GAP ACCEPTANCE BEHAVIOUR AND CAPACITY ANALYSIS OF URBAN UNCONTROLLED INTERSECTIONS FOR MIXED TRAFFIC CONDITIONS

Submitted in partial fulfilment of the requirements

for the award of the degree of

Doctor of Philosophy

by

D. ABHIGNA

701401

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY, WARANGAL

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WARANGAL

CERTIFICATE

This is to certify that the thesis entitled **"GAP ACCEPTANCE BEHAVIOUR AND CAPACITY ANALYSIS OF URBAN UNCONTROLLED INTERSECTIONS FOR MIXED TRAFFIC CONDITIONS"** being submitted by **Mrs. D. ABHIGNA** for the award of the degree of **DOCTOR OF PHILOSOPHY** to the Department of **Civil Engineering** of **NATIONAL INSTITUTE OF TECHNOLOGY, WARANGAL** is a record of bonafide research work carried out by her under my supervision and it has not been submitted elsewhere for award of any degree.

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APPROVAL SHEET

This Thesis entitled **"GAP ACCEPTANCE BEHAVIOUR AND CAPACITY ANALYSIS OF URBAN UNCONTROLLED INTERSECTIONS FOR MIXED TRAFFIC CONDITIONS"** by **Mrs. D. ABHIGNA** is approved for the degree of Doctor of Philosophy.

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DECLARATION

This is to certify that the work presented in the thesis entitled **"GAP ACCEPTANCE BEHAVIOUR AND CAPACITY ANALYSIS OF URBAN UNCONTROLLED INTERSECTIONS FOR MIXED TRAFFIC CONDITIONS"**, is a bonafide work done by me under the supervision of **Dr. K. V. R. Ravishankar**, Assistant Professor, Department of Civil Engineering, NIT, Warangal, Telangana, India and was not submitted elsewhere for the award of any degree. I declare that this written submission represents my ideas in my own words. I have adequately cited and referenced the original sources where others ideas or words have been included. I also declare that I have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea /data/ fact/source in my submission. I understand that any violation of the above will be cause for disciplinary action by the institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

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D. Abhigna

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Date:

Dedicated to

My Parents, Husband and Daughter

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D. ABHIGNA

ABSTRACT

Intersections are the critical zones where vehicles perform different maneuvers in an attempt to share the same space at the same time. As the vehicle reaches the intersection, the driver of the subject vehicle has to take a quick decision by taking into account the intersection geometry, speed and type of the vehicles approaching the intersection from the other legs. At uncontrolled intersections there are no external signs or signals to control the movement of vehicles and the traffic operates based on the priority of traffic movements. In mixed traffic conditions priority rules are often violated by the road users. Vehicular interactions at uncontrolled intersections under mixed traffic conditions are very complex. At higher traffic volumes, the minor road vehicles tend to wait for longer time to cross the intersection that increases the probability of the vehicles to accept the shorter gaps. Hence, the general behavior of different vehicle types and the gap required for each vehicle type approaching from the minor and the major road taking right turn need to be carefully analyzed. Gap acceptance method is used for mixed traffic condition because it is based on the critical gap and follow-up time, which in turn depends on the type of vehicles and traffic conditions. Also, gap acceptance procedure is more suitable for mixed traffic conditions because it can be used for different composition of vehicles.

For this study, data was collected from three three-legged and three four-legged intersections located in various cities in India. The following parameters were extracted from the videographic data at each intersection and for each vehicle type: total volume, gap accepted, gap rejected, follow-up time, stopped delay, and total delay. This study tries to analyze the effect of vehicle type on gap acceptance behaviour of each of the right turning vehicles from the minor and the major road. Also, this study analyzes the major stream vehicle combinations on the gap-acceptance behavior of the minor stream and the major stream vehicles. It is observed that the size of the vehicles and traffic volumes has a significant influence on the critical gap. Depending on the major road vehicle combinations, the critical gap for each right turning subject vehicle varied from 1.4 s to 8.7 s.

Mixed traffic is composed of different vehicle types with varying geometric and acceleration characteristics. The performance of an intersection very much depends on the vehicular composition. This may be due to varying lengths and widths of different vehicle types which influences the intersection capacity. Different vehicle compositions are simulated using VISSIM for different vehicle types with an increment of 10% starting from 0% to 100%. The proportion of each vehicle type for the selected composition is considered in such a way that these proportions matches with the observed field proportion. Thereby, the effect of traffic composition on delay and volume at urban uncontrolled intersections is studied for all the vehicle types. Also, the total delay and the service delay is calculated for all the vehicles at all the intersections considered in this study. It is observed that there is an increase in delay as the size of the vehicle increases. Finally, the field delay data is compared with the simulated delay data and the error observed is less than 10% for all the vehicle types.

Capacity estimation is necessary for designing the intersection facilities and for upgradation of the control facilities to avoid unnecessary delay. Capacity at uncontrolled intersection is measured either by gap acceptance method, empirical regression approach, or conflict technique. Performance of uncontrolled intersection is influenced by the delay caused by low-priority movements on minor roads. In this study, capacity at uncontrolled intersections is estimated by considering the gap acceptance models including Tanner’s model, Drew's model, modified Sieogloch's model, and Luttenin's model. Further, the capacity is also estimated using the HCM (2010) and Indo-HCM methods. Based on the MAPE, Tanner's model is observed to be the best among the selected models for determining the capacity at urban uncontrolled intersections. Also, the performance of each of the six intersections is evaluated using the LOS criteria. HCM (2010) failed to differentiate between the performance of the six intersections. However, the LOS evaluated using the volume-capacity ratio resulted in significant variation in performance of the six intersections.

Keywords – Capacity, critical gap, follow-up time, gap acceptance, service delay, total delay, uncontrolled intersection.

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CHAPTER 1

INTRODUCTION

1.1General

Intersections are the most critical areas on the road network. These are the locations where two or more roads cross each other and the vehicles coming from different directions share the same space at the same time. Activities such as left turning, right turning, and crossing over will occur at same places. The main function of an intersection is to guide vehicles to their respective destinations. According to the road accident report in India (Government of India, 2015), it is observed that 57% accidents occurred at intersections alone in the year 2014, and 49% in the year 2015. This data shows that on an average half of the road accidents are occurring at intersections alone. In order to minimize the causalities at the intersections, drivers should take a quick decision on arriving at the intersection with due consideration to the route, speed, intersection geometry and movement of other vehicles present at the intersection. Even a small error in judging the above intersection scenario can lead to accidents. Similarly, the driver’s decision process can also cause significant delays at the intersection. These delays will further depend on the type of intersection control and also on the intersection geometry. In particular, the performance of an intersection very much depends on the precise estimation of the intersection capacity.

Intersections are the critical zones where conflicting, diverging and merging movements influences the intersection capacity. Uncontrolled intersections in particular pose dangerous situations to the vehicular traffic where interaction between the vehicles is very complex. As per the Washington State Department of Transportation (WSDOT) design manual, most of the uncontrolled intersections are typically located on local streets where the volumes of the intersecting roadways are less and roughly equal. During the peak hour traffic, the unpredicted crossing behaviour of the minor stream and the major stream right turning vehicles causes significant delays and reduces the intersection capacity.

Traffic in India is mixed in nature consisting of the slow and fast moving vehicles, where the size, speed and the operational characteristics of the vehicles are significantly different. Apart from this, the vehicular characteristics at the uncontrolled intersections very much depend on the driver's behavior. Due to these variations, vehicles typically do not follow the lane discipline and occupy any available lateral position on the road. This results in significant delays at urban uncontrolled intersections and subsequent formation of longer queues side-by-side within the available road space. Priority rule is often violated at uncontrolled intersections which lead to increase in the delay for the minor road right turning vehicles. Because of the higher delays to the minor stream traffic, they tend make forced entry into the intersection which in turn can cause delays to the major stream vehicles. Thus, the traffic delays at the uncontrolled intersection affects the overall performance of the road network.

In general, the analysis of traffic flow on the road links is less complex in nature when compared to the intersections as there are no conflicting movements on the road links. Whereas, there are many traffic conflicts at an intersection. The conflicting flow at an intersection is handled by some of the control measures including the control signs and traffic signals. Traffic signals are installed mostly on the major arterial intersections whereas significant proportion of the major road-minor road intersections are still uncontrolled in most of the cities in India. The uncontrolled intersection gives priority to major road movement, while the minor road drivers have to find suitable gaps between the vehicles plying on the major road in order to complete their maneuvers.

Figure 1.1 Traffic scenario at urban uncontrolled intersection

Typically the drivers do not care about the conflicting traffic and try to make forced enter into the intersection while accepting higher risk. The vehicular speeds on the major road varies widely during lean hours. The vehicles plying on the major road crosses the uncontrolled intersection at relatively higher speeds and the gap accepted to cross the stream tends to be higher. Similarly, at higher traffic volumes, the vehicles on minor road tend to wait for longer time to cross the intersection. Longer waiting time will increase the probability of the vehicles to accept the shorter gaps. The traffic flow scenario at a typical uncontrolled urban intersection is as shown in Figure 1.1.

1.2 Gap Acceptance

Gap estimation is an integral part in capacity estimation of uncontrolled intersections. Gap acceptance procedure is used for mixed traffic conditions because it is based on critical gap and follow-up time, which depends on driver's characteristics, vehicle characteristics, site characteristics and other factors which include time of the day. Gap acceptance is widely used to analyze the behavior of drivers at uncontrolled intersections. Gap acceptance is the method in which a vehicle from the minor street accepts the gaps available in the major street to complete the desired manoeuver. Driver approaching the intersection from the minor street tries to find a safe gap to cross the intersection. It is the decision made by the driver either to accept or to reject the gaps available in the major street under the given conditions. The behavior of different vehicle types and the gap available for the subject vehicle approaching from both the minor road and the major road taking right turn to merge with major traffic stream need to be analyzed using the gap acceptance method.

Estimation of critical gap is difficult under mixed traffic conditions compared to homogeneous traffic conditions. The smaller vehicles approaching from a minor road while attempting to cross a major road try to accept narrow gaps available between the large-sized vehicles. In mixed traffic conditions especially at the uncontrolled urban intersections, many times a number of small-sized vehicles accept the available narrow gap and move parallel to each other in the process of crossing the major road, after which these vehicles move one after the other in a single line. During the process of crossing the major road, minor road vehicles do not follow the "priority rule" and the major stream vehicles are forced to reduce the speeds to give way to minor stream vehicles. Also, when the major road traffic flow is less and priorities are implemented, normal gap acceptance behavior is observed. When there is an increase in the waiting time for all the right turning vehicles, these vehicles tend to accept smaller gaps thus, forcing the through major road traffic flow to slow down or stop, resulting in the forceful gap acceptance.

1.3 Traffic Simulation

Traffic simulation plays an important role in estimation of the capacity of the intersections especially by creating several possible scenarios which could not be observed under normal traffic conditions. Traffic simulation models are one of the latest generations of commercially available traffic models developed in recent years. It models the movements of individual vehicles travelling on road networks by using car following, lane changing and gap acceptance rules. They are becoming increasingly more popular for the development and evaluation of a broad range of road traffic management and control systems. For the simulation of traffic many conventional software are available including Aimsun, MATsim, Paramics, SimTraffic, and VISSIM.

PTV VISSIM is one such microscopic multi-modal traffic flow simulation software package developed by Planung Transport Verkehr (PTV) AG in Karlsruhe, Germany. VISSIM considers several parameters and some of them are driver behaviour, lateral behaviour, overtaking behaviour, lane changing behaviour and right hand, left hand traffic rule. It is important to note here that left hand traffic rule is being followed in India with mixed traffic conditions where lane behaviour is generally not followed. Thus, it is not practically possible to directly use VISSIM for Indian traffic conditions. VISSIM can be adopted for Indian conditions by making several changes in settings, parameters and traffic conditions. In VISSIM, a network need to be created and the corresponding parameters need to be calibrated to replicate the field conditions. Several alternate possible scenarios of vehicular compositions for different vehicle types can then be simulated in VISSIM for further analysis. Thus, the effect of vehicle composition on delay and volume at uncontrolled intersections can be thoroughly analyzed through the VISSIM simulation.

1.4 Capacity of Uncontrolled Intersections

The capacity of uncontrolled intersection is required for upgradation of existing “uncontrolled intersection” to “controlled intersection” to minimize the accidents and unwarranted delays.

The estimation of capacity of uncontrolled intersections in mixed traffic is more complex because the performance of the vehicles varies widely. Even though some of the works in the past considered the influence of geometric (Shahi and Amini 1998) and control features (Shahi and Amini 1998; Faghri 1995) on the quality of traffic service including the driver behavior (Vaziri 1998), these works are valid only for controlled intersections or for high speed corridors.

Capacity of uncontrolled intersections is normally estimated either by empirical or gap-acceptance models. Considering the advantages of the gap-acceptance models, several researchers have focused on identifying the parameters affecting the capacity of uncontrolled intersections (Troutbeck and Kako 1999; Brilon and Wu 2001; Pollatschek et al. 2002; Li et al. 2003; Li et al. 2009). Several gap acceptance methods are available in which the following parameters are considered including critical gap, follow up time, conflicting flow, major stream flow, major stream headway, and minimum headway. Driver behavior plays a crucial role on the gap acceptance or rejection. Thus, the behaviour of the driver maneuvering the intersection has influence on the delays, headways and capacity of the intersection. In given traffic conditions, the driver has to accept a gap in the conflicting stream at some point of time at an uncontrolled intersection and this decision is influenced by certain behavioral conditions of the driver. Generally, the type of vehicle on the major stream influences this behaviour apart from the type of right turning vehicle in the traffic stream. Thus, it is necessary to relate the vehicle type to analyze the gap-acceptance behaviour at uncontrolled intersections. These findings are expected to improve the accuracy with which the capacity and performance of an urban uncontrolled intersection can be estimated.

1.5 Need for the Study

In general, the traffic in developing countries such as India is of mixed in nature where the type of vehicles varies widely starting from slow moving non-motorized vehicles such as bicycles and tricycles to fast moving motorized vehicles including two-wheelers, three-wheelers, small cars, big cars, light commercial vehicles, medium commercial vehicles such as tractor-trailers, buses, and heavy commercial vehicles such as fully-loaded trucks. As the vehicular mix consists of both non-motorized and motorized vehicles, their dimensional and performance characteristics varies widely. Further, most of the intersections that are observed in urban areas are still uncontrolled in most of the Indian cities. Because of the mixed nature of traffic at these uncontrolled intersections, the small-sized vehicles has the tendency to penetrate into the available gaps in order to cross an intersection. During this process, because of higher vehicular conflicts, the delays increase significantly especially at uncontrolled intersections leading to formation of queues side-by-side within the available road space. It is possible that one single gap within the major stream traffic flow can be accepted by several number of small-sized vehicles that move parallel to each other. After clearing the conflicting traffic at the intersection, these small-sized vehicles typically travel in one single lane, that is, they move one after the other. It is also worth to note here that, during this intersection crossing maneuvers, the priority rule is often violated by the minor stream vehicles that might make a forced entry into the available gaps after waiting for sufficiently longer duration to clear the intersection. In other words, the minor stream vehicles can accept even shorter gaps if the waiting time at the intersection increases significantly.

In the last few decades, several researchers focused on determining the gap acceptance with main emphasis on homogeneous traffic conditions. However, performing gap acceptance studies for heterogeneous traffic conditions prevailing in India is very difficult especially in understanding the gap acceptance behavior of right turning vehicles. Gap acceptance is affected by various factors which primarily include vehicle type and vehicle arrival rate. Availability of the gaps within the major traffic stream and the subsequent acceptance or rejection of these gaps by the through and right-turning vehicles plays a significant role in the estimation of capacity of uncontrolled urban intersections especially using the gap acceptance methods. Thus, it is very much essential to determine the influence of each right-tuning vehicle type on the gap acceptance behaviour which further depends on the major stream vehicle type and combinations. Once the capacity of the intersection is estimated precisely by taking into account the actual field conditions, the performance of the intersection can further be evaluated for subsequent upgradation, if required.

1.6 Objectives of the Study

The objectives of the research work are as follows:

i. To determine the critical gap for minor and major stream right turning movements at uncontrolled urban intersections.

ii. To evaluate the effect of right turning vehicle types and major stream vehicle combinations on the gap acceptance behaviour of right turning vehicles at uncontrolled urban intersections.
iii. To determine the effect of traffic composition on delay and volume at uncontrolled urban intersections.
iv. To assess the capacities for right turning movements obtained through various models at uncontrolled urban intersections under mixed traffic conditions.
v. To evaluate the performance of uncontrolled urban intersections under mixed traffic conditions.

1.7 Scope of the Current Study

The present work is confined only to uncontrolled urban intersections with four-lane divided major streets, two-lane undivided minor streets, and four-lane divided minor streets. The study is restricted to plain terrain, with mixed traffic flow. Further, the study considers both three-legged and four-legged urban intersections.

1.8 Organization of the Thesis

The research work is presented in eight chapters and the contents of the report are organized as follows:

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

Chapter 2, provides an overview of the earlier studies related to the subject matter of this research work. First, research works related to the study on the gap acceptance behavior of the vehicles are reviewed, followed by the review of various critical gap estimation methods and their significance. Then, the research works related to VISSIM simulation is reviewed. This is followed by the review of various delay studies and capacity estimation methods for the urban uncontrolled intersections. Finally, the entire literature is summarized highlighting the gaps observed in the literature.

Chapter 3, deals with the present study methodology and the pertinent discussion with the help of a flow chart.

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

Chapter 5, provides the analysis of field data including critical gap estimation for all the vehicles types and for each leg of the intersection, effect of major stream vehicle combinations on the gap acceptance behaviour, calculation of follow-up time, the statistical analysis for the critical gap, and fitting of distributions for both the accepted and available gaps.

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

Chapter 7, deals with estimation of the capacity using various existing models and the performance of uncontrolled urban intersections for mixed traffic conditions.

Chapter 8, provides summary of the work, conclusions drawn from the study, limitations of the study, and finally the scope for further work.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Over the past few decades, researchers across the world are mostly focusing on uncontrolled urban intersections because of the higher delays experienced by the road users especially under mixed traffic conditions. Even though the primitive studies focused on determining the gap acceptance behaviour with an emphasis on homogeneous traffic conditions, the present day researchers are concentrating more on quantifying the gap acceptance behavior at uncontrolled intersections under mixed traffic conditions due to the inherent complexity involved in the analysis of the gap acceptance behavior. This chapter presents the comprehensive review of research work covering the studies related to the gap acceptance behaviour of the vehicles, critical gap estimation, VISSIM simulation, delay, and capacity estimation at urban uncontrolled intersections.

2.2 Studies on Gap Acceptance

Most of the researchers in the past conducted gap acceptance studies for homogenous traffic conditions. However, performing gap acceptance studies for mixed traffic conditions prevailing in India is very complex in nature especially in understanding the gap acceptance behavior of the right turning vehicles. Homogeneous traffic follow lane discipline and priority rule whereas the rule of priority is often violated under mixed traffic conditions. Small-sized vehicles are most likely to accept the shorter available gaps causing delay to the major stream traffic. In order to clearly define the critical gap, it is very much essential to introduce the term 'gap'. Gap is the time, in seconds, from the front bumper of the second of two successive vehicles to reach the starting point of the front bumper of the first (HCM 1985; 1994). Gap is also defined as the minimum time between successive major stream vehicles in which, minor stream vehicle can make a maneuver (HCM 2000). In early 1960’s, it was observed that 50% of the gaps of 6.5 s on the major road were accepted by the drivers and this acceptance increased to 90% when the gap was 10 s. It was observed that none of the drivers reject a gap of 12 s or more (Ashworth and Green 1966).

Raff's, probit analysis and Bissell's method were widely used to determine the driver behaviour at stop-controlled intersections. A significant difference was observed between the median lag-acceptance and gap acceptance for the various types of maneuver (Solberg and Oppenlander 1966). The effect of the major stream traffic volume on the gap acceptance behaviour was observed to be significant (Ashworth 1968) whereas, the gap accepted by the minor road driver is mainly affected by the maneuvering type, major road speed, type of vehicle and the presence of the opposite traffic (Sinha and Tomaik 1971). In early 1980's, researchers developed gap acceptance models by comparing existing methods of critical gap estimation and obtained satisfactory results using Ashworth method (Miller 1972) and maximum likelihood technique (Miller 1972; 1974). In an attempt to find the gap acceptance behaviour of the drivers, two types of models including the consistent model and the inconsistent model were used to check which one is more realistic and the results indicated that neither of the models described the true situations (Ashwoth and Bottom 1977). Maximum likelihood method was also used to focus on the factors which mainly influences the gap acceptance behavior of the drivers (Daganzo 1981). The flow on the major road was observed to influence the gap acceptance behaviour of the drivers on the minor roads, and the flow on the major road sensitiveness of the driver was recommended as an important factor (Adebisi 1982). Presence of the 'STOP' sign increased the mean accepted and rejected gaps for the right turn from the minor road to the major road (Polus 1983). A significant influence of the approaching vehicle speed was observed on the gap acceptance behavior and no significant effect was observed for the lag acceptance behavior of the drivers (Uber and Hoffmann 1988). Also, the gap acceptance behaviour was influenced by the duration of the stopped delay experienced by minor road drivers while trying to find gaps in the major traffic stream (Adebisi and Sama 1989). Further, based on the microscopic analysis of the individual behaviour of the minor road vehicles, it was reported that the mean accepted gap was less for through moving vehicles when compared to the turning vehicles (Kyte et al. 1991).

Binary logit model was developed to determine the drivers gap acceptance probabilities (Hamed et al. (1997). The probabilities of accepting or rejecting a gap for each driver can be determined using a binary probit model which is used in the calculation of critical gap for all the drivers. This method can be used to calculate critical gaps of individual drivers and also the mean critical gap at a particular intersection. It was observed that critical gaps differ between the two major lanes. Limited priority for the major stream was also used as a criteria to develop a gap acceptance model and it was reported that there would be an increase in major stream headways due to the merging vehicles especially at higher traffic flows (Troutbeck and Kako 1999). On evaluating the influence of driver behaviour on delay at unsignalized intersections, it was observed that the model with inclusion of different driver behaviour can better explain the performance of the traffic when compared to the simple gap acceptance model (Kaysi and Alam 2000). The logit model was again used to study the turn gap acceptance where it was observed that the probability of gap accepted by the driver for a given duration increased with increase in the speed of the oncoming vehicle (Davis and Swenson 2004). The aggressive driver behaviour of the minor road vehicles at partially unsignalized intersections was attributed mainly to the age of driver, average speed on the major roads and car performance (Kaysi and Abbany 2007). Different types of models were developed in the past while studying the merging behaviour of the vehicles from major to minor road at uncontrolled intersections under mixed traffic conditions where separate probabilistic models were developed for all the merging behaviour of vehicles and a combined model was developed for normal and forced merging (Venkatesan 2011). Different gap acceptance models were also developed for analyzing the behaviour of the right turning vehicles by considering different classes of vehicles, different age groups and also based on gender using SYSTAT (Karthika and Bino 2014).

Recently, researchers focused on improving the safety at the intersections by providing realistic gap values for all the right turning vehicles on the basis of the acceleration capabilities, which can be used for the design purposes (Dabbour 2015). The influencing parameters affecting the drivers gap acceptance behaviour were identified to estimate the critical gap accepted by the drivers where the increase in the delay and decease in the capacity was observed to be due to lack of sufficient gap acceptance (Mhairat et al. 2016). Further, aggressive behaviour of the minor road drivers at uncontrolled T-intersections were considered for analyzing the gap acceptance behavior and it was observed that the gap acceptance behavior is influenced by the clearing time, gap duration and aggressive nature of drivers. It was reported that such method would be very much helpful in differentiating the aggressive and non-aggressive behaviour of drivers at uncontrolled intersections (Dutta and Ahmed 2018). Realistic parameters that were not considered in the previous studies such as the heterogeneous driving behavior, driver impatience and the service time dependent on the vehicle arrival were also considered to develop a gap acceptance model where Poisson arrival process was observed on the major road apart from the formation of vehicle clusters that affect the capacity of the minor road (Abhishek et al. 2018).

2.3 Studies on Critical Gap Estimation

Several methods are available to estimate the critical gap including Siegloch's method, Greenshield's method, Raff's method, acceptance curve method, lag method, Ashworth's method, Harder's method, logit procedure, probit procedure, Hewitt's method, maximum likelihood procedure, clearing behaviour, and equilibrium of probabilities. The critical gap is considered as an important parameter in understanding the gap acceptance behavior of the drivers (Ashalatha and Chandra 2011). Critical gap is defined as the minimum time difference between the arrivals of major street vehicle during which a minor street vehicle can make its entry into the intersection (HCM 2000). Recently, the term critical gap is replaced with critical headway and is defined as the minimum headway in the major traffic stream that will allow entry of one-minor street vehicle into the intersection (HCM 2010). Several critical gap methods are available for analyzing the gap acceptance behavior of drivers for right turning traffic from the minor road.

Raff's method is the earliest method used for estimating the critical gap, which is simple and popular method and is widely being used in several countries. Raff's method is based on macroscopic model that depends on the accepted and rejected gaps. According to Raff's method, a critical gap is the time at which the sum of the cumulative number of accepted gaps and rejected gaps is equal to 1 (Raff and Hart 1950). Few of the researchers used maximum likelihood method to determine the influence of various factors on the gap acceptance behavior of the drivers (Mahmassani and Sheffi 1981; Maze 1981). The type of maneuver has a significant influence on the length of the gap being accepted by the drivers (Ashworth and Bottom 1977; Fitzpatrick 1991). Further, the critical gap does not vary with the change in the approaching vehicle speed (Kyte et al. 1994).

Comparison between various methods of critical gap estimation continued even in late 1990's (Brilon et al. 1999; Gattis and Low 1999). It is important to note here that the conclusion made by the researchers was essentially based on the relative comparison between the methods they considered. On comparing acceptance curve, Geenshield's, logit, Raff's, and Siegloch's methods, it was observed that Raff's method yielded lower critical gap values and logit method yielded higher critical gap values (Gattis and Low 1999). On comparing Ashworth, Harder, Hewitt, lag, logit, maximum likelihood, probit, Raff's, and Siegloch's methods based on the condition that critical gap estimation shall be independent of major approach traffic volume, it was observed that Hewitt and maximum likelihood methods are considered to be the best for evaluation of critical gaps (Brilon et al. 1999). Maximum likelihood method was also used in the past to estimate critical gaps for two major lanes and it was observed that critical gaps can be different in two major lanes (Hagring 2000). However, conflicting traffic speed has a significant influence on the mean accepted gap (Alexander et al. 2002). Similarly, probability equilibrium method based on the accepted and rejected gaps was used for calculation of the critical gap (Wu 2006). Sotware aaSIDRA was used for analyzing the unsignalized intersection by considering total traffic volumes, critical gap, follow-up time, and the proportion of heavy vehicles where it was observed that change of value in any of these parameters affected the average time delay and level of service. Further, the critical gap was found to be a major factor affecting the LOS (Jamil and Ibrahim 2009).

Recently, a group of researchers estimated critical gap using a new method termed "clearing behaviour of vehicles" in concurrence with gap acceptance that can be used to estimate entry capacity. Using the existing critical gap estimation methods including lag, Harders, logit, modified Raff and Hewitt methods, it was reported in the literature that the critical gap can be as low as 1.6 s (Ashalatha and Chandra 2011). In the similar manner, binary logit model was used to estimate the probability of vehicles accepting or rejecting the available gap or lag (Devarasetty et al. 2011). New critical gap estimation methods were also proposed in the recent years based on a survey method for accepted and rejected gaps. It was observed that exponential model representing the rejected proportion is better than the linear model (Guo and Lin 2011). Gap forcing behaviors were also analyzed in the past in an attempt to differentiate from conventional gap acceptance models (Xiao et al. 2011). In one of the previous study, a gap acceptance model was developed using adaptive neuro-fuzzy interface technique for two-wheelers that are turning right at uncontrolled intersection. These researchers estimated the critical gap using Raff's method and the critical gap was observed as 2.47 s (Sangole et al. 2011). However, these studies focused on modeling driver's behaviour. Recently, works are also reported on modeling gap acceptance behaviour of right turning vehicles at partially controlled three-legged intersections using adaptive neuro-fuzzy interface system and found that the predictions for major right turning ranged from 75% to 82% whereas for minor right turning, it is 87% to 89% (Sangole and Patil 2014). Similarly, a group of other researchers used Raff's, logit, lag, Ashworth's, and maximum likelihood methods to estimate the temporal and spatial critical gaps and observed a variation in these values ranging from 3 to 3.9 s and 29 to 36 m, respectively. They reported that these critical gaps are lower than those in the Highway Capacity Manual and other published literature. Based on these observations, they commented that "the drivers in India are more aggressive" (Patil and Pawar 2014).

Critical gap is a significant parameter that affects the delay and capacity of uncontrolled intersections. Different drivers display different critical gaps under different scenarios. Among the critical gap methods that exist, it was observed that the Maximum Likelihood Method (MLM) gives accurate results for critical gap whereas, Probability Equilibrium Method (PEM) (considering only maximum rejected gaps) and Raff's method provides good estimates for critical gap. However, when compared to MLM and PEM, Raff's method was observed to be the simplest method of estimation (Mohan and Chandra 2016). Critical gaps between drivers at two different locations in India and U.S. were also compared and it was observed that, the critical gap of drivers in the U.S are 2 s greater than the critical gap of drivers in India at 5% level of significance (Boyapati and Ardekani 2016). Critical gap estimated using clearing behavior of vehicles was used to analyze the effect of two and four wheelers on the gap and lag acceptance behaviour of drivers where it was observed that, the critical gap and lag varied depending on the intersection type, and the critical lag values were found to be less than the critical gap values (Dutta and Ahmed 2016).

2.4 Studies on VISSIM Simulation

Many studies were carried out for analyzing the uncontrolled intersections using VISSIM simulation. Traffic simulation is an important tool in modeling transport systems (Sun et al. 2013). In the past studies related to traffic micro-simulation, many researchers calibrated the simulation models based on trial and error method and also used default parameters with different traffic scenarios resulting in huge errors in the output (Park and Schneeberger 2003). Simulation models are generally divided into three categories such as microscopic, macroscopic, and mesoscopic. Microscopic simulation model simulates traffic by tracking individual vehicles, macroscopic simulates by taking into the section, and mesoscopic simulation models combine the properties of microscopic and macroscopic simulation models. Thus, simulation results depends on choice of the selected model (Fang et al. 2005).

Recently, many researchers focused on improving the reliability of micro-simulation models through the calibration process by proposing guidelines and requirements for calibration (Park and Schneeberger 2003; Hourdakis et al. 2003; Dowling et al. 2004; Park and Qi 2005). The calibration process of the main capacity parameters that affect the route choice were reported apart from various other parameter optimization algorithms including the genetic algorithm (Dowling et al. 2004). Further, a general methodology that includes an actuated signalized intersection was used to calibrate the VISSIM (Park and Qi 2005). Researchers developed a simulation model apart from developing a relation between flow on major and minor roads, mean delay, total delay and queue lengths by considering the critical gap as a constant (assumed critical gaps as 4 s). It was observed that the minor road vehicle enters the main stream, if the available gap > 4 s or else the vehicle will wait or slow down (Popat et al. 1989). Further, the mean acceptable gap was taken as a constant for each vehicle type and it was observed that the driver accepts the gap whenever the gap is greater than the mean acceptable gap. In the past, simulation model was used to determine the total delay on minor and major flows by varying both the minor and major road volumes (Agarwal et al. 1994). The interaction between the pedestrians and the vehicles at unsignalized intersections was also evaluated by developing a simulation model in terms of delay experienced at an urban uncontrolled intersection under mixed traffic conditions (Raghavachari et al. 1993). In another study, simulation model was developed to estimate the traffic conflicts and their characteristics by including the interactions between the vehicle types at urban uncontrolled intersections where it was concluded that for a fixed traffic volume on minor road, the conflicts will increase for the increments in the traffic volume on the major road. Similarly, for unchanged minor and major traffic volume, the conflicts increases with increase in the right turning traffic (Rao and Rengaraju 1998).

Except few cases, most of the models developed in the past were for homogeneous traffic conditions. Simulation was used in the past to evaluate the traffic operations at signal-controlled intersections, but none are available to model for All-way-stop-controlled (AWSC) intersections. A new AWSC model was developed in the late 1990's for predicting the queue length, vehicle delay, and saturation headway where a good correlation was observed with the field data (Kyte et al. 1996). In the past, researchers adopted a methodology to simulate the heterogeneous traffic flow for varied ranges of static and dynamic characteristics of the vehicles (Thamizh and Reebu (2005). In the recent years, researchers developed a service delay model based on microscopic analysis for the delay data at uncontrolled intersection under mixed traffic conditions where it was observed that the service delay did not depend on the vehicle type, while it depends on the conflicting traffic and percentage of heavy vehicles

(Chandra et al. 2009). The service delay at TWSC intersection was analyzed with the variation of conflicting traffic through VISSIM microscopic simulation by considering four vehicle types including cars, 2w, 3w and heavy vehicles. It was observed that the average service delay for each vehicle type increased exponentially with the increase in the conflicting traffic (Ashalatha and Chandra 2011). The difference between the stop sign and yield sign intersection based on delay was analyzed using VISSIM where the delays due to the two-way stop sign, yield sign, and all-way stop sign intersection were evaluated by considering the lowest delay and the critical traffic volume where stop sign or yield sign were recommended (Yun et al. 2013). In one of the recent study, the performance of uncontrolled intersection using microscopic simulation was evaluated by introducing the J-turn at the intersection. It was reported that the performance improved in terms of increased speed, reduced delay and increased capacity (Hemavathy et al. 2015).

2.5 Studies on Delay

The pioneering work on uncontrolled intersections was started by Tanner (1953, 1962 and 1967). The first attempt was made to study the delay at single lane intersections for two conflicting traffic streams (Tanner 1953). Further, the work focused on developing a model considering the queuing concept for estimating the average delay for the minor road vehicles (Tanner 1962) and estimating the capacity of the minor road (Tanner 1967). Cowan (1987) extended Tanner's work by considering wider classes of arrival patterns and two types of delay including the service delay and the queuing delay. Further, Fisk (1989) extended the Tanner's capacity formula for non-priority movements considering different critical gaps for different lanes.

A theoretical model was derived based on the gap acceptance behavior for determining the average service time (Surti 1970). A relationship between delay and traffic intensity was proposed for minor roads at unsignalized intersections (Kimber 1977). Also, queue length and delays at road junctions were computed by taking into account the stochastic nature of traffic and the capacity (Kimber and Holis 1979). Researchers also tried to analyze the influence of stopped delay on the gap acceptance. However, such studies mainly focused on the left turning minor traffic and limited to the conditions where there was less or no queue (Adebisi and Sama 1989). Further, the delay and capacity of the minor street approaches at two-way stop controlled intersections were evaluated and it was observed that the service delay depends on the volume of the conflicting approaches (Kyte et al. 1991). For a stop-controlled intersection, a probabilistic delay model reflecting the driver gap acceptance behavior was developed that was applicable only to right-turning traffic at T-intersection. This was used as an input to estimate the total delay using the queuing model (Madanat et al., 1994). Later, Researchers developed a service delay model based on the microscopic approach and concluded that the proportion of heavy vehicles in the conflicting traffic stream is mainly affected by the service delay (Chandra et al. 2009). Further, the service delay under different priority movements was analyzed by considering two wheelers, three wheelers, passenger cars and heavy vehicles at unsignalized intersections (Ashalatha and Chandra 2011). In a latest study, delay model was developed for urban uncontrolled intersections where it was observed that with increase in the conflicting flow rate, the service delay increased significantly and average service delay was found to be more for heavy vehicles irrespective of the movement type (Praveen et al. 2016).

2.6 Studies on Capacity Estimation

Several researchers carried out studies on the delay and also estimated the capacity at uncontrolled intersections using different methods where different parameters were used for estimating the capacity of the intersection. Capacity of uncontrolled intersections is normally estimated using either empirical or gap-acceptance models. Considering the advantages of the gap-acceptance models, several researchers focused on identifying the parameters affecting the capacity of uncontrolled intersections. The theory of gap-acceptance is the predominant concept for the analysis of unsignalized intersections. This method is based on the critical gap acceptance and follow up times of right turning vehicles from the minor road. Some of the methods developed based on this concept are presented below:

a) Tanner's Model:

Tanner (1962) developed a model [Equation (2.1)] for finding the capacity of unsignalized intersections:

$$C_p = \frac{q_M(1-\lambda t_p)e^{-\lambda(t_c-t_p)}}{1-e^{-\lambda t_f}}, \tag{2.1}$$

where, $\lambda = q_m/3600$ (veh/s),

t_p = minimum headway in the major traffic stream,

t_c = critical gap,

q_m = number of major stream headways, and

t_f = follow-up time.

b) Drew's Method

Drew (1968) developed the Equation (2.2) to find the capacity of unsignalized intersections.

$$C = \frac{v_p e^{-v_p * t_c}}{e^{-v_p * t_c}}, \quad (2.2)$$

where, v_p= major traffic flow rate (veh/sec),

t_c = critical gap, and

t_f= follow-up gap respectively.

c) Harder's model

Harder (1972) developed the Equation (2.3) for estimating the capacity of minor stream which is given as,

$$C = q_c \frac{e^{\frac{-q_c}{3600}(t_c - t_f)}}{e^{\frac{-q_c}{3600} t_f - 1}}, \quad (2.3)$$

where, q_c= conflicting flow,

t_c = critical gap, and

t_f= follow-up gap.

Harder's equation is valid only for exponentially distributed headways in the major stream. This model also requires that all drivers have the same critical gaps and move-up times. However, these values are statistically distributed. Harder also investigated the influence of a statistical distribution of 't_c' and 't_f' on the capacity and incorporated a factor 'f' into the Equation (2.4).

$$C = f.q_c \frac{e^{\frac{-q_c}{3600}(t_c - t_f)}}{e^{\frac{-q_c}{3600} t_f - 1}}, \quad (2.4)$$

where, $f = 1\text{-}q_c^2.10^{-7}$.

This model assumes that the driver keeps the critical gap constant during the queuing time at the intersection.

d) Siegloch model

Siegloch (1973) proposed a consistent framework for the theory of capacities at unsignalized intersections. The capacity of the minor stream 'C' is given by Equation (2.5):

$$C = q_p \int_{t=0}^{\infty} h(t).\, g(t) dt, \tag{2.5}$$

where, g(t) = function for the number of minor street vehicles that can enter the conflict area during one minor stream gap of size 't',

q_p = expected number of gaps of size 't' within the major stream, and
h(t) = statistical density function of all gaps (or) headways in the major stream.

This equation for the capacity of unsignalized intersections forms the foundation of the whole gap-acceptance theory. A linear regression function is used to represent the observation data as shown in Equation (2.6):

$$t = a + b.g(s), \tag{2.6}$$

where, t = dependent variable,

g = independent variable parameters, and

a, b = regression analysis coefficients,

If 't_c' and 't_f' were constant values, then the above expression is given as:

$$g(t) = f(x) = \begin{cases} 0, \; for t < t_o \\ \frac{t-t_0}{t_f}, \; for t \geq t_0 \end{cases} \tag{2.7}$$

where, $t_0 = t_c - \frac{t_f}{2}$,

t_c = critical gap (s), and
t_f = follow-up time (s).

From the combination of these two equations ‘h(t)’ is assumed as exponentially distributed which gives the simple form of Siegloch capacity formula as:

$$C = \frac{3600}{t_f} e^{-p.t_c}. \quad (2.8)$$

The advantage of this is that, it has close relation to the subsequent capacity theory. This method can only be applied for saturated conditions, which are difficult to find in many practical cases.

e) Modified Siegloch Model

Siegloch modified (1974) the capacity equation by assuming the major stream headways as exponentially distributed, and the modified version is given by Equation (2.9).

$$C = \frac{3600}{t_f} e^{-\frac{q_c}{3600}\left(t_c - \frac{t_f}{2}\right)}, \quad (2.9)$$

where, q_c = conflicting flow,

t_c = critical gap (s), and

t_f = follow-up time (s).

f) Jacob’s Method

It is referred as the Troutbeck modification to the Siegloch equation. Changing the ‘α’ term has a pronounced effect on capacity.

$$C = \left(1 - v_p t_m\right) * e^{-\lambda(t_c - t_f)} \Big/ t_f, \quad (2.10)$$

where, v_p = major traffic flow rate (veh/s),

t_c = critical gap, and

t_f = follow-up gap respectively.

g) HCM Method

The analysis procedure for two-way stop controlled and yield-controlled intersections contained in HCM is based on the one developed in Germany in the early 1970’s. Some modifications were made to the previous German method including those based on a limited number of validation studies in the US as well as the addition of the LOS criteria. The HCM method determines minor road capacity based on the availability of gaps in the major traffic stream to vehicles crossing or turning through that stream. Depending on the type of minor movement being made, a critical gap is chosen based on a number of criteria. The gap is used to extract the potential capacity from a family of curves. The procedure requires determination of priority traffic volumes and the potential capacities for each movement, adjustment of potential capacities based on impedance factors and calculation of reserve capacity available to measure the level of service.

h) Troutbeck Model

Troutbeck (1986) proposed a model [Equation (2.11)] to estimate the capacity of minor stream in the case of a single major street.

$$C_n = \frac{v_p.\alpha.e^{-\lambda(t_c-\Delta)}}{1-e^{-\lambda t_f}}, \tag{2.11}$$

where, $\lambda = \frac{\alpha v_p}{1-\Delta v_p}$,

α = proportion of free vehicles,
Δ = minimum headway between two successive vehicles in platoon,
v_p = major traffic flow rate (veh/sec), and
t_p = follow-up time.

i) Polish Method (Choudur 1989)

It contains the procedure for capacity analysis of two-way stop controlled and yield controlled intersections. Level of service estimation is based on reserve capacity and average delay. Gaps are utilized by vehicles in the following priority order:

i. Right turns from the minor road and the left turns from the major road.
ii. Through movements from the minor road.
iii. Left turns from the minor road

The potential capacity, impedance factor and effects of separate or share lanes were derived using the simulation technique, whereas the model itself was calibrated empirically.

j) German Method (Brilon, 1990)

A set of curves for various average road speeds was developed to relate major road volumes to basic capacity for each of the four movements (major road left turns, minor road through and minor road left turns). Capacity curves were calculated from formula developed by Siegloch with experimentally distributed headways in the major flow. Average major road speeds represent the critical gap and move-up times. Significant effect of major road speed on capacity was noticed. After adjusting for impedance, movement capacity was calculated followed by the reserve capacity.

k) Luttenin' Model

Luttenin's model (1990) as shown by Equation (7.12) is used to calculate the minor stream capacity (TL research report, 2003).

$$C_p = \frac{q_M \exp\left(\frac{q_M(t_c - t_p)}{3600 - q_M t_p}\right)}{1 - \exp\left(\frac{-q_M t_f}{3600 - q_M t_p}\right)}, \tag{2.12}$$

where, $\lambda = q_M/3600$ (veh/s),

t_p= minimum headway in the major traffic stream,

t_c = critical gap,

q_M = number of major stream headways, and

t_f = follow-up gap.

A simple formula presented by Catchpole and Plank (1986) calculates the maximum capacity at an intersection for a single stream of minor and major road traffic. The method is applicable for all the vehicle types and for inconsistent behaviour of drivers in the minor road. However, for the cases of inconsistent behaviour of drivers and mixed traffic scenario for the minor road vehicles, Tanner's formula was suggested to calculate the capacity at the intersection. Tanner's formulae for the cases of inconsistent and/ or non-homogeneous minor stream vehicle was generalized in the capacity formulae suggested by these researchers. Further, the influence of minor stream queue formation was quantified by modifying the delay equation developed by Adam. The researcher observed that the Adam's delay, average delay and the minor stream maximum flow gets affected by the grouping in the major stream (Troutbeck 1986). Using the HCM (1985) method, capacity was estimated for the unsignalized intersection which considers the impedance of the minor road flow and use of shared lanes by two or three minor street movements (Jain et al. 1991). The left turning traffic from the minor road and the gap acceptance was observed to be affected significantly by the directional distribution of the conflicting major stream traffic. Thus, the capacity of the minor street left turn movement was observed to be affected significantly by the distribution of major road traffic (Kittelson and Vandehey 1991). The presence of heavy vehicles influenced the entry capacity of unsignalized intersections (Troutbeck et al. 1993).

Considering the advantages of the gap-acceptance models, several researchers have focused on identifying the parameters affecting the capacity of uncontrolled intersections. Gap acceptance model was developed based on limited priority for the major stream (Troutbeck and Kako 1999) and it was reported that there would be an increase in major stream headways due to the merging vehicles especially at higher traffic flows. An alternate method is reported in the literature called addition of conflict stream for determining the capacity of unsignalized intersections (Brilon and Wu, 2001). Driver’s behavior while waiting on the minor road at unsignalized intersection was considered to estimate the capacity of the intersection (Pollatschek et al. 2002). It was observed that vehicles will enter on to the main road only when the risk is lower than the benefits and different populations will have different entry capacities. Mixed vehicular flows consisting of only heavy and light vehicles were also considered to determine the capacity of unsignalized intersections especially by taking into account the Chinese traffic conditions (Li et al. 2003).

In the subsequent years, a relationship between delay and traffic flow on the minor roads was developed (Luttinen 2004) followed by the evaluation of parameters in the capacity models especially for the unsignalized urban intersections (Chodur 2005). Addition conflict flow method (ACF) was observed to be suitable for determining the capacity and headway departure for each of the approaches. Here, capacities were calculated using the IHCM procedure and ACF methods and results were compared using both the methods (Prasetijo 2005). Apart from this, conflict technique method can be used to analyze the influence of non-motorized (pedestrian and bicycle) movements on the capacity of Two-way stop controlled (TWSC) and All-way stop controlled (AWSC) intersections. It was observed that the estimated capacities of vehicular movement reduced with increase in the bicycle and the pedestrian volume and it was reported that the influence of the non-motorized vehicles cannot be ignored (Shi and Li 2008). Researchers also observed that, the pedestrian and the bicycles movements has a significant influence on the capacity. Further, capacity models for the vehicular movements were used under mixed traffic condition for both AWSC and TWSC intersections (Wu 2000; Li et al. 2009). Also, the conflict technique was used to model TWSC intersections (Brilon and Wu 2001; Li and Deng 2008; Li et al. 2009) and multiple-lane approaches (Li et al. 2011). Along with the cars, multiple vehicles such as bicycles and pedestrians were incorporated in analyzing the capacity by using the conflict technique method (Brilon and Miltner 2005; Li et al. 2009).

For estimating the delay, a detailed procedure is presented in the Highway capacity manual 2000 (HCM 2000). Based on the existing capacity and delay models, the results were tested in the past with the data collected in Croatia and it was observed that the capacity and delay values of uncontrolled intersection with normal traffic and road condition yielded good results according to Highway Capacity Manual 2000 procedure (Cvitanic et al. 2007). A new method termed 'conflict technique' was developed to determine the capacity of unsignalized intersections under mixed traffic conditions considering the parameters such as driver behavior, composition of traffic, and roadside activities. On comparing the results with the IHCM, it was observed that for speed range of 11-12 kmph, the capacity values were similar as per IHCM (Prasetijo et al. 2011). Further, the capacity of unsignalized intersection reduces by 11.26% in slightly snow weather condition when compared with sunny weather condition by using gap acceptance procedure (Xu and Cheng 2012). Capacity was used to evaluate the effect of LOS by taking into account the driver behaviour variables such as saturation headway and critical gap which results in high degree of uncertainty. For intersections with LOS ranging from average to poor, forecasting the variation in the volume was difficult resulting in wide variation of the LOS (Zaher et al. 2013). Finally, the capacity of uncontrolled intersections under mixed traffic conditions prevailing in India was determined using the Additive-Conflict-Flow (ACF) method. The important parameter for the ACF method is the occupation time which is defined as the time spent by a vehicle for occupying the conflict area at the intersection. Relationship between the occupation time and various types of vehicles with the conflicting flow of vehicles for different movements were developed. The occupation time increased with increase in the conflicting flow. Thus, the occupation time can be used to estimate the capacity of the priority movements for mixed traffic conditions and non-lane base behavior (Anuroop and Asaithambi 2016).

2.7 Inference from the Literature Review

From the literature, it is observed that critical gap for different types of movement is dependent on the availability of the gaps at the intersection. In the given traffic conditions, the driver has to accept a gap in the conflicting stream and this decision is influenced by the behavioral conditions of the driver. Generally the type of vehicle in the major stream influences this behaviour. Also, the types of turning vehicle also influence the gap acceptance behaviour in the traffic stream. Thus, it is necessary to relate the vehicle types to analyze the gap-acceptance behaviour. The influence of critical gaps significantly affects the delays and in turn affects the capacity of an intersection. Critical gap and follow up time depends on the type of vehicles crossing the intersection. Behavior of driver maneuvering the intersection has influence on the delays, headways and capacity of the intersection. In addition, for various types of vehicles, the accepted and rejected gaps will be different. Thus, the effect of combinations of major stream vehicle-types on the gap-acceptance behaviour of the minor and major stream right-turning vehicle need to be evaluated.

