89	0.88	0.9

PCE values of HMV, MThW, and MTW moving on six-lane divided road along with the error in f_{HV} estimation are shown in Table 4.10. The MAPE came out to be 2.30%, using area occupancy as the equivalency criterion.

Table 4.10: PCE and Error values for six-lane divided road using area occupancy

Area Occupancy (%)	HMV	MThW	MTW	Estimated f_{HV}	Actual f_{HV}	Percentage Error
1	2.07	1.05	0.58	0.89	0.87	2.24
2	2.09	1.15	0.54	0.89	0.87	1.99
3	2.09	1.18	0.50	0.89	0.87	2.37
4	2.12	1.20	0.48	0.89	0.87	2.22
5	2.16	1.26	0.45	0.89	0.86	2.67

PCE value of HMV, MThW increases with the increasing area occupancy whereas the PCE value of MTW decreases with the increase in area occupancy. It can be seen from Figure 4.8 that the PCE value of MThW and MTW on four-lane and six-lane divided roads do not vary with the change in area occupancy. The variation of Error values with speed drop in the case of four-lane and six-lane divided roads are shown in Figure 4.10. The variation of error values with area occupancy in the case of four-lane and six-lane divided roads are shown in Figure 4.11.

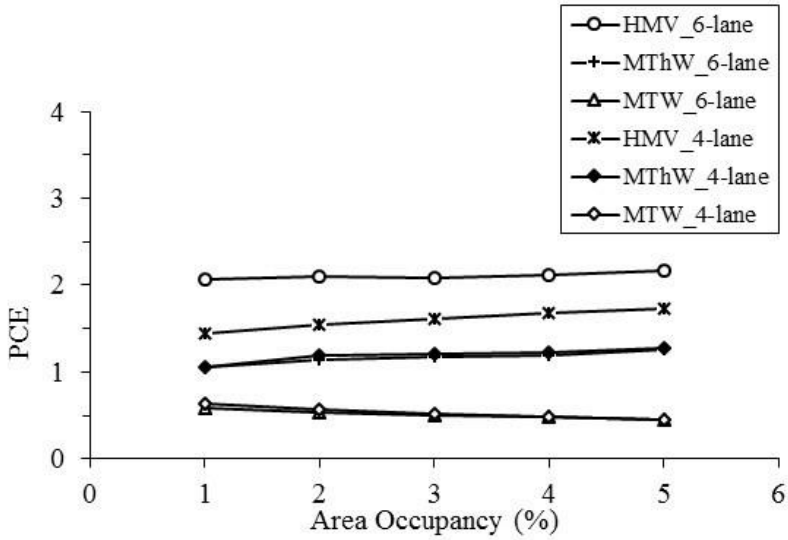


Figure 4.10 Variation of PCEs with area occupancy for four-lane and six-lane divided roads

The percentage error in f_{HV} exhibits a decreasing trend with the increasing area occupancy for four-lane divided road whereas in the case of the six-lane divided road, the error increases with the area occupancy.

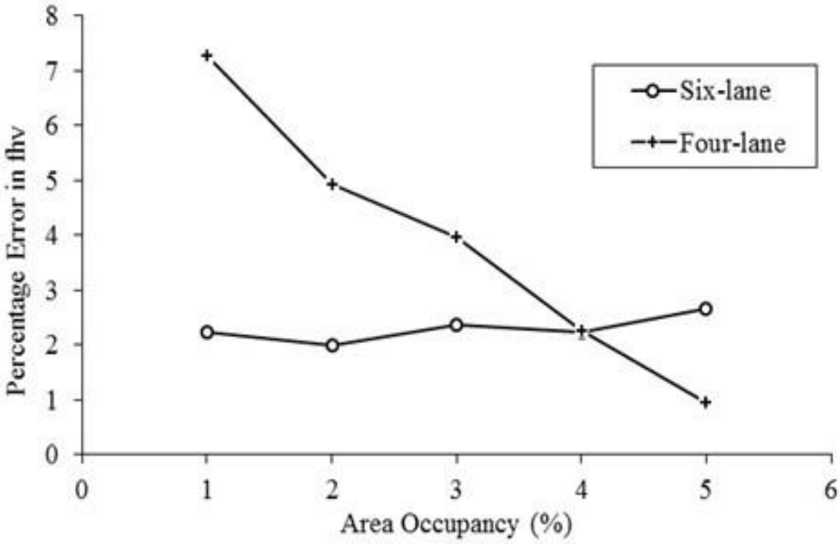


Figure 4.11 Variation of Error values with speed drop of four-lane and six-lane divided roads

4.9 Summary

Selection of the equivalency criterion is one of the main challenges of the PCE estimation. PCE factors are derived for a particular level of service. The equivalency criterion refers to the performance measure which defines the different levels of service for which the PCE values are derived. Several performance measures such as the stream speed, the speed of passenger car, density, area occupancy have been used as the equivalency criterion for uninterrupted facilities. However, there is no consensus among the researchers regarding the use of a particular equivalency criterion. US HCM suggests density as the equivalency criterion since the level of service is defined in terms of density in the case of freeways and multilane highways. Indonesian HCM uses the speed of passenger car as the equivalency criterion on interurban highways (IHCM, 1997). Huber (1982) stated that the equivalency criterion should be decided on the basis that the PCE value of trucks should increase with the increase in traffic flow rate for a given traffic mix. Mallikarjuna and Rao (2006b) further stated that performance measure should be easily measurable in the field. Still, there is no quantitative analysis of the different equivalency criteria over the estimation of the PCEs as far as the previous studies are concerned. Most of the studies selected the equivalency criterion on the basis of the qualitative assessment of the different performance measures. The present study therefore compared the different performance measures in terms of MAPE in f_{HV} estimation and finally selected the equivalency criterion for further analysis. Stream speed and speed of passenger car exhibits a decreasing trend of the PCE values. The change in the value of a particular performance measure is happening due to the change in traffic flow rate.

Table 4.11: MAPE values for the different performance measures

Road type	Density (Veh/km)	Speed Drop (%)	Area Occupancy (%)	Stream Speed (Km/hr)	Speed of Passenger Car (Km/hr)
Four-lane divided road	4.31	3.40	3.88	9.61	28.59
Six-lane divided road	3.86	4.28	2.30	11.07	34.93

Further, it was found that the stream speed and the speed of passenger car result in high error values as shown in Table 4.11. Speed drop, area occupancy, and density provided the increasing trend of the PCEs (with the flow rate) and also less error values. However, density is difficult to measure form the field. Therefore, the present study selects area occupancy and speed drop as equivalency criterion for estimating the PCE values for rural highways.

Chapter 5

Results and Analysis

5.1 General

This chapter presents the PCE calculations for varying compositions of the traffic stream containing Passenger Cars, Heavy Motor Vehicle, Motorised Three-Wheelers, and Motorised Two Wheelers, using Sumner et al.'s approach. Performance measures such as Speed Drop (SD) and Area Occupancy (AO) were used for estimating the PCE values for Four-lane and Six-lane divided rural highways. After estimating the PCEs, the accuracy of the PCE values was checked using the error in f_{HV} estimation. Finally, the aggregate PCE values were calculated for the four-lane and six-lane divided rural highways using the speed drop and area occupancy as performance measures. This chapter is divided into six sections. Section 5.2 discusses the traffic parameters used as inputs to the simulation model. Section 5.3 includes the PCE values estimated using speed drop and area occupancy as performance measures for the different traffic mixes on rural highways. Section 5.4 further includes the analysis of constant PCEs obtained after certain approximation. Section 5.5 provides the aggregate PCE values for different traffic mixes and their variation with increasing traffic flow rates on rural highways. Section 5.6 finally summarises the work presented in this chapter.

5.2 CA model Parameters

For simulating the heterogeneous traffic observed on rural highways passing through level terrain, input data corresponding to the different traffic and vehicular characteristics were adapted from a previous study (Mehar, 2013). These road sections have a width of 7 m (four-lane divided) and 10.5 m (six-lane divided) and located on the Chennai-Villupuram highway and Mumbai-Pune Expressway, respectively. The inputs such as the speed parameters and traffic composition for four-lane and six-lane divided highways are shown in Table 5.1. The parameters of CA model which are used to generate the macroscopic relationships for four-lane and six-lane divided roads are shown in Table 5.2.

