UNIVERSITY OF SOUTHAMPTON

FACULTY OF ENGINEERING, SCIENCE & MATHEMATICS

School of Civil Engineering and the Environment

SIMULATION STUDY OF DIFFERENT OPERATIONS OF MIXED TRAFFIC WITH MOTOR VEHICLES AND BICYCLES AT AT-GRADE INTERSECTIONS IN BEIJING, CHINA

by

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This thesis is dedicated to my family

UNIVERSITY OF SOUTHAMPTON

ABSTRACT

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SIMULATION STUDY OF DIFFERENT OPERATIONS OF MIXED TRAFFIC WITH MOTOR VEHICLES AND BICYCLES AT AT-GRADE INTERSECTIONS IN BEIJING, CHINA

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In Beijing, China, the bicycle remains one of the major transport modes. Because of the significant differences in characteristics between motor vehicles and bicycles, mixed traffic operations at at-grade intersections have caused major urban traffic management problems for a long time. The objective of this research has been to study the influence of bicycle flow on traffic within at-grade signalised intersections in Beijing, China. Microscopic simulation modelling has been used with local calibration/validation data to assess the potential of alternative approaches to a solution.

This thesis describes the research undertaken in the definition, behavioural modelling, coding, validation and application of a microscopic traffic simulation model, which was developed for the investigation of operations at at-grade signalised intersections with mixed traffic in China. The following scenarios have been investigated to find out how the different operations are likely to affect traffic operations: 1. Bicycles use the same signal phase as motor vehicles of the same direction; 2. Second stop line for bicycles; 3. Exclusive bicycle signal; 4. Building a separate lane for prohibiting of left-turn bicycles.

The results illustrated that temporal-segregation-related solutions can significantly reduce the conflicts between motor vehicles and bicycles, lessen the influences caused by bicycles, and enhance safety of both motor vehicles and bicycles. Each method has specific conditions for application. The results from the study may be used as guidelines for traffic operations at at-grade signalised intersections with mixed traffic in Beijing, China.

ACKNOWLEDEMENTS

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1. Introduction and Research Objectives

1.1 Background of the Study

Bicycle travel has played an historic role in transportation and remains an important mode in many countries, particularly in some developing countries. Traffic made up of both motorised and non-motorised vehicles is one of the major characteristics of urban traffic in Beijing, China (Liu et al., 1993A; Duan, 1993). Conflicts between motor vehicles and bicycles at intersections are a main cause of traffic congestion and accidents in China (Gao, 2003). Different measures have been proposed to alleviate the situation, but very little fundamental analysis has been undertaken to estimate the impact of such measures/operations for the management of mixed traffic. Inadequate understanding of traffic operations and poor management and operation of mixed traffic of at-grade intersections has led to serious accidents, capacity reduction, and increased environmental pollution.

In China, the bicycle is a very important mode of transport, and has taken up a substantial part of the urban traffic volume in the last few decades. Even now, the trend to use bicycles is increasing because of energy efficiency, cost effectiveness, health benefits and environmental advantages (Jin, 2004). Bicycle flows are most common and will continue to be one of the major modes of private transportation for urban transport in China, in the foreseeable future.

The interactions between motor vehicles and non-motorised vehicles at intersections have become one of the main causes of traffic congestion and accidents in China. However, less research has been carried out on bicycle traffic than on motorised traffic. Of the limited research which is discussed later, most concerns cyclist behaviour, such as perception-reaction, deceleration and gap acceptance. Very little research has been undertaken on the ways of dealing with bicycles and the impact on urban traffic.

1.2 Statement of the Problem

Few studies have been made to analyse and estimate the impact of different measures/operations on both motor vehicle and bicycle traffic. Three types of research approach are generally used in the field of traffic engineering. They are:

- Empirical approach
- Analytical approach

- Simulation approach

An empirical approach needs the collection and analysis of extensive field data (Hossain and McDonald, 1998A). As the more complicated characteristics of behaviour and traffic conditions, this type of investigation can be extremely time and labour consuming, and the result may be uncertain because of the limitation of data and the lack of alternative measures which can be valuated on the ground.

An analytical approach may be based on macroscopic theories, e.g. the fluid flow analogy, which helps to understand traffic behaviour as a whole. However, because of the need to make many assumptions, an analytical approach has very limited applications and cannot readilly be developed for mixed traffic operations (Hossain and McDonald, 1998A).

In general, "simulation is defined as dynamic representation of some part of the real world achieved by building a computer model and moving it through time (Drew, 1968)". The simulation approach may be either macroscopic or microscopic in nature. Macroscopic approaches deal with traffic in an aggregate, while microscopic simulation replicates the individual vehicle movements along the road system by processing every vehicle with its own characteristics. Simulation modelling is an increasingly important approach in research, demonstration, analysis, and consultancy of transport.

It is impossible to evaluate all potential solutions by implementing them in real systems. And neither the empirical nor analytical approaches may be suitable in many cases. However, microscopic simulation offers a much better way of considering the large variation in vehicle characteristics especially those found in the mixed traffic situation in developing countries. Therefore, a microscopic simulation approach was adopted in this research to investigate the mixed traffic operations in at-grade intersections in Beijing, China.

1.3 Objectives of the Study

The objective of this study has been to investigate the potential impact of different ways of managing mixed traffic, i.e. with motor vehicles and bicycles, within at-grade urban intersections in Beijing, China. To achieve the objective, the research has included definition of the problems, simulation model development and coding, model validation and application study to a range of possible traffic management measures. The following gives an outline description of each part of the research:

1. Identification of the problem: This includes specifying the objectives and approach to study the characteristics of bicycle traffic and the interactions with automobile traffic.

2. The development of the microscopic simulation model: the purpose of this stage is to develop a microscopic simulation model of mixed motor vehicles and bicycle traffic over an urban road network based on the understanding of driving and cycling behaviour and traffic characteristics.

3. Coding: translating the theoretical behaviour models into a working computer program.

- To provide a user-friendly editor to create the traffic network of sufficient size and details to present all link/junction types and control features;
- To provide a software environment for traffic modelling to represent the dynamic nature of traffic systems by simulating the movement of individual motor vehicles, bicycles and the interaction between motor vehicles and bicycles;
- To provide a detailed output of performance parameters for further evaluation.

4. Data collection and validation: To validate the microscopic simulation model on the basis of data collected in real traffic network.

5. *Simulation Application: This is the main part of the research in which the microscopic* simulation model is used to investigate the impact of the different operations for the management of mixed traffic in China.

6. Recommendation: The recommendation and further research were proposed for the design and evaluation of mixed traffic at at-grade signalised intersections in Beijing, based on the results from the simulation study.

1.4 Outline of the Thesis

The thesis is organized into seven chapters.

The first chapter specifies the objectives.

The second chapter describes background details that led to the current research. It was understood that bicycles play an important role and mixed traffic is one of the major characteristics of urban traffic in Beijing. Also, the conflicts between motor vehicles and bicycles at intersections are a major cause of accidents, traffic congestion and traffic capacity reduction. This chapter further explores some proposed measures to alleviate the conflict between bicycles and motor vehicles at intersections in an urban network. Microscopic traffic simulation models are basic tools in research, demonstration, analysis, and evaluation of transport, which save cost, labour and time. Chapter Two also reviews some existing microscopic simulation models and shows that no existing microscopic simulation models can adequately represent the real mixed traffic situation as it exists in Beijing, China.

Chapter Three describes the structures of the simulation model. The features of a typical simulation model are discussed generally. The choice of simulation language and the time management technique within the simulation model are explained in detail. At the end of the chapter, the organisation scheme for the simulation model developed here illustrated.

Chapter Four establishes behaviour models for the simulation system. Both motor vehicle and bicycle operators' behaviour are developed by combining the analysis results based on the data collected in the study and existing research results from the literature review.

The developed microscopic simulation model should be capable of representing mixed traffic with motor vehicles and bicycles under complex traffic scenarios for reasonably complicated traffic control scenarios. A verification and validation procedure was carried out using the data collected in a real road network in Beijing, China. The verification and validation processes are discussed and described in Chapter Five.

Chapter Six describes the study of the interactions of bicycle traffic and motor vehicles using the microscopic simulation model. Some scenarios were developed on the basis of real networks with different traffic management measures. After simulation, data was recorded, and the result compared and analyzed.

Chapter Seven describes the development of proposed guidelines for the design and evaluation of mixed traffic, and highlights conclusions, recommendations and further research. Additional information is provided in the Appendices. The literature referred to is listed in the References section.

2. Background

2.1 Introduction

Although there has been a significant development in traffic engineering related to traffic in developed countries, little progress has been made regarding traffic conditions in the developing world (Jing and Wang, 2004). Urban traffic authorities in developing countries are under growing pressure to decide on controversial issues such as whether or not to ban non-motorised vehicles, the form of mass-transit to implement, and the relevant weight of investment in building physical infrastructure or in the pursuit of efficient management of the existing system. There are few engineering tools and guidelines available for the traffic situation in this decision process.

Bicycle travel has played an historic role in transportation. In China, the bicycle is a very important traffic mode, and has taken up a large part of the urban traffic volume in the last few decades. The mix of bicycles and motor vehicles is one of the major characteristics of urban traffic in Beijing. Bicycling is the transportation mode of choice for up to 70 percent of the urban passenger trips in China (Liu et al., 1993A). From 1980 to 1992, the ownership of bicycles in Beijing doubled (Duan, 1993). Mixed traffic exists on about 70 percent of the roads and at more than 85 percent of intersections (Liu et al., 1995). According to the survey, the average volume of bicycle traffic at intersections within the second-beltway expressway is 16,000 bicycles per hour. The volume between the second- and third-beltway expressways is 12,000 bicycles per hour. Inside the first-beltway, more than 90 percent of intersections formed by primary and secondary streets have a flow of over 10,000 bicycles in the peak hour. Moreover, 14 percent of the intersections have a flow of more than 20,000 bicycles per hour (Li, 1993).

Even now, the trend to use bicycles is increasing because of the energy efficiency, cost effectiveness, health benefits and environmental advantages. According to the survey, the ownership of bicycles in Beijing in 2004 was 11,000,000 (Jin, 2004). Bicycle traffic is common and will continue to be one of the major modes of private transportation for urban transport in Beijing, China, in the coming decades.

Collisions between bicycles and motor vehicles have caused severe loss of life and property in many countries. Intersections are high-risk locations for bicycle-motor vehicle collisions because of the frequent conflicts between bicycle flows and motor vehicle flows. It was reported that bicycle–motor vehicle accidents killed more than 10% of all traffic accident fatalities in Delhi in 1993 (Fazio and Tiwari, 1995). Approximately 18% of all casualty accidents at intersections were bicycle-motor vehicle accidents in Tokyo. According to Traffic Safety Facts 2000, 32.6% of fatal accidents and 56.6% of injury bicycle-motor vehicle collisions occurred at intersections in the US (Wang and Nihan, 2004). In Beijing, with the growth in bicycle traffic, conflicts between bicycles and motor vehicles have also increased. Bicycle traffic contributes more than 50% of passenger transportation and more than 30 percent of traffic accident fatalities. Nearly 70 percent of the traffic accidents were related to bicycles (Liu et al., 1995). These conflicts bring many hazards to the cyclists' safety and decrease intersection capacity and the level of service for motor vehicle traffic. The interactions between motor vehicles and bicycles at intersections have become one of the main causes of traffic congestions and accidents in China. According to survey (Gao, 2003), delay at intersections contributes more than 80 percent of traffic delay. From the above it can be seen that the study of the cyclists' behaviour at intersections is very important.

However, less research has been carried out on bicycle traffic than on automobile traffic. Most of the current research concerns cyclist behaviour, such as perception-reaction, deceleration and gap acceptance, and very little has been undertaken into the ways of dealing with bicycle traffic and its impact on urban traffic. Taylor and Davis (1999) made a comprehensive review of published basic research in bicycle traffic science, and found "Recent emphasis on alternatives to automobile transportation has brought to light deficiencies in basic research performed in bicycle traffic science." They also suggested some further research priorities. The topics involved in the review include bicycles' characteristics, traffic flow, intersection control, capacity, networks, computer models, and geometric design.

Extensive literature reviews on behaviour of motor vehicle drivers, cyclists and microscopic simulation will be found in the following chapters.

2.2 Review

2.2.1 Requirements for cyclists

Safety is clearly an important consideration for cyclists and potential cyclists, but perceptions of safety will vary considerably by type of cyclist and journey purpose. Reports (CROW, 1993; Institution of Highways and Transportation, 1996) suggest that the following factors are all important for cyclists:

- Coherence — the infrastructure should be easy to use and links origins and destinations for cyclists;
- Directness — facilities offer the cyclist as direct a route as possible;
- Attractiveness — the facilities fit into their surroundings and make cycling an attractive option;
- Safety — the infrastructure is safe for cyclists and other road users; and
- Comfort — surfaces and facilities allow cyclists to travel quickly, smoothly, comfortably and well maintained.

2.2.2 Bicycle facilities

Segregated bicycle facilities may consist of separate roads, tracks, paths or lanes designated for use by cyclists and from which motorised traffic is generally excluded. There are various types of bicycle facility and different countries use differing, often legally defined, terms to distinguish them. In essence, segregated bicycle facilities fall into two categories: “Off-road” and “On-road”. “Off-road” facilities may exist on their own dedicated right-of-way or else run alongside an existing roadway. In the United States, off-road unsurfaced trails are commonly called "bike trails" or "mountain-bike trails", while surfaced trails that are separate from roadways and which meet more rigorous standards for width, grade and accessibility are commonly called "bike paths." In the United Kingdom and some other places, the terms "cycle path" or "cycle track" are sometimes used as a blanket term for any off-road facility. "On-road" facilities are typically termed "cycle lanes" and consist of portions of a roadway or the shoulder, which have been designated for use by cyclists.

Off-road facilities:

- Cycle way: Road (UK) or path (USA) dedicated to cyclists on separate right of way.
- Cycle track/Cycle path or Sidepath: Roadside converted-footway type structure alongside

(but not on) a carriageway (UK) or sidepath alongside (but not on) a roadway (USA). Bicycle path may be found in some countries. In the Netherlands bicycle paths are widespread. The current definition of a bicycle path in the HCM (TRB, 1994) is "a bikeway physically separated from motorized traffic by an open space or barrier, either within the highway right-of-way or within an independent right-of-way."

On-road facilities:

- Cycle lane (UK): is a traffic lane marked on an existing roadway or carriageway and generally restricted to cycle traffic.
- Designated bicycle lane (USA): is a portion of a roadway or shoulder which is separated from traffic lanes by the use of a solid white stripe on the pavement and has been designated for preferential use by cyclists (FHWA, 1988).
- Paved shoulder (USA): is part of the cross section of the street but not part of the traveled way for motor vehicles. Bicycles are separated from motor vehicles by the right edge line. Paved shoulders are often constructed on new roadway facilities when allowed by right-of-way requirements. Bicycles generally use paved shoulder as one-way facilities in the same direction as motor vehicle traffic, much like bicycle lanes (Allen et al., 1998).

Bicycle roads in China:

There are two major types of bicycle road in Chinese cities. One is the separated bicycle road, in which the motor vehicle traffic and bicycle traffic are divided by an open space, barrier or raised median (Wei et al., 2003B). The other is the mixed bicycle road, in which bicycles share the roadway with motor vehicle traffic without any physical or nonphysical separations (Wang and Wei, 1993). The separated bicycle road can be further classified into three types:

- Exclusive bike paths: mostly positioned on one or both sides of motor vehicle lanes with designed outer separations.
- Independent bike lanes: positioned on one or both sides of motor vehicle lanes with barrier separations.
- Nonindependent bike lanes: positioned on one or both sides of motor vehicle lanes without physical separation but separated by pavement markings.

Bicycle signals:

In recent years, a number of new traffic control devices have been proposed for use on roadways with bicycle facilities. These devices are often proposed in order to remedy an operational problem caused by a specific facility, or to address the real or perceived needs of cyclists. Bicycle signals are one of these traffic control devices.

Bicycle signals resemble standard traffic signal heads, with the distinguishing feature being that the bicycle signal displays the outline of a bicycle (see Figure 2.1) instead of the standard ball or arrow indication.

In Europe, there are a number of intersections that contain arms with separated parallel bicycle pathways or mandatory bike lanes. Because the presence of parallel paths complicate traffic flow patterns at intersections, bicycle signals were developed to regulate bicycle movements at these intersections. Bicycle signals are commonly used around the world in such places as the Netherlands, England, Germany, and China. In the Netherlands, separate bicycle signals are commonly used at arterial intersections that have bike lanes and high volumes of bicyclists and motor vehicle traffic. The bicyclist signals are vertical red, amber, and green bicycle symbols mounted on a pole (Zegeer et al., 1994). A few cities in the United States, such as Davis, California, have installed bicycle traffic signals in locations with similar circumstances (City of Davis Public Works Department, 1996).

Figure 2.1 Bicycle signal

Signals for cyclists are particularly appreciated when cyclists have to cross busy roads and junctions. However, if traffic is light, the cyclist can be asked to wait for a long period when no traffic is coming. At such locations it might be sufficient, at least outside the peak, just to have a “flashing amber” cycle signal, which is used in Holland (Institute for Transport Studies, 2000B).

2.2.3 Some solutions

Interactive conflicts and disturbances are main factors in traffic congestion, accidents and capacity reduction on urban networks, especially at intersections. One of the ways to alleviate the problem is to segregate motor vehicles and bicycles and minimize the interaction between them. Generally, the segregation of mixed traffic at intersections into motor vehicles and non-motorized vehicles may be by temporal and/or spatial means. "Real segregation of modes on intersections can be realised by the use of flyovers or underpasses, and by segregation in time, i.e. by the use of traffic lights" (Godefrooij, 1997). Spatial segregation is usually related to grade-separation by using interchanges over roadways. "Temporal segregation is the means via signal control and other traffic engineering control facilities, and usually costs less without large scale of construction and expansion of land use" (Wei et al., 2003B). Since spatial segregation always means construction with high costs and large land occupation, temporal separation becomes a better option in managing mixed traffic of at-grade intersections in urban network in developing cities. Wei et al. (2003B) suggested temporal-segregation-related solutions to mixed traffic problems. All the solutions have some merits and drawbacks.

Advantages of segregation at the intersections:

- Segregation gives cyclists their own space at the intersection.
- One of the main points of segregation is to avoid encounters with motorists.
- Segregation can improve the safety.
- Segregation sometimes enables priority to be given to cyclists by the adjustment of traffic lights (Godefrooij, 1997).

Drawbacks of segregation at the intersections:

- Segregation sometimes makes the situation more complex and need better transportation management.
- Segregation sometimes forces cyclists to split a left turn manoeuvre into two separated crossing manoeuvres, which lengthens the cyclists riding distance, and might cause some delay as well.

Some proposed and commonly used measures are introduced below:

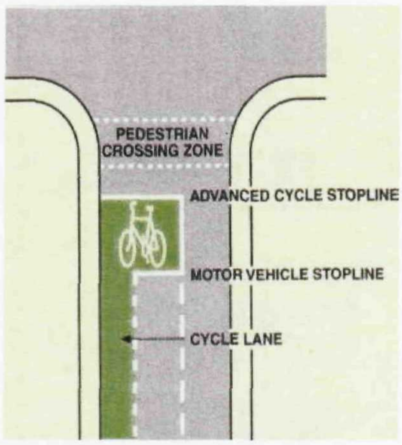
2.2.3.1 Advanced stop line

An Advanced Stop Line (ASL) is a facility that allows cyclists to position themselves at the front of the queuing traffic at a signalized junction stop line. A bicycle approach lane is normally, but not always, provided to allow bicycles to reach the reservoir when motor vehicles are queuing. The bicycle approach lane can be located on the nearside or in the centre.

ASLs were first installed in UK in Oxford in 1986 (Wheeler, 1995), and have increased with many installed in several cities. Since the first ASL was introduced, a further three cities, namely Newark, Bristol and York, have had them installed as part of the Department of Transport's (DOT) investigation into the performance of this facility (DFT, 2005). Although not forming part of the DOT trials, similar ASLs have also been provided in Cambridge and Newcastle. In addition to these sites Avon County Council have developed a simplified form of ASL, which has been installed extensively throughout Bristol. Many other authorities have also tried this layout, examples being Manchester and Cambridge.

ASLs have also been used as traffic control devices in other countries, such as the US, Germany and Netherlands (CROW, 1993; Clarke and Tracy, 1994; Zegeer et al., 1994; Moeur, 1999).

The general layout of an advanced stop line is shown in Figure 2.2 (e. g. driving on the left in UK/Irl and driving on the right in USA etc). Advance stop lines benefit from the existence of a bike lane or other similar facility to permit bicyclists to pass other queued traffic on the intersection approach arm. The provision of advanced stop lines creates a reservoir area in which bicycles can wait in front of other traffic. In the UK, the layout of an ASL has also developed and been simplified, from the use of two sets of traffic signals, to just a standard set, with the minimum use of roadside signs. All these layouts consist of a 5 m deep reservoir and a mandatory cycle lane. The reasoning is that this puts the cyclists clearly into the view of HGV (heavy goods vehicles) drivers, who have a blind spot up to 4 m directly in front of the cab. 1.5 m should generally be regarded as the minimum width for the cycle lane, though in Oxford the lane was only 1.2 m wide (DFT, 2005).



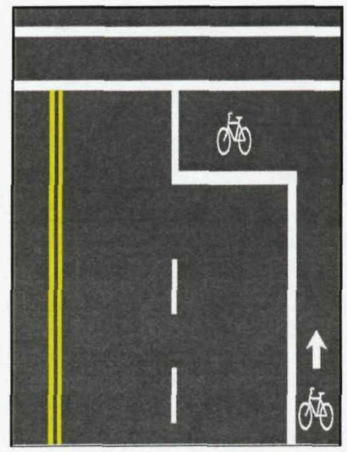


Figure 2.2 Advanced stop line (left figure is for driving on the left)

The purpose of ASL is to improve conditions and safety for bicycles and motor vehicles at intersections. The stated advantages of the advanced stop line are:

- Increasing the visibility of cyclists;
- Helping cyclists make safer turns and crossings;
- Encouraging cyclists to make more predictable approaches to and through an intersection;
- Providing space at the front of an intersection to help cyclists avoid breathing motor vehicle fumes (Wilkinson et al., 1994).

Advanced stop lines also have some disadvantages:

- Drivers of motor vehicles generally operate in a manner consistent with where they expect things to be at intersections. This usually works to enhance smooth and consistent traffic flow. The advance stop line works on the presumption that vehicles will obey the markings and other traffic control devices, and stop prior to crossing the advanced stop line, hence entering the area for bicycle queuing. However, this conflicts with ingrained driver behavior. When the stop line is moved a significant distance back from the "typical" place, such as in this case, then drivers may not react appropriately, unless enhanced signing and pavement markings are used. Advocates of advance stop lines are optimistic in their belief that driver behavior can be modified through the use of signs and markings. Even under these circumstances, though, it was reported that in countries where advance stop lines have been installed that violation rates by drivers entering the area of the advance stop line are high (Camden Cycling Campaign, 1999).

- Advance stop lines will also only function in the manner intended if right turns on red are prohibited at these locations (for driving on the right). However some locations, such as in the United States or China, permit right turns on red unless specifically prohibited, this can create a conflict between driver behavior and the signs and markings at these locations. This may also significantly affect the traffic flow and capacity of the intersection.

Therefore, a full analysis of all junctions where ASLs are proposed should be carried out before a final decision is made. ASLs will not necessarily be of benefit to cyclists at all signal-controlled junctions, and a careful study of the signal timings and flows is necessary.

2.2.3.2 Special waiting area for left-turn bicycles

The principle is almost the same as ASLs. A special waiting area for vehicles driving on the right can be set before the motor vehicle stop line. Left-turn bicycles can stay in this waiting area during the red period. When the lights turn to green, left-turn bicycles can pass the intersection before the motor vehicles. A special waiting area for vehicle driving on the right is shown in Figure 2.3.

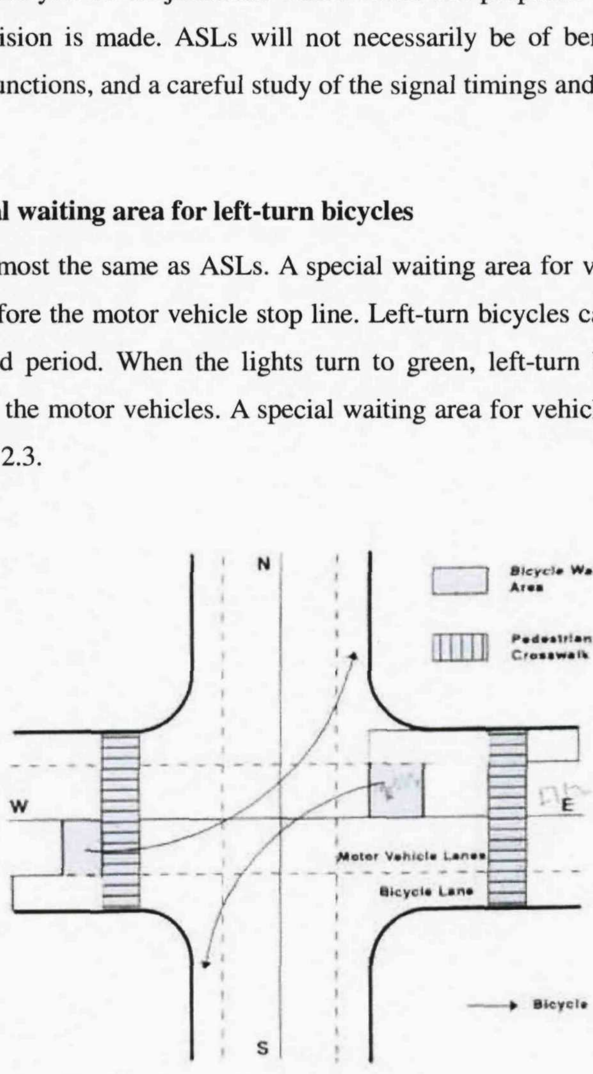


Figure 2.3 Special waiting area for left-turn bicycles

The treatment has some advantages:

- Increases cyclist visibility, allowing them to move to the front of the line where they are in full view of motorists on all sides of the intersection. Allows cyclist to move to the correct position for turning movements during the red signal phase.

- Does not significantly delay motorists since cyclists are usually able to accelerate quickly through intersections. "A study done by Chinese experts indicates that a bicycle starts to move more than 1 sec faster than a motor vehicle" (Wang and Wei, 1993).
- Reduces conflicts between turning bicycles and vehicles by clearly separating motor vehicles and bicycles temporally.

The measure also has some disadvantages:

- Effectiveness may be decreased because some unfamiliar motorists may encroach into the bicycle waiting area.
- Allowing motorists to turn right on red may be hazardous to cyclists (left-turning vehicles in Britain and other countries where motorists drive on the left)
- If the signal turns green as a cyclist is approaching the intersection, they may not have enough time to position themselves properly to effectively and safely use the waiting area.

2.2.3.3 The "Model Bicycle Intersection"

The "model bicycle intersection" was developed and being successfully used in Australia (AUSTROADS, 1993). To simplify the design of bicycle lanes at intersections, six intersection elements have been identified after considering the different phases that cyclists pass through as they deal with road intersections. In simple terms, as cyclists are approaching an intersection, they will ride along midblock in a bicycle lane and then undertake a manoeuvre to place them in the right position for the approach to the intersection. They will ride along the approach to the intersection, and if required to stop, they will store at the intersection until they are able to proceed. The cyclist will then proceed through the intersection, departing the intersection before ending up in the next midblock bicycle lane. From this understanding, the "model bicycle intersection" and its six elements were created.

These six elements and the various positions that cyclists may need to place themselves given their need to travel through the intersection (VICROADS, 2001) are illustrated in Figure 2.4.

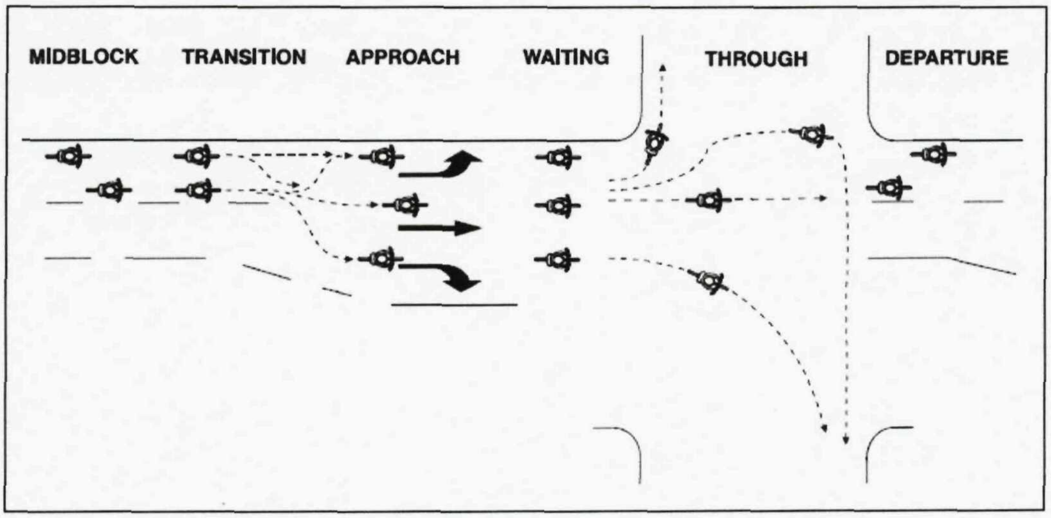


Figure 2.4 The "model bicycle intersection" and its six elements

The six elements of the "model bicycle intersection" are:

1. Midblock bicycle position on the road;
2. Transition from the midblock to the approach position;
3. Approach position at the intersection;
4. Storage of bicycles at the intersection when stopped;
5. Through movement to travel across the intersection;
6. Departure from the intersection.

The practical application of the "model bicycle intersection" is to provide a simplified approach to designing bicycle facilities at intersections. It allows a complex problem to be considered as six smaller and simpler problems.

2.2.3.4 Bicycle banned area

Bicycles maybe banned from an area located at the centre of the intersection. All bicycles are required to travel outside the banned area while motor vehicles maintain their original routes in using the banned area. For safety, a bicycle protection pathway may be created at the periphery of the banned area. The general layout of bicycle banned area is shown in Figure 2.5.

A similar solution, called a "hook turn" is also found in Australia. The "bicycle hook turn" in Australia (Australian National Road Transport Commission, 1999) is shown in Figure 2.6.

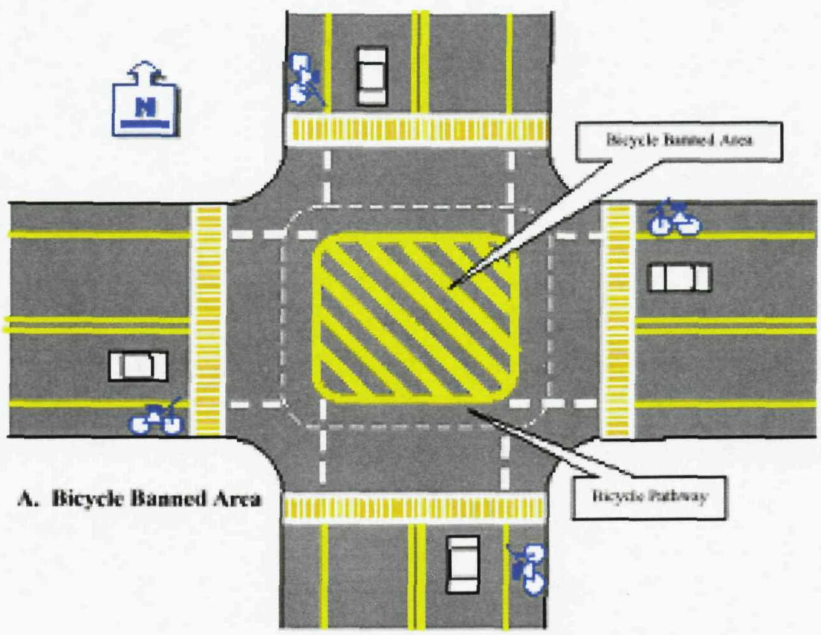


Figure 2.5 Bicycle banned area

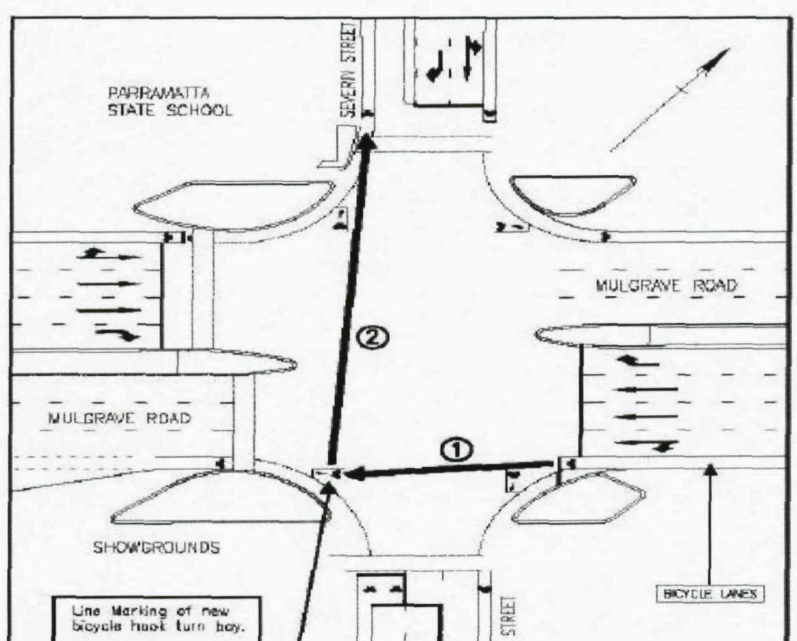


Figure 2.6 Bicycle hook turn in Australia

As shown on the figure, a cyclist would:

- Move forward through the intersection from the bicycle lane on a "green" light.
- Stop in the box at the opposite corner – turn right (in the direction of the marked arrow).
- When light turns "green", move forward through the intersection into the bicycle lane ahead.

Such measures can limit the left-turn bicycles, and reduce the conflicts between motor vehicles and left-turn bicycles accordingly. Sun and Yang (2004) compared the capacity of a typical intersection under two different operations. The first operation was with bicycles using the same signal phase with vehicles of same direction, and the second operation was about adopting a second stop line for bicycles. The results indicated that bicycle capacity at this intersection was 36% higher by using a second stop line for bicycles than by bicycles using the same signal phase with vehicles of same direction.

However, this method requires larger space. If there is no enough storage space before the second stop line, some bicycles may stop outside the storage space and block the movement of motor vehicles, thus causing unpredictable delay and congestion. "Bicycle banned area is applicable to the wider intersections where large bicycles and obvious non-motorized disturbances exist" (Wei et al., 2003A). It was suggested that the method of using a second stop line for bicycles is more suitable for large intersections (Wang and Wei, 1993).

This method also requests a higher requirement from traffic management facilities. Some left-turning cyclists may not obey the rules and move with left-turning vehicles. How to guide cyclists to following the rules would be a problem. It was also mentioned in Wang and Wei (1993) research that the method of left-turn prohibition has been tried in Beijing but proved to be unpopular with cyclists. This is consistent with findings in this simulation study.

2.2.3.5 Left-turn prohibitions

In addition to the bicycle-banned area that can limit the left-turn bicycles, there is another measure to avoid the conflicts between through motor vehicles and left-turn bicycles. That is left-turn prohibitions. A new route can be built near the intersection. Through the route, left-turn bicycles change their route to a right-turn path.

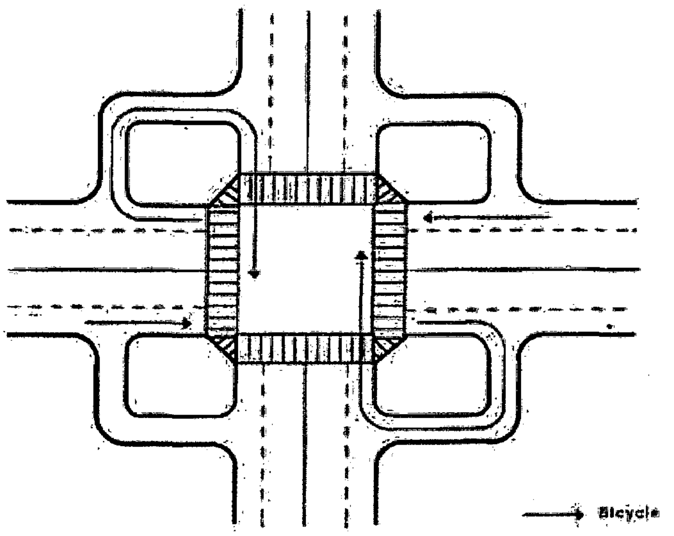


Figure 2.7 Left-turn Bicycle Prohibition

The method can avoid the conflicts between through motor vehicles and left- turn bicycles and this can be helpful to the improvement of safety.

However, it is clear that this method increases the travel distance of left-turn cyclists, and consequently increases the average journey time. Also, the location of the intersection and the layout around the intersection need to be taken into account. The cyclists may not accept this approach if the separate lane is too long or too far from the intersection.

2.2.3.6 Changing route/Divert bicycle traffic flow

This solution is to change the path of cyclists before large intersections with high motor vehicle flows. In big intersections, motor vehicle flow is always heavy. Therefore, bicycles turning with motor vehicles often cause severe conflicts. One potential solution is to divert the bicycle flow outside the intersection.

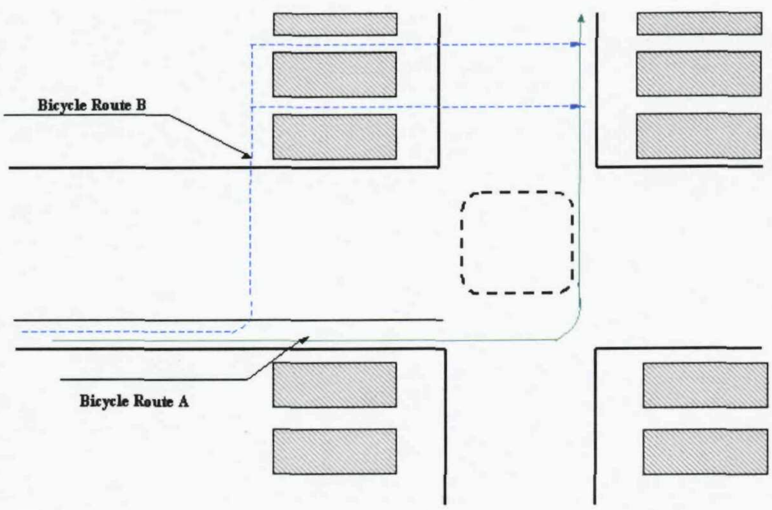


Figure 2.8 Change Bicycle Route

2.2.3.7 Cycle network

The main principle for a sustainable safe infrastructure is that every road is appointed a specific function and is designed such that the road or street in question meets the specific functional requirements as optimally as possible. The goal of the creation of a network of bicycle routes is to concentrate the cyclists on these routes, where the conditions of safety and comfort are satisfactory. The cycle network routes are designed to accommodate commuter and recreational cyclists by creating linkages to major sites related to employment, recreation, culture and tourism, retail, and institutional uses, as well as to provide connections to existing and proposed bicycle routes.

The concept of a "cycle network" has been adopted by several countries. The "cycle network" can encourage automobile trip reduction by providing planned access to an alternative means of transportation.

A bicycle route network for London is planned. The London Cycle Network (LCN+) will provide 900km of safer, faster bicycle routes through the Capital (London Assembly Transport Committee, 2005). The National Cycle Network is a comprehensive network of safe and attractive routes to bicycle, running throughout the UK. 10,000 miles have recently been completed, one third of which is on traffic-free paths, the rest follows quiet lanes or traffic-calmed roads (Sustrans, 2002).

The City of Philadelphia in the United States is developing a city-wide network of "bike friendly" streets to serve cyclists. The goals of the Network Plan are to increase the percentage of trips made by bicycle to 5% of all trips, and to simultaneously reduce by 10% the number of cyclists killed or injured in traffic accidents.

Switzerland opened a nation-wide network of cycle routes in 1998 covering the whole country, and the links between cycles and public transport are very well developed (SWOV, 2001).

In 1996-1997 some 100 kilometers of on-road cycle routes were built in Paris. These routes are mostly taken from road space, not pedestrian space. Cycle use there was 1% in June 1996, by September 1997 it had risen to 5% (SWOV, 2001).

Munich has doubled cycle use from 6% in 1976 to 15% in 1992 by building up a good network of cycle routes (SWOV, 2001).

According to the Dutch guide, the cycle network must be arranged according to a three-tier hierarchy: access, distributor and through roads (CROW, 1993).

It is possible to apply this strategy to Chinese cities, for the following reasons:

- It is preferable that cyclists take minor roads in big Chinese cities. In the downtown area of Chinese cities, it is usually very difficult to travel through the intersections by bicycle, because the red light period will usually consume a lot of time. So, cyclists always wish to avoid these major roads with busy intersections, preferring minor roads, in which they come across fewer red lights and crossings. This condition can already be regarded as a prototype or the internal dynamic of the "cycle network".
- There are large numbers of minor roads and alleys, which are very suitable for bicycle traffic, but too narrow to accommodate automobile traffic in large Chinese cities, which, though has not form a traffic network yet, can be taken as a potential "cycle network".

In order to develop a good bicycle travel environment, the following improvements were supposed:

- The road should be made wider so as to make more streets accessible to cyclists and the surface of roads needs to be improved to make it better to ride bicycles on.

- Because of the complexity of the road configuration, some direction signs will be needed to show directions to avoid confusion.
- Road planning should be improved, with the whole traffic network better organized to make a complete traffic system, which can access varied destinations of the city.

Some researchers (Guan et al., 2001) undertook a hypothetical study on "cycle network" in Beijing, China. They proposed a scenario of bicycle network using existing alleys in east HuangChengGen district of Beijing, and estimated the motor vehicle delay and capacity of bicycle traffic with and without the bicycle network. The theoretical analysis results showed that the vehicle delay reduced and the bicycle capacity increased 3.7% with the bicycle network.

2.2.3.8 Signal phase control

Letting the left-turn bicycles share the same stage with left-turn motor vehicles is one of the common used methods in segregating motor vehicles and bicycles.

The treatment has some advantages:

- The conflicts between left-turn and through motor vehicles and bicycles can be avoided and the capacity and safety can be improved accordingly. Jing and Wang (2004) collected data at a typical intersection in China, and compared the capacity before and after the implementation of bicycles using the same signal phase with vehicles of same direction. The research results showed that bicycle capacity increased around 24%.
- The movements of bicycles are not disturbed by traffic from other directions.
- The bicycles can cross the intersection continuously without stopping inside the intersection.

The disadvantages to this measure are:

- During the left-turn green phase, some through bicycles wait before the stop line, which blocks left-turning bicycles. Therefore, some left-turn bicycles cannot cross the intersection during the green period and have to wait for the next cycle. The through bicycles were blocked by the left-turn bicycles in the same way during the through green phase.

- Because the left-turn bicycles cross the intersection with the left-turn motor vehicles synchronously, there are some potential disturbances caused as bicycles intend to compete with motor vehicles by occupying the space within the intersection. In order to avoid such potential disturbances, a certain size is required.

The channelization of a bicycle lane before the stop line to divide the left-turn and through bicycle flows can lessen the influence between left-turn and through bicycles, and improve the bicycle capacity of an intersection accordingly.

2.2.3.9 Bicycle green phase

A bicycle green phase is a time period giving bicycles their own time to cross an intersection in the signal cycle time. A special bicycle phase allows bicyclists in the bike lane to proceed straight before motor vehicles are allowed to proceed. The purpose is to separate motor vehicles and bicycles by signal timing. "It only applies to intersections with small non-motorized volumes" (Wei et al., 2003A). Such a phase is used widely in Europe and Asia, and is advocated in the Dutch design manual (Zegeer et al., 1994; CROW, 1993).

At such an "all green" phase, bicycles from all directions would proceed at the same time, without the danger of conflicting with motor vehicles. It has the particular benefit of helping cyclists turn left (turn right in UK).

To minimize delay to traffic it may be possible to integrate cyclists' movements with pedestrians. This could increase cycle time to motor vehicles and hence the overall capacity of an intersection. Delay to motor vehicles could be minimized with cyclists sharing an all red pedestrian phase, and crossing an intersection at the same time as pedestrians, in the same direction and with much care.

Researchers in Davis, in the USA researched the effects of integration a bicycle phase into an existing large urban intersection signal cycle and to quantify and compare the effects in a benefit-cost setting. The research results showed that the estimated saving due to crash reductions were the dominant result of the bicycle phase installation, overshadowing the construction costs and increased vehicle delay. However, the CO analysis indicated that with the bicycle phase in place the CO emissions increased due to increased vehicle delay and stops (Korve and Niemeier, 2002).

Wei et al. (2003B) conducted a research on bicycle signal phase. Before-and-after studies showed that since the exclusive bicycle signal phases were assigned, the abeyance percentage of the control regulations by bicycles increased up to 97%, where low volumes of bicycle traffic existed.

All the solutions above try to reduce the chances of traffic conflicts between motor vehicles and bicycles, and get the highest capacity for both motor vehicles and bicycles.