From the literature it is observed that gap acceptance methods developed to find the capacity of unsignalized intersection are mainly based on the calculation of critical gap. Conflict technique is considered to be the good method for calculating the capacity of the intersection. Gap acceptance methods include the mathematical calculations but it does not take into account the driver behaviour. The empirical methods require very large amount of data for which vast surveys are required. Thus, the method should be such that the main parameter for determining capacity should be simple and should involve less number of surveys. Thus, it can be concluded from the review of literature that it is very much essential to evaluate the gap acceptance behaviour of right-turning vehicles at urban uncontrolled intersections under mixed traffic condition. This in turn can be used to estimate the uncontrolled intersection capacity through which the performance of an uncontrolled intersection can be evaluated in terms of the LOS for the possible upgradation to a controlled intersection.

CHAPTER 3

METHODOLOGY

3.1 General

The detailed methodology for the present research work is discussed in this chapter. The traffic flow characteristics at uncontrolled intersections vary significantly with and without priority movements. Most of the vehicles typically move on the road without following the rules of priority. Due to absence of the priority movements at the intersection, the behaviour of the driver plays an important role in analyzing the flow at the intersection. Critical gap is an important parameter for determining the capacity of each movement at uncontrolled intersections. The critical gap estimation and capacity estimation are very complex at uncontrolled intersections particularly in heterogeneous traffic conditions. This can be attributed to widely varying performance including the static and dynamic characteristics of the vehicles. Generally, most of the vehicles accept the gap in a zigzag manner thus making the critical gap determination challenging in these traffic conditions. Even a micro second difference in gap calculation leads to error in capacity estimations. Most of the studies on critical gap estimation are reported for the homogenous traffic conditions, where the rules of priority are truly followed. The subsequent sections of this chapter deals with different stages considered in the current methodology.

3.2 Methodology Flow Chart

The methodology adopted for the present study was executed in different stages and the flowchart is presented in Figure 3.1.

3.2.1 Data Collection

Data was collected from six urban uncontrolled intersections located in various cities in India consisting of both three-legged and four-legged intersections. The sites selected for the study included three four-legged and three three-legged intersections and are located in four states (geographical regions) in India. Two intersections were selected in Warangal city and one intersection was selected in Karimnagar city. Both these cities are located in the south-central state, Telangana. One intersection was selected in Mysore city in Karnataka state and one intersection was selected in Kozhikode city in Kerala state. Karnataka and Kerala states are located in south-western region of India. The last intersection was selected in New Delhi, the capital city of India. Altogether, the six intersections considered in this study are located in four different states of India. In each intersection, the video camera was mounted on a high-raised building such that the entire intersection area is clearly captured that can become effective for further analysis. These six intersections were selected by taking into account the points specified below:

i. Vehicles are not parked at the intersection,
ii. High-raised buildings should be accessible to record the traffic flow,
iii. There should not be any tree cover over the intersection, and
iv. Selected intersection should suitably be away from other intersections such that the traffic flow at the current intersection is not affected.
v. All the intersections are in plain terrain.

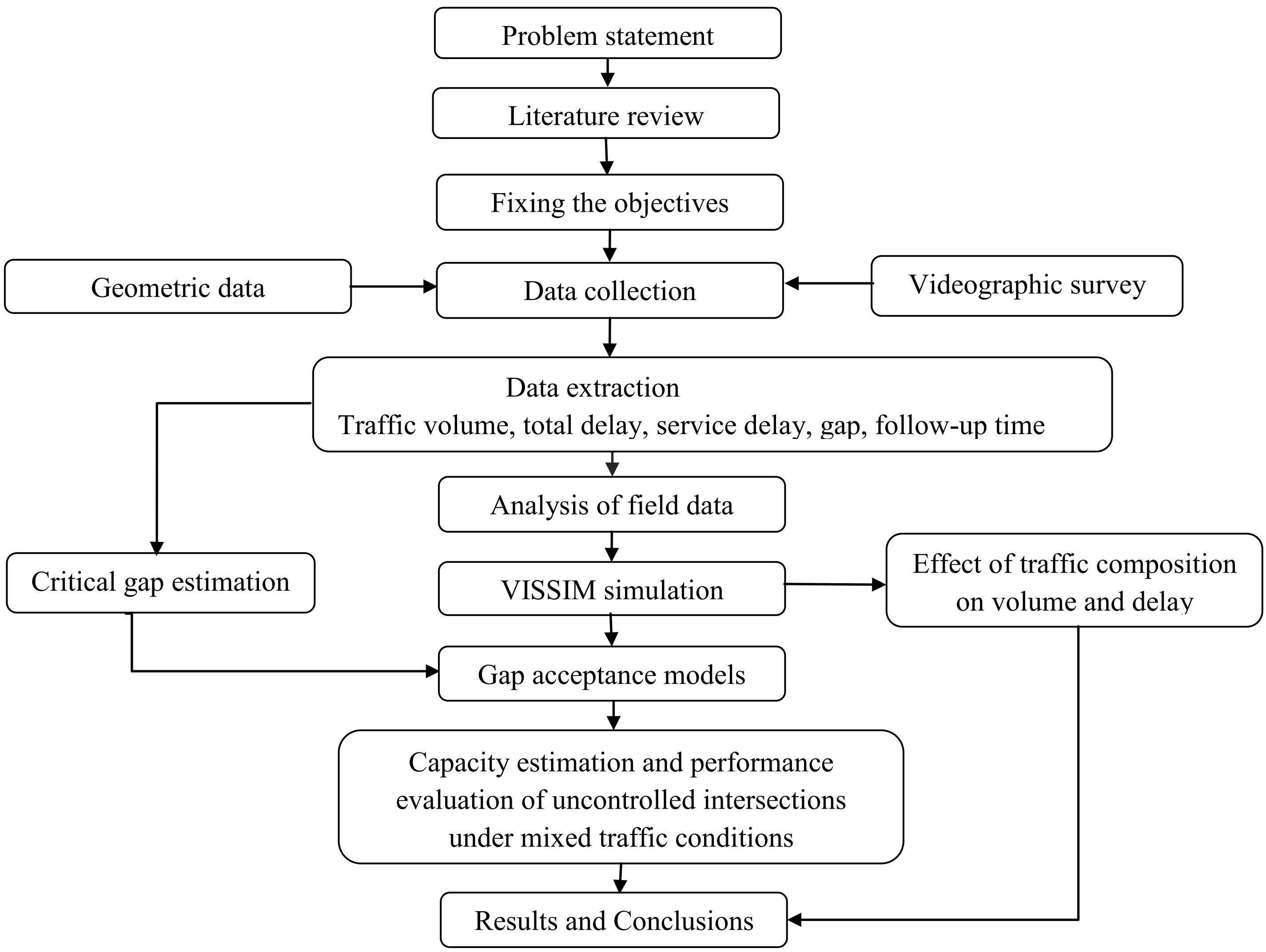


Figure 3.1 Flow chart of study methodology

3.2.2 Data Extraction

The data was extracted using Media Player Classic (MPC-HC Player) and the following parameters were extracted for further analysis:

i. Volume and composition of the vehicles,
ii. Gap accepted and gap rejected by vehicles taking right turn from major and minor road,
iii. Follow-up time,
iv. Total delay and service delay.

3.2.3 Field Data Analysis

It is very much essential to know the gap acceptance behaviour of each right turning vehicle from the major and minor roads to analyze the traffic data at the uncontrolled intersections for the subsequent analysis of these intersections. Apart from this, preliminary analysis was carried out to know the traffic composition, gaps accepted, gaps rejected and the possible outcomes. This data is further used for estimation of critical gap for each of the right turning vehicles from the major and minor road. The data is also used to determine the effect of each right turning vehicle types and major stream vehicle combinations on the gap acceptance behaviour of right turning vehicles at uncontrolled urban intersections. In order to determine the effect of subject vehicle type and conflicting vehicle type on the gap acceptance process, it is necessary to perform the vehicle to vehicle interaction analysis.

3.2.4 Critical Gap Estimation

In this study, critical gap was estimated using the Raff's method. The accepted and rejected gaps were determined for each and every vehicle taking right turn from the major and the minor roads. The accepted and rejected gaps are categorized into different vehicle combinations for the major stream flow. For each of the combinations, critical gaps were determined. These critical gaps are then grouped for each vehicle type and again grouped into leg-wise critical gap.

3.2.5 Simulation Analysis

Base network was developed in VISSIM using the field data and simulation was run to get the required data. Field and simulated data was compared and the resultant error was estimated. The default parameters are used for further analysis if the error is within the permissible limits. Otherwise, calibration of the parameters was required to proceed further. Further, the analysis was carried out to determine the effect of traffic composition on delay and volume at uncontrolled urban intersections.

3.2.6 Calibration Methodology

The microscopic traffic simulation software, VISSIM uses the gap acceptance theory. Figure 3.2 shows the calibration methodology for calibration of the model parameters.

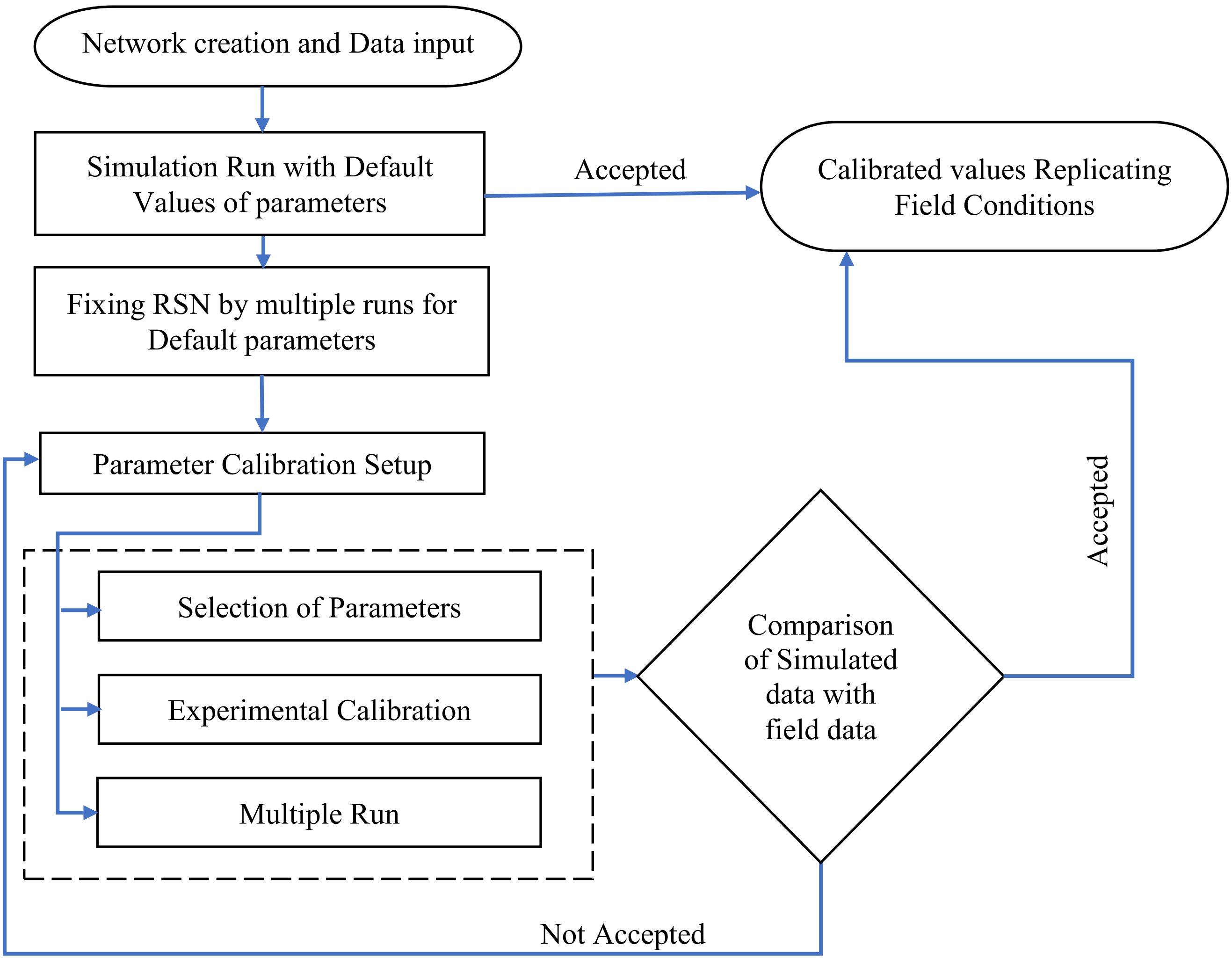


Figure 3.2 Calibration methodology

To develop microscopic simulation model, initial steps will be taken for input data extracted from the field studies. The input data includes the traffic composition, leg-wise volume, movement-wise volume, number of lanes, lane width, and dimension of each vehicle type. The calibration will be executed based on the field data and analyzed to replicate the mixed traffic conditions.

3.2.7 Capacity Estimation and Performance Evaluation

The data obtained from calibrated VISSM model is further used for estimation of the capacity. The calibrated and field capacities for each of the minor and major road right turning vehicles are calculated. Capacity is estimated using existing gap acceptance models including HCM 2010 and Indo-HCM. The model with least MAPE error is considered as the better model for calculating the capacity of the urban uncontrolled intersections and the performance of uncontrolled intersection is subsequently evaluated using the volume-capacity ratio.

3.3 Summary

This chapter provides the methodology adopted for the current research study that involved various stages therein to analyze the traffic at urban uncontrolled three and four-legged intersections. The data collection, extraction and preliminary analysis of the present work will be discussed in the next chapter.

CHAPTER 4

DATA COLLECTION AND EXTRACTION

4.1 General

This chapter deals with the process adopted for data collection at urban uncontrolled intersections and subsequent extraction of the pertinent parameters required for the preliminary analysis. The data thus extracted is used for calculating the critical gap of all the vehicle types considered in this study. Further, the extracted data is also used to determine the effect of various compositions on the delay and volume at the intersection. Using the existing gap acceptance models, capacities are calculated for each of the right turning vehicles from the minor and major roads to select a better model for calculating the capacity of the urban uncontrolled intersections.

4.2 Data Collection

Data was collected from six All-way-stop-controlled (AWSC) intersections in India with two intersections in Warangal city, Telangana state (Intersections 1 and 2), one intersection in Mysore city, Karnataka state (Intersection 3), one intersection near the outskirts of New Delhi (Intersection 4), one intersection in Kozhikode city, Kerala state (Intersection 5), and one intersection in Karimnagar city, Telangana state (Intersection 6). Out of the selected six intersections, three intersections are four-legged (Intersections 1, 2 and 3) and three intersections are three-legged (Intersections 4, 5 and 6). All the four-legged intersections consists of four-lane divided major streets (dual carriageway with two lanes in each direction) and two-lane undivided minor streets (single carriageway with single lane roads). Intersections were selected such that there is a clear difference in the proportion of heavy vehicles across the selected intersections.

At Intersection-1, proportion of heavy vehicles is less compared to that at Intersection-2. Intersection-1 is located on section of NH-163 connecting Warangal and Hyderabad. As the intersection is situated near to the educational institutes, it is very busy in the morning and evening peak hours. Intersection-2 is located on a bypass road in Warangal city where the percentage of heavy vehicles is higher as compared to Intersection -1. Intersection-3 is located in Mysore where the percentage of three-wheelers are very less as compared to Intersections-1 and 2 located in Warangal. All these three intersections are located within the urban area with similar road geometry.

Figure 4.1: Intersection-1 near forest office, Warangal, Telangana

Figure 4.2: Intersection-2 near 100 feet road, Warangal, Telangana

Figure 4.3: Intersection-3 near Kantrajurs, Mysore, Karnataka

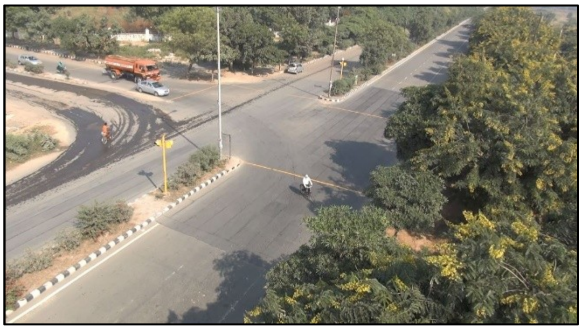


Figure 4.4: Intersection-4 near New-Delhi

Figure 4.5: Intersection-5 near Kozhikode, Kerala

Figure 4.6: Intersection-6 near Karimnagar, Telangana

Intersections- 4, 5 and 6 are three-legged intersections with four lane divided carriageway for the major road whereas the minor road at Intersection-5 is a single carriageway with one lane in each direction and the minor road at Intersections-4 and 6 is a four lane divided with two lanes in each direction. Intersection-4 is located near the outskirts of New Delhi. Intersection-

5 is located in Kozhikode city, in the south-western state of Kerala and Intersection-6 is located in Karimnagar city, in the south-central state of Telangana. All the intersections are located within the urban area with heavy vehicular traffic. The traffic volume at Intersection-4 is less compared to Intersections-5 and 6, as this intersection is located on the outskirts of New Delhi whereas Intersections- 5 and 6 are located within the urban area. Figures 4.1 to 4.6 shows the snapshots of the six intersections selected for the study. These intersections were selected such that there is clear difference between the gap acceptance behaviour because of the heavy vehicles and other vehicle types on the major road at each of these six intersections. Further, enough care was taken during the selection process of these intersections such that each of the intersection is located in a plain terrain with availability of adequate sight distance for each turning movement. It was also ensured that there are no parked vehicles and bus bays at each of the six intersections and these intersections are sufficiently far away from the upstream and downstream signalized intersections.

The videographic data was captured from elevated positions typically from the roof-top of the high-raised buildings near each intersection. Traffic data was extracted from the videographic data and the geometrical factors including the lane width and median width were measured at each intersection. Traffic data including traffic volume, turning volume in each direction, gap accepted, rejected, and follow-up times were extracted by using video player. The data was collected on a typical weekday for a period of four hours covering the morning peak (8.00 a.m. to 10.00 a.m.) and the evening peak (4:00 p.m. to 6.00 p.m.) near the intersection covering all the approaches of the intersection including the area of merging. The geometrical details of all the intersections are shown in Table 4.1.

Table 4.1 Geometrical details of the intersection

Intersection	Number of legs	Major Road		Minor Road	
		No. of lanes in each direction	width of each lane (m)	No. of lanes in each direction	width of each lane (m)
Forest office Junction (Warangal)	4	2	3.5	1	3
100 ft road (Warangal)	4	2	3.5	1	2.25
Kantarajurs road (Mysore)	4	2	3.5	1	3.5
New Delhi	3	2	3.5	2	3
Kozhikode	3	2	3.5	1	3.5
Karimnagar	3	2	3.5	2	3.5

4.3 Data Extraction

The video recordings were played at slow speed on a screen and extracted using Media Player Classic, at an accuracy of 1 in 1000 seconds (0.001 s). Various parameters were extracted from the captured video including: subject vehicle type, gap accepted, gap rejected, conflicting vehicles type, total delay, stopped delay, and traffic volume. The directional distribution of the traffic which includes two-wheelers (2w), three-wheelers (3w), four-wheelers (4w), light commercial vehicles (LCV), buses, bicycles, heavy vehicles (HV), and tractors at all the four-legged and three-legged intersections is shown in Tables 4.2 and 4.3, respectively. Where, EB represents East bound, WB represents West bound, NB represents North bound, and SB represents South bound respectively. Traffic volume is then extracted for four hours duration at all the six intersections. The modal share at all the six intersections are shown in the Figures 4.7 to 4.12. It can be observed that the proportions of two and three wheelers are very high compared to other modes due to the location of these intersections closer to the residential areas.

Table 4.2 Directional distribution of traffic for four-legged intersections (veh/hr)

Different legs of intersection	Intersection-1			Intersection-2			Intersection-3		
	LT	TH	RT	LT	TH	RT	LT	TH	RT
EB	733	7359	1214	465	2865	384	228	2011	343
WB	453	7948	651	388	2827	424	622	2965	1105
NB	62	461	624	477	569	335	416	1295	594
SB	440	508	903	700	705	436	1101	1357	306

Table 4.3 Directional distribution of traffic for three-legged intersections (veh/hr)

Different legs of intersection	Intersection-4			Intersection-5			Intersection-6		
	TH	RT	LT	TH	RT	LT	TH	RT	LT
EB	1891	0	845	4566	0	984	7500	0	891
WB	2239	292	0	8261	837	0	6964	2168	0
SB	0	1234	243	0	833	337	0	1463	929

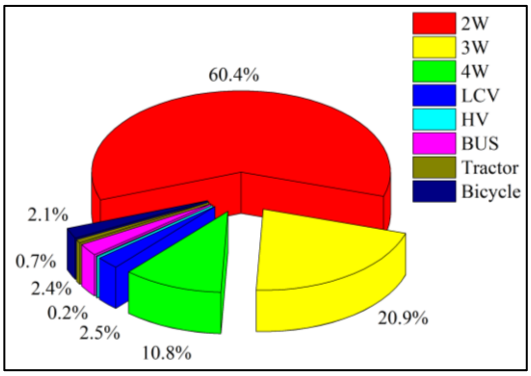


Figure 4.7 Proportion of each vehicle type at Intersection-1

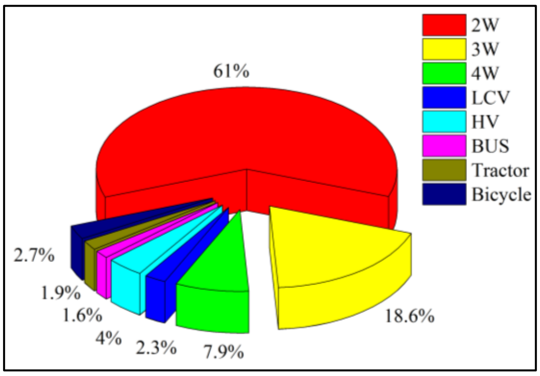


Figure 4.8 Proportion of each vehicle type at Intersection-2

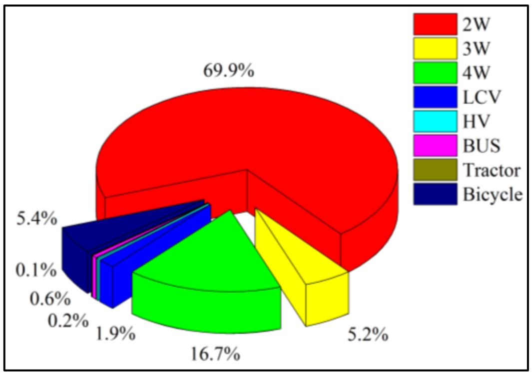


Figure 4.9 Proportion of each vehicle type at Intersection-3

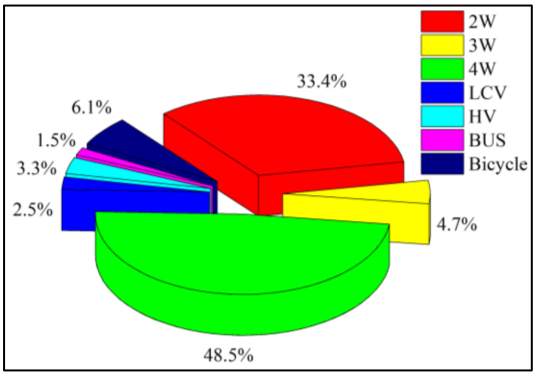


Figure 4.10 Proportion of each vehicle type at Intersection-4

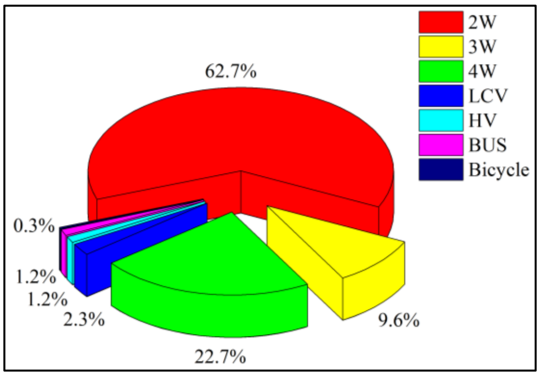


Figure 4.11 Proportion of each vehicle type at Intersection-5

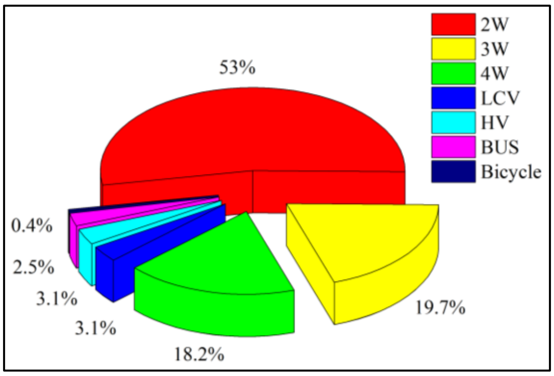


Figure 4.12 Proportion of each vehicle type at Intersection-6

Gap accepted and rejected, follow-up time for each vehicle type was extracted from the videographic data. In addition, the effect of gap available for each vehicle type and with other vehicle types in the stream was extracted from the video. HCM (2010) defines follow up headway as the time between the departure of one vehicle from the minor street and departure of the next vehicle using the same major-street headway, under a condition of continuous queuing on the minor street. The follow-up time is measured from the field as the average of the total readings.

Considering the heterogeneous traffic conditions in India, the minor street vehicles were observed to move about half of the lane width into the intersection area from the stop line as shown in Figure 4.13. Position A is considered as the reference line for recording the arrival of minor street vehicles.

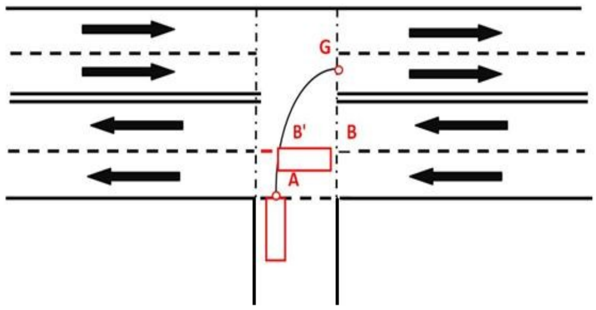


Figure. 4.13 Representation of gap-acceptance measurement for the right turning vehicles

Lag was measured as the time interval between the arrival of a vehicle on the minor road at position A and the arrival of the vehicle on the major street. The video was played with media player classic. Whenever the vehicle reaches point A, the time was noted. The time for the arrival and departure of the vehicles were noted at point G as shown in Figure 4.13. For major stream vehicles, readings were taken at point B which is considered as the reference line for the major stream vehicles.

For determining the critical gap, the recorded video was played on the computer. All the vehicles were divided into seven categories which includes bicycles, two wheelers, three wheelers, four wheelers including cars and jeeps, light commercial vehicles, heavy commercial vehicles, buses and tractors. The data was extracted for each vehicle type for right turning movements on both the major road as well as the minor road. The traffic in India is mixed in nature that lacks in lane discipline at uncontrolled intersections. Even though stop sign posts are available at majority of the intersections, vehicles were not stopping before entering into the intersection area. This warranted for defining a reference point for the minor street vehicle looking for a gap in the major traffic stream. For collecting the data, it is essential to prepare the vehicle combinations for the major traffic stream. Initially, the line of reference for major road and minor road need to be defined. The priority rule is often violated in the mixed traffic conditions wherein most of the major road vehicles are forced to reduce their speeds in order to provide sufficient gap for the vehicles entering the intersection from the minor road as shown in Figure 4.14. This phenomenon affects the critical gap and is taken into account for estimation of the capacity for right-turning vehicles.

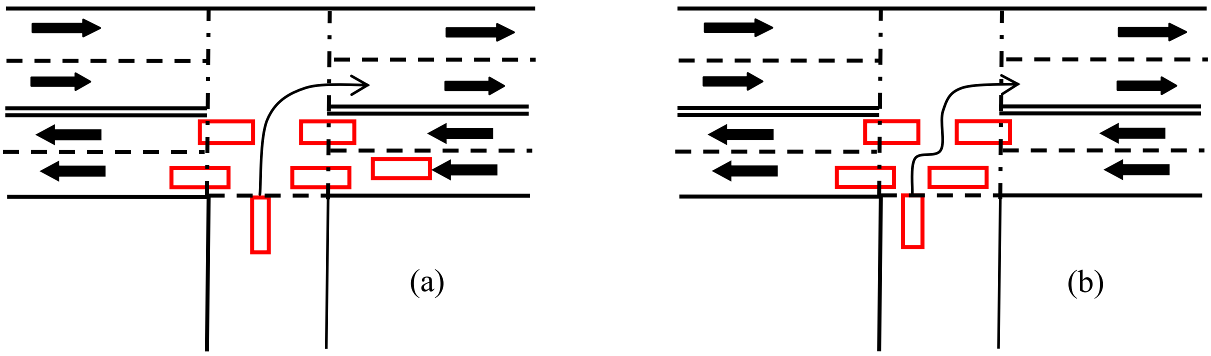


Figure. 4.14 Representation of forced gap-acceptance behaviour for the right turning vehicles

For determining the critical gap value, it is necessary to find the gap accepted and gap rejected for each vehicle type from the recorded video. Gap incorporates both the time and space that a subject vehicle needs to cross safely between two vehicles. Critical gap is the minimum time between successive major stream vehicles in which minor street vehicle can maneuver safely. The follow-up time is calculated for all the right turning vehicles from the minor and major road. Follow-up time is the time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same gap under a condition of continuous queuing. Also, entry time and exit time of vehicles performing right-turn from the minor road and the vehicles performing right-turn from the major road were recorded. The difference between the reading for first vehicle from major road and the arrival time of minor road vehicle results in lag. Some of the gaps are accepted and some are rejected.

Further, the major road vehicles are divided into different combinations that are observed on the major road. The major combinations observed are: 2w-2w, 2w-3w, 2w-4w, 3w-2w, 3w-3w, 4w-2w, and 4w-4w and so on. Gap accepted, gap rejected, lag accepted and lag rejected by major stream vehicle combinations were calculated for each type of right turning vehicles on both the major and minor roads. After determining the accepted and rejected value, critical gap of each type of vehicle was calculated with each major road combination and the critical gap for each vehicle without considering any combinations was also calculated.

From the video recordings, following data is extracted:

a) Traffic volume,

b) Turning volume in each direction with type of vehicle,

c) Width of minor and major roads,

d) Delay for right turning vehicles from major and minor stream at minor road entrance and after leaving the intersection,

e) Proportion of heavy vehicles for each right-turning movement, and traffic composition.

f) Gap accepted and gap rejected, and

g) Follow-up time.

4.4 Summary

The data collected as reported in this chapter is very much useful in analyzing the behaviour of both the minor and major stream vehicles. From this data, critical gaps were calculated for all the right turning vehicles from the minor and major roads. The effect of major stream vehicle combinations on the critical gap of each minor and major road right turning vehicle was also calculated and will be discussed in detail in the next chapter.

CHAPTER 5

DATA ANALYSIS

5.1 General

In general, the presence of varied sizes of vehicles adversely affects the performance of the intersections. Tractors, heavy vehicles and buses require more time to maneuver because of poor acceleration and speed capabilities whereas two wheelers and three wheelers utilize smaller gaps available in the traffic stream. Also, the auto rickshaws are aggressive in nature and are able to adapt the available smaller gaps. In this chapter, the gap acceptance behaviour of each right turning vehicle is studied. Also, the critical gap for each right turning vehicle type is calculated and compared. There are several methods for determining the critical gap such as Siegloch method, Ashworth method, Raff's method, Probit method, Logit method, Probability Equilibrium method, and Maximum Likelihood method. Among these methods, it is reported in the literature that the Maximum Likely Method (MLM) gives accurate results for critical gap. Probability Equilibrium Method (PEM) and Raff's method also gives good estimates for critical gap. However, compared to MLM and PEM, Raff's method is a simplest estimation method (Mohan and Chandra 2016). In this study, Raff's method is used to estimate the critical gap for each of the major and minor road right turning movements at urban uncontrolled intersections. Statistical tests are performed to determine the variation in the critical gaps for all the vehicle types at each of the six intersections. This is followed by fitting various probability distributions for the accepted and available gaps for both the major and minor road right turning vehicles. Further, the effect of right turning movements from both the minor and major traffic streams with respect to the major stream vehicle combinations are analyzed.

5.2 Estimation of Critical Gap and Follow-up Time

As per HCM, critical gap is defined as the bare minimum gap required for the subject vehicle to cross the major stream of vehicles. However, Raff's method defines the critical gap in a different way. According to Raff and Hart (1950), critical gap is defined as "the time when the sum of cumulative probabilities of accepted and rejected gaps is equal to 1". However, based on the graphical solution, Raff (1950) defined the critical gap as "the time at which the cumulative gap accepted and gap rejected cross each other". This time is defined as the critical gap. It is important to note here that, there may be several vehicles, which may be accepting the gap less than critical gap (tc) defined by Raff, i.e., the Raff's method overestimates the critical gap. In a realistic scenario, the actual critical gap will be very much less than the critical gap defined by Raff's method.

The main limitation of Raff's method is that this method is "bias towards cautious drivers". Even though other methods of critical gap estimation including PEM and MLM tries to minimize the bias under different scenarios, Raff's method considers all the accepted and rejected gaps and hence, this method is preferred in this study. It is also important to note here that, the critical gap also depends on the volume of the traffic and also the behavior of the driver. For the same available gap between the major stream vehicles a cautious driver (low risk accepting drivers) may reject a gap whereas non-cautious drivers (high risk accepting drivers) may accept the gap and cross the intersection with higher risk. Thus Raff's method may be considered as much safer method (low risk) for analyzing the gap acceptance behaviour. Here, rejected gaps is not equal to 100% - accepted gaps (or) accepted gaps is not equal to 100% - rejected gaps for any time less than (or) more than the critical gap. Thus, rejected gap = (100% - accepted gap) at the point of intersection, which is defined as the critical gap. Many researchers have defined critical gap in different ways. As per Raff and Hart (1950), critical gap is defined as that size of the gap for which number of accepted gaps shorter than it is equal to the number of rejected gaps longer than it. Also, as per Raff's method, the gap value for which both the density function attains the same value is defined as the critical gap (Raff and Hart 1950). HCM (2000) defines the gap acceptance as the process by which a minor street vehicle accepts an available gap in conflict stream to complete his/her maneuver. As per HCM (2000), critical gap is the minimum time, 'in seconds', between successive major stream vehicles in which minor street vehicle can make a maneuver. A particular driver can reject all the gaps less than the critical gap and accepts all the gaps greater than or equal to 't_c'. In other words, the critical gap (t_c) is defined as the gap that has equal probability of being accepted or rejected (Polus et. al., 2005). In HCM method, gap acceptance behaviour is used for capacity estimation of intersection where the capacity of the intersection is likely to be overestimated because the HCM method considers lower critical gap acceptance. Because of the lower gap acceptance (risk acceptance) behaviour of the drivers, the Raff's method delay will be higher (gap rejected will be more) which in turn leads to a reduction in the capacity of the intersection. That is, the lower risk acceptance results in over estimation of critical gaps and under estimation of capacity. The extracted rejected and accepted gap data are sorted by a gap length of 0.1 second. For every gap length, cumulative number of gap accepted and gap rejected in the corresponding intervals are tabulated. Then the percentage accepted and rejected gaps are determined for the tabulated data. Later the cumulative percentage accepted and rejected gaps are determined. A graph is plotted using the percentage accepted and rejected gaps where the intersecting point of gap accepted curve and gap rejected curve gives the critical gap (t_c) value. Figure 5.1 shows the critical gap of 2w in NB for minor road right turn at Intersection-1. HCM (2010) defines follow up headway as the time between the departure of one vehicle from the minor street and departure of the next vehicle using the same major-street headway, under a condition of continuous queuing on the minor street. Total delay is the time taken by the subject vehicle to cross the intersection. Follow-up time for each leg is determined by taking the average of each follow-up time of all the consecutive entry of vehicle pairs accepting same gap.

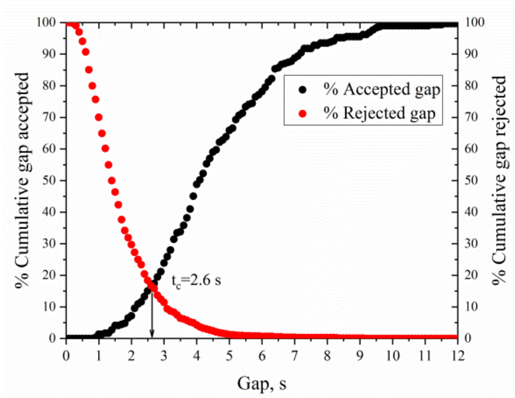


Figure 5.1 Critical gap of 2w in NB for minor road right turn at Intersection-1

5.2.1 Critical gap analysis for Intersection-1

At intersection-1 (Forest office junction) as shown in Table 5.1, for right turning minor stream vehicles from North bound (NB) approach, the critical gap is observed to be lowest for three-wheelers (3w) i.e., 2.275 s, whereas the highest critical gap is observed for bus, i.e., 4.2 s. Similar trends are observed even for turning minor stream vehicles from South bound (SB) approach, where lowest critical gap of 2.15 s is observed for 3w and maximum 4 s critical gap is observed for buses. This shows that for a given volume and proportion of vehicles, lowest critical gaps are observed for small sized vehicles, whereas highest critical gaps are observed for large sized vehicles like buses and Tractors. This shows that the critical gap increases with increase in size of vehicles. It is important to note that, this trend is seen for a particular volume and proportion of vehicles. Such trends may not be observed at other intersections because of different volumes and proportions of vehicles.

Table 5.1 Critical gaps (in seconds) for each vehicle type at Intersection-1

Minor stream vehicles	**Right turning from minor stream**		**Right turning from major stream**	
	NB	**SB**	**EB**	**WB**
2w	2.6	2.3	2.2	2.4
3w	2.275	2.15	2.35	2.9
4w	2.8	2.9	2.7	3.2
LCV	3.4	2.3	2.35	3.2
Tractor	3.85	-	4	4.8
Bus	4.2	4	3.8	-
HV	-	-	5	-

Similar to the minor street right turning vehicles, the critical gap increased with increase in the size of the vehicles for right turning major stream vehicles. However, for major stream right turning vehicles, the lowest critical gap is observed for two-wheelers (2w) when compared to 3w. Highest critical gap is observed for large-sized vehicles which is 5 s for EB heavy vehicle and 4.8 s for WB tractor. In the Table 5.1, the lowest and the highest critical gaps in both the minor stream and the major stream right-turning vehicles were identified irrespective of the minor stream subject vehicles. That is, the lowest critical gaps of 2.275 s and 2.15 s are observed for 3w in the minor stream right-turning vehicles. Similarly, the lowest critical gaps of 2.2 s and 2.4 s are observed for 2w in the major stream right-turning vehicles. The average of the above four lowest critical gaps irrespective of the vehicle-type is obtained as 2.26 s and is termed as the average lowest critical gap. A similar analysis was done to determine the average highest critical gap. The highest critical gaps of 4.2 s and 4.0 s are observed for Bus in the minor stream right-turning vehicles. The highest critical gap of 5.0 s is observed for HV in the major stream right-turning vehicles from the East bound. The highest critical gap of 4.8 s is observed for Tractor in the major stream right-turning vehicles from the West bound. The average of the above four highest critical gaps irrespective of the vehicle-type is obtained as 4.5 s and is termed as the average highest critical gap. The average lowest and average highest critical gaps are calculated based on the observed sample size of 23 critical gaps for various modes in both the minor stream and the major stream right-turning vehicles. Whereas, the critical gap for LCV and 4w are lying within these minimum and maximum values.

Irrespective of the approach (bound), a critical gap of 2.6 s is required for all the right turning 2w to cross the intersection. Similarly, 2.9 s is required for right turning 3w, for 4w it is 3.2 s, 3.4 s for LCV, 4.8 s for tractor, 4.2 s for Bus, and 5 s for HV. It can be observed from the above critical gaps that the critical gap increases with increase in size of the vehicles irrespective of the approaches. It is observed that irrespective of the vehicle type and the bound direction, a 5 s critical gap is required for the right turning vehicles to cross the intersection.

5.2.2 Critical gap analysis for Intersection-2

At intersection-2 (100 ft junction) as shown in Table 5.2, no significant difference between the minimum and maximum critical gaps could be observed. The critical gap for each bound at this intersection is found to be independent of the vehicle types i.e., the critical gaps more or less remained constant irrespective of the vehicle size. When the size of the vehicle is increased there is no significant difference in the critical gaps. The average critical gap for this intersection is 3.98 s whereas the standard deviation is only 0.23 s. This shows that there is no significant variation in the critical gap of different vehicle types for all the approaches at this intersection.

Irrespective of the particular approach, the average critical gaps are computed for each vehicle type and the lowest critical gap is observed to be 3.58 s for a 2w whereas the highest critical gap is observed to be 4.3 s for tractor. The variation in terms of standard deviation from the mean critical gap, i.e., μ-σ and μ+σ are 3.75 s and 4.21 s respectively. Thus, it can be concluded that in order to observe the effect of vehicle type on the critical gap acceptance behavior there should be significant volume of traffic. The total volume of traffic observed at this intersection for four hours duration is 10575 vehicles i.e., 2644 veh/hr whereas, at Intersection-1, the volume of traffic is observed to be 5310 veh/hr. It is also important to note here that apart from volume of vehicles, the proportion of vehicles is also likely to play a significant role in the critical gap acceptance behavior of vehicles at a particular intersection. It is also hypothesized that, at a relatively higher volumes of traffic the critical gap is likely to be vehicle type dependent and also depends on the proportion of vehicles types (vehicle size).

Table 5.2 Critical gaps (in seconds) for each vehicle type at Intersection-2

Major stream vehicles	Right turning from minor stream		Right turning from major stream	
	NB	SB	EB	WB
2w	2.75	4.45	3.2	3.9
3w	4.05	3.95	3.35	4.55
4w	4.8	4.5	2.3	4.6
LCV	3.95	4	4	-
Tractor	4.6	-	4	-
Bus	4.5	-	3.5	-

If the proportion of vehicles between Intersection 1 and 2 are compared, it is more or less similar except that the volume of vehicles at Intersection-1 is much higher than Intersecton-2. Because, Intersection-2 happens to be located on the bypass road and the proportion of HV is bit higher at this intersection.

5.2.3 Critical gap analysis for Intersection-3

At intersection-3, the 3w proportion is only 5% whereas at Intersections- 1 and 2, the 3w proportion is close to 20%. Intersections- 1 and 2 are located in the Warangal city, where most of the commuters share the 3w and the public transport usage is relatively less. Intersection-3 is located in Mysore city, where the commuters only hire the 3w and in general the public transport and 2w usage is significantly higher. Thus, the proportion of 3w is much lower at intersection-3. Further, the 2w proportion is 70% at this intersection, whereas it is 60% at both the Intersections- 1 and 2. Also, the cars proportion is 17% at Intersection-3, whereas the proportion is between 8-11% for the Intersections- 1 and 2. The proportion of buses is 0.7% at Intersectin-3, whereas at Intersections 1 and 2 the proportions ranged between 1.6 - 2.4%. The total volume of traffic observed at this intersection for four hours duration is 12243 vehicles i.e., 3085 veh/hr. Intersecton-3 has volumes more or less same as that of Intersection-2, however there is a significant variation in the proportion of the vehicles at both the Intersections (2 and 3) as discussed above.

As show below in Table 5.3, for right turning minor stream from the NB approach, the critical gap increased with increase in size of the vehicles. The volume of right turning vehicles from NB minor stream is 594 vehicles over four hours duration i.e., 149 veh/hr. However, the right turning vehicles from SB minor stream is only 306 vehicles i.e., 77 veh/hr. The proportion of 2w for SB approach is 70% whereas for 4w it is 15% and the rest 15% includes the remaining vehicle types. Because of the relatively low volumes of traffic when compared to NB approach and significantly higher proportions of 2w and 4w, critical gaps could be observed only for these two vehicle types. Because of the low volumes and biased proportions, no significant observation could be drawn regarding the critical gap acceptance behavior.

Table 5.3 Critical gaps (in seconds) for each vehicle type at Intersection-3

Minor stream vehicles	Right turning from minor stream		Right turning from major stream	
	NB	SB	EB	WB
2w	2.6	3.1	1.8	2.3
3w	2.5	-	0.8	2.3
4w	4.4	1.9	3.25	3.5
LCV	-	-	3	-
Bus	-	-	0.7	-

Because of lower traffic volumes of vehicles at this intersection, no significant observations could be drawn for right turning major stream approaches. Out of 100% proportion of vehicles at this intersection, 70% consists of 2w, 15% are 4w and the rest 15% includes the remaining vehicle types. If the critical gap of 2w and 4w are compared irrespective of the approaches, the critical gap for car is observed to be slightly higher (3.26 s) than that of 2w (2.45 s). This shows that the critical gap increases with increase in size of the vehicle.

5.2.4 Critical gap analysis for Intersection-4

At intersection-4 (New Delhi) as shown in Table 5.4, for right turning minor stream vehicles of South bound approach, the critical gap is observed to be lowest for two-wheelers (2.9 s), whereas the highest critical gap is observed for bicycles (5.1 s). Similar trends are observed even for major stream right turning vehicles from west bound approach, where lowest critical gap of 3 s is observed for small sized vehicle such as 3w and maximum 4.6 s critical gap is observed for bicycles. It is important to note here that, in contrast to other intersections, because of higher proportion of bicycles (6%) and lower speeds of bicycles when compared to other motorized vehicles, the critical gaps are observed to be much higher for bicycles.

Further, the 3w proportion is also relatively less at this intersection (5%) when compared to other intersections. Ignoring the relatively small proportion of 3w, for a given volume and proportions of vehicles lowest critical gaps are observed for small sized vehicles, whereas highest critical gaps are observed for large sized vehicles such as HV. As mentioned earlier, because of the slower speeds of the non-motorized vehicles such as bicycles the critical gap is observed to be much higher than the large sized vehicles. This shows that the speed of the vehicles also play a critical role in estimation of the critical gaps at uncontrolled urban intersections.

Table 5.4 Critical gaps (in seconds) for each vehicle type at Intersection-4

Type of Vehicle	Right turning from minor stream	Right turning from major stream
	SB	**WB**
2w	2.9	3.2
3w	4.9	3
4w	3.5	4.1
LCV	4	3.5
HV	4.5	4.25
Bus	-	4.25
Bicycle	5.1	4.6

Similar to the minor street right turning vehicles, the critical gap increased with increase in the size of the vehicles for right turning major stream vehicles. If the right turning vehicles at these two (minor and major) streams are compared, the lowest critical gap of 2.9 s is observed for two-wheelers. Similarly, the highest critical gap of 5.1 s is observed for bicycles. The average lowest critical gap for all the approaches is observed to be 2.95 s whereas the standard deviation is 0.07 s. Similarly, average highest critical gap for all the approaches is observed to be 4.85 s whereas the standard deviation is 0.35 s. Thus, the average lowest critical gap at this intersection is observed to be 3.05 s for small sized vehicles (2w). Similarly, the average highest critical gap is observed to be 4.85 s for bicycles.

Because of the lower volumes of traffic at this three-legged intersection, no specific increase in the critical gaps of major right turning streams are observed when compared to minor stream right turning vehicles. This intersection has a unique feature with considerable proportion of bicycles (6.1%). Further, the proportion of 3w is much less at this intersection when compared to the other two three-legged intersections.

Irrespective of the approach (bound), a critical gap of 3.2 s is required for all the right turning 2w to cross the intersection. Similarly, 4.9 s is required for right turning 3w, for 4w it is 4.1 s, 4 s for LCV, 4.5 s for HV, 4.25 s for Bus, and 5.1 s for Bicycle. It can be observed from the above critical gaps that the critical gap increases with increase in size of the vehicles irrespective of the approaches.

5.2.5 Critical gap analysis for Intersection-5

At intersection-5 (Kozhikode) as shown in Table 5.5, for the right turning minor stream vehicles of South bound approach the critical gap is observed to be lowest for three-wheelers (2.4 s), whereas the highest critical gap is observed for LCV (4 s). Similar trends are observed even for major stream right turning vehicles from west bound approach, where lowest critical gap of 2.5 s is observed for 3w and maximum 4.95 s critical gap is observed for HV. This shows that for a given volume and proportion of vehicles, lowest critical gaps is observed for small sized vehicles, whereas highest critical gaps are observed for large sized vehicles. This shows that the critical gap increases with increase in size of vehicles. It is important to note that, this trend is seen for a particular volume and proportion of vehicles. Such trends may not be observed at other intersections because of different volumes and proportions of vehicles.

Similar to the minor street right turning vehicles, the critical gap increased with increase in the size of the vehicles for right turning major stream vehicles. If the right turning vehicles at these two (minor and major) streams are compared, the lowest critical gap of 2.4 s is observed for three-wheelers. Similarly, the highest critical gap of 4.95 s is observed for HV. The average lowest critical gap for all the approaches is observed to be 2.45 s whereas, the standard deviation is 0.07 s. Similarly, average highest critical gap for all the approaches is observed to be 4.48 s, whereas the standard deviation is 0.67 s. Thus, the average lowest critical gap at this intersection is observed to be 2.45 s for small sized vehicles (3w). Similarly, the average highest critical gap is observed to be 4.95 s for large sized vehicle (HV).