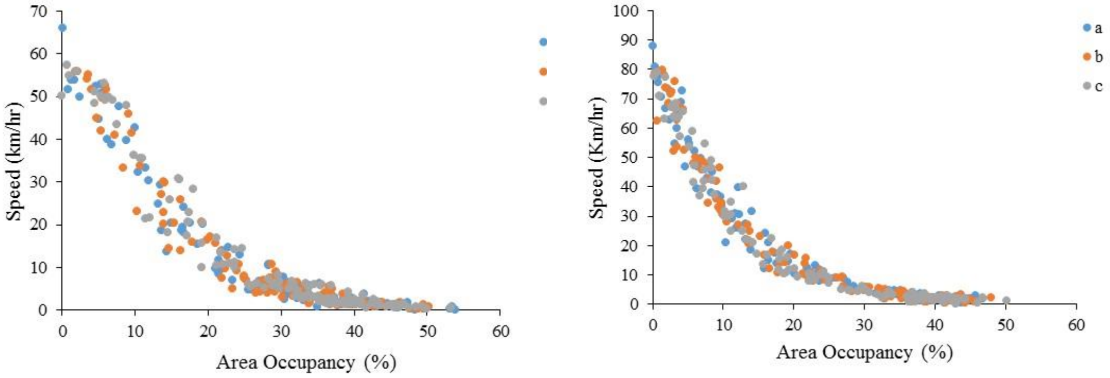
Table 5.1: Traffic parameters used in the CA model

Vehicle type	Average Traffic composition (%)		Speed Parameters (km/h)			
			Four-lane		Six-lane	
	Four-lane	Six-lane	Maximum speed	Mean Free-flow speed	Maximum speed	Mean Free-flow speed
Passenger Car	45	72	102	67	131	87
HMV	28	15	85	50	87	59
MThW	8	3	70	49	76	45
MTW	19	10	100	61	105	75

Table 5.2: CA parameters for the Four-lane & Six-lane divided roads

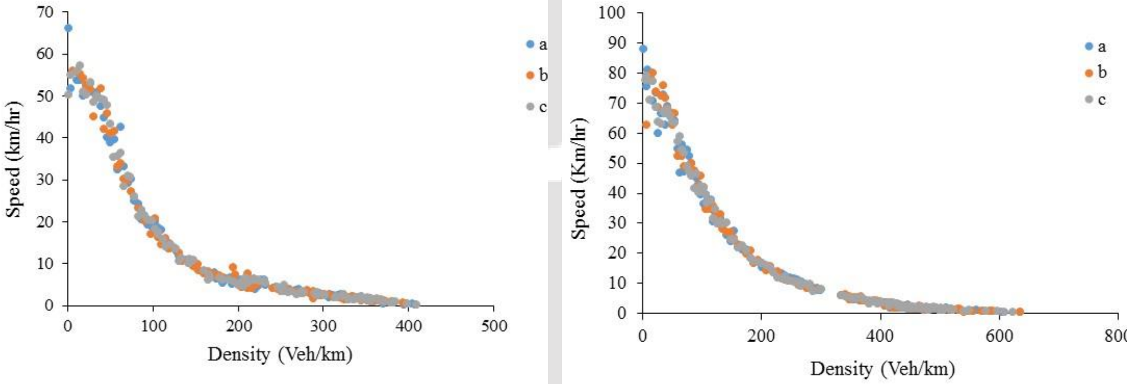
Parameters corresponding to Updating Model								
Parameters	Passenger Car		HMV		MThW		MTW	
	Four-lane	Six-lane	Four-lane	Six-lane	Four-lane	Six-lane	Four-lane	Six-lane
Slow down probability (P_{dec})	0.3	0.3	0.1	0.1	0.3	0.3	0.1	0.1
Slow-to-start probability (P_o)	0.1	0.2	0.5	0.5	0.5	0.5	0.1	0.1
Break light probability (P_{bl})	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Minimum Gap (Cells)	4	4	4	4	4	4	4	4
Interaction Headway (t^{th}) (Sec)	2	4	3	5	3	4	2	3
Security distance ($d_{security}$)	10	10	12	12	12	12	10	10
Parameters corresponding to Lane change Model								
Probability in lane change (P)	0.95	0.95	0.6	0.6	0.6	0.7	0.95	0.95
Multiplication parameter (α)	1.0	1.0	1.1	1.1	1.0	1.2	1.0	1.0
Back gap ($\beta * v_t^b + \Delta$) factor (β)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

From the CA model, data corresponding to the speed, flow, density, and area occupancy was extracted. Figures 5.1 (a) and (b), showing the output from multiple simulations (a, b, c), prove that the variation among the macroscopic relations for the different simulation runs is negligible.



Four-lane Six-lane

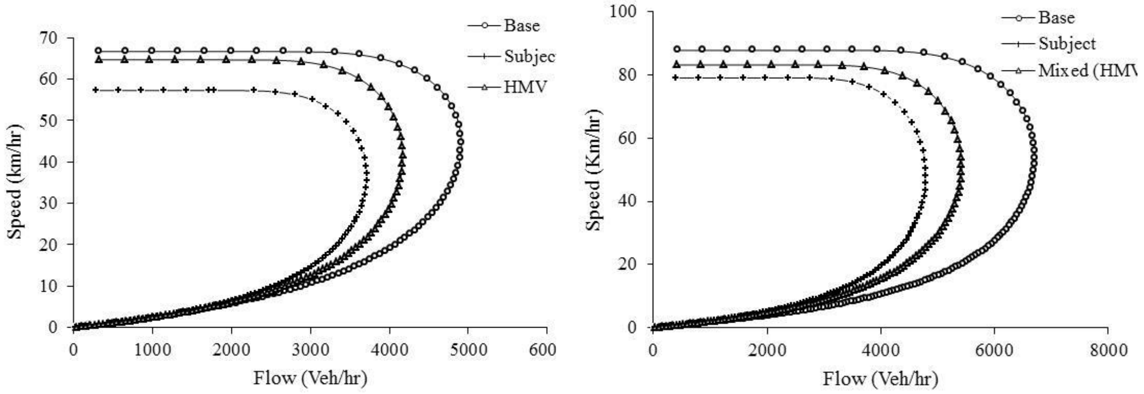
Figure 5.1(a) Speed-Area occupancy relations for multiple runs on Four-lane and Six-lane divided roads



Four-lane Six-lane

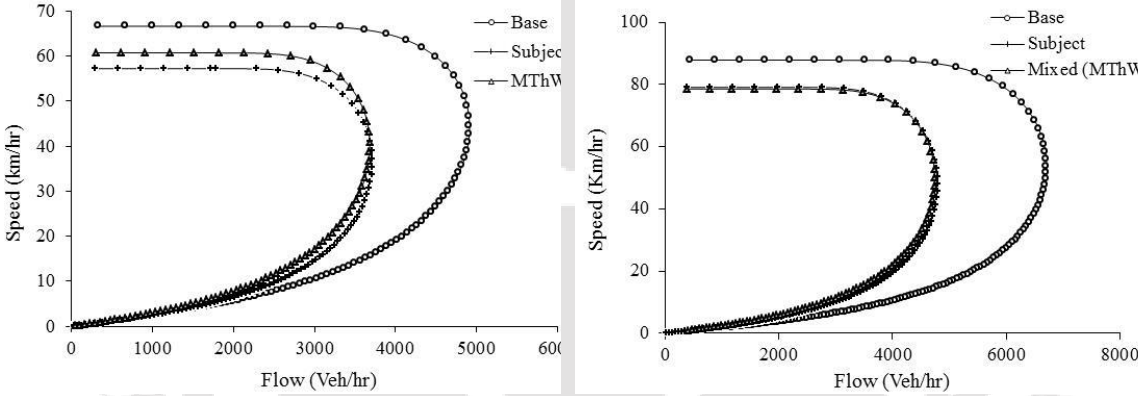
Figure 5.1(b) Speed-Density relations for multiple runs on Four-lane and Six-lane divided roads

Macroscopic relations corresponding to various mixes were obtained based on the speed-density model of Del Castillo and Benitez (1995). Speed-Flow and Flow-Area occupancy relations of the base, mixed and subject streams used for the PCE estimation of HMV, MThW, and MTW, meant for the Sumner et al.'s method, are shown in the Figures 5.2 to 5.7 respectively.