2.3 Simulation approach

Most of the traffic characteristics and parameters are dependent on traffic conditions and environments. An empirical approach would require a huge database, and in mixed traffic situations more variations in traffic conditions are expected, data requirement could be even higher. The empirical and analytical approach may not be suitable for all conditions. The simulation approach is not dependent on any such site and can be adapted to changing situations more easily. A simulation approach is useful to study traffic conditions which do not exist in reality, and provides a powerful tool for a wide range of experimentation.

2.3.1 Two simulation approach

Traffic simulations can be classified into two branches: one is macroscopic simulation and the other is microscopic simulation.

Macroscopic approach: This approach describes the traffic in terms of aggregate parameters such as traffic volumes, speed and density. Vehicles are considered collectively rather than individually.

Microscopic approach: In this approach, each vehicle is considered individually and reacts to the stimuli given by the traffic environments.

Microscopic simulation is more useful in the analysis of traffic capacity and understanding of drivers' behaviour and offers a much better way of considering the high variation of vehicle characteristics especially in mixed traffic situations in developing countries. It can also be used effectively to consider Intelligent Transport Systems (ITS). However, in the long run, microscopic simulation offers better possibilities than macroscopic simulation to model various dynamic phenomena. Microscopic simulation models are becoming an increasingly important tool for traffic control research and development. "They permit the traffic engineer to have a "bird's eye" view of the traffic and an instant feel for current problems and possible solutions" (Institute for Transport Studies, 2000A). Controversial or new techniques can be tried and tested without any disruption to traffic in a real network. The APAS (1995) has identified four main uses:

- Simulating networks including interactions between vehicles and new responsive control and information systems.
- Short term forecasting.
- Enhancing assignment models.
- Providing inputs to car driving simulators.

2.3.2 Some existing microscopic simulation models for mixed traffic flow

There are many microscopic simulation models available. However, most existing models focus on motor vehicles, with very few considering mixed traffic including motor vehicle and bicycle. The need for a simulation model considering mixed traffic has been recognized by some researchers.

In its review of micro-simulation models, the SMARTEST project team identified only two of the thirty micro-simulation models it reviewed as claiming to be able to represent bicycles (Algers et al., 1997). These two packages are FLEXSYT II and HUTSIM. Even though a vehicle type such as a 'bicycle' may be represented in these European simulation models; however, bicyclist behavior is not modeled based on any empirical studies and is very simplistic in nature.

HUTSIM (Kosonen, 1996) is a simulation program for adaptive traffic signal control developed by the Helsinki University of Technology. In this model, bicycles are treated as "fast-moving pedestrians," which may be appropriate in Finland where bicyclists and pedestrians share the same facilities and they interfere with the motor-vehicular traffic only at crosswalks. However, this is not generally the case elsewhere.

FLEXSYT II developed by the Dutch Ministry of Traffic (Algers et al., 1997). FLEXSYT II is an event-based, microscopic simulation tool for traffic management studies. In the simulation eight vehicle types can be present. These types are person cars, small trucks, large trucks, busses, trams, bicycles, pedestrians and carpool vehicles.

Besides these two commercial simulation products, there are some research products considering mixed traffic flow. Such as MORTAB (model for depicting road traffic behaviour) (Ramanayya, 1988), TRASMIC (traffic simulation of mixed condition) (Sutomo, 1992), MIXSIM (Hoque, 1994) and MIXNETSIM (Mixed Traffic Network Simulation Model for Developing Cities) (Hossain, 1996). These first three simulation models are lane- or strip-based. In these models, non-motorised vehicles are treated as slow-moving vehicles, without considering their own characteristics. Interactions between motor vehicles and non-motorized vehicles are not described in detail.

MORTAB is a microscopic simulation model developed by Ramanayya (1988). It can replicate a stream of mixed traffic in a lane with a certain width in order to obtain speed-flow-density relationships. This model is capable of presenting eight different vehicle types including bicycles in Indian traffic conditions. It also considers overtaking behaviour within the same lane for narrower vehicles. The model was validated by comparing the output with measured speed distributions at particular points. However, bicycles are still treated as slow-moving vehicles, and the model only considers behaviour in a lane. It cannot deal with more complicated road conditions such as intersections.

Sutomo (1992) developed TRASMIC model for microscopic simulation of mixed traffic at signalised intersections. In this model, the road width was divided into smaller strips (1m in this model) and a lane analogy was applied to strips. However, it is difficult to find a strip width which evenly fits road width, vehicle width and side clearances, because a vehicle with side clearances was required to be fitted within a whole number of strips in this model.

MIXSIM is a microscopic simulation model developed by Hoque (1994). MISIM is used for studying isolated signalised intersections with mixed traffic conditions. In this model, the strip-based approach similar to TRASMIC model was adopted to replicate the mixed traffic flow. However, the MIXSIM model considered narrower strip widths (half a meter) to reduce the accompanying error. MIXSIM was calibrated on data from Bangladesh.

MIXNETSIM is a microscopic simulation model developed by Hossain (1996). MIXNETSIM is applied to study the effect of non-motorised vehicles on the performance of a corridor of developing cities. The model was calibrated and validated using data collected in Dhaka, Bangladesh. In this model, a co-ordinate based technique was adopted to model mixed traffic vehicle movement. With co-ordinate based referencing all the vehicles in a single road link can be represented in a single list. Also, each vehicle can be tracked using the co-ordinates of one of the corners of the vehicle. Generally, it will be much slower. For example, to search a car follower will take long time in X, Y coordinator system, but would be very simple in a link-based system. The calculation and simulation would be much more difficult for complete interaction. Besides, there would also be a problem in determining the conflicted vehicles or bicycles from the surrounding vehicles or bicycles within the intersections.

BICSIM (Faghri and Egyháziová, 1999) was developed in 1999 at the University of Delaware. BICSIM "is the first in the world to allow the inclusion of cyclists in the analysis of urban transportation by means of microscopic simulation." "The problem in creating such a program is identifying and understanding the interactions between bicycles and motor vehicles." However, the model is too simplistic. The bicycle-following logic is simply an adaptation of car-following logic. In BICSIM, the bicycle-following representation reported is based on the total safety distance model, originally developed for motor traffic. It is based on the hypothesis that the distance headway a cyclist maintains with a leading bicycle is the distance that may be traveled during a following bicyclist's reaction time and the difference between the braking distance of the lead and the following bicycles. This approach assumes that interaction between bicycles occur as the distance headway between bicycles reduces to less than or equal to this minimum distance headway estimated. The characteristics of group waiting before the stop line and group riding behaviour have not been taken into consideration either. The model also ignores the different behaviour according to different characteristics of cyclists, such as age and gender, and gap/lag acceptance of bicycles. Besides, all the behaviour models in BICSIM are developed and validated based on the data collected in Delaware in the United States, which may not reflect the mixed traffic situation in Beijing reasonably.

The characteristics of mixed traffic in developed countries are different to that in Beijing. In developed countries, the percentage of bicycles in the traffic flow is very low compared with that in Beijing. Besides, bicycles move like motor vehicles and often share the lane with the motor vehicles in developed countries and there are fewer conflicts between motor vehicles and bicycles. In Beijing, bicycle traffic contributes more than 50 percent of passenger transportation. Unlike motor vehicles, which move or wait in a linear queue, bicycles often appear with a "group" characteristic in Beijing. The microscopic simulation models, which developed on the basis of developed countries' traffic situations, cannot represent such real characteristics of mixed traffic.

Some researchers in China have recognized the importance of building microscopic simulation models that can reflect the real mixed traffic characteristics in Beijing, China.

Yang et al. (2004) introduced a preliminary study on mixed traffic microscopic simulation in China. In this simulation system, the behaviour model was too simple because of inadequate understanding of characteristics of both motor vehicles and bicycles. The model can only be applied in un-signalized junctions. And the model was neither calibrated nor validated.

Ning et al. (2005) developed a new mixed traffic microscopic simulation model based on the characteristics in China. The system prototype was based on the cellular automata model. This simulation model proposed to divide the lane into cells, and each vehicles and bicycles cover several cells. There are some improvements in the network editor and display. However, there was no adequate underlying research on traffic behaviour, such as car-following and lane-changing model, to support this simulation model.

There are a lot of efforts in developing microscopic simulation models for mixed traffic in China. However, it generally led only to largely simplified models because of a lack of adequate understanding of the characteristics of mixed traffic, which is the basis of the microscopic traffic simulation model developed here.

Since the existing microscopic simulation models could not replicate the real traffic behaviour under mixed traffic conditions in Beijing, a new microscopic simulation model that can represent the mixed traffic characteristics is required.

2.4 Summary

Mixed traffic is one of the major characteristics of urban traffic in Beijing. The conflicts between motor vehicles and bicycles at intersections are the main causes of traffic congestion and accidents. Some measures were proposed to try to alleviate the situation, while few studies were made to analyse and estimate the effect of different measures/operations on both motor vehicle and bicycle traffic. The inadequate understanding of the traffic operations and improper managing and operating mixed traffic of at-grade intersections has led to some serious problems, such as accidents, capacity reduction and environmental pollution. Therefore, the objective of this work is to study the effect of bicycle flow to the traffic within at-grade intersections in an urban network in Beijing, China, under different operations using a new simulation model specifically focused on the introduction of appropriate cyclist behaviour.

3. Model structure

A microscopic simulation model needs to realistically describe the road network, the traffic conditions and road user characteristics and represent the interactions among them dynamically. In this chapter, the simulation language, the scanning technique, and the structure of the simulation are discussed.

3.1 Introduction

Simulation is a process based on building a computer model that suitably represents a real system, which enables the extraction of valid inferences on the behaviour of the system modelled, from the outcomes of the computer experiments conducted on its model. In recent years, simulation has become one of the most used and powerful tools for systems analysis and design, by its proven ability to find solutions to build new systems, or assess the impact of proposed changes on an already existing system. A simulation study usually has the objective of helping to get a better understanding on how a system behaves, evaluating the impact of changes in the system, or in values of the parameters governing the system, or of decisions on the policies controlling the system. In the context, a general microscopic traffic simulation model should always meet the following requirements:

- Ability to simulate the appropriate geographic scope or study area for the analysis, including an isolated intersection, single roadway, or network.
- Capability to model various facility types, such as freeways, ramps, urban arterials, bicycle lanes, etc.
- Ability to represent various travel modes, such as car, bus, bicycle, and pedestrian traffic.
- Ability to accurately model traffic behaviour, such as car following, lane changing, etc.
- Ability to model various traffic management strategies, such as signal control, priority control, incident management, etc.
- Ability to produce and output performance measures, such as volumes, travel time, speed, etc.
- Capability to provide a software platform interfacing with the user in a friendly and efficient way.

The simulation model developed in this research considered all these requirements and had great universality and transferability.

3.2 Simulation language

Microscopic models simulate the movement of individual vehicles based on car-following and lane-changing theories. Typically, vehicles enter a transportation network using a statistical distribution of arrivals (a stochastic process) and are tracked through the network over small time intervals (e.g., 1 second or a fraction of a second). Typically, upon entry, each vehicle is assigned a destination, a vehicle type, and a driver type. Computer time and storage requirements for microscopic models are large, usually limiting the network size and the number of simulation runs that can be completed.

Java, C, C++, Visual Basic, and .Net are the most popular programming languages at present. Java, Visual Basic and .Net are essentially compiled at run-time by the system's virtual machine, therefore, the speed of these three programming languages is somewhat slower than C/C++. In the case of traffic simulation, due to large scale of the calculations involved, efficiency and the speed are very important.

C and C++ are core languages, which can compile alone and therefore are faster than Java, Visual Basic and .Net programming languages. C++ can be said to be a superset of the C programming language, and C++ supports object-oriented programming. Object-oriented techniques are the best way to develop large, complex software applications and systems, especially for a traffic simulation system, in which the road, vehicles and control facilities can be treated as objects. C++ is a practical well-established object-oriented programming language. In the world of industrial software, C++ is viewed as a solid, mature, mainstream tool. It has widespread industry support.

Visual C++ is the most popular IDE (Integrated Development Environment) and compiler for C++ on Microsoft Windows operating system. Reviewing the facilities of Visual C++, it was found to be suitable for the development of the simulation model. In recent years, Visual C++ has been successfully adopted to develop simulation models by a number of researchers. On the basis of the above, the Visual C++ programming language was chosen for the simulation model.

3.3 Scanning technique

In a real traffic system, the traffic entities undertake certain functions simultaneously. However, in a simulation model, all functions must be updated following certain sequences. Therefore, a suitable scanning technique is required. There are two commonly used scanning techniques in simulation modelling. One is event scanning, the other is time scanning.

Event scanning updates the time status of the system according to the occurrence of specific events or activities. In this method, the scanning time interval is variable.

Time scanning technique divides the duration of the simulation process into a number of successive time intervals. The system updates the status according to the time intervals which are fixed.

Traffic simulation system is a continuous dynamic system, and time scanning is more suitable than event scanning in reflecting the characteristic of continuity of traffic flow. Besides, it is easier to develop a computer program using a fixed time interval. Therefore the time scanning technique was chosen for the development of the simulation model described in this thesis.

The choice of interval length is important. Generally, the smaller the scanning time interval, the more accurate the simulation results. However, if the intervals are too short, the increased calculation time will be dramatically. On the other hand, the precision of the results will be reduced as the scanning interval increases. Therefore, it is important to find a scanning time interval, which produces simulation results with a satisfactory level of accuracy without unnecessarily increasing the computing time.

From sensitivity tests, Miyahara (1994) research concluded that time increments shorter than 0.5 seconds do not make a significant difference in overall vehicle movement. In Gipps' study, it was found that the car-following model behaves well even if recalculation interval is the same as a driver's reaction time that is normally 0.5 to 1.5 seconds (Gipps, 1981). In order to observe the effects of different scanning time intervals, Wu (1994) carried out three repeated simulations running with three different scanning time intervals, 0.5, 0.25 and 0.125 seconds, based on the same geometric layout. The results showed that the differences in the average vehicle delay between these three scanning time intervals were negligible. Therefore, a recalculation interval of 0.5 seconds was thought to be suitable in the simulation model and has been adopted in this research.

3.4 Structure of the simulation model

In this microscopic simulation model, each link, vehicle, bicycle, signal, detector and so on was modelled as object. Motor vehicles and bicycles can enter the network from the origin section and leave through the destination section. In order to implement the functions, some associated components must be formulated. For convenience and better understanding of model development all the related components can be grouped into three major processors, each performing specific and independent functions. The following processors go to make up the simulation model:

- Network Edit Processor: Consists of four categories: geometric data, signal and detector data, traffic data and simulation control data. The "Network Edit Processor" can be used to develop the road network, O-D matrix, signals, detectors, and so on, whose locations and properties can be set inside.
- Simulation Processor: The geometric of background of road network developed in "Network Edit Processor" can be presented in "Simulator". Load the data from "Network Edit Processor", and build up the running conditions of the simulation model. The movement of vehicles along the network can be modelled during the simulation process.
- Output Processor: The simulation results recorded during the simulation process are dealt with and output via this processor.

The structure of the simulation model is shown in Figure 3.1.

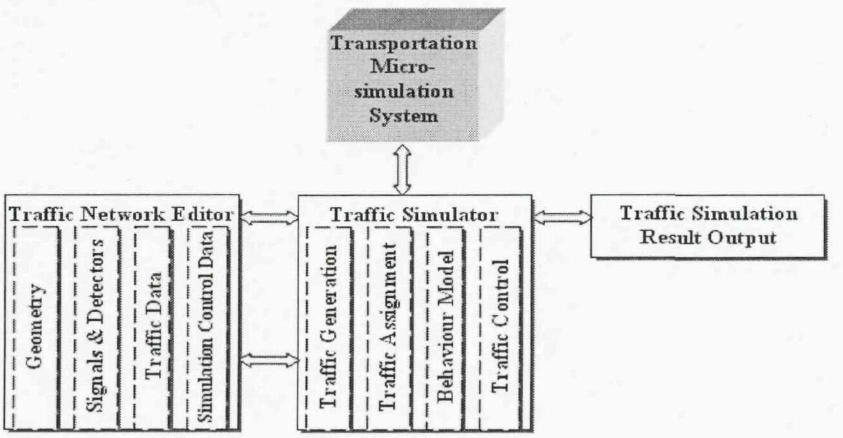


Figure 3.1 Model structure

The network editor in the simulation model can support a wide range of road types and geometry, such as urban road, freeway, bicycle lanes, signalized and unsignalized intersections, roundabouts and road networks. However, pavement hasn't been considered in this model. A number of vehicle types including cars, trucks, buses, and bicycles are provided in this model. The model can simulate a range of traffic management features including fixed-signal traffic control, actuated signal control, priority control, ramp control and traffic data detection. Bus priority strategy is also potentially feasible. Data can be recorded for any time period and interval within that time period. Data can be recorded at any location in the network, at an intersection, and along any path. It can also be reported for individual or aggregated vehicles. The decision on which data to be recorded and how the data is reported, is based on the needs of the research.

The simulation model developed in this research can simulate many common, geometric and operational conditions throughout the transportation system. Some functions and applications may not be fully displayed in this research due to the limitation of simulation study purpose. In practice, the simulation model has been used in some practical applications. "Poole project" is an example of the simulation model being used in a small urban road network. Poole is a city in the south of England. Located in Poole, the study area was a small urban road network including signalized intersections, unsignalized intersections and roundabouts. The simulation model was used in this project to observe the effects of different traffic management on the study area. Verification and validation process were conducted using field data. "J13 project" is another example of the model being used in motorway simulation. The J13 junction is a roundabout on M4, which is a motorway in UK linking London with Wales. Based on data collected in the field, a simulation study was conducted by using the simulation model developed in this research. These examples showed that the simulation model can be used in different road type and different places after sufficient calibration and validation.

3.5 Traffic simulation system's flowchart

The general logic of the simulation process is illustrated in Figure 3.2. Time-interval scanning approach was employed to update the state of system. Before initialization of the timer, the geometric data, traffic control data and traffic data built up in Network Edit Processor are imported, and the running conditions are set out. At each time interval, the simulation cycle scans the events scheduling list. During this process, the states of vehicles and traffic controls are updated.

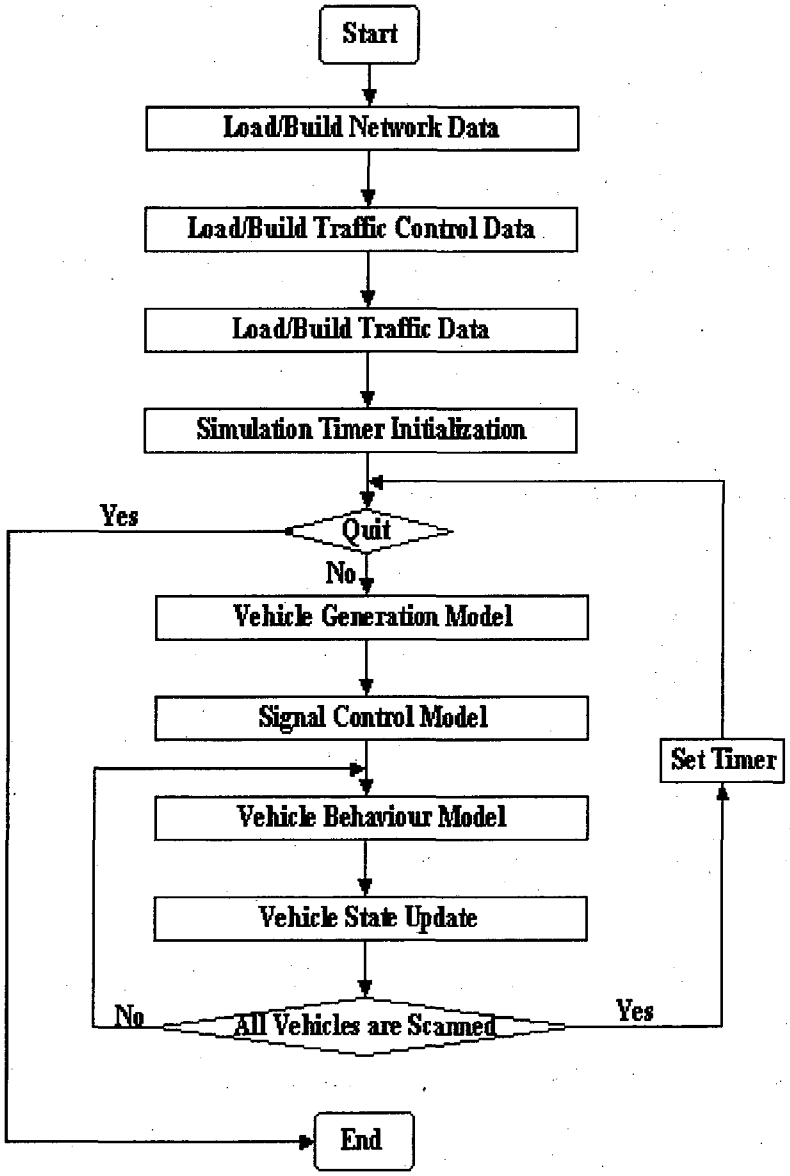


Figure 3.2 Traffic simulation system flowchart

Step 1: Creating and editing traffic networks based on background images; or importing an existing network, which is already developed. The traffic network data includes the geometric data of all the traffic facilities, such as lane, road, signal, detector and so on, and the connections among them.

Step 2: Setting up / importing the traffic control data. The traffic control data includes signal control type (fixed or vehicle actuated), the cycle of signals, green time, priority rules, etc.

Step 3: Loading/building traffic data, including traffic flow, desired speed, vehicle type, turning proportions, etc.

Step 4: Initializing the simulation timer. If the counter value is equal to the "system simulation time", then quit; otherwise, the counter value increases by an interval and the process goes to step 5.

Step 5: New vehicles enter the simulation system using a statistical distribution of arrivals, and each vehicle is assigned unique characteristics, such as a destination, a vehicle type, and a driver type.

Step 6: Updating the states of signals and other controls.

Step 7: Gaining the new position of each vehicle after a series decision process, such as car-following model, lane-changing model, and gap acceptance model.

Step 8: Updating the states of all vehicles and goes to step 4.

4. Behavioural model development

In order to produce a reliable, realistic and easy-to-use simulation model, the following functions, such as vehicle characteristics, driver behaviour, interactions between bicycles and motor vehicles, signal controls, detection techniques, and geometric layout are essential. The main emphasis of this research is based on mixed traffic. Therefore, both motor vehicle and bicycle behaviour models are considered.

4.1 Motor vehicle models

4.1.1 Static characteristics

Vehicle size is an important factor that will affect both the road design and the behavioural abilities, such as acceleration, deceleration and so on (Drew, 1968). Motor vehicle type is the essential attribute of vehicles in microscopic simulation.

Motor vehicles are usually classified into the following four categories.
Light vehicle (LV): 3 to 4 wheeled vehicles;
Medium commercial vehicle (MV): vehicles with 2 axles but more than 4 wheels;
Heavy commercial vehicle (HV): vehicles with more than 2 axles;
Buses and coaches;

This classification standard is also applicable for China. In China, the motor vehicles are also classified into four categories: light vehicle, Light commercial vehicle, heavy commercial vehicle and buses and coaches (Lu et al., 2006).

Due to sharp restrictions on motorcycle use in Beijing, levels of motorcycle use in Beijing are relatively low. The Beijing government released a policy to prohibit new motorcycles to apply for a license from 1985. According to a survey in Beijing in 1987, 75000 motorcycles were owned. More than 88% were distributed in the suburban area and were prohibited from entering the area within the third ring road and the city zone (Liang and Wu, 1991). Beijing's regulation forbad motorcycles with license numbers stating with "Jing B", to enter the area within the forth ring road from July 2001(Beijing Traffic Management Bureau, 2001). Thus, as motorcycle flow is very low in Beijing urban area compared with motor vehicles and bicycles, motorcycles have not been taken into consideration in this model. Buses and coaches were not considered for reasons of simplicity. Therefore, motor vehicles were classified into three categories for use in this study, LV, MV and HV.

4.1.2 Generation model

4.1.2.1 Initial headway models

In microscopic simulation models, each vehicle will have assigned an arrival time at the entry to the system. Vehicle arrival times can be calculated according to the initial headway. "Headway is the time interval between the arrivals of the corresponding point in a pair of moving vehicles, such as from front bumper to front bumper (Mei and Bullen, 1993)." There are many existing headway models, as discussed below:

4.1.2.1.1 Some existing headway models

1. Negative Exponential Distribution

The negative exponential distribution is the interarrival time distribution of the Poisson process. In traffic flow theory, the negative exponential distribution has been used since Adams (Adams, 1936; Luttinen, 1996).

$$f(t)=\begin{cases}\lambda\ell^{-\lambda t} & (t\geq 0)\\ 0 & (t<0)\end{cases} \tag{4.1}$$

where,

λ is the mean arrival rate of vehicles (inverse of mean headways), unit is veh/sec;

It assumes random arrivals, and is not suitable for heavy traffic flows or cyclic arrival patterns such as may be found downstream of a signal controlled junction.

2. Shifted Negative Exponential Distribution

Adding a phase is a possible way to avoid the extremely short headways, which is a major problem of the negative exponential distribution. Shifted negative exponential distribution was derived from the negative exponential distribution.

The density function is:

$$f(t)=\begin{cases}\lambda\ell^{-\lambda(t-\tau)} & (t\geq\tau)\\ 0 & (t<\tau)\end{cases}$$

(4.2)

where,

λ is the mean arrival rate of vehicles (inverse of mean headways), unit is veh/sec;

τ is the minimum headway;

3. Lognormal Distribution

If ln(t) follows the normal distribution $N(\mu,\sigma^2)$, then t is said to have a lognormal distribution.

$$f(t)=\frac{1}{t\sigma(2\pi)^{0.5}}e^{\left[-\frac{(\ln(t)-\mu)^2}{2\sigma^2}\right]}(t>0)$$

(4.3)

where,

μ is the mean of the normally distributed ln(t);

σ^2 is the variance of the normally distributed ln(t);

4. Mixed Distribution

Some researchers believe that vehicle headways could be best described in terms of more than one distribution to represent the range of flow conditions in a traffic stream. The mixed distributions typically have two distinct headway distributions: one for free-moving and the other for following vehicles. The simplest format of mixed distribution is

$$f(t)=r\cdot f_1(t)+(1-r)f_2(t)$$

(4.4)

where,

r is the proportion of the distribution;

$f_1(t)$ and $f_2(t)$ are the density functions of the two different distributions;

Schuhl first proposed this distribution considering two sub-distributions which are both exponentially distributed (Schuhl, 1955; Hoque, 1994). Buckley (1962) proposed a mixed model derived from a combination of a negative exponential distribution and a normal distribution. The DDNED (Double Displaced Negative Exponential Distribution) model was proposed by Griffiths (1991), which consists of two shifted negative exponential distributions. As compared with the negative exponential and the shifted exponential distributions, the mixed distribution has the advantage of being able to produce a larger coefficient of variation. It also has separate models for free and follower headways.

4.1.2.1.2 Proposed initial headway model

Some of the distribution models are usually cumbersome for simulation purposes because of the complexity of the form and require many parameters to describe. The shifted exponential distribution has been widely used in simulation studies (Luttinen, 1996). In the shifted negative exponential distribution, the parameter τ prevents the occurrence of extremely short headways, which is a major problem of the negative exponential distribution. Chin (1983) suggested the use of shifted negative distribution to generate vehicle headways in simulation model. The shifted negative exponential distribution was used to generate initial motor vehicle headways in several simulation models due to its simplicity and practical accuracy (Wu, 1994; Rathi and Nemeth, 1986). The distribution is shifted to avoid the small headways which would be unrealistic in practice.

According to the shifted negative exponential distribution, the probability distribution function is:

$$f(t)=\begin{cases}\lambda\ell^{-\lambda(t-\tau)} & (t\geq 0)\\ 0 & (t<0)\end{cases} \tag{4.5}$$

the distribution is:

$$F(t)=\begin{cases}1-\ell^{-\lambda(t-\tau)} & (t\geq 0)\\ 0 & (t<0)\end{cases} \tag{4.6}$$

the vehicle headway is

$$t=\tau-\frac{1}{\lambda}\ln(u)(0<u<1) \tag{4.7}$$

where,

λ is the mean arrival rate of vehicles (inverse of mean headways to be supplied by user), unit is veh/sec;

τ is the minimum headways;

u is the random deviate;

In Mei & Bullen's research, the statistical test on a high flow data set gave an excellent fit with a 0.3- or 0.4- seconds shift (Mei and Bullen, 1993).

Faghri and Egyháziová (1999) proposed the shift should be 0.5 seconds because it was observed that individual headways are rarely less than 0.5 seconds. Therefore, in this model, 0.5 seconds was adopted as the shift in initial headway generation process.

4.1.2.1.3 Flow chart of initial headway model

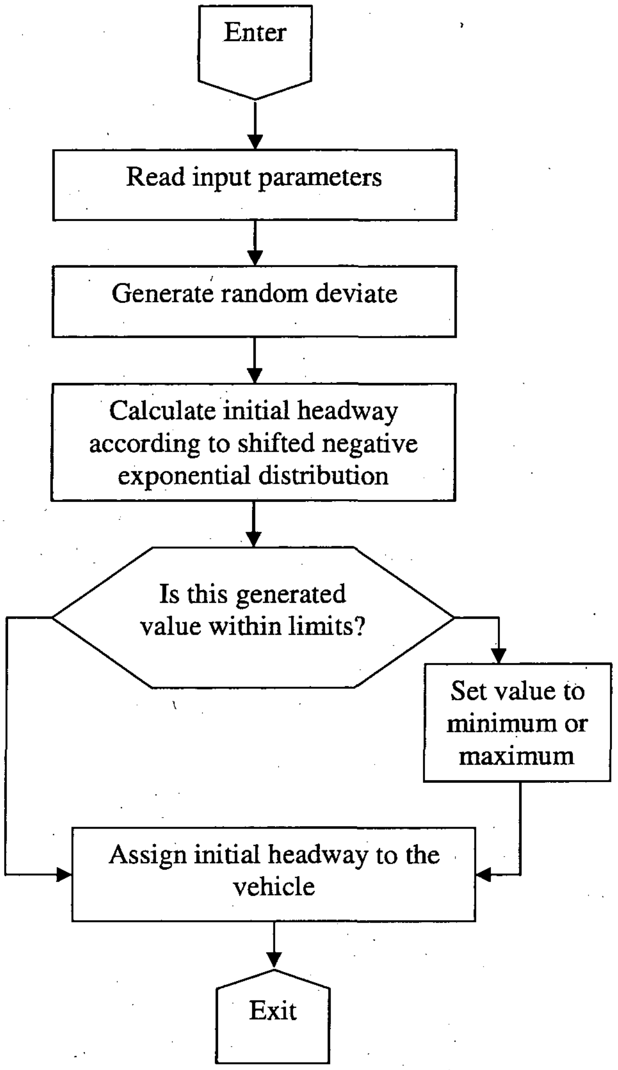


Figure 4.1 Flow chart of initial headway model

4.1.2.2 Speed

4.1.2.2.1 Initial speed

The initial speed is the speed of a vehicle when it just enters the simulation system. The initial speed depends on three factors: the vehicle's desired speed, the speed of front vehicle and the headway to it. Once the vehicle enters the simulated section, the speed will be adjusted according to the car following model.

4.1.2.2.2 Desired speed

The desired speed of a vehicle is the speed selected by a driver when travelling under free flow conditions. The vehicle will never be allowed to have a speed greater than the desired speed. There are many factors that may affect vehicle's speed, such as road layout, weather, and traffic. The desired speed is limited only by the capabilities of vehicles and by the wishes of their drivers. Research shows vehicle speed has a normal distribution (Chien et al., 2001). Therefore, normal distribution was applied for desired speed in this study.

If $X \sim N(\mu,\sigma^2)$, the probability formula is :

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma} e^{-\frac{(x-\mu)^2}{2\sigma^2}}, (\sigma > 0) \tag{4.8}$$

where,

x is motor vehicle speed;

μ is expectation, that is the mean vehicle speed;

σ is standard deviation;

Many researchers have reported that the coefficient of variation (the ratio of standard deviation to mean) to be constant over a wide range of mean speeds. Results were found by Mintsis (1982), with the values of 0.137 and 0.125 for cars travelling on single and dual carriageways respectively. The corresponding values for heavy vehicles were 0.118 and 0.11. According to data collected in Beijing, China, the mean speed was 10.87 m/s, and the sd was 1.2, therefore the coefficient of variation was 0.11, was close to the value in the literature.

In this study case, the mean of desired speed and standard deviation have to be supplied with the local value because it varies from place to place.

4.1.2.2.3 Flow chart of desired speed model

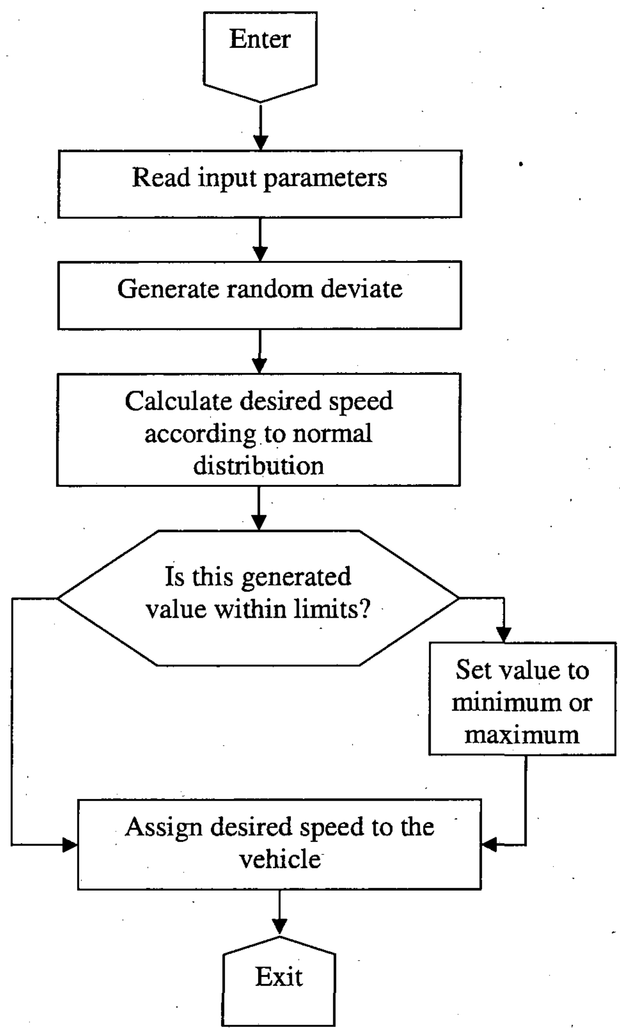


Figure 4.2 Flow chart of desired speed model

4.1.3 Car-following model

The way in which the driver of one vehicle in a stream of traffic reacts to the preceding vehicle is called car following. "Car-following itself, forms one of the main processes in all microscopic simulation models as well as in modern traffic flow theory (Brackstone and McDonald, 1999)." There have been numerous efforts to develop a satisfactory car-following model. In the 1950s to 70s, the car-following models attempted to mathematically describe the reaction of a driver to the motion of the immediately preceding vehicle in which the following driver either accelerates or decelerates according to the magnitude of the stimulus. Most of the car-following models at that time were based on a combination of mathematical and psychological reasoning and extensive experiments. In other words, they were constructed on a descriptive basis. From the early 1980s, as computer technology developed rapidly, more complicated simulation could be handled in personal computers. Car-following models included more driver and vehicle characteristics such as the behaviour of traffic in congested and non-congested conditions.

Some existing commonly used car-following models are reviewed and described below. After a comparison, one car-following model is proposed for application in the program for this research.

4.1.3.1 Review of car-following models

4.1.3.1.1 Linear car-following model

Most of the early period descriptive car-following models have the similar form of

$$a_n(t+\tau) = \lambda(t) \cdot \beta(t) \tag{4.9}$$

Where,

$a_n(t+\tau)$ is the acceleration or deceleration of the follower at time t+τ;

τ is reaction time;

$\lambda(t)$ is the sensitivity at time t;

$\beta(t)$ is the stimulus at time t;

The simplest car-following model is the linear model extended by Chandler (Chandler et al., 1958; Hoque, 1994).

$$a_n(t+\tau) = \lambda(V_n(t) - V_{n+1}(t)) \tag{4.10}$$

Where,

λ is the constant of sensitivity;

$V_n(t) - V_{n+1}(t)$ is the relative speed of following and leading vehicle at time t;

4.1.3.1.2 Non-linear car-following model (GHR model)

Gazis et al. (1961) proposed a non-linear model (GHR) where the sensitivity is inversely proportional to the space between vehicles. The model has the form:

$$a_n(t+\tau) = \frac{\lambda(V_n(t) - V_{n-1}(t))}{X_n(t) - X_{n+1}(t)} \tag{4.11}$$

Where,

λ is the constant of sensitivity;

$X_n(t) - X_{n+1}(t)$ is the spacing of following and leading vehicle at time t;

4.1.3.1.3 Collision avoidance models (CA model)

The base relationship of CA model does not describe a stimulus-response type function as proposed by the GHR model, but seeks to specify a safe following distance (through the manipulation of the basic Newtonian equations of motion), within which a collision would be unavoidable, if the driver of the vehicle in front were to act 'unpredictably'.

Gipps (1981) proposed a CA model, which was derived by setting limits on the performance of driver and vehicle and using these limits to calculate a safe speed with respect to the preceding vehicle. In this model, it is assumed that the driver of the following vehicle selects his or her speed to ensure that he can bring his vehicle to a safe stop should the vehicle ahead come to a sudden stop. The original Gipps's following model is:

$$v_n(t+\tau) = \min\{v_n(t) + 2.5a_n\tau(1 - v_n(t)/V_n)(0.025 + v_n(t)/V_n)^{1/2},$$
$$b_n\tau + \sqrt{(b_n^2\tau^2 - b_n[2[x_{n-1}(t) - s_{n-1} - x_n(t)] - v_n(t)\tau - v_{n-1}(t)^2/\hat{b}])}\ \}$$

(4.12)

where,

$v_n(t+\tau)$ is the speed of subject vehicle n at time t+τ;

$v_n(t)$ is the speed of subject vehicle n at time t;

a_n is the maximum acceleration which the subject vehicle n wished to undertake;

V_n is the desired speed of subject vehicle n;

b_n is the maximum deceleration which the subject vehicle n wished to undertake;

$x_{n-1}(t)$ is the location of the front vehicle n-1 at time t;

$x_n(t)$ is the location of the subject vehicle n at time t;

s_{n-1} is the reserve safety distance, that is the vehicle length plus a margin into which the subject vehicle is not willing to intrude, even at rest;

$\hat{b}$ is the deceleration of front vehicle n-1;

4.1.3.1.4 FLOWSIM car-following model

Fuzzy logic based models typically divided their inputs into a number of overlapping "fuzzy sets"; each one describing how adequately a variable fits the description of a "term". A set of driving rules have been compiled and produced for implementation in a fuzzy environment.

The FLOWSIM car-following model is a fuzzy logic based model developed at the University of Southampton. Emphasis was placed on the research undertaken to establish fuzzy sets and systems for driving behaviour models, the collection of data on appropriate driving behaviour, fuzzy sets and systems calibration, and model validation.

The FLOWSIM car-following model has two principal variables to the decision making process, which are relative speed (DV) and distance divergence (DSSD). Each variable consists of several overlapping fuzzy sets. The rule base of the car-following model describes a following vehicle's response to the change of relative speed and headway to the leading vehicle according to its free speed and desired safe following distance. The model was successfully validated on a large independent data base.

4.1.3.2 Proposed car-following model

It is important to select a suitable car-following model in microscopic simulation, as car-following plays the most important role of representing an individual vehicle's movement in a traffic stream.

Linear models, GHR model and Gipps model are all commonly used car-following models.

A great deal of work has been performed on the calibration and validation of the GHR model. However, it is now being used less frequently, significantly because of the large number of contradictory findings as the correct values of m and l. The parameters l, k, and m have no obvious connection with identifiable characteristics of vehicle.

Since the developments by Gipps, the CA model continues to see widespread use in simulation models. Part of the attractiveness of this model is that it may be calibrated using common sense assumptions about driver behaviour, needing only the maximum breaking rates that a driver wish to use, and predicts other drivers will use, to allow it to fully function. Although it produces acceptable results on, for example, comparing the simulated propagation of disturbances against empirical data, there are a number of problems which cannot easily be solved. For example, if one examines the `safe headway' concept, we see that this is not a totally valid starting point, as in practice a driver may consider conditions several cars down stream, basing his assumption of how hard the vehicle in front will decelerate on the `preview information' obtained.

Fuzzy logic based car-following models allow a more realistic representation to the uncertainties of driver perception and decisions than conventional models (Wu et al., 2003). Model validation results have shown that the fuzzy logic based model (FLOWSIM) could closely replicate real systems and in test cases performed better than other common deterministic models such as the "GHR", "Gipps", and "MISSON" models (Wu et al., 1998; Wu et al., 2003). Therefore, the FLOWSIM type fuzzy logic car-following processes have been employed in this simulation.

During the car following decision making processes there are two significant variables of relative speed (DV) and distance divergence (DSSD).

$$DSSD = DS / sd \tag{4.13}$$

where,

DS is vehicle separation;

sd is desired following distance;

Fuzzy set terms used in the car-following model are listed below:

Table 4.1 Fuzzy Sets

Relative speed (DV)	Distance divergence (DSSD)	Driver response (acceleration rate)
Opening fast (V1)	Much too far (S1)	Strong acceleration
Opening (V2)	Too far (S2)	Light acceleration
About Zero (V3)	Satisfied (S3)	No action
Closing (V4)	Too close (S4)	Light deceleration
Closing fast (V5)	Much too close (S5)	Strong deceleration

A typical fuzzy rule for the car-following model in natural language has the form below:
If the distance divergence, DSSD, is too great and the relative speed, DV, is closing, then the driver's response is no action (i.e. keep current speed).

As relative speed (DV) and distance divergence (DSSD) are the two basic car-following variables, vehicle accelerations and decelerations will be decided according to the relative speed between the leading vehicle and the subject vehicle, the relative distance between the leading vehicle and subject vehicle, and the desired speed of subject vehicle.

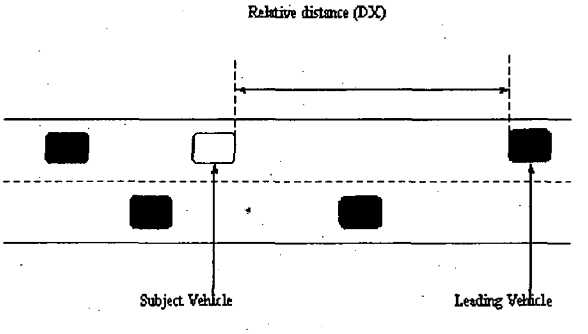


Figure 4.3 Car following

1. Variable 1: Relative speed (DV)

$$DV = V\_LeadingVehicle - V\_SubjectVehicle \tag{4.14}$$

When calculating the relative speed, several cars in front of the subject vehicle will be taken into consideration, which fits drivers' ability to look ahead in the traffic stream in most situations.

2. Variable 2: Distance divergence (DSSD)

$$DSSD = DS / sd \tag{4.15}$$

Where,

sd is the desired gap, which follows a normal distribution;

$$DS = DX / V\_SubjectVehicle \tag{4.16}$$

Where,

DX is the relative distance;

The detailed definitions of parameters which were used to calculate the variables in the FLOWSIM car-following model are illustrated in Appendix A.

In the FLOWSIM fuzzy logic based car-following model, 5-group experimental values for fuzzy set (DV) and 5-group experimental values for fuzzy set (DSSD) were set. Corresponding 25-group results for acceleration/deceleration were set in 25-group experimental values. When calculating the motor vehicle's acceleration/deceleration, the pair of (DSSD, DV) will be compared with the fuzzy sets.