It is observed that irrespective of the vehicle types, the critical gaps for major stream right turning vehicles are higher than the minor stream right turning vehicles with exception for

LCV due to low proportion (2%). It is important to note here that, the volume of minor street right turning vehicles is 208 veh/hr, whereas the volume for major street right turning vehicles is 210 veh/hr. This shows that the volume of traffic is almost same for both the right turning approaches. Even though the critical gaps for major stream right turning vehicles is slightly higher than the minor stream right turning vehicles, the difference is marginal.

Table 5.5 Critical gaps (in seconds) for each vehicle type at Intersection-5

Type of Vehicle	Right turning from minor stream	Right turning from major stream
	SB	**WB**
2w	2.45	2.6
3w	2.4	2.5
4w	3.15	3.5
LCV	4	3.05
HV	-	4.95
Bus	-	-
Bicycle	-	-

Irrespective of the approach (bound), a critical gap of 2.6 s is required for all the right turning 2w to cross the intersection. Similarly, 2.5 s is required for right turning 3w, for 4w it is 3.5 s, 4 s for LCV, and 4.95 s for HV. It can be observed from the above critical gaps that the critical gap increases with increase in size of the vehicles irrespective of the approaches.

5.2.6 Critical gap analysis for Intersection-6

At Intersection-6 (Karimnagar) as shown in Table 5.6, for right turning minor stream vehicles of south bound approach the critical gap is observed to be lowest for two-wheelers (1.7 s), whereas the highest critical gap is observed for HV (3.0 s). Similar trends are observed even for major stream right turning vehicles from west bound approach, where lowest critical gap of 2.2 s is observed for 2w and maximum 6.2 s critical gap is observed for HV. This shows that for a given volume and proportion of vehicles lowest critical gaps are observed for smaller sized motorized vehicles, whereas highest critical gaps are observed for large sized vehicles. This shows that the critical gap increases with increase in size of vehicles. It is important to note that, this trend is seen for a particular volume and proportions of vehicles. Such trends may not be observed at other intersections because of different volumes and proportions of vehicles.

Table 5.6 Critical gaps (in seconds) for each vehicle type at Intersection-6

Type of Vehicle	Right turning from minor stream	Right turning from major stream
	SB	WB
2w	1.7	2.2
3w	1.9	2.3
4w	2.2	2.7
LCV	2.2	2.8
HV	3	6.2
Bus	2.65	4.2
Bicycle	-	3.6

Similar to the minor street right turning vehicles, the critical gap increased with increase in the size of the vehicles for right turning major stream vehicles. If the right turning vehicles at these two (minor and major) streams are compared, the lowest critical gap of 1.7 s is observed for two-wheelers. Similarly, the highest critical gap of 6.2 s is observed for HV. The average lowest critical gap for all the approaches is observed to be 1.95 s whereas, the standard deviation is 0.35 s. Similarly, average highest critical gap for all the approaches is observed to be 4.6 s, whereas the standard deviation is 2.26 s. Thus, the average lowest critical gap at this intersection is observed to be 1.95 s for small sized vehicles (2w). Similarly, the average highest critical gap is observed to be 4.6 s for large sized vehicle (HV).

It is observed that irrespective of the vehicle types, the critical gaps for major stream right turning vehicles are always higher than the minor stream right turning vehicles. It is important to note here that, the volume of minor street right turning vehicles is 366 veh/hr, whereas the volume for major street right turning vehicles is 542 veh/hr. Because of the higher volumes of right turning vehicles from the major stream these vehicles are waiting for relatively longer time when compared to the minor stream vehicles to complete the right turning maneuver.

Irrespective of the approach (bound), a critical gap of 2.2 s is required for all the right turning 2w to cross the intersection. Similarly, 2.3 s is required for right turning 3w, for 4w it is 2.7 s, 2.8 s for LCV, 6.2 s for HV, 4.2 s for Bus, and 3.6 s for Bicycle. It can be observed from the above critical gaps that the critical gap increases with increase in size of the vehicles irrespective of the approaches. As highlighted above, because of the higher volume of major stream right turning vehicles, the critical gap for HV is 6.2 s whereas the critical gap for minor stream right turning HV is only 3 s.

5.2.7 Critical gap analysis at all the Intersections

In order to compute the critical gap for both the minor stream and the major stream right-turning vehicles, the entire set of right-turning vehicles from that particular approach are considered by following the same procedure as described earlier. For four and three-legged intersections, it is important to see the effect of volume, proportions of vehicles and geometrics on the critical gap values as discussed below for different scenarios. Table 5.7 shows the critical gap values for each leg of the six intersections considered in the study.

Table 5.7 Critical gaps (in seconds) for each leg at all the six intersections

Traffic stream bound	**Int-1**	**Int-2**	**Int-3**	**Int-4**	**Int-5**	**Int-6**
NB minor stream	2.45	3.0	3.0	-	-	-
SB minor stream	2.45	4.35	2.2	3.8	2.9	2.1
EB major stream	2.3	3.1	2.7	-	-	-
WB major stream	2.35	3.55	2.5	4.5	3.6	2.0

a. Effect of traffic volume on critical gap for a four-legged intersection with similar proportion of vehicles and geometry.

On comparing Intersections- 1 and 2 as shown in Table 5.7, the proportions of all vehicle types are more or less similar whereas, the volume of traffic in all the approaches for Intersection-1 is much higher than Intersection-2. As the volume of traffic decreases, the right turning vehicles from a particular approach will have more opportunity and time to complete the right turning maneuver resulting in higher critical gaps. It can be seen that the critical gap for right turning NB approach is 2.45 s for Intersection-1 whereas it is 3 s for Intersection-2. Similarly, for SB approach, the critical gap is 2.45 s for Intersection-1 whereas it is 4.35 s for Intersection-2. Similar trends are observed even for major stream right turning approach for both these intersections.

b. Effect of proportion of vehicles on critical gap for a four-legged intersection with similar traffic volume and geometry.

On comparing Intersections- 2 and 3 as shown in Table 5.7, the traffic volumes on both the intersections are more or less similar. The proportions of 2w, bicycles and cars on Intersection-3 are much higher than those at Intersection-2. Similarly, the proportion of 3w's, HV's and tractors at Intersection-2 are higher than those at Intersection-3. Now it can be seen that, for similar volumes of traffic on Intersections- 2 and 3, the proportions of smaller vehicles are much higher on Intersection-3 when compared to Intersection-2. The smaller sized motorized vehicles can accept the available smaller gaps within the major stream vehicles and can complete the right turning maneuver in very less time when compared to the large sized vehicles resulting in lower critical gaps for higher proportions of smaller sized motorized vehicles at Intersection-3.

c. Effect of traffic volume on critical gap for a three-legged intersection with similar proportion of vehicles and no effect of number of lanes on critical gap.

On comparing Intersections- 5 and 6, the number of lanes for the minor approach are two for Intersection-6 for each direction whereas there is only one lane per direction for Intersection-5 apart from higher proportions of 3w's at Intersecton-6 when compared to Intersection-5 and higher proportions of 2w's and cars at Intersection-5 when compared to Intersection-6. It is important to note here that, even though the number of lanes is more than one for the minor approach, the right turning vehicles are likely to occupy the extreme right lane and thus the number of lanes for a minor street approach is expected to have insignificant effect on the critical gap. On comparing Intersections- 5 and 6, the proportions of all vehicle types are more or less similar except 10% difference in 2w and 3w proportion. Whereas, the volume of traffic in all the right turning and through movement approaches for Intersection-6 is much higher than Intersection-5. As the volume of traffic decreases, the right turning vehicles from a particular approach will have more opportunity and time to complete the right turning maneuver resulting in higher critical gaps. It can be seen that the critical gap for right turning south bound approach is 2.1 s for Intersection-6 whereas it is 2.9 s for Intersection-5. A similar trend is observed for major stream right turning approach for both these intersections.

d. Effect of traffic volume and proportion of vehicles on critical gap for a three-legged intersection with same number of lanes.

On comparing Intersections- 4 and 6 as shown in Table 5.7, where both are three-legged intersections with same geometry, the traffic volume is much higher for Intersecton-6 when compared to Intersection-4 including relatively higher proportions of 2w's and 3w's at Intersection-6, whereas the proportion of cars and bicycles is higher at Intersection-4. Considering the higher proportions of 2w's and 3w's at Intersection-6 for relatively higher volumes of traffic when compared to Intersection-4 i.e., for south bound approach, the critical gap is 2.1 s for Intersection-6 whereas it is 3.8 s for Intersection- 4. Similarly, the critical gap is 2 s for west bound approach at Intersection-6 whereas it is 4.5 s at Intersection-4.

Finally, on comparing the four and three-legged intersections as shown in Table 5.7, for example on comparing Intersections- 1 and 6 with more or less similar traffic volumes and proportions, Intersection-1 is four-legged whereas Intersection-6 is three-legged. It can be observed that because of the higher number of conflicts points at a four-legged intersection i.e., Intersection-1, the critical gaps are much higher at this intersection when compared to three legged intersection i.e., Intersection-6. Thus, the critical gap at Intersection-6 for south bound approach is 2.1 s whereas it is 2.45 s for south bound approach at Intersection-1. Also, the critical gap at Intersection-6 for west bound approach is 2 s whereas it is 2.35 s for west bound approach at Intersection-1.

Table 5.8 Critical gap for all legs at different intersections

Intersection	**Critical Gap (s)**
Intersection-1	2.5
Intersection-2	3.2
Intersection-3	2.8
Intersection-4	3.9
Intersection-5	2.8
Intersection-6	2.1

Table 5.8 shows the critical gap values for all the six intersections. Finally, it is concluded that as traffic volume increases critical gap of the intersection decreases. Also, the vehicle size is significantly influencing the accepted critical gaps. Table 5.9 shows the follow-up times for

all the legs at different intersections considered in the study. The follow-up times are observed to higher for Intersection-3 because the EB through-traffic on the major road is very less. Thus, the minor road right turning vehicles get more opportunity to clear the intersection using the same available gap on the major stream.

Table 5.9 Follow-up time (in seconds) for all legs at different intersections

Name of the Intersection		Follow-up Time (s)
Intersection-1	NB	1.989
	SB	1.904
	EB	1.475
	WB	1.428
Intersection-2	NB	6.305
	SB	3.942
	EB	4.238
	WB	4.424
Intersection-3	NB	3.465
	SB	9.509
	EB	4.773
	WB	11.899
Intersection-4	WB	3.564
	SB	3.984
Intersection-5	WB	2.713
	SB	3.049
Intersection-6	WB	3.185
	SB	2.773

5.2.8 Statistical analysis of the critical gap

For statistical analysis, ANOVA test was performed using SPSS. Critical gap of eight different vehicle types are considered for the analysis which includes 2w, 3w, 4w, LCV, HV, bus, tractor and bicycle. From Table 5.10, through statistical analysis it can be observed that significant variation is observed between the critical gap values for all the vehicle types at Intersections- 1, 4, 5 and 6 based on the p-value ($p < 0.05$). Whereas, Intersections- 2 and 3 did not show significant variation in the critical gap values for all the vehicle types considered in the study as the p-value is greater than 0.05.

Table 5.10 Statistical analysis for critical gap

Name of the Intersection	Source of Variation	Sum of Squares	df	Mean Square	F	P-value	Sig.
Int-1	Between Groups	61.439	5	12.288	3.681	.003	Yes
	Within Groups	15211.684	4557	3.338			
	Total	15273.123	4562				
Int-2	Between Groups	28.287	5	5.657	.701	.623	No
	Within Groups	4190.052	519	8.073			
	Total	4218.339	524				
Int-3	Between Groups	10.450	4	2.613	.322	.863	No
	Within Groups	4942.346	610	8.102			
	Total	4952.796	614				
Int-4	Between Groups	84.645	6	14.108	2.124	0.049	Yes
	Within Groups	2510.469	378	6.642			
	Total	2595.114	384				
Int-5	Between Groups	43.142	4	10.786	2.469	.043	Yes
	Within Groups	16370.950	3747	4.369			
	Total	16414.092	3751				
Int-6	Between Groups	71.194	7	10.171	2.965	.004	Yes
	Within Groups	21483.279	6262	3.431			
	Total	21554.473	6269				

5.2.9 Distribution of gaps

In general, fitting an appropriate distribution for the accepted gaps and available gaps will be very much useful: (i) for traffic flow simulation, (ii) for evaluating the traffic flow on the transportation facility, and (iii) for evaluation of the uncontrolled intersections. The data can also be used for estimating the critical gap. Various probability distributions are considered in this study to the fit the accepted gaps and the available gaps. Figures A.1 and A.3 respectively shows the histogram of the accepted and available gaps superimposed with various distributions pertaining to the major road right turning vehicles. Also, the histogram for the accepted and available gaps superimposed with various distributions pertaining to the minor road right turning vehicles are shown respectively in Figures A.2 and A.4. The probability density functions (pdf) of normal, exponential, lognormal and gamma distributions are fitted to all the six intersections as shown in Figures A.1 to A.4. K-S test (Kolmogorov-Smirnov) is used to measure the goodness of fit for the fitted distribution. Here, K-S test value is the maximum distance between the empirical distribution function of the sample and the cumulative distribution function of the reference distribution.

The statistical parameters of the distributions pertaining to accepted gaps for both the major road and the minor road right turning and the corresponding critical value at 95% confidence level for each distribution is shown respectively in Tables 5.11 and 5.12. Also, the corresponding mean, standard deviation of the gap data and K-S test values for the fitted distributions for all the six intersections are also shown in Tables 5.11 and 5.12.

From Table 5.11 it can be seen that at 95% confidence level at almost all the intersections, major road right turning vehicles followed gamma distribution with K-S test values that are less than the critical values. However, for few cases the K-S test values are observed to be less than the critical values for lognormal and normal distributions. That is, for Intersection-1 EB, lognormal distribution is followed. Further, for Intersection-2 WB and Intersection- 4 WB, the K-S test values are observed to be less and followed normal distribution. It is important to note here that even though for the above three cases the gamma distribution is not followed, the K-S values are observed to be within the threshold values.

Table 5.11 Statistical parameters of the distributions pertaining to accepted gaps for major road right turn

Intersection		**Accepted gaps**		**K-S test values**				**Critical values for K-S test****	**No. of observations**
		Mean	**std.dev**	**Normal**	**Lognormal**	**Exp.**	**Gamma**		
For Major road RT									
INT-1	EB	4.438	1.806	0.093	**0.023**	0.338	0.043	0.069	382
	WB	4.248	2.319	0.099	0.054	0.271	**0.044**	0.102	164
INT-2	EB	6.287	2.858	0.132	0.098	0.286	**0.081**	0.134	102
	WB	5.965	2.456	**0.064**	0.128	0.342	0.105	0.21	40
INT-3	EB	5.436	2.816	0.174	0.105	0.326	**0.105**	0.172	60
	WB	5.796	2.603	0.097	0.081	0.299	**0.058**	0.138	94
INT-4	WB	7.492	2.798	**0.104**	0.172	0.364	0.125	0.224	35
INT-5	WB	5.097	2.191	0.094	0.06	0.311	**0.049**	0.084	262
INT-6	WB	4.415	2.282	0.071	0.069	0.248	**0.022**	0.048	817

Bold data indicates significant values and are the lowest K-S test values among four distributions

** at 95% confidence level

From Table 5.12 it can be seen that at 95% confidence level at almost all the intersections, minor road right turning vehicles followed gamma distribution with lowest K-S test values that are less than the critical values. However, for few cases the K-S test values are observed to be less than the critical values for lognormal and normal distributions. That is, for Intersection-2 SB lognormal distribution is followed. Further, for Intersection-2 NB and Intersection- 3 NB and SB, the K-S test values are observed to be less and followed normal distribution. It is important to note here that even though for the above three cases the gamma distribution is not followed, the K-S values are observed to be within the threshold values except Intersection-3 NB. Thus, it is concluded that, gap accepted by the major road and the minor road right turning vehicles follows gamma distribution.

Table 5.12 Statistical parameters of the distributions pertaining to accepted gaps for minor road right turn

Intersection		Accepted gaps		K-S test values				Critical values for K-S test**	No. of observations
		mean	std.dev	Normal	Lognormal	Exp.	Gamma		
For Minor road RT									
INT-1	NB	4.581	2.03	0.092	0.057	0.298	**0.033**	0.079	293
	SB	3.995	2.256	0.1	0.04	0.224	**0.029**	0.073	346
INT-2	NB	6.155	2.832	**0.103**	0.133	0.268	0.108	0.148	82
	SB	6.609	2.21	0.099	**0.061**	0.396	0.072	0.215	38
INT-3	NB	5.806	2.979	**0.077**	0.09	0.091	0.237	0.14	91
	SB	5.683	2.651	**0.089**	0.148	0.281	0.116	0.254	27
INT-4	SB	6.871	2.872	0.096	0.122	0.31	**0.065**	0.126	117
INT-5	SB	5.851	2.392	0.067	0.058	0.294	**0.028**	0.067	411
INT-6	SB	5.26	2.845	0.071	0.096	0.218	**0.049**	0.08	287

Bold data indicates significant values and are the lowest K-S test values among four distributions
** at 95% confidence level

The statistical parameters of the distributions pertaining to available gaps for both the major road and the minor road right turning and the corresponding critical value at 95% confidence level for each distribution is shown respectively in Tables 5.13 and 5.14. Also, the corresponding mean, standard deviation of the gap data and K-S test values for the fitted distributions for all the six intersections are also shown in Tables 5.13 and 5.14.

From Table 5.13 it can be seen that at 95% confidence level at almost all the intersections, major road right turning vehicles followed lognormal distribution with K-S test values that are less than the critical values. However, for few cases the K-S test values are observed to be less than the critical values for gamma and exponential distributions. That is, for Intersection-2 WB gamma distribution is followed. Further, for Intersection-3 EB and WB, the K-S test values are observed to be less and followed exponential distribution. It is important to note here that even though for the above three cases the lognormal distribution is not followed, the K-S values are observed to be within the threshold values except for Intersection-3.

Table 5.13 Statistical parameters of the distributions pertaining to available gaps for major road right turn

Intersection		**Available gaps**		**K-S test values**				**Critical values for K-S test****	**No. of observations**
		mean	**std.dev**	**Normal**	**Lognormal**	**Exp.**	**Gamma**		
For Major road RT									
INT-1	EB	2.044	1.858	0.184	**0.044**	0.12	0.083	0.036	1398
	WB	2.422	2.078	0.133	**0.045**	0.051	0.109	0.061	497
INT-2	EB	4.07	3.217	0.14	**0.065**	0.12	0.071	0.099	190
	WB	3.637	2.694	0.13	0.084	0.137	**0.063**	0.126	117
INT-3	EB	3.383	3.064	0.15	0.137	**0.092**	0.117	0.128	112
	WB	3.438	3.017	0.157	0.113	**0.075**	0.098	0.098	191
INT-4	WB	4.24	3.269	0.164	**0.077**	0.128	0.086	0.132	101
INT-5	WB	2.122	2.047	0.195	**0.032**	0.095	0.08	0.039	1228
INT-6	WB	1.644	1.766	0.201	**0.048**	0.117	0.147	0.02	4755

Bold data indicates significant values and are the lowest K-S test values among four distributions

** at 95% confidence level

From Table 5.14 it can be seen that at 95% confidence level at almost all the intersections, minor road right turning vehicles followed lognormal distribution with K-S test values that are less than the critical values. However, for few cases the K-S test values are observed to be less than the critical values for gamma and normal distributions. That is, for Intersection-2 NB gamma distribution is followed. Further, for Intersection-6 SB, the K-S test values are observed to be less and followed normal distribution. It is important to note here that even though for the above two cases the lognormal distribution is not followed, the K-S values are observed to be within the threshold values except for Intersection-6. Thus, it is concluded that, gap acceptance by the major and the minor road right turning vehicles follows Lognormal distribution.

Table 5.14 Statistical parameters of the distributions pertaining to available gaps for minor road right turn

Intersection		Available gaps		K-S test values				Critical values for K-S test**	No. of observations
		mean	std.dev	Normal	Lognormal	Exp.	Gamma		
For Minor road RT									
INT-1	NB	2.322	1.816	0.139	**0.025**	0.158	0.059	0.037	1337
	SB	2.263	1.887	0.146	**0.035**	0.061	0.137	0.037	1313
INT-2	NB	4.24	3.085	0.157	0.096	0.122	**0.084**	0.111	150
	SB	3.962	2.827	0.142	**0.078**	0.152	0.085	0.134	103
INT-3	NB	3.244	2.917	0.157	**0.049**	0.062	0.059	0.084	260
	SB	3.085	2.833	0.216	**0.111**	0.131	0.125	0.163	67
INT-4	SB	3.516	3.075	0.15	**0.054**	0.084	0.057	0.069	385
INT-5	SB	2.203	2.161	0.198	**0.03**	0.091	0.084	0.027	2529
INT-6	SB	2.12	2.233	**0.021**	0.042	0.116	0.138	0.035	1525

Bold data indicates significant values and are the lowest K-S test values among four distributions
** at 95% confidence level

5.3 Effect of Right Turning Vehicle Type and Major Stream Vehicle Combinations on the Gap Acceptance Behaviour of Right Turning Vehicles at Uncontrolled Urban Intersections

The methodology of the current study includes suitable site selection for the field survey, collection and extraction of field data, followed by identification and statistical analysis of the factors leading to gap acceptance. For collecting the gap acceptance data, combination of vehicles are prepared for the major stream traffic. For the gap-acceptance study, only those vehicles that are taking right turn from both the minor and major streams are considered independently. The time of arrival and exit time for each vehicle type are noted for both the minor and major stream vehicles. For the major stream vehicles, the readings are taken at the reference line. The gap values obtained from the video are then extracted. In general, at all the six intersections considered in this study, the left turning traffic is not predominant. Further, left turning traffic is quite smooth as there is no gap acceptance involved and it is a simple merging process. However, for right turning vehicles, gap acceptance phenomenon is influential. Therefore, the data was extracted specifically for the right turning vehicles. Through movement is outside the scope of this study. According to the available data, possible combinations of major stream vehicles are considered and critical gaps are determined for each combination for both the minor stream and major stream right turning subject vehicles. Radar plots are generated for all the intersections by considering the observed major stream vehicle combinations and the resulting gap acceptance behaviour of each type of right turning vehicle from both the minor and major roads separately.

Total traffic is divided into eight categories: two-wheelers (2W), three-wheelers (3W), four-wheelers (4W) including cars and jeeps, Light Commercial Vehicles (LCV), Heavy Commercial Vehicles (HV), Buses, Tractors and Bicycles. These eight categories of vehicles plying on the major road can be grouped into 64 combinations. However, all these 64 combinations of vehicles could not be observed at all the six intersections. The combinations of vehicles observed at Intersection-1 are: 2w-2w, 2w-3w, 2w-4w, 2w-lcv, 2w-hcv, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-lcv, 4w-2w, 4w-3w, 4w-4w, 4w-lcv, 4w-hcv, lcv-2w, lcv-4w, Bus-4w. The combinations of vehicles observed at Intersection-2 are: 2w-2w, 2w-3w, 2w-4w, 2w-lcv, 2w-hcv, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-lcv, 4w-2w, 4w-3w, 4w-4w, 4w-lcv, 4w-hcv, lcv-2w, lcv-4w, and Bus-4w. The combinations of vehicles observed at Intersection-3 are: 2w-2w, 2w-3w, 2w-4w, 2w-lcv, 2w-hcv, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-lcv, 4w-2w, 4w-3w, 4w-4w, 4w-lcv, 4w-hcv, lcv-2w, lcv-4w, and Bus-4w. The combinations of vehicles observed at Intersection-4 are: 2w-2w, 2w-3w, 2w-4w, 2w-lcv, 2w-hcv, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-lcv, 4w-2w, 4w-3w, 4w-4w, 4w-lcv, 4w-hcv, lcv-2w, lcv-4w, and Bus-4w. The combinations of vehicles observed at Intersection-5 are: 2w-2w, 2w-3w, 2w-4w, 2w-LCV, 2w-HV, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-LCV, 4w-2w, 4w-3w, 4w-4w, 4w-LCV, 4w-HV, LCV-2W, LCV-4w, and Bus-4w. Similarly, the combinations of vehicles observed at intersection-6 are: 2w-2w, 2w-3w, 2w-4w, 2w-lcv, 2w-hcv, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-lcv, 3w-Bus, 4w-2w, 4w-3w, 4w-4w, 4w-lcv, 4w-Bus, lcv-2w, lcv-3w, lcv-4w, Bus-2w, Bus-3w, Bus-4w, hcv-3w, hcv-4w. The sample size for different combinations of the vehicle types at all the six intersections are tabulated in APPENDIX- B. In this study, critical gap of each right turning vehicle is determined using Raff's method. The accepted and rejected gaps are sorted by 0.1 s interval. For every 0.1 s interval, gaps accepted and gaps rejected are tabulated. Later, critical gap for each vehicle is determined using cumulative percentage of gap accepted and rejected at 0.1 s interval.

For each type of minor stream vehicle, the gaps accepted and the gaps rejected by the major stream vehicle combinations are calculated. For instance, for 2w in a minor stream, the gap accepted and gap rejected for different combinations of major stream vehicles are observed. Further, the accepted and rejected gaps are matched for a particular combination of major stream vehicle type and the critical gap is determined for a particular minor stream right-tuning vehicle. The critical gap is calculated for each vehicle type in the minor stream taking right turn with respect to the major stream vehicle combinations. The variation of the critical gap for each combination of major stream vehicle is analyzed. Figure 5.2 shows the critical gap of 2w for minor road right turn at Intersection-5 (Kozhikode intersection).

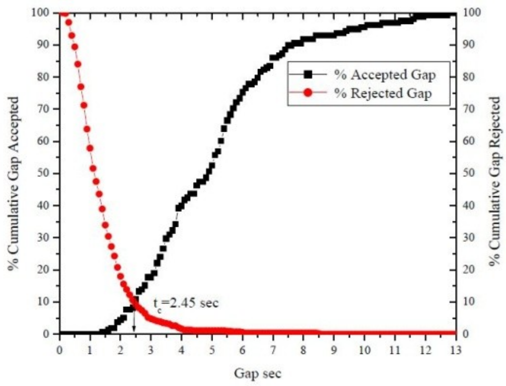


Figure 5.2 Critical gap of 2w for minor road right turn at Intersection-5

By using Raff's method, the critical gap of each vehicle type accepting the major road gap for different combinations is determined. The critical gap values of each vehicle type at each intersection are represented in the form of a radar plot and are shown in Figures 5.3 to 5.62. These figures show the critical gap values for right turning of minor stream vehicles and major stream vehicles separately. These radar plots are generated for all the six intersections by considering the observed major stream vehicle combinations and the resulting gap acceptance behaviour of each type of right turning vehicle from the minor road (Figures 5.3 to 5.11 for Intersection-1, Figures 5.21 to 5.24 for Intersection-2, Figures 5.30 to 5.33 for Intersection-3, Figures 5.38 to 5.40 for Intersection-4, Figures 5.43 to 5.46 for Intersection-5, and Figures 5.51 to 5.56 for Intersection-6). In all these radar plots, each radial line represents a major stream vehicle type combination and the scale represents the critical gap in seconds. Similarly, radar plots are generated for all the six intersections by considering the observed major stream vehicle combinations and the resulting gap acceptance behaviour of each type of right turning vehicle from the major road. (Figures 5.12 to 5.20 for Intersection-1, Figures 5.25 to 5.29 for Intersection-2, Figures 5.34 to 5.37 for Intersection-3, Figures 5.41 to 5.42 for Intersection-4, Figures 5.47 to 5.50 for Intersection-5, and Figures 5.57 to 5.62 for Intersection-6).

At Intersection-1, for the minor stream right turning 2w as the subject vehicle, the critical gap will be least for 2w-2w combination. As the size of the vehicle combination increases, the distance required to cross, in other words, the gap length also increases resulting in higher critical gaps for large vehicle combinations. It is important to note here that, the size of the following vehicle plays a major role in the gap acceptance behaviour of the right turning subject vehicles. For the right turning minor stream vehicles, with increase in size of the vehicle, the critical gap increases. Smaller sized vehicles tend to accept the smaller gaps whereas large size vehicles require higher gaps to perform the right turning maneuver.

When 2w is the subject vehicle, irrespective of the major stream vehicle combinations, the average critical gaps for a minor stream right turning 2w is observed to be 3.3 s. Similarly for 3w, 4w, LCV, Tractor and bus the average critical gap is observed to be 3.21 s, 3.74 s 3.8 s, 3.96 s and 4.1 s respectively as shown in the Figures 5.3 to 5.8. It can be seen that irrespective of the vehicle combination plying on the major road, the critical gaps increases with increase in size of the vehicle. It is important to note here that, the above trends may not be observed at some of the approaches at Intersection-1 due to non-availability of complete set of vehicle combinations and also the required number for each combination. Similar trends are observed at other approaches of Intersection-1. Similar kind of analysis is performed at other five intersections considered in this study.

Similar trends as that of NB approach of Intersection-1 could be observed at all the approaches of remaining five intersections considered in this study.

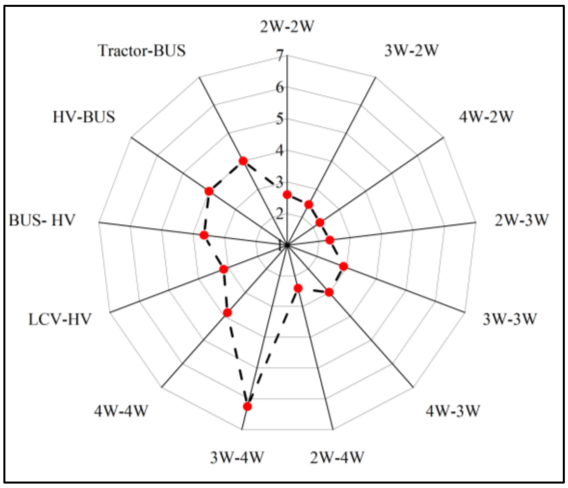


Figure 5.3 Critical gap in seconds for 2w in the minor leg (NB) for Intersection-1

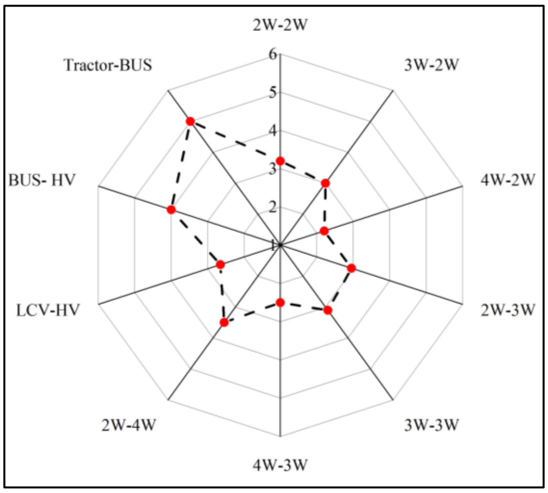
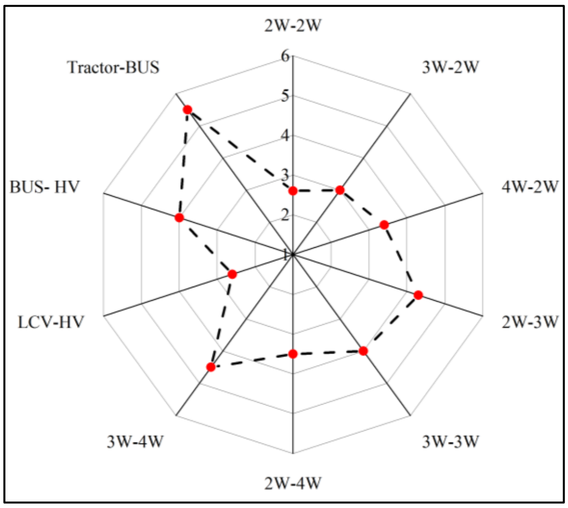


Figure 5.4 Critical gap in seconds for 3w in the minor leg (NB) for Intersection-1



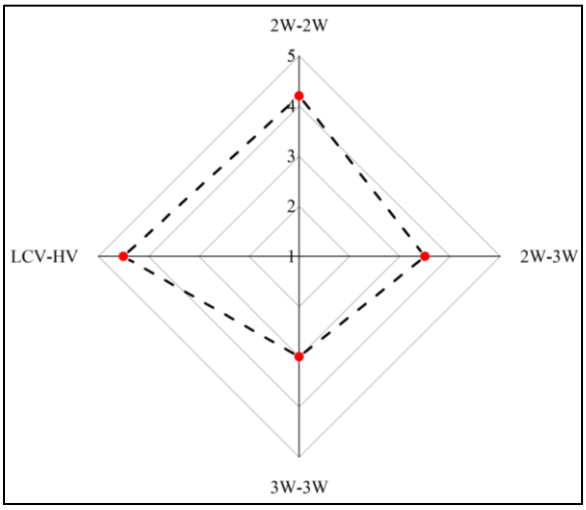
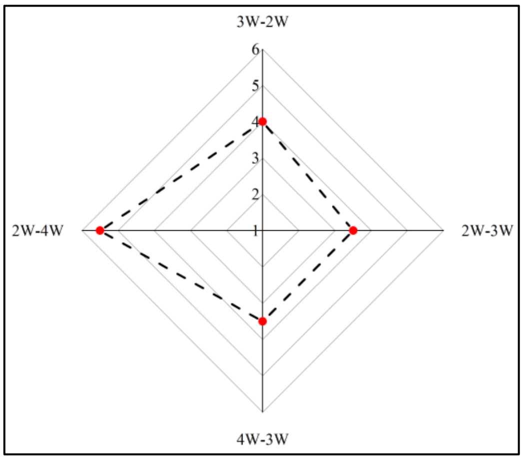


Figure 5.5 Critical gap in seconds for 4w in the minor leg (NB) for Intersection-1

Figure 5.6 Critical gap in seconds for LCV in the minor leg (NB) for Intersection-1



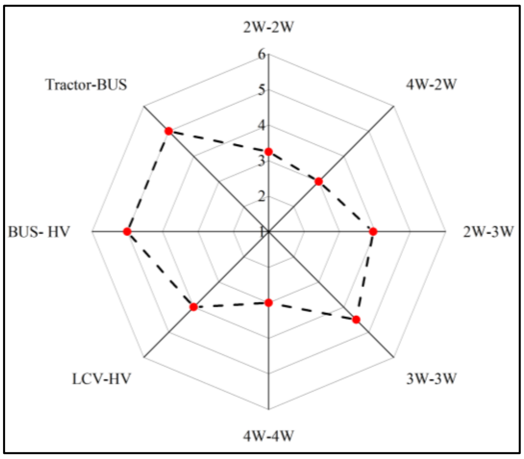


Figure 5.7 Critical gap in seconds for Bus in the minor leg (NB) for Intersection-1

Figure 5.8 Critical gap in seconds for Tractor in the minor leg (NB) for Intersection-1

From the above plots as shown in the Figures 5.3 to 5.8, for 2-wheelers, maximum critical gap is available for the combination of 3w-4w and minimum for 4w-2w combination. The sample size observed for 3-wheelers is relatively less, i.e., 22, which resulted in higher critical gaps. For 4-wheelers the maximum critical gap is available for the combinations related to the tractors and minimum for 2w-2w combination. The critical gap for buses is higher than that of the tractors.

From the Figures 5.9 to 5.11, for 2w, higher critical gap values are observed for 3w-4w combination, whereas, lower are observed for 4w-4w combination, as the sample size is inadequate. For 2w-2w combination, minimum critical gap values are observed for 2w. For

3w, the minimum critical gap is observed for 2w-2w combination. For 4w, lower critical gap is observed for 2w-2w combination and higher for Bus-HV combination.

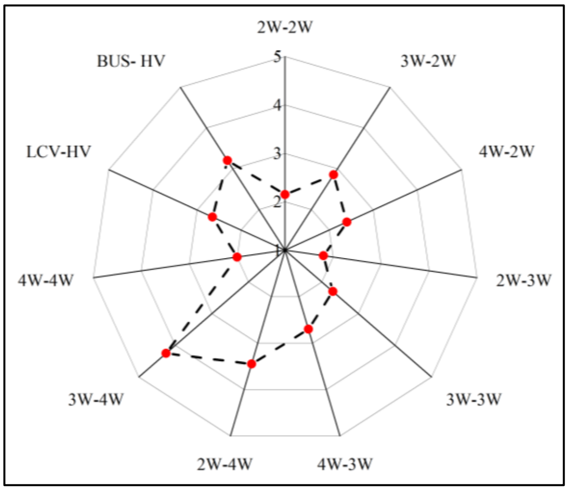


Figure 5.9 Critical gap in seconds for 2w in the minor leg (SB) for Intersection-1

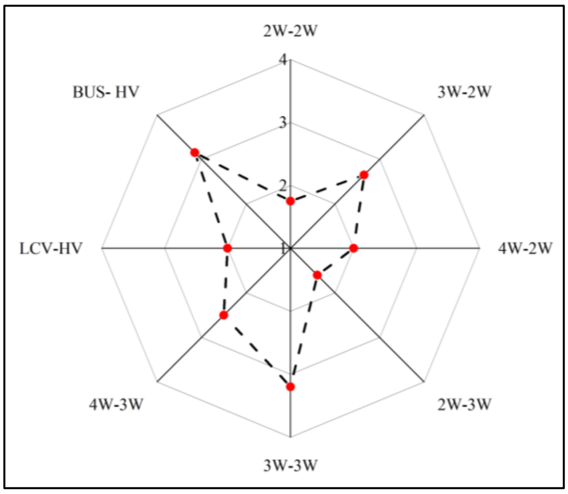


Figure 5.10 Critical gap in seconds for 3w in the minor leg (SB) for Intersection-1

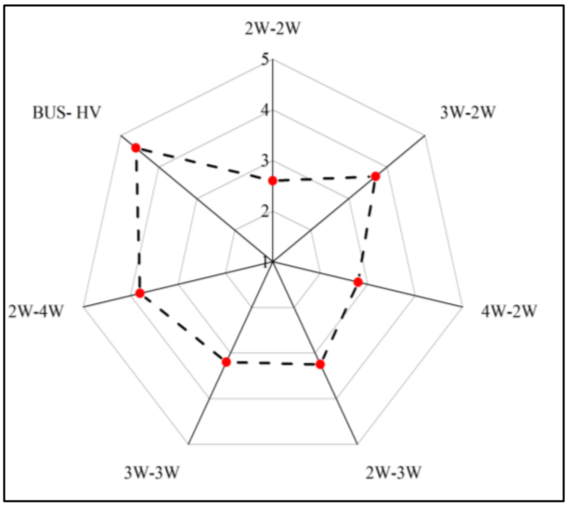


Figure 5.11 Critical gap in seconds for 4w in minor leg (SB) for Intersection-1

From Figures 5.12 to 5.17, it can be seen that, for 2w lower critical gap is observed for 3w-4w combination, since the sample size is inadequate. But, for 2w-2w combination the sample size is adequate, hence less critical gap value is observed and higher values are obtained for the combinations of Bus-HV. For 3w, minimum critical gap is observed for 2w-2w combination. For 4w, higher critical gap values are obtained for Bus-HV and less for 2w-2w combination. For other vehicle types the sample size is inadequate.

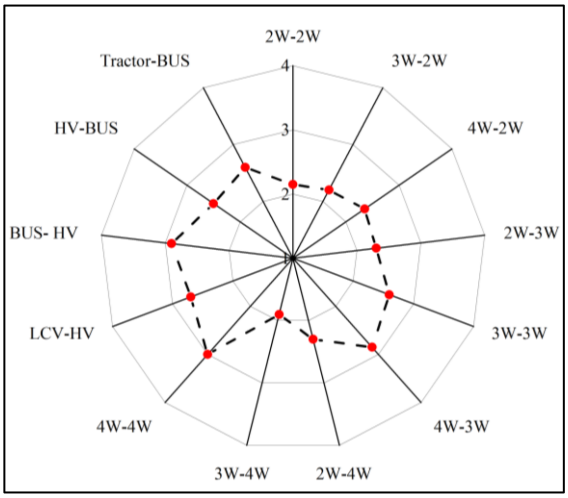


Figure 5.12 Critical gap in seconds for 2w in the major leg (EB) for Intersection-1

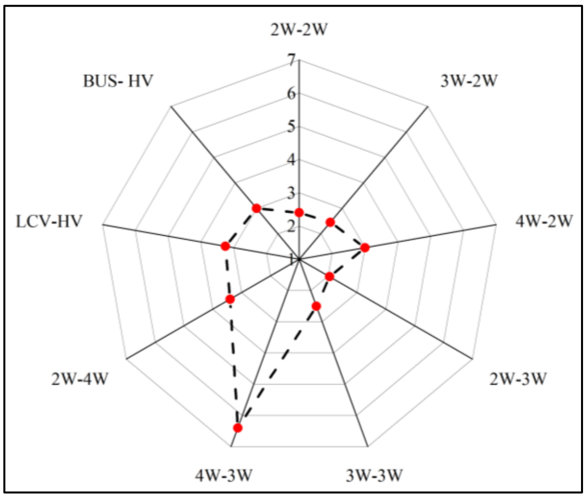


Figure 5.13 Critical gap in seconds for 3w in the major leg (EB) for Intersection-1

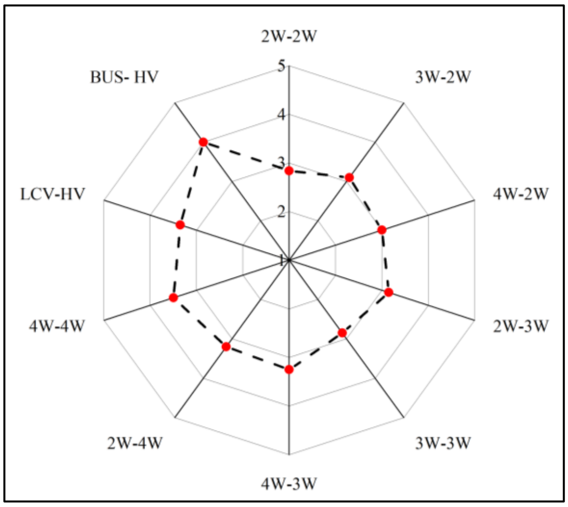


Figure 5.14 Critical gap in seconds for 4w in the major leg (EB) for Intersection-1

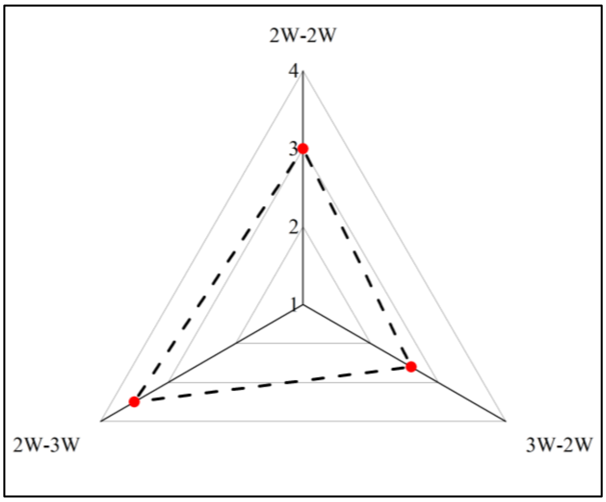


Figure 5.15 Critical gap in seconds for LCV in the major leg (EB) for Intersection-1

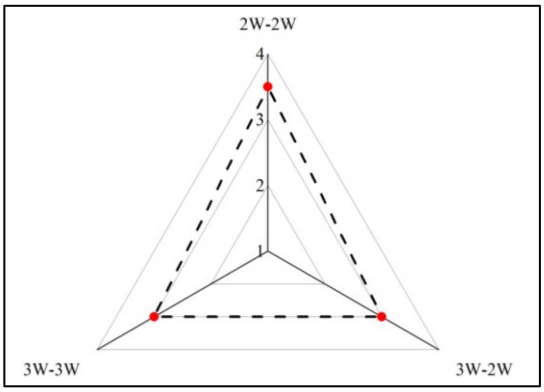


Figure 5.16 Critical gap in seconds for Bus in the major leg (EB) for Intersection-1

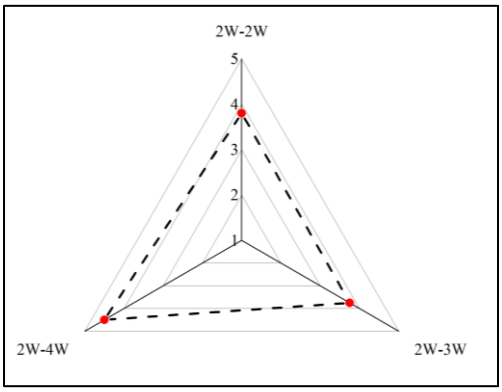


Figure 5.17 Critical gap in seconds for Tractor in the major leg (EB) for Intersection-1

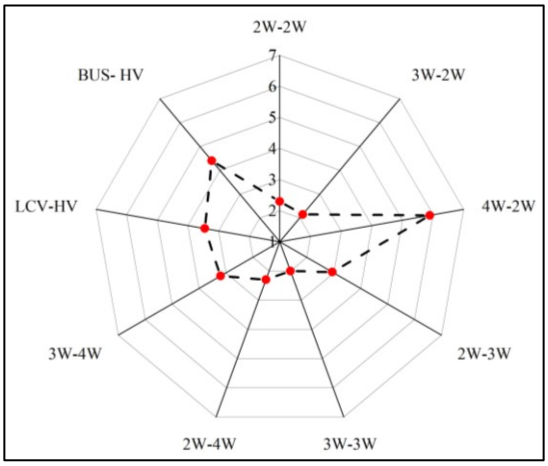


Figure 5.18 Critical gap in seconds for 2w in the major leg (WB) for Intersection-1

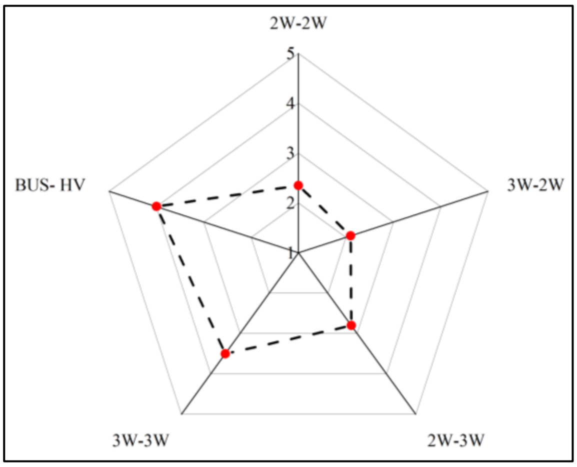


Figure 5.19 Critical gap in seconds for 3w in the major leg (WB) for Intersection-1

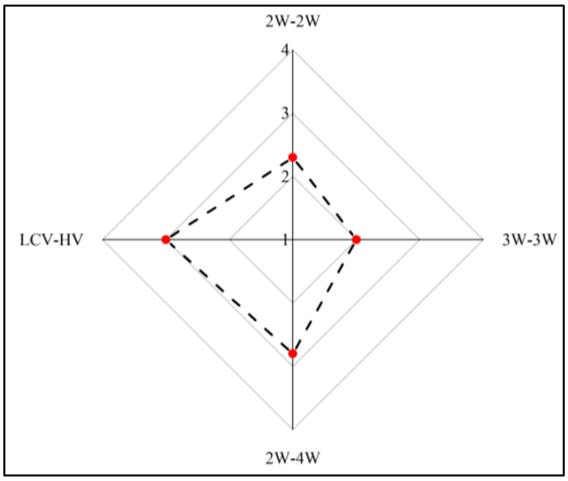


Figure 5.20 Critical gap in seconds for 4w in major leg (WB) for Intersection-1

From the Figures 5.18 to 5.20, it can be seen that, for 2w less critical gap is observed for 2w-2w combination and higher for 4w-2w combination. For 3w, higher critical gap is observed for Bus-HV, whereas less for 3w-2w combination. For 4w, higher critical gap is observed for LCV-HV, whereas less for 3w-3w combination.

For Intersection-2, the geometrics of the intersection are same as that of Intersection-1. From the Figures 5.21 and 5.22, it can be seen that, for 2w as the subject vehicle, lower critical gap is observed for 3w-2w combination and higher for LCV-HV combination. For other vehicle types the sample size is observed to be inadequate. Similar procedure is followed for the right turning vehicles from the SB minor stream.