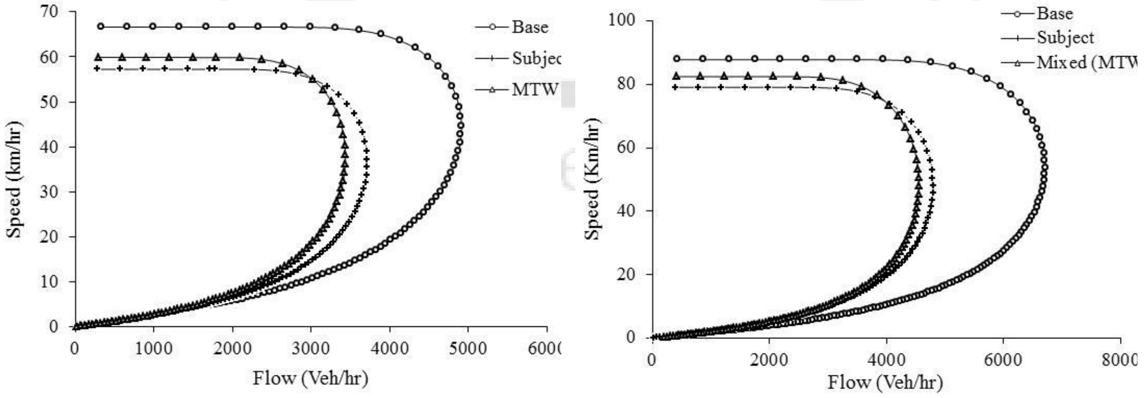
(a) Four-lane divided road (b) Six-lane divided road

Figure 5.2 Speed-Flow relations for the PCE estimation of HMV moving on (a) Four-lane and (b) Six-lane divided roads



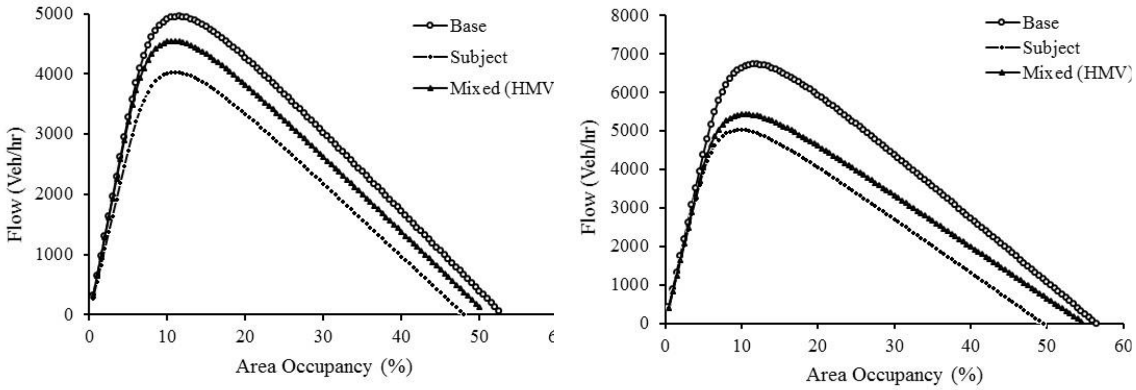
(a) Four-lane divided road (b) Six-lane divided road

Figure 5.3 Speed-Flow relations for the PCE estimation of MThW moving on (a) Four-lane and (b) Six-lane divided roads



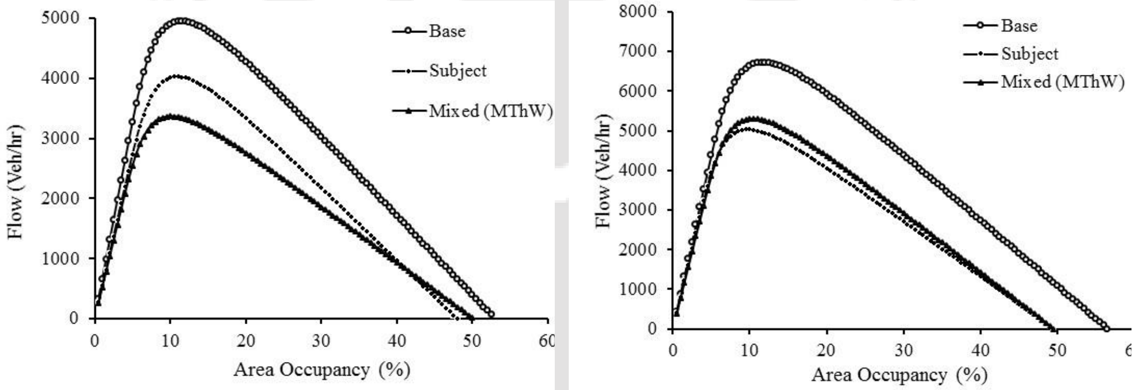
(a) Four-lane divided road (b) Six-lane divided road

Figure 5.4 Speed-Flow relations for the PCE estimation of MTW moving on (a) Four-lane and (b) Six-lane divided roads



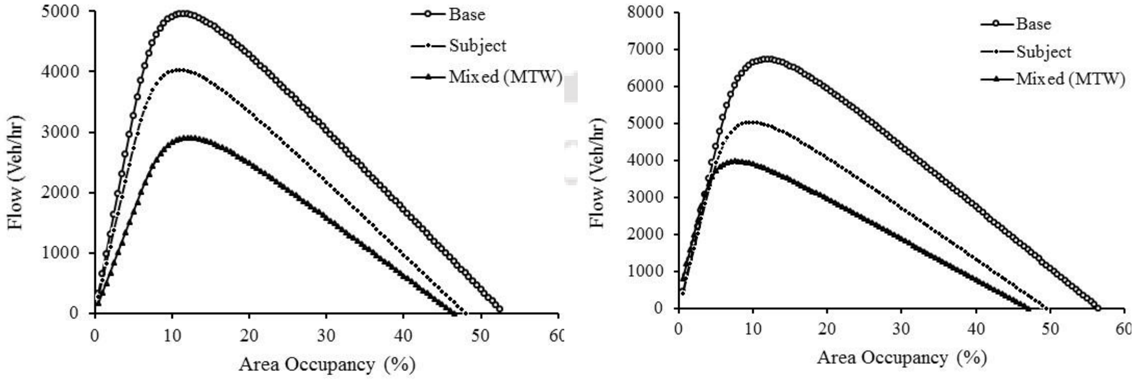
(a) Four-lane divided road (b) Six-lane divided road

Figure 5.5 Area Occupancy-Flow relations for the PCE estimation of HMV moving on (a) Four-lane and (b) Six-lane divided roads



(a) Four-lane divided road (b) Six-lane divided road

Figure 5.6 Area Occupancy-Flow relations for the PCE estimation of MThW moving on (a) Four-lane and (b) Six-lane divided roads