4.1.3.3 Flow chart of car-following model

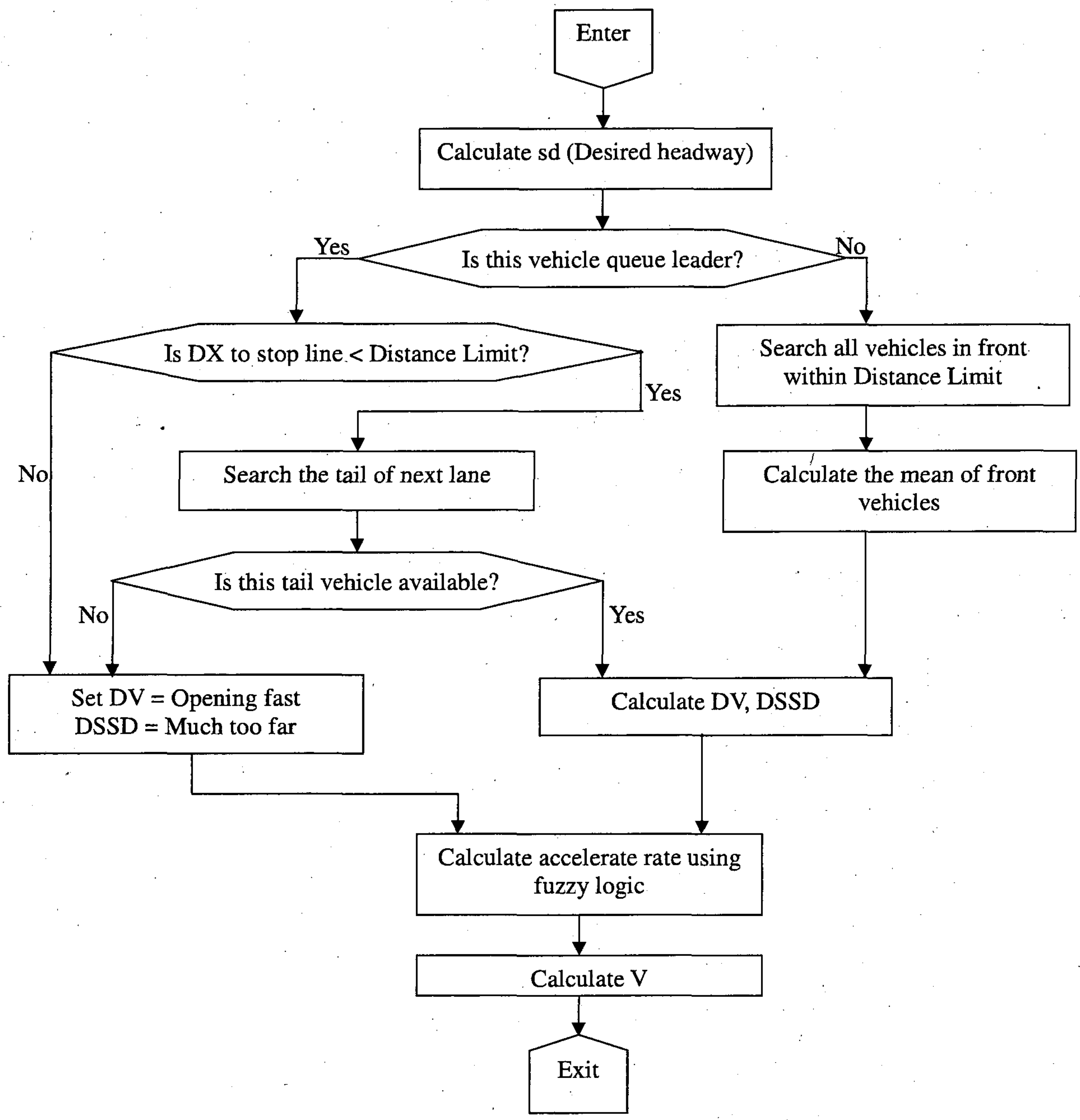


Figure 4.4 Flow chart of car-following model

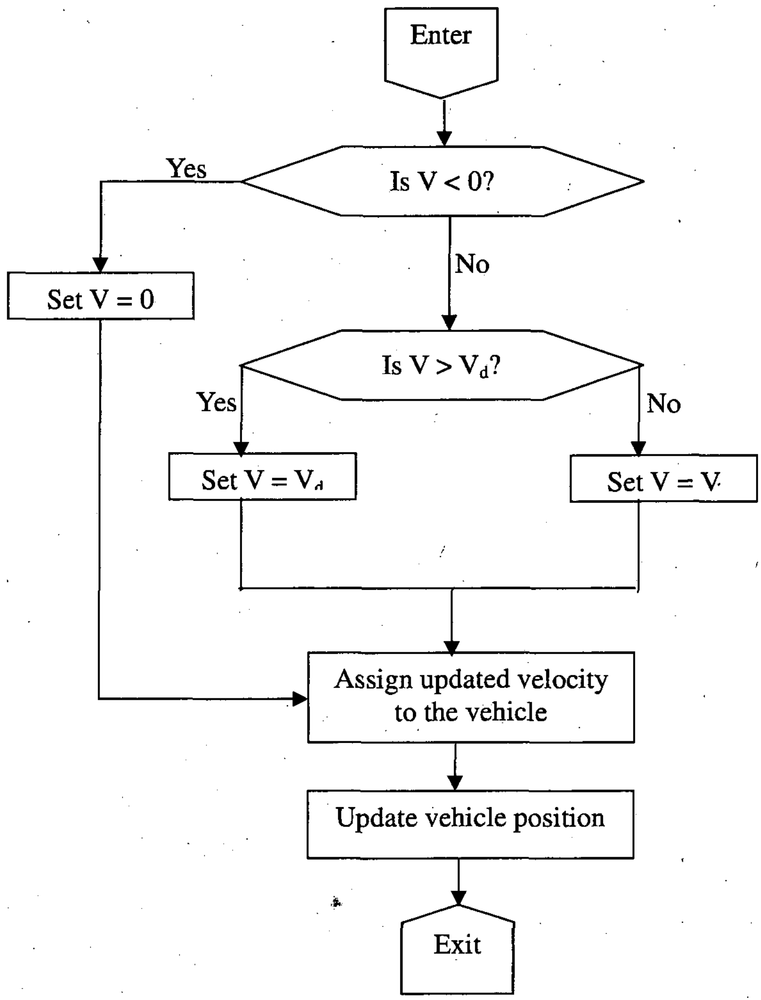


Figure 4.5 Flow chart of update of vehicle speed and position

4.1.4 Lane-changing model

4.1.4.1 Introduction

Lane changing is defined as the transfer of a vehicle from one lane to an adjacent lane. The mechanism of lane changing is more complex than car following, because "the decision to change lane depends on a number of objectives, and at times these may conflict (Gipps, 1986)." Thus, in coming to a decision concerning lane changing, a driver must be able to reconcile both short-term and long-term aims.

Some factors that influence drivers to change lanes are listed below:

Whether it is physically possible and safe to change lanes;

The location of permanent obstructions;

The presence of transit lanes;

The driver's intended turning movement;

The presence of heavy vehicles;

Speed;

The decision for lane changing originates from a desire or demand and can be optional or forced. If a lane changing is unavoidable, it is defined as "forced lane-changing". However, if the desire of lane changing is only to gain the advantage of speed, it is called "optional lane-changing". "One of the main purposes of changing lanes is to gain some speed advantages (Gipps, 1986)".

4.1.4.2 Proposed lane-changing model

Gipps (1986) lane-changing model is one of the most commonly used models. However, Gipps' model does not consider an increase in a driver's desire to change lane when he or she is delayed behind a slower vehicle (Sultan and McDonald, 2001). Also, does not consider the influence of driver's character. FLOWSIM lane-changing model is a new lane-changing model based on fuzzy logic, and model validation results have shown that FLOWSIM fuzzy logic based lane-changing model can closely replicate real systems and in test cases have performed better than some common models such as the "Gipps" model (Wu et al., 1998). Therefore, FLOWSIM fuzzy logic based lane-changing model has been employed in this simulation.

In the FLOWSIM lane-changing model, two logics are employed for forced lane changing and optional lane changing. Which logic is applied depends on some factors. These factors that affect the lane-changing decision making process are listed below:

- Turn intention: Turn intention is the first factor that affects the lane-changing decision-making. If the motor vehicle is not in the target lane (the correct lane through which the subject vehicle can enter the next lane that is in its route), then it's possible for this subject vehicle to change lane. Whether changing to the nearside or offside lane depends on whether the target lane is in the offside or nearside of subject lane.

- The distance to the stop line: This is another factor that affects the lane-changing manoeuvre of a motor vehicle. If the subject vehicle is not in the target lane and its position is close to the stop line, then forced lane changing should be conducted. Otherwise, it's optional for the subject vehicle to change lane.

- The desired speed of the subject vehicle, the speed of front vehicle and rear vehicle in lateral lane.

Lane-changing behaviour falls into three patterns, depending on the distance to his intended turn.

- If the turn is remote, the optional lane-changing model works and the driver concentrates on gaining the advantage of speed.
- If the turn enters the zone the driver regards s middle distance, both the opportunity to improve speed and the intention to stay in the correct lane should be considered.
- If the turn is close, the speed is unimportant and the forced lane-changing model works.

4.1.4.2.1 Forced Lane Changing

For forced lane changing, the major consideration is safety. The drivers start to seek gaps to change lane to reach their desired direction. The gap between the front vehicle and the rear vehicle in the target lane should be measured. If the driver has been unable to find an acceptable gap and is, for example approaching a stop line, the lane changing logic allows the lane changer to look behind the adjacent vehicle and adjust the relative speed with respect to a later available gap.

4.1.4.2.2 Optional Lane Changing

The motivation for lane changing from the offside to the nearside lane and from the nearside to the offside lane is different. Therefore, two different models were taken into consideration in FLOWSIM lane-changing model: one is lane change to offside (LCO) and the other is lane change to nearside (LCN).

1. The logic for LCO

A driver's motivation to move to the offside lane is to get some speed benefit. In the FLOWSIM LCO model, there are two variables: "overtaking benefit" and "opportunity". The "overtaking benefit" can be obtained by measure of speed gain on the assumption that a LCO is carried out. "Opportunity" can be measured by the headway to the rear-approaching vehicle in the offside lane.

Table 4.2 Fuzzy set terms for the LCO model (Lane-changing)

Overtaking benefit	Opportunity	Intention of LCO
High (OB1)	Good (OP1)	High
Medium (OB2)	Moderate (OP2)	Medium
Low (OB3)	Bad (OP3)	Low

A typical fuzzy rule for the LCO model in natural language has the form:
If overtaking benefit is high and opportunity is good, then intention of LCO is high.

If the target lane of the subject vehicle is on the offside of the subject lane, the LCO will be carried out. Under optional conditions, whether this lane changing can be conducted depends on two variables, "overtaking benefit" and "opportunity". The "overtaking benefit" is the increased speed gained given that a LCO is carried out. "Opportunity" is to check whether there is enough gap between the front vehicle and the rear-approaching vehicle in the offside lane. "Opportunity" can be measured by the headway to the rear-approaching vehicle in the offside lane. The higher "overtaking benefit" and the higher "opportunity" result in higher potential to change to the offside lane.

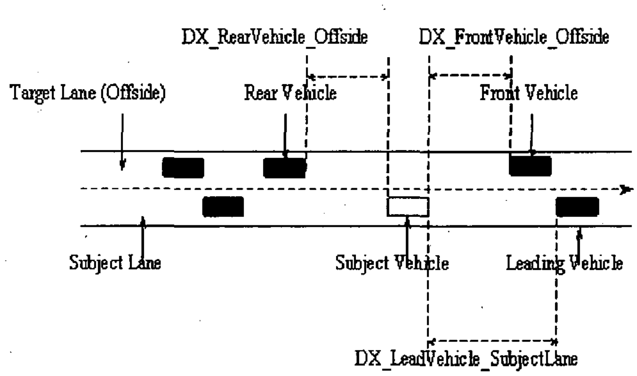


Figure 4.6 Lane changing (LCO)

2. The logic for LCN

A driver's motivation to move to the nearside lane is to reduce impedance to fast moving vehicles approaching from behind. In the FLOWSIM LCN model, there are two variables, "pressure from rear" and "gap satisfaction". The "pressure from rear" is the time headway to the following vehicle. "gap satisfaction" can be obtained by evaluating how long the vehicle can stay in the nearside lane without reducing speed.

Table 4.3 Fuzzy set terms for the LCN model (Lane-changing)

Pressure from rear	Gap satisfaction	Intention of LCN
High (PR1)	High (GS1)	High
Medium (PR2)	Medium (GS2)	Medium
Low (PR3)	Low (GS3)	Low

A typical fuzzy rule for the LCN model in natural language has the form:
If pressure from rear is high and gap satisfaction is high, then intention of LCN is high.

If the target lane of the subject vehicle is on the nearside of the subject lane, the LCN will be carried out. Whether this lane changing can be conducted depends on two variables, "pressure from rear" and "gap satisfaction". The "pressure from rear" is the pressure caused by the difference of speed and distance between the subject vehicle and the following vehicle. The "gap satisfaction" can be obtained by evaluating how long the vehicle can stay in the nearside lane without reducing speed. The higher "pressure from rear" and the higher "gap satisfaction" result in higher potential to change to the nearside lane.

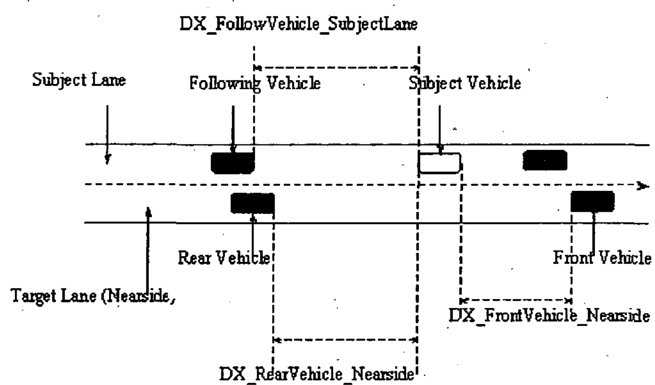


Figure 4.7 Lane changing (LCN)

The detailed definitions of some parameters, which are used to calculate the variables in FLOWSIM lane-changing model, are illustrated in Appendix B.

4.1.4.3 Lane-changing model flow chart

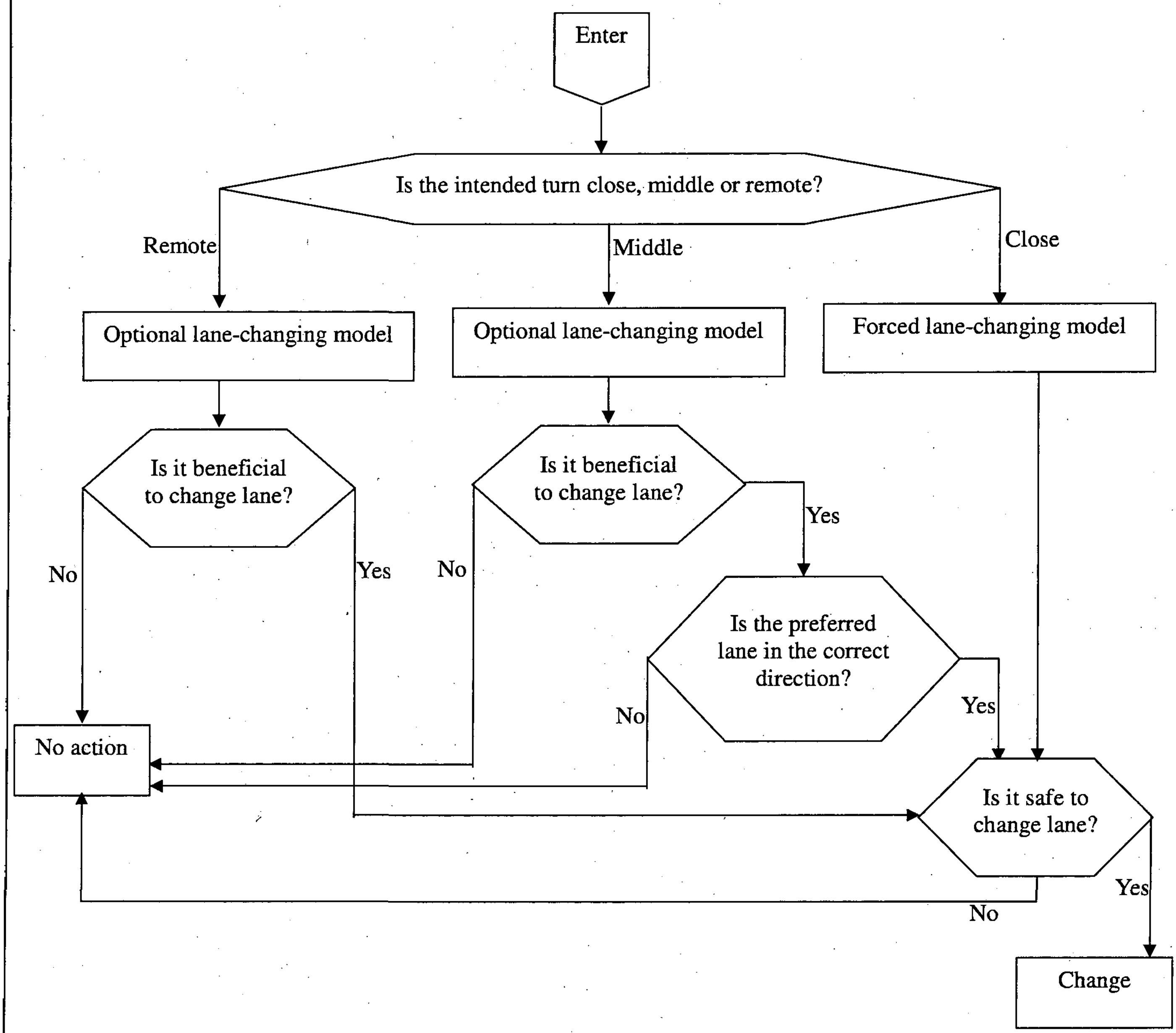


Figure 4.8 Flow chart of lane-changing model

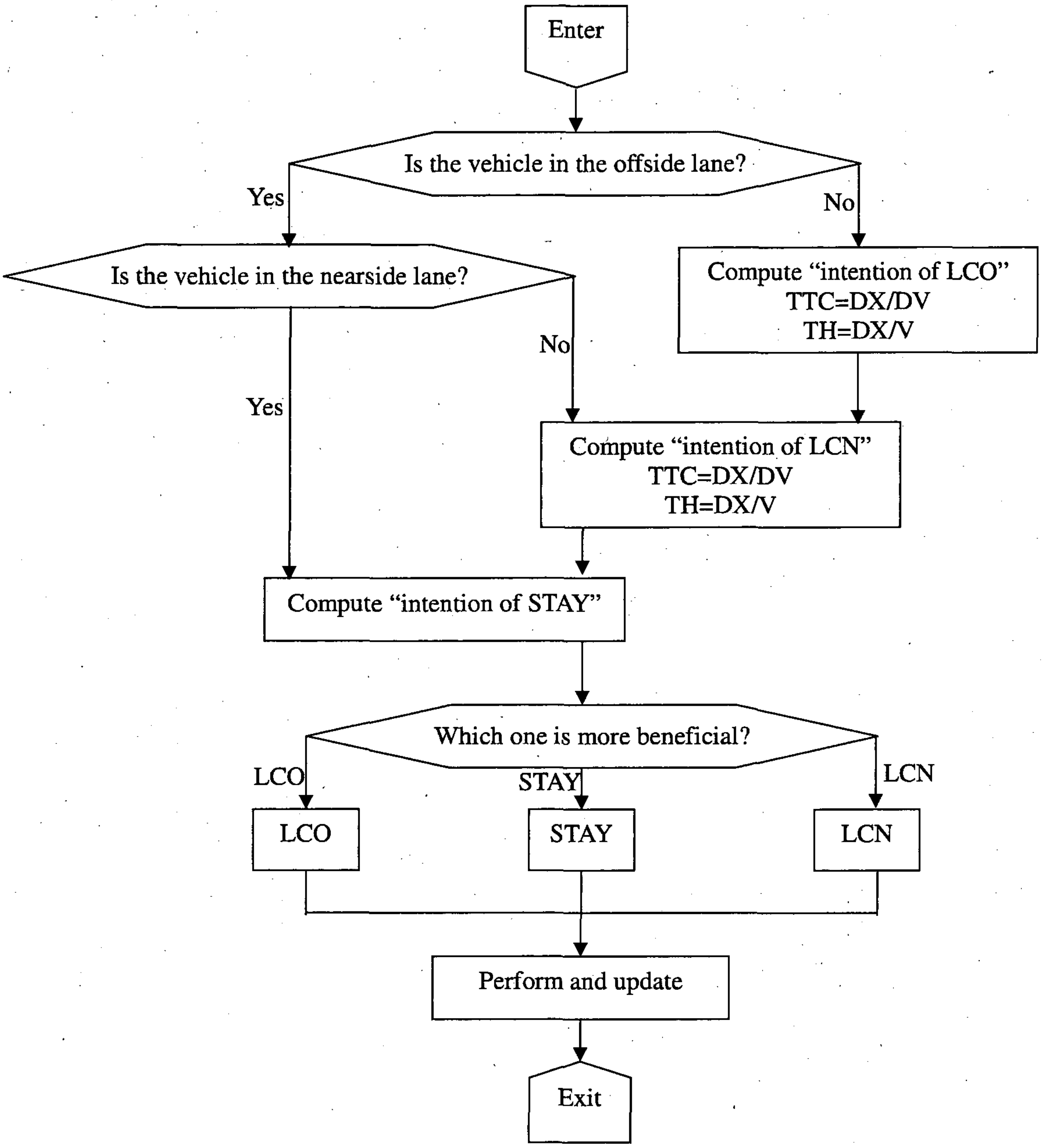


Figure 4.9 Flow chart of optional Lane-changing model

4.2 Bicycle models

4.2.1 Static characteristics

Bicycle types, size, the age and gender of the cyclist are factors, which may affect the behaviour of cyclists. "The classification of bicyclists into groups A, B, and C should be incorporated into the model (Faghri and Egyháziová, 1999)." Bicyclists were divided into three categories: Group A — advanced bicyclists; Group B — basic bicyclists; Group C — children. Group C is defined as a bicyclist who is too young to obtain a driver license. Group B is defined as someone who is old enough to possess a driver license, is moderately skilled, and has a basic, but not extensive, knowledge of bicycling. Group A is defined as an experienced, knowledgeable and skilled bicyclist who is old enough to possess a driver license.

A typical bicycle in the United States is 1.75 m in length with a handlebar width of 0.6 m. In the Netherlands, it has been reported that 95 percent of bicycles are less than 1.9 m in length and that 100 percent of bicycle handlebar widths are less than 0.75 m (Allen et al., 1998). The general size of bicycles in China is 1.9m in length, 0.6m wide and 2.25m high (Highway and Transportation Society of China, 1998). Because what is studied is the mixed traffic in Beijing, the unique size of 1.9 m in length and 0.6 m in width has been adopted in this model.

In addition, a bicyclist needs a certain amount of operating space. No bicyclist, at any speed, can ride a bicycle in a perfectly straight line. One U.S. source reports that a typical bicycle needs between 0.75m and 1.40m of width to operate. An older study in Davis, California, recommends a minimum width of 1.28m for bicycles with additional width at higher volumes (Miller and Ramey, 1975). In the Netherlands, 1.0 m of clear space is generally recommended for bicycles (Allen et al., 1998). In Germany, 1.0 m is reported as the normal width of one bicycle lane (Brilon, 1994). In China, the width when moving is 1.0m (Highway and Transportation Society of China, 1998).

4.2.2 Generation model

4.2.2.1 Initial headway

There are available distributions for bicycle arrivals, such as negative exponential, shifted negative exponential, gamma, log normal and so on. According to Opiela's (Opiela et al., 1980) research, the negative exponential and gamma distributions provide an acceptable representation of the actual arrival distribution in all cases. Because of the characteristics of the group, no shift was used. The gamma distribution is more complex and possesses a high degree of randomness. Therefore, a negative exponential distribution was chosen in generating the bicycle initial headway due to its simplicity and practical accuracy.

According to the negative exponential distribution, the probability distribution function is:

$$f(t)=\begin{cases}\lambda\ell^{-\lambda t}\ (t\geq 0)\\ 0\quad (t<0)\end{cases}$$

(4.17)

the distribution is:

$$F(t)=\begin{cases}1-\ell^{-\lambda t}\ (t\geq 0)\\ 0\quad (t<0)\end{cases}$$

(4.18)

According to the negative exponential distribution, the bicycle headway is

$$t=-\frac{1}{\lambda}\ln(u)(0<u<1)$$

(4.19)

where,

> λ is the mean arrival rate of bicycles (inverse of mean headways to be supplied by user), unit is bic/sec;
>
> u is the random deviate;

4.2.2.2 Bicycle desired speed

4.2.2.2.1 Bicycle speed review

"Bicycle travel speed influences stopping distances, clearance intervals, and turning radii at intersections (Opiela et al., 1980)". It is necessary to determine bicycle travel speed for accurate replication of operation. Bicycle travel speed has been measured by several researchers, including Opiela et al. (1980), Forester (1983), Taylor (1993), and Pein (1997), in various environments.

In Sweden, the 85th percentile free-flow speed of bicycles is reported to be between 16 km/h and 28 km/h (Vagverk, 1977).

Opiela et al. (1980) observed the intersection approach speeds of 486 bicycles. The study was conducted primarily in Michigan on university campuses in the United States. "The locations were selected to be sufficiently distant from the intersections to minimize the effect of traffic controls on approach speed". The results noted that the average speed of bicycle traffic was 20.8 km/hr, and the overall speeds ranged from a high of 39.6 km/hr to a low of 3.4 km/hr. Data obtained in Opiela's study were collected in campus, therefore the result is biased to the young adult age group. The standard deviation of bicycle speeds was 4.2km/h.

Forester (1983) recommended bicycle speeds of 24.1 km/hr to 32.2 km/hr for adult and 16.1 km/hr for children.

Taylor (1993) suggested the bicycle "speeds in the range of 16.1 km/hr to 32.2 km/hr from the data collected in his study, previously collected data, AASHTO, and Forester".

Pein (1997) researched the cyclists crossing time from a full stop at a variety of trail-roadway intersections along the Pinellas Trail in Pinellas County, Florida. In his study, the estimated crossing speed is 12.7 km/hr, which compares favourably with values noted in the AASHTO Guide.

The AASHTO Guide (AASHTO, 1999) recommends speed of 19km/h for Group A cyclists, 13 km/h for Group B cyclists, and 10 km/h for Group C cyclists in the United States.

One study in China reported observed average bicycle speeds at various locations between 10 km/h and 16 km/h with an overall mean of approximately 12 km/h (Lui, 1991).

Another Chinese study reported observed average bicycle speeds between 12 km/h and 16.3 km/h, with an overall mean of approximately 14 km/h (Liu et al., 1993B).

A more recent Chinese study reported peak-hour free-flow speeds of 18.2 km/h where bicycle traffic was separated from motor vehicles by a barrier and 13.9 km/h at locations without a lane barrier (Wei et al., 1997).

Huang and Wu (2004) observed 531 bicycles, and the crossing speed for bicycle traffic at signalized intersections in Beijing was measured. The maximum through speed was 20.4 km/h and the minimum through speed is 5.3 km/h. The maximum left-turning speed is 22.3 km/h and the minimum left-turning speed is 6.9 km/h. "The average through speeds for male and female cyclists were 11.3 km/h and11.6 km/h, respectively, but the difference between them was tested to be not significant at a 90% level. The average left-turning speeds of male and female cyclists were 12.9 km/h and 13.4 km/h, respectively, and the difference found was not significant at a 90% level, either."

In summary, free-flow bicycle speed appears to be somewhere between 10 km/h and 28 km/h with a majority of the observations being between 12 km/h and 20 km/h (Allen et al., 1998).

Opiela et al. (1980) researched the distributions of bicycle speeds and "the normal distribution was found to provide a close approximation to the data collected".

Huang and Wu (2004) also analysed the speed distributions of the through bicycle speeds and the left-turning bicycle, and "the normal distribution was proven to provide the closest approximation to all classes of the collected data by using the chi-square test".

4.2.2.2.2 Proposed bicycle desired speed

According to the review of past-published research in characteristics of bicycle, a common conclusion is that the normal distribution can provide a good fit to the distribution of bicycle speeds. Therefore, the normal distribution was employed in this study to calculate the desired speed of bicycles.

If $X \sim N(\mu,\sigma^2)$, the probability formula is :

$$f(x)=\frac{1}{\sqrt{2\pi}\sigma}e^{-\frac{(x-\mu)^2}{2\sigma^2}},(\sigma>0) \tag{4.20}$$

where,

x is bicycle speed;

μ is expectation, that is the mean bicycle speed;

σ is standard deviation;

The average speed of bicycles and standard deviation are different in different areas. For example, the average speed in China is slower than in UK. That is because the bicycle speed is affected by the quality of facilities and the performance of bicycles. The mean speed and standard deviation also vary in different environments. Therefore, the mean bicycle speed and standard deviation should be provided as local value in the model.

4.2.2.2.3 Flow chart of bicycle desired speed model

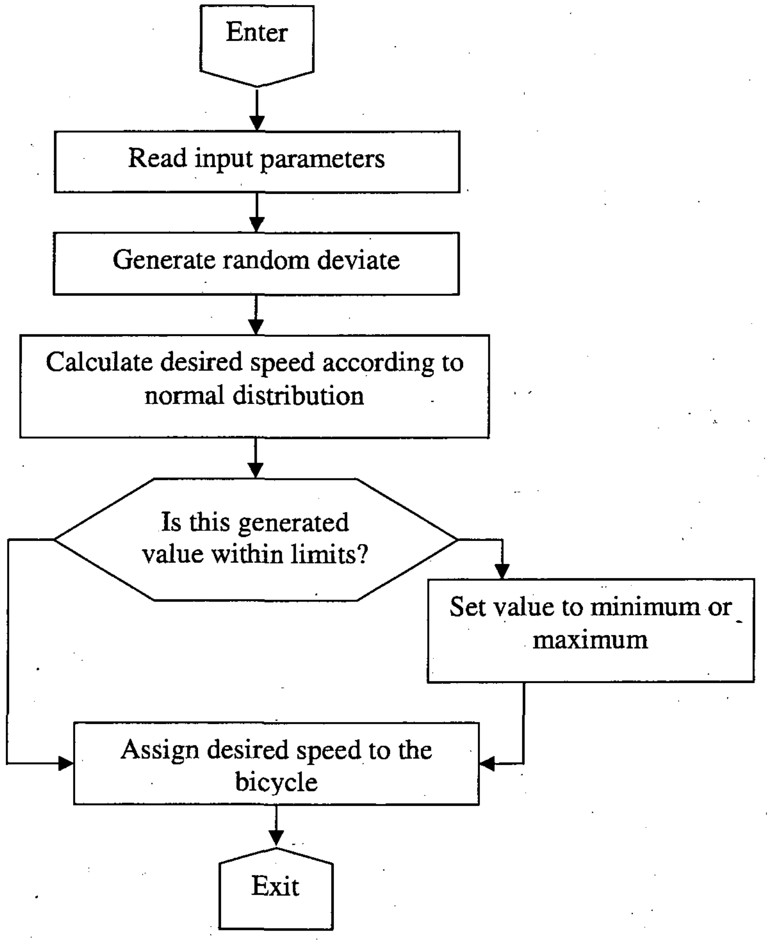


Figure 4.10 Flow chart of bicycle desired speed model

4.2.3 Bicycle acceleration/deceleration behaviour model

Mathematically, bicycle deceleration/acceleration model can be expressed as:

$$a = \begin{cases} a^{cf} & (Not\_Head\_Bicycle) \\ a^{ff} & (Head\_Bicycle) \end{cases} \qquad (4.21)$$

where,

a^{cf} is the bicycle following acceleration/deceleration;

a^{ff} is the bicycle free-flow acceleration/deceleration;

4.2.3.1 Bicycle deceleration review

AASHTO Guide (AASHTO, 1999) notes cyclist deceleration is 1.2 to 2.5 m/sec^2.

Taylor (1993) measured comfortable bicycle deceleration rates by experimentation on willing cyclists who were told they had just seen a yellow light. The maximum bicycle deceleration was considered to be a bit less than 5.5 m/sec^2 on the safe side. "If one considers braking in wet weather, the maximum attainable deceleration is about 4 m/sec^2."

From observation, the bicycle braking situations before a stop line can be divided into two major types: one is easy deceleration where the cyclist expects to stop when approaching the intersection, the other type is emergency deceleration where a cyclist attempting to cross the intersection has to stop in a comparatively short length when the signals change suddenly or is stopped by the guide. In this data analysis, the average braking length was used to separate the two types of braking situations.

Forester (1983) recommended bicycle deceleration of 4.6 m/sec^2 for adults and 2.4 m/sec^2 for children.

Huang and Wu (2004) studied 414 bicycles before they crossed the intersection, and got 408 valid deceleration data sets. The maximum deceleration was -1.92 m/sec^2, and the minimum deceleration was -0.16 m/sec^2. The average deceleration was -0.57 m/sec^2. In Huang's research, "the distribution of bicycle decelerations were analysed and the log normal distribution was found to provide a close approximation to the data collected." Huang suggested the values of two parameters for the log normal distribution. μ=0.66, σ=0.42.

4.2.3.2 Bicycle acceleration review

AASHTO Guide (AASHTO, 1999) notes cyclist acceleration is 0.5 to 1.0 m/sec^2.

Taylor (1993) measured bicycle acceleration from cruising speed for a small number of experimental participants and the maximum acceleration rate was 1.95 m/sec^2.

Pein (1997) estimated bicycle acceleration from a standing start from trail-side observations. His estimation method also might be used with similar on-road data. The estimated mean acceleration is 1.07 m/sec^2, which compare favourable with values noted in the AASHTO Guide.

Huang and Wu (2004) researched 201 bicycles crossing the intersection in Beijing, and 195 valid acceleration data were obtained. The maximum acceleration is 1.23 m/sec^2, and the minimum acceleration is 0.23 m/sec^2. The average acceleration is 0.52 m/sec^2. In Huang's research, "the distribution of bicycle acceleration was analysed and the log normal distribution was found to provide a close approximation to the data collected." Huang suggested the values of two parameters for the log normal distribution. μ=0.69, σ=0.32.

4.2.3.3 Bicycle free-flow deceleration

If the subject bicycle is the head of a platoon of bicycles in the subject link, and it needs to decelerate when approaching the stop line of the subject link, the deceleration of the subject bicycle is called free-flow deceleration.

Because there are few researches on bicycle free-flow deceleration model, some existing motor vehicle deceleration models are referred to.

1. Uniform deceleration model

The general form of uniform deceleration model is:

$$a = c \tag{4.22}$$

where,

c is constant parameters;

2. Linear Deceleration model

The general form of linear deceleration model is:

$$a = c_1 + c_2 t \tag{4.23}$$

$$a = c_1 + c_2 v \tag{4.24}$$

$$a = c_1 + c_2 x \tag{4.25}$$

where,

c_1 and c_2 are constant parameters;

3. Proposed free-flow deceleration model

Lee found that the uniform deceleration model does not represent the observed behaviour of actual drivers when considered at a microscopic level (Lee et al., 1977; Hoque, 1994). A comparison between observed data and that from uniform deceleration model demonstrated that the uniform deceleration model generated a higher speed during the initial stages of the deceleration (Wu, 1994).

Free-flow deceleration can be calculated from a free-flow deceleration model based on Hossain. Hossain and McDonald (1998B) found the rate of deceleration depends on the free flow deceleration distance, which is between the end of the approach and the point from which the lead bicycle starts to react to the junction. The subject bicycle is given a constant deceleration value from a distance. "The distance from the end of the approach was found to be normally distributed from the field data". According to Hossain's research, the parameters of the free-flow deceleration distance model for non-motorised traffic is below:

Table 4.4 Free-flow deceleration distance

Traffic Type	Fitted Distribution	Minimum (m)	Maximum (m)	Mean (m)	Standard Deviation
Non-motorised	Normal	44.5	60.96	51.57	3.98

Therefore, in this model, the free-flow deceleration distance is determined according to the normal distribution, and the two parameters for the normal distribution are set to be μ=51.57 and σ=3.98 respectively. The bicycle free-flow deceleration can be calculated from the following equation:

$$a_d^{ff} = -\frac{v^2}{2\Delta S} \tag{4.26}$$

where,

a_d^{ff} is the calculated bicycle free-flow deceleration;

v is the bicycle current speed;

ΔS is the deceleration distance to the stop line;

The bicycle deceleration values in this model are limited to be no more than the maximum bicycle deceleration. According to previous research on bicycle deceleration, the maximum bicycle deceleration varied according to different environments and should be provided by users with local value.

4.2.3.4 Bicycle free-flow acceleration

If the subject bicycle is the leader in the subject lane, and can accelerate freely, its acceleration is called free-flow acceleration.

Because there are few researches on bicycle free-flow acceleration model, some existing motor vehicle acceleration models are referred to.

1. Uniform acceleration model

The uniform acceleration model is the simplest model. Greenshields et al. (1947) proposed a constant acceleration model.

$$a = 3 \qquad (ft/\sec^2) \tag{4.27}$$

2. Non-uniform acceleration model

Drew (1968) suggested an acceleration model where the acceleration is proportional to the speed as follows:

$$a = a_0(1 - v/v_d) \qquad (v \le v_d) \tag{4.28}$$

where,

a_0 is the initial acceleration;

v is the current speed;

v_d is the desired speed;

3. Linear acceleration model

Anderson et al. (1968) proposed a linear acceleration model. This model assumes a constant rate of acceleration α,

$$a = a_0 + \alpha t \qquad 0 \le t \le T$$

(4.29)

where,

a_0 is the initial acceleration;

T is time taken to reach desired speed;

Substitute equation 4.29 into kinematic relationship,

$$v = v_0 + a_0 t + \frac{\alpha t^2}{2}$$

(4.30)

Equation 4.29 is replaced by equation 4.30,

$$a^{ff} = a_0 (1 - \frac{v}{v_d})^{½} \qquad (0 \le v \le v_d)$$

(4.31)

where,

a^{ff} is the calculated free-flow acceleration;

a_0 is the initial acceleration;

v is the current speed;

v_d is the desired speed;

4. Proposed free-flow acceleration model

Whilst the uniform acceleration model is simple, Anderson et al. (1968) found it to be unsuitable for the purpose of microscopic simulation.

Hossain and McDonald (1998B) proposed using a linear acceleration model to compute bicycle free-flow acceleration. Linear acceleration model was adopted in this study due to its accuracy and simplicity.

$$a^{ff} = a_0 (1 - \frac{v}{v_d})^{½} \qquad (0 \le v \le v_d)$$

(4.32)

where,

a^{ff} is the calculated bicycle free-flow acceleration;

a_0 is the initial bicycle acceleration;

v is the bicycle current speed;

v_d is the desired speed of the bicycle;

The bicycle acceleration values in this model are limited to be no more than the maximum bicycle acceleration. According to previous research on bicycle acceleration, the maximum bicycle acceleration varies according to different environments and should be provided by users with local value.

4.2.3.5 Bicycle following deceleration/acceleration

For the situations of bicycle following a bicycle or a motor vehicle, there is less published research work. Therefore, some commonly used car-following models were used for guidance. The bicycles in Beijing are in competition with each other to gain space and speed, and the bicycle following process can best be described as the maintenance of a safe situation based on a collision avoidance model. Therefore, a simple bicycle model based on the Gipps's (Gipps, 1981) car-following model was adopted in this simulation model. Another reason using Gipps's car-following model is the parameters in that model correspond to obvious characteristic of drivers and vehicles so that most can be assigned values without resorting to elaborate calibration exercises. The Gipps's car-following model originally was developed for motor vehicle following, therefore some adjustment should be made for the specific characteristics of bicycles. Because of the high lateral flexibility of bicycles, the reserve safety margin was eliminated. The original Gipps's following model is:

$$v_n(t+\tau) = \min\{v_n(t) + 2.5a_n\tau(1 - v_n(t)/V_n)(0.025 + v_n(t)/V_n)^{1/2},$$
$$b_n\tau + \sqrt{(b_n^2\tau^2 - b_n[2[x_{n-1}(t) - s_{n-1} - x_n(t)] - v_n(t)\tau - v_{n-1}(t)^2/\hat{b}])}\ \} \tag{4.33}$$

where,

$v_n(t+\tau)$ is the speed of subject vehicle n at time t+τ;

$v_n(t)$ is the speed of subject vehicle n at time t;

a_n is the maximum acceleration which the subject vehicle n wished to undertake;

V_n is the desired speed of subject vehicle n;

b_n is the maximum deceleration which the subject vehicle n wished to undertake;

$x_{n-1}(t)$ is the location of the front vehicle n-1 at time t;

$x_n(t)$ is the location of the subject vehicle n at time t;

s_{n-1} is the reserve safety distance, that is the vehicle length plus a margin into which the subject vehicle is not willing to intrude, even at rest;

$\hat{b}$ is the deceleration of front vehicle n-1;

The bicycle following deceleration/acceleration values in this model are limited to be within the boundary between the maximum deceleration and the maximum acceleration.

4.2.3.6 Flow chart of bicycle acceleration/deceleration model

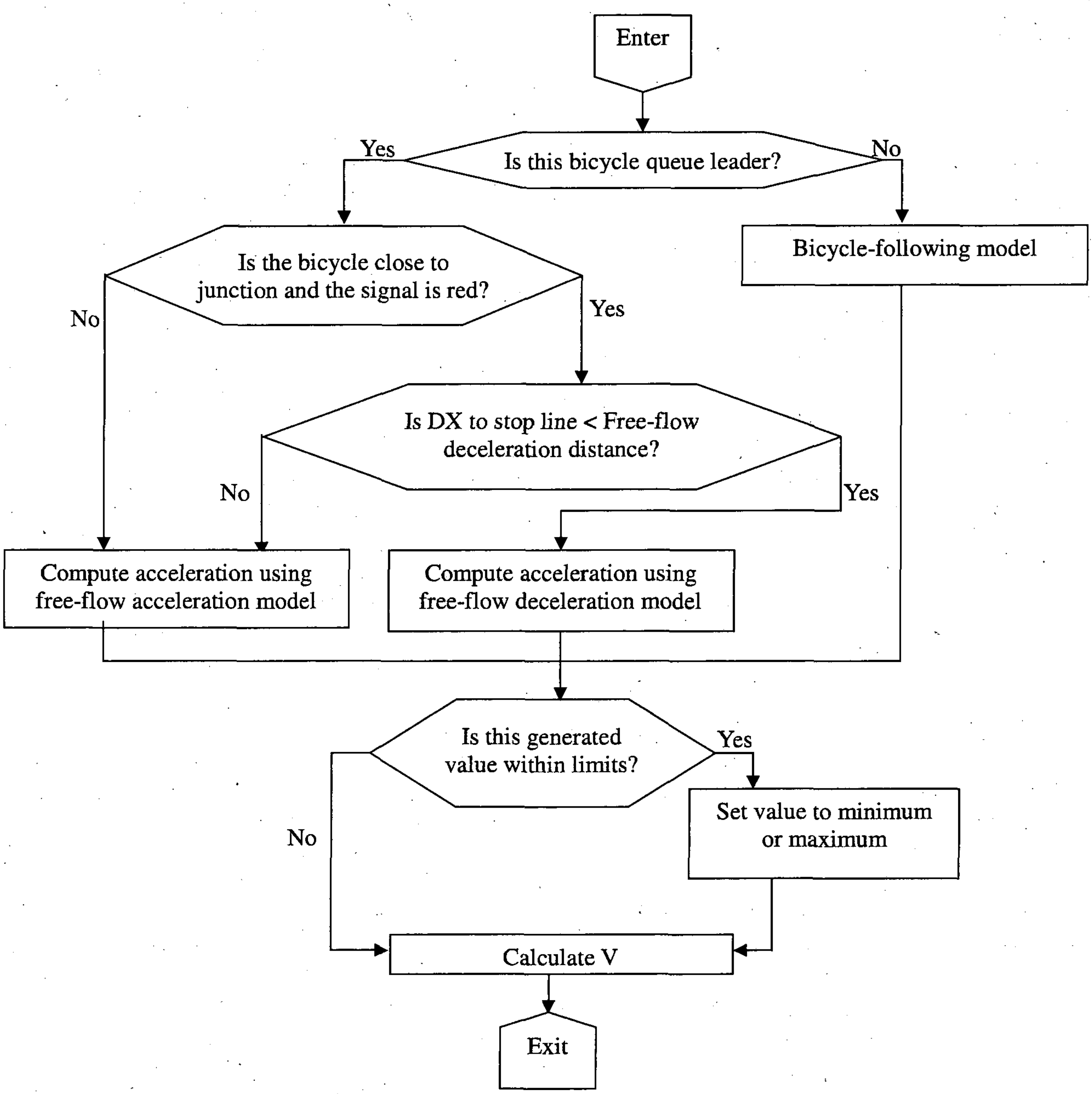


Figure 4.11 Flow chart of bicycle acceleration/deceleration model

4.2.4 Bicycle group characteristic

Unlike motor vehicles, which move or wait in a linear queue, bicycles appear as a "group" characteristic under three conditions below:

- A number of bicycles riding close together when approaching to the intersection where the signal for cyclists is red;
- A number of bicycles departing from intersection where the signal for cyclists turning from red to green;
- A number of bicycles waiting before the stop line before the traffic light turns green;

Xu (1995) observed the bicycle waiting group density at eight intersections in Beijing, and found that the average of maximum waiting density was 0.63 bic/m^2 and the mean waiting group density was 0.54 bic/m^2.

Huang and Wu (2004) researched bicycle waiting group density before the stop line, and found that the maximum waiting group density was 0.67 bic/m^2 and the mean waiting group density was 0.46 bic/m^2.

Considering bicycle group characteristics, a parameter "maximum parallel bicycles in each bicycle lane" N_w was adopted. The bicycle lane is divided into N_w strips. Bicycles can ride along these strips. The maximum riding and waiting numbers of paralleled bicycles are N_w. N_w is affected by some factors such as size of bicycles, width of bicycle lanes, behaviour of cyclists, and so on. In this model, only size of bicycles and width of bicycle lanes are taken into consideration.

$$N_w = \rho_b * L_b * W_l \tag{4.34}$$

where,

N_w is maximum number of bicycles that can ride or wait together in lateral direction, the unit is bic;

ρ_b is the density of bicycles that can ride or wait together in lateral direction, the unit is bic/m^2;

L_b is the general length of bicycle;

W_l is the width of bicycle lane;

Sun and Yang (2004) researched the theory of traffic design for signalised intersection of mixed traffic in China and found a relationship between the widths of bicycle lane and the density of paralleled bicycles according to the data analysis.

$$\rho_b = 0.886 - 0.069 * W_l \tag{4.35}$$

where,

ρ_b is the density of bicycles that can ride or wait together in lateral direction, the unit is bic/m^2;

W_l is the width of bicycle lane;

The general size of bicycles in China is 1.9m long, 0.6m wide and 2.25m high. The waving width when moving is 1.0m (Highway and Transportation Society of China, 1998).