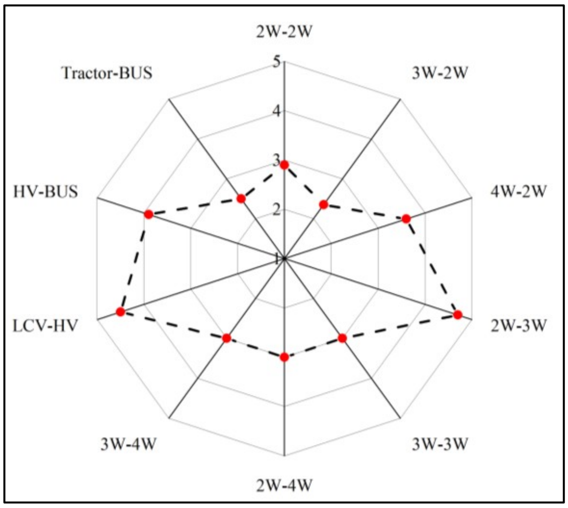


Figure 5.21 Critical gap in seconds for 2w in the minor leg (NB) for Intersection-2

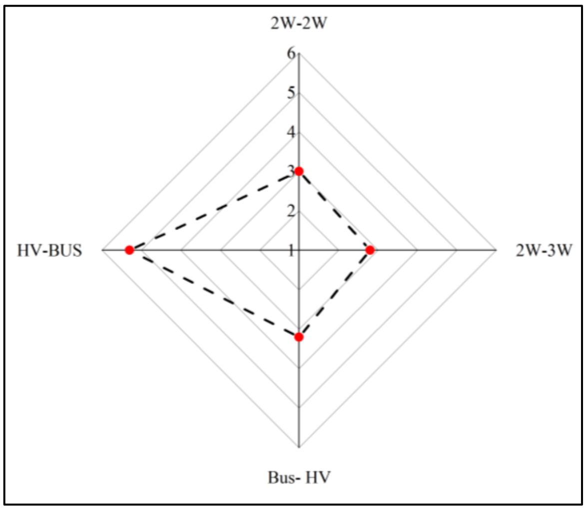


Figure 5.22 Critical gap in seconds for 4w in in the minor leg (NB) for Intersection-2

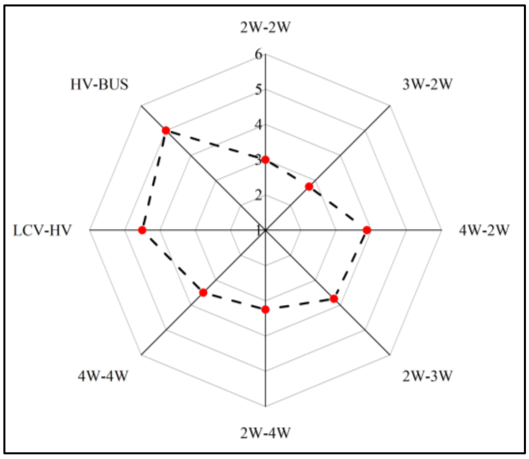


Figure 5.23 Critical gap in seconds for 2w in the minor leg (SB) for Intersection-2

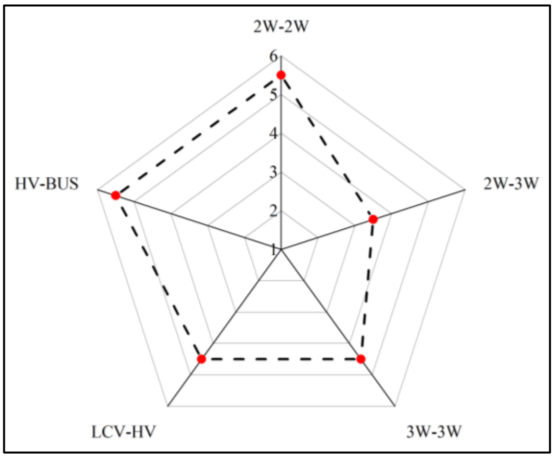


Figure 5.24 Critical gap in seconds for 3w in the minor leg (SB) for Intersection-2

From the Figures 5.23 and 5.24, it can be seen that, for 2w as the subject vehicle, lower critical gap is observed for 2w-2w combination and higher for HV-Bus combination. For 3w, lower critical gap is observed for 2w-3w combination and higher for 2w-2w combination.

The critical gap analysis for right turning vehicles from EB major stream is shown in Figures 5.25 to 5.27. For 2w, higher critical gap value is obtained for HV-Bus and lower for 2w-2w combination. For 3w, higher critical gap value is obtained for HV-Bus and lower for 2w-3w combination.

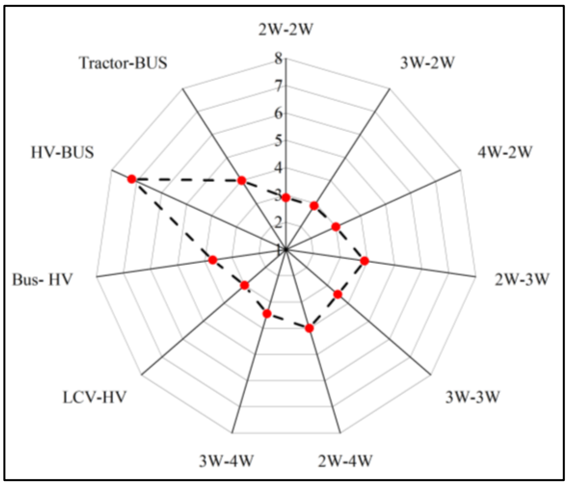


Figure 5.25 Critical gap in seconds for 2w in the major leg (EB) for Intersection-2

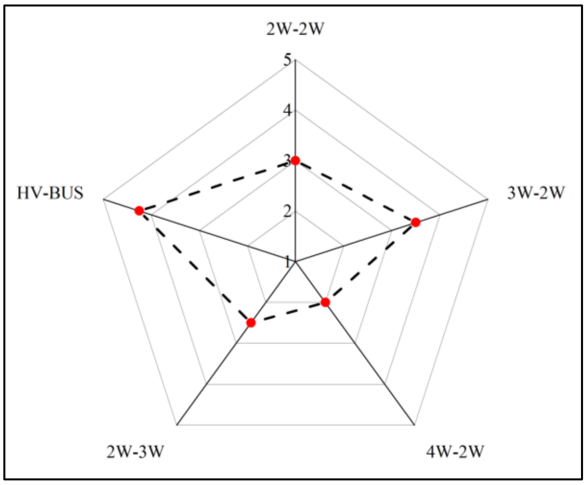


Figure 5.26 Critical gap in seconds for 3w in the major leg (EB) for Intersection-2

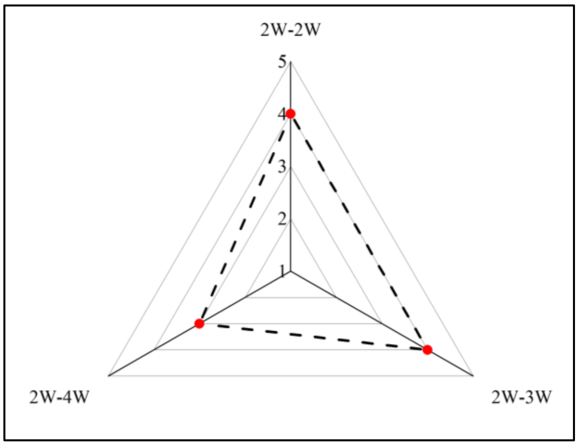


Figure 5.27 Critical gap in seconds for 4w in the major leg (EB) for Intersection-2

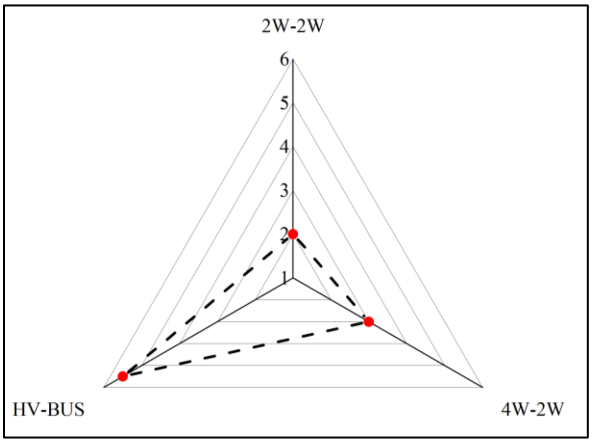


Figure 5.28 Critical gap in seconds for 2w in the major leg (WB) for Intersection-2

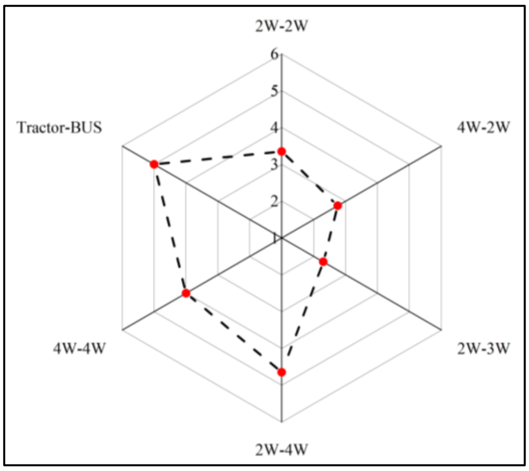


Figure 5.29 Critical gap in seconds for 3w in the major leg (WB) for Intersection-2

Similar procedure is followed for the right turning vehicles from WB major stream vehicles. The critical gaps for different vehicle combinations are shown in Figures 5.28 and 5.29. For 2w, higher critical gap value is obtained for HV-Bus and lower for 2w-2w combination. For 3w, higher critical gap value is obtained for 2w-4w and lower for 4w-2w combination.

For Intersection-3, the critical gap values for minor leg right turning vehicles with major stream combinations are shown in the Figures 5.30 to 5.33. It can be seen that, for 2w as the subject vehicle for NB, lower critical gap is observed for 3w-2w combination and higher for 2w-3w combination. For 4w, lower critical gap is observed for 3w-2w combination and higher for 2w-2w combination.

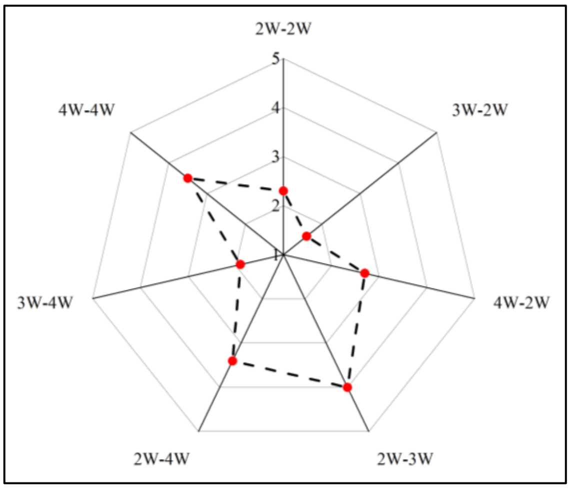


Figure 5.30 Critical gap in seconds for 2w in the minor leg (NB) for Intersection-3

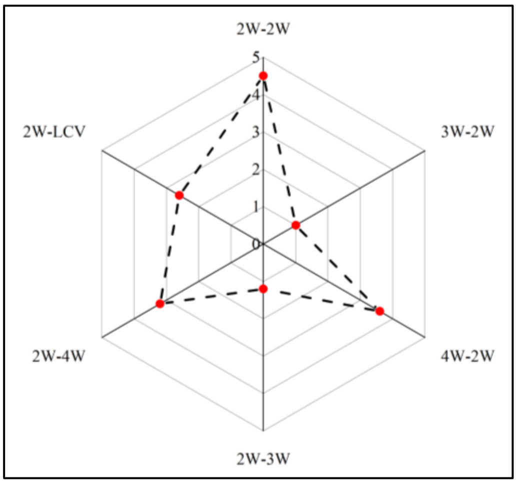


Figure 5.31 Critical gap in seconds for 4w in the minor leg (NB) for Intersection-3

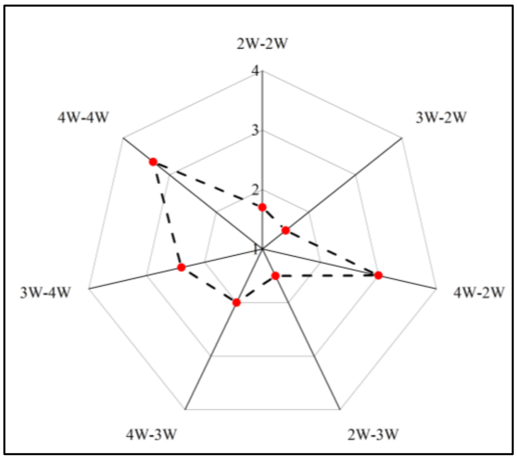


Figure 5.32 Critical gap in seconds for 2w in the minor leg (SB) for Intersection-3

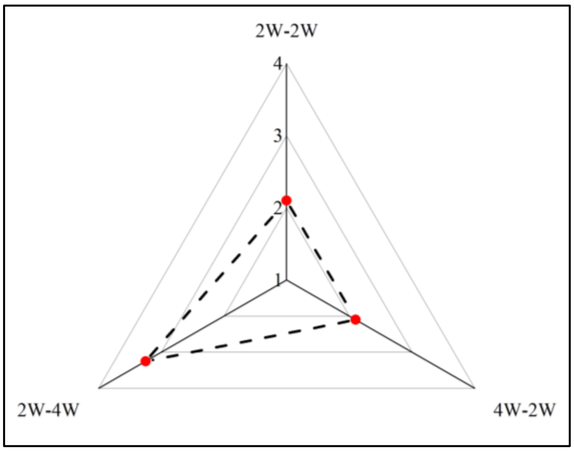


Figure 5.33 Critical gap in seconds for 4w in the minor leg (SB) for Intersection-3

The critical gap values for major leg right turning vehicles with major stream combinations are shown in the Figures 5.34 to 5.37. It can be seen that, for 2w as the subject vehicle for EB, lower critical gap is observed for 3w-4w combination and higher for 4w-4w combination. For 4w, lower critical gap is observed for 4w-2w combination and higher for 2w-2w combination. For WB, 2w as the subject vehicle, lower critical gap is observed for 2w-3w combination and higher for 4w-4w combination. For 4w, lower critical gap is observed for 4w-2w combination and higher for 2w-4w combination.

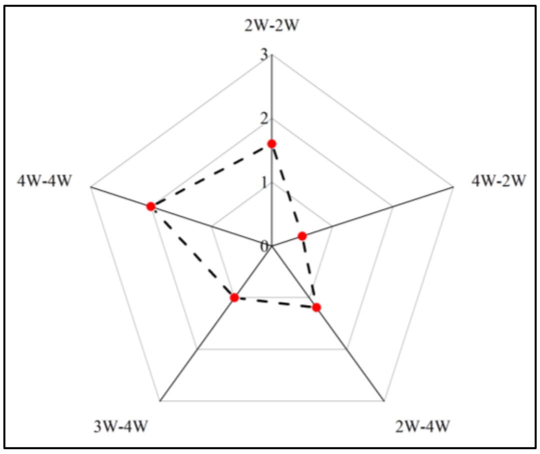


Figure 5.34 Critical gap in seconds for 2w in the major leg (EB) for Intersection-3

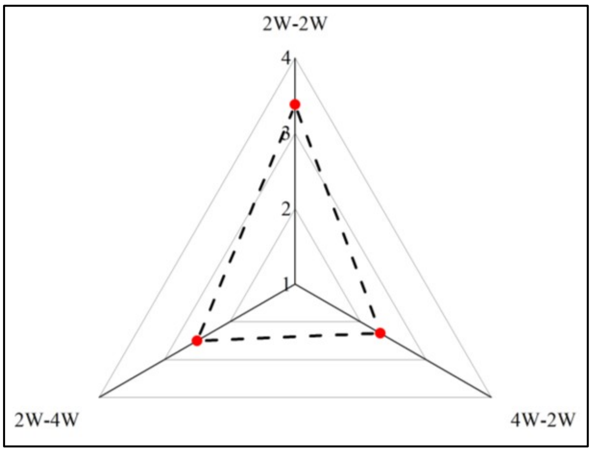


Figure 5.35 Critical gap in seconds for 4w in the major leg (EB) for Intersection-3

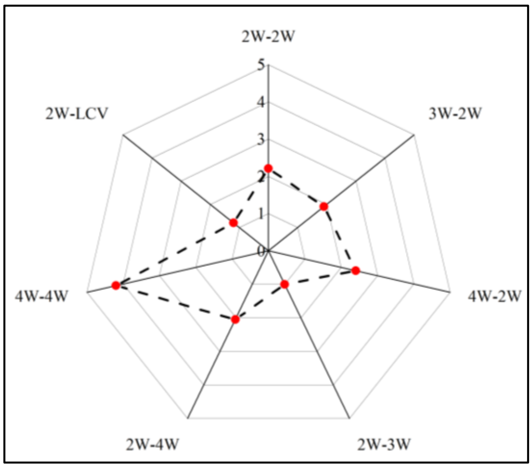


Figure 5.36 Critical gap in seconds for 2w in the major leg (WB) for Intersection-3

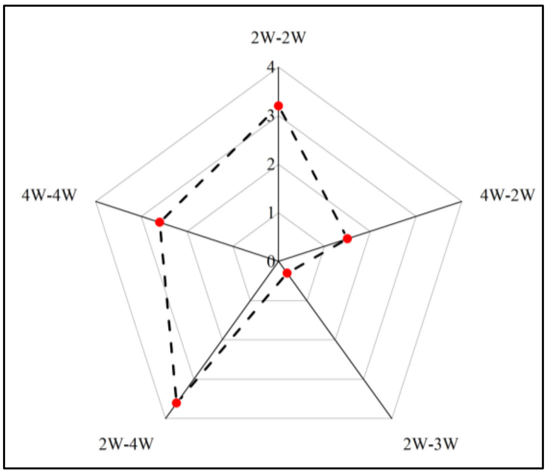


Figure 5.37 Critical gap in seconds for 4w in the major leg (WB) for Intersection-3

For three-legged intersection the critical gap values for the entire minor right turning vehicles with respect to major stream combinations are shown in Figures 5.38 to 5.62. Total traffic is divided into seven categories: two-wheelers (2w), three-wheelers (3w), four-wheelers (4w) including cars and jeeps, Light Commercial Vehicles (LCV), Heavy Commercial Vehicles (HV), Buses, and Bicycles. These seven categories of vehicles plying on the major road can be grouped into 49 combinations. Similar to the four-legged intersections, all the possible 49 combinations could not be seen at all the approaches of intersections. The combinations of vehicles observed at Intersection-4 are: 2w-2w, 2w-4w, 4w-2w, 4w-3w, 4w-4w, 4w-Bicycle, LCV-4w, Bus-4w, and HV-4w. The combinations of vehicles observed at Intersection-5 are: 2w-2w, 2w-3w, 2w-4w, 2w-LCV, 2w-HV, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-LCV, 4w-2w, 4w-3w, 4w-4w, 4w-LCV, 4w-HV, LCV-2W, LCV-4w, and Bus-4w. Similarly, the combinations of vehicles observed at Intersection-6 are: 2w-2w, 2w-3w, 2w-4w, 2w-LCV, 2w-HV, 2w-Bus, 3w-2w, 3w-3w, 3w-4w, 3w-LCV, 3w-Bus, 4w-2w, 4w-3w, 4w-4w, 4w-LCV, 4w-Bus, LCV-2w, LCV-3w, LCV-4w, Bus-2w, Bus-3w, Bus-4w, HV-3w and HV-4w. By using Raff's method, the critical gap of each vehicle type accepting the major road gap for different combinations is determined. The critical gap values of each vehicle type at each intersection are shown in Figures 5.38 to 5.62. These figures show the critical gap values for right turning of minor stream vehicles and major stream vehicles separately. These radar plots are generated for all the intersections by considering the observed major stream vehicle combinations and the resulting gap acceptance behaviour of each type of right turning vehicle from the minor road (Figures 5.38 to 5.40 for Intersection-4, Figures 5.43 to 5.46 for Intersection-5, and Figures 5.51 to 5.56 for Intersection-6). Similarly, radar plots are generated for all the intersections by considering the observed major stream vehicle combinations and the resulting gap acceptance behaviour of each type of right turning vehicle from the major road (Figures 5.41 to 5.42 for Intersection-4, Figures 5.47 to 5.50 for Intersection-5, and Figures 5.57 to 5.62 for Intersection-6). For example, as shown in Figure 43, when the subject vehicle 2w is approaching towards the intersection from the minor road and turning right onto the major road, the critical gap for 2w-2w combination on major stream is observed as 2.6 s, and the critical gap for 4w-lCV is 7 s. Similarly, the critical gap of each vehicle taking right turn from minor road with respect to major stream combinations and major stream right turn vehicles with respect to major stream through vehicles combinations are calculated. The critical gaps obtained from all the intersections are compared. The variation of the critical gap for each combination of major stream vehicle is analyzed.

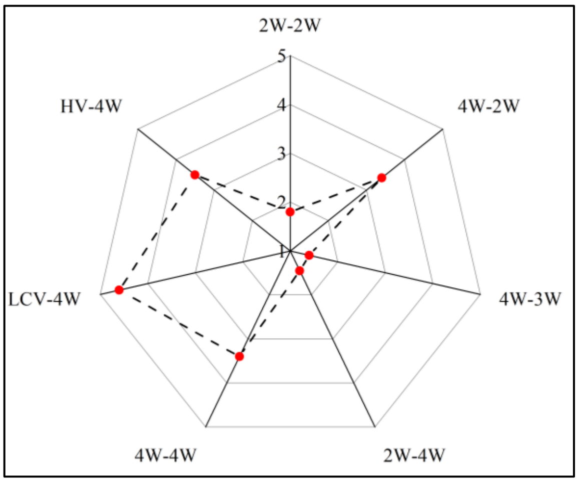


Figure 5.38 Critical gap in seconds for 2w in the minor leg (SB) for Intersection-4

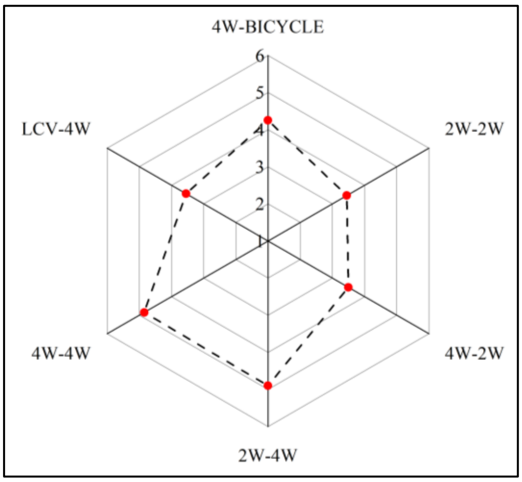


Figure 5.39 Critical gap in seconds for 4w in the minor leg (SB) for Intersection-4

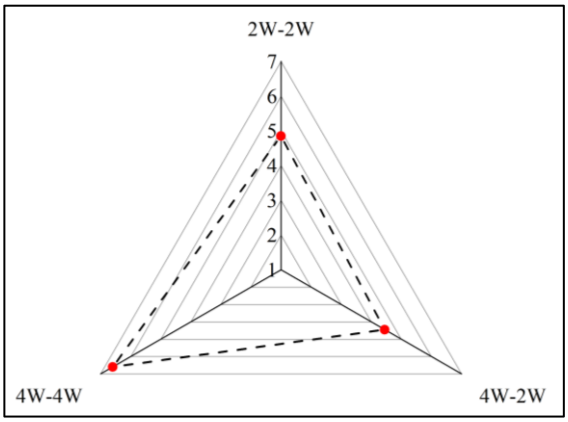


Figure 5.40 Critical gap in seconds for Bicycle in the minor leg (SB) for Intersection-4

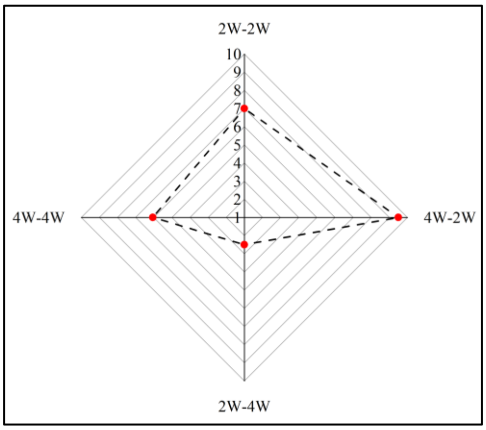


Figure 5.41 Critical gap in seconds for 2w in the major leg (WB) for Intersection-4

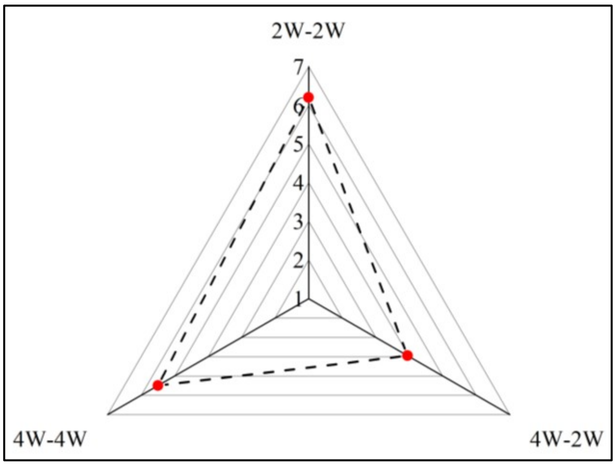


Figure 5.42 Critical gap in seconds for 4w in the major leg (WB) for Intersection-4

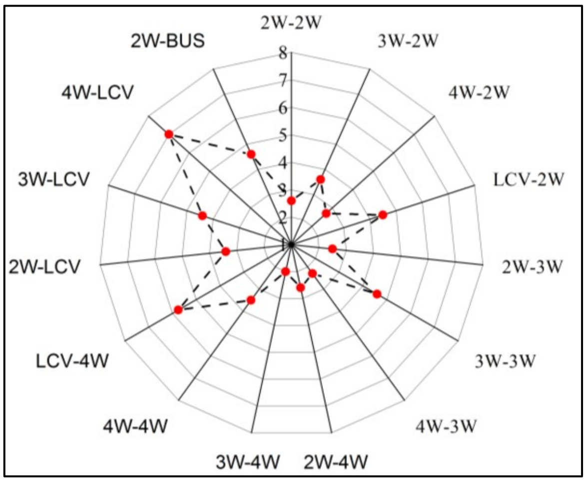


Figure 5.43 Critical gap in seconds for 2w in the minor leg (SB) for Intersection-5

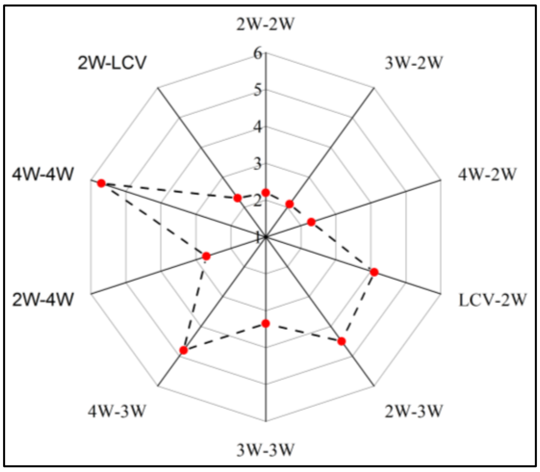


Figure 5.44 Critical gap in seconds for 3w in the minor leg (SB) for Intersection-5

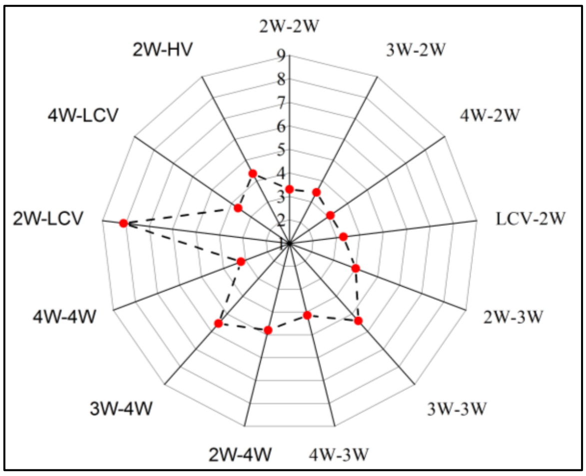


Figure 5.45 Critical gap in seconds for 4w in in the minor leg (SB) for Intersection-5

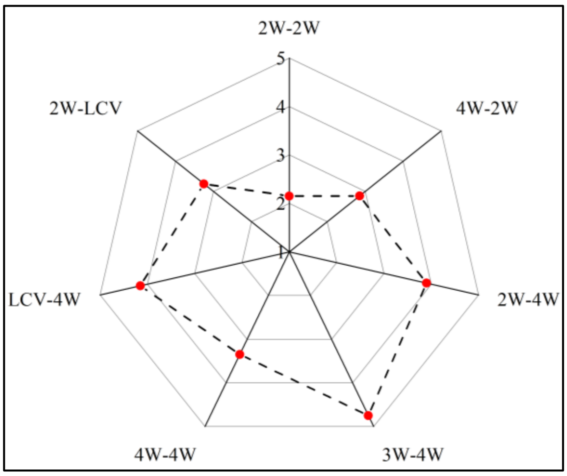


Figure 5.46 Critical gap in seconds for LCV in the minor leg (SB) for Intersection-5

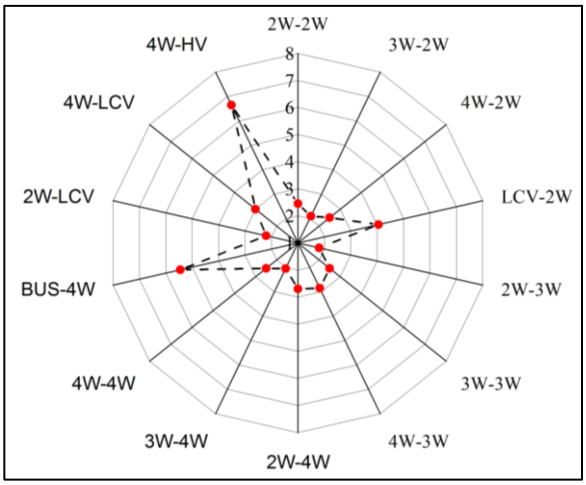


Figure 5.47 Critical gap in seconds for 2w in the major leg (WB) for Intersection-5

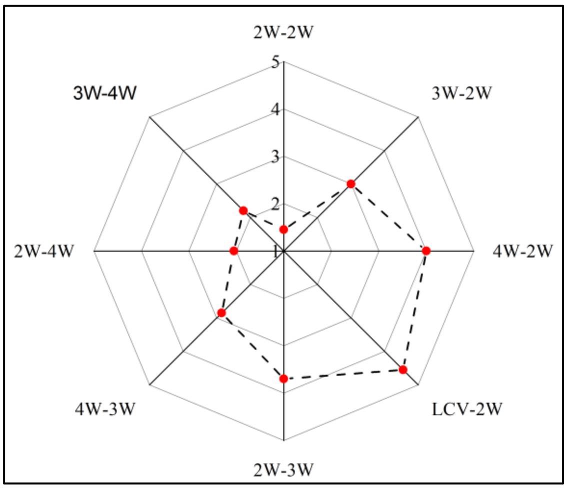


Figure 5.48 Critical gap in seconds for 3w in the major leg (WB) for Intersection-5

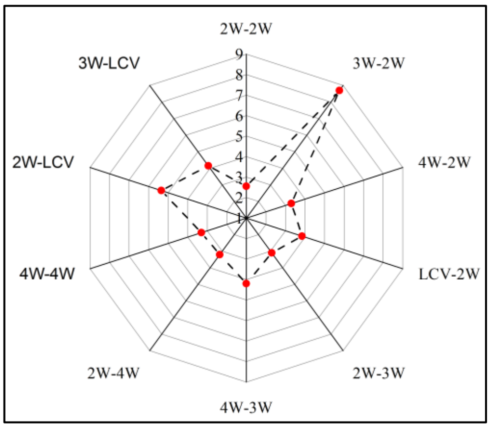


Figure 5.49 Critical gap in seconds for 4w in the major leg (WB) for Intersection-5

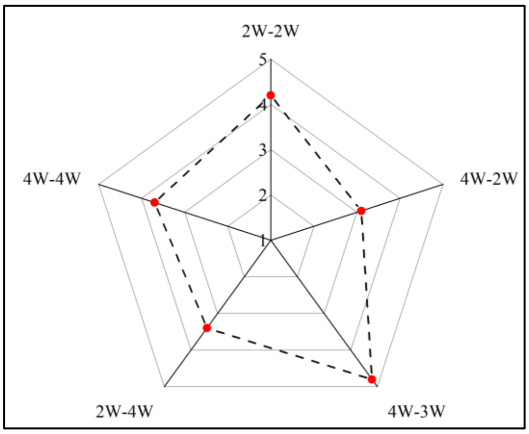


Figure 5.50 Critical gap in seconds for LCV in the major leg (WB) for Intersection-5

For Intersection-5, 2w, 3w, 4w and LCV are the subject vehicles accepting the gaps for different combinations in the major stream vehicles. From Figures 5.38 to 5.50, it is observed that for 2w and 3w as the subject vehicle type with different major stream combinations consisting of 2w, 3w, 4w as either lead or following vehicle are showing less critical gap values because smaller sized vehicles are forced to accept the available gap in the major stream as compared to other vehicle combinations consisting of LCV, HCV and buses as the lead or the following vehicle types. It is also observed that irrespective of subject vehicle, the combination with LCV as the leading vehicle or following vehicle are showing the higher critical gap values. When 4w is the following vehicle, this combination also shows higher critical gap. Due to lower acceleration characteristics of LCV, the critical gap values are observed to be higher. Similar observation can be made for other larger vehicles including HCV and Buses.

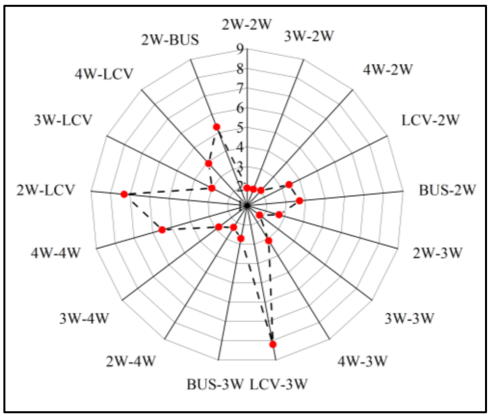


Figure 5.51 Critical gap in seconds for 2w In the minor leg (SB) for Intersection-6

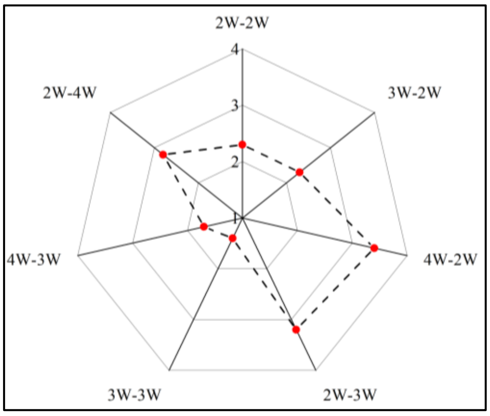


Figure 5.52 Critical gap in seconds for 3w in the minor leg (SB) for Intersection-6

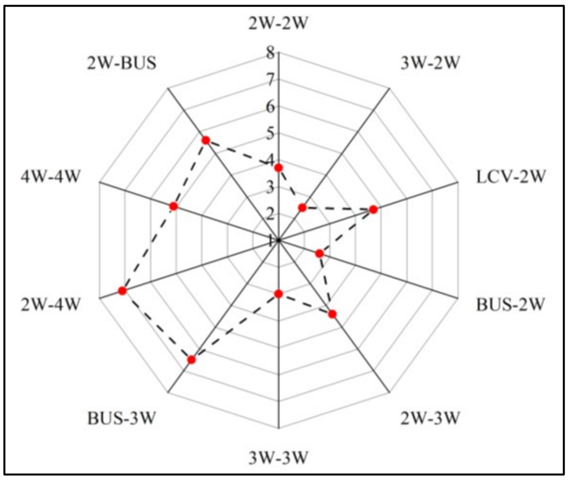


Figure 5.53 Critical gap in seconds for 4w in the minor leg (SB) for Intersection-6

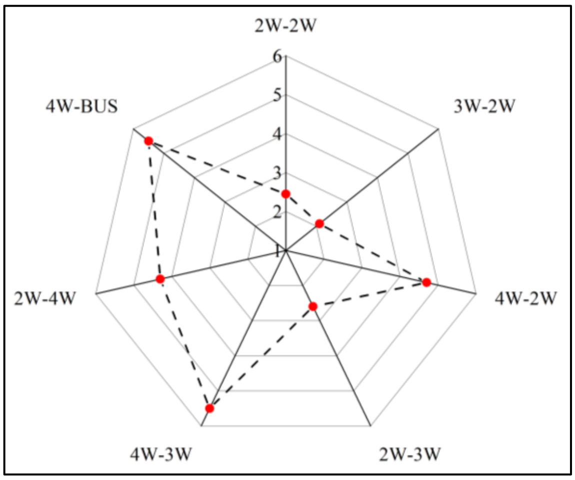


Figure 5.54 Critical gap in seconds for LCV in the minor leg (SB) for Intersection-6

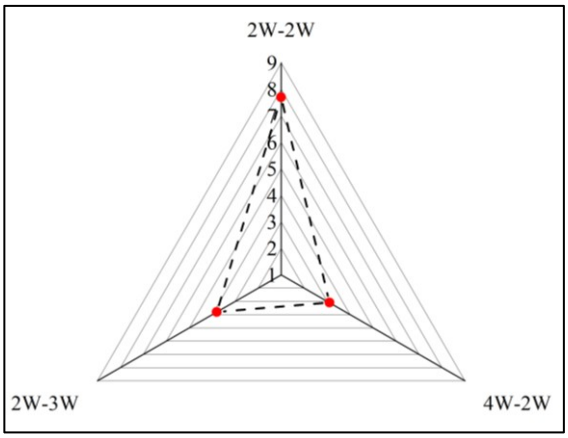


Figure 5.55 Critical gap in seconds for Bus in the minor leg (SB) for Intersection-6

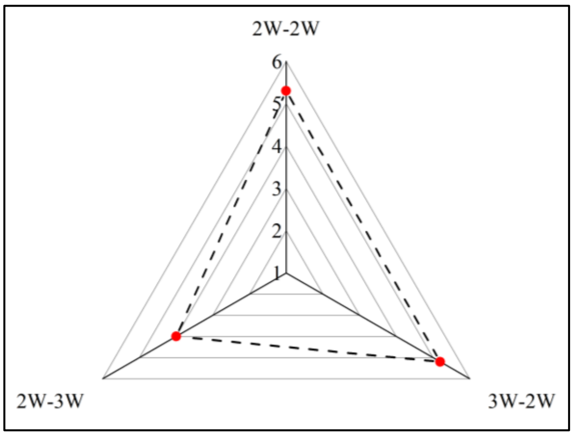


Figure 5.56 Critical gap in seconds for HV in the minor leg (SB) for Intersection-6

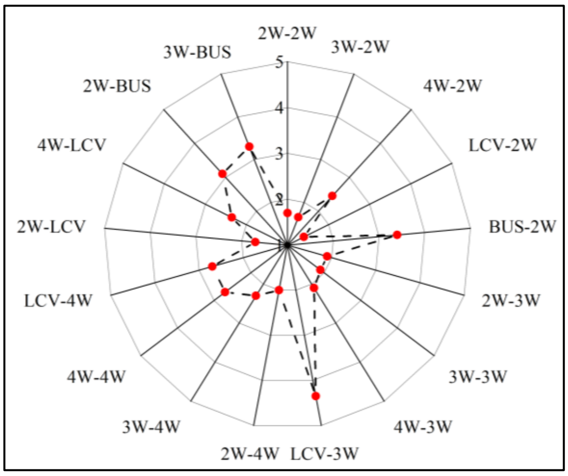


Figure 5.57 Critical gap in seconds for 2w in the major leg (WB) for Intersection-6

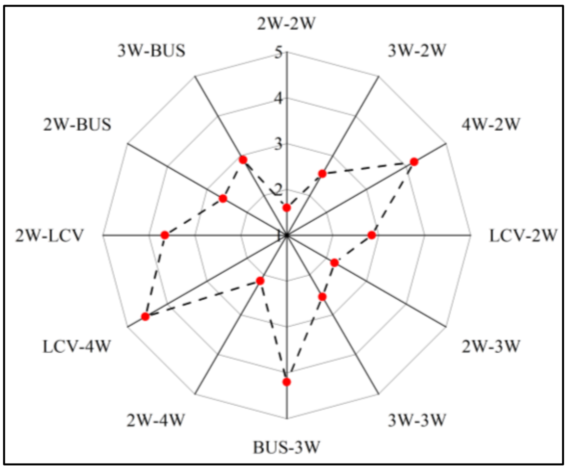


Figure 5.58 Critical gap in seconds for 3w in the major leg (WB) for Intersection-6

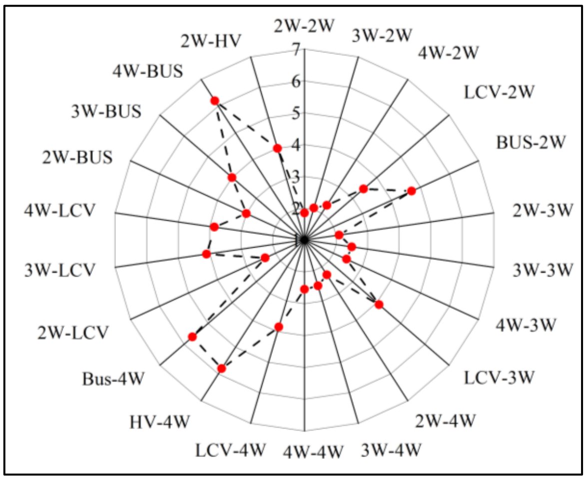


Figure 5.59 Critical gap in seconds for 4w in the major leg (WB) for Intersection-6

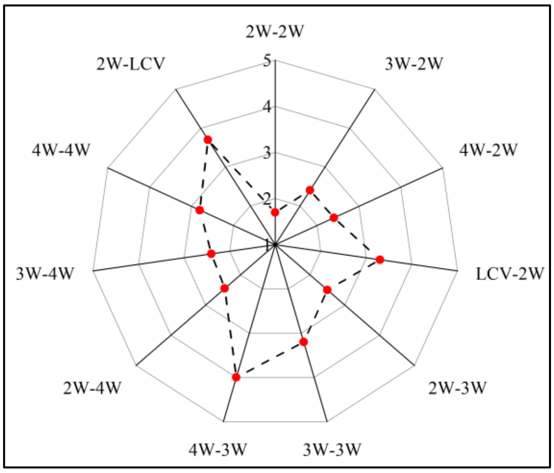


Figure 5.60 Critical gap in seconds for LCV in the major leg (WB) for Intersection-6

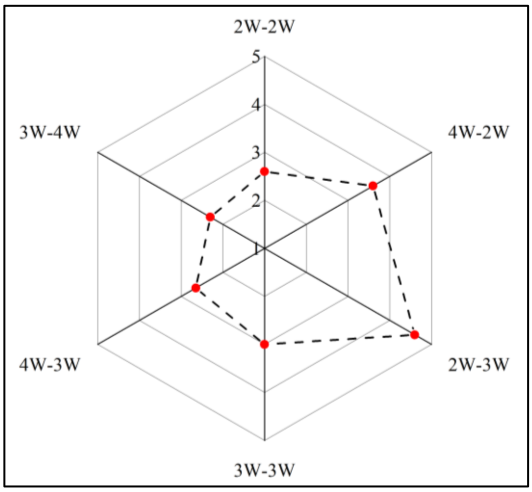


Figure 5.61 Critical gap in seconds for Bus in the major leg (WB) for Intersection-6

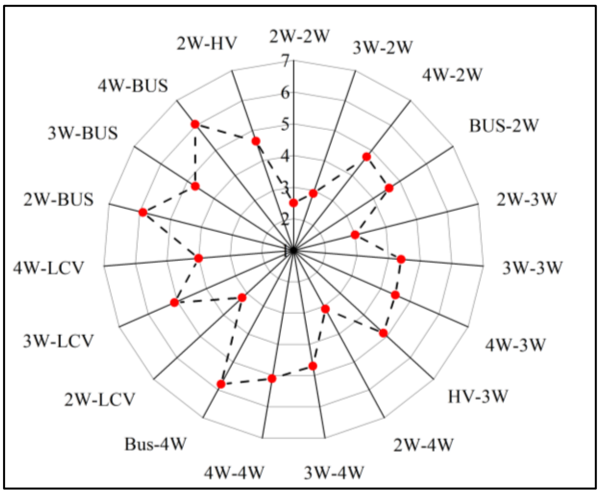


Figure 5.62 Critical gap in seconds for HV in the major leg (WB) for Intersection-6

At Intersection-6, 2w, 3w, 4w, LCV, HCV and Buses are the subject vehicles accepting the gaps for different combinations in the major stream as shown in the above Figures 5.51 to 5.62. The presence of different sized vehicles adversely affects the performance of the intersection. Larger vehicles require more time to maneuver because of lower acceleration and speed capabilities whereas 2w and 3w utilize smaller gaps available in the traffic stream. It is important to note that even though the gap acceptance depends very much on the dimensional and performance characteristics of the subject vehicle, the driver behaviour also plays a significant role in accepting or rejecting the gap. The critical gap analysis is performed for each right turning vehicle from both the minor and the major roads. The critical gap is more for LCV, HCV and Bus as the subject vehicles, which shows that the presence of large sized vehicles in the minor stream will lead to reduction in the capacity of the intersection.

From the above Figures 5.51 to 5.62, it is observed that irrespective of subject vehicle, the combination with large vehicle size irrespective of whether it is a leading vehicle or a following vehicle are showing higher critical gaps.

For Interersection-5, the critical gap for the minor road right turning traffic varies between 2 to 8.1 s with major road combinations. Whereas, the critical gap for the major road right turning traffic varies between 1.45 to 8.7 s with major road combinations. Similarly, for Intersection-6, critical gap for the minor and major road right turning traffic varies between 1.4 to 8.2 s and 1.4 to 6.2 s. Further, 2w-2w, 2w-3w, 3w-2w, 3w-3w combinations resulted in less critical gaps when compared with other vehicle combinations. However, the critical gap for different vehicle combinations for Intersection-5 and Intersection-6 varies between 1.45 to 8.7 s and 1.4 to 8.2 s, respectively.

After observing the critical gap of all types of vehicles and its combinations at all the three intersections, it is observed that two-wheelers and three wheelers are more aggressive than other vehicle types irrespective of vehicle combinations, therefore, their critical gap is lower than other type of vehicles. The critical gap for HCVs and buses are high because of their larger size and lower acceleration characteristics. The critical gap for four-wheelers and LCVs are moderate compared to other vehicles because they tend not to take risk as compared with two-wheelers and three-wheelers.

5.4 Summary

For each type of right turning vehicles in the minor road, gaps accepted and gaps rejected by the major road vehicle combinations are calculated. Similarly, the accepted and rejected gaps are calculated for each type of right turning vehicles in the major road and the critical gap for each vehicle with different combinations is calculated. It is observed that the critical gap of the subject vehicle taking right turn depends on the vehicle type of the major stream combination. Gap acceptance has been shown to vary with the conflicting vehicle type. If the size of the major stream vehicle is small, such as 2w and 3w, then the small sized subject vehicles tries to accept shorter gaps and the same set of subject vehicles accept higher gaps in the combinations having large sized vehicles including buses and HCVs. If the following vehicle size is small in the major traffic stream, the subject vehicle is more likely (aggressive) to accept the gap.

It is concluded that the subject vehicle from the minor and major road right turning vehicle and the conflicting vehicle type strongly influences the gap acceptance behaviour. It is also observed that the accepted gaps follows gamma distribution for the major and minor road right turning vehicles, whereas the available gaps followed lognormal distribution for the major and minor road right turning vehicles.

The gap accepting behavior of the right turning subject vehicles depends on the combinations of the major stream. For the conflicting major stream combination having large sized vehicles like buses or HCVs, the critical gap value is higher compared to other small sized vehicle type combinations. If the conflicting major stream vehicle is 2w, then the subject vehicles are more likely to accept shorter gaps. As the total data analysis is done using only the field conditions, the results obtained is totally based on field conditions. The analysis provides the effect of vehicle type on the gap acceptance and rejection behaviour.

In order to minimize the waiting time at the urban uncontrolled intersections, most of the minor stream vehicles prefer to cross the intersection during the lag period rather for waiting for sufficiently longer duration of time to identify suitable gaps within the major traffic stream. For these vehicles, there are number of gaps available for different major vehicle combinations. For every minor stream vehicle, the preferences of gaps accepted and rejected depend on the type, speed and size of the vehicle. For 2w, it is very easy to accept any gap, and 3w are found to be as aggressive as 2w. The main concern comes for cars, LCV, buses, tractors and heavy vehicles. The preferences of acceptance and rejection can be analyzed by the critical gap that has been accepted by each minor stream vehicle for each major stream vehicle combination.