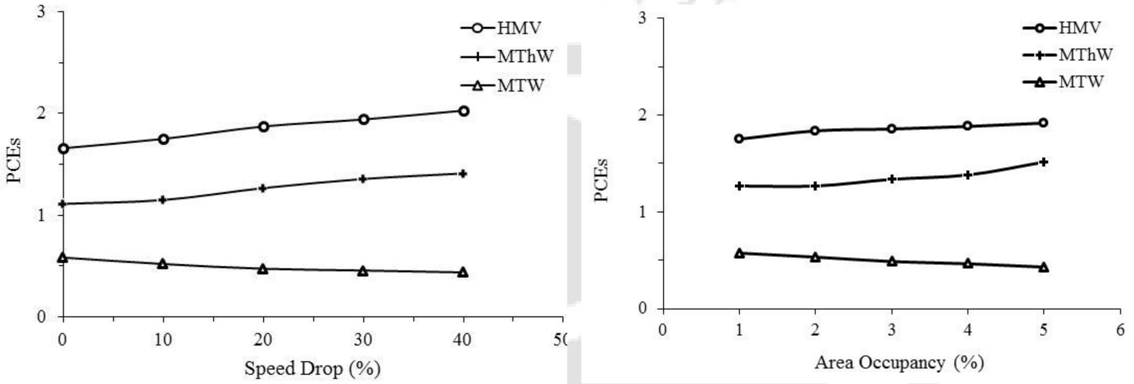


(a) Four-lane divided road (b) Six-lane divided road

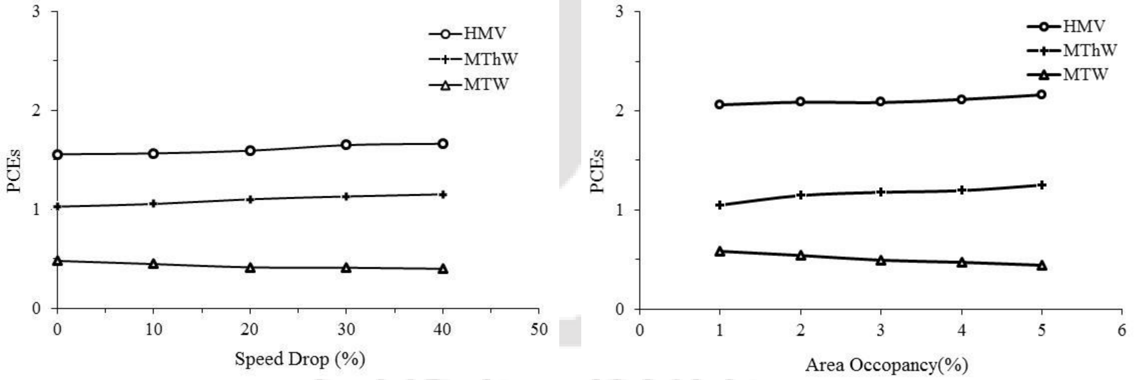
Figure 5.7 Area Occupancy-Flow relations for the PCE estimation of MTW moving on (a) Four-lane and (b) Six-lane divided roads

5.3 Individual PCE Values using Speed Drop and Area Occupancy

At the equal speed drop or area occupancy, the PCE values of HMV, MThW, and MTW moving on the rural highways are estimated using the speed-flow and flow-area occupancy relationships of the base, mixed and subject streams as shown in Figures 5.2 to 5.7. The PCE values are calculated for the different levels of speed drop starting from zero to the forty percent at an equal interval of ten percent speed drop. The zero percentage speed drop incorporates LOS A and B, and the speed tends to drop after LOS B in rural highways. This drop in speed happens due to the significant increase in traffic flow rate which leads to the increased interaction between the vehicles present in the traffic stream. Similarly, at the equal area occupancy, the PCE values are estimated for four-lane and six-lane divided roads. The variation of the PCE values of HMV, MThW, and MTW with speed drop and area occupancy for the four-lane and six-lane divided roads are shown in Figure 6.8. For the actual traffic composition (Table 5.1), the PCE values of HMV, calculated using speed drop, vary from 1.66 to 2.03 for the four-lane divided road and 1.75 to 1.92 for the six-lane divided road as shown in Figure 5.8. For MThW, the values range between 1.11 to 1.41 for four-lane divided road and 1.26 to 1.51 for the six-lane divided road. MTWs’ PCEs vary between 0.58 to 0.57 and 0.58 to 0.52 for four-lane and six-lane divided roads, respectively. Area occupancy resulted in the PCEs ranging from 1.58 to 1.83, 1.15 to 1.35, and 0.46 to 0.35 for HMV, MThW, and MTW, respectively, for a four-lane divided road. The PCE values vary between 1.23 to 1.47, 1.01 to 1.15, and 0.40 to 0.28 for HMV, MThW, and MTW, respectively, on a six-lane divided road. The PCE value of HMV increases with the increase in speed drop or area occupancy for both four-lane and six-lane divided roads. The PCE value of MThW also increases with the increase in speed drop or area occupancy (increase in flow rate) for four-lane and six-lane divided roads. Furthermore, the PCE value of MTW decreases with the increase in speed drop or area occupancy for four-lane and six-lane divided roads. Mehar (2013) found that the PCE value of HMV falls in the range of 4.18-3.78 for four-lane divided road and 4.88-3.97 for six-lane divided road corresponding to the percent share of HMV present in the actual traffic mix. The PCE value of HMV was very high at free-flow conditions, i.e., LOS A to B (four-lane divided road: 4.18-4.13; six-lane divided road: 4.88-4.75) which is questionable because the impact of a particular type of vehicle should be less during free-flow conditions. Further, Mehar (2013) found that the PCE value of HMV decreases with the increase in traffic flow rate. As stated by Huber (1982), Krammes and Crowley (1986), Webster and Elefteriadou (1999) that the PCE values of HMV should increase with the increase in traffic flow rate due to the increased interaction among the different vehicle classes.



(a) Four-lane divided road



(b) Six-lane divided road

Figure 5.8 Variation of PCE values of HMV, MThW, and MTW with the speed drop and area occupancy on (a) Four-lane and (b) Six-lane divided roads

5.4 Constant PCE Values Using Speed Drop and Area Occupancy

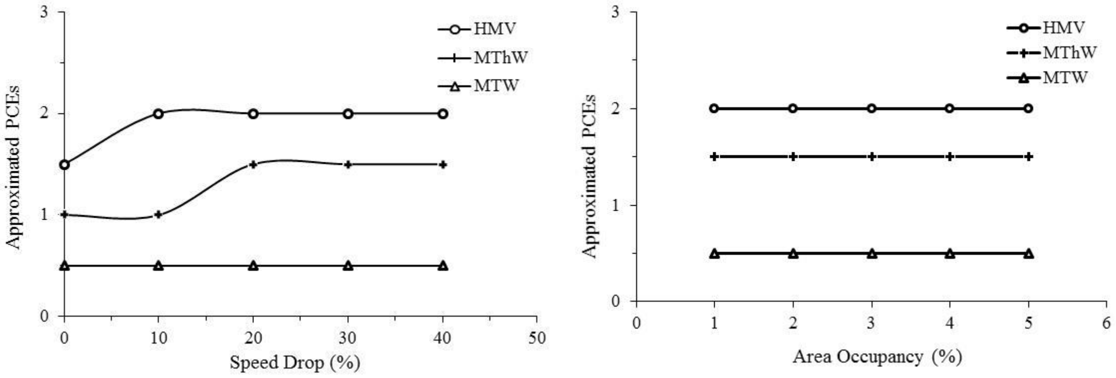
US HCM 1997, 2000 and 2010 provided the PCE values of trucks to the nearest 0.5 for the freeways and multilane highways passing through the level terrain. Further, Roess and

Messer (1984) mentioned that the use of PCEs varying with LOS complicates computations and poses serious problems in the capacity analysis of highways. However, researchers such as Chandra and Sikdar (2000), Arasan and Arkatkar (2008, 2010), Mehar et al. (2014), Dhamaniya and Chandra (2016) stated that for the heterogeneous traffic stream the PCE values might not be the same across the different levels of service.