For example, a bicycle lane's width is 3.5m. The general length of bicycle is 1.9m. According to equation 4.34 and 4.35, the N_w should be 4.2 bic. Then the maximum numbers of bicycles can ride or wait paralleled is 4. The movement of bicycles riding in the bicycle lane and waiting before the stop line is shown in Figure 4.12.

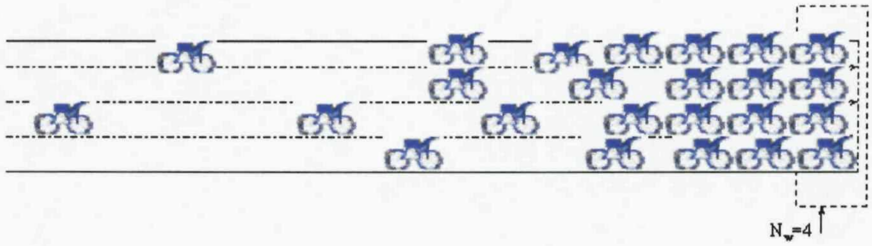


Figure 4.12 Bicycle waiting group density

4.2.5 Lateral movement model

4.2.5.1 Strip-changing model

Unlike motor vehicles, which move or wait in an obvious linear queue, bicycles appear as a "group" characteristic. The lateral movement of bicycles is not so disciplinary as motor vehicles. However, the bicycles still have some cut-in or overtaking characteristics in order to avoid the obstacle or gain higher speed. The lateral movement of bicycles is more flexible than motor vehicles.

Though there is no obvious link for bicycles as in motor vehicles, the single bicycle lane was divided into several strips in order to model bicycles easier. The number of strips in each bicycle lane depends on the N_w, i.e., the maximum number of bicycles that can ride together in lateral direction. The platoon of bicycles on each strip forms a link. The lateral movement of bicycles can be simplified to a change strip decision process.

The benefit is the basic and important factor, which will affect the strip-changing decision-making process, needs to be considered. The purpose of changing strip for bicycles is simpler than that for motor vehicles, i.e., to get speed benefit or avoid an obstacle. There are some characteristics that is different between bicycle strip-changing and motor vehicle lane-changing behaviour:

- Turn intention doesn't affect the behaviour of cyclists. Therefore, there is no difference between bicycle move to the offside and to the nearside.
- The leading bicycle won't consider the pressure of following bicycle when making decision.
- When a bicycle tries to overtake, the subject bicycle may not change to a lateral strip but cut in the front of leading bicycle due to the small size and flexibility.
- Three parameters need to be considered when evaluating the benefit of change strip, i.e., increased speed in subject strip, possible increased speed in nearside strip, and possible increased speed in offside strip. Therefore, the leading bicycle in subject strip and the leading bicycles in lateral strips need to be considered.

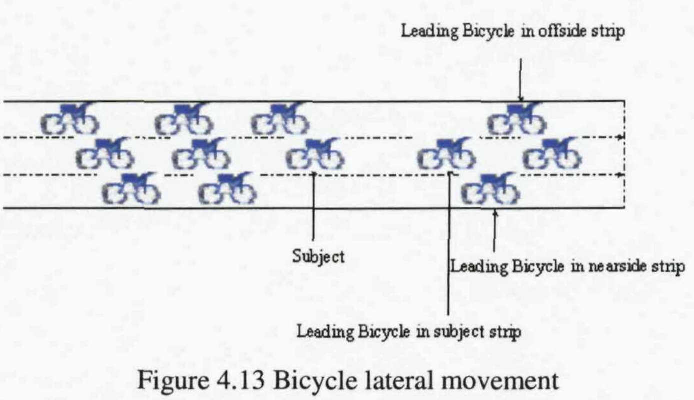


Figure 4.13 Bicycle lateral movement

4.2.5.2 Bicycle strip-changing flow chart

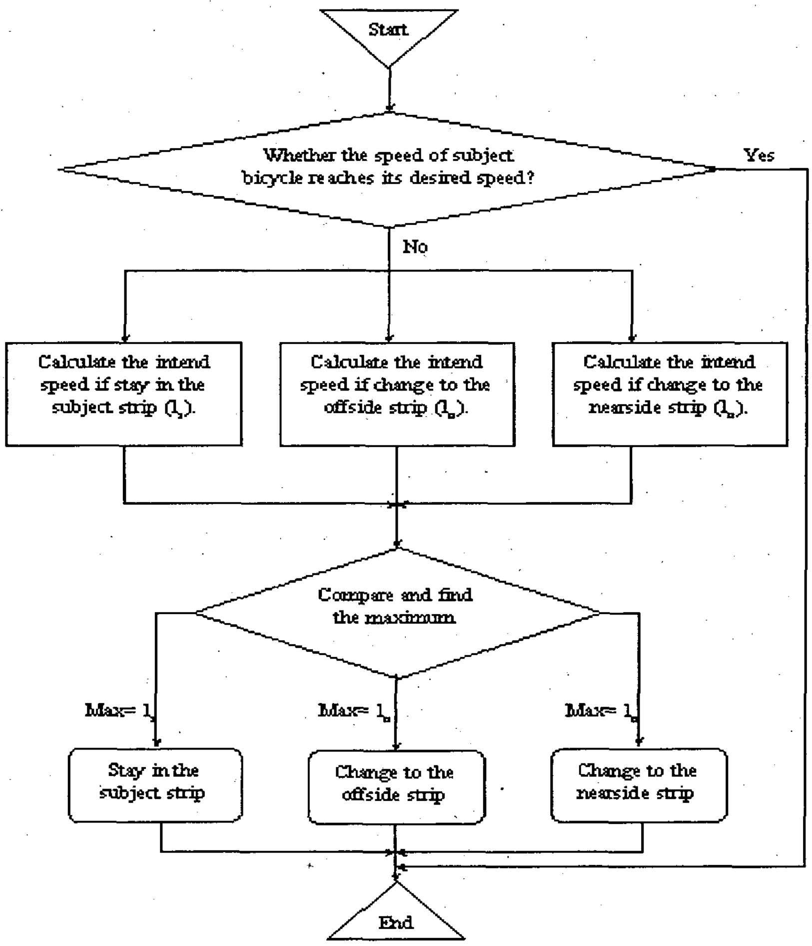


Figure 4.14 Bicycle strip-changing model flow chart

4.3 Gap/lag acceptance

Mixed traffic flows with motor vehicles and bicycles are very common in Chinese cities. The interactions between these flows at signalised intersections often result in conflicts and delays and become one of the main causes of traffic congestion in urban networks. Therefore, in order to represent the conflict phenomenon, gap/lag acceptance decision becomes an essential behavioural characteristic in microscopic simulation model development.

4.3.1 Gap/lag acceptance characteristics

Gap acceptance is an important behavioural parameter of drivers who wish to merge into other traffic streams for lane changing or crossing traffic streams at intersections. Gap acceptance describes the behaviour of individual traffic entities in accepting or rejecting gaps in the traffic stream.

A gap is the time or length between two successive vehicles, beginning with the end of the leading vehicle and ending with the front of the following vehicle. A lag is the time or length from the moment the crossing vehicle reaches the point of conflict until the closing lag vehicle arrives or the time between the moment the crossing or merging vehicle arrives at the intersection until the first gap vehicle arrives.

The acting vehicle is the vehicle attempting to cross or merge into other traffic streams at an intersection. The gap vehicles are the vehicles defining the gap. The first gap vehicle that opens the gap is called the opening gap vehicle, while the second gap vehicle that ends the gap is called the closing gap vehicle. Both the acting vehicle and the gap vehicles can be either a bicycle or a motor vehicle.

"The prevailing notion of the critical gap is that it is the minimum gap duration that will be accepted by an individual vehicle/operator unit in a specific situation (Miller, 1971)." A measure of the centre of this distribution is often chosen as the design critical gap (Taylor and Mahmassani, 1998).

Many references concern the drivers' gap acceptance behaviour. Some factors are found to influence driver's gap acceptance. Small motor vehicles are more likely to accept smaller gaps than heavy vehicles. Critical gap time may decrease when drivers are delayed for too long and become impatient. The accepted gap may also be influenced by the available gaps in the major stream. For example, a merging vehicle when presented with a number of gaps would choose the larger gap rather than the first sufficient one. Critical gap time may change by time of day (Taylor and Mahmassani, 1998; Gao et al., 2001; Huang and Wu, 2004).

4.3.2 Hypothesis on critical gap/lag acceptance

Time hypothesis is that each minor road driver has a fixed critical gap measured in time units irrespective of the speed of the approaching major road vehicle that ends the gap. Drivers will accept all gaps larger than a fixed length in time, and will reject all gaps that are less than the critical time.

Modified time hypothesis suggested that the critical gap of each minor road driver is not fixed, but is composed of at least two terms, a constant time term and a variable component which is dependent on the speed of the approaching major road vehicles.

The pure time model is provided by Gibbs (1968), who related the critical distance gap D, in meters, to the approach speed V, in meters per second, by

$$D = 5.4V \tag{4.36}$$

This corresponds to a constant time gap T of 5.4 seconds.

Cooper et al. (1976) proposed a modified time hypothesis model for merging situations:

$$D = 11.5 + 5V \tag{4.37}$$

where,

D is the median accepted gap/lag distance in meters;

V is the speed of approaching vehicle, unit is m/s;

The distance hypothesis concept is that each minor road driver has a fixed critical gap/lag measured in distance units to the approaching major road vehicle regardless the speed of the approaching vehicle.

4.3.3 Consistent and inconsistent behaviour

Assuming a pure time hypothesis, the formulation of the gap acceptance model may be approached in two ways: consistent and inconsistent behaviour. The consistent behaviour approach assumes that drivers behave consistently so that each driver has a consistent critical gap, rejecting all gaps smaller and accepting any gap larger than this critical value. However, the inconsistent behaviour approach assumes that a driver does not have a fixed critical gap, but that it varies with different geometry, traffic and time, with the distribution of such critical gaps being identical throughout the population (Weiss and Maraddadin, 1962).

Ashworth and Bottom (1977) concluded that the real traffic gap/lag acceptance behaviour lies between the two approaches and no one approach is clearly superior to the other. However, the inconsistent model was preferred because it gave relatively better results.

4.3.4 Gap/lag acceptance distribution function

The accepted gap/lag distribution function describes both the inconsistent and consistent behaviour was found to be essentially similar (Hossain, 1996). Some of the suggested gap acceptance distribution functions are described below:

Blumfeld and Weiss (1970) suggested the simplest consistent behaviour model, which assumed a single value critical gap throughout the population. This leads to an accepted gap function in the form of a step function.

$$U(t-t_c)=\begin{cases}1 & while(t>t_c)\\ 0 & elsewhere\end{cases} \tag{4.38}$$

where,

t_c is the critical gap;

A trapezoidal distribution, which is based on the uniform gap density function, was proposed by Drew (1967).

$$\alpha(t)=\begin{cases}0 & t\le t_{\min}\\ (t-t_{\min})/(t_{\max}-t_{\min}) & t_{\min}<t<t_{\max}\\ 1 & t\ge t_{\max}\end{cases} \tag{4.39}$$

where,

t_{min} and t_{max} are the minimum and maximum critical gap respectively;

Herman and Weiss (1961) proposed an exponential gap acceptance function:

$$\alpha(t)=\begin{cases}1-\exp[-\lambda(t-t_{\min})] & t>t_{\min}\\ 0 & elsewhere\end{cases} \tag{4.40}$$

Solberg and Oppenlander (1966) suggested a lognormal gap density function.

$$\alpha(t)=\frac{1}{s\sqrt{2\pi}}\int_0^t\frac{1}{Z}\exp[\frac{-(\ln Z-\ln\bar{t})^2}{2s^2}]dZ \quad t>0 \tag{4.41}$$

where,

$\bar{t}$ and s are the mean and standard deviation of lognormal critical gap distribution respectively.

4.3.5 Some existing research results on gap/lag acceptance

A vehicle making a left (or right) turn from the minor street is making a lateral left (or right) manoeuvre. Similarly, a lateral straight manoeuvre is a through movement from the minor street. A frontal left manoeuvre is a left turn from the major street.

Taylor and Mahmassani (1998) researched the design critical gap of motor vehicle:
Lateral right-turn: 5.0 sec;
Lateral straight manoeuvre: 5.5 sec;
Lateral left-turn: 6.0 sec;
Frontal left-turn: 4.5 sec;

Smith (1976) obtained the design critical gap after analysing the data collected for motor vehicles entering or crossing bicycle traffic at intersection of two-lane street, and found that the critical gap of lateral straight manoeuvre was 3.5 sec and the frontal left-turn was 2.6 sec.

Smith (1976) also estimated the design critical gap for bicycles crossing motor vehicle traffic flow on a two-lane street to be 4.4 seconds.

Opiela et al. (1980) recorded a total of 260 bicycle crossings of two lanes of one-way motor vehicle traffic (7.3 m wide), and determined the gap-acceptance characteristics of the cyclists. The average accepted gap was 3.9 sec, and the minimum was noted to be 1.1 sec. The critical gap was noted to be 3.2 sec. They determined that the distribution of accepted gaps corresponded to a lognormal function.

Taylor and Mahmassani (1998) estimated discrete choice (probit) models of both motorist and cyclist gap acceptance behaviour when crossing and merging with bicycle-automobile mixed traffic at three low-speed stop-controlled intersections near a university campus. They not only estimated the mean value of the overall gap distributions but also the magnitude and direction of the effects of various factors on the mean critical gap.

Hossain and McDonald (1998B) analysed 106 examples of gap/lag acceptance behaviour in signalised junctions with mixed traffic and found that the gap/lag acceptance of both non-motorised and motorised vehicles fits the log normal distribution. He suggested the values of two parameters for the log normal distribution: μ=2.475, σ=0.26 for non-motorised vehicles and μ=3.584, σ=0.219 for motorised vehicles respectively.

Huang and Wu (2004) collected 104 gap acceptance decision observations and 217 lag acceptance decision observations. In this research, 4.52 sec average accepted gap length and 2.93 sec average accepted lag length was obtained respectively. The minimum accepted gap length was noted to be 1.56 sec and the minimum accepted lag length was 0.52 sec. "The distribution of the accepted gaps/lags of bicycles was analysed and the natural log normal distribution was found to provide a close approximation to the data collected." Huang suggested the values of two parameters for the log normal distribution: μ=4.19, σ=0.39 in gap acceptance and μ=2.47, σ=0.51 in lag acceptance respectively.

The accepted gap of cyclists is related to whether the crossing was initiated from a rolling or standing start. "Many cyclists accepted much shorter gaps by not stopping prior to initiating the crossing manoeuvre. When traffic conditions required a stop, the accepted gap tended to be longer in duration (Opiela et al., 1980)." In other words, the accepted gap is shorter when rolling than stopped-start (standing start). Huang and Wu (2004) researched the effects of stopping start before crossing and concluded that the length of accepted gap of the acting vehicle that crossing from stopping is 0.98 sec larger than from rolling, while the length of accepted lag is 0.99 sec larger.

4.3.6 Proposed gap/lag model

Although modified time hypothesis suggests the effect of the approach on gap acceptance, there is little agreement of researchers as to the correct model to relate approach speed to the critical accepted gap size (Hossain, 1996). Therefore, considering the better performance of the inconsistent behaviour model, a time hypothesis along with inconsistent behaviour was selected for developing the gap acceptance model of the present simulation model.

The gap/lag acceptance decision behaviour can be divided into four groups according to the various types of the acting vehicle and the gap vehicles.

1. The acting vehicle is motor vehicle, and the gap vehicles are motor vehicles;
2. The acting vehicle is motor vehicle, and the gap vehicles are bicycles;
3. The acting vehicle is bicycle, and the gap vehicles are motor vehicles;
4. The acting vehicle is bicycle, and the gap vehicles are bicycles;

A common conclusion is that the bicycle gap/lag acceptance is basically log normal distributed. According to log normal distribution, the probability distribution function is:

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma} e^{-\frac{(\ln(x)-\mu)^2}{2\sigma^2}}, (\sigma > 0) \tag{4.42}$$

where,

μ is expectation, that is the ln(mean gap/lap acceptance);

σ is standard deviation;

Because of the lack of lane discipline inside the intersection with mixed traffic flows, gaps could not be effectively defined by the drivers. It was rather easier for drivers to assess the lags (the time gaps between the driver concerned and the nearest conflicting stream vehicle).

The gap/lag acceptance behaviour is characterised by two separate decisions: 1) whether the potential conflict space which the vehicle intends to occupy is clear, 2) whether the vehicle would accept the evaluated lag to enter the potential conflict space. Whenever approaching the potential conflict point, the acting vehicle needs to check whether the potential conflict space is occupied. If the conflict space is determined to be clear, then the acting vehicle will evaluate the lag available to the nearest vehicle in the conflicting stream while occupying the potential conflict space.

According to previous research on bicycle mean gap/lap acceptance, log-normal distributed gap/lag acceptance model performed better than a single critical gap/lag value. Huang's (Huang and Wu, 2004) research was based on the data collected in signalised intersection in Beijing, so, the analysis results can reflect the typical situation of gap/lag acceptance in signalised intersection with mixed traffic in Beijing. Therefore, in this study case, the values proposed in Huang's results were adopted. If the acting vehicle is bicycle, then the mean gap/lap acceptance was set to be 2.47 sec, and the standard deviation σ was set to be 0.51. If the acting vehicle is motor vehicle, then the mean gap/lap acceptance was set to be 3.58 sec, and the standard deviation σ was set to be 0.22.

4.3.7 Flow chart of gap/lag acceptance

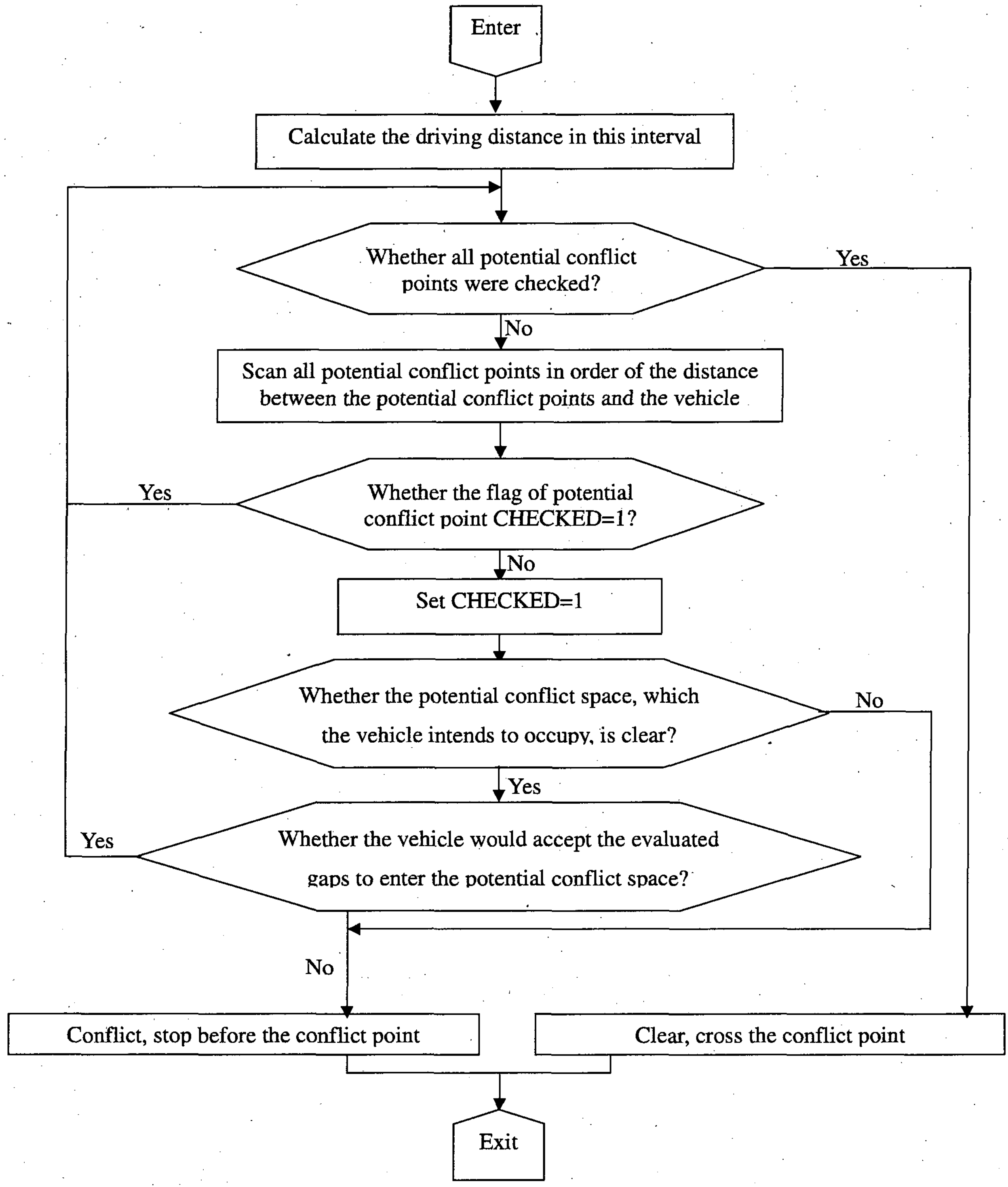


Figure 4.15 Flow chart of gap acceptance

Because of the limited physical space at most intersections, the potential conflict points may be very close to each other. Motor vehicles often occupy more than one potential conflict point, (i.e. potential conflict space of motor vehicle can cover more than one conflict lane), due to the size and speed of motor vehicles. Therefore, motor vehicles need to check all the potential conflict lanes. However, because of the small size and high flexibility of bicycles, bicycles assess gaps lane-by-lane and can stop between lanes.

4.4 Signals

4.4.1 Definitions of signal control

Phase: A phase is a set of movements that can take place simultaneously or the sequence of signal indications received by such a set of movements (Institution of Highways and Transportation, 1997; The Ministry of Public Security of the People's Republic of China, 2002).

Cycle: In each signal controlled there is a normal sequence in which the various phases receive green and one repetition of this sequence is called cycle (Institution of Highways and Transportation, 1997; The Ministry of Public Security of the People's Republic of China, 2002).

Stage: A stage is that part of the cycle during which a particular set of phases receives green (Institution of Highways and Transportation, 1997).

Intergreen period: The time period between the end of green for one phase and the start of green for another phase gaining right of way at the same change of stage is knows as the intergreen period between these phases (Institution of Highways and Transportation, 1997).

The usual sequence of signal aspects or indications in China is red, green, and amber. The amber period is standardized at 3 s (Salter and Hounsell, 1996).

The length of intergreen period must be based on the time required for a vehicle, which passed over the stop line at the start of amber period to clear a potential collision point with a vehicle starting at the onset of green of the following stage and travelling at the normal speed for the intersection (Salter and Hounsell, 1996).

The total lost time = Lost time during the intergreen period + Start and end lost time during the green period

Lost time during the intergreen period = Intergreen period – Amber time

The start and end lost time during the green period = Actual green time + Amber time – effective green time. In practice the lost time due to starting and end lost times is often taken as 2 s in UK (Salter and Hounsell, 1996), which is the most frequently occurring value.

Minimum green period: For each phase there will be a minimum green period which is the minimum amount of time required by any vehicles waiting between the nearest detectors and the stop-line to pass through the junction (Institution of Highways and Transportation, 1997; The Ministry of Public Security of the People's Republic of China, 2002).

Vehicle Extension Period: a vehicle detected on an approach already showing a green signal may cause the green to be extended by a preset vehicle extension period and this can be repeated until the maximum green period has been used up (Institution of Highways and Transportation, 1997; The Ministry of Public Security of the People's Republic of China, 2002).

Maximum green period: For each phase showing a red signal the first vehicle to cross a detector on an approach in that phase will register a demand for a following stage in which that phase has a green signal. This demand will also initiate a preset maximum green period for each phase already displaying a green and for which such a period has not already begun (Institution of Highways and Transportation, 1997; The Ministry of Public Security of the People's Republic of China, 2002).

4.4.2 Modelling of signal control

There are two primary types of signal control. One is fixed-time signal control, and another one is vehicle actuated signal control.

Fixed-time signal operates under a predetermined time schedule. Each signal phase or traffic movement is serviced in a programmed sequence. It is the oldest signal control system, which is still widely used all over the world. It suits intersections with relatively stable traffic flow over the time periods set for the fixed operations.

Traffic signals working under vehicle actuation give green signals for time periods which relate to on-line traffic demand. Traffic demand is measured by vehicle detectors, which are installed on the approaches to the signals. More sophisticated network control systems are available, but as isolated fixed-time and vehicle-actuated signal control are most common, only these will be studied.

For fixed-time signals, the amber period, the green period and the red period follows a pre-determined time schedule.

For vehicle-actuated signals, the green periods are related to the traffic demands, using the detectors that are installed on approaches. The minimum green period, maximum green period and the extension period should be set. The minimum green is not fixed but varies between 7 sec and 13 sec according to the traffic flow. The extension period may be fixed or related to the approach speed measured by the detector. The maximum green was adopted, in order to prevent vehicles waiting on a halted phase indefinitely due to a continuous stream of traffic on the running phase.

4.5 Overview of model development

A microscopic simulation model was written in Visual C++ as described in Chapter Three, which incorporated the following key behavioural elements identified in this chapter. These were:

- Motor vehicle generation model;
- Motor vehicle car-following model;
- Motor vehicle lane-changing model;
- Bicycle generation model;
- Bicycle acceleration/deceleration model;
- Bicycle lateral movement model;
- Gap acceptance model;
- Signal control model;

As mentioned in Chapter Three (Section 3.4), the simulation model developed in this research has a certain universality and transferability. Though the model needs to be further improved in some elements, such as pedestrians' behaviour and lateral disturbance between motor vehicles and bicycles, many common, geometric and operational conditions throughout the transportation system can still be simulated with this simulation model. For example, the model is applicable to both urban roads and motorways, to isolated junctions and road networks. Both fixed-time signal control and vehicle-actuated signal control have been achieved in the simulation model. A simple behaviour model for buses is feasible. Detectors for traffic data and simulation of accident are also available. Some functions and applications may not be displayed in this research due to the limitation of simulation study purpose. For example, only fixed-time signal control was adopted in this research because of the purpose of case studies. Of course, calibration and validation is necessary when the simulation model is applied in different conditions, because no single model can be expected to be equally accurate for all possible local traffic conditions and behaviours. Therefore every model must be adapted for local conditions.

5. Model verification and validation

5.1 Introduction

A simulation model should be reliable, error-free and credible. Although a complete replication of actual microscopic behaviour cannot be expected, a reasonable compliance of the simulation results with the real life situation is required if the model is to be used for experimental studies with confidence. The process of ensuring validity, credibility, and reliability typically consists of three elements: verification, calibration and validation (Rakha et al., 1996). To be credible, a model has to be carefully verified and validated before application.

5.1.1 Verification and validation processes

The literature review below gives an insight into what researchers recommend as procedures that can be followed in calibrating and validating traffic simulation models.

Benekohal (1991) has provided a first step in creating a framework for the verification and validation procedure.

Rakha et al. (1996) attempted to provide traffic model developers and users with a framework for the verification, validation and calibration of traffic simulation models. Examples are provided to illustrate the model verification and validation processes. In their study, verification was defined to be the process of determining if the logic that describes the underlying mechanics of the model, as specified by the model designer, is faithfully captured by the computer code.

Milam and Choa (2002) presented an initial set of recommended guidelines for the calibration and validation of traffic simulation models. Calibration was defined in this study as the process by which the individual components of simulation model are adjusted or tuned so that the model will accurately represent field measured traffic conditions.

Merritt (2004) proposed a methodology for the calibration and validation of the stochastic microscopic traffic simulation model CORSIM, using data connected from a section of the arterial road in the city of Uppsala, Sweden. In this study, data from two traffic scenarios were collected during the AM peak and midday peak. The midday peaks were used for calibration while the AM peaks were used for validation.

Cohen (2004) introduced an approach for calibration and validation of traffic simulation models. According to Cohen, understanding the meaning of calibration and validation and what parameters to use in either case is a necessary step for understanding how to calibrate and validate traffic simulation models for a particular case study. Cohen defined calibration as the adjustment of parameters in a model so as to represent local conditions, and validation as a comparison of measures of effectiveness as computed by a model and as observed in field data under the same traffic conditions. Cohen stressed on the fact that validation can only be attempted after the model has been effectively calibrated.

Dowling et al. (2004) proposed a practical, top-down approach for calibration of microscopic simulation models. According to their research, calibration is necessary because no single model can be expected to be equally accurate for all possible local traffic conditions and behaviours, and therefore every model must be adapted to local conditions. A three-step calibration/validation process was recommended in this study: capacity calibration, route choice calibration and system performance calibration. They suggested that to satisfy these steps, the following data needed to be collected: traffic counts and measures of systems performance such as travel times, speeds, delays and queues. Also for validation, parameters used as measures of system performance must be collected simultaneously with the traffic counts.

As can be seen from the literature review, the verification and validation process of a model should include three stages. The first stage is the verification process to ensure that the internal logic of the program is correct, i.e. that a simulation computer program performs as intended, that the conceptual simulation model was correctly translated into a working program. The second stage is a calibration process, which is concerned with the testing and refinement of the model. The third stage, which is known as validation, was the process of checking whether the model correctly represents the system modelled.

The flow chart below shows the outline of the verification and validation process used in this study with a list of the parameters used in each stage of the process.

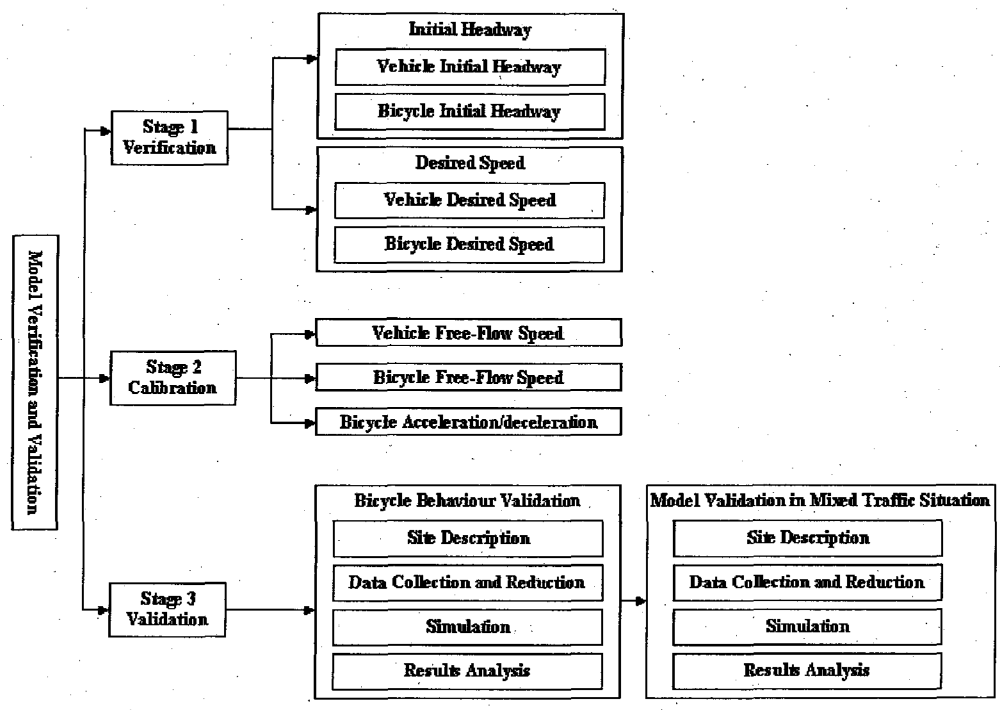


Figure 5.1 Outline of the verification and validation process

In order to estimate typical values of the measured traffic parameters and to investigate their variability, detailed statistical analysis of the data was undertaken. The Kolmogorov-Smirnov test is commonly used to determine whether one data set is compatible with another random sample from a given distribution. Therefore, the Kolmogorov-Smirnov test of goodness-of-fit was used throughout this study to test the goodness of fit of specific distribution to the observed data.

5.1.2 Data requirement

The components or parameters of a simulation model requiring calibration include traffic control operations, traffic flow characteristics, and drivers' behaviour (Milam and Choa, 2002). In general, two different types of data are required to make the application of the traffic simulation model successful. They are data for model input, and data for model validation. These two data types are explained below.

5.1.2.1 Data for Model Input

These input data could be categorized into three major groups as described in the following sections.

A. Transportation network data

The network data include the geometry of the road network such as link length and width, number of lanes per link, lane channelization at intersections, grade and so on. Lane use, lane restrictions, and lane blockage should be taken into consideration as well.

B. Traffic control data

These include signal control type, signal timing data, and detector data such as type and the location of detectors. Priority rules also need to be included.

C. Traffic demand data

These include traffic flow data, and desired speed limits.

Most of these data can obtained from detailed onsite observations and video records.

5.1.2.2 Data for Model Validation

Vehicle characteristics, driver characteristics, and measures of performances are the key parameters for model validation. These are used to evaluate whether the model can realistically represent the traffic environment being analyzed and to compare model results with corresponding data observed in the field. Statistical tests were applied to check the agreement of the field values of validation parameters with those obtained from simulation runs.

1. Driver behaviour data

These can be taken into consideration by using statistical distributions of behaviour related parameters such as desired free flow speed, queue discharge headway, acceptable gaps for lane changing and car following, maximum acceleration, maximum deceleration etc. The best parameter ranges were identified according to the on-site measurements where necessary.

2. Vehicle data

These represent the characteristics and performance of the vehicles in the network. Such as vehicle type, vehicle length and vehicle width.

3. Measures of performance data

These measures usually used in validation include travel time, delay, queue length etc. In this study, travel time was adopted as the parameter to validate the model for signalised intersections. Travel time for the journey through the signalised junction gives a good reflection of the queuing along the approach and subsequent discharge process from that approach. It also indicates the performance of the car-following and free-flow behaviour of the vehicles approaching the intersection.

5.1.3 Proposed procedure of calibration and validation

In the CORSIM user's manual (FHWA, 1997), an informal validation process was demonstrated with the CORSIM simulation model and importance of data quality and visualization was emphasized.

Hellinga (1998) described a calibration process including seven steps: (a) defining study goals and objectives, (b) determining required field data, (c) choosing measures of performance, (d) establishing evaluation criteria, (e) network representation, (f) driver routing behaviour, and (g) evaluation of model outputs.

Chu et al. (2003) presented a systematic, multi-stage procedure for the calibration and validation of PARAMICS simulation models. The procedure was demonstrated in a calibration study with a corridor network in the southern California. In order to analyze network-wide transportation problems, not only driving behaviour model calibration but also dynamic Origin and Destination (OD) demand estimation and route choice calibration were considered in the process.

A procedure was proposed by Park and Schneeberger (2003) for microscopic simulation model calibration and validation and an example case study is presented with real-world traffic data from Route 50 on Lee Jackson Highway in Fairfax, Virginia and VISSIM simulation model. The proposed procedure consisted of nine steps: (a) measure of effectiveness selection, (b) data collection, (c) calibration parameter identification, (d) experimental design, (e) run simulation, (f) surface function development, (g) candidate parameter set generations, (h) evaluation, and (i) validation through new data collection.

Dowling et al. (2004) proposed a three-step calibration/validation process for simulation model. They were: capacity calibration, route choice calibration and system performance calibration. This three-step process was illustrated in an example application for a freeway/arterial corridor.

According to the literature review, the proposed procedure of calibration and validation is a more detailed step-by-step approach. The following steps are adopted for calibration and validation:

1. Determination of measures of performance

The first step in the calibration and validation process is to determine measures of performance appropriate for calibration and validation. Performance measures and uncontrollable input parameters and controllable input parameters have to be determined. The performance measure could be an average travel time between two data collection points in the network.

2. Data collection

Data collection was required to provide simulation program input parameters and output measures of performance for calibration and validation of the microscopic simulation model. Uncontrollable input parameters and measures of performance should be collected from the field. Uncontrollable input parameters may include existing geometry, traffic counts, current signal timing plans, and so on (Park and Schneeberger, 2003).

3. Identification of calibration parameters

All calibration controllable parameters within the microscopic simulation model must be determined. Controllable input parameters in the simulation program may include free-flow speed, minimum headways, minimum and maximum acceleration/deceleration, and so on.

4. Evaluation of candidate parameter sets with simulation runs

In this step, multiple runs are conducted to verify whether the parameter sets identified in the previous step generate statistically significant results.

5. Validation with new data

To perform validation of the microscopic simulation model, a new set of field data should be collected. One way of collecting validation data would be to collect data for different time periods or conditions (Merritt, 2004).

5.2 Model verification

A simulation model is composed of several individual procedures and functions. To make sure the model is logically correct and can represent the real system accurately, all the procedures and functions have to be tested to examine whether they are correct in logic and perform properly as expected. For example, if a procedure or a function was intended to generate random values, which should follow certain distribution, a series of random outputs could be produced and represented in a histogram to check whether the outputs fit the expected distribution.

Two verification procedures have been carried out on the initial headway generation function and the desired speed generation function.

5.2.1 Initial headway

The initial headway generation procedure will affect the accuracy of the number and the distribution of generated motor vehicles and bicycles. The verifications of initial headway of motor vehicles and bicycles were conducted separately.

5.2.1.1 Initial vehicle headway

Motor vehicle's initial headways should follow the shifted negative exponential distribution (see chapter 4). According to previous research (see 4.1.2.1.2), the shift was set to be 0.5 sec. If the initial headway generation processor is correct, the ratio of N_{sim} (simulated motor vehicle arrival) to N_{exp} (expected motor vehicle arrival) should be about 1.0 and independent of the traffic demand. In order to examine the number of generated motor vehicles, 26 different traffic demands were selected randomly within the range from 100 veh/h to 6000 veh/h, which can generally represent the requirement of traffic demand on urban network. 26 simulations were conducted for 26 different traffic demands.

The data of expected motor vehicle arrival (N_{exp}), the simulated motor vehicle arrival (N_{sim}) and the ratios of N_{sim} to N_{exp} were given in Table 5.1.

Table 5.1 The ratios of N_{sim} to N_{exp}

Expected Arrival (N_{exp}) (veh/h)	Simulated Arrival (N_{sim}) (veh/h)	N_{sim} / N_{exp}
100	97	0.97
200	201	1.005
400	388	0.97
600	587	0.978333
800	803	1.00375
1000	1010	1.01
1250	1250	1
1500	1511	1.007333
1750	1759	1.005143
2000	2003	1.0015
2250	2246	0.998222
2500	2489	0.9956
2750	2735	0.994545
3000	2990	0.996667
3250	3244	0.998154
3500	3500	1
3750	3760	1.002667
4000	4009	1.00225
4250	4264	1.003294
4500	4519	1.004222
4750	4771	1.004421
5000	5039	1.0078
5250	5303	1.010095
5500	5559	1.010727
5750	5816	1.011478
6000	6071	1.011833

In the last column of Table 5.1, it was found that all the ratios of N_{sim} to N_{exp} fell in the range of 0.97 to 1.012. The maximum difference was only 3.0%.

Statistical analysis results of the rates of N_{sim} to N_{exp} are shown in Table 5.2. The statistics analysis indicates that the rates of N_{sim} to N_{exp} have a mean of 1.000117 and a standard deviation of 0.011256. The F-test shows these two samples' variances are equal, therefore the two-tailed t-test (Two-Sample Assuming Equal Variances t-test) was conducted to evaluate whether the values were statistically equivalent at 5% level of significance. The t-statistic value of 0.023 is less than the critical t-statistic value for 5% significance level (2.008). The result shows there is no significant difference between N_{sim} to N_{exp}. F-test and T-test result are presented in Appendix C.

Table 5.2 Statistics analysis of the rates of N_{sim} to N_{exp}

Item	Value
Size	26
Mean	1.000117
Standard Deviation	0.011256
Minimum	0.97
Maximum	1.012

In order to verify the distribution of motor vehicle initial headways, generated vehicle initial headways were recorded during the 26 simulations. The Kolmogorov-Smirnov test was used to check the assumed model. The test result confirmed that the bicycle initial headway was exponentially distributed. The Kolmogorov-Smirnov test results are presented in Appendix C. Data analysis shows the initial headway for each simulation follow the shifted negative exponential distribution. An example on the distribution of motor vehicle initial headway when the expected arrival was 3000 vehicle/h is shown in Figure 5.2:

From the trend line of the distribution, it's clear to see that the motor vehicle initial headway generation procedure provides a close fit to the shifted negative exponential distribution.

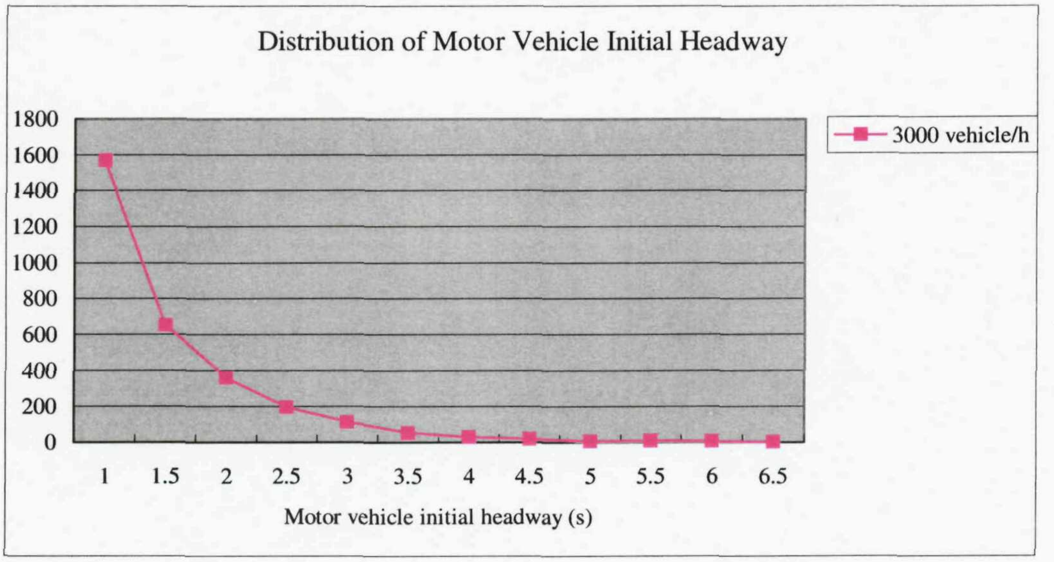


Figure 5.2 Distribution of motor vehicle initial headway

5.2.1.2 Initial bicycle headway

The initial bicycle headways should follow the negative exponential distribution (see Chapter 4). In order to examine the number of bicycles generated, 30 different bicycle traffic demand levels were selected randomly within the range from 100 bic/h to 3000 bic/h, which can generally represent the requirement of bicycle traffic demand on urban network. 30 simulations were conducted for the 30 different bicycle traffic demands.

The data of expected bicycle arrival (N_{exp}), the simulated bicycle arrival (N_{sim}) and the ratios of N_{sim} to N_{exp} is shown in Table 5.3.

Table 5.3 The ratios of N_{sim} to N_{exp}

Expected Arrival (N_{exp})	Simulated Arrival (N_{sim})	N_{sim} / N_{exp}
100	99	0.99
200	201	1.005
300	291	0.97
400	398	0.995
500	498	0.996
600	588	0.98
700	697	0.995714
800	806	1.0075
900	920	1.022222
1000	1017	1.017
1100	1109	1.008182
1200	1214	1.011667
1300	1307	1.005385
1400	1420	1.014286
1500	1521	1.014
1600	1624	1.015
1700	1718	1.010588
1800	1819	1.010556
1900	1891	0.995263
2000	2007	1.0035
2100	2102	1.000952
2200	2203	1.001364
2300	2307	1.003043
2400	2393	0.997083
2500	2486	0.9944
2600	2588	0.995385
2700	2682	0.993333
2800	2786	0.995
2900	2873	0.99069
3000	2970	0.99

In the last column of the table, it was found that all the ratios of N_{sim} to N_{exp} fell in the range of 0.97 to 1.0222. The maximum difference is only 3%. In Table 5.4, the statistics analysis indicates that the rates of N_{sim} to N_{exp} have a mean of 1.000937 and a standard deviation of 0.011359. The F-test shows these two samples' variances are equal. Therefore, the two-tailed t-test (Two-Sample Assuming Equal Variances t-test) was conducted to evaluate whether the values were statistically equivalent at 5% level of significance. The t-statistic value of 0.00514 is less than the critical t-statistic value for 5% significance level (2.0017). The result shows there is no significant difference between N_{sim} to N_{exp}. The F-test and T-test result are presented in Appendix C.

Table 5.4 Statistics analysis of the rates of N_{sim} to N_{exp}

Item	Value
Size	30
Mean	1.000937
Standard Deviation	0.011359
Minimum	0.97
Maximum	1.0222

In order to verify the distribution of initial bicycle headways, initial headways were recorded during the 30 simulations. The Kolmogorov-Smirnov test was used to check the assumed model. The test result confirmed that the bicycle initial headway was exponential distributed. Kolmogorov-Smirnov test are presented in Appendix C. Data analysis shows the initial headway for each simulation follow the negative exponential distribution. An example on the distribution of bicycle initial headway under the expected arrival was 2000 bicycle/h is shown in Figure 5.3.