CHAPTER 6

VISSIM SIMULATION

6.1 General

In the developing country like India, traffic is mixed consisting of vehicles with varied geometric characteristics and performance characteristics including varied acceleration and speeds. The time to cross the intersection depends on the static and dynamic characteristics of vehicles. Different vehicle types will take different durations to cross the intersection. Thus, the composition of the vehicles will also affect the efficiency of the intersection in terms of its capacity and Level of Service (LOS). In case of uncontrolled intersections especially under mixed traffic conditions, the conflicting traffic is high and there is a possibility of accidents. It is very difficult to analyze the uncontrolled intersection because of heterogeneous conditions and due to lack of knowledge to the drivers on the traffic rules. The mixed traffic is composed of different vehicle types with different lengths and widths. The vehicle type also affects the capacity of the intersection because of their different lengths and widths. The analysis of effect of vehicle composition on capacity of uncontrolled intersections using micro simulation is one of the important studies.

6.2 VISSIM Traffic Flow Simulation

Simulation is defined as dynamic representation of some part of real world achieved by building a computer model and moving it through time (Drew 1968). Simulation is the imitation or emulation of some real thing, state of affairs or process. It is a methodology to help achieve educational goals. The most powerful and efficacious simulations are conducted based on strong educational principles, run by expert facilitators and within the context of the objectives of a curriculum. The act of simulating something first requires that a model be developed; this model represents the key characteristics or behaviours/ functions of the selected physical or abstract system or process. The model represents the system itself, whereas the simulation represents the operation of the system over time. Simulation is used in many contexts, such as simulation of technology for performance optimization, safety engineering, testing, training, education, and video games. Often, computer experiments are used to study simulation model.

Simulation is also used with scientific modeling of natural systems or human systems to gain insight into their functioning. Simulation can be used to show the eventual real effects of alternative conditions and courses of action. Simulation is also used when the real system cannot be engaged, because it may not be accessible, or it may be dangerous or unacceptable to engage, or it is being designed but not yet built, or it may simply not exist (Sokolowski and Banks 2009). Traffic micro-simulation models are one of the latest generations of commercially available traffic models developed in recent years. It models the movements of individual vehicles travelling on road networks by using car following, lane changing and gap acceptance rules. They are becoming increasingly popular for the development and evaluation of a broad range of road traffic management and control systems.

6.3 Applications of VISSIM Simulation

For the simulation of traffic, many conventional software are available, Verkehr in staedtn simulation (VISSIM) is one of them. There are many different rules like right hand traffic rule, left hand traffic rule and lane behavior that are inbuilt in VISSIM. However, in India left hand traffic rule is followed with mixed traffic conditions where no particular lane behavior is followed. Thus, VISSIM cannot be used directly for Indian traffic conditions. In order to use VISSIM for Indian traffic conditions, a different setting is required where the parameters and traffic conditions need to changed. Following are the advantages of using VISSIM:

- VISSIM is a useful tool for both microscopic and macroscopic simulation models.
- VISSIM is able to simulate road corridors of heavily populated motorways to identify system performance, bottlenecks, and potential for improvement.
- Corridor studies on arterials with signalized and non-signalized intersections can also be done using VISSIM.
- Signal priority schemes can be analyzed for public transport within multi-modal studies.
- Traffic circulation, public transport operations, pedestrian crossings, and bicycle facilities can be modelled for various layouts of the street network and different options of vehicle detection.

6.4 Driver Behaviour Model in VISSIM

VISSIM contains number of driver behavior parameters that can affect the maneuverability of individual vehicles passing through the network (or section). The change in parameter values can cause a substantial change in simulated output. Therefore, it is important to see the applicability of the VISSIM in replicating the mixed traffic flow on a computer screen. In the calibration process, the VISSIM parameters are adjusted in such a way that it is able to reproduce the traffic flow conditions as observed in field. Sensitivity of each parameter is checked individually and in combination of other parameters and the most influencing parameters are identified.

VISSIM is mainly based on the driver behaviour model and the driver behaviour model uses the car following model and rule based algorithm for lane changing operations and lateral behaviour. VISSIM uses two car following models developed by Wiedemann on the basis of car following theories and behaviours. It consists of two different sets of parameters developed by Weidemann in 1974 and 1999.

In 1974 Wiedemann developed a model (Wiedemann 74 model) by considering only three parameters which were describing the safety distance between two vehicles, average standstill distance (ax), additive part of the safety distance (bx_add) and multiplicative part of the safety distance (bx_mult) and the default parameters are shown in the Figure 6.1. The safety distance can be calculated by using the Equations (6.1) and (6.2),

$$\text{Safety distance, } d = ax + bx \tag{6.1}$$

Where, ax = standstill distance,

$$bx = (bx\_add + bx\_mult * z) * \sqrt{V} \tag{6.2}$$

Where, 'z' is normally distributed around 0.5 with a range of 0 to 1 and a standard deviation of 0.15. The default values of bx_add and bx_mult are 2.0 and 3.0 respectively.

In 1999, Wiedemann developed another model (Wiedemann 99) by considering more number of parameters (CC parameters) based on the action and reaction of driver according to perceived traffic situations. The safe distance can be calculated by using Equation (6.3),

$$dx = CC0 + CC1 * V \tag{6.3}$$

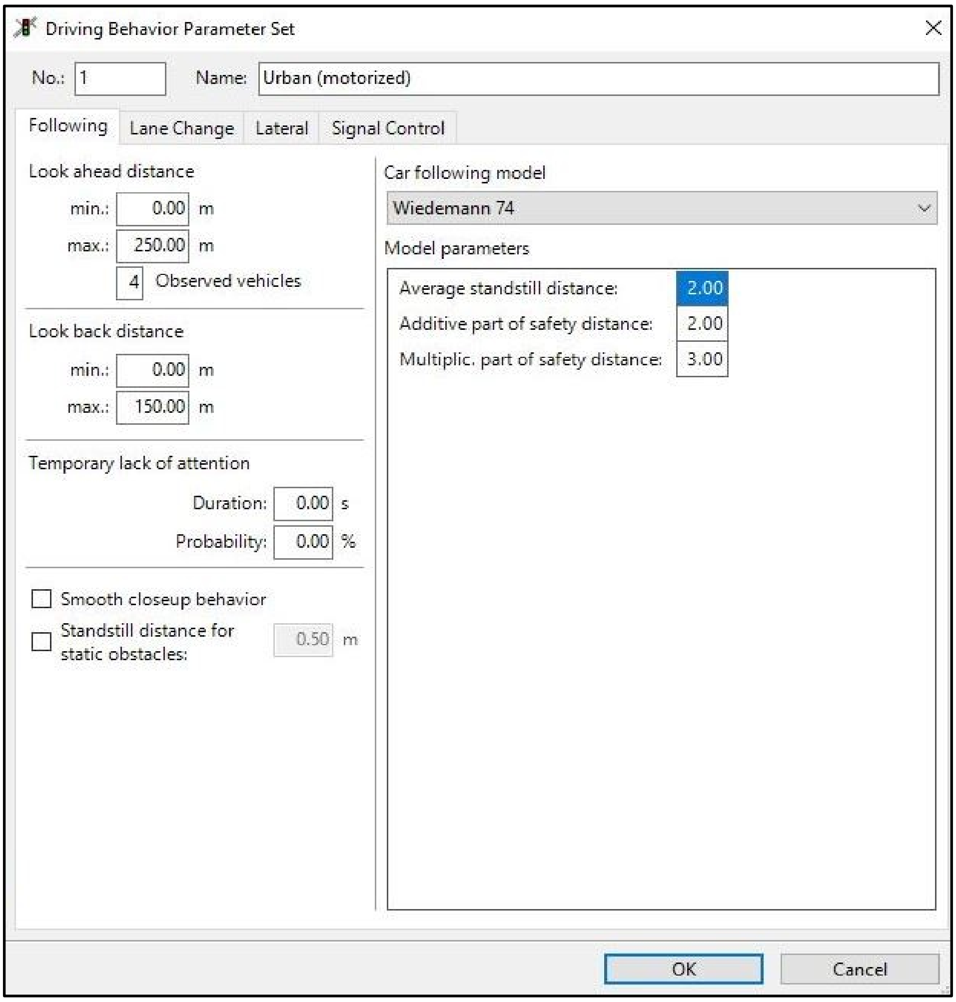


Figure 6.1 Driver behaviour parameters with default values

6.5 Simulation Parameters in VISSIM

There are many simulation parameters in VISSIM to run the simulation accurately and efficiently. Figure 6.2 show some of the simulation parameters which mostly influence the simulation and output.

a) Simulation period: It is the time period for which simulation needs to be run. It will depend on the number of hours of data taken for analysis of field data.
b) Simulation resolution: It is the number of time steps taken for simulation to run one second from the simulation period. If the time period is 3600 s and simulation revolution is 10, it will take 360 s (3600/10) to get the simulation output. It ranges from 1 to 20 time steps/ simulation seconds.

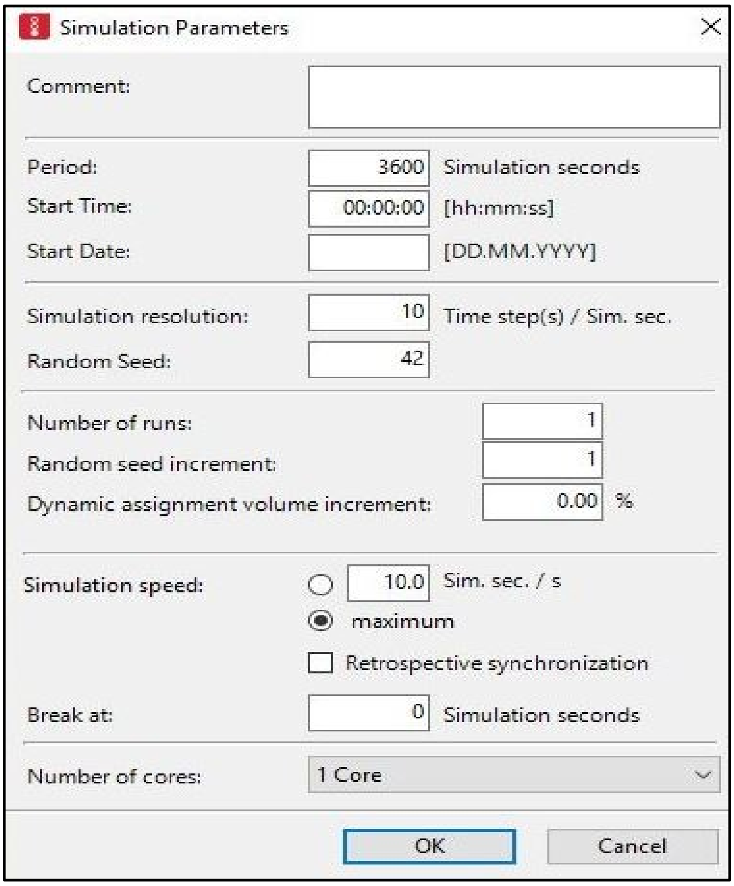


Figure 6.2 Simulation parameters with default values

c) Random seed: VISSIM is a stochastic simulation model, where it generates vehicles based on random seed numbers. The random seed numbers are assigned to follow certain vehicle arrival distribution. The number of simulation runs gives the output for a particular number of times. If the number of runs is one, it will give only one output.

d) Simulation speed: It is the number of simulation seconds required to complete the real time seconds. However, the change in simulation speed will not affect the overall output of the simulation

6.6 VISSIM Simulation for the Current Study

The traffic in India is highly heterogeneous in nature. The conflicting traffic is substantially higher at uncontrolled intersections especially under mixed traffic conditions where the conflicting traffic is high with higher potential for accidents. It is very difficult to analyze the uncontrolled intersections because of the heterogeneous conditions and the drivers lacking knowledge on the traffic rules. Thus, the analysis of uncontrolled intersections is one of the major challenges in the field of transportation engineering. The mixed traffic is composed of different vehicle types with different lengths and widths and operational characteristics like power and maneuverability. The vehicle types also affect the capacity of the intersection because of their different lengths and widths. The effect of vehicle composition on delay and volume of uncontrolled intersections using micro simulation is one of the important studies. Many studies were executed in the past that are related to analysis of un-signalized intersections using VISSIM simulation. Most of the models estimate capacity based on lane-based motorized traffic. VISSIM can be used to estimate capacity with more precision using geometric and driver characteristics unlike the other capacity estimation models. VISSIM provides simulation results that better match with the field conditions and traffic engineering principles.

6.7 Data Analysis and Calibration of VISSIM

It is important to analyze the field data in order to know the share of each vehicle type at the intersection and also to know how the capacity of an intersection is varying for different proportions of each vehicle type. Figures 4.7 to 4.12 shows the typical proportions of each vehicle type at all the intersections considered in this study.

The major proportion of the traffic for all the intersections is mainly occupied by 2w, 3w and 4w compared to other vehicle types. Different vehicle compositions (total 88 compositions for each intersection) are considered by changing the vehicle types with an increment of 10% (11 compositions each for vehicle type starting from 0% to 100%. The proportion of other vehicle types for each selected composition is considered in such a way that these proportions match with the observed field proportion. For example, if 2w is the subject vehicle type for C5 composition at New Delhi intersection, 60% of total volume consists of 2w, 3% for 3w, 28% for 4w, 1% for bus and LCV, 4% bicycle, and 3% HV. Calibration for field volume is one of the important steps in VISSIM. It is the time taking process as it requires a number of multiple simulation runs by changing CC parameters. Before proceeding to calibration, one has to fix the RSN (Random Seed Number). The calibration of VISSIM is performed to suit the field volumes for all the uncontrolled intersections. The simulated volume is compared with the field volume and RMSE (Root Mean Square Error) is calculated for each random seed number. The random seed number with least error is selected for the study. For the calibrated parameters, simulation is performed for different vehicle compositions.

6.8 Comparison of Calibrated Volume with Field Volume

The calibration in VISSIM is done in such a way that the calibrated total and leg-wise volumes must be nearly equal to the field volumes with the variation of 3 to 5% for the entire intersection. For all the intersections, it is observed after calibration that the total volume and the leg-wise volume are almost equal to the actual field volume with negligible percentage error. For the field and calibrated conditions, total volumes of the six selected intersections are shown in Table 6.1. The percentage error for all the six intersections is very much less than 3%.

Table 6.1 Comparison of field and calibrated volumes for all the intersections

Location	Field volume	Calibrated volume	MAPE
Forest office Junction	21356	21298	0.27
100 ft Junction	10575	10560	0.14
Kantrajurs road	12343	12271	0.58
New Delhi	6,744	6,881	2.03
Kozhikode	15,818	15,857	0.25
Karimnagar	19,153	18,845	1.61

The variation of average simulated delay for all the intersections considered in this study for different vehicle compositions and different vehicle types is shown in Figures 6.3 to 6.44.

6.9 Effect of Traffic Composition on Delay and Volume at Uncontrolled Urban Intersections

With increase in the proportion of bicycles as shown in Figures 6.3 and 6.4, it is observed that the intersection volume decreases whereas the delay increases. It is important to note here that, the performance characteristics of the bicycles such as speed play a major role in terms of affecting the volume and delay at the intersection. Because of the lower speeds of the bicycles, with increase in its proportion the number of vehicles per unit time decreases, i.e., the volume of vehicles decreases. Even though there is a decrease in volume of vehicles at the intersection because of the increase in proportion of bicycles, the overall delay at the intersection increases due to lower speeds of the bicycles (due to slowing moving behaviour).

Thus, the performance characteristics play a major role for bicycles when compared to the dimensional characteristics.

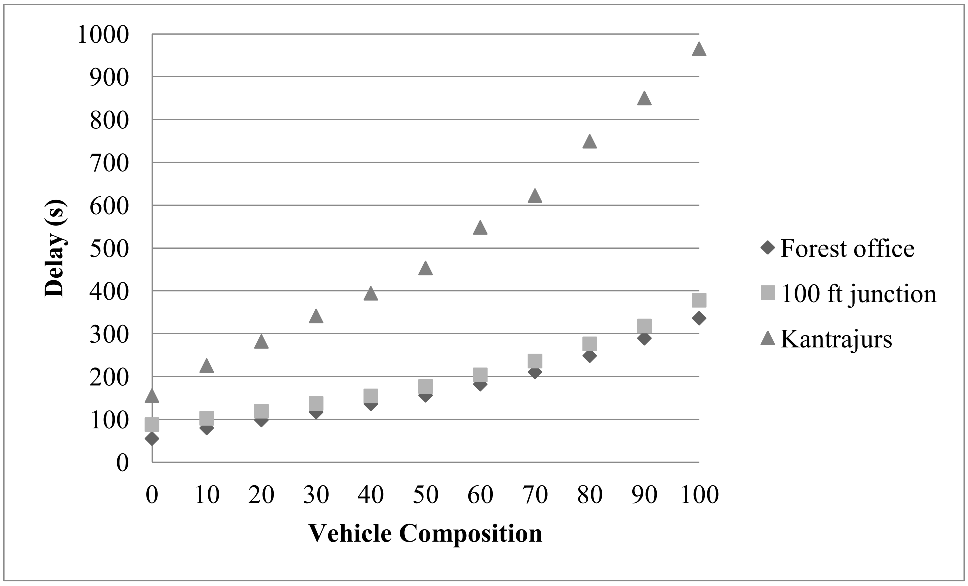


Figure 6.3 Comparison of average delay of Bicycle with different vehicle compositions at four-legged intersections

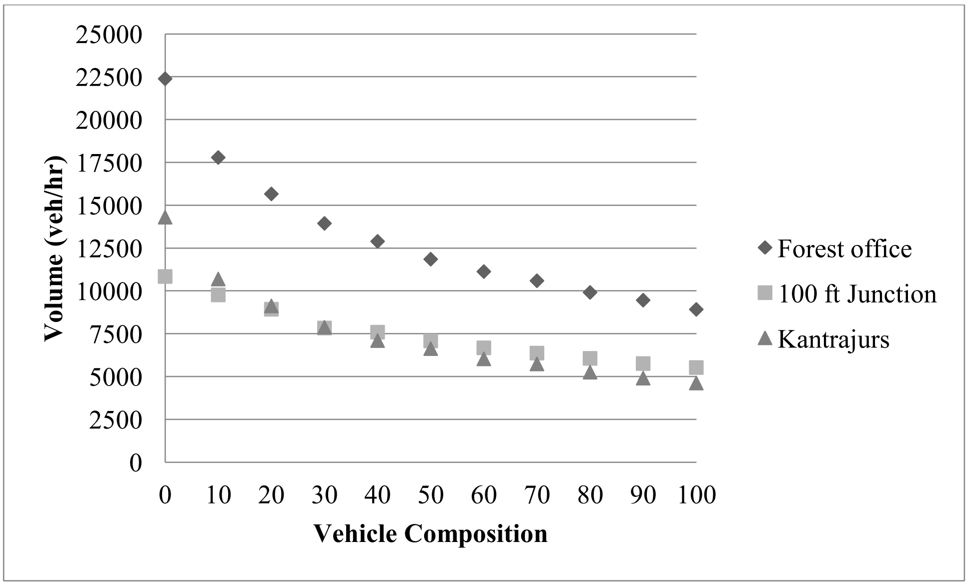


Figure 6.4 Comparison of Volume of Bicycle with different vehicle compositions at four-legged intersections

At all the three four-legged intersections, as shown in Figures 6.5 and 6.6, with increase in proportion of 2w's there is an increase in total volume of vehicles at each of these intersections and at the same time there is an increase in delay at each of these intersections. This is due to the fact that with increase in proportion of 2w, the available gap between these vehicles decreases resulting in an increase in delay.

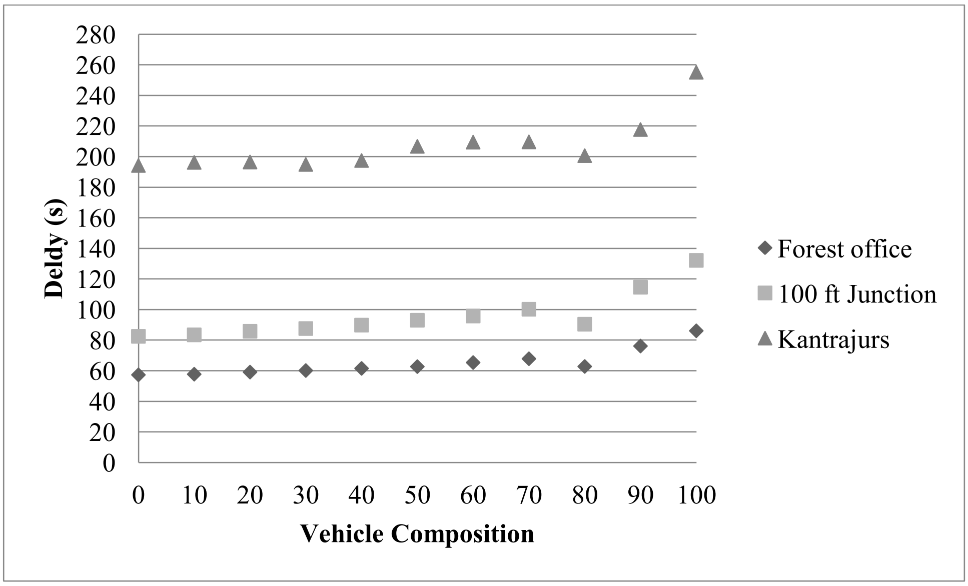


Figure 6.5 Comparison of average delay of 2w with different vehicle compositions at four-legged intersections

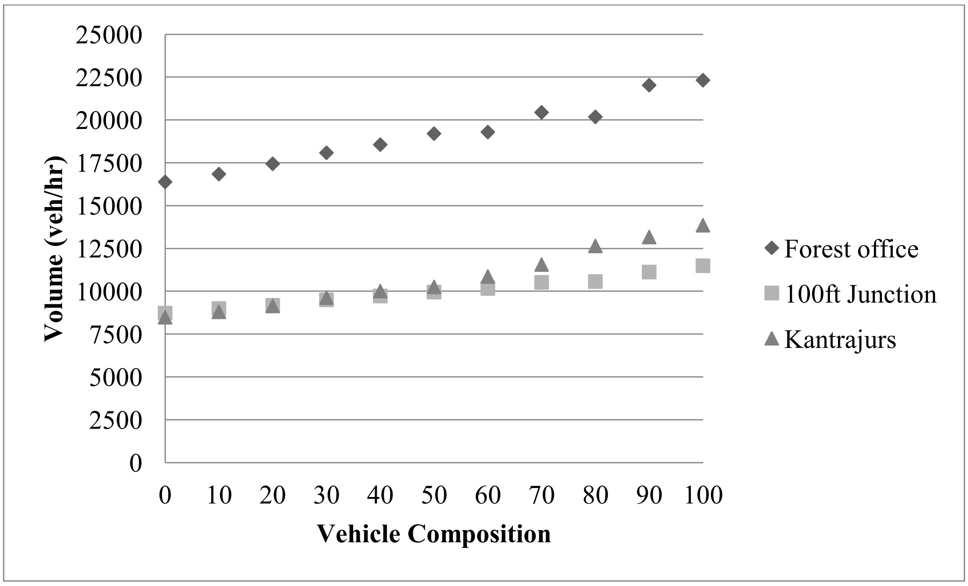


Figure 6.6 Comparison of volume of 2w with different vehicle compositions at four-legged intersections

From Figures 6.7 to 6.10, it can be seen that, with increase in proportion of 3w and 4w, the overall volume of the intersection increases whereas the delay decreases. Because of the relatively larger size of these vehicle types when compared to bicycles and 2w, the gap maintained by these vehicles within the traffic stream will be relatively higher providing more opportunity to cross the intersection easily thereby decreasing the overall delay of the intersection.

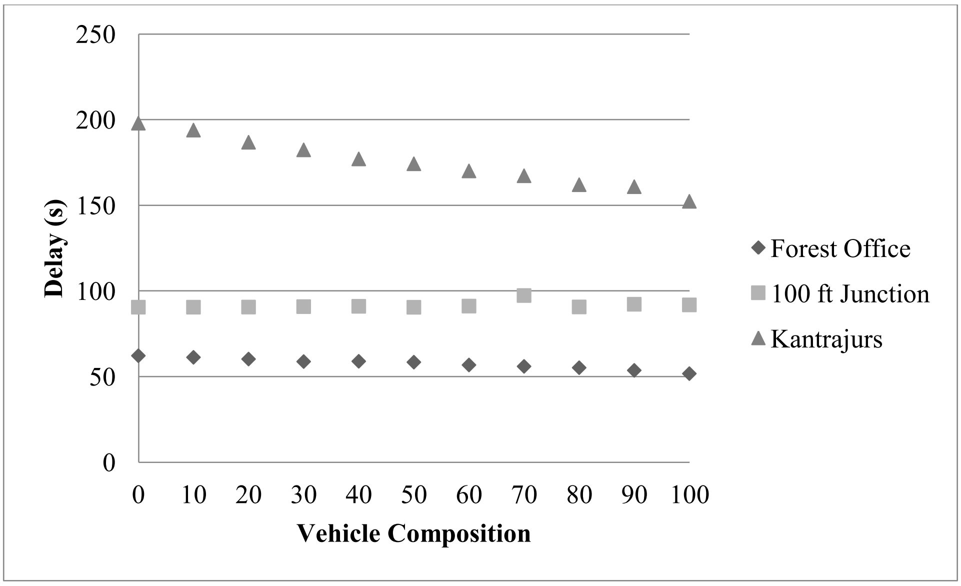


Figure 6.7 Comparison of average delay of 3w with different vehicle compositions at four-legged intersections

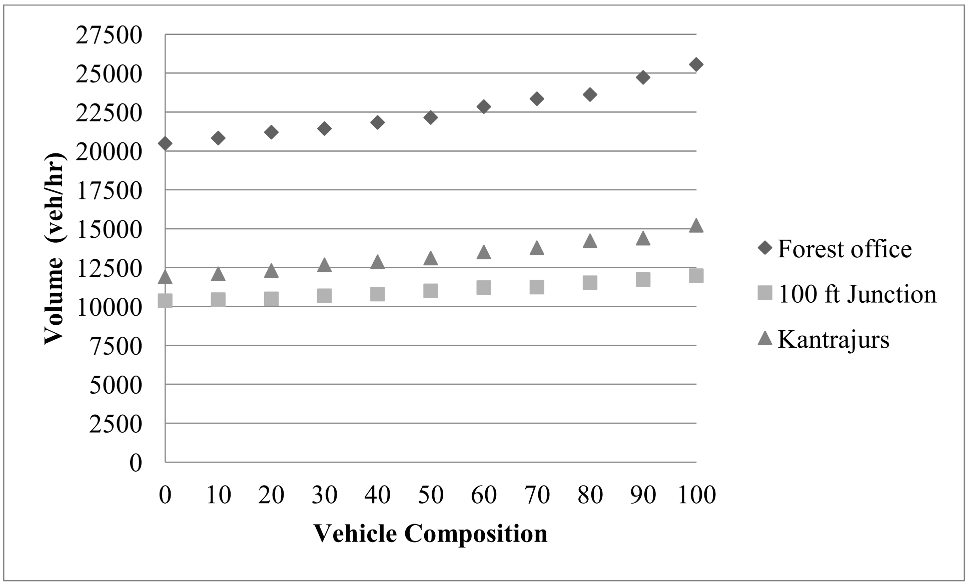


Figure 6.8 Comparison of volume of 3w with different vehicle compositions at four-legged intersections

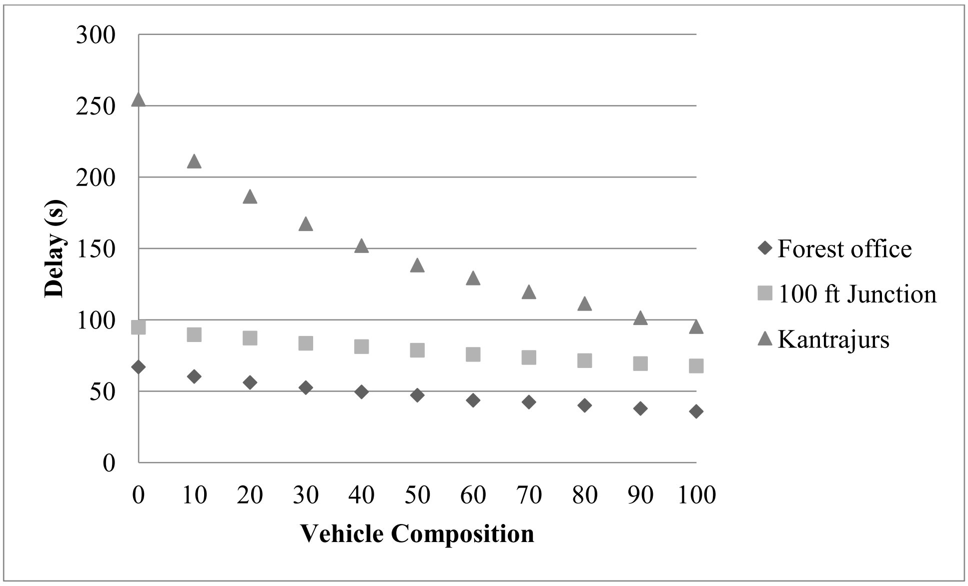


Figure 6.9 Comparison of average delay of 4w with different vehicle compositions at four-legged intersections

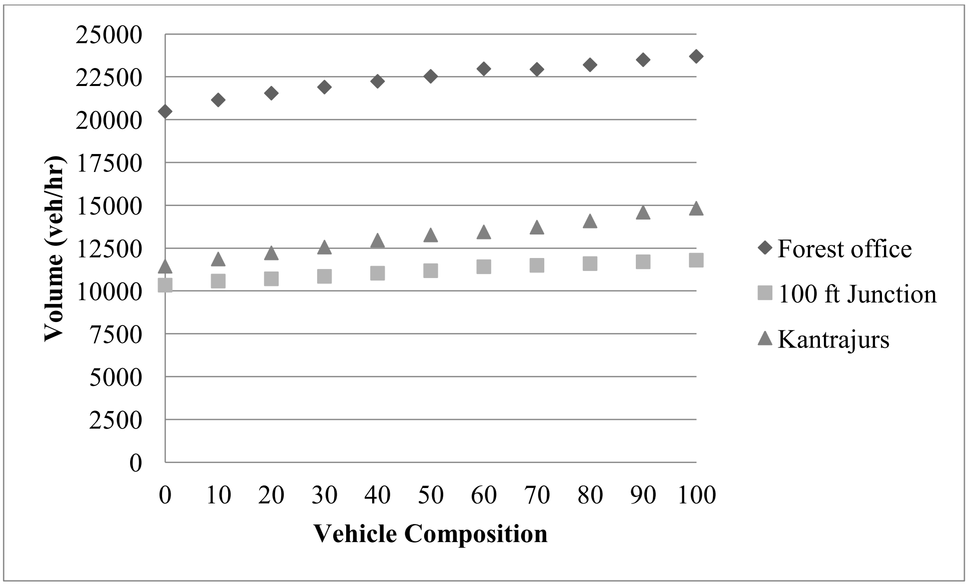


Figure 6.10 Comparison of volume of 4w with different vehicle compositions at four-legged intersections

Based on the above discussion, it can be observed that for 2w, 3w and 4w, with increase in proportion of these vehicle types there is an increase in overall volume of the intersection. This is mainly due to the smaller size and higher acceleration characteristics of 2w, 3w, and 4w.

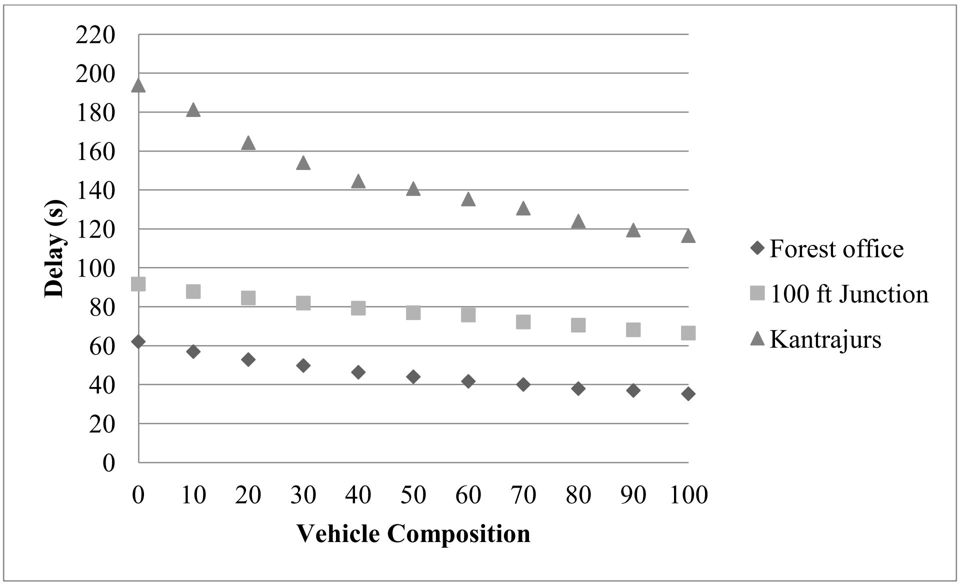


Figure 6.11 Comparison of average delay of LCV with different vehicle compositions at four-legged intersections

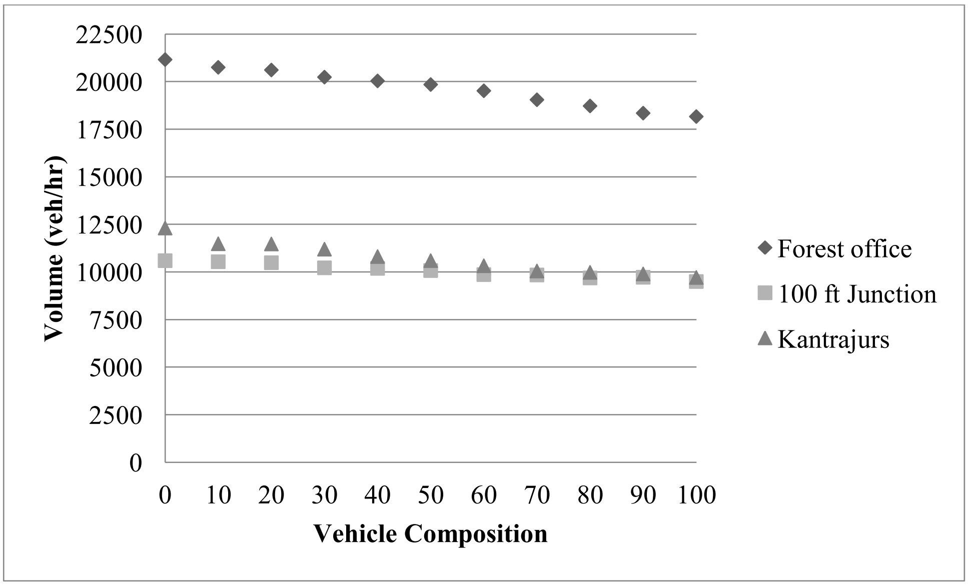


Figure 6.12 Comparison of volume of LCV with different vehicle compositions at four-legged intersections

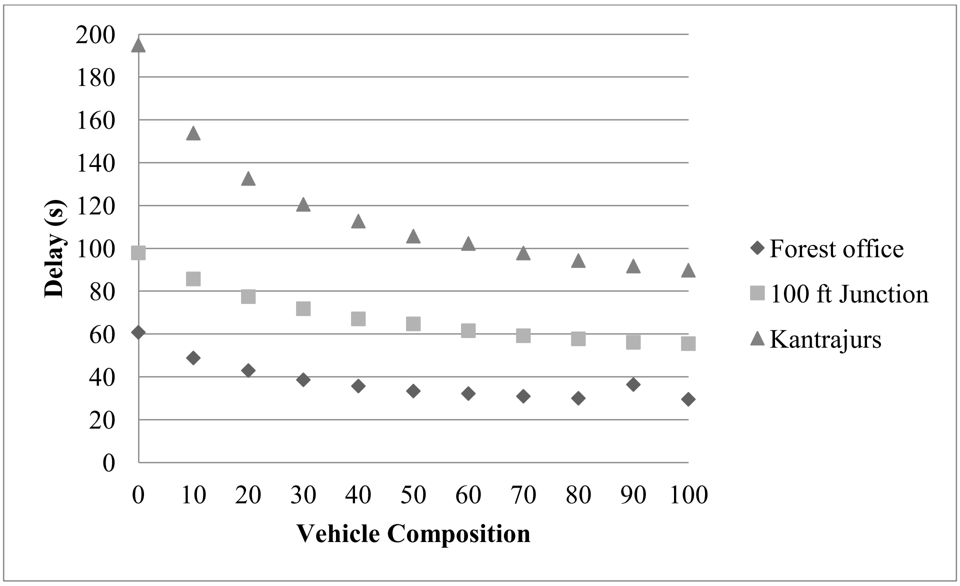


Figure 6.13 Comparison of average delay of HV with different vehicle compositions at four-legged intersections

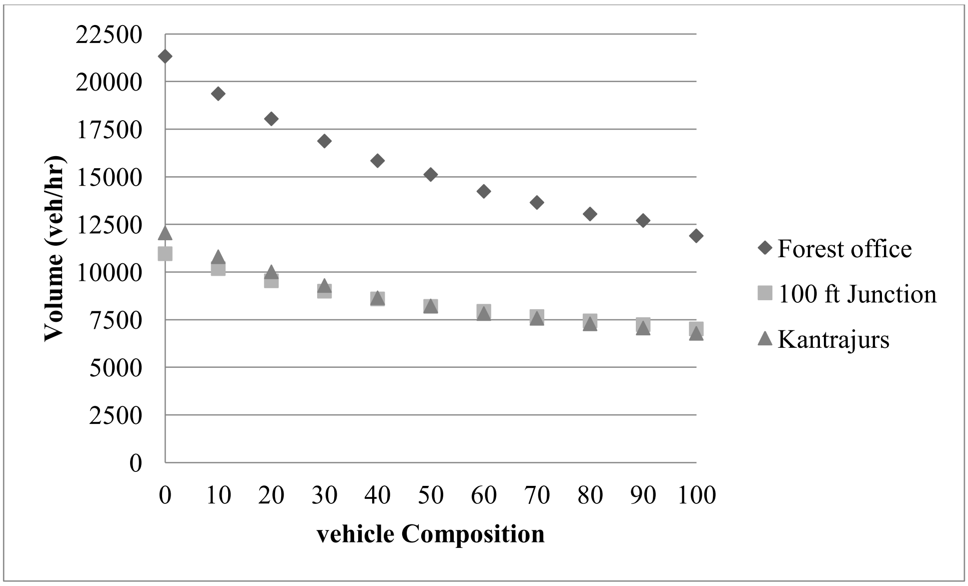


Figure 6.14 Comparison of volume of HV with different vehicle compositions at four-legged intersections

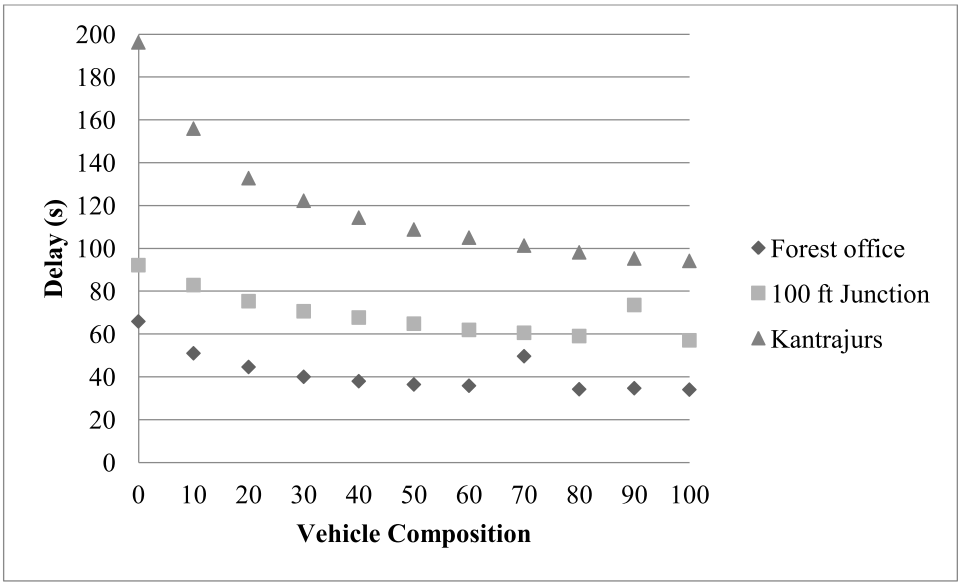


Figure 6.15 Comparison of average delay of BUS with different vehicle compositions at four-legged intersections

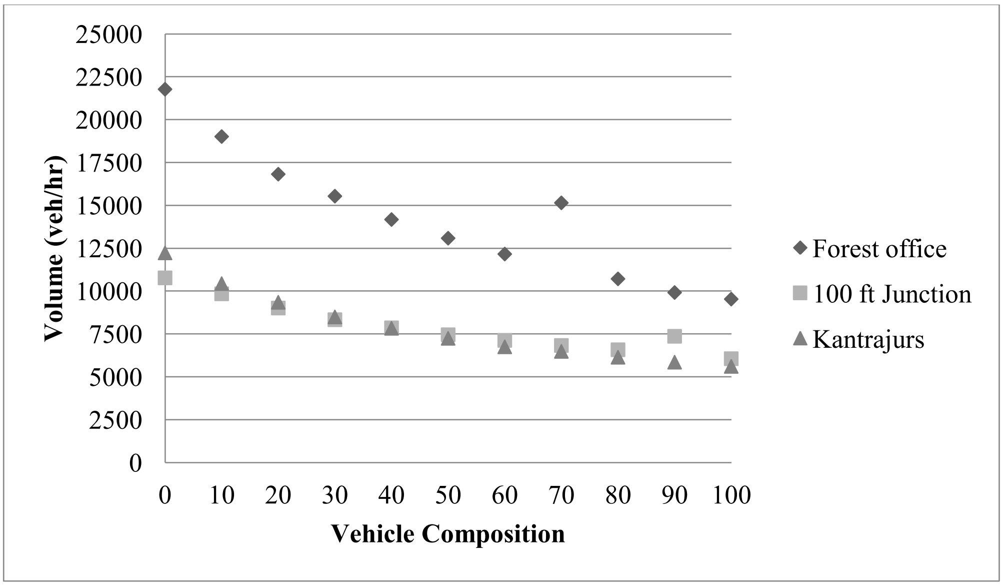


Figure 6.16 Comparison of volume of BUS with different vehicle compositions at four-legged intersection

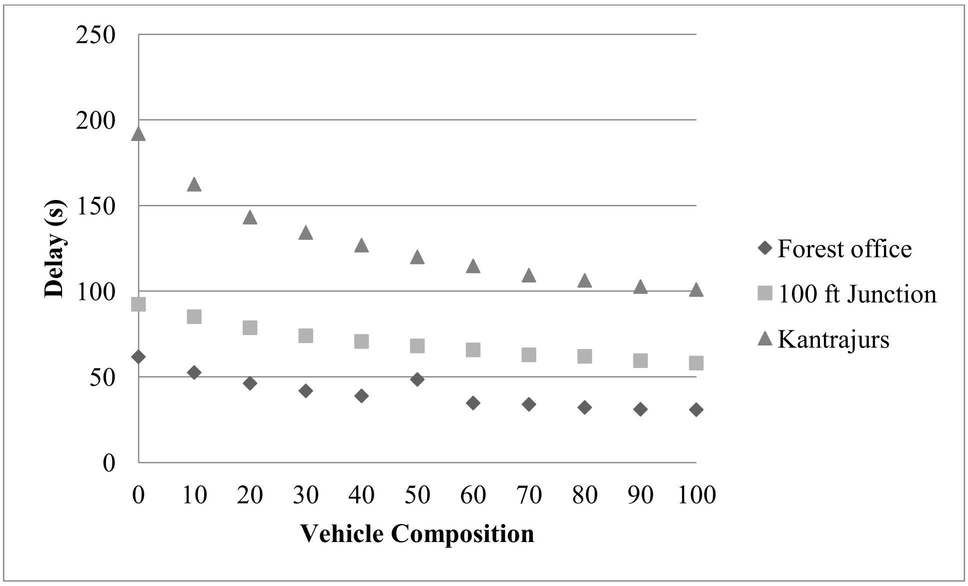


Figure 6.17 Comparison of average delay of Tractor with different vehicle compositions at four-legged intersections

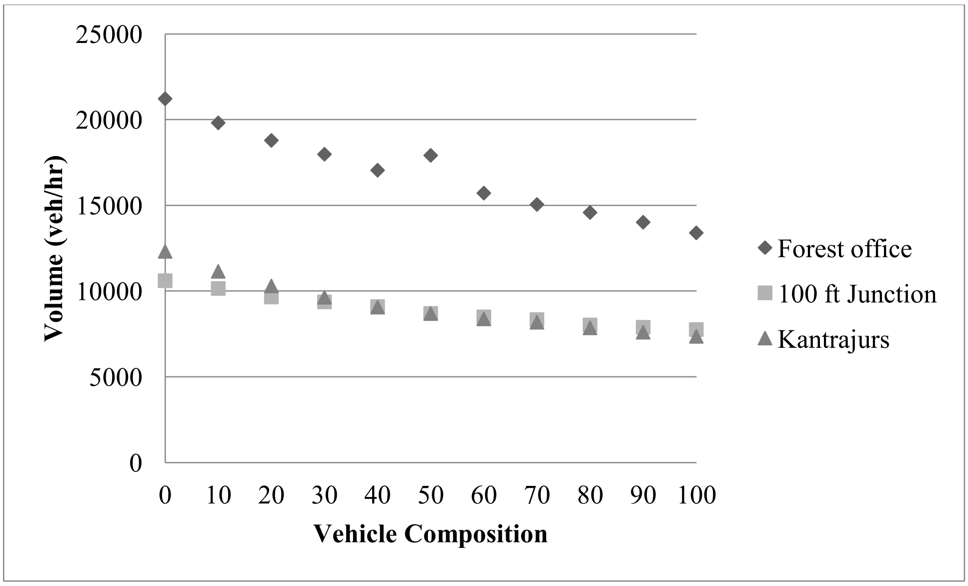


Figure 6.18 Comparison of volume of Tractor with different vehicle compositions at four-legged intersections

For remaining all vehicle types including LCV, HV, Buses and Tractors as show in the Figures 6.11 to 6.18, it can be observed that with increase in proportion of these vehicle types the overall volume of the intersection decreases and at the same time the intersection delay also decreases. This is mainly due to the fact that, all these vehicle types will have relatively higher projected areas when compared to the other vehicle types and has relatively lower acceleration characteristics thus resulting in less number of vehicles per unit time resulting in decrease in overall volume of the intersections. Because of the large size of these vehicle types, these vehicles tend to maintain relatively higher gaps in between the lead and the following vehicle thus creating more opportunities for relatively small sized vehicles to cross the intersection easily thereby decreasing the delay at the intersection.

At all the three three-legged intersection including New Delhi, Kozhikode, and Karimnagar similar trends as that of four-legged intersection are observed except for the 3W as shown in the Figures 6.19 to 6.32. As discussed for four-legged intersections, with increase in proportions of the small sized vehicles, they tend to maintain relatively less gap between the lead and the following vehicle resulting in fewer opportunities for the right turning vehicles to clear the intersection resulting in higher delays at the intersection. Specifically at the three-legged intersection, with the increase in proportion of 3w the delay also increased.

Because of the less number of conflict points at the three-legged intersection when compared to the four-legged intersection, with the increase in proportion of the 3w's, they tend to cause higher delays at the three-legged intersections as these vehicles tend to forcefully enter into the intersection because of less conflicting traffic resulting in higher delays.