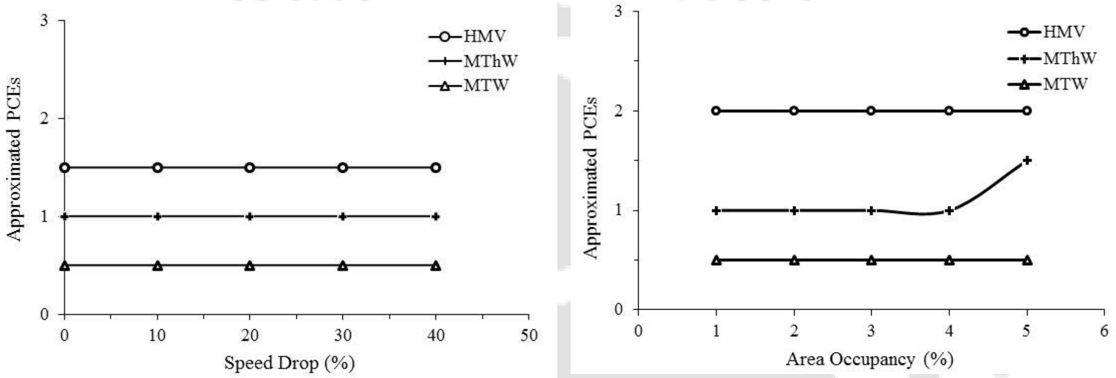
Therefore, to check the suitability of this hypothesis, the present study rounded off the estimated PCEs for the different levels of speed drop/area occupancy. The PCE values are approximated to the nearest 0.5, using a threshold of 0.25 and are shown in Figure 5.9. After rounding off, the PCE values do not turn out to be constant in the present study across the different levels of service. For obtaining the constant PCEs, higher PCE values are considered because, as mentioned by Roess and Messer (1984), "the design benefits of smaller PCE values at low volumes are minimal." Proceeding in this manner, the PCE values of HMV, MThW, and MTW for the actual traffic composition (Table 5.1) came out to be 2, 1.5, and 0.5, respectively for both the speed drop and area occupancy on the four-lane divided road. On the six-lane divided road, the approximated PCE values turned out to be 1.5, 1, 0.5 and 2, 1.5, 0.5 for speed drop and area occupancy, respectively.

Table 5.3: MAPE values of different traffic compositions for constant PCEs

Four-lane divided road			Six-lane divided road		
Traffic composition	MAPE		Traffic composition	MAPE	
	SD	AO		SD	AO
45%Car,47%HMV,8%MThW	7.62	3.12	72%Car,25%HMV,3%MThW	4.23	1.72
45%Car,42%HMV,8%MThW, 5%MTW	0.22	7.79	72%Car,20%HMV,3%MThW, 5%MTW	1.63	3.37
45%Car,37%HMV,8%MThW, 10%MTW	2.75	0.00	72%Car,15%HMV,3%MThW, 10%MTW(*Actual Mix*)	5.40	2.90
45%Car,28%HMV,8%MThW, 19%MTW(*Actual Mix*)	1.99	2.88	72%Car,10%HMV,3%MThW, 15%MTW	2.74	1.02



(a) Four-lane divided road



(b) Six-lane divided road

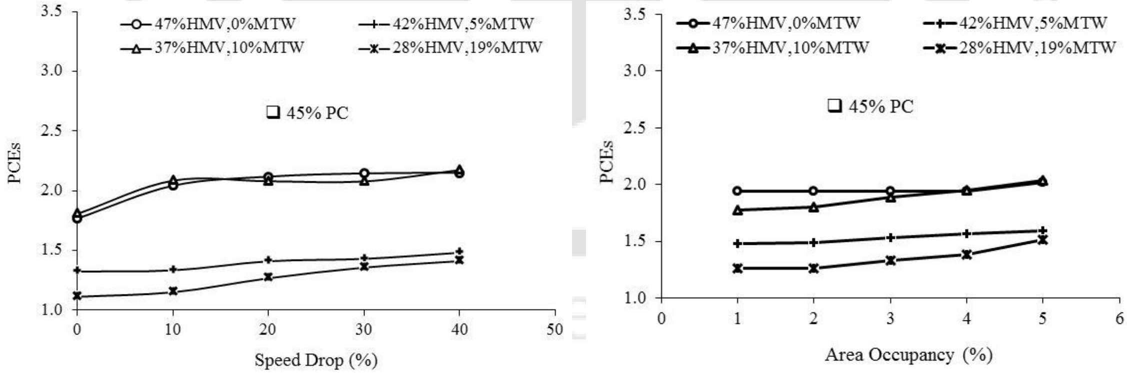
Figure 5.9 Approximated PCE values for the actual mix of (a) Four-Lane and (b) Six-Lane divided roads

Using the above-mentioned procedure, the approximated PCE values are estimated for the other traffic compositions. Further, the corresponding error values are calculated and are shown in Table 5.3. It can be seen from the table that the error values calculated using the approximated PCEs for the different traffic mixes are mostly on the lower side. Therefore, even for the heterogeneous traffic mix, using a single set of PCE values for the varying LOS does not result in significantly higher error values. On the other hand, these PCE values simplify the use of PCE values in the capacity analysis of highways for the field engineers.

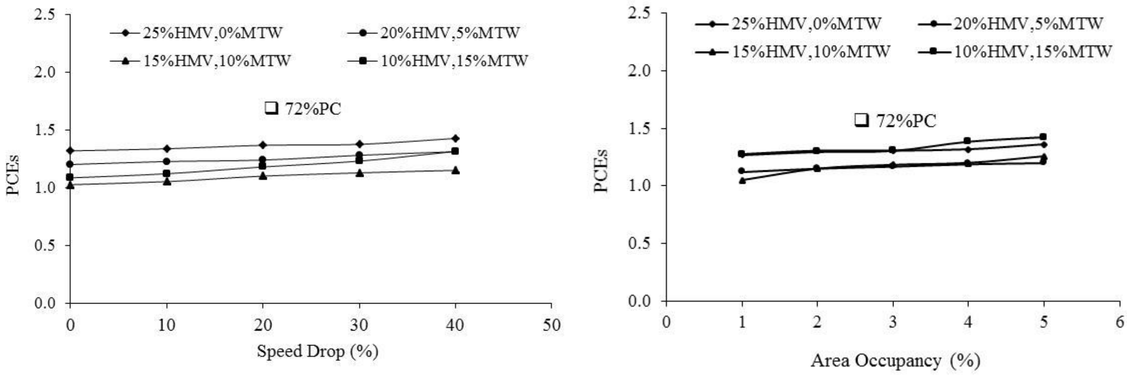
5.5 Aggregate PCE Values Using Speed Drop and Area Occupancy

The PCE values of HMV, MThW, and MTW were estimated for the different traffic mixes. While doing this the proportion of HMV and MTW were varied, but the proportion of MThW and passenger car were kept constant. Representative Speed-flow and flow-area occupancy relationships used for estimating the PCE values at different levels of speed drop and area occupancy are already shown in Figures 5.2 to 5.7. For the heterogeneous traffic mix, getting the individual PCE value for a specified proportion of the vehicle type is tedious. Because the PCE value of a vehicle type changes depending on the proportion of the other types of vehicles present in the traffic stream.

This is true even in the case of approximated PCEs. This can be seen in Figures 5.10 and 5.11, which shows, the individual PCE and approximated PCE values of MThW on four-lane and six-lane divided roads for the proportions of 8% and 3%, respectively. Moreover, both individual PCEs and approximated PCEs impart error while calculating the heavy vehicle adjustment factor (f_{HV}). Roess and Messer (1984) stated that the PCE value of a traffic mix (aggregate PCE) will provide more accurate estimation of the heavy vehicle adjustment factor (f_{HV}).