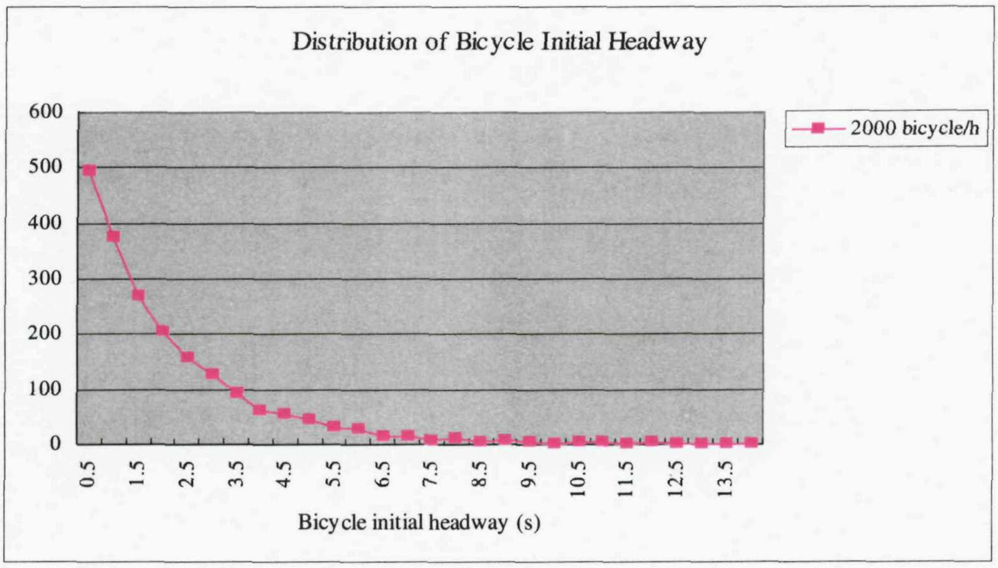


Figure 5.3 Distribution of bicycle initial headway

The analyses above show that the vehicle and bicycle generation processor is reliable.

5.2.2 Desired speed

Desired speed is an essential parameter that will affect the behaviour of motor vehicles and bicycles. The verifications of desired headway of motor vehicles and bicycles were conducted separately.

5.2.2.1 Desired speed of motor vehicle

The motor vehicle desired speed should follow the normal distribution. In order to verify the distribution of motor vehicle's desired speed, 5 simulations were run under five different sets of mean and standard deviation. Most of the speed limit of motor vehicle lanes in urban area is less than 70 km/h, therefore, six values between 30 km/h and 70 km/h were selected as the supposed mean desired speed.

The parameters, which will affect the motor vehicle's desired speed, are listed in Table 5.5.

Table 5.5 The parameters affecting the distribution of desired speed

Expected Traffic Flow (vehicle/h)	3000	3000	3000	3000	3000
Mean (km/h)	30	40	50	60	70
Standard Deviation	1.2	1.2	1.2	1.2	1.2

From the trend line of the distribution, it's clear to see that the motor vehicle desired speeds generation procedure provides a close fit to the normal distribution under different settings. The Kolmogorov-Smirnov test was used to check the assumed model. The test result confirmed that the vehicle desired speed was normally distributed. Kolmogorov-Smirnov test are presented in Appendix C. The output is demonstrated in Figure 5.4.

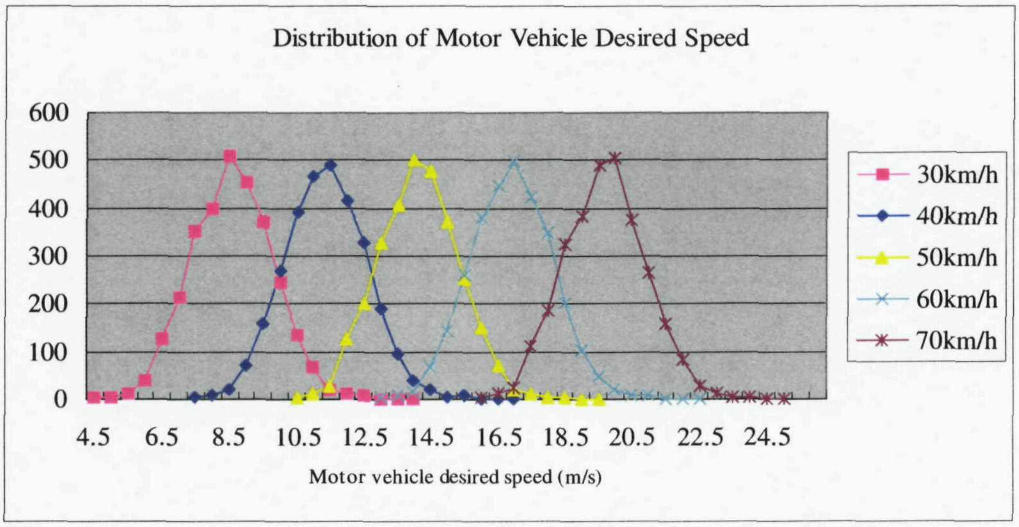


Figure 5.4 Distribution of motor vehicle desired speed

5.2.2.2 Desired speed of bicycle

The bicycle desired speed should follow the normal distribution. In order to verify the distribution of bicycle's desired speed, 6 simulations were run against six different sets of mean and standard deviations. According to the research result (see 4.2.2.2.1), free-flow bicycle speed appears to be somewhere between 10 km/h and 28 km/h with a majority of the observations being between 12 km/h and 20 km/h. Therefore, six values between 10 km/h and 32 km/h were selected as the supposed mean desired speed.

The parameters, which will affect the bicycle's desired speed, are listed in Table 5.6.

Table 5.6 The parameters affecting the distribution of desired speed

Expected Traffic Flow (bicycle/h)	3000	3000	3000	3000	3000	3000
Mean (km/h)	12	16	20	24	28	32
Standard Deviation	0.8	0.8	0.8	0.8	0.8	0.8

The trend line of the distribution illustrates that the bicycle desired speeds generation procedure is in agreement with the expected distribution under different settings. The Kolmogorov-Smirnov test was used to check the assumed model. The test result confirmed that the bicycle desired speed was normally distributed. Kolmogorov-Smirnov test are presented in Appendix C. The output is illustrated in Figure 5.5.

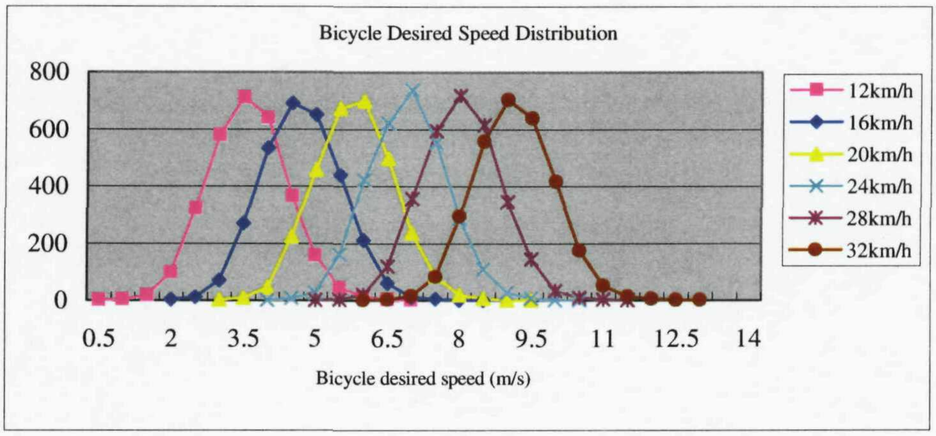


Figure 5.5 Distribution of bicycle desired speed

5.3 Model calibration and validation

In order to make sure the model can represent the real cyclists behaviours, a calibration and validation procedure was conducted based on data collected in Beijing, China as part of the research.

5.3.1 Measure of performance selection

Journey time was selected as the measure of performance for the calibration and validation process. The performance measure was chosen because of its ease of collection from the field and from the simulation output files.

5.3.2 Data collection

Data collection was required to provide simulation program input and output parameters for calibration and validation of the microscopic simulation model. These parameters should include uncontrollable input parameters, controllable parameters and measures of performance. These data were collected directly from the field. The data collection process are introduced with each step of calibration and validation below.

5.3.3 Identification of calibration parameters

Generally, a microscopic traffic simulation model includes some components, such as the roadway network, traffic control systems, and vehicle-bicycle-people units, etc., and associated behavioural models, such as driving behaviour models. These components and models have complex data requirements and numerous parameters. Some of these parameters, such as the roadway network parameter, the traffic data, and the traffic control data, are not affected by the model. Such parameters don't need to be calibrated. However, some parameters related to the behaviour models need to be calibrated for the specific study network and the intended applications. The parameters that influence the dynamics of single vehicles and bicycles in this microscopic simulation model include motor vehicle free-flow speed, bicycle free-flow speed, maximum acceleration, and maximum deceleration.

The parameters of mixed traffic listed in Table 5.7 were estimated using both field and video film observation.

Table 5.7 List of observed data from mixed traffic road site

The parameters for calibration
Motor vehicle free-flow speed
Bicycle free-flow speed
Bicycle maximum acceleration/deceleration

5.3.3.1 Motor vehicle free-flow speed

Low traffic flow situations were chosen to estimate motor vehicle free-flow speed. Data were collected from a typical roadway in Beijing, China. In order to get the average free-flow speed of motor vehicles, the license number plate recognition method was carried out at two timing points along the selected route. Two synchronized cameras were set on the overpass to collect the free-flow traffic flow. The number plate of each motor vehicle and the time passing the special points that were marked in advance was recorded. These two points were set far away from the stop line. The speed of each motor vehicle was determined by noting the time required to traverse a known distance, which is defined by two fixed points. The time required to travel through the known distance was obtained by recording the time difference when the same motor vehicle passed the two fixed points. The speed of each vehicle was calculated by dividing the reference distance by the time difference for each motor vehicle.

The selected road was a three-lane road with a speed limit of 40 km/h. A 358 free-flow speeds of motor vehicles were collected. The maximum speed was 52.2 km/h, the minimum was 18.98 km/h. The average free-flow speed was 39.14 km/h. The results are presented in Table 5.8.

Table 5.8 Analysis results of motor vehicle free-flow speed

Vehicle class	Sample size	Mean (m/s)	Fitted Distribution			Min (m/s)	Max (m/s)
			Name	Mean	Std. Dev.		
Motor vehicle	358	10.87	Normal	10.87	1.2	5.27	14.5

The Kolmogorov-Smirnov test was used to check the model. The test result confirmed that the motor vehicle free-flow speed was normally distributed. Kolmogorov-Smirnov test result is presented in Appendix C. The normal distribution of vehicle free-flow speed is shown in Figure 5.6.

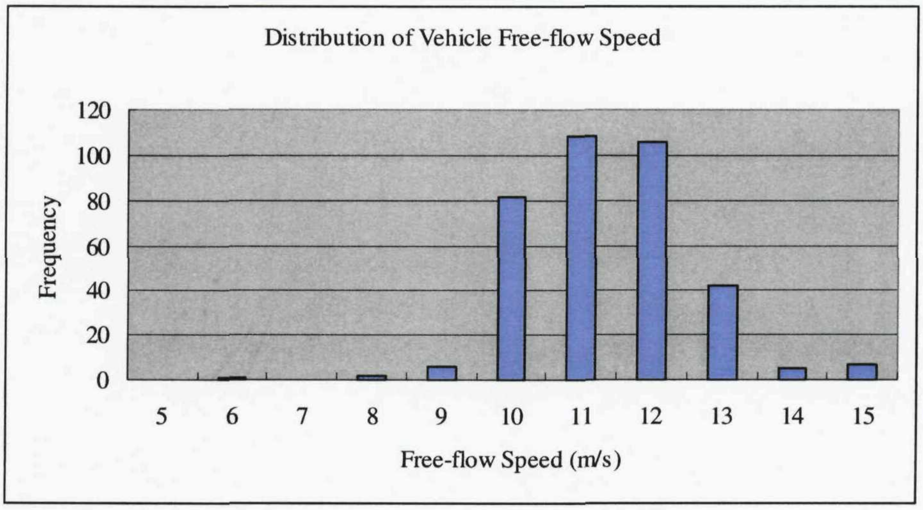


Figure 5.6 Distribution of vehicle free-flow speed

5.3.3.2 Bicycle free-flow speed

Low flow bicycle situations were chosen to estimate bicycle free-flow speed. The survey was undertaken at a bicycle lane inside the campus of Beijing Jiaotong University. This lane connects to the junction that was chosen for validation, and the behaviour of bicycles should be similar. This site was chosen for data collection because it is easier to set cameras inside the campus.

The data used for calibration was collected on 11th May 2006. Cameras were used to record the visual images which were then reduced into useful data by later manual analysis in the laboratory.

The bicycle free-flow speed was obtained by noting the time taken to ride over a known distance defined by two fixed timing points. The speed was calculated by dividing the distance by the time taken.

550 records of bicycles riding between point A and B were collected. The maximum speed was 7.24 m/s, and the minimum speed was 1.6 m/s. The average speed was 4.26 m/s. The standard deviation was 0.8. The results are presented in Table 5.9.

Table 5.9 Analysis results of bicycle free-flow speed

Vehicle class	Sample size	Mean (m/s)	Fitted Distribution			Min (m/s)	Max (m/s)
			Name	Mean	Std. Dev.		
Bicycle	550	4.26	Normal	4.26	0.8	1.6	7.24

The Kolmogorov-Smirnov test was used to check the assumed model. The test result confirmed that the bicycle free-flow speed was normally distributed. Kolmogorov-Smirnov test result is presented in Appendix C. The normal characteristics if the distribution of bicycle free-flow speed is shown in Figure 5.7.

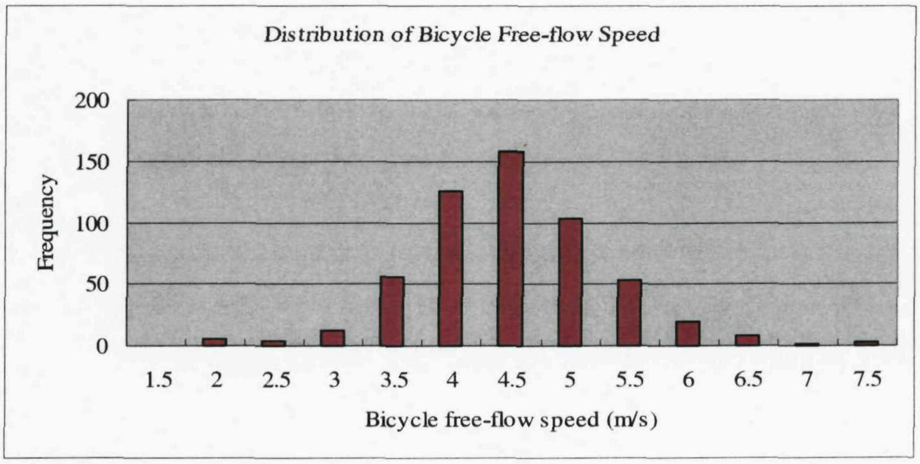


Figure 5.7 Distribution of bicycle free-flow speed

5.3.3.3 Bicycle maximum acceleration/deceleration

Deceleration and acceleration data were collected and analysed for bicycles by dividing the time difference into several time slices, noting the time t_i and location L_i of the braking/speeding process between two fixed points (as shown in Figure 5.8).

Figure 5.8 Illustration of time slice

The deceleration/acceleration can be calculated as:

$$V_2 = \frac{L_2 - L_1}{t_2 - t_1}$$
$$V_3 = \frac{L_3 - L_2}{t_3 - t_2}$$
$$a = \frac{V_3 - V_2}{t_3 - t_2} \tag{5.1}$$

31 sets of bicycle data were analyzed in SET_A. 44 acceleration and 75 deceleration values were obtained. The maximum acceleration was 3.5 m/s^2, and the maximum deceleration was –5 m/s^2.

5.3.4 Evaluation of calibration parameters

Usually, multiple runs should be conducted to reduce stochastic variability because of the large number of combinations among controllable parameters provided by commercial simulation products. The best parameter set should be selected based on the simulation results. However, in this study case, all the parameters related to the behaviour model were obtained directly from the field and had certain values. Therefore, only one simulation run was conducted to determine the simulation model could represent the real conditions.

5.4 Model validation with new data

To perform validation of the microscopic simulation model, a new set of field data was needed. One way of collecting such validation data would be to collect data for different time periods or conditions.

5.4.1 Behaviour validation

5.4.1.1 Validation of motor vehicle behaviour

The behaviour of motor vehicles has been fully calibrated and validated in previous study (Wu et al., 1998; Wu et al., 2003). Results have shown that the fuzzy model (FLOWSIM) can closely replicated real systems and in test cases has performed better than some other common deterministic models.

5.4.1.2 Validation of bicycle behaviour

In order to examine whether the model can represent the real cyclists' behaviour, a validation procedure was conducted based on the real data collected in Beijing, China.

5.4.1.2.1 Site description

The collection survey was undertaken at an unsignalized junction, which is located inside campus in Beijing, China. Site illustration is shown in Figure 5.9.

The survey site is an intersection with four approach arms. There is one lane in each direction, and bicycles and motor vehicles share the lane. Because the site surveyed was located inside the campus, bicycle flows were heavy and motor vehicle flows are very light. This enabled bicycle traffic data to be collected without motor vehicle interference.

Figure 5.9 The surveyed site

5.4.1.2.2 Data collection and reduction

The data used for validation was collected in May 2006. Cameras were used to record the visual images during a peak flow. The visual images were reduced to useful data later in the laboratory.

The bicycle flow was obtained by counting the numbers of bicycles riding between each pair of OD. The OD matrix measured from data is shown in Table 5.10.

Table 5.10 Traffic flow on site (20 minutes sample)

(bicycles/20minutes)	west	south	east	north
west	0	22	28	4
south	19	0	25	242
east	26	12	0	21
north	17	126	34	0

5.4.1.2.3 Simulation

In order to compare collected and simulated bicycle journey times, a 20-minutes simulation was carried out. Simulated journey time which start from the same origin road, end at the same destination road, and drive along the same route as field data were recorded.

According to the calibration results and data reduced, the input of the simulation model is shown in Table 5.11.

Table 5.11 Input for the simulation model

Simulation Period	20 minutes	
Traffic Flow	Table 5.10 for SET_B above	
Two parameters (μ and σ) for the calculation of desired speed	$\mu = 4.26 \quad (m/s)$	$\sigma = 0.8 \quad (m/s)$
Limitation for the acceleration	3.5 m/s^2	
Limitation for the deceleration	-5 m/s^2	

5.4.1.2.4 Result analysis

The average travel time from the model over a distance between south and north of 82 m was 19.27 seconds, with an average speed of 4.25 m/s.

The comparison of the average speed per bicycle between data simulated and surveyed is shown in Figure 5.10. The trace shows a very close fit between the data simulated and surveyed.

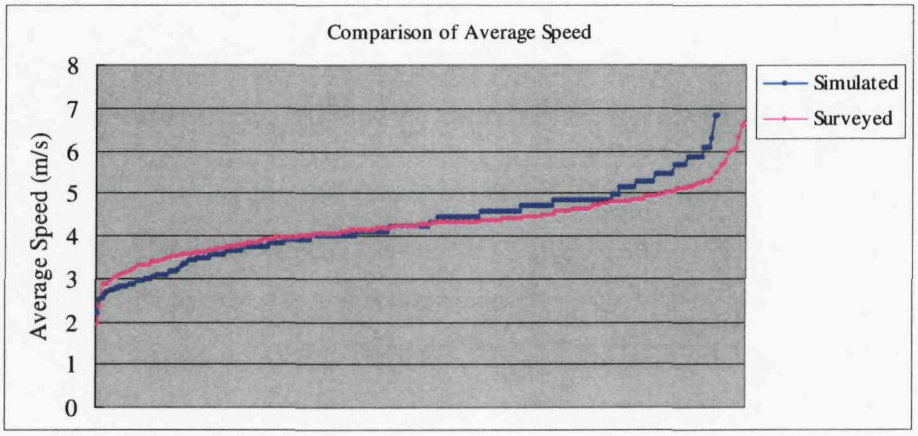


Figure 5.10 Comparison of average speed

The F-test shows these two samples' variances are unequal, therefore two-sample assuming unequal variances t-test was used. The F-test result is presented in Appendix C. The two-tailed t-test was conducted to evaluate whether the average travel time values were statistically equivalent at 5% level of significance. The result of data analysis using t-test (two sample assuming unequal variances) is shown in Table 5.12. The test statistic value of 0.03 is less than the critical t-statistic value for 5% confidence (1.96). As expected, the result shows there is no significant difference between data surveyed and simulated.

Table 5.12 t-Test: Two-Sample Assuming Unequal Variances

t-Test: Two-Sample Assuming Unequal Variances		
	Surveyed	Simulated
Mean	4.26	4.26
Variance	0.48	0.72
Observations	242	232
Hypothesized Mean Difference	0	
df	445	
t Stat	0.04	
P(T<=t) one-tail	0. 48	
t Critical one-tail	1. 65	
P(T<=t) two-tail	0. 97	
t Critical two-tail	1. 96	

5.4.2 Model validation in a mixed traffic situation

In order to validate the model on an urban road network with mixed motor vehicle and bicycle flow, further validation was conducted based on real data collected in Beijing, China. The validation focused on the motor vehicle flow with interference of bicycles. Motor vehicle journey times were used as the measurable index.

5.4.2.1 Site description

The collection survey was undertaken at peak time (7:30 am-8:30 am) at a small network that is a signalized T-junction close to the Beijing Jiaotong University in Beijing, China. The layout of this site is given in Figure 5.11.

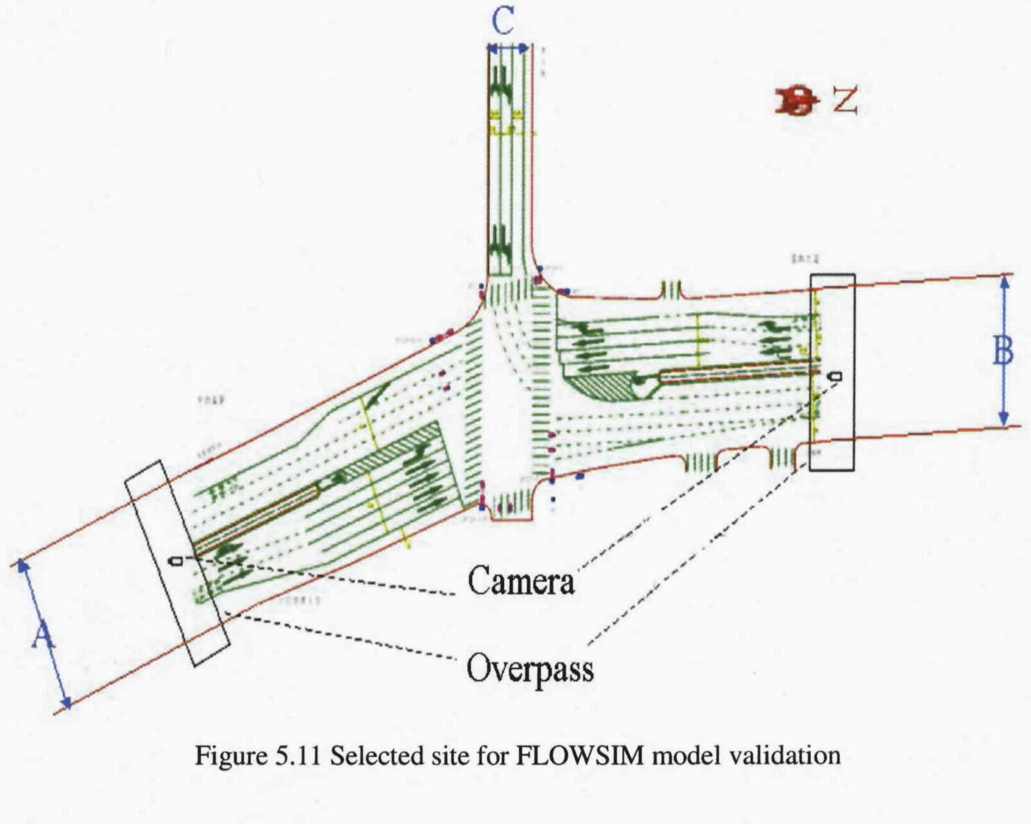


Figure 5.11 Selected site for FLOWSIM model validation

The general situations of the T-junction were shown in Table 5.13.

Table 5.13 General Situation of Selected Road Sections

Location		Near Beijing Jiaotong University
Traffic Volume at Peak Hours (veh/h)	South-North	559
	South-West	246
Signal Settings		2 staged signal Fixed setting
Traffic Facilities		2 pedestrian overpass, 2 crossings, waiting zones for left-turning automobiles etc.
Length of the selected road section (South-North)		580 m (A_B)
Length of the selected road section (West-South)		373 m (C_A)

5.4.2.2 Data collection and reduction

Cameras were used to record the visual images during peak traffic conditions. The visual images were reduced into useful data in the laboratory.

The first data needed to be reduced was traffic flow. Flow was obtained by counting the numbers of motor vehicles and bicycles moving between each pair of OD. The OD matrix measured during 1 hour are shown in Tables 5.14 and 5.15.

Table 5.14 Motor vehicle traffic flow of site

	west	south	east	north
west	0	181 veh/h	0	353 veh/h
south	246 veh/h	0	0	559 veh/h
east	0	0	0	0
north	0	702 veh/h	0	0

Table 5.15 Bicycle traffic flow of site

	west	south	east	north
west	0	0	472 bic/h	0
south	0	0	0	0
east	370 bic/h	0	0	0
north	0	0	0	0

In order to get the average journey time between west and south, two timing points were selected 373 m apart along the selected route. Two synchronized cameras were used with one set on the southbound overpass (point A), and another one was set on the roadside of the west (point C). The number plates of each motor vehicle and the time passing the timing points (that were marked in advance) was recorded. The times required to travel between point C and A were obtained by recording the time difference when the same motor vehicle passes the two fixed points.

Similarly, in order to get the average journey time between south and north, another two timing points along the selected route were selected. The two timing points are point A (south bound of southbound overpass) and point B (north bound of northbound overpass). Two synchronized cameras were used with one set on the overpass of southbound, and another on the overpass of northbound. The number plate of each motor vehicles and the time passing the points that were marked in advance was recorded. The time required travelling between point A and B was obtained by recording the time difference when the same motor vehicle passes the two fixed points.

The average journey time between point C and A was determined from the data collected for 117 motor vehicles. The maximum value is 137 s, and the minimum value is 29 s. The average journey time between point C and A is 54.6 s.

The average journey time between point A and B was determined from the data collected for 133 motor vehicles. The maximum value is 98 s, and the minimum value is 41 s. The average journey time between point A and B is 65.1 s.

Signal setting is another important piece of data needed from on-site observation and video recorded. The detailed signal phase and stage arrangements are given in Figure 5.12.

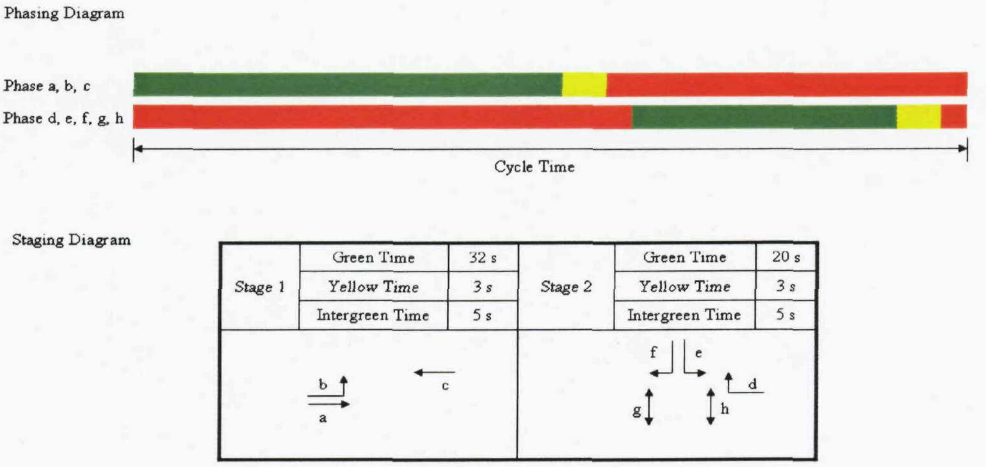


Figure 5.12 Signal phase and stage arrangement

5.4.2.3 Simulation

In order to compare motor vehicle journey times of collected and simulated conditions, a simulation process was carried out. The simulation network was built on a detailed base map as it helped to build network topology accurately and to locate control equipment. Once the basic simulation network was set, the next step was to represent mixed traffic with motor vehicles and bicycles and their routings in the study road section. Simulated journey time which start from the same origin road, end at the same destination road, and drive along the same route as field data were recorded. According to the data reduced, the input of the simulation model is shown in Table 5.16.

Table 5.16 Input of simulation model

Simulation Period	1 hour	
Traffic Volume at Peak Hours (veh/h)	Table 5.14 and 5.15 above	
Two parameters (μ and σ) for the distribution of bicycle desired speed	$\mu = 4.26 \quad (m/s)$	$\sigma = 0.8 \quad (m/s)$
Limitation for the bicycle acceleration	3.5 m/s^2	
Limitation for the bicycle deceleration	–5 m/s^2	
Two parameters (μ and σ) for the distribution of vehicle desired speed	$\mu = 10.87 \quad (m/s)$	$\sigma = 1.2 \quad (m/s)$
Signal Settings	2 staged signal Fixed setting (Figure 5.10)	
Length (South-North)	580 m (A_B)	
Length (Wesr-South)	373 m (C_A)	

5.4.2.4 Results analysis for route A_B

The mean values were used to compare with the field data. The field data, simulated data and the difference between them are shown in Table 5.17.

Table 5.17 The comparison of the results between collected and simulated for route A_B

	Field Data	Simulated Data
Sample size	133	522
Average Journey Time (s)	65.1	64.5

The comparison of frequency distribution of journey time between data surveyed and simulated for route A_B is shown in Figure 5.13.

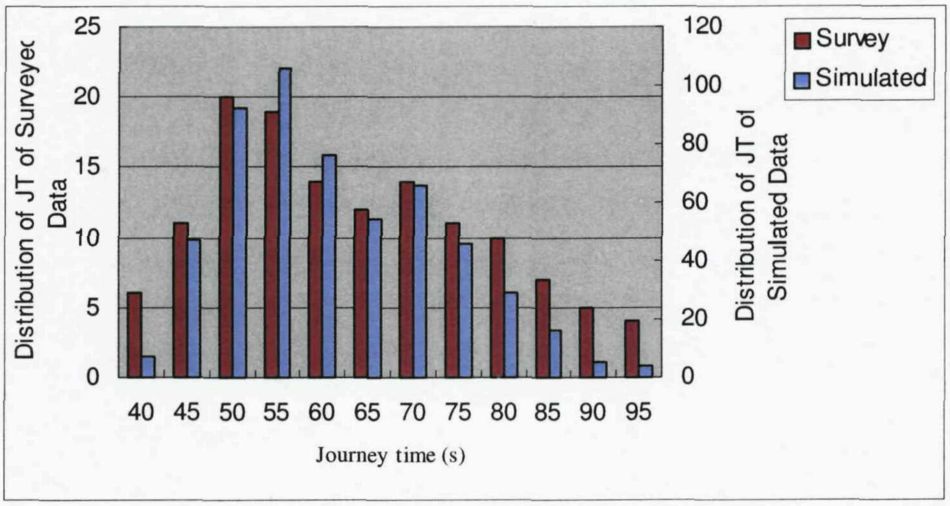


Figure 5.13 Comparison of Frequency Distribution of Journey Time between Data Surveyed and Simulated for route A_B

The F-test shows these two samples' variances are unequal, therefore a two-sample assuming unequal variances t-test was used. F-test result was presented in Appendix C. The two-tailed t-test was conducted to evaluate whether the average travel time values were statistically equivalent at 5% level of significance. The results of data analysis using t-test (two sample assuming unequal variances) are given in Table 5.18.

Table 5.18 t-Test: Two-Sample Assuming Unequal Variances

t-Test: Two-Sample Assuming Unequal Variances		
	Survey	Simulate
Mean	65.19	64.52
Variance	200.67	113.85
Observations	133	522
Hypothesized Mean Difference	0	
df	172	
t Stat	0.51	
P(T<=t) one-tail	0.30	
t Critical one-tail	1.65	
P(T<=t) two-tail	0.61	
t Critical two-tail	1.97	

The test statistic value of 0.51 is less than the critical t-statistic value for 5% confidence (1.97). As expected, the result shows there is no significant difference between data surveyed and simulated.

5.4.2.5 Results analysis for route C_A

The mean values were compared with the field data. The field data, simulated data and the difference between them are given in Table 5.19.

Table 5.19 The comparison of the results between collected and simulated for route C_A

	Field Data	Simulated Data
Sample size	117	178
Average Journey Time (s)	54.6	54.4

The comparison of frequency distribution of journey time between data surveyed and simulated for route C_A is shown in Figure 5.14.

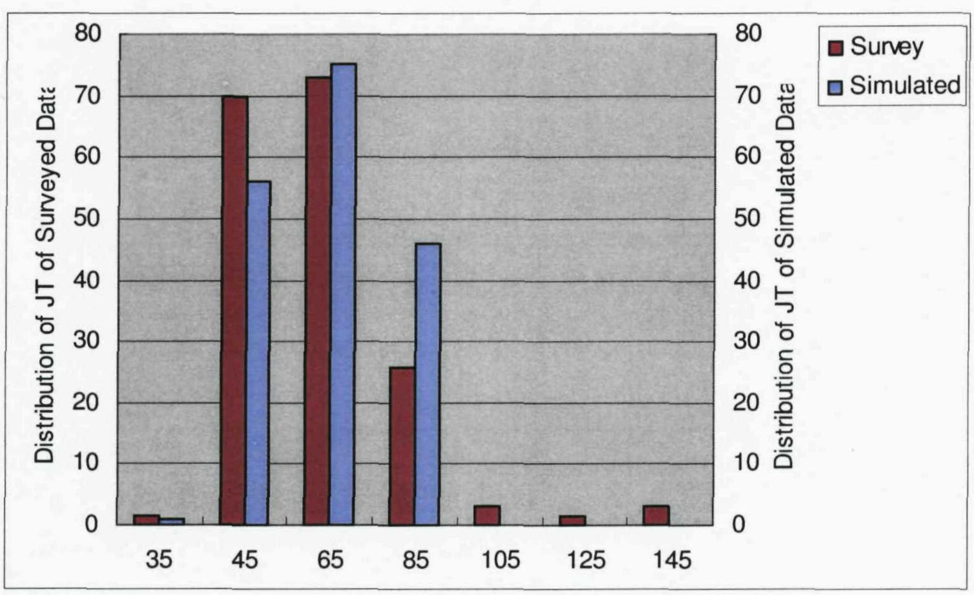


Figure 5.14 Comparison of Frequency Distribution of Journey Time between Data Surveyed and Simulated for route C_A

The F-test shows that the two sample variances are unequal, therefore two-sample assuming unequal variances t-test was used. F-test result was presented in Appendix C. The two-tailed t-test was conducted to evaluate whether the average travel time values were statistically equivalent at 5% level of significance. The result of data analysis using t-test (two sample assuming unequal variances) is given in Table 5.20. As expected, the result shows there is no significant difference between data surveyed and simulated.

Table 5.20 t-Test: Two-Sample Assuming Unequal Variances

t-Test: Two-Sample Assuming Unequal Variances		
	Survey	Simulate
Mean	54.59	54.40
Variance	382.18	182.66
Observations	117	178
Hypothesized Mean Difference	0	
df	188	
t Stat	0.09	
P(T<=t) one-tail	0.46	
t Critical one-tail	1.65	
P(T<=t) two-tail	0.93	
t Critical two-tail	1.97	

5.5 Conclusions

The simulation results show there are no significant differences between data surveyed and simulated with or without considering the conflicts between motor vehicles and bicycles. Calibration and validation results suggest that the simulation model is capable of replicating mixed traffic with motor vehicles and bicycles at signalised intersections in Beijing, China.

There are two pedestrian overpass located at the northbound and southbound of the intersection, and almost all the pedestrian cross the intersection through the overpass. Therefore, these are only bicycle flow and motor vehicle flow in this study field. This satisfies the requirements for the simulation model, which only considers mixed motor vehicle and bicycle traffic. Influences of pedestrian should be considered when this simulation model be applied to more complicated situations.

In this study, only one field collected data point was used during each of calibration and validation tests. It is more desirable to collect multiple data points, collected over time, of field data to consider variability of field data.

Although a sizable signalized intersection was used in this study, more complex intersection could be considered to verify the performance of this procedure. Further research should be conducted to determine whether the parameters are applicable to all networks or simply to this specific one.

Furthermore, it is more desirable to consider many different calibration parameters or other performance measures to understand inherent variability associated with the simulation models.

6. Simulation study

6.1 Introduction

In Beijing, China, the bicycle remains one of the major transport modes. Because of the significant differences in characteristics between motor vehicles and bicycles, the problems of mixed traffic at at-grade intersections has been a major issue of urban traffic management for a long time. Several scenarios to deal with this situation have been developed and are described in this chapter. The potential scenarios have been studied by using the microscopic simulation model developed, calibrated and validated as part of this research and the results reported. Different proportions of bicycle flow were applied to these scenarios. The scenarios studied were:

1. Scenario 1: No bicycle lane signals;
2. Scenario 2: Bicycles use the same signal phase as motor vehicles;
3. Scenario 3: A second advanced stop line for bicycles;
4. Scenario 4: Exclusive bicycle signals;
5. Scenario 5: Building a separate lane of prohibiting of left-turn bicycles;

Each of these scenarios is described below:

6.2 Base scenario

In order to study the effect of different operations of bicycle traffic on traffic at at-grade intersections in Beijing, China, a base scenario was required. Different operations were applied to the basic scenario, based on the simulation processes carried out.

6.2.1 Some assumptions

The design of the base scenario should be able to reflect the real situation of intersections in Beijing, China. Some assumptions were made and described below:

1. In Beijing, China, most of the intersections are four-arm cross intersection. Therefore, a typical four-arm intersection with three vehicle lanes and one bicycle lane in each arm was assumed to be the basic layout. There are two major types of bicycle road in Chinese cities. One is the separated bicycle road, in which the motor vehicle traffic and bicycle traffic are

divided by an open space, barrier or raised median (Wei et al., 2003B). The other is these mixed bicycle road, in which bicycles share the roadway with motor vehicle traffic without any physical or nonphysical separations (Wang and Wei, 1993). The lateral disturbance caused as cyclists intend to compete with motor vehicles usually occurs on the mixed bicycle road. According to the research (Wang and Wei, 1993), the separated bicycle road is the most common type at city level in large and medium city in China and was adopted in this study.

2. There are many factors which affect the performance of the traffic. For example, a queue build up downstream may cause extra delay to the traffic. In order to predigest the influential factors and focus on the influence caused by bicycles and operations, an isolated intersection was used and the influence factors outside this intersection were not considered, i.e. there were no delays related to the queues outside the intersection.

3. Whilst, pedestrian may be considered to have characteristics which are not dissimilar to slower bicycles and share the same signal phase with bicycles, different behaviour (more flexible and more crowded) and different facilities may still influence traffic differently. At the test site, in order to eliminate other factors that influence the traffic, pedestrians were assumed to cross the intersection via an underpass or pedestrian bridge. Therefore, pedestrian interactions were not taken into consideration in this simulation study.

4. The arrival and departure of buses at the bus stop will also influence traffic behaviour. It was assumed that there was no bus stop inside this intersection.

5. According to the China Urban Road Traffic Planning Design Criterion (The People's Republic of China Construction Ministry, 1998), the suggested width of vehicle lane is 3.5 m at at-grade intersection in China. However, the width of general vehicle lane in Beijing, China is 3.75m and was adopted in this study.

6. Because the strip-based approach was adopted for bicycles in this study, the width of bicycle lanes is one of the important factors affecting capacity and saturation flow of bicycles (see 6.2.3.2). According to the design criterion of highway and urban road in Code for Design of Municipal Road (Beijing Municipal Institute of City Planning and Design, 1998), the minimum width of bicycle lane is 1.0 m. China Urban Road Traffic Planning Design Criterion (The People's Republic of China Construction Ministry, 1998) suggests the

reasonable width of bicycle lane is 3.5 m for major road and 2.5 m for minor road. A bicycle lane width of 3.5 m was adopted in this study.

7. If the modelled length of each approach is too short, queues can build up beyond the initial simulation start points, i.e. some vehicles may not be able to enter the road due to the entrance blocked by the queue and vehicle delay may be underestimated. In general, the distance between two intersections in urban network in China is about 500 meters (Jin and Zhai, 2006). Therefore, 500 m was adopted as the length of each lane in this study. The implications of queuing beyond 500 m are discussed below.

8. In order to compare the influence on the traffic under different operations dealing with bicycles, vehicular traffic flows were kept at the same values.

9. The traffic flow in each arm was set to be the same. The left-turn, through and right-turn bicycle flows in each arm were assumed to be the same.

10. Fixed-time signal control was adopted in this research. To avoid the conflicts among motor vehicles and reduce the influence caused by the motor vehicle interaction, four-phase signal setting was employed. The amber period was standardized at 3 sec.

11. In Practice, there may be increased cycle length to accommodate increasing bicycles. However, for study purpose, to make results comparable, all these conclusions are based on the assumption that the cycle time only considered the influence of motor vehicles, without considering the influence of increasing bicycle flow.

6.2.2 Geometry description

Based on the assumptions above, a typical isolated four-arm signalised intersection with three vehicle lanes and one separated bicycle lane in each arm was assumed to be the basic layout. The scenario is illustrated in Figure 6.1.

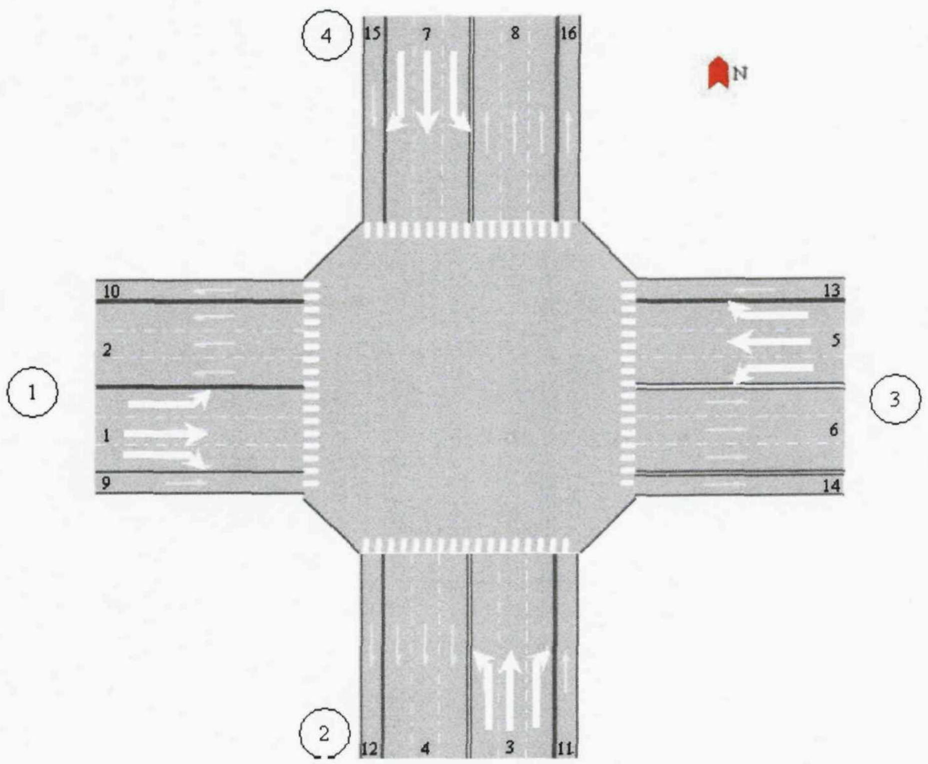


Figure 6.1 Illustration of base scenario

The details of geometric characteristics of basic scenario are given in Table 6.1.

Table 6.1 Geometric characteristics of the basic scenario

Link no.	Type	Length (m)	Width (m)
1	Vehicle lane	500	3.75
2	Vehicle lane	500	3.75
3	Vehicle lane	500	3.75
4	Vehicle lane	500	3.75
5	Vehicle lane	500	3.75
6	Vehicle lane	500	3.75
7	Vehicle lane	500	3.75
8	Vehicle lane	500	3.75
9	Bicycle lane	500	3.5
10	Bicycle lane	500	3.5
11	Bicycle lane	500	3.5
12	Bicycle lane	500	3.5
13	Bicycle lane	500	3.5
14	Bicycle lane	500	3.5
15	Bicycle lane	500	3.5
16	Bicycle lane	500	3.5

6.2.3 Traffic data

For simplicity of discussion, the traffic demands were assigned equally between the OD pairs.

6.2.3.1 Vehicular traffic flow

In general, the saturation flow of vehicular traffic of each lane in China is 1600 veh/h/l (Gao et al., 2003; Luo, 2004). The maximum possible ratio of flow to saturation flow Y_{pract} can be calculated according to the function (Salter and Hounsell, 1996) below:

$$Y_{pract} = 0.9(1 - \frac{L}{120}) \tag{6.1}$$

$$Y = \frac{y}{S} \tag{6.2}$$

where,

L is the lost time (s);

Y is the flow (veh);

S is the saturation flow of vehicle (veh);

The lost time (L) was 16 sec (see section 6.2.4), based on that, Y_{pract} was calculated to be 0.78. Since the flow was assigned to each lane equally, the maximum possible flow was 312 veh/h. Therefore, 312 veh/h was employed as the vehicular traffic flow in each OD pair in this study. The assumed vehicular traffic demand data are listed in Table 6.2.