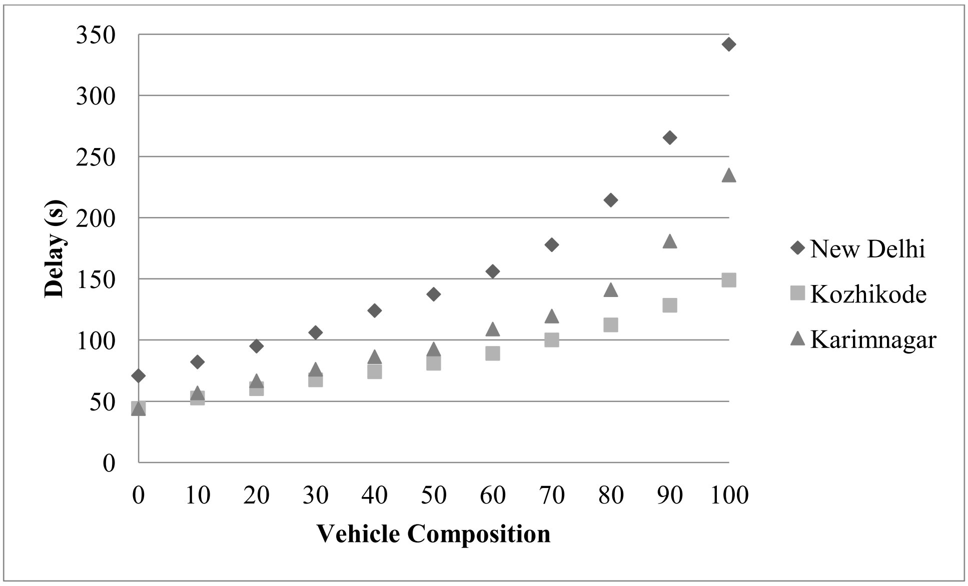


Figure 6.19 Comparison of average delay of Bicycle with different vehicle compositions at three-legged intersections

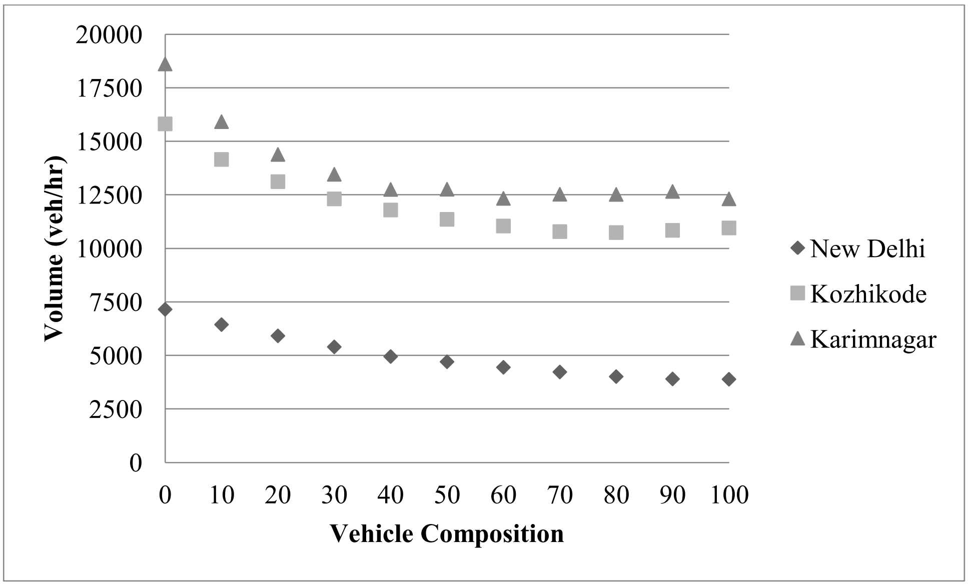


Figure 6.20 Comparison of volume of Bicycle with different vehicle compositions at three-legged intersections

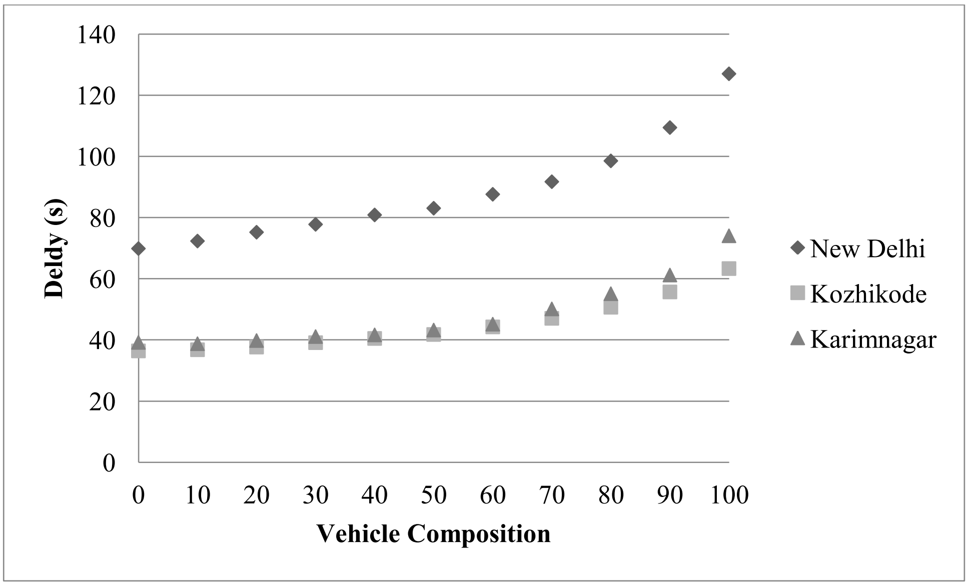


Figure 6.21 Comparison of average delay of 2w with different vehicle compositions at three-legged intersections

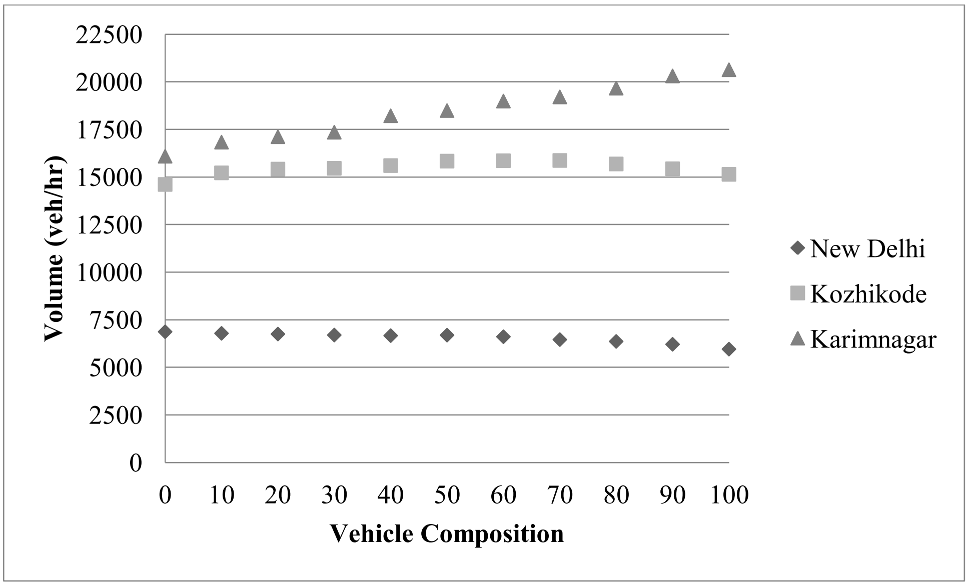


Figure 6.22 Comparison of volume of 2w with different vehicle compositions at three-legged intersections

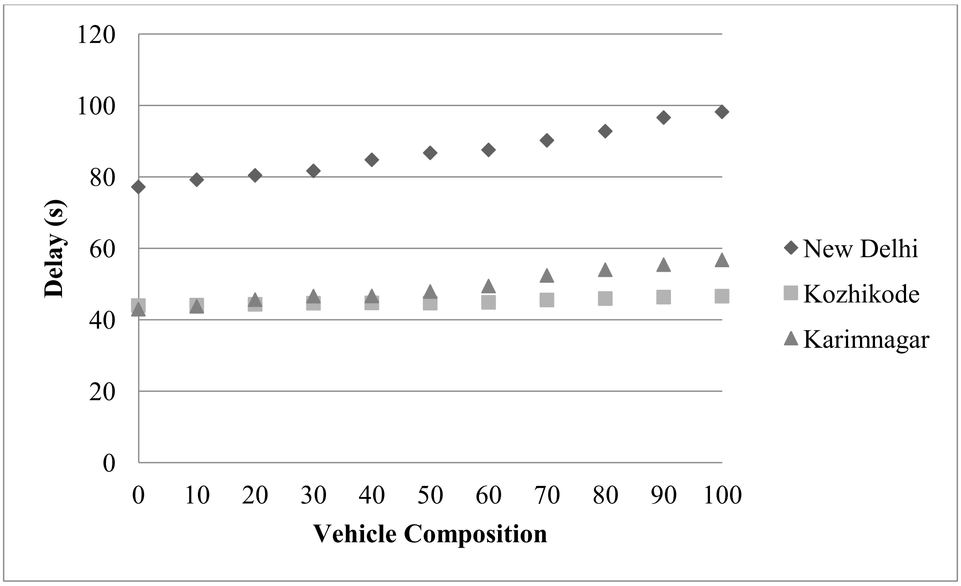


Figure 6.23 Comparison of average delay of 3w with different vehicle compositions at three-legged intersections

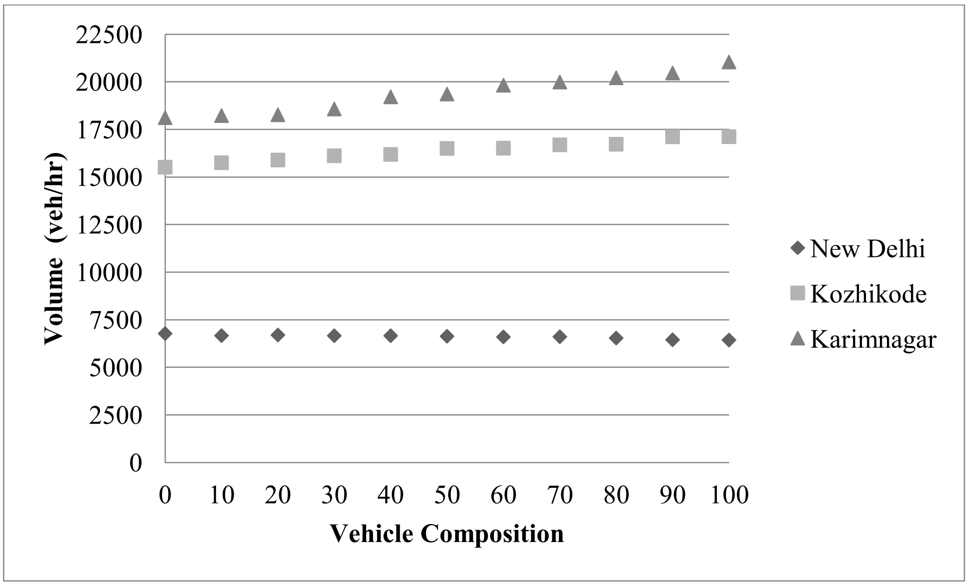


Figure 6.24 Comparison of volume of 3w with different vehicle compositions at three-legged intersections

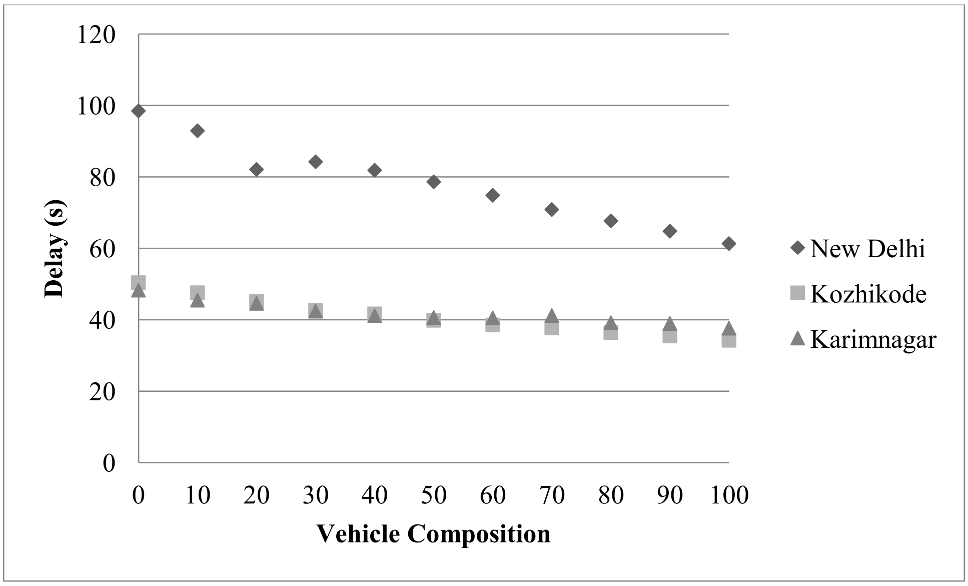


Figure 6.25 Comparison of average delay of 4w with different vehicle compositions at three-legged intersections

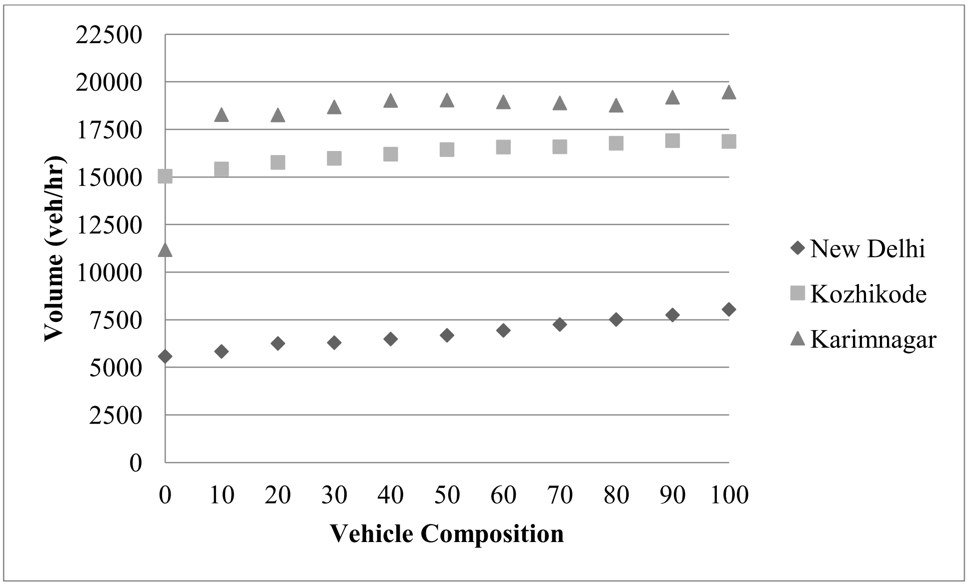


Figure 6.26 Comparison of volume of 4w with different vehicle compositions at three-legged intersections

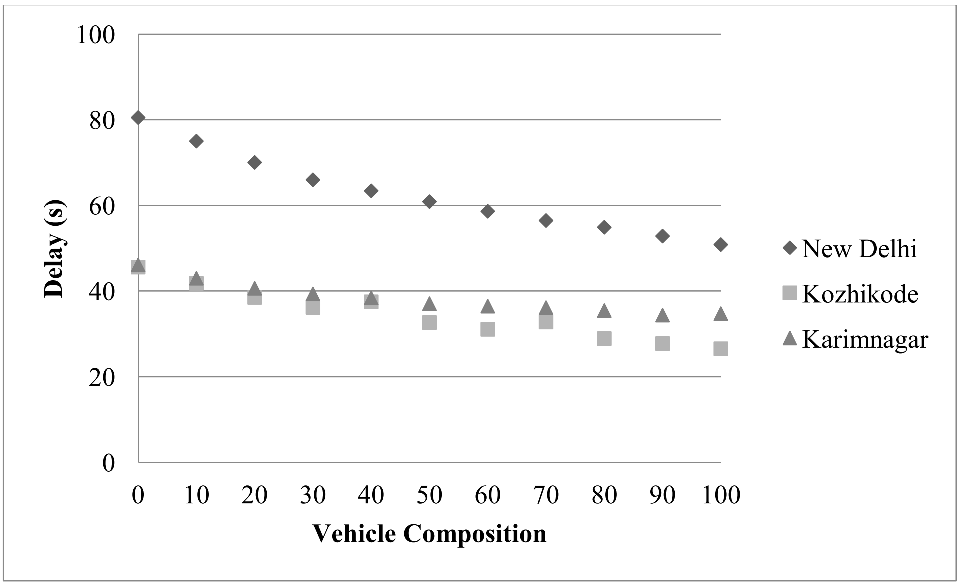


Figure 6.27 Comparison of average delay of LCV with different vehicle compositions at three-legged intersections

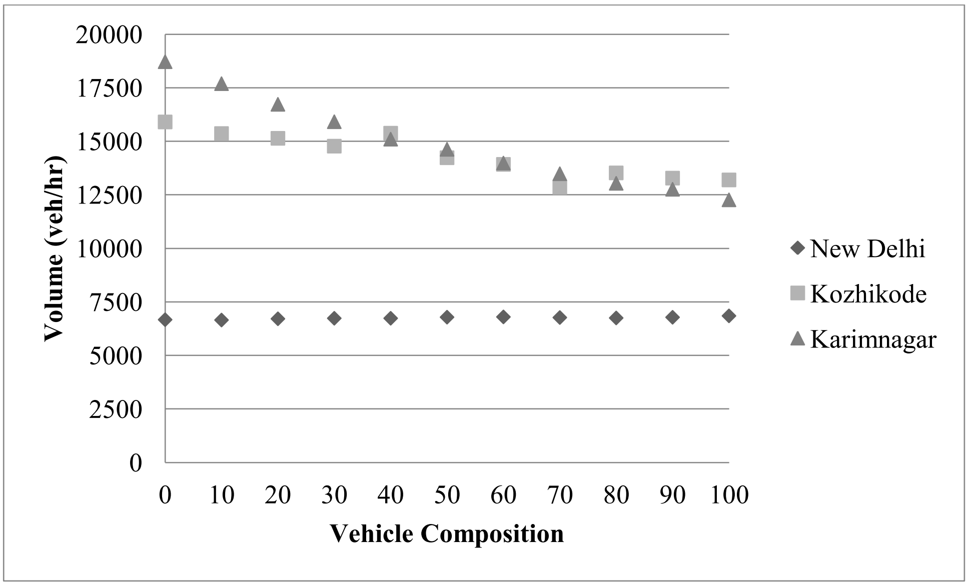


Figure 6.28 Comparison of volume of LCV with different vehicle compositions at three-legged intersections

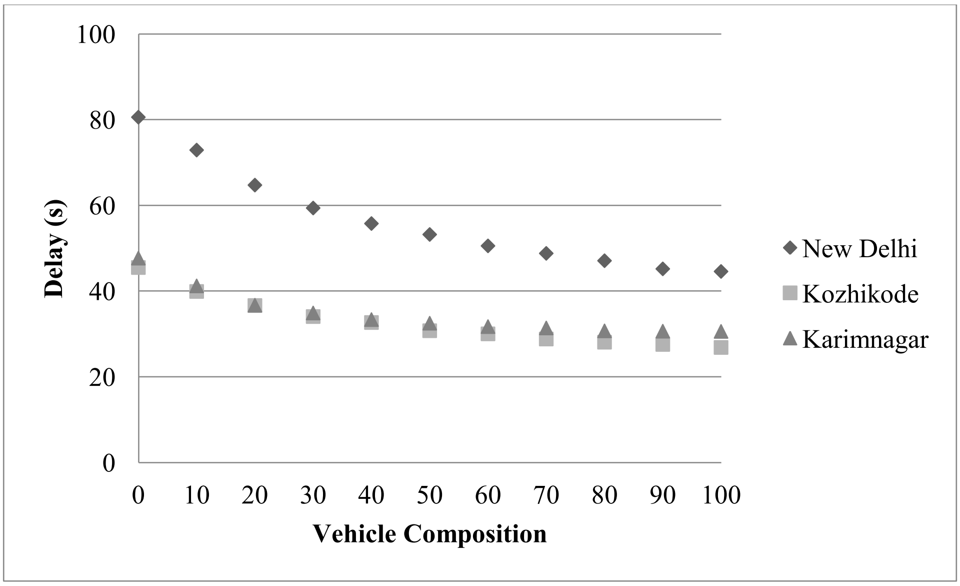


Figure 6.29 Comparison of average delay of HV with different vehicle compositions at three-legged intersections

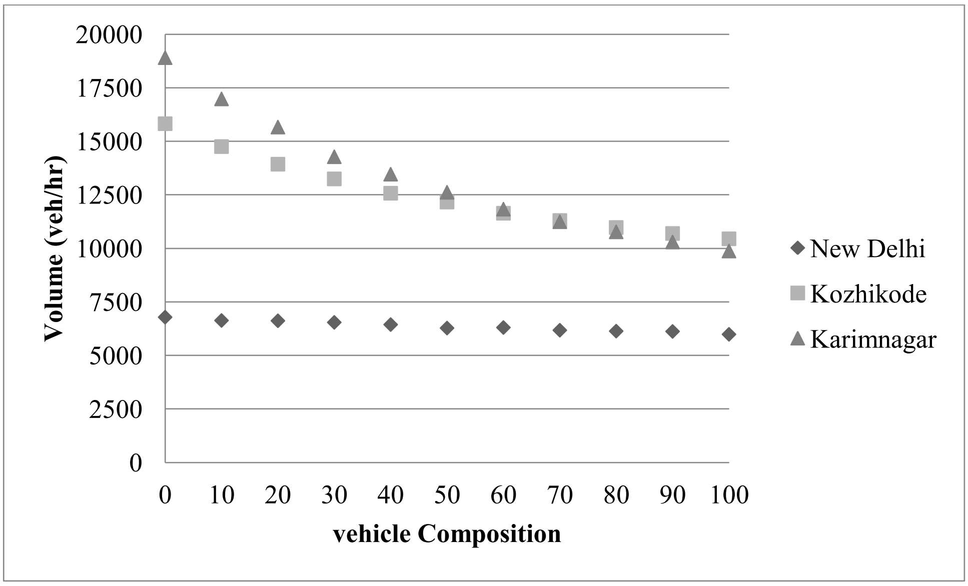


Figure 6.30 Comparison of volume of HV with different vehicle compositions at three-legged intersections

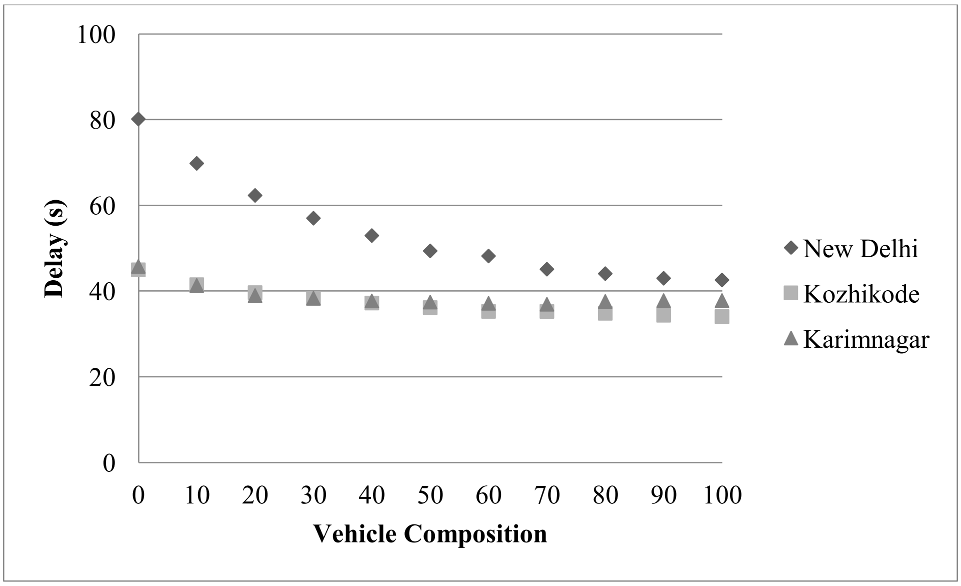


Figure 6.31 Comparison of average delay of Bus with different vehicle compositions at three-legged intersections

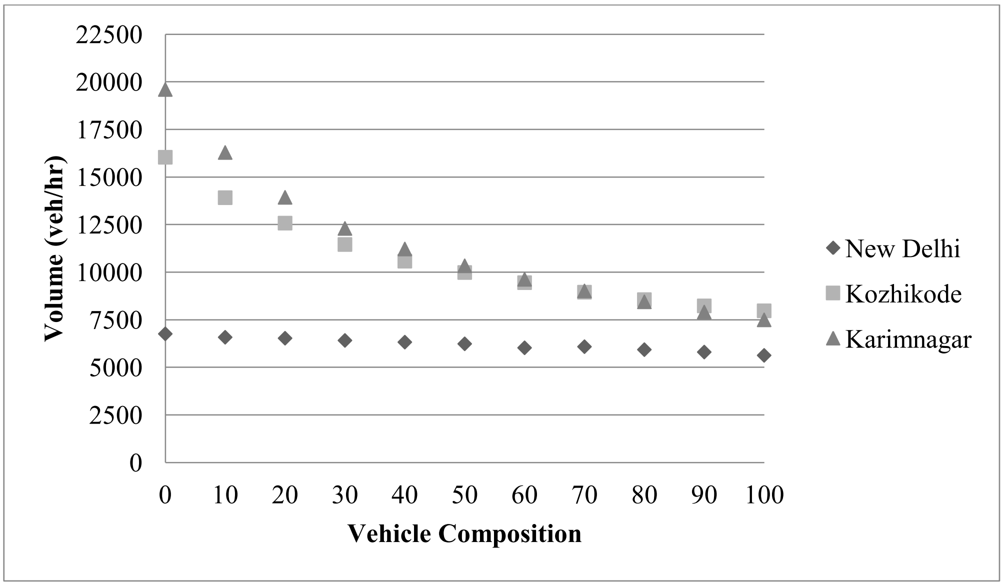


Figure 6.32 Comparison of volume of Bus with different vehicle compositions at three-legged intersections

The variation in overall traffic volume of the intersection with increase in proportion of each vehicle type at Intersection-1 (Forest office junction) is shown in Figure 6.33. Similarly, the variation in overall average delay of the intersection with increase in proportion of each vehicle type at Intersection-1 (Forest office junction) is shown in Figure 6.34.

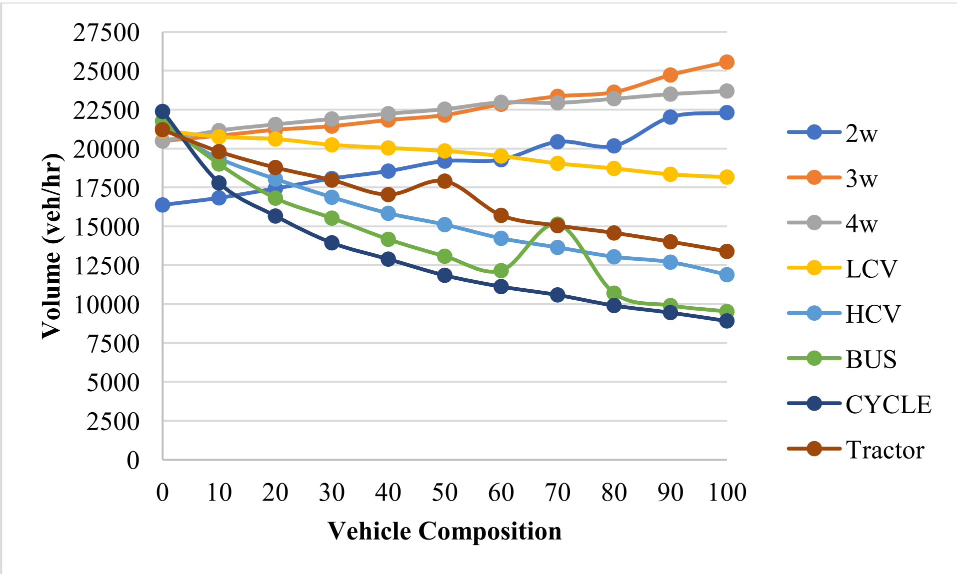


Figure 6.33 Variation of traffic volume with increase in proportions of each vehicle type at Intersection-1

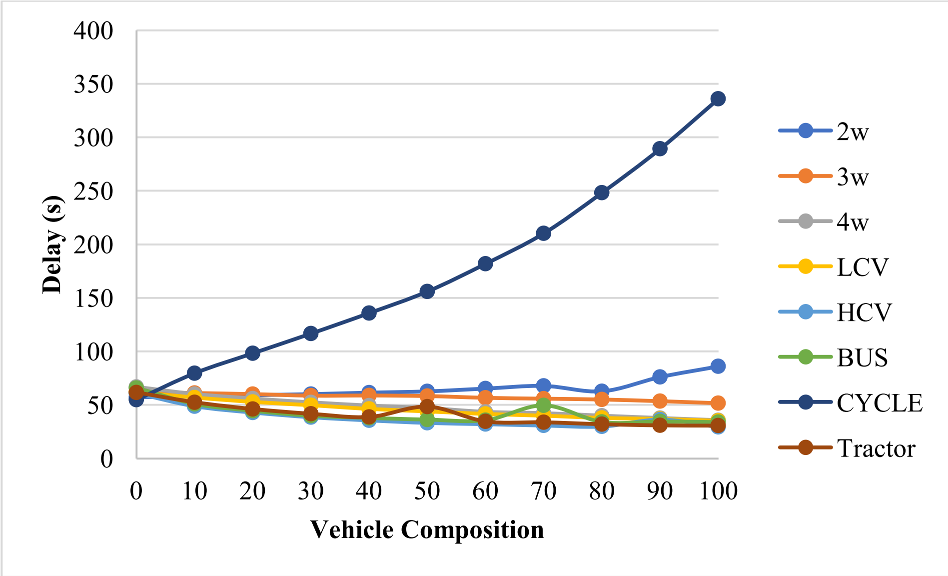


Figure 6.34 Variation of average delay with increase in proportions of each vehicle type at Intersection-1

It can be seen that, increase in proportion of Bicycles is resulting in higher delays at the intersections because of slower speeds of these vehicle types, whereas the influence of other vehicle types on the intersection delay is relatively less when compared to the Bicycles.

Similar trends could be observed at remaining two four-legged intersections i.e, 100 feet junction and Kantrajurs junction as shown in the Figures 6.35 to 6.38. This shows that irrespective of the type of intersection, the proportion of bicycles plays a critical role in the overall delay of the urban uncontrolled intersection especially for the mixed traffic conditions. Similar trends are also observed at all the three three-legged intersections as shown in the Figures 6.39 to 6.44.

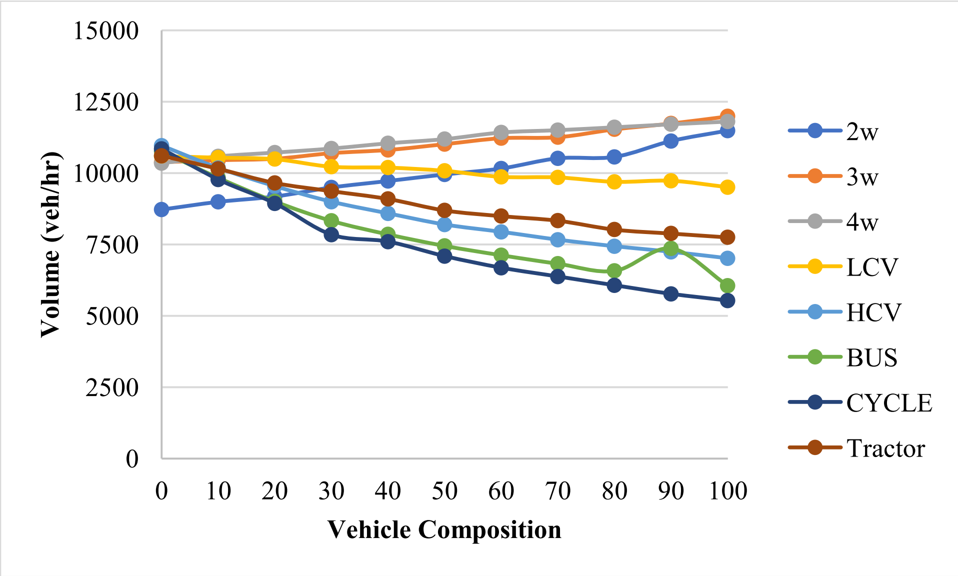


Figure 6.35 Variation of traffic volume with increase in proportions of each vehicle type at Intersection-2

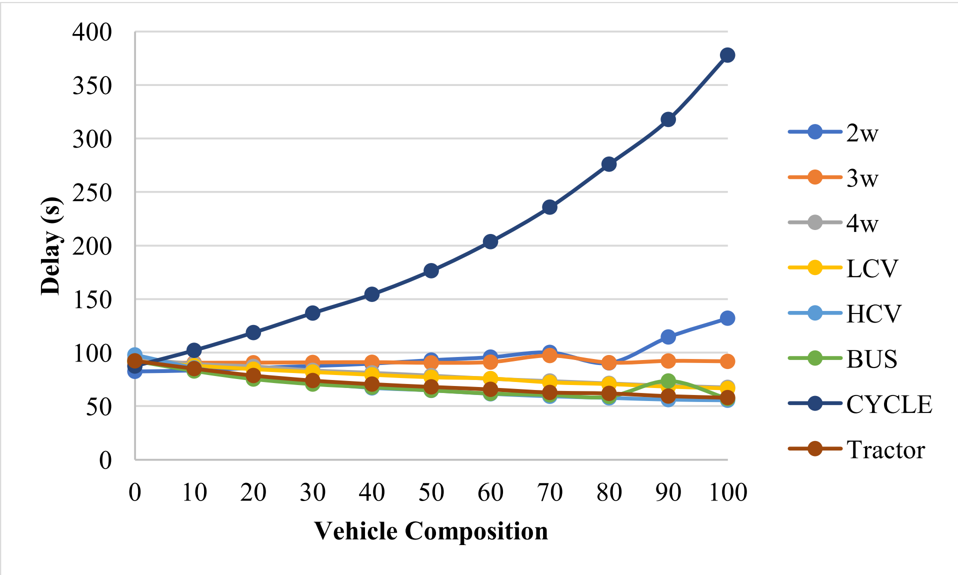


Figure 6.36 Variation of average delay with increase in proportions of each vehicle type at Intersection-2

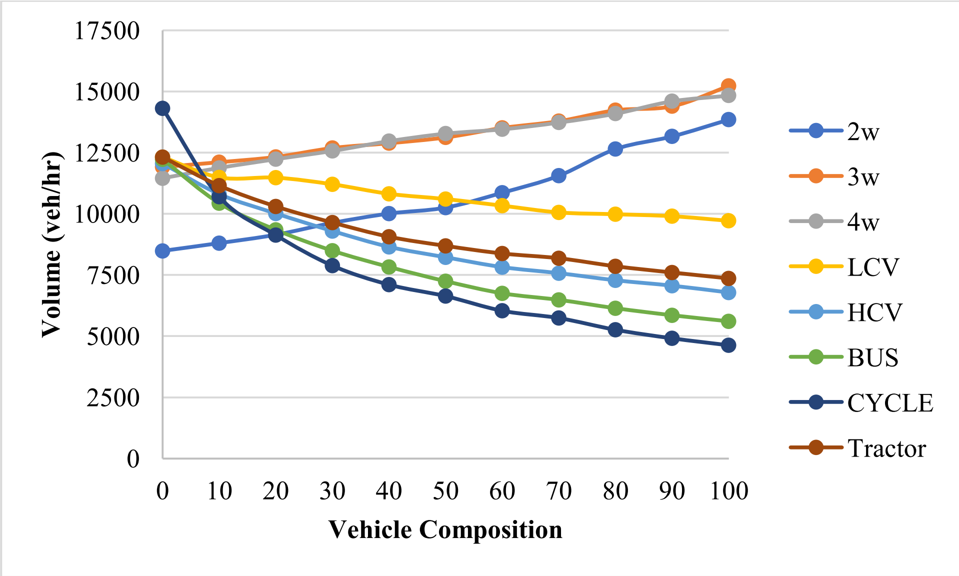


Figure 6.37 Variation of traffic volume with increase in proportions of each vehicle type at Intersection-3

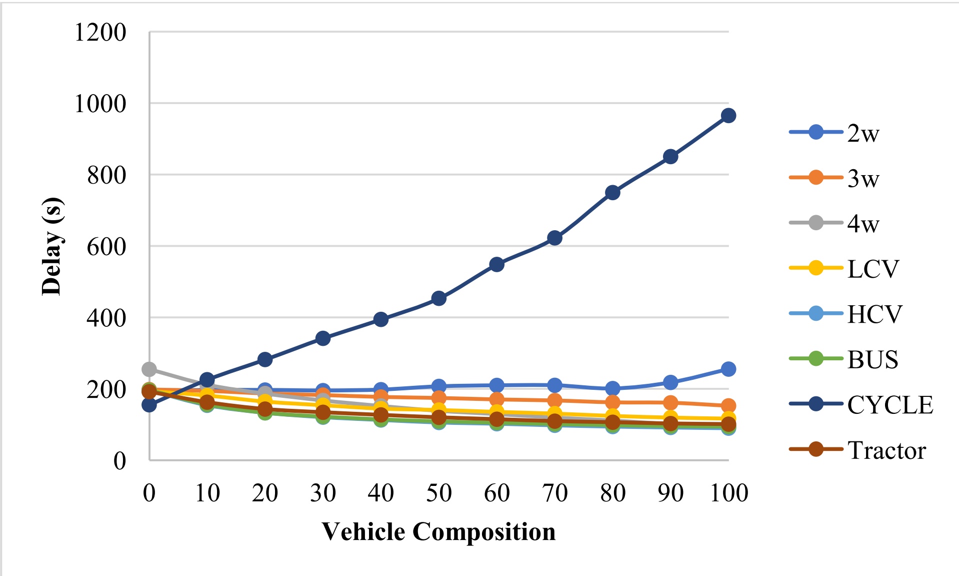


Figure 6.38 Variation of average delay with increase in proportions of each vehicle type at Intersection-3

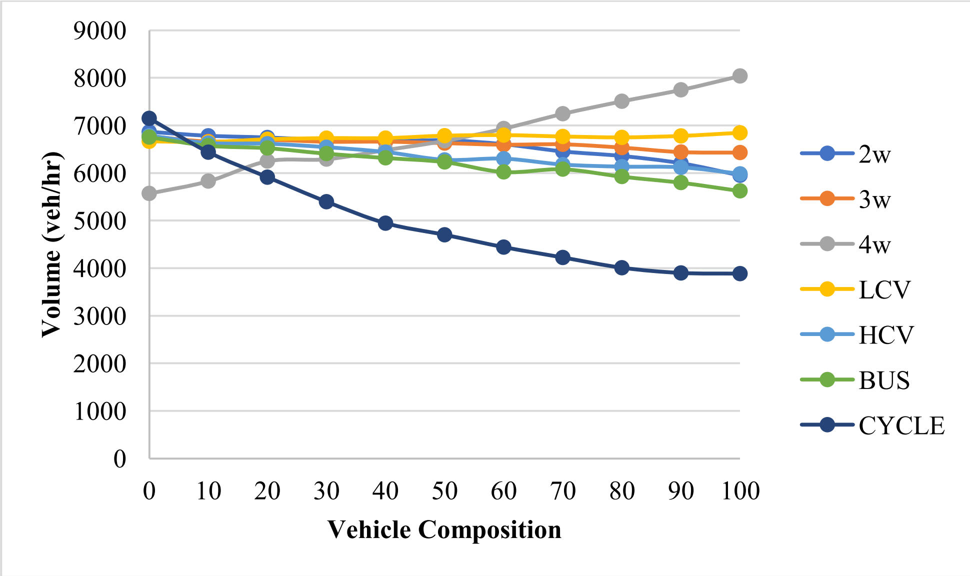


Figure 6.39 Variation of traffic volume with increase in proportions of each vehicle type at Intersection-4

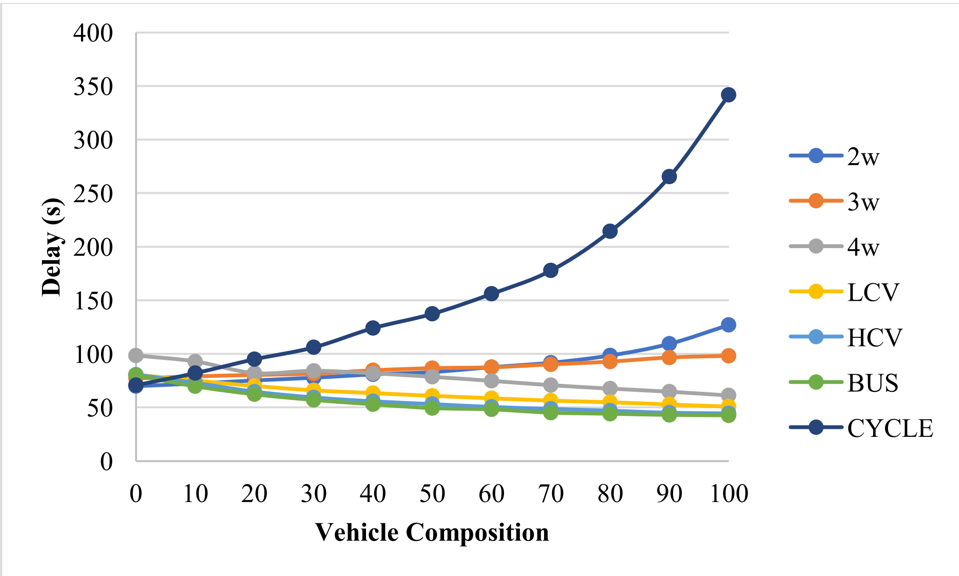


Figure 6.40 Variation of average delay with increase in proportions of each vehicle type at Intersection-4

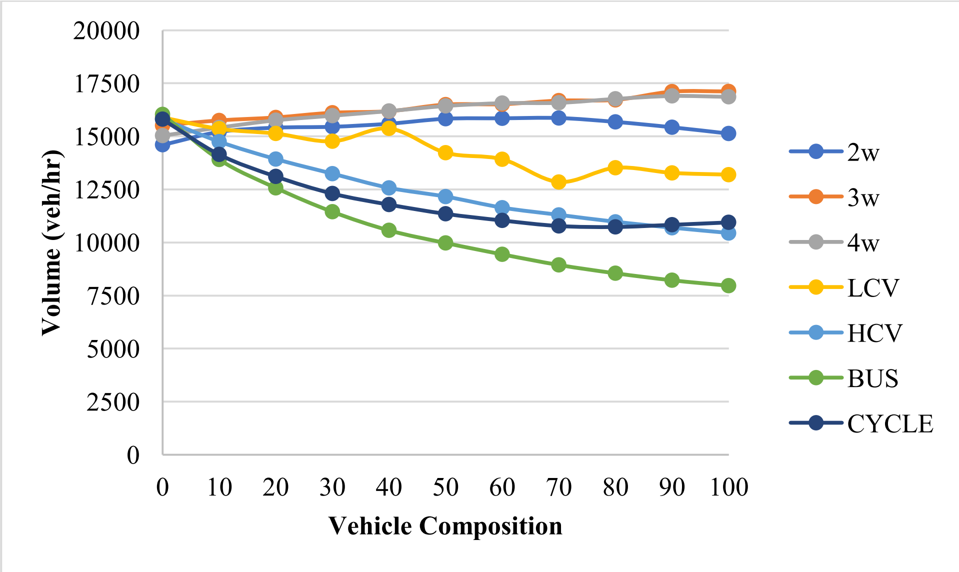


Figure 6.41 Variation of traffic volume with increase in proportions of each vehicle type at Intersection-5

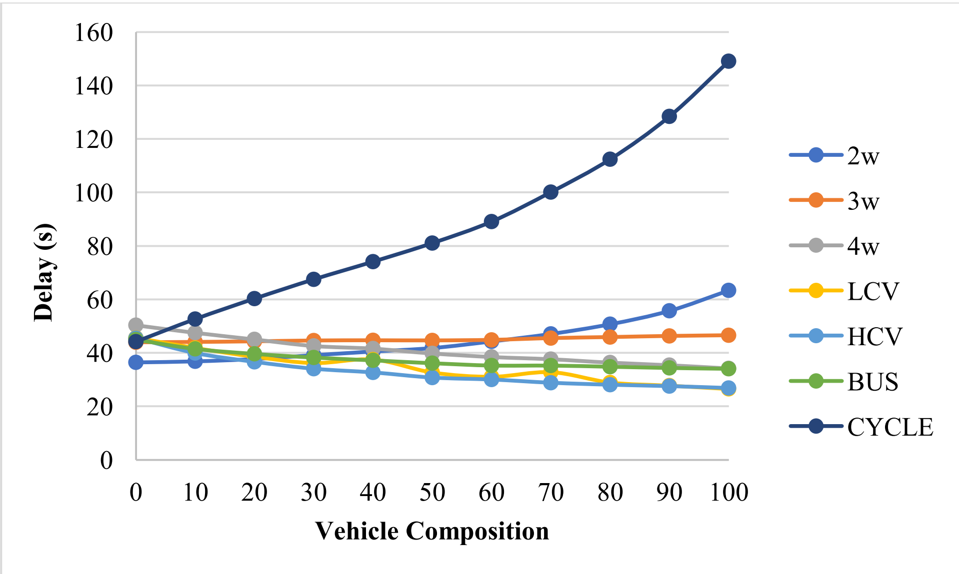


Figure 6.42 Variation of average delay with increase in proportions of each vehicle type at Intersection-5

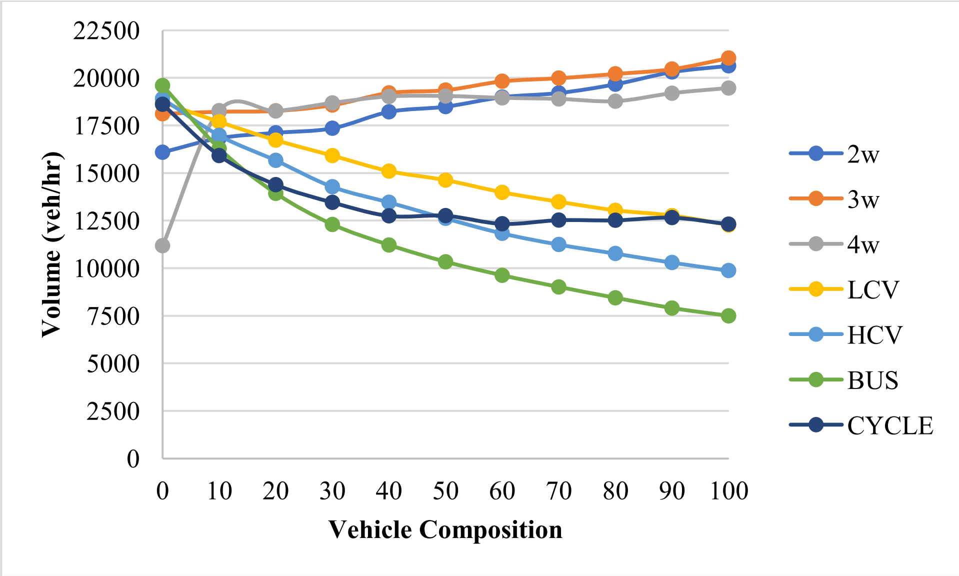


Figure 6.43 Variation of traffic volume with increase in proportions of each vehicle type at Intersection-6

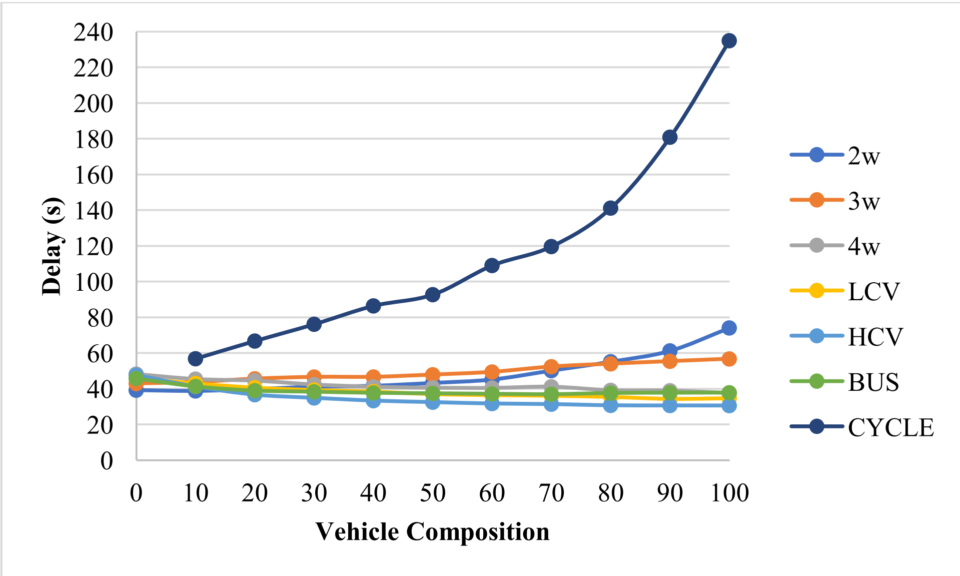


Figure 6.44 Variation of average delay within crease in proportions of each vehicle type at Intersection-6

6.10 Estimation of Total Delay and Service Delay at the Intersections

For each type of vehicle, the service delay or stopped delay and total delay for all the intersections are calculated. Service delay is the time for which a subject vehicle will stop at the intersection entrance while passing through the intersection for appropriate gap. The delay value is noted when the vehicle is fully stopped and ends when the vehicle begins to accelerate. Total delay is the time taken by the subject vehicle to cross the intersection.