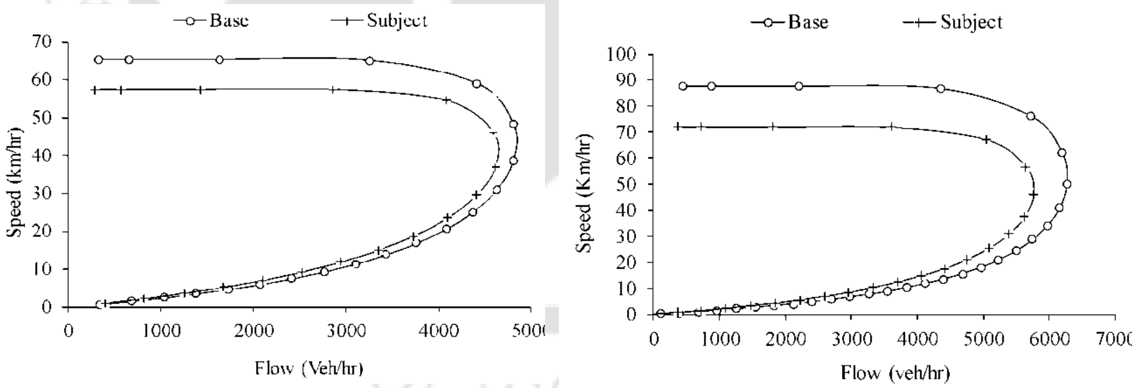
(a) PCE on Four-lane divided road using speed drop and area occupancy



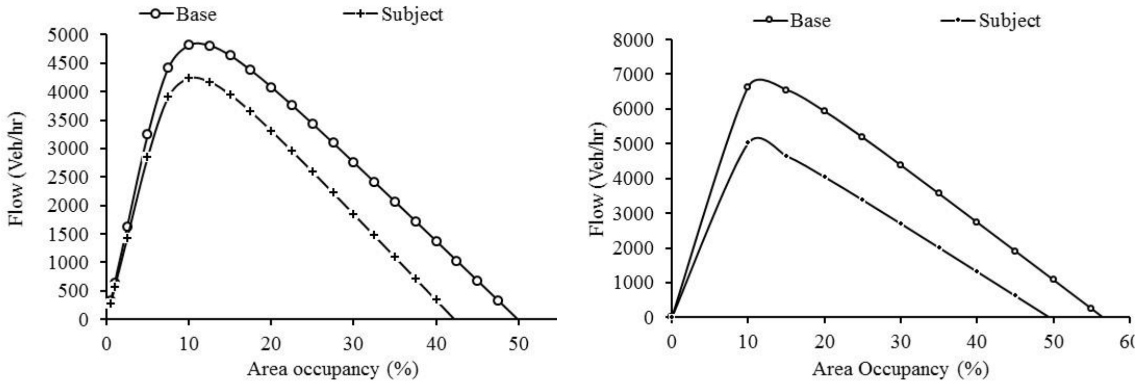
(b) PCE on Six-lane divided road using speed drop and area occupancy

Figure 5.10 Individual PCE values of MThW for different traffic mixes on (a) Four-lane and (b) Six-lane divided roads having a fixed proportion of MThW

Therefore, the present study estimated the aggregate PCE value for a particular traffic mix on four-lane and six-lane divided roads. For estimating aggregate PCEs, speed-flow and flow-area occupancy relationships of base and subject streams are employed and are shown in Figures 5.11 (a) and (b).



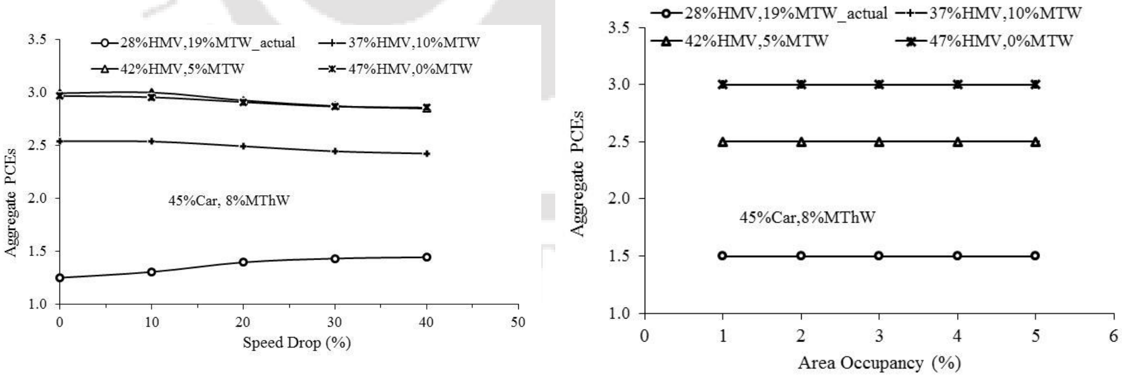
(a) Speed-flow diagrams on Four-lane and Six-lane rural roads



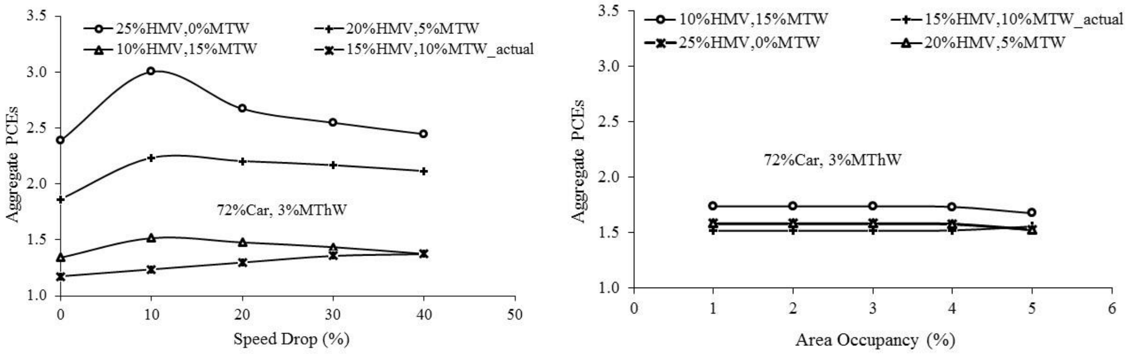
(b) Area occupancy-flow diagrams on Four-lane and Six-lane rural roads

Figure 5.11 Area Occupancy-Flow and Speed-Flow diagrams for Four-lane and Six-lane divided roads used for estimating the Aggregate PCE values

At the same area occupancy, aggregate PCE values corresponding to the traffic mix shown in Table 5.1 came out to be 1.35 for the four-lane divided road and vary from 1.52 to 1.56 for the six-lane divided road. Similarly, equal speed drop results in the aggregate PCE values ranging between 1.25 to 1.45 and 1.17 to 1.37 in the case of four-lane and six-lane divided roads, respectively. The variation of the aggregate PCE values with speed drop and area occupancy corresponding to some of the other traffic compositions is shown in Figure 5.12.