Table 6.2 Vehicular traffic demand data

Veh/h	D_1 (Link 2)	D_2 (Link 4)	D_3 (Link 6)	D_4 (Link 8)
O_1(Link 1)	0	312	312	312
O_2 (Link 3)	312	0	312	312
O_3 (Link 5)	312	312	0	312
O_4 (Link 7)	312	312	312	0

6.2.3.2 Bicycle traffic flow

In this study, the bicycle traffic flow in each arm was set to be the same. The left-turn, through and right-turn bicycles flows in each arm were assumed to be the same. Sun and Yang (2004) collected and analysed large numbers of data in China. A relationship between the width of bicycle lane and the saturation flow of bicycles was obtained in his research. The saturation flow of bicycles S_B can be calculated according to the function (Sun and Yang, 2004) below:

$$S_B = 3600(0.55W + 0.41) \tag{6.3}$$

where,

S_B is the saturation flow of bicycles (bic/h);

W is the width of bicycle lane (m);

Since the width of bicycle lane was 3.5 m in this study and the maximum possible ratio of flow to saturation flow was 0.78, the maximum possible bicycle flow of each bicycle lane was 3278 bic/h. Because one of the purposes of this study is to study the effect of different proportions of bicycle flow, the bicycle flow was divided into six levels from zero to the maximum value. The six levels of bicycle flow are listed in Table 6.3.

Table 6.3 Six levels of bicycle traffic demand

	O_1 (Link 9)			O_2 (Link 11)			O_3 (Link 13)			O_4 (Link 15)		
	Left-turn (Bic/h)	Through (Bic/h)	Right-turn (Bic/h)	Left-turn (Bic/h)	Through (Bic/h)	Right-turn (Bic/h)	Left-turn (Bic/h)	Through (Bic/h)	Right-turn (Bic/h)	Left-turn (Bic/h)	Through (Bic/h)	Right-turn (Bic/h)
Level 1	0	0	0	0	0	0	0	0	0	0	0	0
Level 2	200	200	200	200	200	200	200	200	200	200	200	200
Level 3	400	400	400	400	400	400	400	400	400	400	400	400
Level 4	600	600	600	600	600	600	600	600	600	600	600	600
Level 5	800	800	800	800	800	800	800	800	800	800	800	800
Level 6	1093	1093	1093	1093	1093	1093	1093	1093	1093	1093	1093	1093

6.2.4 Signal setting

Fixed signal control was adopted in this research. To avoid the conflicts among motor vehicles and reduce the influence caused by the motor vehicle interaction, a four-phase signal setting was employed. The basic staging diagram is shown below:

Staging Diagram

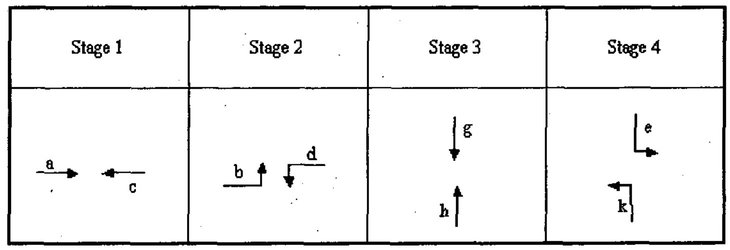
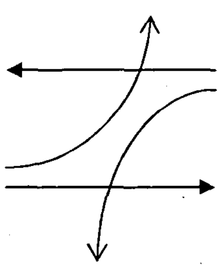
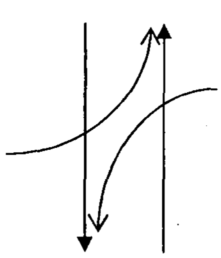
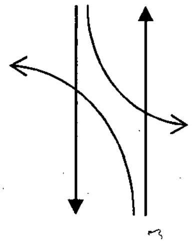
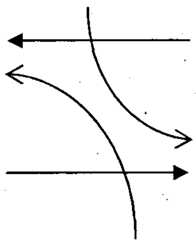


Figure 6.2 Basic signal staging diagram

To calculate the intergreen time, we found out the main conflicts between every two stages. According to the data collected for calibration and validation, the average vehicle speed is 10.87 m/s and the average bicycle speed is 4.26 m/s in the model.

Table 6.4 Calculation of intergreen period

Stage	Intergreen period	
Stage1-2 	Vehicle (west to east; east to west): L=29.75m t1=L/v=29.75/10.87=2.74s Vehicle (west to North; east to south): L=20.05m t2=L/v=20.05/10.87=1.84s Intergreen time: I= t1 -t2=2.74-1.84=0.9s≤5s	Bicycle (west to east; east to west): L=24.09m t1=L/v=24.09/4.26=5.65s Vehicle (west to North; east to south): L=26.6m t2=L/v=26.6/10.87=2.45s Intergreen time: I= t1 -t2=5.65-2.45=3.2s≤5s
Stage2-3 	Vehicle (west to North; east to south): L=20.05m t1=L/v=20.05/10.87=1.84s Vehicle (north to south; south to north): L=18.55m t2=L/v=18.55/10.87=1.71s Intergreen time: I= t1 -t2=1.84-1.71=0.13s≤5s	Biycle (west to North; east to south): L=19.61m t1=L/v=19.61/4.26=4.6s Vehicle (north to south; south to north): L=29.27m t2=L/v=29.27/10.87=2.69s Intergreen time: I= t1 -t2=4.6-2.69=1.91s≤5s (Table continues)

(Continued _Table 6.4) Stage3-4 	Vehicle (north to south; south to north): L=30.24m t1=L/v=30.24/10.87=2.78s Vehicle (south to west; north to east): L=20.24m t2=L/v=20.24/10.87=1.86s Intergreen time: I= t1 -t2=2.78-1.86=0.92s≤5s	Bicycle (north to south; south to north): L=23.61m t1=L/v=23.61/4.26=5.54s Vehicle (south to west; north to east): L=26.85m t2=L/v=26.85/10.87=2.47s Intergreen time: I= t1 -t2=5.54-2.47=3.07s≤5s
Stage4-1 	Vehicle (south to west; north to east): L=20.05m t1=L/v=20.05/10.87=1.84s Vehicle (west to east; east to west): L=18.79m t2=L/v=18.79/10.87=1.73s Intergreen time: I= t1 -t2=1.84-1.73=0.11s≤5s	Bicycle (south to west; north to east): L=18.79m t1=L/v=18.79/4.26=4.41s Vehicle (west to east; east to west): L=29.51m t2=L/v=29.51/10.87=2.71s Intergreen time: I= t1 -t2=4.41-2.71=1.7s≤5s

Some assumptions were made as below:

1. The amber period (A) was standardized at 3 sec;
2. The all red period (IG-A) was assumed to be 2 sec;
3. Start and end lost time is 2 sec for each of the four green periods of the four-phase cycle;
4. The saturation flow of vehicular traffic for each lane is 1600 veh/h/l in China (Gao et al., 2003; Luo, 2004);

Based on the assumptions, the signal time was calculated according to the vehicular traffic flow:

$$Y = y1 + y2 + y3 + y4 = \frac{312}{1600} + \frac{312}{1600} + \frac{312}{1600} + \frac{312}{1600} = 0.78$$

Lost time: $L = \sum_{i=1}^{4} ((IG - A) + 2) = 16$

The optimum cycle time: $C_0 = \dfrac{1.5L+5}{1-Y} = 132$

Effective green time:

$$g_1 = \frac{y_1}{Y} \times (C_0 - L) = 29$$

$$g_2 = \frac{y_2}{Y} \times (C_0 - L) = 29$$

$$g_3 = \frac{y_3}{Y} \times (C_0 - L) = 29$$

$$g_4 = \frac{y_4}{Y} \times (C_0 - L) = 29$$

Actual green time:

$$G_1 = g_1 + 2 - A = 28$$

$$G_2 = g_2 + 2 - A = 28$$

$$G_3 = g_3 + 2 - A = 28$$

$$G_4 = g_4 + 2 - A = 28$$

The detailed signal stage and phase arrangements are shown in Figure 6.3 and Figure 6.4.

Staging Diagram

Stage 1	a c	Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 2	b d	Green Time	28s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 3	g h	Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 4	e k	Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s

Figure 6.3 Signal stage arrangement

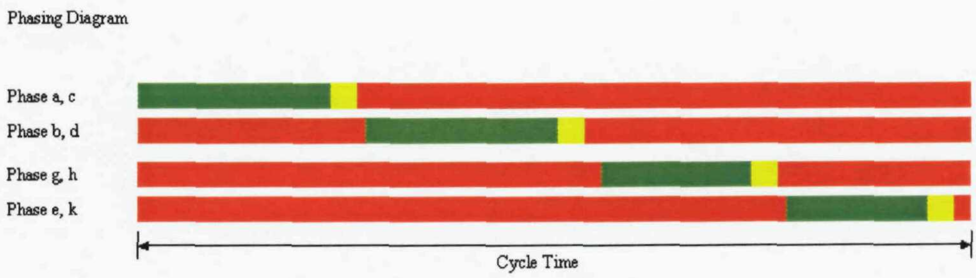


Figure 6.4 Signal phase arrangement

6.2.5 The inputs of simulation model

Based on the data collected and analysed at intersections in Beijing, China (see the calibration and validation process in chapter 5), some parameters used as input to the simulation model are listed in Table 6.5.

Table 6.5 The inputs of simulation model

Simulation Period	1 hour	
Two parameters (μ and σ) for the distribution of bicycle desired speed	$\mu = 4.26 \quad (m/s)$	$\sigma = 0.8 \quad (m/s)$
Limitation for the bicycle acceleration	3.5 m/s^2	
Limitation for the bicycle deceleration	-5 m/s^2	
Two parameters (μ and σ) for the distribution of vehicle desired speed	$\mu = 10.8 \quad (m/s)$	$\sigma = 1.2 \quad (m/s)$

6.2.6 Measures for output

Journey time, vehicle delay and crossing volume were adopted as the measures for the output of simulation results.

1. **Journey Time** was the time used to travel from original section to destination of each vehicle or bicycle in average. The unit for vehicle journey time was sec/veh. The unit for bicycle journey time was sec/bic.

2. **Average Delay** was defined as the time difference between the actual time used to travel from original section to destination and the time needed to finish this trip at its desired speed on average. The unit for vehicle delay was sec/veh. The unit for bicycle delay was sec/bic.

$$d = t_{tr} - \frac{s_{tr}}{v_{des}} \tag{6.4}$$

where,

t_{tr} is the actual travel time of a vehicle;

s_{tr} is the actual travel distance;

v_{des} is the desired speed;

3. In this study, **Crossing Volume** was defined as the number of vehicles or bicycles crossing the stop line of the individual lanes during the simulation period. **Crossing Volume of the intersection** was the sum of the Crossing Volume of the individual lanes comprising the intersection. The unit for motor vehicle was veh. The unit for bicycle was bic.

6.3 Sensitivity testing

In order to test the effects on results of varying traffic flows and turning proportions, three sensitivity analyses were conducted.

In Practice, there may be adjusted cycle time to accommodate different turning proportions. However, for study purpose, to make results comparable, all these conclusions are based on the assumption that the cycle time was fixed. Detailed signal phase and stage arrangement used in sensitivity testing are presented in Figure 6.5 and Figure 6.6 below.

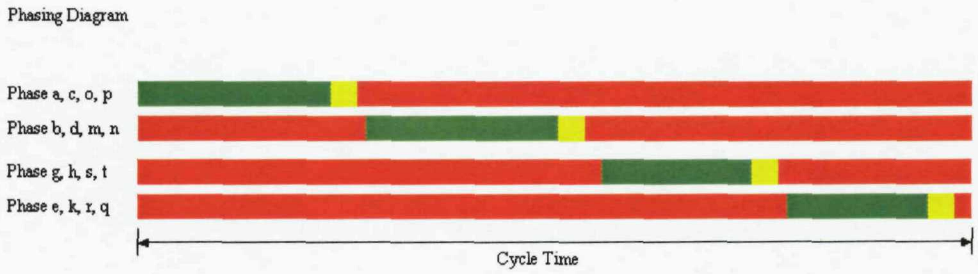


Figure 6.5 Signal phase arrangement

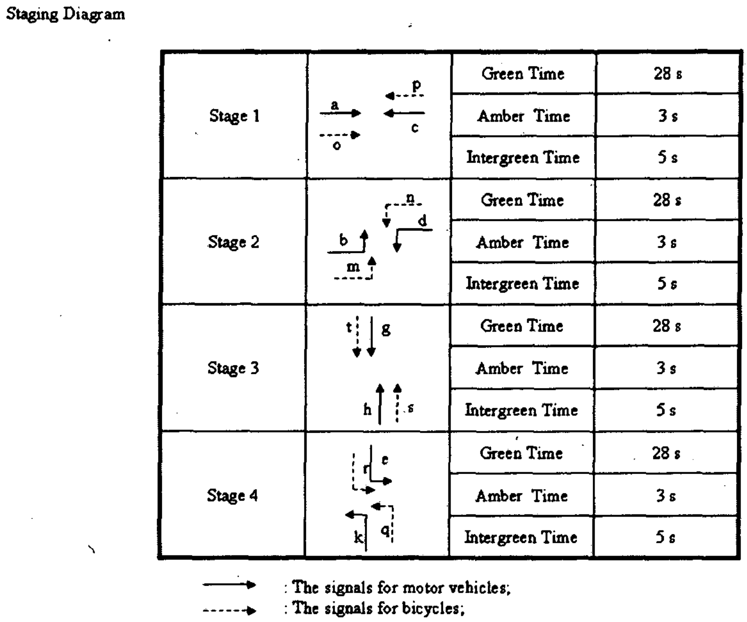


Figure 6.6 Signal stage arrangement

6.3.1 Varying traffic flows

In order to test the effects on results of varying traffic flows, a sensitivity analysis was conducted, where motor vehicle traffic demands were increased and decreased. Factors of 0.80, 0.90, 1.10, 1.20 and 1.30 were applied to all demand inputs, and results were analyzed.

To test the sensitivity of the results to demand changes, six sets of simulation runs were completed, and the average travel time of each run was recorded. One set of runs used motor vehicle traffic demands as the base case (100% demand). One set of runs used traffic demands 10% higher than the base case (110% demand). One set of runs used traffic demands 20% higher than the base case (120% demand). One set of runs used traffic demands 30% higher than the base case (130% demand). One set of runs used traffic demands 10% lower than the base case (90% demand). The other set used traffic demands 20% lower (80% demand).

The six different levels of vehicle traffic demands are shown in Table 6.6.

Table 6.6 Motor vehicle traffic demand levels for sensitivity testing

Veh/h	Demand level					
	80%	90%	100%	110%	120%	130%
Left-turn motor vehicle	249	280	312	343	374	405
Through motor vehicle	249	280	312	343	374	405
Right-turn motor vehicle	249	280	312	343	374	405

In order to research the effects of motor vehicle traffic demands, bicycle flow was fixed to be at level 2 (Table 6.3).

The results of a sensitivity analysis for different vehicle traffic demands are illustrated in Figure 6.7. The travel time with base case (100% demand) was scored 1 while the others were given a relative score using the ratio of the travel times. For example, if the total travel time with 100% demand was 30 minutes, that value was given a score of 1. Then, if another run had a total travel time of 40 minutes, that value would be given a score of 1 * 40/30 = 1.33.

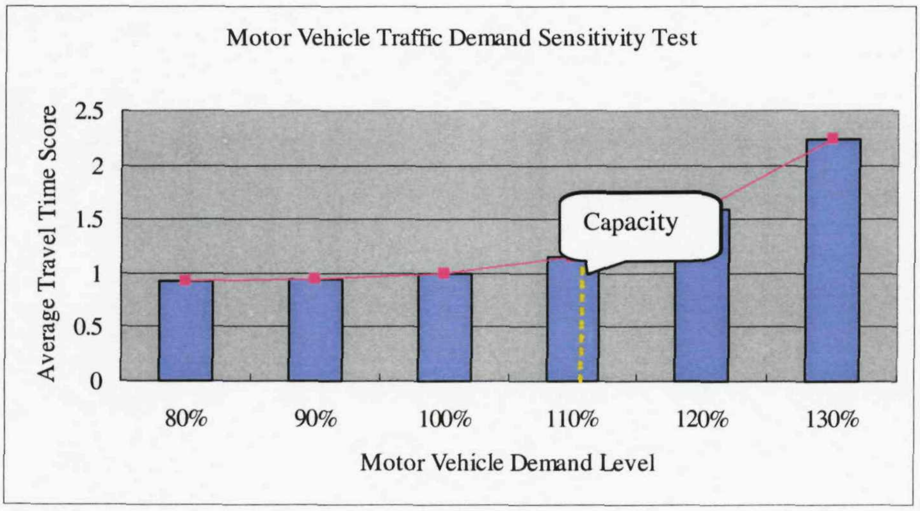


Figure 6.7 Motor vehicle traffic demand sensitivity test

Because 1600 veh/h/l was adopted as the saturation flow of vehicular traffic of each lane in this study (see 6.2.3.1), the maximum number of vehicles crossing the stop line per lane per hour was 351/h/l (1600*29/132) based on the signal setting used in this test. It can be seen that when the traffic flow is lower than the capacity, the influence of increased demand on travel time is small. When the traffic demand, which is higher than 110% demand, exceeds the capacity, the travel time increased sharply. It is suggested in Figure 6.7 that travel times are sensitive to the level of traffic demand, as would be expected.

6.3.2 Varying motor vehicle turning proportions

In order to test the effects on results of varying turning proportions, a sensitivity analysis was conducted, where motor vehicle turning proportions were changed. Five sets of simulation runs were completed, and the average travel time of each run was recorded. The five different levels of turning proportions are shown in Table 6.7.

In order to research the effects of motor vehicle traffic demands, bicycle flow was fixed to be at level 2 (Table 6.3).

Table 6.7 Five different levels of turning proportions for sensitivity testing (motor vehicle)

	level				
	Level 1	Level 2	Level 3 (Base case)	Level 4	Level 5
Left-turn motor vehicle	20% (187 veh/h)	30% (281 veh/h)	33.4% (312 veh/h)	40% (374 veh/h)	50% (468 veh/h)
Through motor vehicle	40% (375 veh/h)	50% (468 veh/h)	33.3% (312 veh/h)	30% (281 veh/h)	20% (187 veh/h)
Right-turn motor vehicle	40% (374 veh/h)	20% (187 veh/h)	33.3% (312 veh/h)	30% (281 veh/h)	30% (281 veh/h)
Total volume in each approach	100% (936 veh/h)	100% (936 veh/h)	100% (936 veh/h)	100% (936 veh/h)	100% (936 veh/h)

The results of a sensitivity analysis for varying motor vehicle turning proportions are illustrated in Figure 6.8 and 6.9. The travel time with base case (Level 3) was scored 1 while the others were given a relative score using the ratio of the travel times. For example, if the average travel time with level 3 was 30 minutes, that value was given a score of 1. Then, if another run had an average travel time of 40 minutes, that value would be given a score of 1 * 40/30 = 1.33.

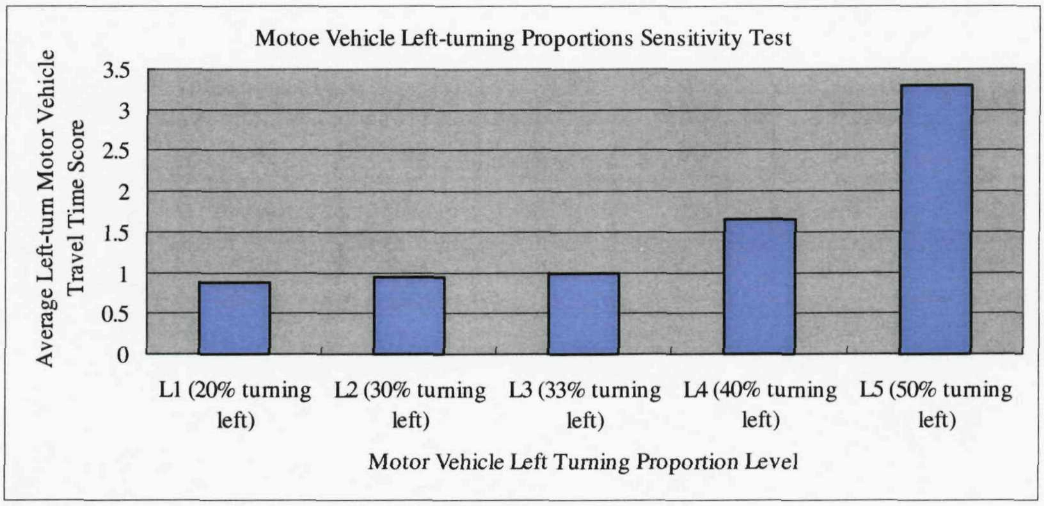


Figure 6.8 Motor vehicle sensitivity test (Left-turning proportions)

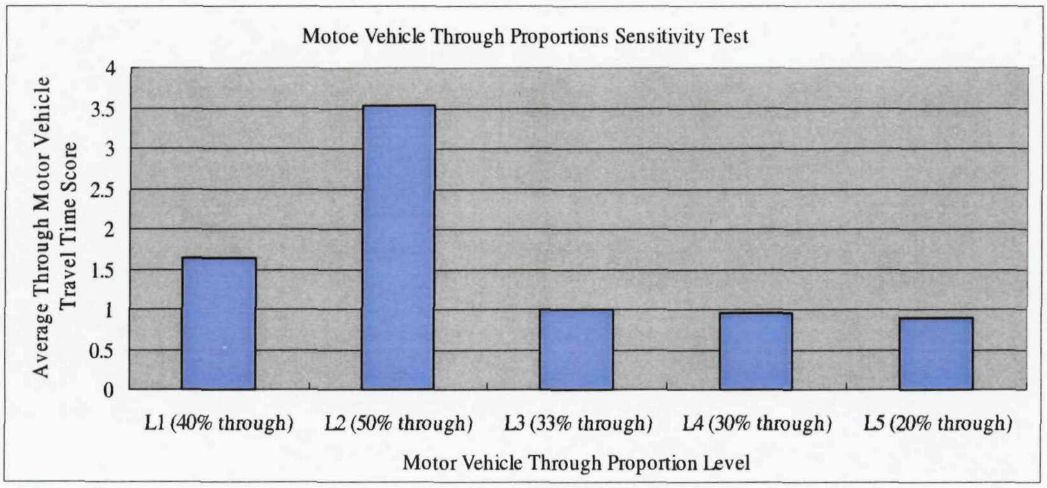


Figure 6.9 Motor vehicle sensitivity test (through proportions)

Motor vehicles can perform right turn on red without effect on through and left-turn bicycles based on the fixed signal setting used in this test. Therefore, in order to research the effect of changing proportions, and to make results comparable, only left-turn and through vehicle travel time were compared. It can be seen that the average travel time increased with the increasing turning proportions. It is suggested in Figure 6.8 and 6.9 that travel times are sensitive to the turning proportions, as would be expected.

6.3.3 Varying bicycle turning proportions

In order to test the effects on results of varying bicycle turning proportions, a sensitivity analysis was conducted, where bicycle turning proportions were changed.

Bicycles in each approach with different turning intentions share the same bicycle lane and affect each other. However, bicycles can perform right turn on red with minor effect on through and left turning bicycles. Therefore, in order to observe the effect of changing proportions, and to make results comparable, two tests were conducted. The total bicycle volume in each approach was fixed. Through bicycles proportion was assumed to be fixed at 40% in the first test to observe the effect of changing left turning proportions, and left-turn bicycles proportion was assumed to be fixed at 40% in the second test to examine the effect of changing through proportions. Six sets of simulation runs were completed, and the average travel time of each run was recorded. The six different levels of turning proportions in two tests are shown in Table 6.8 and 6.9.

Table 6.8 Levels of bicycle turning proportions for sensitivity testing (Test 1)

	Test 1 (Through proportions were fixed)		
	Level 1	Level 2	Level 3
Left-turn bicycle	20% (480 bic/h)	30% (720 bic/h)	40% (960 bic/h)
Through bicycle	40% (960 bic/h)	40% (960 bic/h)	40% (960 bic/h)
Right-turn bicycle	40% (960 bic/h)	30% (720 bic/h)	20% (480 bic/h)
Total volume in each approach	100% (2400 bic/h)	100% (2400 bic/h)	100% (2400 bic/h)

Table 6.9 Levels of bicycle turning proportions for sensitivity testing (Test 2)

	Test 2 (Left turning proportions were fixed)		
	Level 1	Level 2	Level 3
Left-turn bicycle	40% (960 bic/h)	40% (960 bic/h)	40% (960 bic/h)
Through bicycle	20% (480 bic/h)	30% (720 bic/h)	40% (960 bic/h)
Right-turn bicycle	40% (960 bic/h)	30% (720 bic/h)	20% (480 bic/h)
Total volume in each approach	100% (2400 bic/h)	100% (2400 bic/h)	100% (2400 bic/h)

In order to research the effects of bicycle traffic demands, motor vehicle flow was fixed as listed in Table 6.2.

The results of a sensitivity analysis for varying bicycle turning proportions are illustrated in Figure 6.10 and 6.11. The shortest travel time was scored 1 while the others were given a relative score using the ratio of the travel times. For example, if the travel time with level 3 was 30 minutes, that value was given a score of 1. Then, if another run had a travel time of 40 minutes, that value would be given a score of 1 * 40/30 = 1.33.

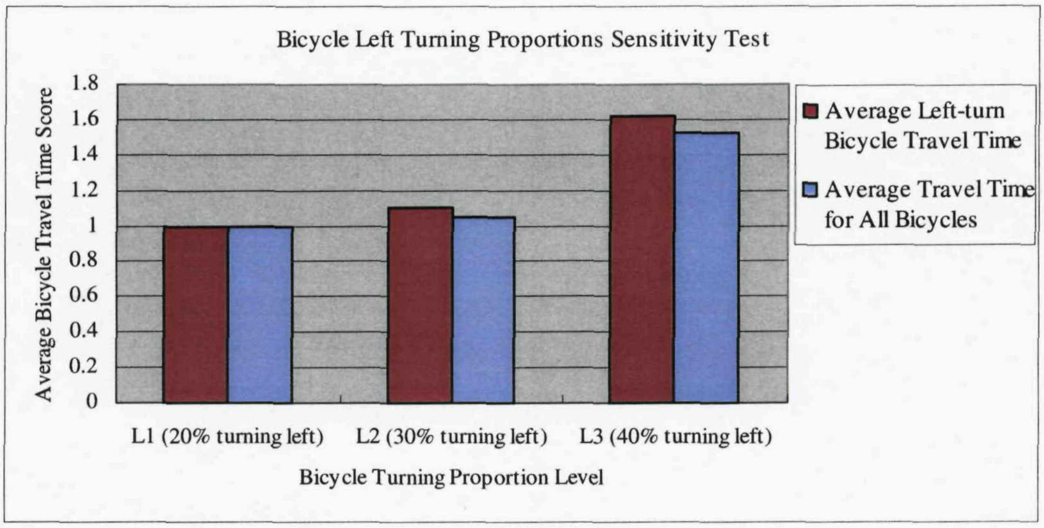


Figure 6.10 Bicycle left turning proportions sensitivity test

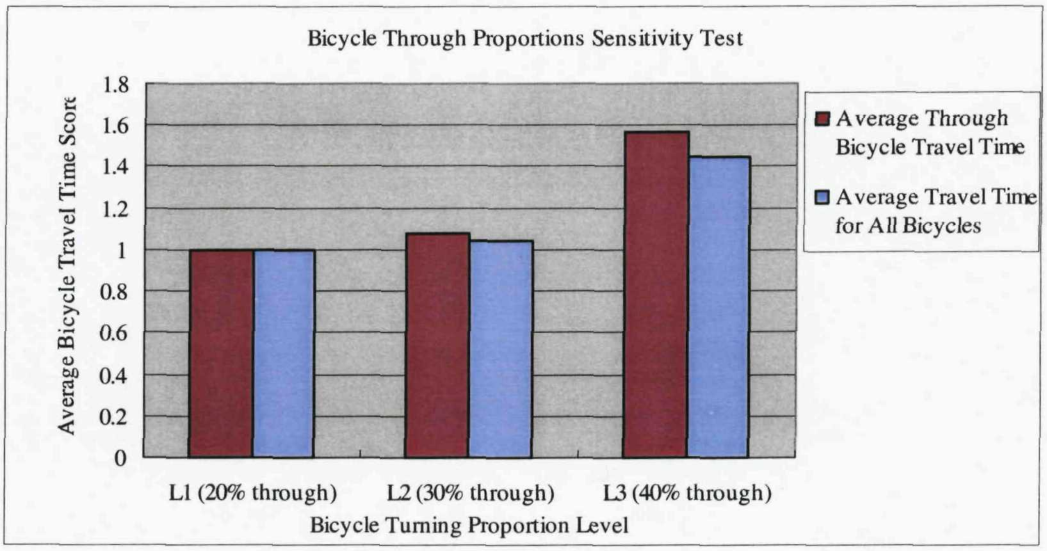


Figure 6.11 Bicycle through proportions sensitivity test

It can be seen that the average left-turn bicycle travel time increased with the increasing left turning proportions. Also, it can be seen that the average through bicycle travel time increased with the corresponding increasing through proportions. The overall average bicycle travel time still increased, though the total bicycle volume was fixed and the right turning proportions decreased with the increase of left turning proportions (when through proportions were fixed) and through proportions (when left turning proportions were fixed). However, the rate of increase in the overall average bicycle travel time was lower than that in the left-turn/through bicycle travel time. It can be seen that the effect of right-turn bicycles is less obvious than that of left-turn/through bicycles. And compared with a situation in which the total bicycle volumes were at a low level, the effect of increasing proportions was greater when the total bicycle volumes were heavy. The reason lies in the fact that bicycles can perform right turn on red with minor effect on through and left-turn bicycles. When the bicycle volumes were at a high level, the left-turn/through bicycles had to wait before the stop line, thereby blocking bicycles with other turning intentions. It is suggested in Figure 6.10 and 6.11 that travel times are sensitive to the turning proportions, as would be expected.

6.4 Scenario1: The influence of different proportion of bicycle flow without signals for bicycle lane

In this scenario, it was assumed that there were no special signals for bicycles at the intersection. Bicycles waited for a suitable gap when approaching to the intersection. Once the gap was accepted, bicycles crossed the intersection. In order to study the impact of different proportion of bicycle flow on traffic at intersections, several levels of bicycle flow were adopted.

6.4.1 Geometry description

The geometry was the same as the base scenario.

6.4.2 Traffic data

In this scenario, the left-turn, through and right-turn bicycle traffic flows in each arm were assumed to be the same. Six different levels of bicycle flow were used as listed in Table 6.10.

Table 6.10 Six levels of bicycle traffic demand

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Left-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Through bicycle flow (Bic/h)	0	200	400	600	800	1093
Right-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Total volume in each approach (Bic/h)	0	600	1200	1800	2400	3279
Total volume in the intersection (Bic/h)	0	2400	4800	7200	9600	13116

6.4.3 Signal setting

In this scenario, it was assumed that there were no signals for bicycles. A four-phase fixed signal control was employed, and the signal setting was the same as that adopted in the base scenario.

6.4.4 Results

When observing the simulation process, there were serious conflicts between motor vehicles and bicycles, and very serious congestion occurred. The traffic was delayed over a few hundred meters, and the intersection was in a state of near-paralysis when the bicycle traffic demand reached level 3.

The average vehicle journey time between different OD pairs are shown in Figure 6.12. It can be seen that the journey time increased dramatically after introducing bicycles.

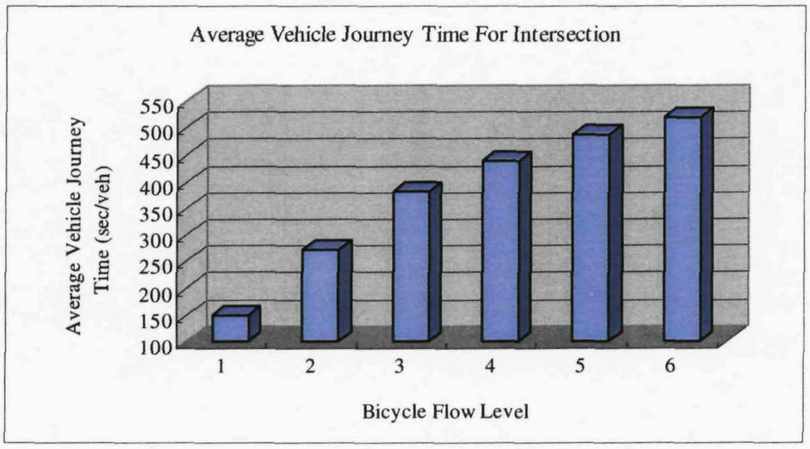


Figure 6.12 The average journey time of vehicles at different levels of bicycle flow

Because there were no signals for bicycle traffic, the conflicts between motor vehicles and bicycles were serious. Because the bicycles are more flexible and the speed of vehicles is often very slow within intersections, the bicycles always accept smaller gaps than motor vehicles. Furthermore, because the bicycles have a "group' characteristic, several bicycles can be served by a single gap. Once a bicycle occupies the collision point, some close-by bicycles will clear the collision point. Therefore, the number of discharged vehicles crossing the intersection dropped dramatically with increasing bicycle flow. Only one or two vehicles could cross the intersection during each green period, sometimes even lower. The vehicles started queuing before the stop lines, and the queue length kept increasing. When the bicycle flow reached level 5 or higher level, the vehicles queued over 500 meters, which was beyond the original point. The new generated vehicles had to wait to enter the system at the original point. The waiting time were counted in the journey time as a vertical queue. The average journey time above only counted in the vehicles that finished their journey within one hour. Because of the serious congestions, a lot of vehicles could not finish their journey and arrive the destination. The higher bicycle traffic flow, the more vehicle delay. Therefore, not all the vehicles, which generated within the first hour, are counted in when calculating the average journey time. The trend line of average vehicle journey time above underestimated the vehicle delay due to the high miss probability of the vehicles. In order to count all the delays in, a modification was made. The simulation period was extended to two hours, and all the vehicles that generated during the first hour are recorded. It was found that all vehicles generated during the first hour could finish their journey between original point and destination by the time simulation ended. Therefore, the average journey time can be calculated based on the same number of vehicles. The new data are shown in Figure 6.13.

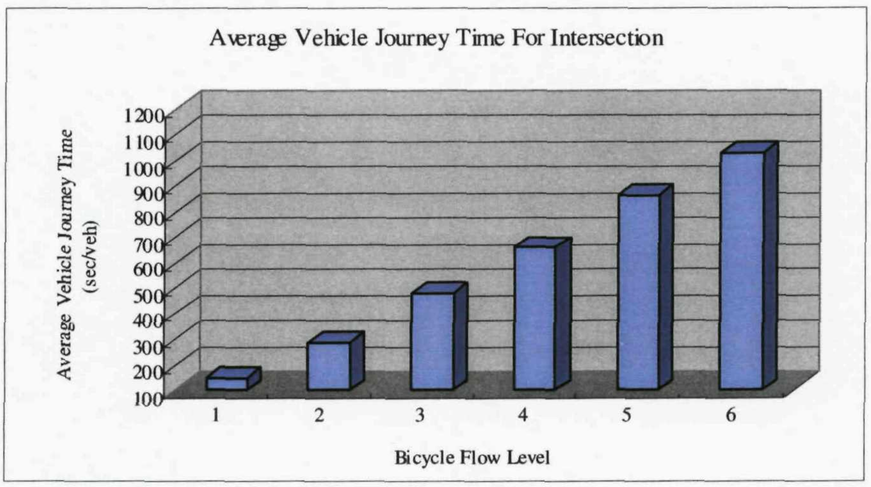


Figure 6.13 The average journey time of vehicles at different levels of bicycle flow

The average vehicle journey time calculated in modified method is obviously higher than that in method without modification.

The average vehicle delay between different OD pairs was calculated to check the effect of bicycles on vehicular traffic. The result is shown in Figure 6.14. It was clear that bicycles had a significant effect on motor vehicles. The influence increased with increasing bicycle flow.

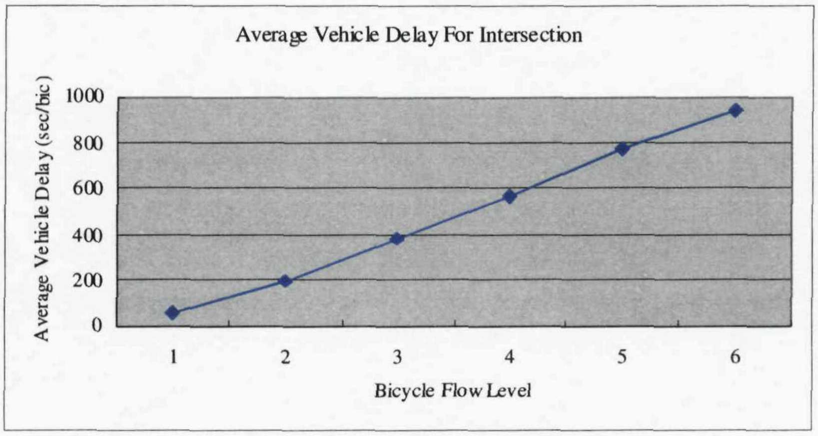


Figure 6.14 The average vehicle delay for intersection at different levels of bicycle flow

The number of motor vehicles crossing the intersection during simulation was recorded and calculated. As expected, a decreasing number of motor vehicles crossing the intersection were seen with the increasing bicycle flows. It is showed in Figure 6.15 that when bicycle flow increased, the vehicle delay increased, while the vehicle crossing volume decreased.

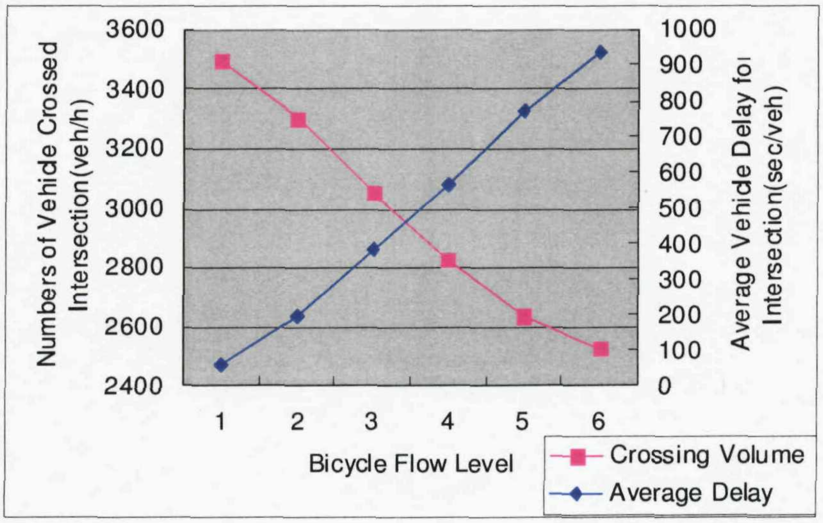


Figure 6.15 The relationship between average vehicle delay, number of vehicles crossing intersection and the bicycle flow

The ratio of decreased crossing volume of the intersection and the ratio of increased average vehicle delay at different levels of bicycle traffic were calculated and are given in Table 6.11. When the bicycle traffic demand was at level 1, i.e. 0 bic/h, the vehicle delay was 56.67 sec/veh. When the bicycle traffic demand was increased to level 2, the vehicle delay was 195.35 sec/veh, i.e. a 244.3% increased in vehicle delay and 5.4% decreased in crossing volume. The vehicle delay increased with the increasing bicycle flow. When the bicycle traffic demand reached level 6, the crossing volume decreased 27.8% and the average delay increased by 1551.2% compared with level 1.

Table 6.11 Ratio of increased average vehicle delay and decreased crossing volume

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Decreased vehicle cross volume	0	5.4%	12.7%	19.2%	24.7%	27.8%
Increased average vehicle delay	0	244.3%	571.9%	894.7%	1258.6%	1551.2%

The increase of vehicle delay and reduction of capacity is probably caused by the growing conflicts between motor vehicles and bicycles when the bicycle traffic demand was increasing. Observing the simulation process, there were serious conflicts between motor vehicles and bicycles, and serious congestions happened after introducing bicycles. Because there were no signals for bicycles, the conflicts between motor vehicles and bicycles were inevitable, which increased the delay of vehicular traffic. Accordingly the crossing capacity of motor vehicles decreased. The results showed that even low level of bicycles caused dramatic influence on motor vehicles without signals for bicycles, which will lead to serious congestions and capacity reduction.

The average bicycle delay was also recorded during the simulation process. The ratio of increased average bicycle delay at different levels of bicycle traffic demand were calculated and listed in Table 6.12.

Table 6.12 Ratio of increased average bicycle delay

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Increased delay	0	0	63.14%	161.57%	272.02%	403.13%

6.5 Scenario2: The influence of different proportion of bicycle flow with signals for bicycle lane (Bicycles use the same signal phase as motor vehicles of the same direction)

The results in scenario 1 show that even low level of bicycles cause dramatic influence on motor vehicles without signals for bicycles, which increases not only the delay sharply, but also the risk of accident. In reality, because the bicycle volumes are high at intersections in Beijing urban network, the signals for bicycles are necessary. There are several proposed methods dealing with bicycles to avoid the conflicts between motor vehicles and bicycles. Letting the left-turn bicycles share the same stage with left-turn motor vehicles is one of the common used methods in segregating motor vehicles and bicycles. In this scenario, it was assumed that there were signals for left-turn and through bicycles respectively. Bicycles used the same signal phase as motor vehicles of the same direction. The influence of different proportion of bicycle flow on traffic was studied based on the scenario, where the method of bicycles using the same signal phase as motor vehicles of the same direction was applied.

6.5.1 Geometry description

The geometry was the same as the base scenario.

6.5.2 Traffic data

In this scenario, the left-turn, through and right-turn bicycle traffic flows were assumed to be the same. Six different levels of bicycle flow are listed in Table 6.13.

Table 6.13 Six levels of bicycle traffic demand

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Left-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Through bicycle flow (Bic/h)	0	200	400	600	800	1093
Right-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Total volume in each approach (Bic/h)	0	600	1200	1800	2400	3279
Total volume in the intersection (Bic/h)	0	2400	4800	7200	9600	13116

6.5.3 Signal setting

In this scenario, four-phase fixed signal control was employed, and the signal setting for vehicles was the same as what was adopted in the base scenario. It was assumed that there were signals for left-turn and through bicycles respectively. Bicycles used the same signal phase as motor vehicles of the same direction.

The detailed signal phase and stage arrangements are given in Figure 6.16 and Figure 6.17.

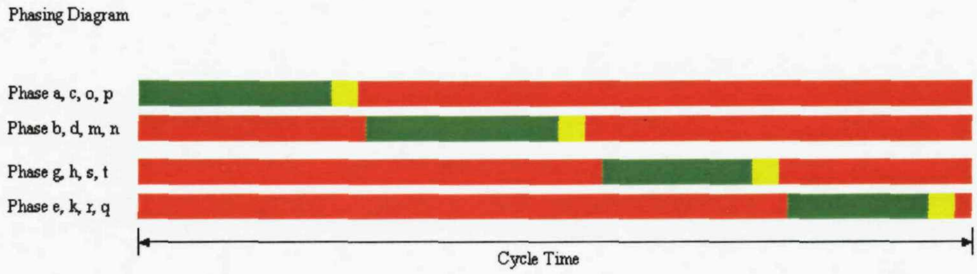


Figure 6.16 Signal phase arrangement

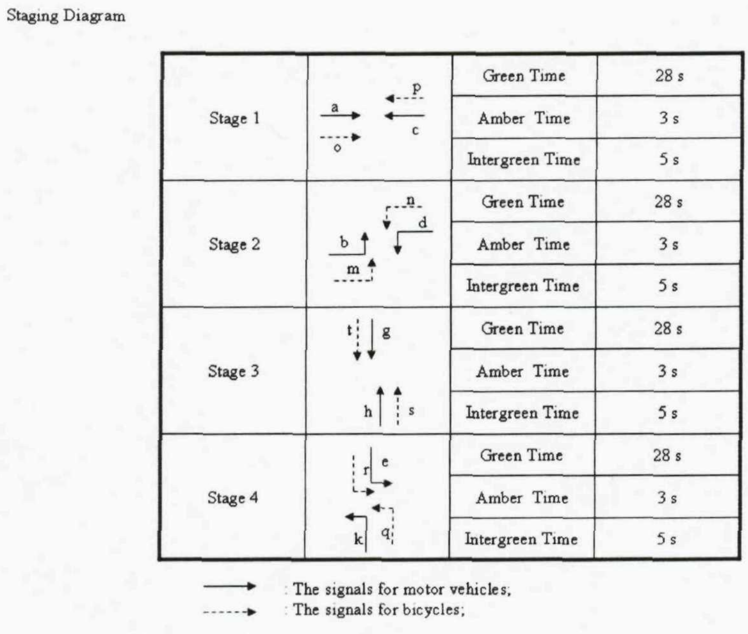


Figure 6.17 Signal stage arrangement

6.5.4 Results

The results presented in Figure 6.18, 6.19, 6.20 and 6.21 illustrate the influence of different levels of bicycle traffic demand on traffic of the intersection.

The average vehicle journey time between different OD pairs is shown in Figure 6.18. It can be seen that the increasing bicycle volumes caused little influence in average vehicle delay and journey time.