Table 6.2 Delay in seconds for all vehicles types at Forest office Intersection

Vehicle type	NB		SB		EB		WB	
	Service delay	Total delay	Service delay	Total delay	Service delay	Total delay	Service delay	Total delay
2W	5.95	11.13	8.07	13.82	5.5	8.92	6.94	9.49
3W	6.59	13.13	9.171	18.34	5.29	9.23	6.2	10.8
4W	8.34	16.22	16.69	22.63	7.02	11.97	7.51	13.05
LCV	6.68	14.17	9.66	24.22	4.01	9.04	13.28	15.63
Tractor	8.42	18.33	3.33	19.11	5.37	11.28	6.96	11.81
Bus	11.05	25.83	12.99	17.6	9.96	17.81	7.78	16.81
HV	1.83	12.39	-	-	7.2	16.13	-	-

Table 6.3 Delay in seconds for all vehicles types at 100 feet Intersection

Vehicle type	NB		SB		EB		WB	
	Service delay	Total delay	Service delay	Total delay	Service delay	Total delay	Service delay	Total delay
2W	4.31	8.94	6.87	8	4.49	8.16	8.35	10.06
3W	4.9	10.27	6.94	10.93	4.9	9.75	12.94	15.22
4W	6.42	12.31	9.83	11.96	2.43	7.23	11.07	13.65
LCV	4.18	9.93	7.9	12.21	7.15	12.75	8.62	11.58
Tractor	5.87	13.86	5.27	10.33	6.27	13.3	7.23	11.11
HV	3.32	9.46	-	-	4.36	10.65	-	-
Bus	8.25	12.95	6.97	10.66	3.01	11.99	8.43	14.04

Table 6.4 Delay in seconds for all vehicles types at Kantrajrus Intersection

Vehicle type	NB		SB		EB		WB	
	Service delay	Total delay	Service delay	Total delay	Service delay	Total delay	Service delay	Total delay
2W	8.76	6.03	8.25	7.91	3.86	8.54	7.64	9.77
3W	6.82	6.7	4.17	7.64	5.27	9.49	9.57	13.01
4W	17.21	9.26	8.14	10.29	9.47	9.6	9.29	13.35
LCV	-	12.05	-	13.21	10.96	10.9	10.95	13.6
Bus	-	14.65	-	13.8	-	12.29	3.89	8.41

Table 6.5 Delay in seconds for all vehicles types at New Delhi Intersection

Vehicle type	SB		WB	
	Service Delay	Total Delay	Service Delay	Total Delay
2W	4.355	5.306	4.1	6.091
3W	1.195	6.428	3.794	8.336
4W	4.909	6.524	5.105	7.304
LCV	5.87	8.493	-	6.932
HV	4.936	7.667	-	8.718
Bus	4.21	8.515	3.554	7.664
Bicycle	8.298	10.021	5.75	9.458

Table 6.6 Delay in seconds for all vehicles types at Kozhikode Intersection

Vehicle type	SB		WB	
	Service Delay	Total Delay	Service Delay	Total Delay
2W	8.623	8.149	7.277	6.492
3W	6.715	9.953	5.096	6.508
4W	9.53	11.771	10.188	9.534
LCV	5.898	8.731	3.309	8.58
HV	9.094	23.469	-	-

Table 6.7 Delay in seconds for all vehicles types at Karimnagar Intersection

Vehicle type	SB		WB	
	Service Delay	Total Delay	Service Delay	Total Delay
2W	11.891	9.438	5.997	7.025
3W	12.223	12.107	6.205	8.413
4W	12.917	12.251	6.951	8.66
LCV	10.145	12.2	8.382	8.677
HV	17.186	13.438	8.548	9.296
Bus	19.419	12.573	8.75	10.279
Bicycle	13.675	11.999	-	-

From the six intersections as shown in Tables 6.2 to 6.7, it is observed that two-wheelers and three-wheelers have less total delay as well as stopped delay because two-wheelers and three-wheelers are more forced to accept the available gap and also tend to accept shorter gaps.

HCVs and buses will have large delay because of their enormous size, and hence need more time to cross the intersection. LCVs and four-wheelers have medium delay compared to other vehicles because of their equivalent size. Finally, it is concluded that as traffic volume increases, there is an increase in the vehicular delay. Hence, vehicle size significantly affects the delay and as size of the vehicle increases, delay also increases.

The field delay is compared with the simulated delay for all the six intersections considered in this study. Table 6.8 and 6.9 shows the average total delay for all the intersections and Table 6.10 and 6.11 shows the average service/ stopped time delay for all the intersections. It is observed that, the field and the simulated values showed less than 10% error for all the vehicle types.

Table 6.8 Average total delay in seconds for 4-Legged Intersections

Vehicle Type	Intersection-1			Intersection -2			Intersection -3		
	Field delay	**Simulated delay**	**MAPE**	**Field delay**	**Simulated delay**	**MAPE**	**Field delay**	**Simulated delay**	**MAPE**
Bicycle	-	-	-	-	-	-	-	-	-
2W	43.36	60.798	0.003	35.160	44.429	0.004	32.250	47.335	0.005
3W	51.5	59.044	0.003	46.170	48.277	0.002	36.840	54.119	0.076
4W	63.87	60.801	0.002	45.150	48.327	0.008	42.500	51.823	0.010
Bus	78.05	57.118	0.054	49.640	46.691	0.036	49.150	52.327	0.096
LCV	63.06	56.116	0.019	46.470	47.108	0.006	49.760	50.203	0.003
HV	28.52	54.177	1.874	20.11	48.944	0.331	-	-	-
Tractor	60.53	65.472	0.060	48.6	47.012	0.018	-	-	-

Table 6.9 Average total delay in seconds for 3-Legged Intersections

Vehicle Type	Intersection -4			Intersection -5			Intersection -6		
	Field delay	**Simulated delay**	**MAPE**	**Field delay**	**Simulated delay**	**MAPE**	**Field delay**	**Simulated delay**	**MAPE**
Bicycle	19.479	18.109	0.003	-	-	-	11.999	20.752	1.403
2W	11.397	18.409	0.205	14.641	20.662	0.004	16.463	19.615	0.002
3W	14.764	21.588	0.014	16.461	22.519	0.024	20.520	20.260	0.000
4W	13.828	7.818	0.150	21.305	25.255	0.005	20.911	26.013	0.007
Bus	16.179	20.051	0.142	-	-	-	22.852	29.529	0.064
LCV	15.425	19.786	0.131	17.311	24.048	0.105	20.877	26.649	0.044
HV	16.385	18.806	0.138	23.469	24.659	0.026	22.734	28.088	0.039
Tractor	-	-	-	-	-	-	-	-	-

Table 6.10 Average service/stopped delay in seconds for 4-Legged Intersections

Vehicle Type	Intersection-1			Intersection -2			Intersection -3		
	Field delay	Simulated delay	MAPE	Field delay	Simulated delay	MAPE	Field delay	Simulated delay	MAPE
Bicycle	-	-	-	-	-	-	-	-	-
2W	26.460	19.551	0.002	24.020	30.188	0.004	25.230	20.928	0.002
3W	27.251	12.052	0.013	29.680	28.254	0.002	14.670	23.505	0.097
4W	39.560	15.961	0.025	29.750	24.002	0.023	9.290	22.196	0.064
Bus	41.780	10.200	0.151	26.660	18.215	0.194	3.890	19.680	6.058
LCV	33.630	13.378	0.105	27.850	25.920	0.029	10.950	22.194	0.403
HV	9.030	12.794	0.868	7.680	18.253	0.318	-	-	-
Tractor	24.080	24.755	0.020	24.640	30.109	0.124	-	-	-

Table 6.11 Average service/stopped delay in seconds for 3-Legged Intersections

Vehicle Type	Intersection -4			Intersection -5			Intersection -6		
	Field delay	Simulated delay	MAPE	Field delay	Simulated delay	MAPE	Field delay	Simulated delay	MAPE
Bicycle	14.048	29.180	0.048	-	-	-	13.675	8.333	0.751
2W	8.455	29.442	0.827	15.900	8.078	0.005	17.888	10.247	0.004
3W	4.989	31.055	0.153	11.811	8.401	0.019	18.428	10.175	0.012
4W	10.014	14.927	0.170	19.718	12.019	0.011	19.868	16.261	0.005
Bus	7.764	28.102	1.550	-	-	-	28.169	18.760	0.073
LCV	5.870	24.250	1.456	9.207	11.356	0.063	18.527	19.500	0.008
HV	4.936	21.093	3.059	9.094	10.296	0.067	25.734	18.707	0.045
Tractor	-	-	-	-	-	-	-	-	-

6.11 Summary

The VISSIM simulation is calibrated to the field volume for all the six intersections considered in this study. Based on the field data analysis, it is observed that majority of the total traffic at all the six intersections consists of only three vehicle types including 2W, 3W and 4W. Different vehicle compositions are considered for each vehicle type based on their observed field proportions. Total 88 compositions are prepared considering all the vehicle types and the same are simulated in the VISSM. From the results it is observed that the average delay of all the three intersections decreased as the vehicle composition changes from C1 to C11. Delay is significantly influenced by the size of the vehicle because large sized vehicle take more time to maneuver. It is observed that, total time taken to cross the intersection is observed to be higher for the traffic from minor road when compared to the traffic from the major road. Total delay for LCV, Bus and Bicycle are high while, total delay for two-wheeler, three-wheeler, and four-wheeler is observed to be less. Finally, the field delay data is compared with the simulated data, and observed error is less than 10% for all the vehicle types.

CHAPTER 7

CAPACITY ESTIMATION AND PERFORMANCE EVALUATION

7.1 General

Capacity at unsignalized intersections is defined as a result of the basic capacity within ideal traffic conditions related to various adjustment and correction factors, which included the impact of road environment, geometric design, and traffic conditions. Unsignalized intersections are defined as the intersections where traffic operates on the basis of the priority of traffic movements. The left-turning movement (in contrast with the straight on or right-turn movements) from the minor street has, for example, the lowest priority according to the corresponding traffic laws in many countries. The performance of an unsignalized intersection is strongly influenced by the delay caused by low-priority movements on minor roads. They are the major source for vehicular conflict resulting in delay, accidents and congestion. User cost and delay can be reduced by improving the design and operation of the unsignalized intersection. Improvement in design and operation largely depends on how accurately capacity and delay are estimated in response to alternative policies and design.

The capacity of uncontrolled intersections is estimated by three methods. The methods developed to determine the capacity of uncontrolled intersection are mainly based on the calculation of critical gap and follow up time of minor stream vehicles. Gap-Acceptance method (GAP) was developed in Germany (Harders, 1968) and is widely used in United States and several European countries. This method is based on the critical gap acceptance and follow-up times of vehicles from the minor road. Other countries like Sweden and Germany also use the GAP method in their own capacity manuals. It includes mathematical calculations and it can be used for different types of vehicles and involves less number of surveys.

Conflict technique is considered to be the easier method to handle. This method is based on "addition of critical movement flows". This method considers all the traffic streams and conflict points at intersections simultaneously. It includes a simplified concept where interaction and impact between flows at intersection is mathematically formulated. The Empirical Method was developed in UK (Kimber and Coombe, 1980), and is based on regression analysis on field data collected from modern British streets to get the representative result and it require very large amount of data, for which vast surveys are required.

Hence, comparing the three methods, GAP is more suitable for mixed traffic conditions because it can be used for different composition of vehicles. Hence, GAP method is considered to be more suitable for the current work.

7.2 Gap Acceptance Capacity Models

Even though there are several gap acceptance capacity models that are discussed in the literature review, only four models are considered in this study including Tanner's model, Drew's model, modified Siegloch model, and Luttenin model. These models are selected based on the condition that all these four models satisfied the unique criteria that is specific to each type of intersection especially for mixed traffic conditions.

7.2.1 Tanner's Model

Tanner (1962) developed a model [Equation (7.1)] for finding the capacity of unsignalized intersections:

$$C_p = \frac{q_M(1-\lambda t_p)e^{-\lambda(t_c-t_p)}}{1-e^{-\lambda t_f}}, \quad (7.1)$$

where, $\lambda = q_m/3600$ (veh/s),

t_p= minimum headway in the major traffic stream,

t_c= critical gap,

q_m= number of major stream headways, and

t_f = Follow-up time.

7.2.2 Drew's Method

Drew (1968) developed the Equation (7.2) to find the capacity of unsignalized intersections.

$$C = \frac{v_p e^{-v_p * t_c}}{e^{-v_p * t_c}}, \tag{7.2}$$

where, v_p= major traffic flow rate (veh/sec),

t_c= critical gap, and

t_f= follow-up gap respectively.

7.2.3 Modified Siegloch Model

Siegloch modified (1974) the capacity equation by assuming the major stream headways as exponentially distributed, and the modified version is given by Equation (7.3).

$$C = \frac{3600}{t_f} e^{-\frac{q_c}{3600}(t_c - \frac{t_f}{2})}, \tag{7.3}$$

where, q_c = conflicting flow,

t_c = critical gap (s), and

t_f = follow-up time (s).

7.2.4 Luttenin's Model

Luttenin's model (1990) as shown by Equation (7.4) is used to calculate the minor stream capacity.

$$C_p = \frac{q_M \exp\left(\frac{q_M(t_c - t_p)}{3600 - q_M t_p}\right)}{1 - \exp\left(\frac{-q_M t_f}{3600 - q_M t_p}\right)} \quad , \tag{7.4}$$

where, λ= q_M/3600 (veh/s),

t_p= minimum headway in the major traffic stream,

t_c= critical gap,

q_M = number of major stream headways, and

t_f= follow-up gap.

7.3 Indo-HCM

Capacity (C_x) for any movement at an uncontrolled intersection is computed based on the gap acceptance model as presented in Equation (7.5). Capacity of a movement can be deduced from the estimated values of critical gap, follow-up time and conflicting flow rates. The adjustment factors in the equation are to be taken based on the intersection geometry as shown in the Table 7.1.

$$C_x = \frac{a * V_{c,x} * e^{-V_{c,x}(t_{c,x} - b)/3600}}{1 - e^{-V_{c,x} * t_{c,x}/3600}}, \quad (7.5)$$

where, C_x= Capacity of movement 'x' (in PCU/h),

$t_{c,x}$= critical gap of standard passenger cars for movement 'x' (s),

$t_{f,x}$= follow-up time for movement 'x' (s),

$V_{c,x}$= Conflicting flow rate corresponding to movement 'x' (PCU/h), and

'a' and 'b' = adjustment factors based on intersection geometry

Table 7.1 Adjustment factors for capacity model

Major Street Configuration	**Adjustment factors**	**Subject Movement**		
		Right Turn from Major	**Right Turn from Minor**	**Through on Minor**
Four-lane divided	a	0.80	1.00	0.90
	b	1.30	2.16	5.04
Two-lane undivided	a	0.70	0.80	1.10
	b	-0.11	0.72	0.72

7.4 HCM 2010

The HCM 2000 method also explains the gap acceptance theory for the computation of the capacity and level of service of the All Way Stop Controlled (AWSC) intersection. According to the geometric group, the saturation headway and departure headway are calculated using different adjustment factors and using exhibits in HCM 2000. The pedestrian impedance factors are also considered from which the capacity and level of service are determined for Two Way Stop Controlled (TWSC) intersections. The method explained in this report is based on the AWSC intersection.

7.5 Estimation of Capacity of Urban Uncontrolled Intersections

The theory of gap-acceptance is the predominant concept for unsignalized intersections analysis. This method is based on critical gap acceptance and follow up times of right turning vehicles from the minor road. Four gap acceptance models are used for determining the capacity of urban uncontrolled intersections which includes Tanner's model, Drew's model, modified Siegloch model, and Luttinen's model, as the selected intersections are satisfying the underlying conditions behind these chosen models. Also, HCM 2010 and Indo-HCM are used for determining the capacity of urban uncontrolled intersections. The capacity of minor stream right turning vehicles based on field and simulated data are calculated for the above six capacity models and are shown respectively in Tables 7.2 and 7.3. Similarly, the capacity of major stream right turning vehicles based on field and simulated data are calculated for the above six capacity models and are shown respectively in Tables 7.4 and 7.5.

Tanner proposed a theoretical model to relate various parameters connected with the delay problem in dealing with an intersection of a major and minor road and for finding the capacity of minor stream. Tanner's model is developed for single lane Major Street for calculating the capacity of right turning vehicles, whereas modified tanner's model is developed considering n number of vehicles in the major stream. Modified Tanner's model considers the following parameters: minimum headway and number of major stream headways, critical gap, and follow-up time. Luttenin's model considers the following parameters: major stream flow rate, critical gap, follow-up time, and the minimum headway. Drew's model considers the following parameters: major traffic flow rate (veh/hr), major stream flow rate (veh/s), critical gap, and follow-up time.

Table 7.2 Capacity of minor stream right turning vehicles based on field data

Parameters	Intersection-1		Intersection-2		Intersection-3		Intersection-4	Intersection-5	Intersection-6
	NB	**SB**	**NB**	**SB**	**NB**	**SB**	**SB**	**SB**	**SB**
Tanner's Model (veh/h)	762	774	541	573	670	497	649	568	754
Drew's Model(veh/h)	666	575	521	478	563	535	564	609	718
Modified Siegloch Model (veh/h)	585	584	601	451	549	953	638	659	804
Luttinen's Model (veh/h)	586	517	513	464	361	568	522	453	512
HCM 2010 (veh/h)	544	560	488	520	688	896	526	413	405
Indo-HCM (PCU/h)	241	259	503	549	859	885	546	343	64

Table 7.3 Capacity of minor stream right turning vehicles based on simulated data

Parameters	Intersection-1		Intersection-2		Intersection-3		Intersection-4	Intersection-5	Intersection-6
	NB	**SB**	**NB**	**SB**	**NB**	**SB**	**SB**	**SB**	**SB**
Tanner's Model (veh/h)	672	717	522	490	720	535	651	568	754
Drew's Model(veh/h)	591	539	487	404	608	579	558	561	703
Modified Siegloch Model (veh/h)	541	558	608	387	584	1223	638	659	804
Luttinen's Model (veh/h)	504	478	471	384	451	630	513	377	485
HCM 2010 (veh/h)	492	612	376	512	1064	748	499	416	418
Indo-HCM (PCU/h)	222	235	424	435	670	687	498	274	126

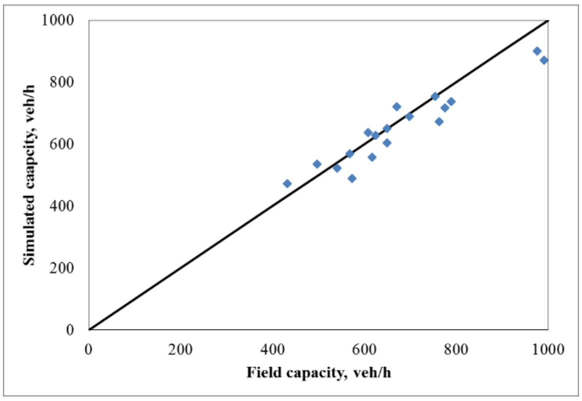
Table 7.4 Capacity of major stream right turning vehicles based on field data

Parameters	Intersection-1		Intersection-2		Intersection-3		Intersection-4	Intersection-5	Intersection-6
	EB	**WB**	**EB**	**WB**	**EB**	**WB**	**WB**	**WB**	**WB**
Tanner’s Model (veh/h)	990	975	649	616	608	432	624	788	697
Drew’s Model(veh/h)	863	717	601	549	565	471	511	735	732
Modified Siegloch Model (veh/h)	819	798	609	552	659	883	599	790	890
Luttinen’s Model (veh/h)	760	643	583	539	442	503	456	574	535
HCM 2010 (veh/h)	1496	1373	478	576	375	857	553	1412	2021
Indo-HCM (PCU/h)	728	800	923	1042	1313	1339	981	848	674

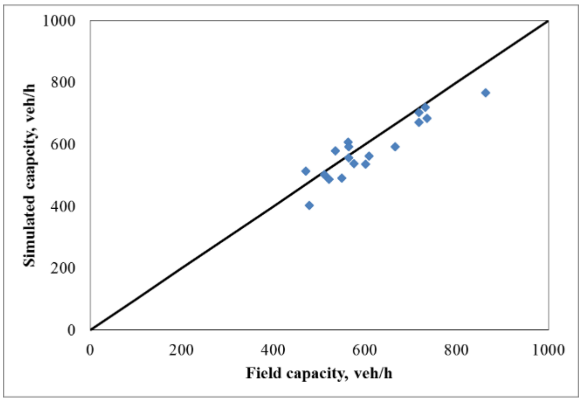
Table 7.5 Capacity of major stream right turning vehicles based on simulated data

Parameters	Intersection-1		Intersection-2		Intersection-3		Intersection-4	Intersection-5	Intersection-6
	EB	**WB**	**EB**	**WB**	**EB**	**WB**	**WB**	**WB**	**WB**
Tanner’s Model (veh/h)	872	901	604	558	637	473	627	737	690
Drew’s Model(veh/h)	767	671	536	492	592	513	502	684	719
Modified Siegloch Model (veh/h)	685	701	588	512	655	1124	599	790	890
Luttinen’s Model (veh/h)	653	593	509	476	514	555	445	490	508
HCM 2010 (veh/h)	1514	1341	588	674	557	772	515	1205	1551
Indo-HCM (PCU/h)	733	789	918	942	1158	1199	928	776	692

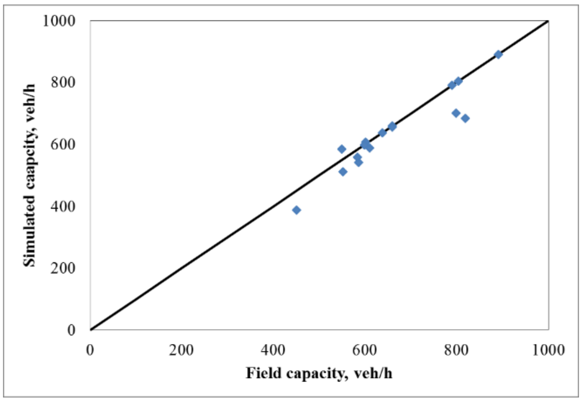
Modified Siegloch model considers the following parameters: conflicting flow rate, critical gap, and follow-time, while Siegloch model is applied only for saturated conditions that are very difficult to find in many practical cases. HCM (2010) is an iterative process for which the input data from the field is used and the methodology is applied through a set of four worksheets that involves saturation headways, departure headways, service time, capacity, and level of service. In HCM 2010, the capacity values are different because it is dependent on heavy vehicle composition and total traffic flow. Indo-HCM is more empirical in nature, where adjustment factors are considered. Priority rankings are given for different movements and based on these rankings, the conflicting flow is calculated for each movement. The adjustment factors are given for different turning movements for both 4-lane divided and 2-lane undivided major streets. The capacity very much depends on these adjustment factors because the same adjustment factors are considered in the current study for all the six intersections. The existing capacity calculated using Indo-HCM is observed to be much different from the flow observed on the field study that resulted in higher MAPE values. However, the capacities estimated by the Tanner's method and modified Siegloch method are nearly equal but, in modified Siegloch method, conflicting volume is inversely affecting the capacity of the intersection. Whereas, in Tanner's model the minimum headway of major stream traffic is greatly influencing the capacity. Tanner's model is more realistic in nature and resulted in closer capacities for both field and simulated data. The field capacity and the capacities estimated using the above six models are compared in Figure 7.1. It is observed that for the existing heterogeneous traffic conditions prevailing at all the six intersections, Tanner's model is resulting in better capacity estimation when compared to other models considered in this study as shown in Table 7.6.

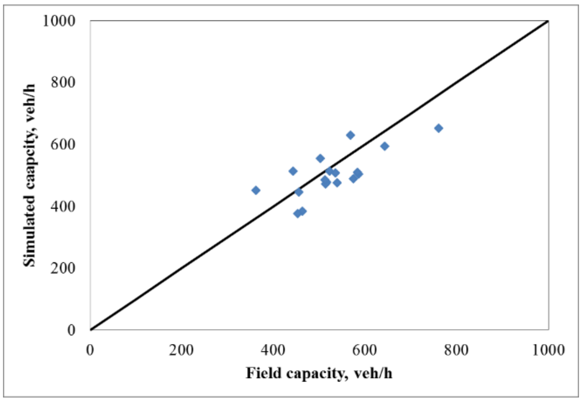


(a)

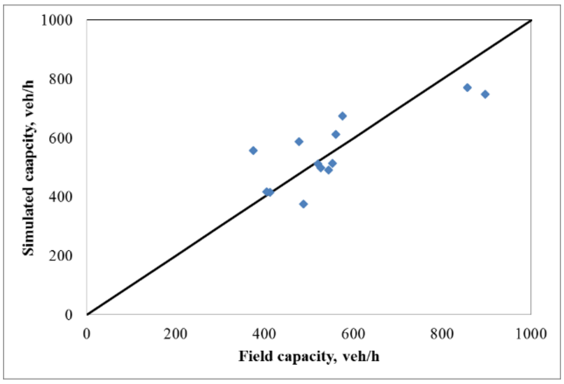


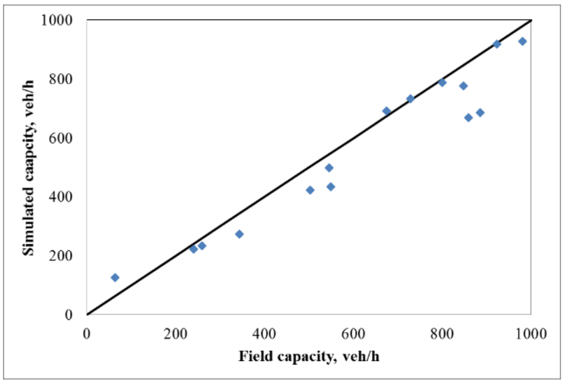
(b)





(c) (d)





(e) (f)

Figure 7.1 Comparison between field and simulated capacity using (a) Tanner's model, (b) Drew's model, (c) modified Seilogch's model, (d) Luttenin's model, (e) HCM 2010 method, and (f) Indo-HCM model.

Table 7.6 MAPE in % obtained from the six models

Model	MAPE
Tanner's Model	6.18
Drew's Model	7.22
Modified Siegloch Model	7.16
Luttinen's Model	11.21
HCM 2010	15.02
INDO-HCM	15.27

7.6 Performance of the Urban Uncontrolled Intersections

The performance of the six intersections evaluated through LOS criteria for both minor road and major road right turning vehicles based on delay and v/c are shown respectively in Tables 7.7 and 7.8.

HCM 2010 resulted in same LOS (F) for all the approaches at all the six intersections considered in this study. LOS (F) is defined when the service delay per vehicle is more than 50 s. It is observed that for all the approaches at all the six intersections considered in this study, the service delay per vehicle is exceeding 50 s resulting in LOS (F). However, Indo-HCM defines LOS based on v/c. It is observed that, the performance of each of the approaches at all the six intersections could be quantified clearly using the v/c as recommended by Indo-HCM.

Table 7.7 LOS of minor road right turning vehicles based on delay and v/c

Method	Intersection-1		Intersection-2		Intersection-3		Intersection-4	Intersection-5	Intersection-6
	NB	**SB**	**NB**	**SB**	**NB**	**SB**	**SB**	**SB**	**SB**
HCM 2010	F	F	F	F	F	F	F	F	F
Indo-HCM	A	C	A	A	A	A	F	E	F
Tanner's Model	B	B	B	B	B	B	D	C	B

Table 7.8 LOS of major road right turning vehicles based on delay and v/c

Method	Intersection-1		Intersection-2		Intersection-3		Intersection-4	Intersection-5	Intersection-6
	EB	**WB**	**EB**	**WB**	**EB**	**WB**	**WB**	**WB**	**WB**
HCM 2010	F	F	F	F	F	F	F	F	F
Indo-HCM	C	C	A	A	B	A	E	E	F
Tanner's Model	C	B	B	B	B	D	A	B	E

It is important to note that, adjustment factors are used for critical gap calculation and based on these critical gap values the follow up time is calculated. Also, high priority is given to through movement on the major road for calculating the conflicting flow for all the right turning vehicles and adjustment factors are also provided for the intersection geometry in Indo-HCM. Based on these conditions, capacity calculated using Indo-HCM is overestimating resulting in less v/c ratio. Thus, the LOS computed using Tanner's model is more realistic than the LOS computed using Indo-HCM. Also, the variation in the performance of all the intersections using Tanner's model could be seen clearly when compared to Indo-HCM.

Several operational challenges were observed at the six urban uncontrolled intersections selected for the current study. As the traffic is mixed in nature, some of the unique operational characteristics were observed at these intersections including the tendency of the small-sized vehicles to penetrate into the available gaps while crossing an intersection, one single gap within the major stream traffic flow is being accepted by several number of small-sized vehicles that move parallel to each other, and most of the vehicles accept the gap in a zig-zag manner. Such unique operational characteristics especially by the small-sized vehicles including two-wheelers and three-wheelers significantly affected the critical gaps and in turn the capacity. This shows that small-sized vehicles' drivers tend to accept higher risk. As the Tanner's model also takes into account the minimum headway of the major stream traffic, the capacity estimated using this method is more realistic with minimum error. Further, as discussed earlier, there are several adjustment factors that are considered in the capacity estimation using the Indo-HCM method. In the current study, the performance of all the six urban uncontrolled intersections were computed in terms of LOS based on service delay and volume-capacity ratio. Thus, the LOS computed based on the Tanner's model realistically takes into account the unique operational characteristics of the small-sized vehicles and is observed to be much better than the LOS computed using the HCM (2010) and Indo-HCM methods. The higher risk accepting tendency of the small-sized vehicle drivers can be controlled to a greater extent through continuous surveillance and by ensuring proper enforcement. Suitable policy decisions need to evolved to identify the high risk accepting drivers essentially at the urban uncontrolled intersections so that such drivers can be penalized suitably. Further, it is essential to educate such drivers about the possible impacts of their driving behaviour on the safety of the road users in general and the performance of the intersection in particular.

7.7 Summary

Six different models are used in this study to estimate the capacity of urban uncontrolled intersections. Based on the MAPE, it is observed that the Tanner's model is better than other models considered in this study. Performance of all the six intersections is evaluated through LOS. It is observed that HCM fails to differentiate between the performances of the six intersections in terms of the LOS. The performance of the six intersections obtained from the v/c through the Tanner's model is observed to be better when compared to the performance of these intersections obtained from the v/c through the Indo-HCM method.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Summary

At an uncontrolled intersection, the vehicular interactions are very complex. Most of the vehicles move without following the rules of priority. Hence, for these intersections the critical gap estimation and capacity estimation is very difficult in the mixed traffic scenario. Generally, most of the vehicles accept the gap in a "zig-zag" manner making it very difficult to determine the critical gap. Gap estimation is an integral part of capacity estimation at uncontrolled intersections. Most of the studies on critical gap estimation are reported for homogenous traffic conditions and where the rules of priority are truly followed.

In this study, the accepted and rejected gaps are determined for each and every vehicle taking right turn from both the major road and the minor road. The accepted and rejected gaps are categorized into different vehicle combinations for the major stream. For each of the combinations, critical gaps are determined with respect to the subject vehicle taking right turn from both the major road and the minor road. These critical gaps are then grouped for each vehicle type and regrouped into leg-wise critical gap.

Microscopic VISSIM simulation is used to show the eventual real effects of alternative scenarios. Different vehicle compositions are considered by changing the vehicle types with an increment of 10% starting from 0% to 100%. The proportion of each vehicle type for each selected composition is considered in such a way that these proportions matches with the observed field proportion. Thereby, the effect of traffic composition on delay and volume at urban uncontrolled intersections is studied for all the vehicle types. Finally, the field delay data is compared with the simulated delay data for the total delay and the service delay. It is observed that the error is less than 10% for all the vehicle types.

Capacity at uncontrolled intersections is estimated using the gap acceptance models including Tanner's model, Drew's model, modified Sieogloch's model, and Luttenin's model. Further, the capacity is also estimated using the HCM (2010) and Indo-HCM methods. Based on the

MAPE, it is observed that Tanner's model is observed to be the best among the selected models for determining the capacity at urban uncontrolled intersections. Also, the performance of each of the six intersections is evaluated using the LOS criteria. HCM (2010) failed to differentiate between the performances of the six intersections as the service delay per vehicle is much more than 50 s. However, the LOS evaluated using the v/c resulted in significant variation in performance of the six intersections.

8.2 Conclusions

The conclusions drawn from this study are presented below:

i. In general, irrespective of the vehicle combinations, the critical gap increased with increase in size of the vehicle. The critical gaps observed at all the six intersections ranged from 1.7 s to 4.45 s for 2w, 1.9 s to 4.9 s for 3w, 2.2 s to 4.8 s for 4w, 2.2 s to 4 s for LCV, 3.85 s to 4.8 s for tractor, 2.65 s to 4.5 s for Bus, 3 s to 6.2 s for HV, and 3.6 s to 5.1 s for Bicycle. However, it is observed that because of the lower volumes of right turning vehicles at Intersections- 2 and 3, the effect of vehicle size could not be related to the critical gap. From the critical gap obtained for the right turning vehicles from minor road to major road, 3w are observed to be more aggressive than 2w as the critical gap accepted by 3w is less than the critical gap accepted by the 2w.

ii. The critical gap for major road right turning vehicles is observed to be higher when compared to the minor road right turning vehicles. For example, at intersection-6, a significant critical gap of 6.2 s is observed for the major road right turning heavy vehicles whereas the critical gap is only 3 s for minor road right turning heavy vehicles. It is observed that, 2w and 3w are accepting shorter gap, 4w and LCVs are accepting moderate gap, whereas HCVs and Buses accepted larger critical gaps. Also, the critical gap decreased with increase in the traffic volume.

iii. Using ANOVA test, significant variation is observed between the critical gap values for all the vehicles types at Intersections- 1, 4, 5 and 6 based on the p-value ($p < 0.05$). Whereas, Intersections- 2 and 3 did not show significant variation in the critical gap values for all the vehicle types considered in the study as the p-value is greater than 0.05. The accepted gaps at most of the intersections followed gamma distribution and available gaps followed lognormal distribution.

iv. When vehicle combinations are considered, it is observed that with increase in size of the major stream vehicle combinations, the distance required to cross, i.e., the gap length also increases resulting in higher critical gaps for large vehicle combinations. The size of the following vehicle plays a major role in the gap acceptance behaviour of the right turning subject vehicles. For the right turning minor and major stream vehicles, with increase in size of the vehicle, the critical gap increases. Smaller sized vehicles tend to accept the smaller gaps whereas large size vehicles require higher gaps to perform the right turning maneuver.

v. Based on the VISSIM simulation it is observed that with increase in proportion of bicycles, the intersection volume decreases and the overall delay at the intersection increases. Hence, the performance characteristics of the bicycles plays a major role when compared to the dimensional characteristics. The performance characteristics of the bicycles such as speed plays a major role in terms of affecting the volume and delay at the intersection.

vi. It is observed that for 2w, 3w and 4w, with increase in proportion of these vehicle types, there is an increase in overall volume of the intersection. Because of the lower projected areas and higher acceleration characteristics, the number of vehicles per unit time increases at the intersection i.e., the overall volume of the intersection increases thereby the delay decreased for 3w and 4w, whereas the delay increased for 2w. This is due to the fact that with increase in proportion of 2w, the available gap between these vehicles decreases resulting in an increase in overall delay.

vii. For remaining all vehicle types including LCV, HV, Buses and Tractors, it is observed that with increase in proportion of these vehicle types the overall volume of the intersection decreases and at the same time the intersection delay also decreases. Since, all these vehicle types will have relatively higher projected areas when compared to the other vehicle types and with relatively lower acceleration characteristics, result in less number of vehicles per unit time and subsequent decrease in overall volume of the intersection.

viii. As traffic volume increases, there is an increase in the vehicular delay. Also, as size of the vehicle increases, the total delay and service delay also increases. It is observed that, on comparing the average total and service delays observed in the field with the corresponding simulated delays, the error is less than 10% for all the vehicle types.

ix. Compared to all the existing models, Tanner's model is observed to be better for calculating the capacity of all the right turning vehicles as the MAPE obtained for the Tanner's model is the least when compared to other models considered in this study. The Tanner's model is observed to be more realistic as this model considers all vehicle types typically observed in the mixed traffic conditions. It is important to note here that, even though Indo-HCM method of capacity estimation is specifically developed for mixed traffic conditions, it is more empirical in nature where adjustment factors are considered and this method also gives priority ranking for different movements that are subsequently used for calculating the conflicting flow. The findings of this study are expected to improve the accuracy with which the capacity of an urban uncontrolled intersection can be estimated.

x. HCM 2010 resulted in same LOS (F) for all the approaches at all the six intersections. This is due to the fact that the LOS (F) is defined when the service delay per vehicle is more than 50 s. It is observed that for all the approaches at all the six intersections considered in this study, the service delay per vehicle is exceeding 50 s resulting in LOS (F). The performance of the six intersections obtained from the volume-capacity ratio through the Tanner's model is considered to better when compared to the Indo-HCM method. In Indo-HCM, adjustment factors and priority movements are considered which is overestimating the capacity resulting in less v/c ratio.

8.3 Limitations of the Study

Following are the limitations of the present study:

i. This study ignores the driver characteristics including age of the driver, gender and driver behavioral characteristics.

ii. This study focussed only on the motorized traffic and the effect of pedestrians is ignored.

8.4 Scope for Further Work

The current research work can be extend further by taking into account the following points:

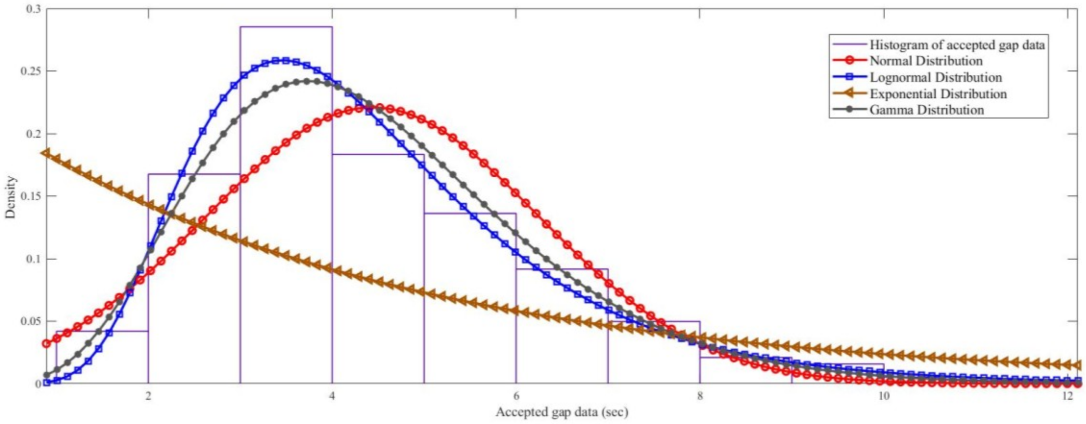
i. The effect of parking, bus-stops near the intersections, side friction, and pavement condition can be explored.

ii. The effect of pedestrian-vehicle interaction on the gap acceptance behaviour at uncontrolled intersections can be studied.

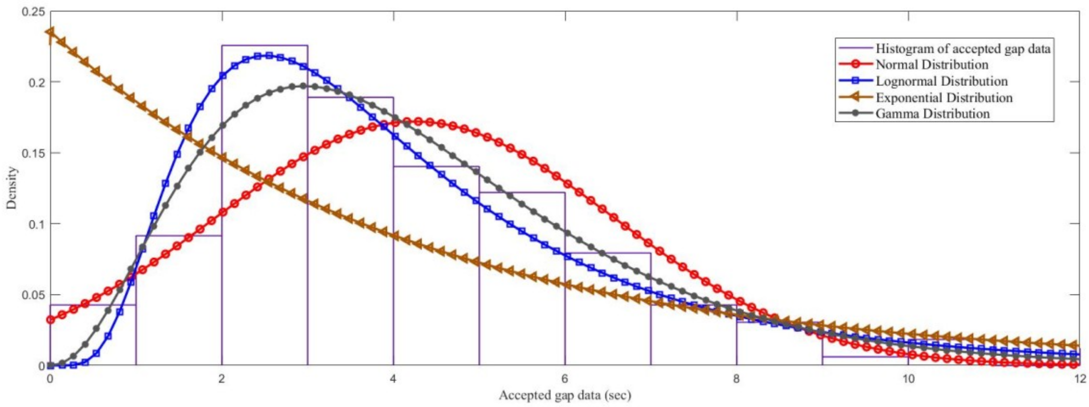
iii. Influence of weather conditions, off-peak traffic conditions, presence of median on gap acceptance behaviour can be studied.

APPENDIX-A

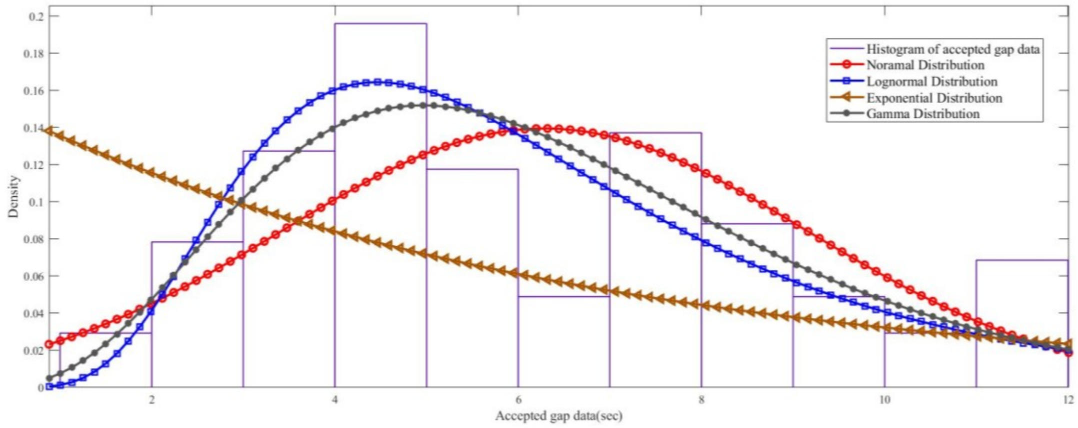
A.1 Distribution of Gaps



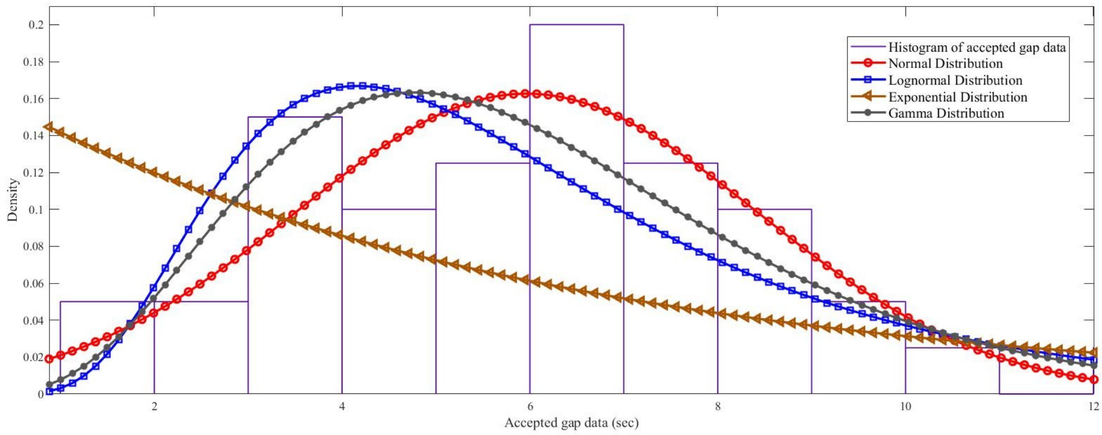
(a) EB for Intersection-1



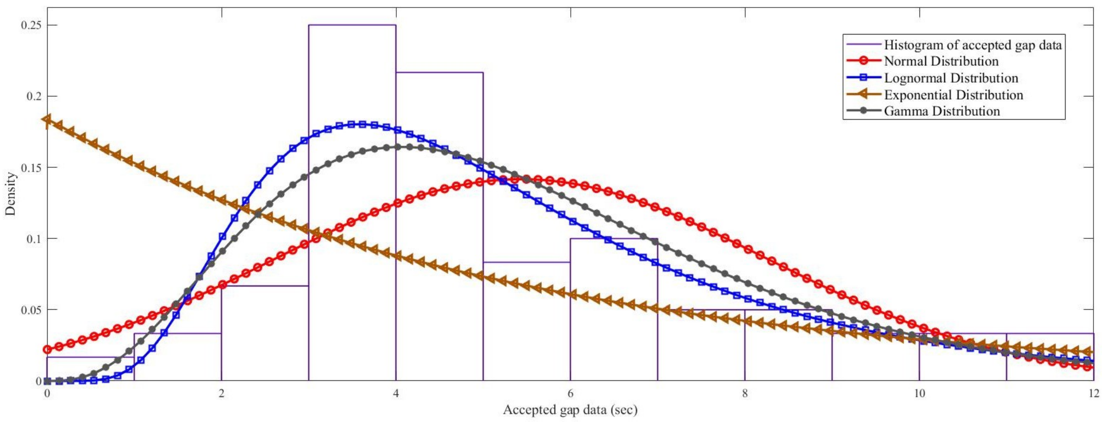
(b) WB for Intersection-1



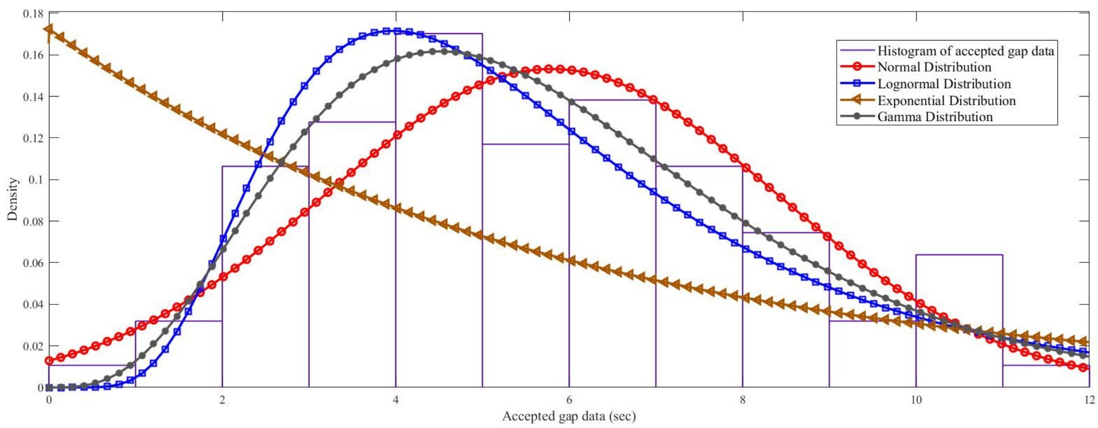
(c) EB for Intersection-2



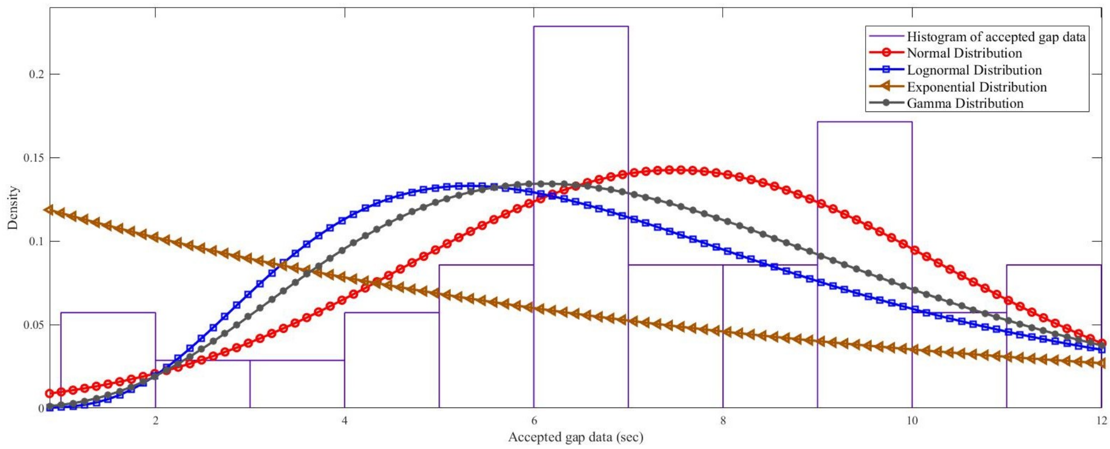
(d) WB for Intersection-2



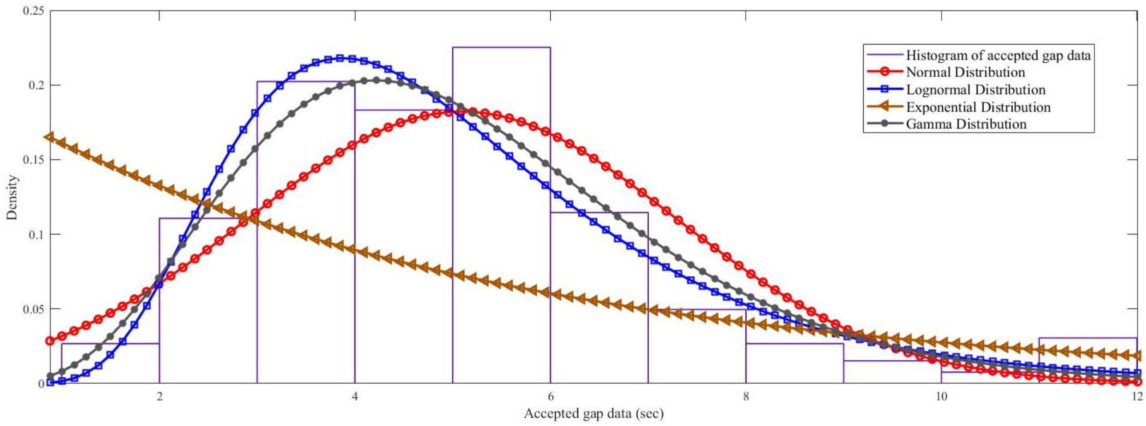
(e) EB for Intersection-3



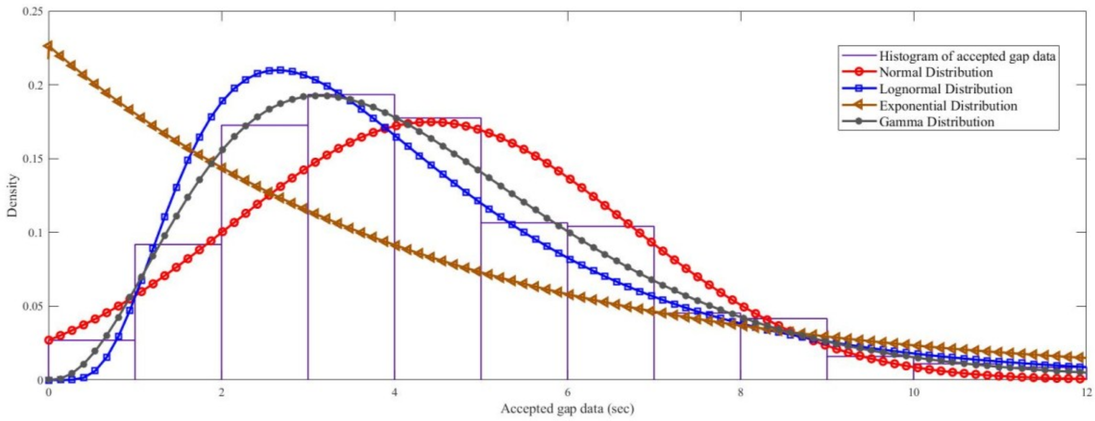
(f) WB for Intersection-3



(g) WB for Intersection-4

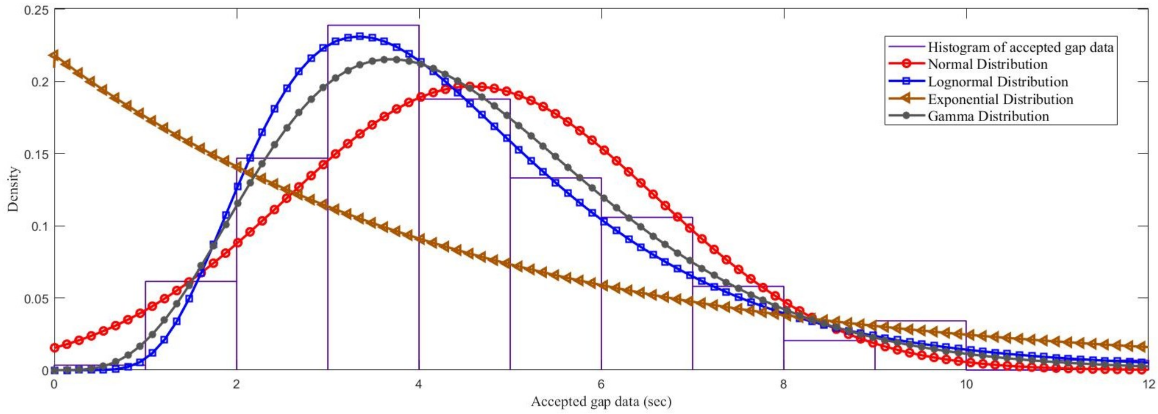


(h) WB for Intersection-5

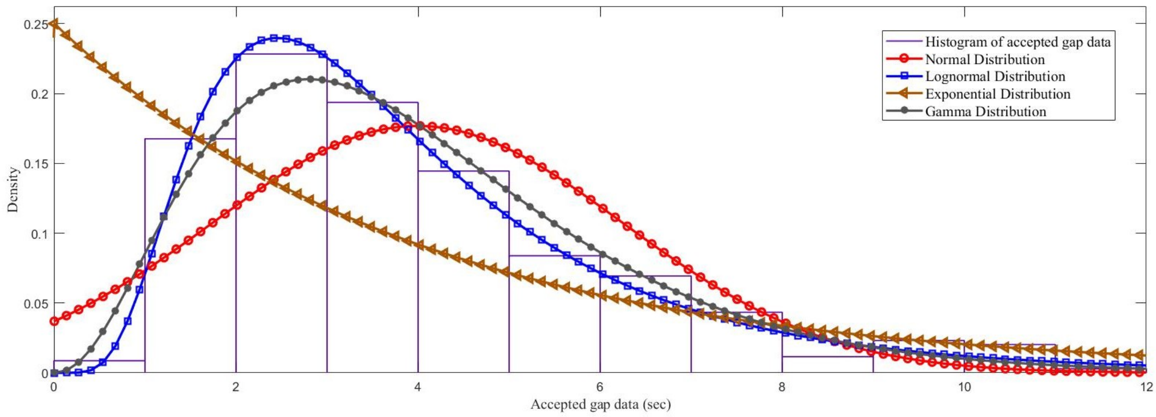


(i) WB for Intersection-6

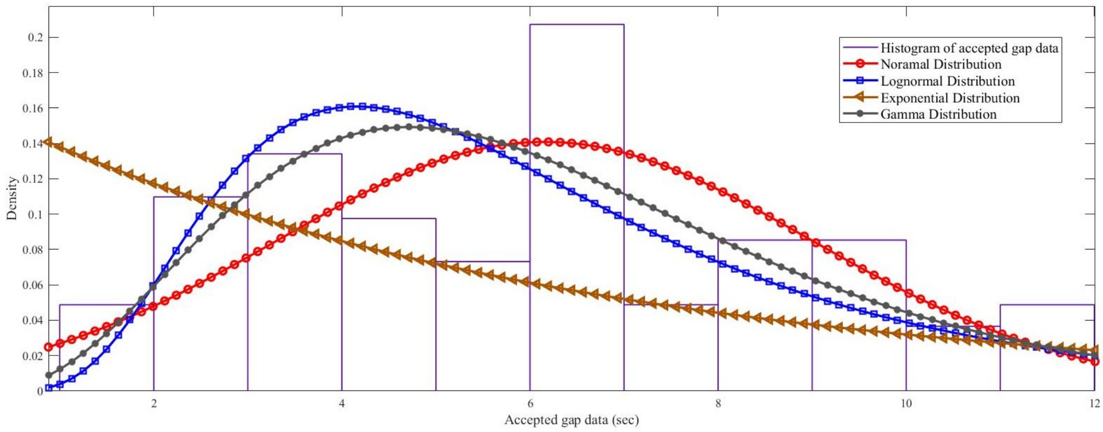
Figure A.1 Distributions fitted to accepted gap data for major road right turning vehicles