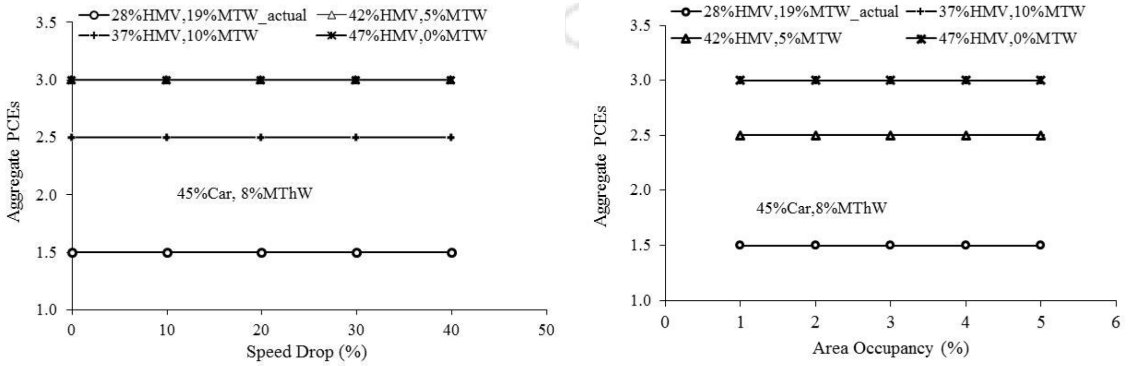
(a) Four-Lane divided road



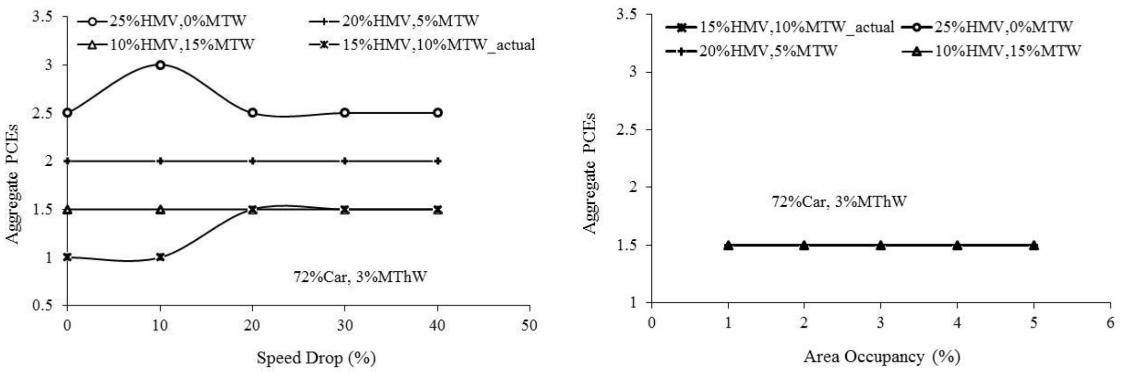
(b) Six-Lane divided road

Figure 5.12 Aggregate PCE values for different traffic mixes on (a) Four-Lane and (b) Six-Lane divided roads

From the Figure 5.12, it can be stated that the aggregate PCE values do not vary with area occupancy in the case of the four-lane divided road corresponding to a particular traffic mix, whereas speed drop exhibits slight variation. The range of area occupancy (upper limit is 0.05%) employed in this study falls within the speed drop range of ten percent. Therefore, the variation among the aggregate PCEs with the area occupancy is negligible. Moreover, for the six-lane divided road, the aggregate PCEs exhibit very little change in their magnitudes even for the varying compositions. However, the aggregate PCE values estimated using the speed-drop exhibits variation. The speed difference on a six-lane divided highway will be more due to the greater road width and also more so under free-flow conditions as the different vehicle classes will move at varying free-flow speeds. Therefore, the speed drop provides large variability in the lower range of speed drop in the case of the six-lane divided road.



(a) Four-Lane divided road



(b) Six-Lane divided road

Figure 5.13 Aggregate PCE values after rounding off for the different traffic mixes moving on (a) Four-Lane and (b) Six-Lane divided roads

Figure 5.14 shows the impact of road width on the aggregate PCE values when the traffic stream consists of 72% passenger cars, 15% HMV, 3% MThW and 10% MTW on both four-lane and six-lane divided roads. It can be observed from Figure 5.12 that four-lane divided road has higher aggregate PCE values compared to the six-lane divided road. The aggregate PCE value decreases with the increasing lane width. On a relatively narrow road, the impact caused by different types of vehicles on the passenger car will be more as compared to the roads which are wider. For a given traffic mix on wider roads, passenger cars have lesser interactions with the other vehicles. Therefore, aggregate PCE value decreases with the increasing lane width on rural highways.

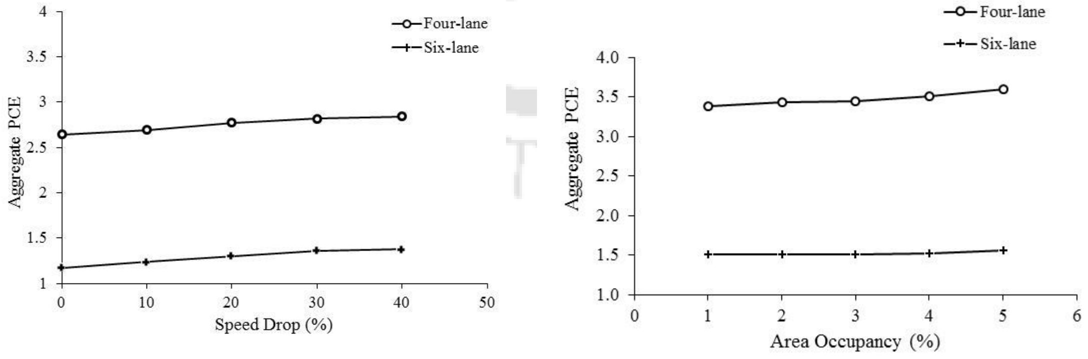


Figure 5.14 Effect of road width on aggregate PCE values using speed drop and area occupancy

5.6 Summary

The widespread use of PCE values contributed to a huge body of research available on PCE. For the design and operational analysis, the PCE values are required to convert the demand or service volume expressed in vehicles per hour to the passenger cars per hour. In the present study, a CA-based simulation model was considered for simulating traffic stream behaviour on the rural highways. For estimating the PCEs of four commonly observed vehicle types, macroscopic relationships corresponding to the base, mixed and subject streams were generated with the help of the CA model. Two performance measures, the speed drop, and area occupancy, were chosen for estimating the PCEs. At equal speed drop and area occupancy, the PCE values were calculated for the different traffic compositions and road geometry using the Sumner et al.'s method. The estimated PCE values were validated using the percentage error in f_{HV} estimation. Thereafter, the constant PCEs were estimated for the different traffic mixes and checked for their adequacy across the different values of speed drop and area occupancy. Finally, the concept of aggregate PCE was employed for estimating the combined impact of different types of vehicles present in the heterogeneous traffic stream on four-lane and six-lane divided roads. The effect of different traffic mixes, traffic flow rate, and road width on aggregate PCE has been studied for the selected four-lane and six-lane divided roads in the present study. Based on the analysis, the aggregate PCE was found to be a viable option as compared to the individual PCE and approximated PCE for the heterogeneous traffic mixes moving on rural highways passing through the level terrain.

Chapter 6

Conclusions

6.1 Summary

The objective of the present study was to estimate the PCE values of different vehicle types for representing the heterogeneous traffic stream moving on the multilane highways in India. From the literature review, it was understood that this can be achieved by either calculating individual PCEs or aggregate PCE. Previous studies carried out in India employed individual PCEs in dynamic and static forms for representing the heterogeneous traffic stream. The present study analysed both individual (static and dynamic) and aggregate PCE for representing the heterogeneous traffic stream on four-lane and six-lane divided highways in India. Sumner et al.'s approach was used for estimating the individual PCE values of different vehicle type. This method uses the macroscopic relationships between flow and a suitable performance measure. Given the problems in field data collection on rural highways, use of the simulation model was thought to be an effective tool for obtaining the macroscopic relationships. From simulation model, macroscopic data corresponding to speed, flow, density, and area occupancy were acquired to generate the macroscopic relationships. Performance measures such as the speed drop and the area occupancy were chosen as the equivalency criteria. These measures were selected based on the trend of PCEs with flow rate and the error in flow rate conversion. The dynamic variability of individual and aggregate PCEs with traffic composition and flow rate were also analysed. Constant PCEs were estimated from individual PCEs by considering a certain amount of approximation and their adequacy was investigated across different levels of speed drop and area occupancy. Aggregate PCE values were calculated for four-lane and six-lane divided roads using the speed drop and area occupancy.

Following sections discuss the major conclusions drawn from the present study, contributions of the study, and recommendations for future work.

6.2 Conclusions

The important conclusions of the present study are as follows:

1. Macroscopic relations estimated based on the limited empirical data were found to be prone to errors. Results show that it is necessary to collect the field data on all the possible traffic states that may exist on a given road stretch. This study has also found that the macroscopic models obtained from the stationary traffic conditions were relatively more accurate.
2. Performance measure affects the estimated PCE values. Analysis of various performance measures has shown that the area occupancy and the speed drop are suitable to describe the LOS experienced by the drivers travelling in the heterogeneous traffic stream.
3. For a given traffic composition, Individual PCEs for various vehicle types were found to vary with the speed drop and area occupancy. Given the complexities in using the dynamic PCEs, the highest PCE value (obtained from different levels of speed drop/area occupancy) were used across different levels of service. For constant and the individual PCEs, the error values range from 0-8% and 1-5%, respectively for different traffic compositions. Therefore, the constant PCEs can be used instead of the individual PCEs without much loss of accuracy for the heterogeneous traffic conditions.
4. For a four-lane divided road and for a particular traffic mix, the aggregate PCE value was found to remain constant with the area occupancy. Whereas the speed drop provides a slight variation. This may be due to the fact that the range of area occupancy (upper limit is 5%) falls within the speed drop range of ten percent.