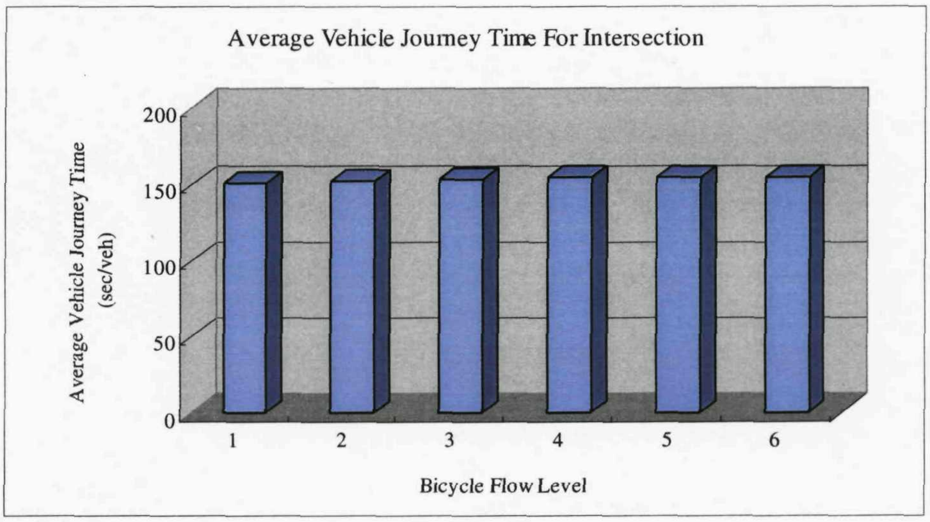


Figure 6.18 The average journey time of vehicles at different levels of bicycle flow

The average vehicle delays of left-turn, through and right-turn vehicles at different levels of bicycle traffic demand are shown in Figure 6.19. It can be seen that the journey time of right-turn vehicles was obviously shorter than that of left-turn and through vehicles. The increasing bicycle volumes caused slight increase in right-turn vehicle delay, but little influence on left-turn and through vehicles. The average delay of left-turn and through vehicles shows signs of levelling off. This is because that the possible conflicts exist only between right-turn vehicles and bicycles under existing signal settings. The increasing bicycle flows led to the increase of the conflicts between right-turn motor vehicles and bicycles.

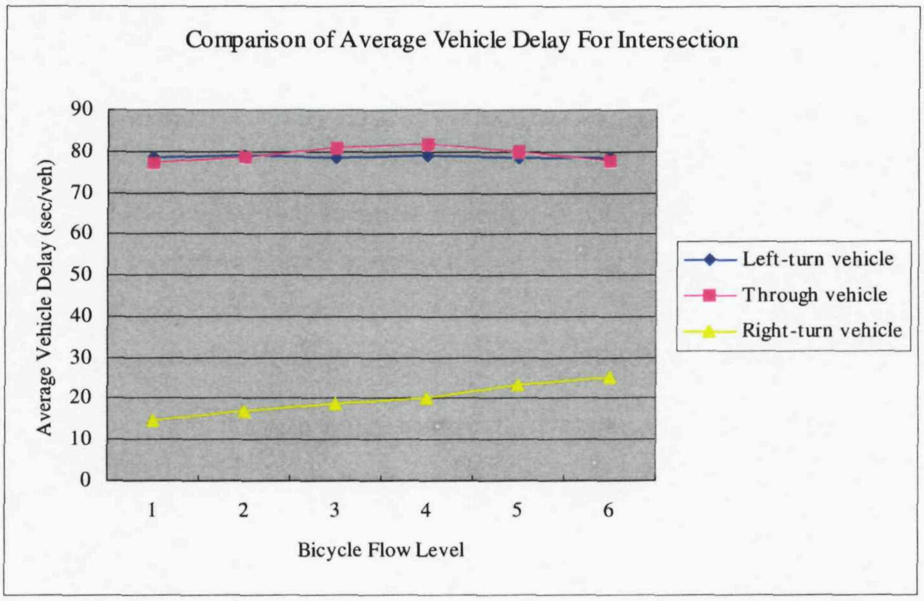


Figure 6.19 The average delay of left-turn, through and right-turn vehicles at different levels of bicycle flow

The average bicycle delay was also recorded during the simulation process. The result is shown in Figure 6.20.

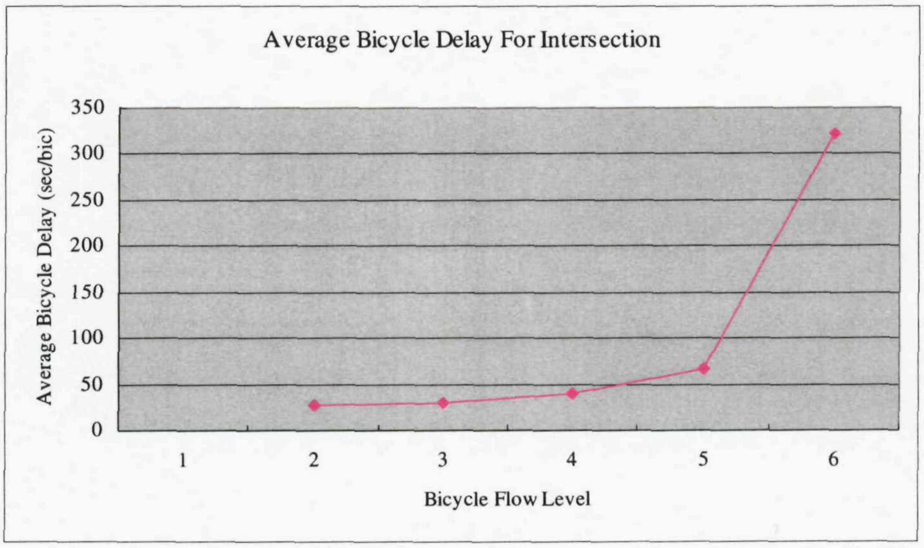


Figure 6.20 The average bicycle delay for intersection at different levels of bicycle flow

The ratios of increased average bicycle delay at different levels of bicycle traffic demand were calculated and listed in Table 6.14.

Table 6.14 Ratio of increased average bicycle delay

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Increased average bicycle delay	0	0	9.6%	41.7%	142.8%	1101.8%

It is clear that the change of bicycle traffic demand had little effect on the vehicular traffic. While the average bicycle delay increased with increasing bicycle flow. When the bicycle traffic demand was at level 2, the bicycle delay was 26.73 sec/bic. When the bicycle traffic demand was increased to level 3, the bicycle delay was 29.31 sec/bic, i.e. a 9.6% increase in vehicle delay. The bicycle delay increased with the increasing bicycle flow. When the bicycle volumes reached level 6, the average delay increased 1101.8% compared with level 2, was 321.24 sec/bic.

It can be seen from Figure 6.20 and Table 6.14 that when the bicycle volumes were lower than level 5, the average bicycle delay increased slowly with the increasing bicycle traffic demand. While the bicycle traffic demand increased to level 5 and higher, the bicycles experienced a significant increase in average delay. This was caused by the interactions among bicycles. When the bicycle volumes were higher than level 5, during the green period of left-turn bicycles, the through bicycles had to wait before the stop line, and some left-turn bicycles blocked by these through bicycles and had to wait for the next green period. The through bicycles were blocked by the left-turn bicycles in the same way.

A comparison between the scenario 1 and scenario 2 was conducted. In order to compare the effect on motor vehicles under the same conditions, the average vehicle delay at six levels of bicycle traffic demand were selected. It can be seen in Figure 6.21 that the average vehicle delays in scenario 1 were obviously higher than that in scenario 2.

Results show that adoption of bicycle signals using the same signal phase as motor vehicles of the same direction were helpful in reducing the conflicts between right-turn and through motor vehicles and bicycles and in lessening the impact caused by bicycles on vehicles. However, on one hand, such signal setting limited the crossing capacity of bicycles and caused higher bicycle delay when the bicycle traffic demands were high. On the other hand, some possible conflicts still existed between right-turn vehicles and bicycles, which would be unsafe for both motor vehicles and bicycles.

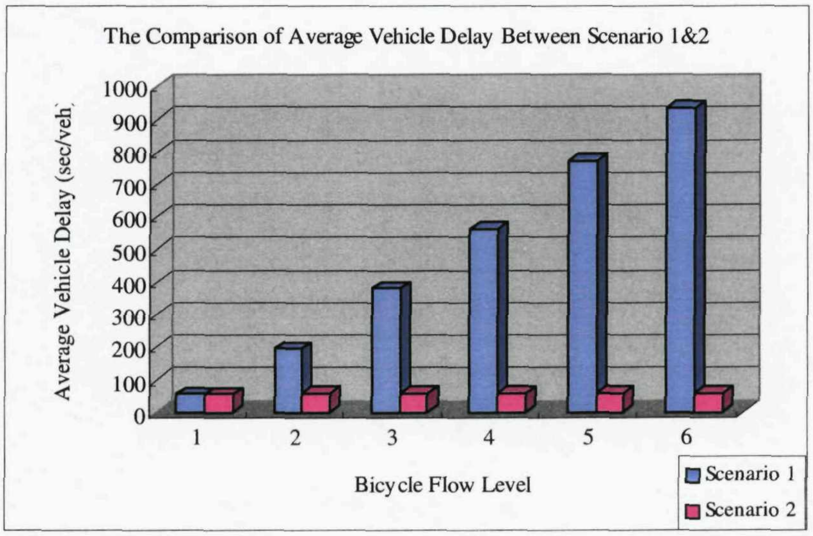


Figure 6.21 The comparison of average vehicle delay between scenario 1 and 2

6.6 Scenario 3: The influence of different proportion of bicycle flow with signals for bicycle lane (a second stop line for bicycles)

Second stop line for bicycles is another method to segregate motor vehicles and bicycles. Bicycles are not allowed to occupy and cross the confined area located at the centre of the intersection. All bicycles are required to travel outside the banned area while motor vehicles maintain their original route in using the banned area. Such measure can limit the left-turn bicycles, and reduce the conflicts between motor vehicles and left-turn bicycles accordingly. In this scenario, the signals for right-turn vehicle lane were set, and bicycles were banned from making left turn. The left-turn bicycles finish left-turn movement by two through movements, as shown in Figure 6.22.

6.6.1 Geometry description

The geometry is shown in Figure 6.22.

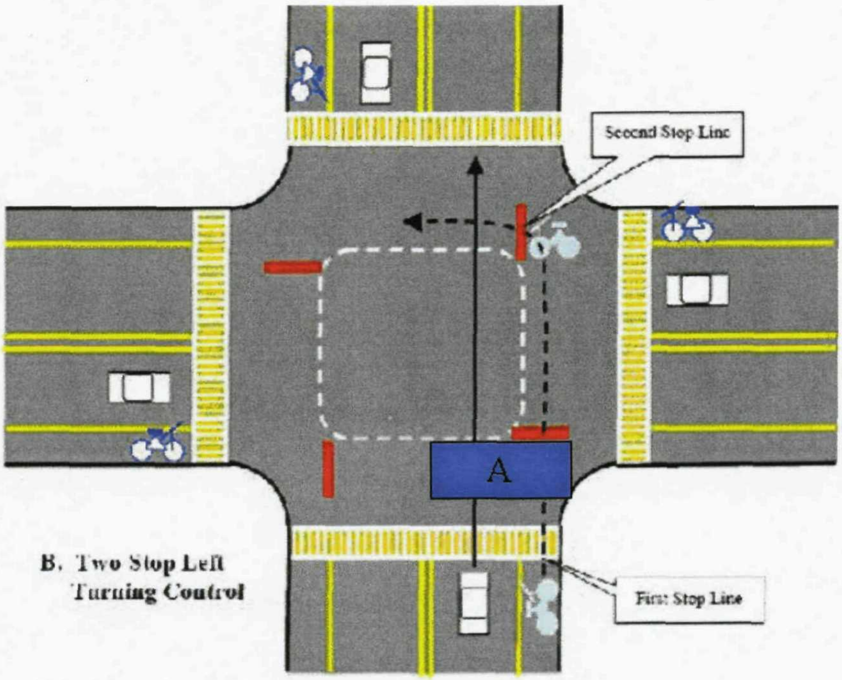


Figure 6.22 Geometry of scenario 3

In reality, the space of the intersection is one of the important issues, which will affect the decision of whether this method can be adopted. If there is no enough storage space before the second stop line, some bicycles may stop outside the storage space and block the movement of motor vehicles, which causes unpredictable delay and congestion.

The maximum number of bicycles that can be accommodated inside the storage space before the second stop line can be calculated according to the function below:

$$N_w = \rho_b * S_s \tag{6.5}$$

where,

N_w is maximum number of bicycles that can be accommodated inside the storage space before the second stop line, the unit is bic;

ρ_b is the density of bicycles, the unit is bic/m^2;

S_s is the size of storage space;

The storage space before the second stop line (Area A shown in Figure 6.22) depends on the size of intersection. In this case, the area of storage space before the second stop line S_s is about 67.5 m^2. According to previous research (Chapter 4.2.4), the maximum waiting group density was 0.67 bic/m^2 and the mean waiting group density was 0.46 bic/m^2, the maximum waiting group density 0.67 bic/m^2 was adopted as ρ_b in calculating the maximum number of bicycles that can stop before the second stop line. Therefore, the maximum number of bicycles that can stop before the second stop line in this case was around 46 bic. After observing the simulation process, it was found that even when the bicycle volumes reach level 6, the parking space in this case was still enough to accommodate the bicycles stopped before the second stop line.

6.6.2 Traffic data

In this scenario, the left-turn, through and right-turn bicycle flows in different directions were assumed to be the same. Six different levels of bicycle flow are listed in Table 6.15.

Table 6.15 Six levels of bicycle traffic demand

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Left-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Through bicycle flow (Bic/h)	0	200	400	600	800	1093
Right-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Total volume in each approach (Bic/h)	0	600	1200	1800	2400	3279
Total volume in the intersection (Bic/h)	0	2400	4800	7200	9600	13116

6.6.3 Signal setting

In this scenario, the signals for right-turn vehicle lane were set. Four-phase fixed signal control was employed, and the signal setting for vehicles was the same as what was adopted in the base scenario. It was assumed that there were signals at the second stop line for left-turn bicycles respectively. Bicycles used the same signal phase as through vehicles of the same direction.

The detailed signal phase and stage arrangement are shown in Figure 6.23 and Figure 6.24.

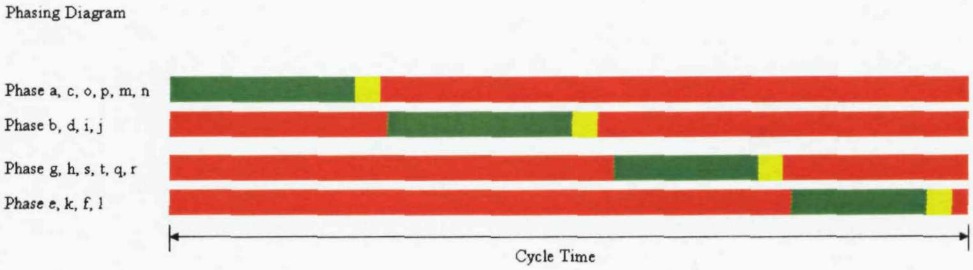


Figure 6.23 Signal phase arrangement

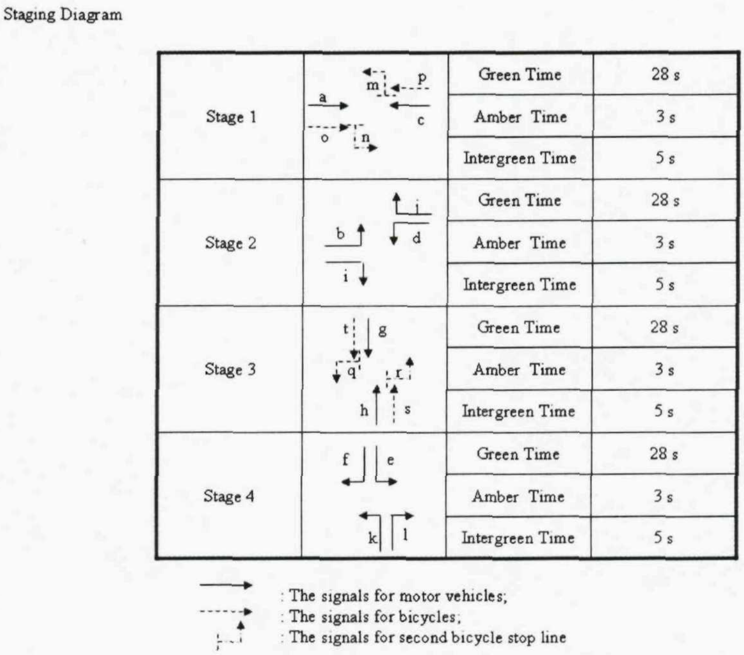


Figure 6.24 Signal stage arrangement

6.6.4 Results

The results are presented in Figure 6.25, 6.26 and 6.27 to illustrate the influence of different levels of bicycle flow on traffic.

The average vehicle journey time between different OD pairs are shown in Figure 6.25. It can be seen that the increasing bicycle volumes caused little influence on average vehicle delay and journey time.

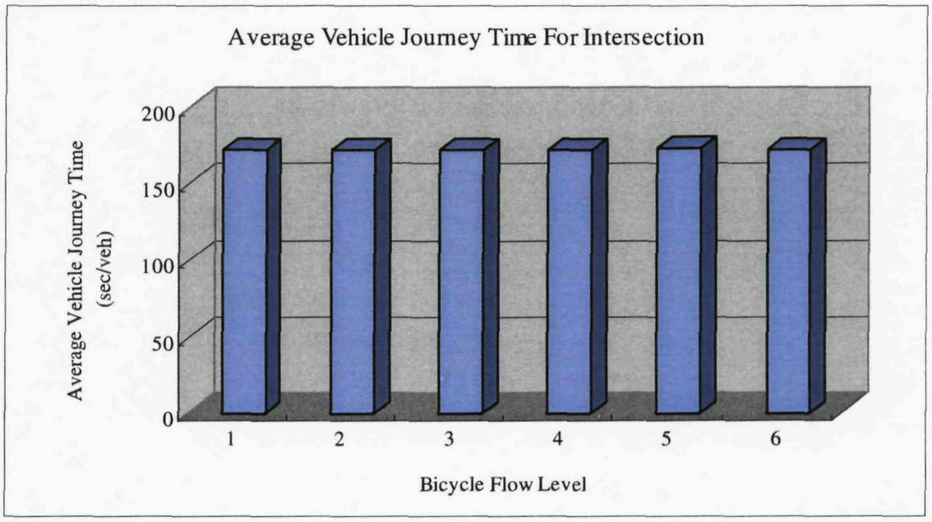


Figure 6.25 The average journey time of vehicles at different levels of bicycle flow

The average vehicle delay was calculated to check the impact of bicycles on the whole intersection. The result is shown in Figure 6.26. It is clear that the bicycles caused little effect on motor vehicles.

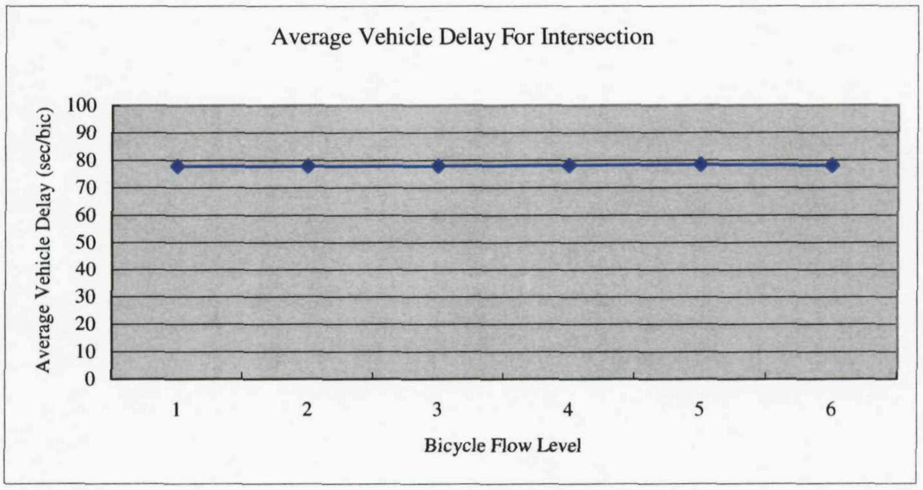


Figure 6.26 The average vehicle delay for intersection at different levels of bicycle flow

The average bicycle delay was also recorded during simulation process. The result is shown in Figure 6.27. It is clear that the average bicycle delay increased slowly with the increasing bicycle flow.

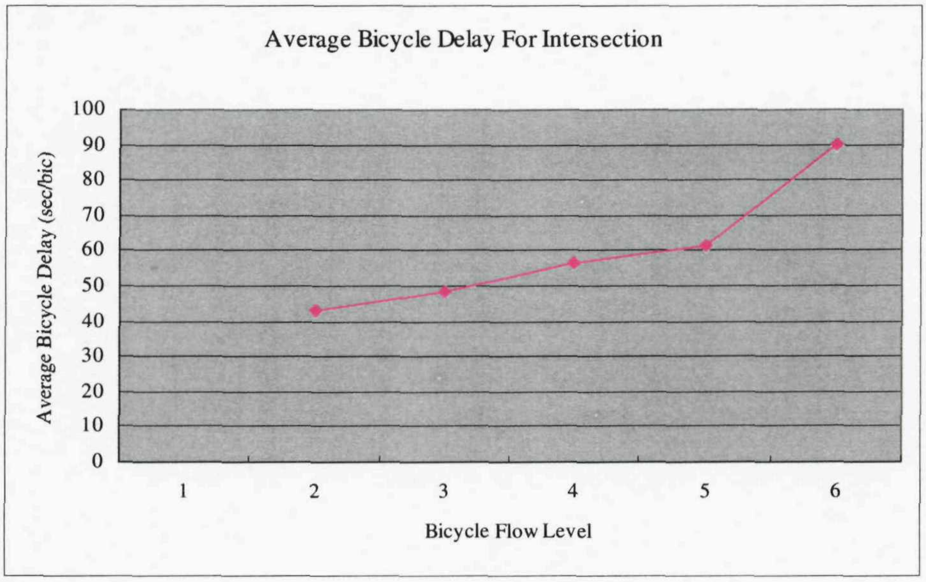


Figure 6.27 The average bicycle delay for intersection at different levels of bicycle flow

The ratios of increased average bicycle delay at different levels of bicycle traffic demand are calculated and listed in Table 6.16.

Table 6.16 Ratios of increased average bicycle delay

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Increased average bicycle delay	0	0	11.5%	30.7%	42.1%	107.5%

It is clear that changes in bicycle volume have little effect on both the vehicle delay and the bicycle delay. When the bicycle traffic demand was at level 2, the bicycle delay was 43.27 sec/bic. When the bicycle traffic demand was increased to level 3, the bicycle delay was 48.25 sec/bic, i.e. an 11.5% increase in vehicle delay. The bicycle delay increased with the increasing bicycle flow. When the bicycle volumes reached level 6, the average delay increased 107.5% compared with level 2, was 89.79 sec/bic.

A comparison between the scenario 2 and scenario 3 was conducted below. From the result shown in Figure 6.28, the average vehicle delay increased slightly in scenario 3 as compared with scenario 2.

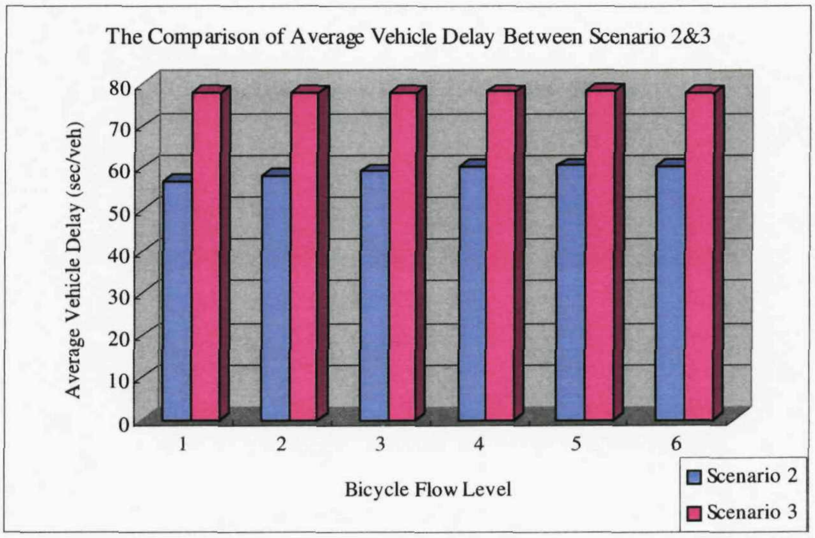


Figure 6.28 The comparison of average vehicle delay between scenario 2 and scenario 3

The comparison of average bicycle delay between scenario 2 and scenario 3 is shown in Figure 6.29. It can be seen that when the bicycle volumes was lower than level 5, the average bicycle delay in scenario 2 was lower than that in scenario 3. When the bicycle volumes increased higher than level 5, a dramatic increase in the average bicycle delay can be observed in scenario 2 and the value was obviously higher than that in scenario 3.

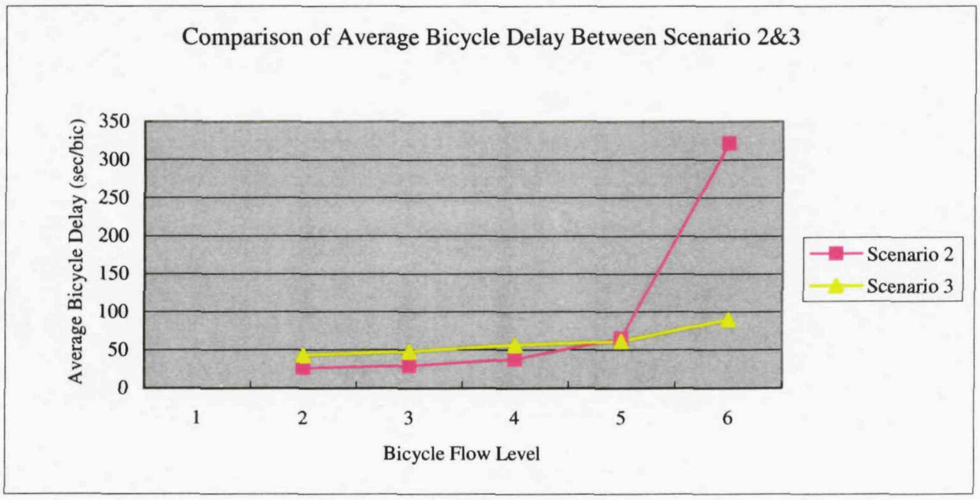


Figure 6.29 The comparison of average bicycle delay between scenario 2 and scenario 3

The comparison between scenario 2 and scenario3 indicated that the second stop line for bicycles could avoid the conflicts between motor vehicles and bicycles completely and have least influence on both motor vehicles and bicycles, giving there was enough space for bicycles waiting before the second stop line, especially when the bicycle volumes were high.

6.7 Scenario 4: The influence of different proportion of bicycle flow with exclusive bicycle signal

Exclusive bicycle signal was proposed as another approach in separating motor vehicles and bicycles in at-grade intersections. Bicycle green phase allows bicycles their own time to cross an intersection in the signal cycle time. In this scenario, the influence of an exclusive bicycle signal was studied.

6.7.1 Geometry description

The geometry is the same as the base scenario.

6.7.2 Traffic data

In this scenario, the left-turn, through and right-turn bicycles flows in different directions were assumed to be the same. Six different levels of bicycle flow are listed in Table 6.17.

Table 6.17 Six levels of bicycle traffic demand

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Left-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Through bicycle flow (Bic/h)	0	200	400	600	800	1093
Right-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Total volume in each approach (Bic/h)	0	600	1200	1800	2400	3279
Total volume in the intersection (Bic/h)	0	2400	4800	7200	9600	13116

6.7.3 Signal setting

In this scenario, four-phase fixed signal control was employed, and the signal setting for vehicles was the same as what was adopted in the base scenario. 20 seconds exclusive green time was assigned for bicycles. Bicycles can only cross the intersection during the exclusive bicycle phase.

The detailed signal phase and stage arrangements are shown in Figure 6.30 and Figure 6.31.

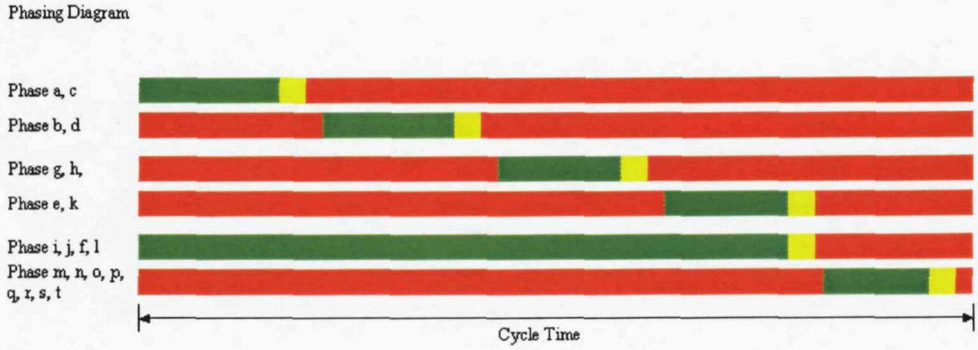
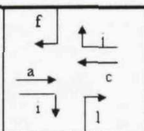
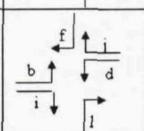
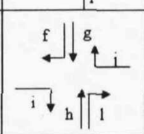
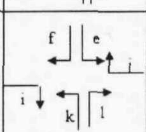
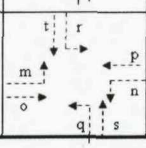


Figure 6.30 Signal phase arrangement

Staging Diagram

Stage 1		Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 2		Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 3		Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 4		Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 5		Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s

——→ : The signals for motor vehicles;
- - - -→ : The signals for bicycles;

Figure 6.31 Signal stage arrangement

6.7.4 Results

The results presented in Figure 6.32 and 6.33 illustrate the influence of different level of bicycle flows on traffic.

The average journey time between different OD pairs are shown in Figure 6.32. It can be seen that increasing bicycle volumes causes little influence on average vehicle delay and journey time.

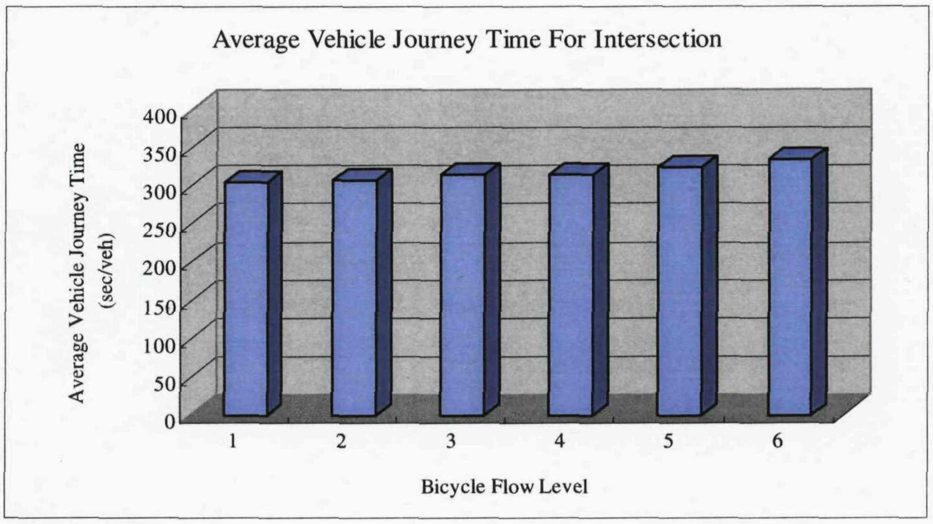


Figure 6.32 The average journey time of vehicles at different levels of bicycle flow

The average bicycle delay was also recorded during the simulation process. The result is shown in Figure 6.33. It was clear that the average bicycle delay increased slowly with the increasing bicycle flow when the bicycle volumes were below level 5, and then there was a sharp increase in bicycle delay when the bicycle traffic demands were higher than level 5. This was because some bicycles cannot cross the stop line within one bicycle phase and have to wait for the next bicycle phase when the bicycle volumes were high.

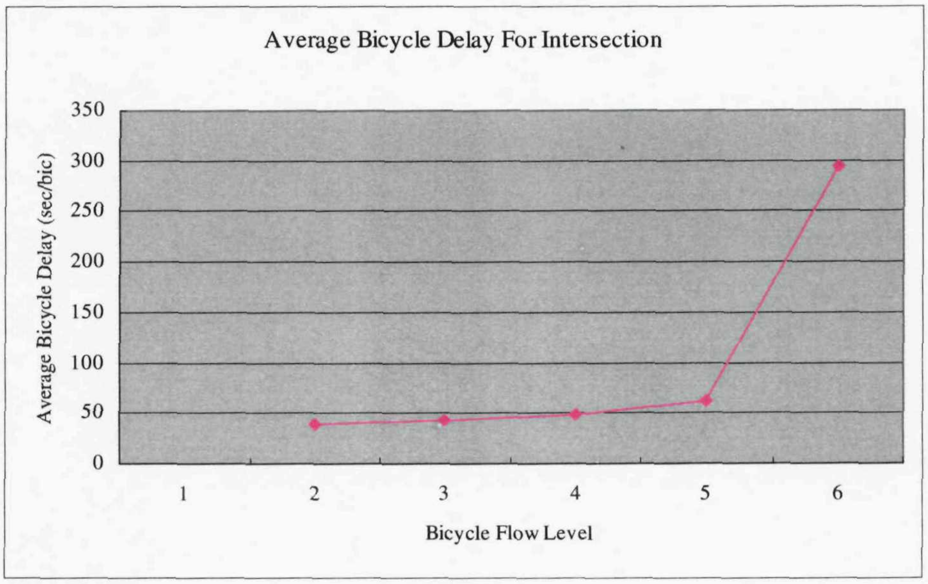


Figure 6.33 The average bicycle delay for intersection at different levels of bicycle flow

The ratios of increased average bicycle delay at different levels of bicycle traffic demands are calculated and listed in Table 6.18.

Table 6.18 Ratios of increased average bicycle delay

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Increased delay	0	0	8.2%	20.3%	56.2%	635.2%

It is clear that changes in bicycle volume had little effect on both the vehicle delay and the bicycle delay when the bicycle volumes were below level 5. When the bicycle traffic demand was at level 2, the bicycle delay was 40.07 sec/bic. When the bicycle traffic demand was increased to level 3, the bicycle delay was 43.35 sec/bic, i.e. an 8.2% increase in vehicle delay. The bicycle delay increased with the increasing bicycle flow. When the bicycle volumes reached level 6, the bicycles experienced a significant increase on the average delay, which was 294.6138 sec/bic increased 635.2% compared with level 2.

A comparison between scenario 3 and scenario 4 was conducted and is shown in Figure 6.34. The average vehicle delay increased a lot in scenario 4 as compared with in scenario 3. This is because the signal cycle time increased with an additional bicycles phase. The average vehicle delay increased consequentially.

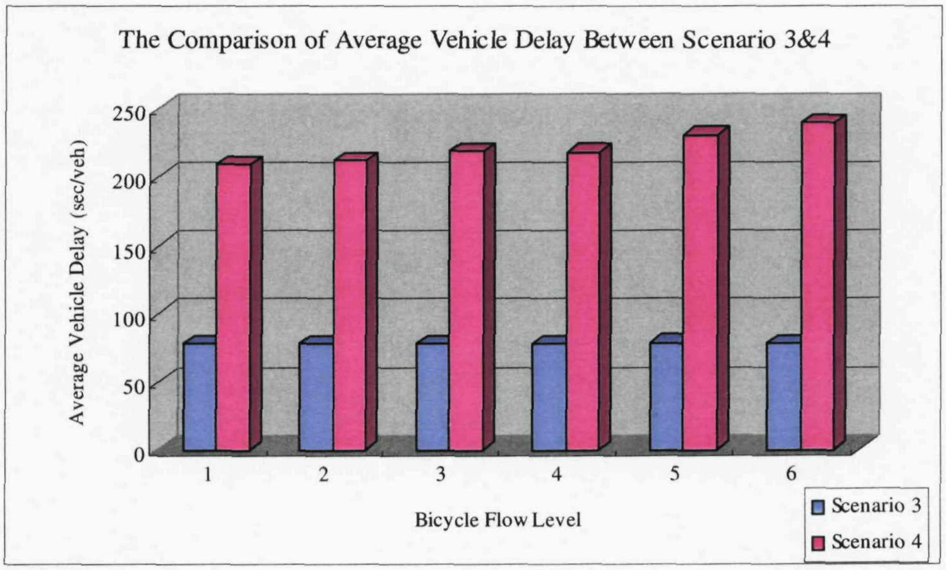


Figure 6.34 The comparison of average vehicle delay between scenario 3 and scenario 4

The comparison of average bicycle delay between scenario 3 and scenario 4 is shown in Figure 6.35. It can be seen that when the bicycle volumes was lower than level 5, the average bicycle delay in scenario 4 was slightly lower than that in scenario 3. When the bicycle volumes increased higher than level 5, the average bicycle delay increased dramatically and was significantly higher than that in scenario 3.

Both the second stop line for bicycles and exclusive bicycle phase can avoid the conflicts between motor vehicles and bicycles completely. It is therefore safer for traffic. In scenario 4 (exclusive bicycle phase), the ignoring of conflicts among bicycles results in underestimating the negative influence caused by bicycles. Because during the exclusive bicycle phase, bicycles in different directions interact inside the intersection, which lead to disorder to some extent and causes some bicycles to remain inside the intersection when bicycle phase finished. While scenario 3 (second stop line for bicycles) requires large space to accommodate the stopped left-turn bicycles. The comparison between scenario 3 and scenario 4 indicated that the second stop line for bicycles caused least influence on both motor vehicles and bicycles given there was enough space for bicycles waiting before the second stop line, especially when the bicycle volumes were high. Therefore if the bicycle volumes are low and the intersection is small, an exclusive bicycle phase is a reasonable option, otherwise a second stop line for bicycles could be a good alternative.

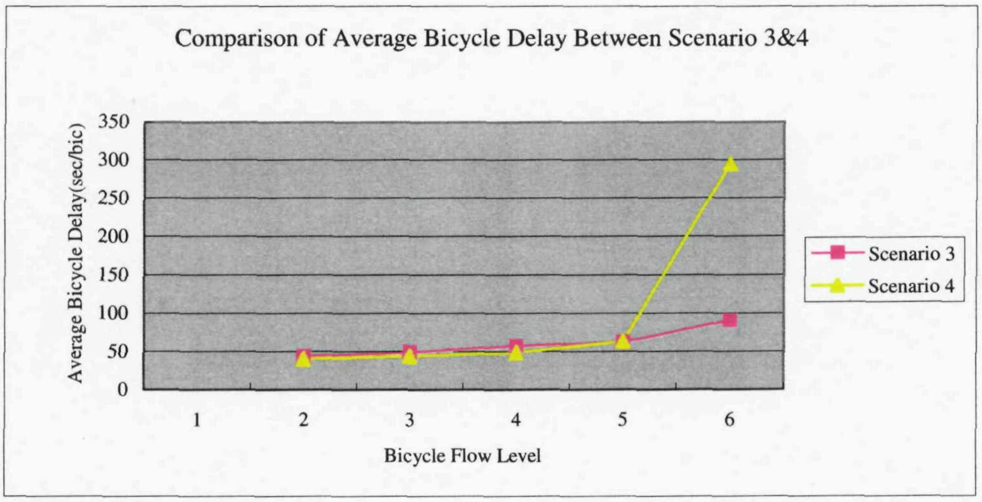


Figure 6.35 The comparison of average bicycle delay between scenario 3 and scenario 4

6.8 Scenario 5: The influence of different proportion of bicycle flow with signals for bicycle lane (a separate lane was built near the intersection to prohibit left-turn bicycles)

In this scenario, another left-turn prohibition approach was studied. A separate lane was built near the intersection. Through the lane, the left-turn bicycles change their route to a right-turn path. The method can also avoid the conflicts between through motor vehicles and left- turn bicycles.

6.8.1 Geometry description

The geometry is shown in Figure 6.36.

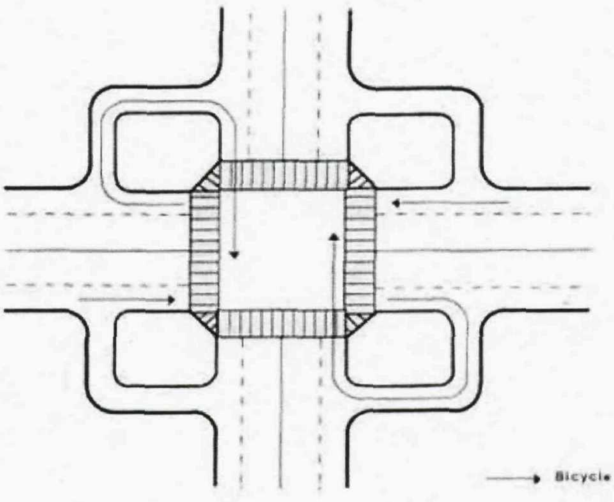


Figure 6.36 Geometry of scenario 5

6.8.2 Traffic data

In this scenario, the left-turn, through and right-turn bicycle traffic flows in different directions were assumed to be the same. Six different levels of bicycle flow are listed in Table 6.19.

Table 6.19 Six levels of bicycle traffic demand

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Left-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Through bicycle flow (Bic/h)	0	200	400	600	800	1093
Right-turn bicycle flow (Bic/h)	0	200	400	600	800	1093
Total volume in each approach (Bic/h)	0	600	1200	1800	2400	3279
Total volume in the intersection (Bic/h)	0	2400	4800	7200	9600	13116

6.8.3 Signal setting

In this scenario, the signals for right-turn vehicle lane were set. Four-phase fixed signal control was employed, and the signal setting for vehicles was the same as what was adopted in the base scenario. Bicycles used the same signal phase as through vehicles of the same direction.

The detailed signal phase and stage arrangements are shown in Figure 6.37 and Figure 6.38.

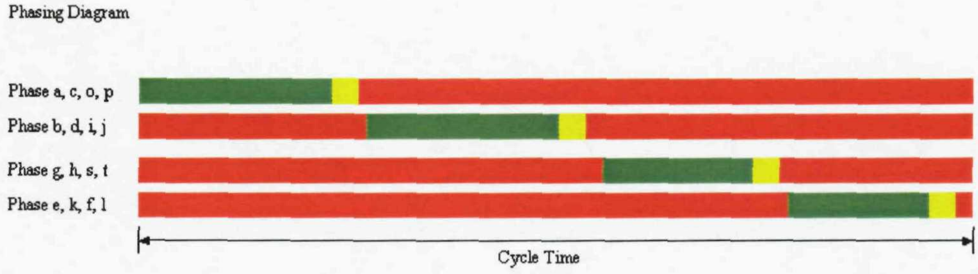


Figure 6.37 Signal phase arrangement

Staging Diagram

Stage 1	a, o, p, c	Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 2	b, i, j, d	Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 3	t, g, h, s	Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s
Stage 4	f, e, k, l	Green Time	28 s
		Amber Time	3 s
		Intergreen Time	5 s

——→ : The signals for motor vehicles;
-----→ : The signals for bicycles;

Figure 6.38 Signal stage arrangement

6.8.4 Results

The results are presented in Figure 6.39 and 6.40 to illustrate the impact of different levels of bicycle flow on vehicles and bicycles.

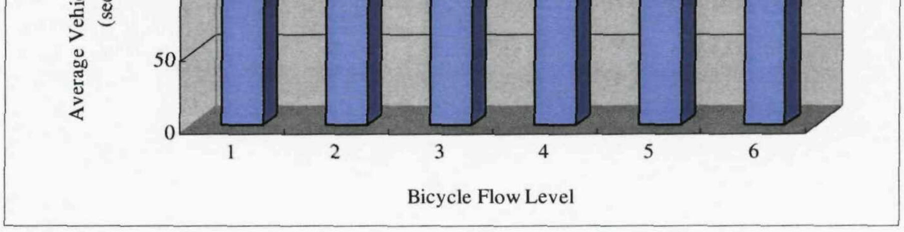


Figure 6.39 The average journey time of vehicles at different levels of bicycle flow

The average vehicle journey time between different OD pairs are shown in Figure 6.39. It can be seen that the increasing bicycle traffic demands caused little influence on average vehicle delay and journey time.

The average bicycle delay was also recorded during simulation process. The result is shown in Figure 6.40. It was clear that the average bicycle delay increased slowly with the increase of the bicycle flow when bicycles volumes were lower than level 5. There was a sharp increase for bicycle delay when the bicycle volumes were higher than level 5. This was because some bicycles can't cross the stop line within one bicycle phase and had to wait for the next bicycle phase when the bicycle volumes were high.