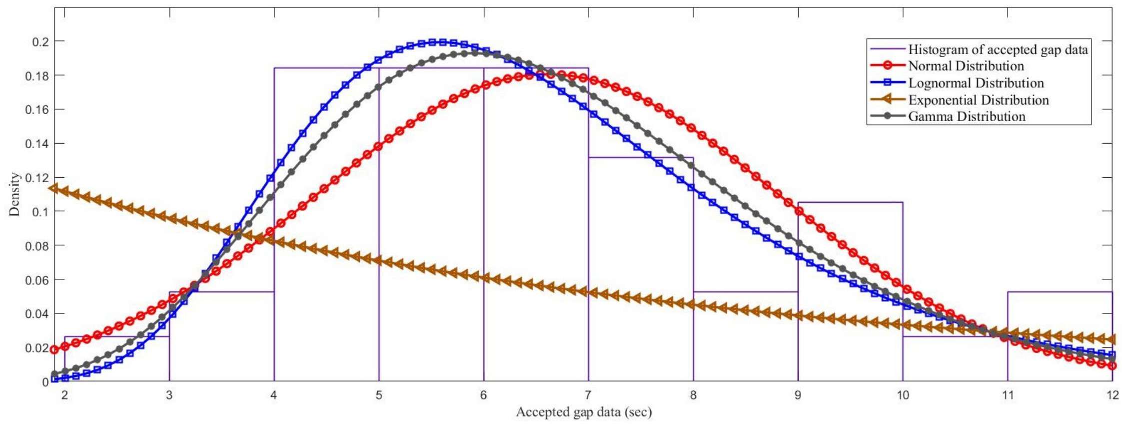
(a) NB for Intersection-1



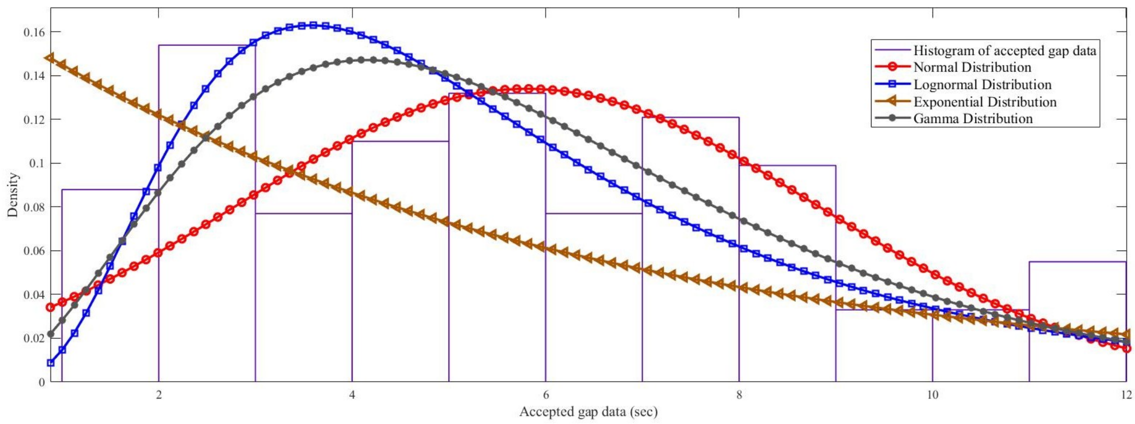
(b) SB for Intersection-1



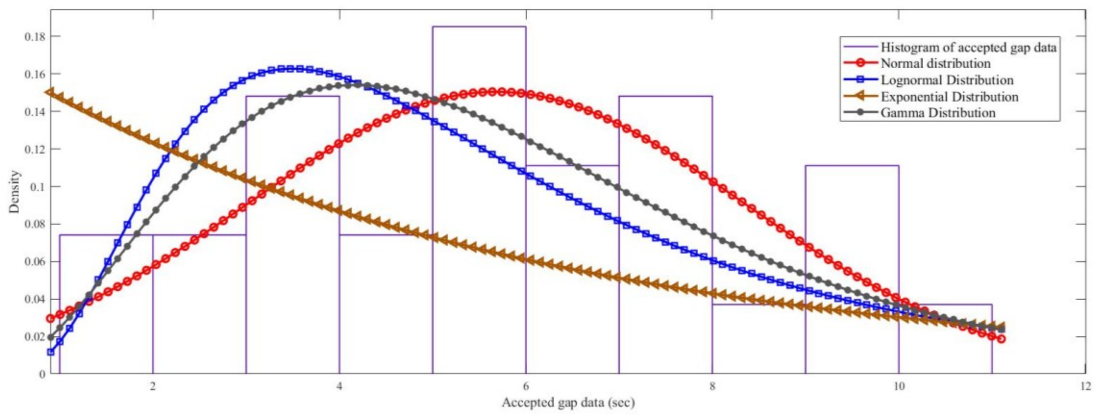
(c) NB for Intersection-2



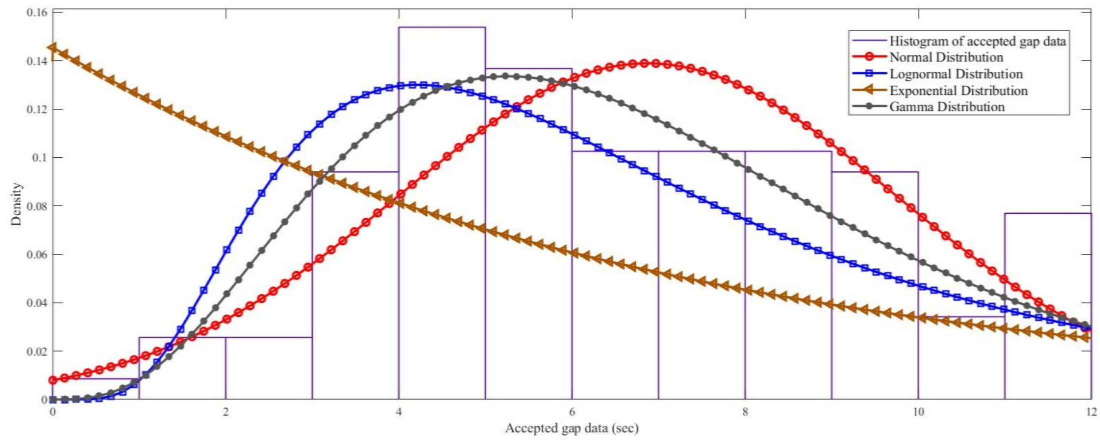
(d) SB for Intersection-2



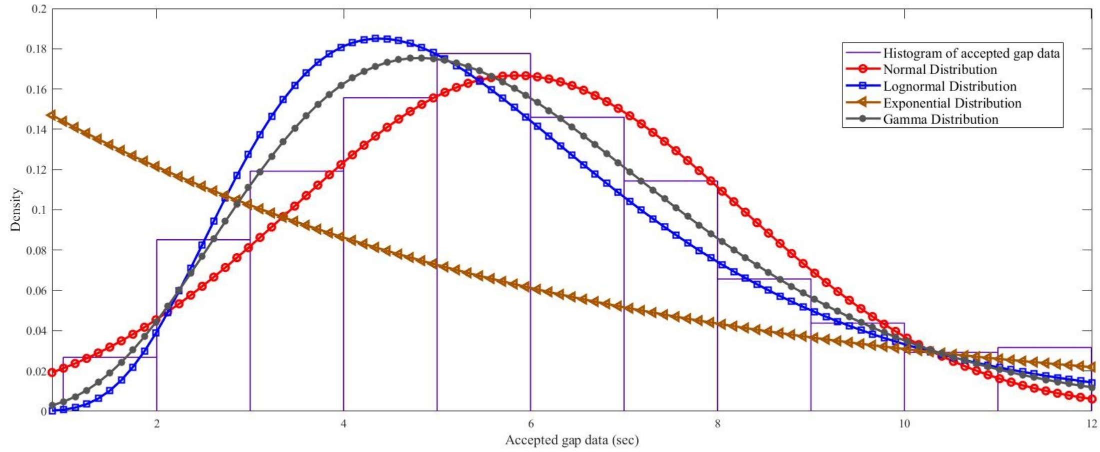
(e) NB for Intersection-3



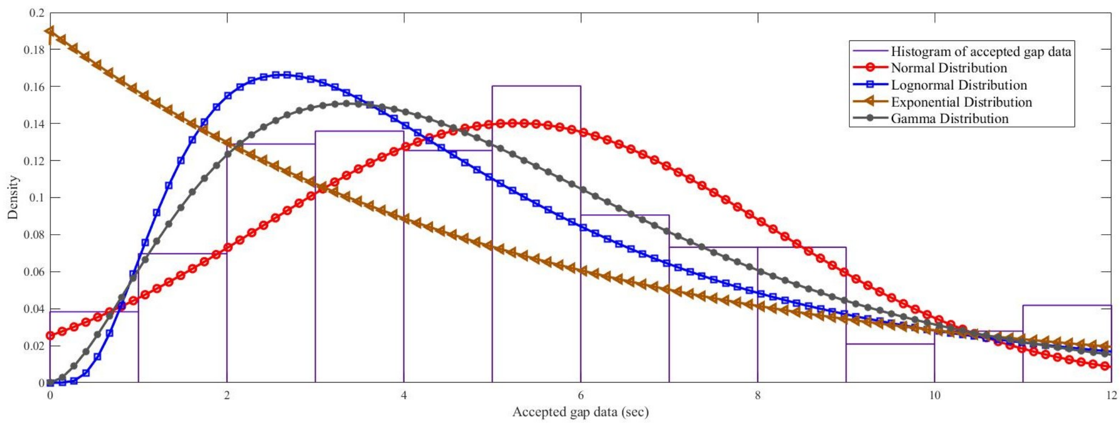
(f) SB for Intersection-3



(g) SB for Intersection-4

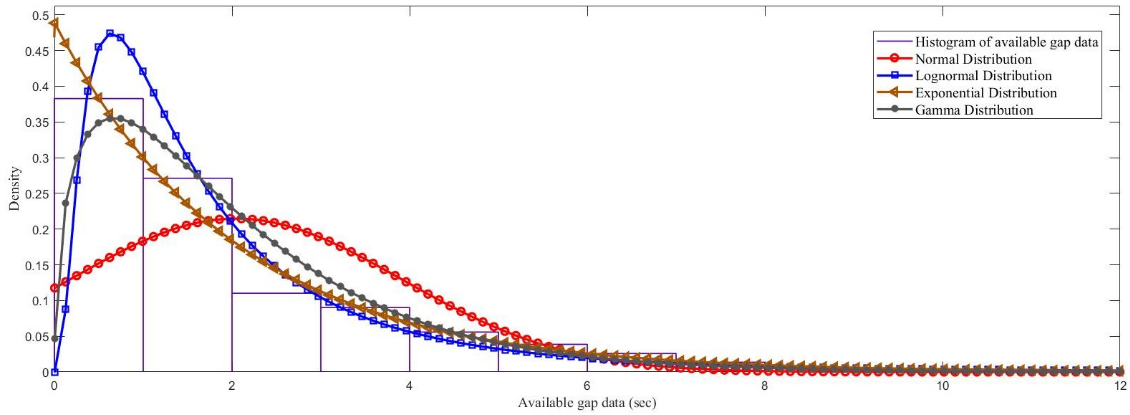


(h) SB for Intersection-5

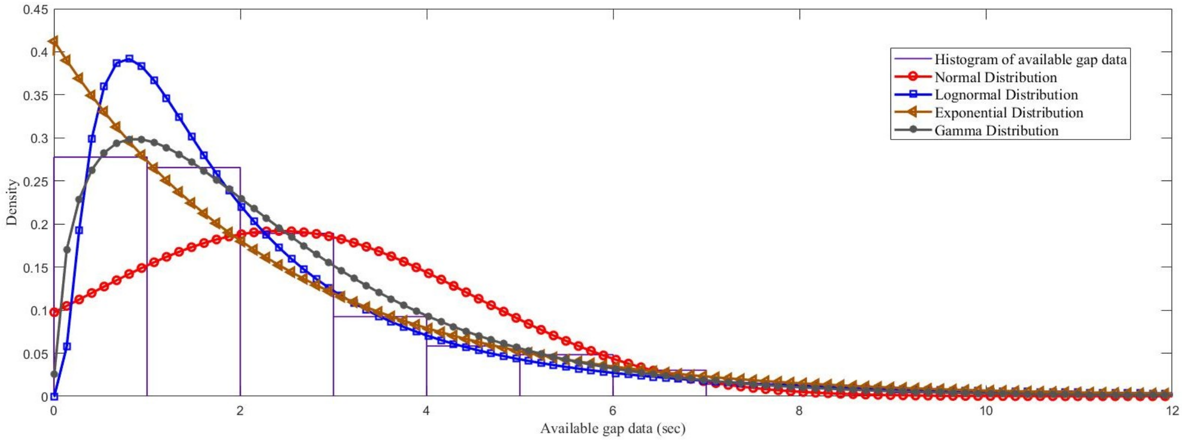


(i) SB for Intersection-6

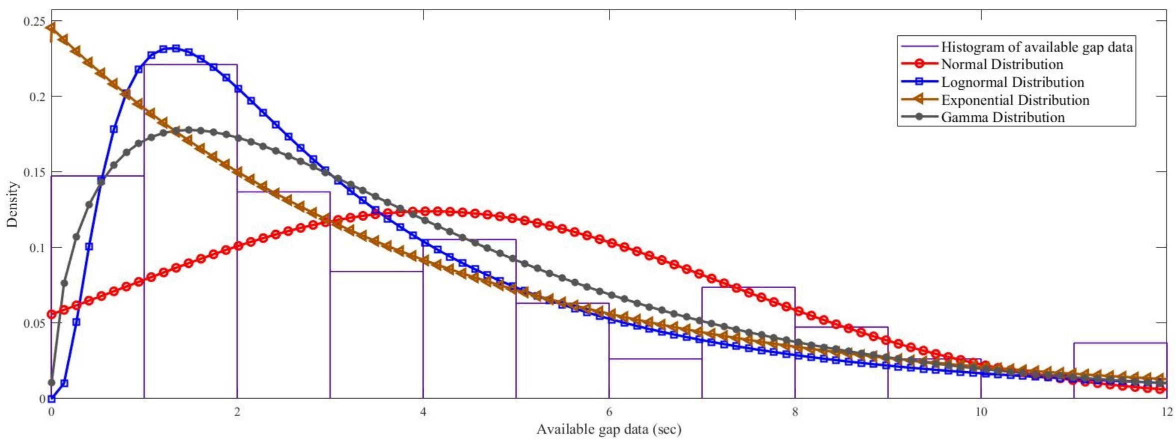
Figure A.2 Distributions fitted to accepted gap data for minor road right turning vehicles



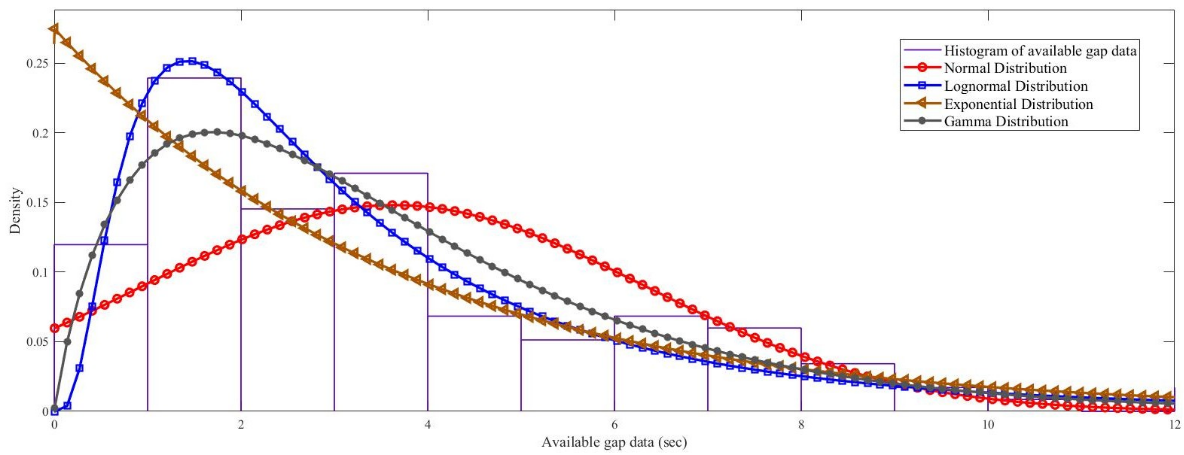
(a) EB for Intersection-1



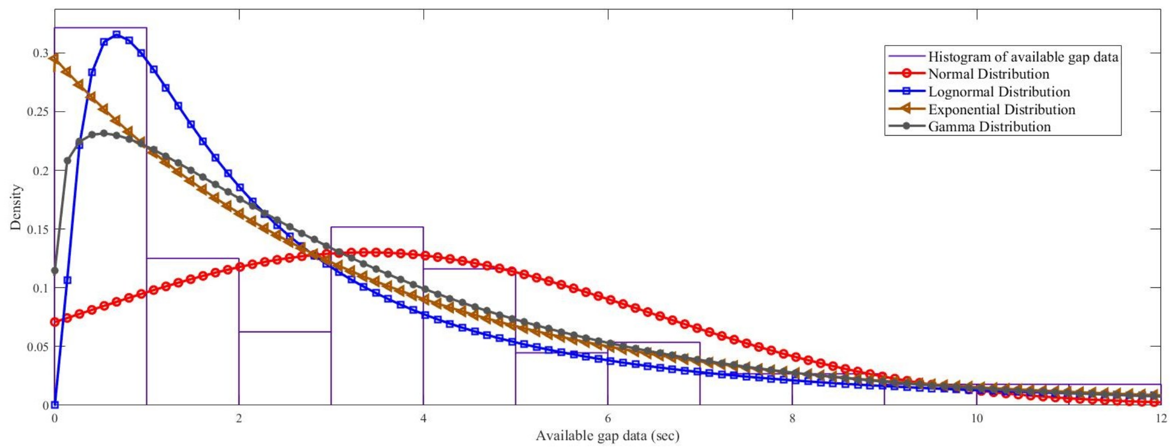
(b) WB for Intersection-1



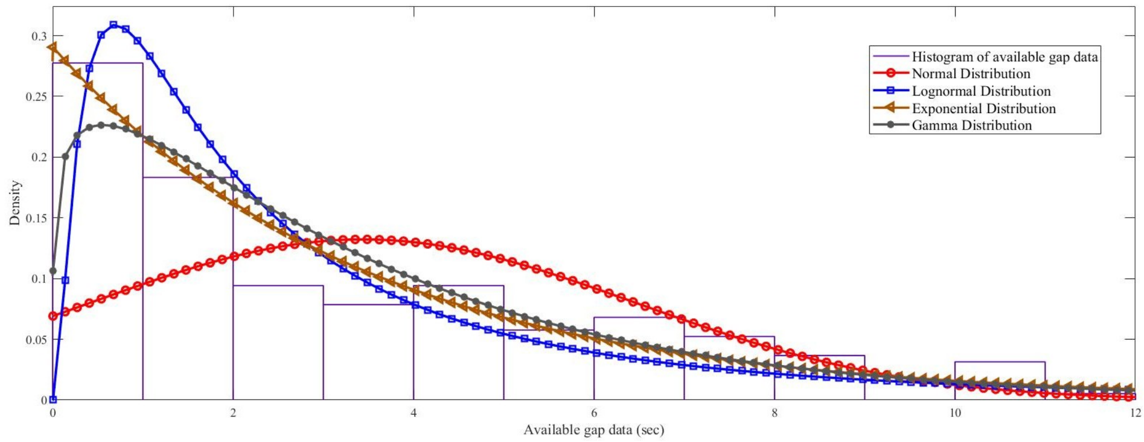
(c) EB for Intersection-2



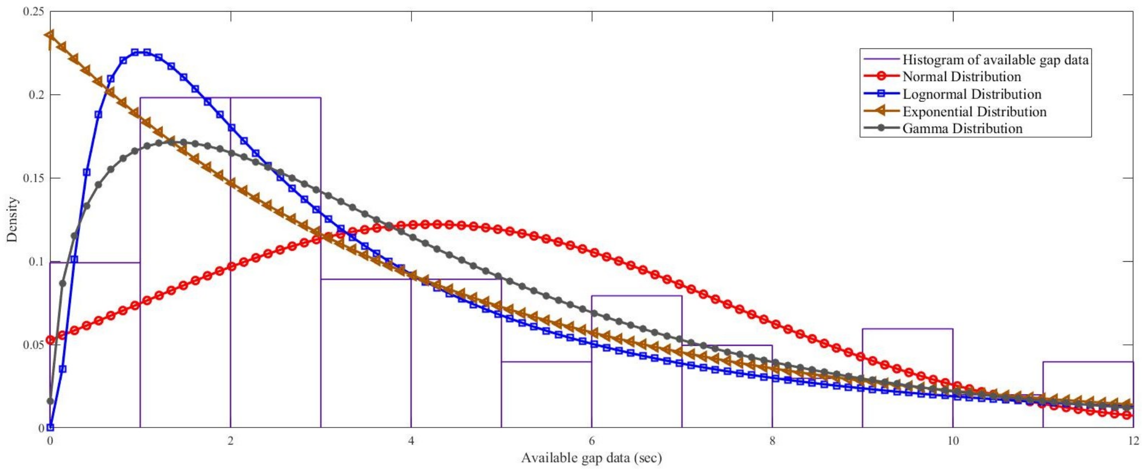
(d) WB for Intersection-2



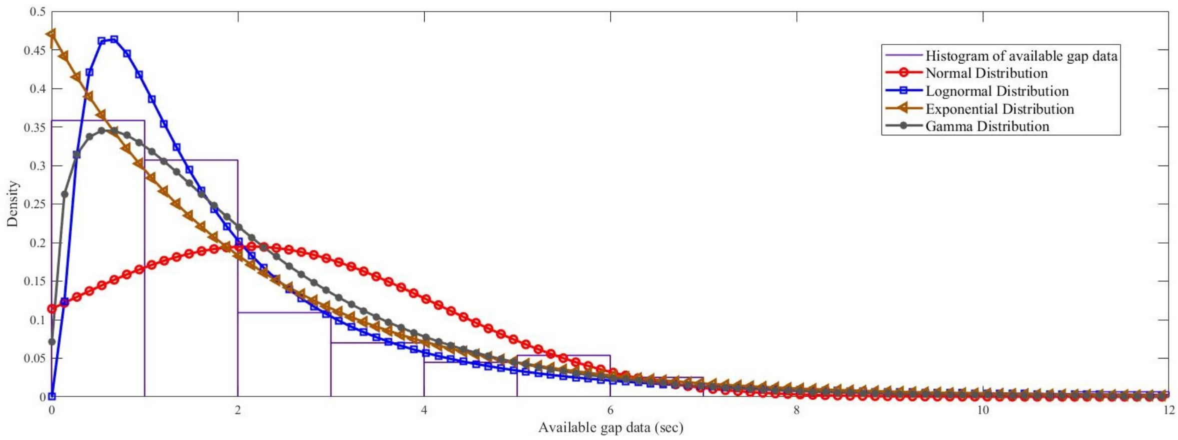
(e) EB for Intersection-3



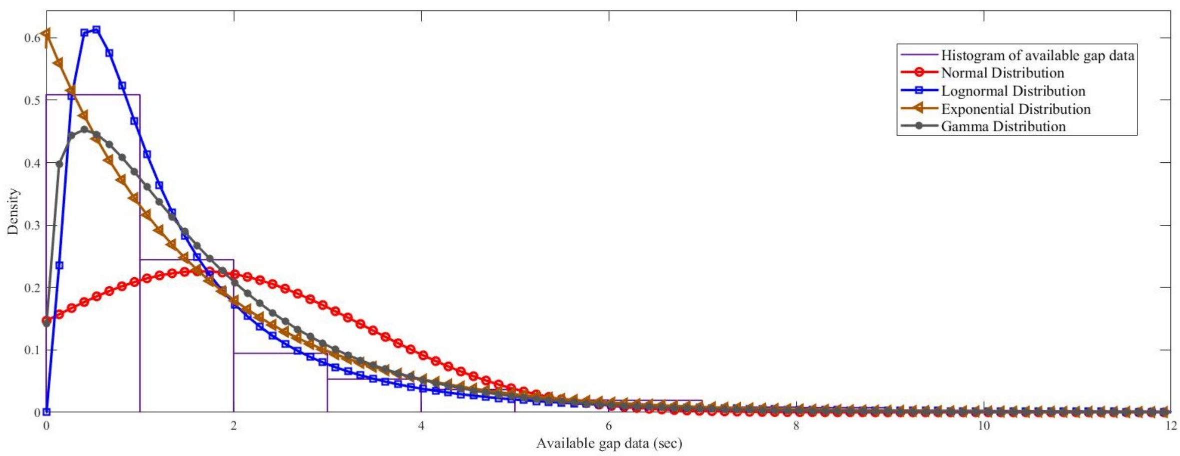
(f) WB for Intersection-3



(g) WB for Intersection-4

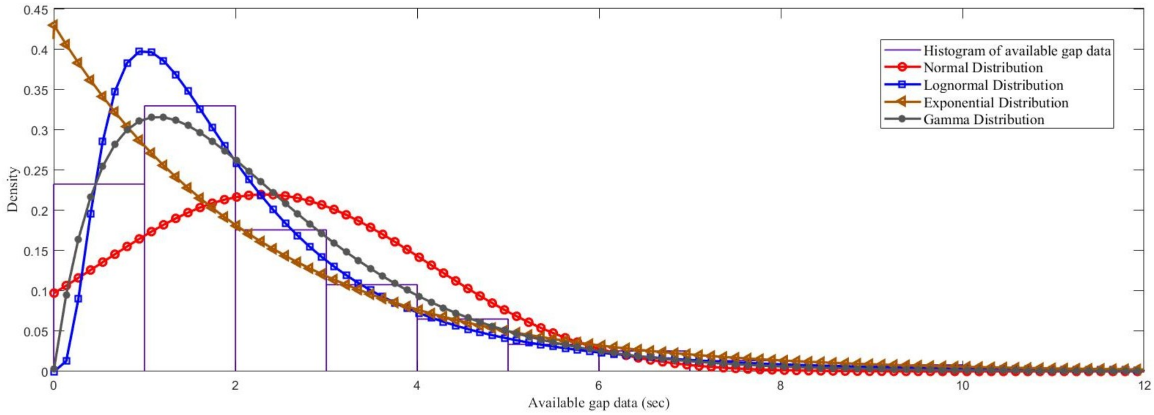


(h) WB for Intersection-5

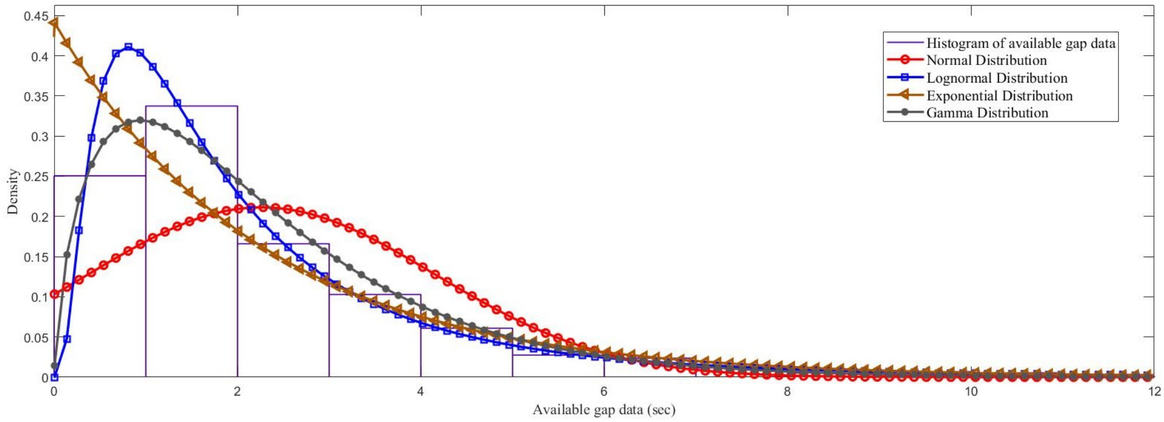


(i) WB for Intersection-6

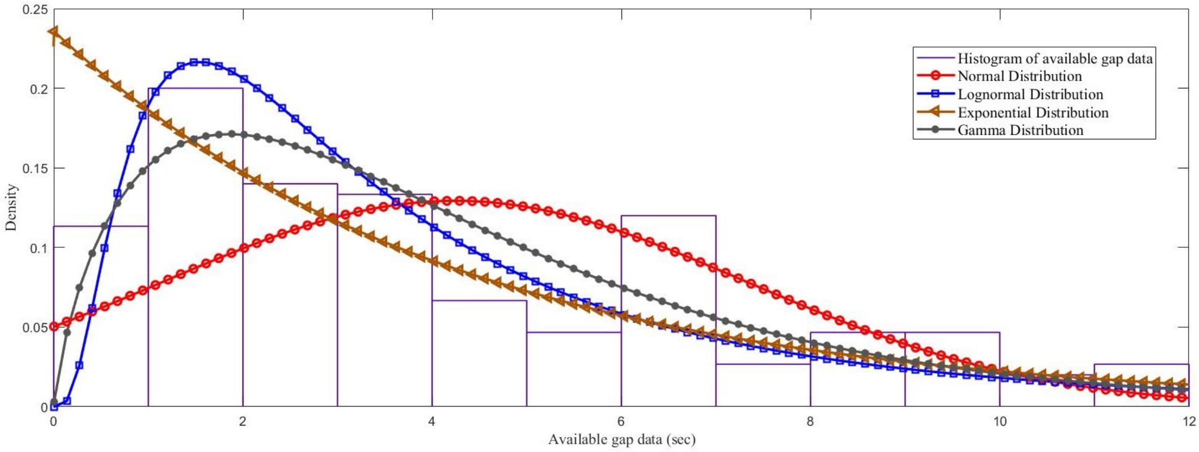
Figure A.3 Distributions fitted to available gap data for major road right turning vehicles



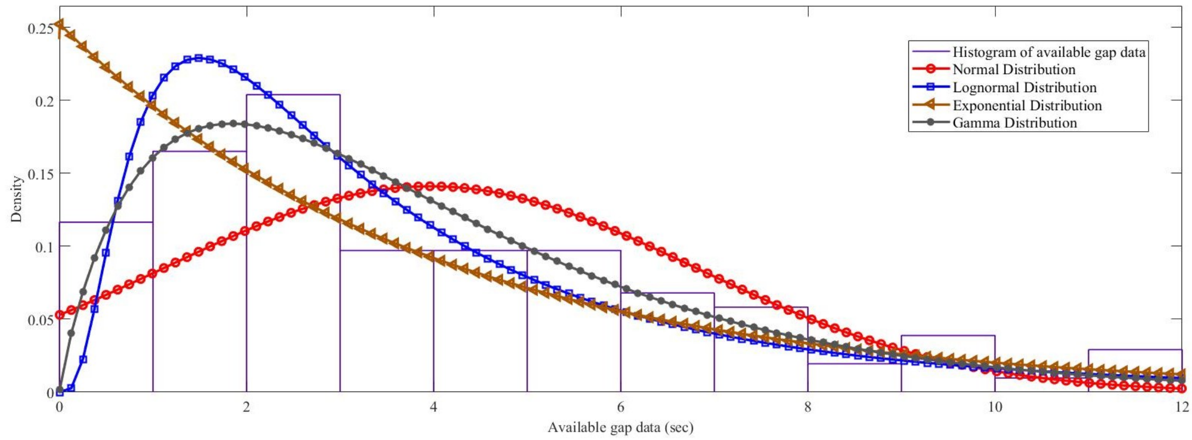
(a) NB for Intersection-1



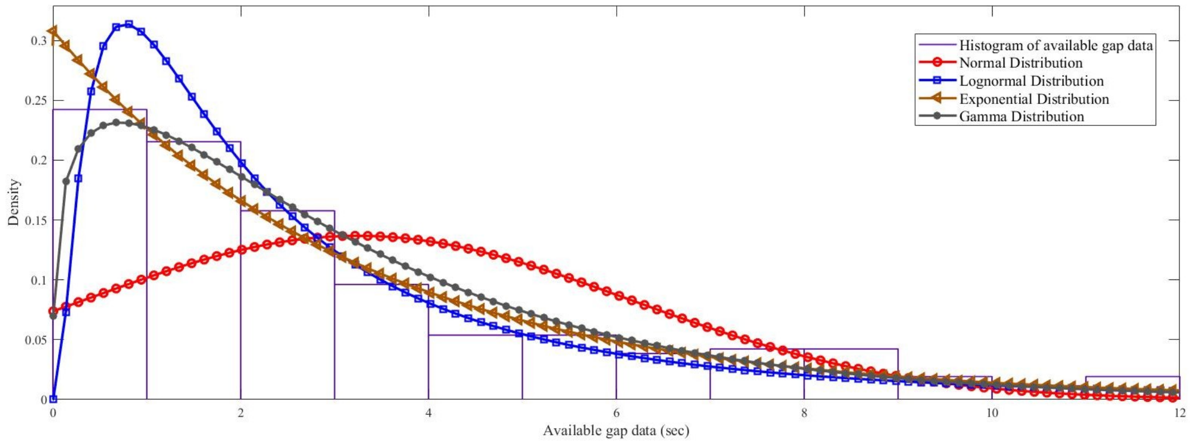
(b) SB for Intersection-1



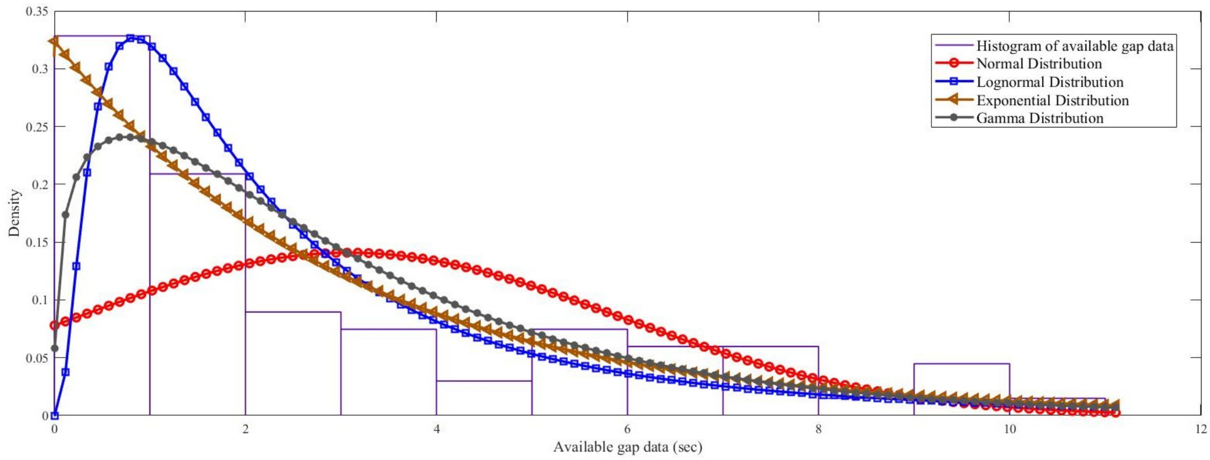
(c) NB for Intersection-2



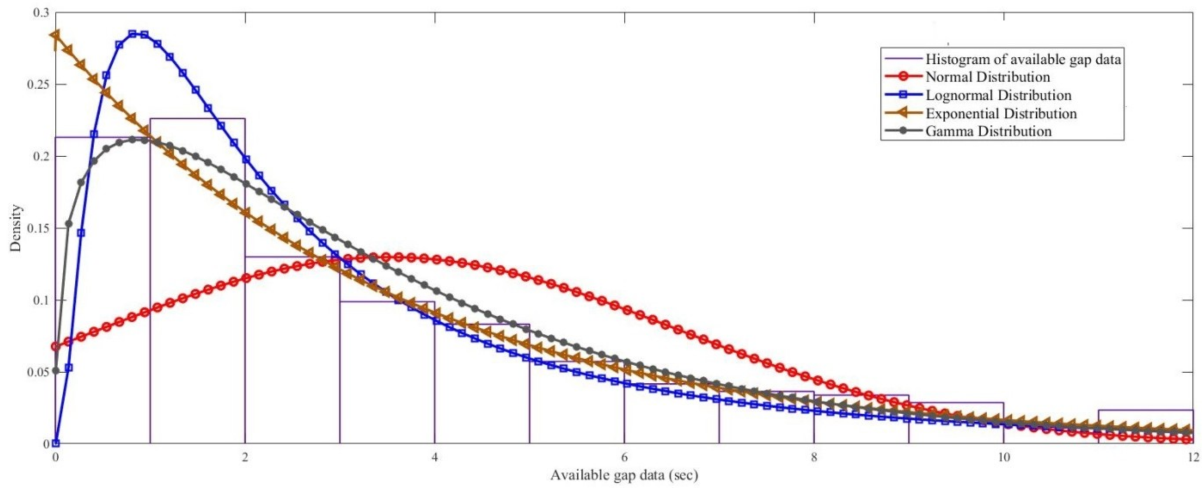
(d) SB for Intersection-2



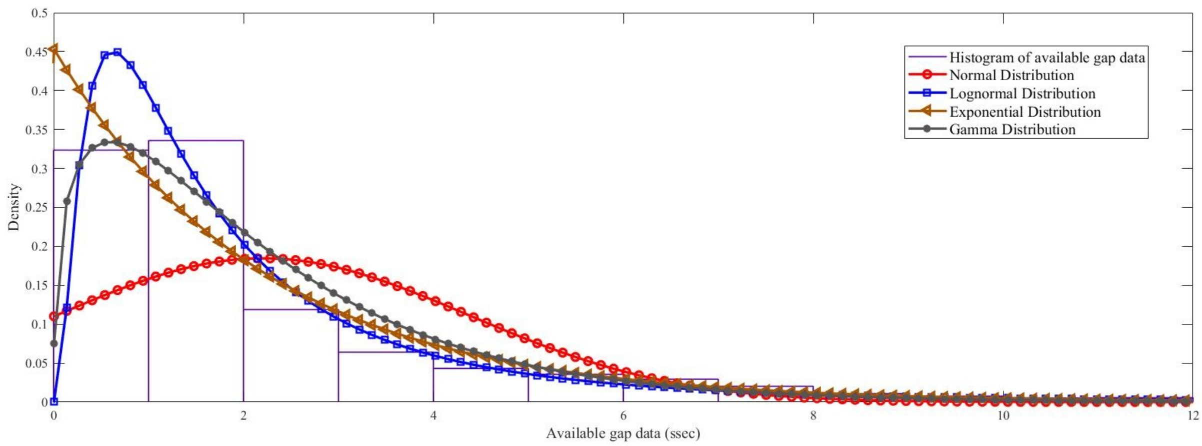
(e) NB for Intersection-3



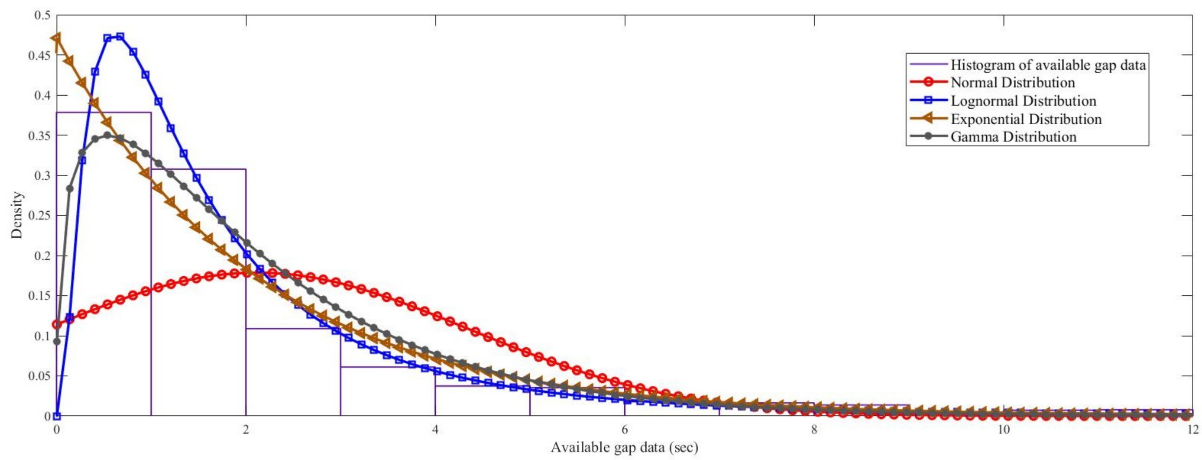
(f) SB for Intersection-3



(g) SB for Intersection-4



(h) SB for Intersection-5



(i) SB for Intersection-6

Figure A.4 Distributions fitted to available gap data for minor road right turning vehicles

APPENDIX-B

Table B.1 Sample sizes for different combinations of vehicle types

Major stream vehicles	Sample size					
	Int-1	Int-2	Int-3	Int-4	Int-5	Int-6
2w-2w	1391	127	315	48	1059	1954
2w-3w	492	50	23	-	199	720
2w-4w	248	23	87	50	574	523
2w-LCV	-	-	5	-	35	80
2w-BUS	-	-	-	-	14	86
2w-HV	-	-	-	-	17	14
3w-2w	604	196	17	-	203	672
3w-3w	309	7	-	-	47	291
3w-4w	80	9	10	-	121	228
3w-LCV	-	-	-	-	15	19
3w-Bus	-	-	-	-	-	43
4w-2w	292	25	97	72	588	600
4w-3w	125	-	4	6	143	209
4w-4w	74	6	26	191	415	191
4w-LCV	-	-	-	-	25	27
4w-HV	-	-	-	-	7	-
4w-BUS	-	-	-	-	-	20
4w-Bicycle	-	-	-	4	-	-
LCV-2w	-	-	-	-	58	57
LCV-3w	-	-	-	-	-	28
LCV-4w	-	-	-	9	18	8
LCV-BUS	-	8	-	-	-	-
LCV-HV	289	10	-	-	-	-
HV-3w	-	-	-	-	-	6
HV-4w	-	-	-	4	-	5
HV-BUS	7	68	-	-	-	-
BUS-2w	-	-	-	-	-	109
BUS-3w	-	-	-	-	-	8
Bus-4w	-	-	-	2	15	21
Bus- HV	280	6	-	-	-	-
Tractor-BUS	26	10	-	-	-	-

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