   For a six-lane divided road, aggregate PCEs show a large variation with the speed drop, particularly at lower flow rates. This can be attributed to the speed difference between the passenger car and the slow moving vehicles on a six-lane divided road due to the greater road width which will be more at the lower flow rates.

5. For the different traffic compositions on four-lane divided road, aggregate PCEs range from 1.5-3.0 with the change in HMV composition from 25%-50% for both speed drop and area occupancy.

   For the six-lane divided road, with the area occupancy the aggregate PCEs remain 1.5 and do not depend on the HMV composition. But with the speed drop the aggregate PCEs vary from 1.0-3.0 with the changing HMV composition from 10%-25%.
6. As the road width increases the aggregate PCE value decreases due to the lesser interaction between the passenger cars and the other types of vehicles present in the traffic stream.

6.3 Contributions to the Field of Research

The present study is one of the few studies carried out on the rural highways in India using simulation-based approach. The major contributions of the study are:

1. The present research revealed the difficulties in getting the macroscopic relationships from empirical observations. It also manifested the biasedness in the macroscopic model parameters resulting due to the lack of data corresponding to particular traffic states. The analysis will surely help researchers in understanding the problems associated with the development of macroscopic relationships from the field data.

2. The study suggested speed drop and area occupancy as the performance measures for estimating the individual PCE values of multiple types of vehicles, moving on the rural highways. Also, the shortcomings of performance measures such as stream speed, the speed of passenger car, and density have been pointed out in this study. Most of the PCE studies selected the performance measure based on the qualitative assessment rather than performing quantitative analysis of the different performance measures. The present study bridges that gap by performing a comparative analysis of different performance measures.

3. Most of the studies carried out in India suggested the use of varying PCEs with LOS. The present study shows that the constant PCE values obtained after rounding off to the nearest 0.5 could be used across the entire LOS without much loss of accuracy. This is a

really useful finding given that the application of varying PCEs in the field for LOS analysis is cumbersome.

4. The present study also proposed the use of aggregate PCE for the LOS analysis of rural highways given that the individual PCEs do not provide any specific advantage over aggregate PCE for the heterogeneous traffic mix. Instead, it is plausible that the summation of individual PCEs may not equate to the combined impact of all those vehicle types.

6.4 Recommendations for Further Research

Developing macroscopic relationships based on empirical data needs to be further analyzed for the heterogeneous traffic stream. Speed drop & area occupancy can be further explored as LOS measures on rural highways. Speed drop analysis can be done based on the field data. The effect of the other important factors on the aggregate PCEs needs to be studied.

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Appendix A

In general terms, most adjustment factors are multiplicative. They are used to take a demand flow, service flow rate, or capacity value stated in terms of base conditions and converts it to an equivalent value that recognizes existing or projected prevailing conditions.

$$q_p = q_b \prod f_i$$

Where, q_p = flow rate (demand, service, or capacity) under prevailing conditions (veh/h or veh/h/ln),

q_b = flow rate (demand, service, or capacity) under base conditions (pc/h or pc/h/ln), and

f_i = adjustment factor for prevailing conditions i.

The adjustment factor for any given prevailing conditions is defined as

$$f_i = \frac{q_p}{q_b}$$

For adjusting flow rate due to the presence of heavy vehicles, the above equation becomes,

Heavy vehicle adjustment factor,

$$f_{HV} = \frac{q_p}{q_b}$$

Suppose p1, p2, p3 are the proportion of vehicle types present in the traffic stream, then

$$q_b = q_p \times p_1 \times E_1 + q_p \times p_2 \times E_2 + q_p \times p_3 \times E_3$$

Where, E1, E2, E3 are the PCE values of three vehicle types respectively.

For a passenger car, E1 =1, and p1 = 1- p2- p3, then

$$q_b = q_p \times (1 - p_2 - p_3)\ \times 1 + q_p \times p_2 \times E_2 + q_p \times p_3 \times E_3$$

Or

$$^{q_b}/_{q_p} = (1 - p_2 - p_3)\ + p_2 \times E_2 + p_3 \times E_3$$

Or

$$^{1}/_{f_{HV}} = 1 + p_2\,(E_2 - 1) + p_3\,(E_3 - 1)$$

Or

$$f_{HV} = \frac{1}{1 + p_2\,(E_2 - 1) + p_3\,(E_3 - 1)}$$

Table A1: PCE Values of four-lane divided road for different traffic compositions

Traffic composition	Speed Drop (%)	Area Occupancy (%)	HMV		MThW		MTW	
			SD	AO	SD	AO	SD	AO
45%Car,47%HMV,8%MThW	0	1	3.24	3.15	1.77	1.94	NA	NA
	10	2	3.24	3.15	2.04	1.94	NA	NA
	20	3	3.24	3.15	2.12	1.94	NA	NA
	30	4	3.25	3.17	2.15	1.94	NA	NA
	40	5	3.25	3.24	2.15	2.01	NA	NA
45%Car,42%HMV,8%MThW, 5%MTW	0	1	3.58	3.26	1.32	1.48	0.57	0.75
	10	2	3.60	3.29	1.33	1.49	0.54	0.74
	20	3	3.61	3.31	1.41	1.53	0.51	0.72
	30	4	3.61	3.31	1.43	1.57	0.45	0.70
	40	5	3.61	3.35	1.48	1.59	0.36	0.64
45%Car,37%HMV,8%MThW, 10%MTW	0	1	3.17	2.93	1.81	1.78	0.58	0.73
	10	2	3.18	2.93	2.08	1.81	0.52	0.70
	20	3	3.19	2.94	2.08	1.89	0.47	0.65
	30	4	3.22	2.95	2.08	1.95	0.46	0.57
	40	5	3.23	3.01	2.17	2.04	0.44	0.54

Table A2: PCE Values of six-lane divided road for different traffic compositions

Traffic composition	Speed Drop (%)	Area Occupancy (%)	HMV		MThW		MTW	
			SD	AO	SD	AO	SD	AO
72%Car,25%HMV,3%MThW	0	1	2.52	1.63	1.32	1.28	NA	NA
	10	2	2.57	1.63	1.34	1.31	NA	NA
	20	3	2.61	1.64	1.37	1.31	NA	NA
	30	4	2.61	1.64	1.38	1.32	NA	NA
	40	5	2.62	1.65	1.43	1.36	NA	NA
72%Car,20%HMV,3%MThW, 5%MTW	0	1	2.22	1.79	1.20	1.12	0.64	0.87
	10	2	2.24	1.83	1.23	1.15	0.62	0.84
	20	3	2.24	1.86	1.24	1.17	0.62	0.83
	30	4	2.29	1.87	1.28	1.19	0.58	0.81
	40	5	2.32	1.91	1.31	1.20	0.53	0.77
72%Car,10%HMV,3%MThW, 15%MTW	0	1	2.37	2.65	1.09	1.27	0.57	0.77
	10	2	2.45	2.65	1.12	1.30	0.54	0.71
	20	3	2.46	2.69	1.18	1.31	0.51	0.64
	30	4	2.49	2.71	1.23	1.38	0.50	0.60
	40	5	2.51	2.79	1.32	1.42	0.45	0.56

Publications

List of Publications in Conferences

Syed Omar and Mallikarjuna, C (2015), "Analysis of Macroscopic Relations for Heterogeneous Traffic Stream," Presented at the National Conference and Workshop On Recent Advances in Traffic Engineering (RATE 15) July 03-04, 2015 Civil Engineering Department, SVNIT, Surat, Gujarat.

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Syed Omar B., Pranab K., and Mallikarjuna, C (2018), "Estimation of the Passenger Car Equivalents: A Review of the Existing Approaches." (Manuscript under preparation for Transport Reviews).

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