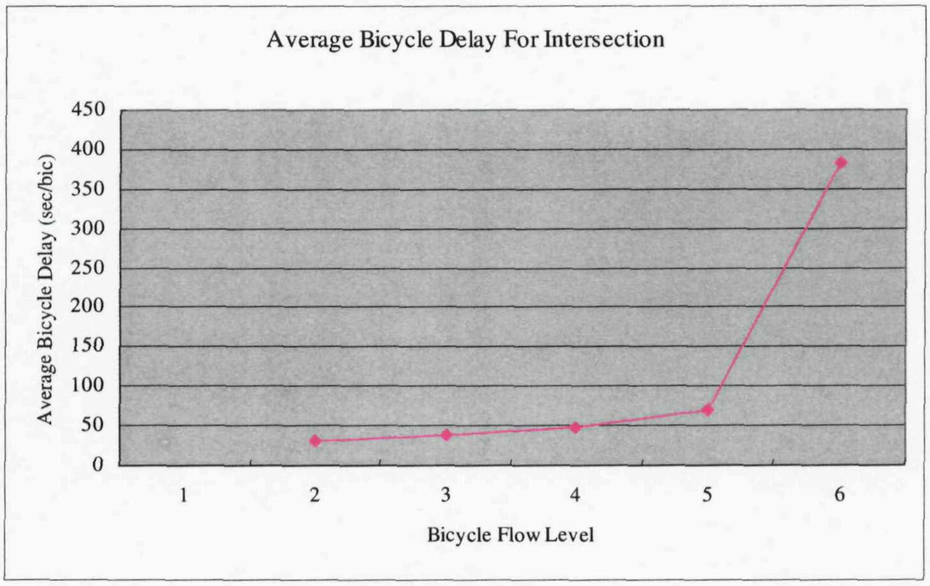


Figure 6.40 The average bicycle delay for intersection at different levels of bicycle flow

The ratios of increased average bicycle delay at different levels of bicycle traffic flow are calculated and listed in Table 6.20.

Table 6.20 Ratios of increased average bicycle delay

	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6
Increased delay	0	0	24.9%	53.6%	126.3%	1139.2%

When the bicycle traffic demand was at level 2, the bicycle delay was 30.82 sec/bic. When the bicycle traffic demand was increased to level 3, the bicycle delay was 38.50 sec/bic, i.e. a

24.9% increase in vehicle delay. The bicycle delay increased with the increasing bicycle flow. When the bicycle volumes reached level 6, the average delay increased 1139.2% compared with level 2, was 381.90 sec/bic.

A comparison of scenario 3, scenario 4 and scenario 5 is conducted and shown in Figure 6.41, it can be seen that the average vehicle delay in scenario 3 and scenario 5 was on the same level, and the average vehicle delay increased a lot in scenario 4 as compared with in scenario 3 and 5.

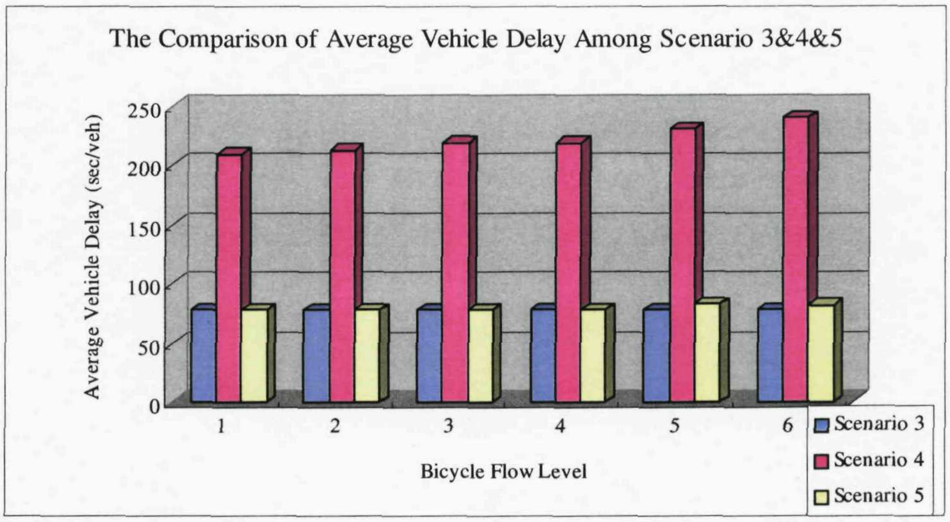


Figure 6.41 The comparison of average vehicle delay among scenario 3, 4 and 5

The comparison of average bicycle delay among scenario 3, scenario 4 and scenario 5 is shown in Figure 6.42. It can be seen that when the bicycle volumes was lower than level 5, there is no much difference among scenario 3, 4 and 5. At level 6, the average bicycle journey time in scenario 5 was obviously higher than that in scenario 3 and 4. When the bicycle volumes increased higher than level 6, the average bicycle delay in scenario 5 increased about 325.33% as compared with that in scenario 3, and the average bicycle delay in scenario 4 increased about 228.11% as compared with that in scenario 3.

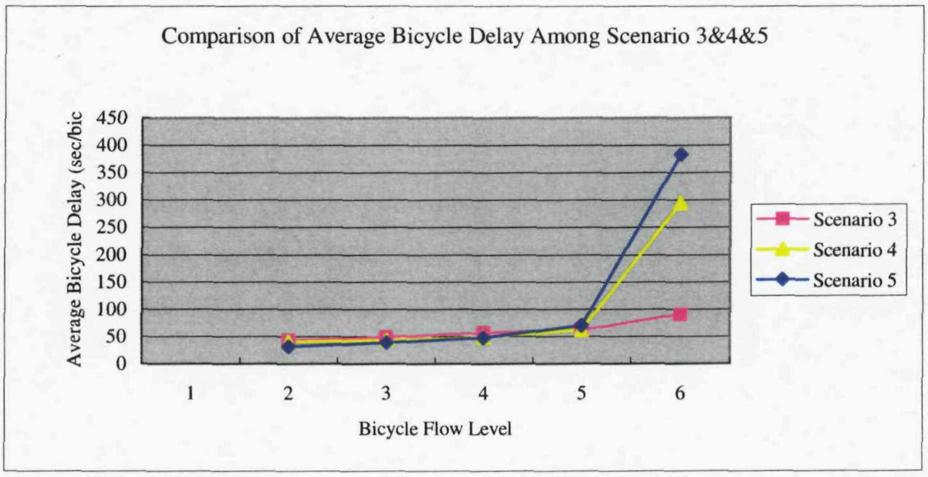


Figure 6.42 The comparison of average bicycle delay among scenario 3, 4 and 5

When the bicycle volumes were lower than level 5, the influence on both vehicle traffic and bicycle traffic caused by bicycles in scenario 5 is the same as that in scenario 3, however, the average bicycle journey time increased dramatically when the bicycle volumes reach level 6. There is no need to consider the space of the intersection in the approach that prohibits the left-turn bicycles by building a separate lane, however, the cyclists may not accept this approach if the separate lane is too long or too far from the intersection, because this control would increase the travel distance of left-turn cyclists definitely, and increase the average journey time of left-turn bicycles consequently. So, the bicycle and vehicle volumes, the space of the intersection and the location of the separate lane should be taken into consideration when designing the control strategy for intersection of mixed traffic.

6.9 Comparison between scenario 2&3&4&5

The results in scenario 1 shows that even low level of bicycles caused dramatic influence on motor vehicles without signals for bicycles, which increased not only the delay sharply, but also the risk of accident. In reality, because the bicycle volumes are high at intersections in the Beijing urban network, signals for bicycles are absolutely necessary.

There are several operations, which were applied and simulated in scenario 2, 3, 4, and 5, dealing with bicycles to avoid the conflicts between motor vehicles and bicycles. Some analyses were made based on the simulate results.

The comparison of average vehicle delay among these four scenarios under six levels of bicycle traffic demand is shown in Figure 6.43. The value in scenario 2 is the lowest, and the value in scenario 4 is obviously higher than that in other three scenarios. The average vehicle delay in scenario 2 and 5 are at the same level.

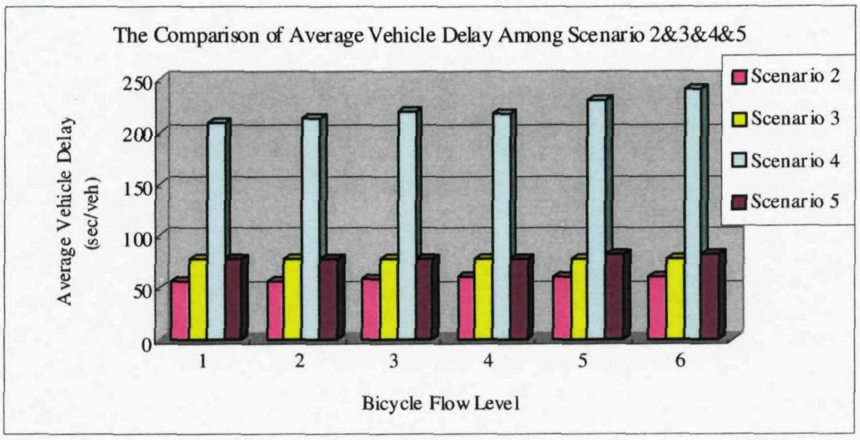


Figure 6.43 The comparison of average vehicle delay among scenario 2, 3, 4 and 5

The number of vehicles crossing the intersection is another factor, which can reflect the vehicular capacity of the intersection. The comparison of vehicle crossing volume of the intersection among these four scenarios under six levels of bicycle traffic demand is shown in Figure 6.44. The figure shows that the vehicular crossing volume of intersection in scenario 2, 3 and 5 are at the same level. The vehicle crossing volume of the intersection in scenario 4 is lower than the other three scenarios.

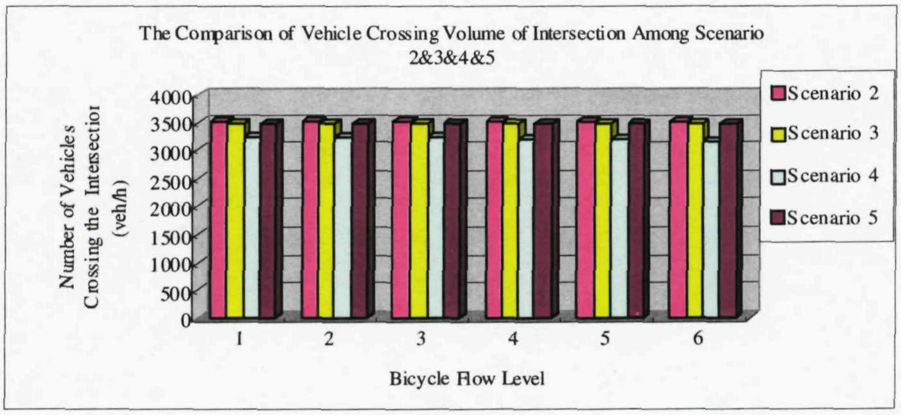


Figure 6.44 The comparison of vehicle crossing volume of the intersection among scenario 2, 3, 4 and 5

From Figure 6.43 and 6.44, it is clear that scenario 4 causes more influence on vehicular traffic than other three scenarios. This is because in scenario 4, the signal cycle time is extended with an exclusive bicycle phase.

The comparison of average bicycle delay among these four scenarios under six levels of bicycle traffic demand is shown in Figure 6.45. When the bicycle volumes are lower than level 5, the influence on bicycle traffic in these four scenarios is almost at the same level. However, the average bicycle delay increased dramatically when the bicycle volumes reach level 6. At level 6, the average bicycle delay in scenario 2, 4 and 5 was obviously higher than that in scenario 3. When the bicycle volumes increased higher than level 6, the average bicycle delay in scenario 5 increased about 325.3% as compared with that in scenario 3, the average bicycle delay in scenario 2 increased about 257.8% as compared with that in scenario 3, and the average bicycle delay in scenario 4 increased about 228.1% as compared with that in scenario 3.

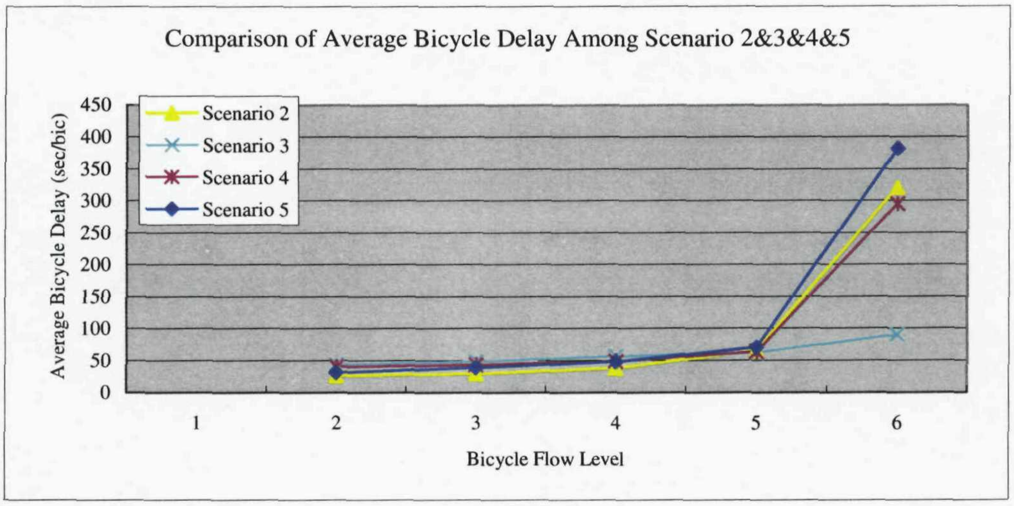


Figure 6.45 The comparison of average bicycle delay among scenario 2, 3, 4 and 5

The number of bicycles crossing the intersection is another factor. It can reflect the bicycle capacity of the intersection. The comparison of bicycle crossing volume of intersection among these four scenarios under six levels of bicycle traffic demand is shown in Figure 6.46. It shows that the bicycle crossing volumes of intersection in scenario 2, 3, 4 and 5 are at the same level when the bicycle traffic is lower than level 5. When the bicycle volumes increased to level 6, the bicycle crossing volume of intersection in scenario 5 is the lowest one, and the bicycle crossing volume of the intersection in scenario 3 is the highest one.

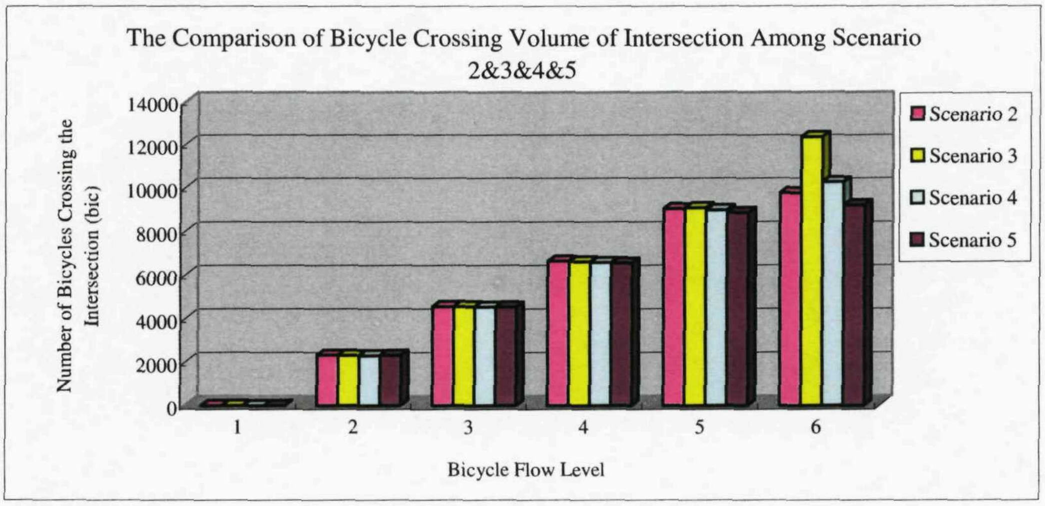


Figure 6.46 The comparison of bicycles crossing volume of intersection among scenario 2, 3, 4 and 5

6.10 Summary

Without signals for bicycles in at-grade signalised intersections, even a low level of bicycles can exert a dramatic influence on motor vehicle journey times. The conflicts between motor vehicles and bicycles not only increase the delay sharply, but also lead to serious congestion and capacity reduction. The risk of accident increases as well. In reality, as the bicycle volumes are high at intersections in Beijing urban network, the signals for bicycles become absolutely necessary.

The segregation of motor vehicles and bicycles is a basic principle for managing and operating mixed traffic in most at-grade intersections. Some temporal-segregation-related solutions can effectively avoid the conflicts between motor vehicles and bicycles, weaken the effect caused by bicycles, and enhance the safety of both motor vehicles and bicycles.

However, because of the different traffic conditions of intersections, no certain controlling method matches all the intersections. Each method has its own applicable condition. Based on the above-mentioned simulation results and analyses, the characteristics and applicable conditions are recommended as follows:

6.10.1 The method of bicycles using the same signal phase as motor vehicles of the same direction

For bicycles using the same signal phase as motor vehicles of the same direction, left-turn bicycles go through the intersection with left-turn motor vehicles and through bicycles go through the intersection with through motor vehicles. The conflicts between left-turn and through motor vehicles and bicycles can be avoided completely. Since these are no signals for right-turn vehicles, the vehicles can turn right when signals for left-turn and through vehicles are at red. The possible conflicts exist only between right-turn vehicles and bicycles under existing signal settings. Therefore, the negative effect caused by the change of bicycle traffic flow on motor vehicles is not remarkable.

However, the change of bicycle traffic flow has some obvious effect on the bicycle capacity of the intersection. During the left-turn green phase, some through bicycles wait before the stop line, which block the left-turn bicycles. Therefore, some left-turn bicycles cannot cross the intersection during the green period and have to wait for the next cycle. The through bicycles were blocked by the left-turn bicycles in the same way during the through green phase. According to the simulation results, the bicycle capacity of the intersection decreased 27% compared with the method using a second stop line for bicycles.

With the method of bicycles using the same signal phase as motor vehicles, the bicycles can cross the intersection continuously without stopping inside the intersection. The movements of bicycles will not be disturbed by traffic from other directions. The driving path is the shortest in all solutions and this is also in conformity with the psychology of cyclists.

Due to the fact that the left-turn bicycles cross the intersection with the left-turn motor vehicles, some potential disturbances exist caused as bicycles compete with motor vehicles by occupying the space within the intersection. In order to avoid such potential disturbances, intersections may need to be large.

Separating the left-turn and through bicycle flows, the channelization of bicycle lane before the stop line can minimise the influence between left-turn and through bicycles, thus improve the bicycle capacity of intersection accordingly.

6.10.2 The method of the second stop line for bicycles

In the method with the second stop line for bicycles, bicycles are not allowed to occupy and cross the confined area located at the centre of the intersection. All bicycles are required to travel outside the banned area while motor vehicles maintain their original route in the banned area. The left-turn bicycles will finish left-turn movement by two through movements. In this way, the left-turn bicycles will be limited, and the conflict between motor vehicles and bicycles will be avoided completely. Therefore, the negative effect caused by the change of bicycle traffic flow on motor vehicles remains unremarkable.

The change of bicycle traffic flow also brings very little effect on bicycle capacity of intersection. This method can provide the highest bicycle capacity of intersection and the lowest average bicycle delay among these proposed measures. According to the simulation results, the bicycle capacity of intersection, with the second stop line is assigned, is 27% higher than that of bicycles using the same signal phase as motor vehicles of the same direction, and 20% higher than that of using exclusive bicycle phase, and 34% higher than that with the method of building a separate lane near the intersection to prohibit left-turn bicycles.

In reality, the space available in the intersection is one of the important issues, which will decide whether this method can be adopted. Lacking enough storage space before the second stop line, some bicycles may stop outside the storage space and block the movement of motor vehicles, thus causes unpredictable delay and congestion.

In general, this method exerts the least negative effect on both motor vehicles and bicycles. Provided the intersection space is large enough to accommodate the stopped bicycles without disturbing or blocking the movement of motor vehicles, the use of a second stop line for bicycles is an effective measure to control mixed traffic in at-grade intersections and will be a good option especially when the bicycle traffic flow is very high.

6.10.3 The method of exclusive bicycle signal

The use of an exclusive bicycle signal is another approach to separating motor vehicles and bicycles in at-grade intersections. A green phase gives bicycles their own time to cross an intersection in the signal cycle time, thus avoiding the conflicts between motor vehicles and bicycles completely. Therefore, it is safer for traffic, although, the safety is achieved at the price of the reduction of vehicular capacity of intersection. The additional bicycle phase increases the signal cycle time, and the average vehicle delay increases accordingly, and consequentially the vehicular capacity of intersection is reduced as well. According to the simulation results, since the exclusive bicycle signal was assigned, the vehicular capacity of intersection decreased 11% compared with other solutions.

Based on exclusive bicycle phase, ignorance of conflicts between bicycles will result in underestimating the negative effect caused by bicycles. During the exclusive bicycle phase, bicycles in different directions interact inside the intersection, which leads to disorder to some extent and causes some bicycles to remain inside the intersection when bicycle phase finished. Therefore the method of exclusive bicycle signal is only suitable to small intersections with low level of bicycle traffic flow.

6.10.4 The method of building a separate lane for prohibiting of left-turn bicycles

Building a separate lane is a left-turn prohibition approach in which separate lane is built near the intersection. Through the lane, the left-turn bicycles change their route to a right-turn path, which help to avoid the conflicts between through motor vehicles and left- turn bicycles. Therefore, the negative effect caused by the change of bicycle traffic flow on vehicle traffic is small. This approach is safe for both motor vehicles and bicycles.

However, safety is achieved at the price of sacrificing cyclists' benefit. It is clear that this method increases the travel distance of left-turn cyclists, and consequently increases their average journey time. And the bicycle flow intending to cross the stop line increases as well. For example, the left-turn bicycles from south to west have to cross the stop line of the east approach. Therefore, the bicycle flow in east arm increases on account of the join of left-turn bicycles from south to west. As a result, some bicycles cannot cross the stop line within one bicycle phase and have to wait for the next bicycle phase when the bicycle volumes are high.

The location of the intersection and the layout around the intersection need to be taken into account. Cyclists may not accept this approach if the separate lane is too long or too far from the intersection.

The above-mentioned simulation results are quite similar to the results of some research by other researchers.

Jing and Wang (2004) collected data at a typical intersection in China, and compared the capacity before and after the implementation of bicycles by using the same signal phase with vehicles of same direction. The research results showed that the bicycle capacity increased around 24%.

Sun and Yang (2004) compared the capacity of a typical intersection under two different operations. The first operation was about bicycles using the same signal phase with vehicles of same direction, while the second operation was about adopting second stop line for bicycles. The results indicated that by using second stop line for bicycles, bicycle capacity at this intersection was 36% higher than bicycles adopting the same signal phase with vehicles of same direction. The findings in this simulation study are similar to Sun and Yang's results.

Wang and Wei (1993) suggested that the method of second stop line for bicycles is more suitable for large intersections. It was also mentioned in this research that the method of left-turn prohibition had been tried in Beijing but proved to be unpopular with cyclists. This is consistent with the findings in this simulation study.

According to Godefrooij's study (Godefrooij, 1997), there is a growing practice of a less comprehensive form of segregation at intersections in the Netherlands, which is similar to the solution of second stop line for bicycles: bicycle tracks or bicycle lanes are continued across the intersection, thus providing the cyclists their own space in the intersection area. This method forces cyclists to split up a left turn manoeuvre into two separated crossing manoeuvres, which lengthens the riding curve, and might cause some delay as well.

Godefrooij also mentioned in his study that segregation in time using traffic lights will cause delay for all road users (Godefrooij, 1997). The more signal phases there are in the signal cycle, the smaller the proportion of total time which can be given to each traffic movement, and hence the greater probable delay for each movement.

Wei et al. (2003B) conducted a research on bicycle signal phase. Before-and-after studies showed that since the exclusive bicycle signal phases were assigned, the abeyance percentage of the control regulations by bicycles increased up to 97%, where low volumes of bicycle traffic existed.

Researchers in Davis, in the USA researched the effects of integration a bicycle phase into an existing large urban intersection signal cycle and to quantify and compare the effects in a benefit-cost setting. The research results showed that the vehicle delay increased nearly 37% with bicycle phase, however, the estimated saving due to crash reductions were the dominant result of the bicycle phase installation, overshadowing the construction costs and increased vehicle delay. The analysis results indicate the installation of the bicycle phase was a beneficial project and implies that other such projects would also be beneficial under similar conditions, which including high bicycle volume, and high frequency of motor vehicle-bicycle conflicts. However, the CO analysis indicated that with the bicycle phase in place the CO emissions increased due to increased vehicle delay and stops (Korve and Niemeier, 2002).

The results provide strong support for the segregation of motor vehicles and bicycles at at-grade intersections in Beijing, China. As much of delay occurs at intersections, the temporal-segregation-related operations may provide appropriate solutions. The efforts trying to limit the conflicts between motor vehicles and bicycles bring to some extent benefits in reducing the traffic delay and increasing the capacity of intersections. In this way, the safety of both motor vehicles and bicycles may be improved consequently.

7. Conclusions and recommendations

7.1 Introduction

This thesis has described the development, validation and application of a microscopic simulation model dealing with mixed traffic with motor vehicles and bicycles. The thesis starts with a background introduction that leads to this study. And there follows, an extensive literature review and models definition on characteristics of vehicles and bicycles, driver behaviour, and control measures were carried out. Simulation model was developed based on the definition. A verification and validation procedure was carried out using the data collected in real road network in Beijing, China. A series of applications were carried out to explore how the different operations affect traffic.

The following sections include a summary of main study results and design guidance for mixed traffic operations and the recommended further work.

7.2 Main findings of the research

A microscopic traffic simulation model is often considered to be the most efficient tool in research, demonstration, analysis, and evaluation of transport operations and designs. However, because of the unique characteristics of mixed traffic situations, existing microscopic simulation models can not generally represent real mixed traffic situations such as those found in Beijing, China. A new microscopic simulation model dealing with mixed traffic with motor vehicles and bicycles was developed. This new model can demonstrate many operations on mixed traffic at at-grade intersection in urban network in Beijing.

The interactive conflicts and disturbances are one of the main factors result in traffic congestion, accidents and capacity reduction on urban networks, especially at intersections. The problem of mixed traffic at at-grade intersections is a major issue of urban traffic management. The simulation results illustrated that even a low level of bicycles causes dramatic influence on traffic conditions at at-grade signalised intersections, if the motor vehicles and bicycles are not separated properly, assuming there are no changes in vehicular traffic demand. Very serious traffic congestions happen because of the substantial conflicts between motor vehicles and bicycles. In such a situation, the temporal or spacial separation of motor vehicles and bicycles should be considered.

The segregation of motor vehicles and bicycles is a basic principle of managing and operating mixed traffic in most at-grade intersections. The simulation results give strong support for the segregation of motor vehicles and bicycles at at-grade intersections in Beijing, China. These temporal-segregation-related solutions can reduce the conflicts between motor vehicles and bicycles, reduce the influences caused by bicycles, and enhance the safety of both motor vehicles and bicycles.

However, because of the different traffic conditions found at different intersections, no single control method works for all. Each method has its own applicable condition. For the operation design for mixed traffic at at-grade intersections in China, the following conclusions from the simulation results and analyses may be helpful to traffic planners and engineers in China.

1. When bicycles use the same signal phase as motor vehicles of the same direction, the increasing bicycle traffic demand causes only slight impacts on right turning vehicular traffic. A change of bicycle traffic demand brings some obvious effect on bicycle capacity. When bicycle traffic demand is heavy, average bicycle delay increases sharply. The driving path of cyclists is the shortest in all solutions. Therefore, this operation is more acceptable for cyclists.

2. Among all the temporal-segregation-related solutions, the method of using a second stop line for bicycles to prohibit left-turns for bicycles produces the minimum average bicycle delay without causing significant extra vehicle delay. Whether there is enough storage space before the second stop line for the bicycles is an important issue which needs to be considered. This method would be a good option especially when the bicycle traffic flow is very high, if the intersection space is large enough to accommodate the stopped bicycles.

3. Exclusive bicycle green phases gives bicycles their own time to cross an intersection in a signal cycle time. When the bicycle traffic demand is low, this method produces similar bicycle delay to the method of using a second stop line. When the bicycle traffic demand is high, the bicycle delay increases sharply. This method also leads to the maximum vehicle delay due to the increase of cycle time. This method is only suitable to intersections with small bicycle volumes.

4. The method of building a separate lane to prohibit left-turn bicycles, though producing similar vehicle delay to the method of using second stop line for bicycles, results in maximum bicycle delay. Because the cyclists need to ride an extra distance, the acceptance of this operation is the lowest. The location of the intersection and the geometric layout will affect the bicycle delay and needs to be taken into consideration.

The development of the simulation model is mainly based on the characteristics of mixed traffic in Beijing, China. The guidelines recommended above may be helpful to traffic planners and engineers in China in solving the urban mixed traffic problems. However, the scenarios developed and used in simulation study are based on the conceptual design. In practice, when dealing with the traffic design for intersection of mixed traffic, simulation studies based on the real traffic situation are required. A procedure of traffic design is recommended.

1. Survey of the intersection. The purpose of survey is to know the traffic conditions. Some data can be obtained via the survey. The data includes the demand of motorized and non-motorised traffic, the geometry of the intersection, channelization of the intersection, control settings and so on.

2. Analysis. Based on the information obtained via the survey, the key issues that affect the capacity of the intersection need to be found. Referring to the guidelines recommended above, appropriate methods are proposed as the potential solutions.

3. Estimate. Apply all the potential solutions to the simulation model and get the results. Analyze and estimate the simulation results. After comparison among these potential solutions and considering the real situation, the optimal solution can be obtained. The future development of traffic needs to be taken into account. Whether the operation is easy to manage and whether the method fits the people's psychology also need to be considered.

7.3 Further work

The following are recommended for further consideration.

1. Most of the roads in Beijing are physically separated lanes for motor vehicles and bicycles, therefore, the lateral disturbances caused as cyclists intend to compete with motor

vehicles were not considered in this research. However, mixed bicycle roads, in which bicycles share the roadways with motor vehicle traffic without any physical or nonphysical separations, are very common in some small cities in China. Better understanding of the lateral disturbances on mixed bicycle roads is necessary. It is necessary to improve the simulation behaviour model with further research on the disturbance between motor vehicles and bicycles from lateral side.

2. The further extension of the model should concern simulating multi-modal traffic behaviours that also includes pedestrians and buses. In real situations, pedestrians always have an effect on traffic. Conflicts among motor vehicles, bicycles, and pedestrians at intersections are serious in China. Also, the disturbances caused by buses are common in urban areas. These disturbances include the arrival and departure of buses, bus priority control, and exclusive bus lanes. In practice, when dealing with the traffic design for intersections with mixed traffic, simulation study based on the real traffic situation is required. Therefore, research on pedestrians and buses' behaviour and interactions among motor vehicles, bicycles and pedestrians should be included in the further work.

3. Although the case studies in Chapter Six represent typical intersection situations of Beijing, there are still many other types of intersections that have not been investigated, such as T-junctions and roundabouts. Also, in this research, the simulation studies were based on conceptual scenarios with typical characteristics. In real situations, the conditions may be more complex and with additional limitations, such as different turning proportions, vehicle actuated signal control, different road geometry, and various layouts. Therefore, it is suggested to conduct further research using the proposed solutions at the real sites with different size and configurations.

4. For this research, only isolated intersections were considered. In reality, there exist many influences from downstream and upstream which affect the performance of the traffic. For example, a queue build up downstream may cause extra delay to the traffic. Also, some practical traffic solutions and management are designed for road network and may bring better effect, such as "cycle network". For a comprehensive assessment of measures, network level rather than isolated intersection level was supposed to be more appropriate. Therefore, it is suggested that further research should include consideration for the situation of road network. Also, in this research, data were collected at several isolated points during each of

calibration and validation tests. It is more desirable to collect multiple data points at a network level, collected over time, of field data to account for the variability.

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APPENDIX A

FLOWSIM Car-Following Model

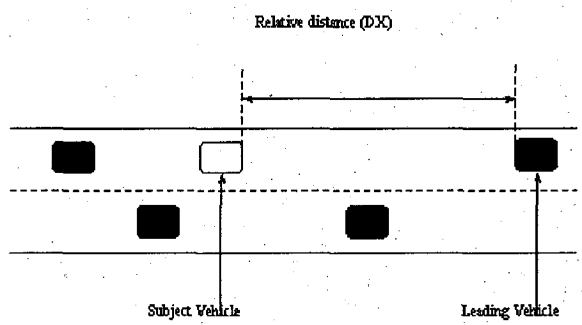

Figure A.1 Car Following

1. Variable 1: Relative speed (DV)

$$DV = V\_LeadingVehicle - V\_SubjectVehicle \tag{A.1}$$

If the subject vehicle is the head vehicle in this lane, the last vehicle in the next lane (which is on the route of the subject vehicle) will be searched and the retrieved vehicle will be regarded as the leading vehicle. If there are still no vehicles in the next lane on the route of subject vehicle, then the relative speed is set to be opening fast.

If the subject vehicle is not the head vehicle in this lane, the speed of leading vehicle is the mean speed of all vehicles in front of the subject vehicle.

2. Variable 2: Distance divergence (DSSD)

$$DSSD = DS / sd \tag{A.2}$$

Where,

sd is the desired gap, which follows a normal distribution;

$$DS = DX / V\_SubjectVehicle \tag{A.3}$$

Where,

DX is the relative distance;

If the subject vehicle is the head vehicle in this lane, the last vehicle in the next lane (which is on the route of the subject vehicle) will be searched and the retrieved vehicle will be regarded as the leading vehicle. If there are still no vehicles in the next lane on the route of subject vehicle, then the distance divergence is set to be much too far.

APPENDIX B

FLOWSIM Lane-Changing Model

1. The logic for LCO

If the target lane of the subject vehicle is on the offside of the subject lane, the LCO will be carried out. Under optional conditions, whether this lane changing can be conducted depends on two variables, "overtaking benefit" and "opportunity". The "overtaking benefit" is the increased speed gained giving that a LCO is carried out. "Opportunity" is to check whether there is an enough gap between the front vehicle and the rear-approaching vehicle in the offside lane. "Opportunity" can be measured by the headway to the rear-approaching vehicle in the offside lane. The higher "overtaking benefit" and the higher "opportunity" result in higher potential to change to the offside lane.

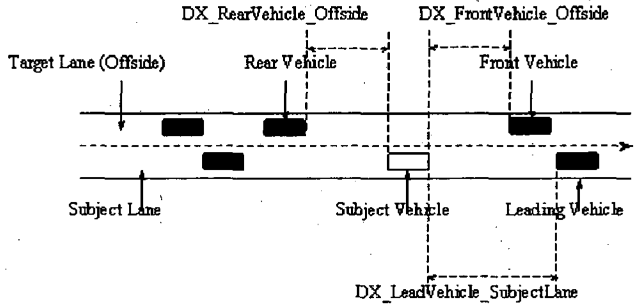

Figure B.1 Lane Changing (LCO)

- Parameters for estimation of "overtaking benefit" and "opportunity" (TTC & TH)

$$TTC\_Front\_Offside = DX\_FrontVehicle\_Offside \,/\, DV\_FrontVehicle\_Offside \tag{B. 1}$$

$$TH\_Front\_Offside = DX\_FrontVehicle\_Offside \,/\, V\_SubjectVehicle \tag{B. 2}$$

$$DV\_FrontVehicle\_Offside = V\_FrontVehicle\_Offside - V\_SubjectVehicle \tag{B. 3}$$

V_FrontVehicle_Offside is the speed of front vehicle in the offside lane. If there is no front vehicle before the subject vehicle in the offside lane, V_FrontVehicle_Offside is set to be the desired speed of the subject vehicle.

$$DX\_FrontVehicle\_Offside = S\_FrontVehicle\_Offside - S\_SubjectVehicle \tag{B. 4}$$

S_FrontVehicle_Offside is the position of front vehicle in the offside lane. If there is no front vehicle before the subject vehicle in the offside lane, S_FrontVehicle_Offside is set to be the stop line of the offside lane.

$$TTC\_\mathrm{Re}ar\_Offside = DX\_\mathrm{Re}arVehicle\_Offside / DV\_\mathrm{Re}arVehicle\_Offside \tag{B. 5}$$

$$TH\_\mathrm{Re}ar\_Offside = DX\_\mathrm{Re}arVehicle\_Offside / V\_\mathrm{Re}arVehicle\_Offside \tag{B. 6}$$

$$DV\_\mathrm{Re}arVehicle\_Offside = V\_\mathrm{Re}arVehicle\_Offside - V\_SubjectVehicle \tag{B. 7}$$

V_RearVehicle_Offside is the speed of rear vehicle in the offside lane. If there is no rear vehicle after the subject vehicle in the offside lane, V_RearVehicle_Offside is set to 0.

$$DX\_\mathrm{Re}arVehicle\_Offside = S\_SubjectVehicle - S\_\mathrm{Re}arVehicle\_Offside \tag{B. 8}$$

S_RearVehicle_Offside is the position of rear vehicle in the offside lane. If there is no rear vehicle after the subject vehicle in the offside lane, S_ReartVehicle_Offside is set to be 0.

$$TH\_LeadVehicle\_SubjectLane = DX\_LeadVehicle\_SubjectLane / V\_SubjectVehicle \tag{B. 9}$$

$$DV\_LeadVehicle\_SubjectLane = V\_LeadVehicle\_SubjectLane - V\_SubjectVehicle \tag{B. 10}$$

V_LeadVehicle_SubjectLane is the speed of leading vehicle in the subject lane. If there is no leading vehicle before the subject vehicle in the subject lane, V_LeadVehicle_SubjectLane is set to be the desired speed of the subject vehicle.

$$DX\_LeadVehicle\_SubjectLane = S\_LeadVehicle\_SubjectLane - S\_SubjectVehicle \tag{B.11}$$

S_LeadVehicle_SubjectLane is the position of leading vehicle in the subject lane. If there is no leading vehicle before the subject vehicle in the subject lane, S_LeadVehicle_SubjectLane is set to be the stop line of the subject lane.

2. The logic for LCN

If the target lane of the subject vehicle is on the nearside of the subject lane, the LCN will be carried out. Whether this lane changing can be conducted depends on two variables, "pressure from rear" and "gap satisfaction". The "pressure from rear" is the pressure caused by the difference of speed and distance between the subject vehicle and the following vehicle. The "gap satisfaction" can be obtained by evaluate how long the vehicle can stay in the nearside lane without reducing speed. The higher "pressure from rear" and the higher "gap satisfaction" result in higher potential to change to the nearside lane.

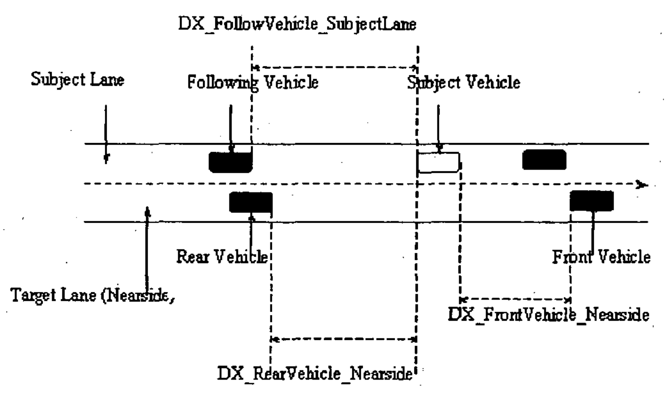

Figure B.2 Lane Change (LCN)

- Parameters for estimation of "pressure from rear" and "gap satisfaction" (TTC & TH)

$$TTC\_Front\_Nearside = DX\_FrontVehicle\_Nearside / DV\_FrontVehicle\_Nearside \tag{B. 12}$$

$$TH\_Front\_Nearside = DX\_FrontVehicle\_Nearside / V\_SubjectVehicle \tag{B. 13}$$

$$DV\_FrontVehicle\_Nearside = V\_FrontVehicle\_Nearside - V\_SubjectVehicle \quad \text{(B. 14)}$$

V_FrontVehicle_Nearside is the speed of front vehicle in the nearside lane. If there is no front vehicle before the subject vehicle in the nearside lane, V_FrontVehicle_Nearside is set to be the desired speed of the subject vehicle.

$$DX\_FrontVehicle\_Nearside = S\_FrontVehicle\_Nearside - S\_SubjectVehicle \quad \text{(B. 15)}$$

S_FrontVehicle_Nearside is the position of front vehicle in the nearside lane. If there is no front vehicle before the subject vehicle in the nearside lane, S_FrontVehicle_Nearside is set to be the stop line of the nearside lane.

$$TTC\_\mathrm{Re}ar\_Nearside = DX\_\mathrm{Re}arVehicle\_Nearside / DV\_\mathrm{Re}arVehicle\_Nearside \quad \text{(B. 16)}$$

$$TH\_\mathrm{Re}ar\_Nearside = DX\_\mathrm{Re}arVehicle\_Nearside / V\_\mathrm{Re}arVehicle\_Nearside \quad \text{(B. 17)}$$

$$DV\_\mathrm{Re}arVehicle\_Nearside = V\_\mathrm{Re}arVehicle\_Nearside - V\_SubjectVehicle \quad \text{(B. 18)}$$

V_RearVehicle_Nearside is the speed of rear vehicle in the nearside lane. If there is no rear vehicle after the subject vehicle in the nearside lane, V_RearVehicle_Nearside is set to 0.

$$DX\_\mathrm{Re}arVehicle\_Nearside = S\_SubjectVehicle - S\_\mathrm{Re}arVehicle\_Nearside \quad \text{(B. 19)}$$

S_RearVehicle_Nearside is the position of rear vehicle in the nearside lane. If there is no rear vehicle after the subject vehicle in the nearside lane, S_ReartVehicle_Nearside is set to be 0.

$$TH\_FollowVehicle\_SubjectLane = DX\_FollowVehicle\_SubjectLane / V\_FollowVehicle\_SubjectLane \quad \text{(B. 20)}$$

$$DV\_FollowVehicle\_SubjectLane = V\_SubjectVehicle - V\_FollowVehicle\_SubjectLane \quad \text{(B. 21)}$$

V_FollowVehicle_SubjectLane is the speed of following vehicle in the subject lane. If there is no following vehicle before the subject vehicle in the subject lane, V_FollowVehicle_SubjectLane is set to be 0.

$$DX\_FollowVehicle\_SubjectLane = S\_SubjectVehicle - S\_FollowVehicle\_SubjectLane \tag{B. 22}$$

S_FollowVehicle_SubjectLane is the position of following vehicle in the subject lane. If there is no following vehicle before the subject vehicle in the subject lane, S_FollowingVehicle_SubjectLane is set to be 0.

APPENDIX C

Table C 1 F-Test for Vehicle Initial Headway

F-Test Two-Sample for Variances		
	Variable 1	Variable 2
Mean	2920.153846	2907.692308
Variance	3542366.695	3470538.462
Observations	26	26
df	25	25
F	1.020696568	
P(F<=f) one-tail	0.479780942	
F Critical one-tail	1.955449136	

Table C 2 Two tailed T-Test for Vehicle Initial Headway

T-Test: Two-Sample Assuming Equal Variances		
	Variable 1	Variable 2
Mean	2920.153846	2907.692308
Variance	3542366.695	3470538.462
Observations	26	26
Pooled Variance	3506452.578	
Hypothesized Mean Difference	0	
df	50	
t Stat	0.02399437	
P(T<=t) one-tail	0.490476303	
t Critical one-tail	1.675905423	
P(T<=t) two-tail	0.980952605	
t Critical two-tail	2.008559932	

Table C 3 One-Sample Kolmogorov-Smirnov Test for Distribution of Vehicle Initial Headway

			IH_V
N			2990
Exponential parameter.(a,b)		Mean	.8040371
			4
Most Extreme Differences		Absolute	.008
		Positive	.008
		Negative	-.007
Kolmogorov-Smirnov Z			.452
Asymp. Sig. (2-tailed)			.987

a Test Distribution is Exponential.

b Calculated from data.

Table C 4 F-Test for Bicycle Initial Headway

F-Test Two-Sample for Variances		
	Variable 1	Variable 2
Mean	1551.166667	1550
Variance	765213.3161	775000
Observations	30	30
df	29	29
F	0.987372021	
P(F<=f) one-tail	0.486468633	
F Critical one-tail	0.537399458	

Table C 5 Two tailed T-Test for Bicycle Initial Headway

T-Test: Two-Sample Assuming Equal Variances		
	Variable 1	Variable 2
Mean	1551.166667	1550
Variance	765213.3161	775000
Observations	30	30
Pooled Variance	770106.658	
Hypothesized Mean Difference	0	
df	58	
t Stat	0.00514893	
P(T<=t) one-tail	0.497954718	
t Critical one-tail	1.671553491	
P(T<=t) two-tail	0.995909436	
t Critical two-tail	2.001715984	

Table C 6 One-Sample Kolmogorov-Smirnov Test for Distribution of Bicycle Initial Headway

		IH_B
N		2007
Exponential parameter.(a,b)	Mean	1.7940
Most Extreme Differences	Absolute	.011
	Positive	.011
	Negative	-.007
Kolmogorov-Smirnov Z		.485
Asymp. Sig. (2-tailed)		.973

a Test Distribution is Exponential.

b Calculated from data.

Table C 7 One-Sample Kolmogorov-Smirnov Test for Distribution of Vehicle Desired Speed (Mean = 40 km/h)

		DS_V
N		2970
Normal Parameters(a,b)	Mean	11.1111
	Std. Deviation	1.19965
Most Extreme Differences	Absolute	.011
	Positive	.011
	Negative	-.009
Kolmogorov-Smirnov Z		.595
Asymp. Sig. (2-tailed)		.871

a Test distribution is Normal.

b Calculated from data.

Table C 8 One-Sample Kolmogorov-Smirnov Test for Distribution of Bicycle Desired Speed (Mean = 12 km/h)

		DS_B
N		2950
Normal Parameters(a,b)	Mean	3.3327
	Std. Deviation	.79280
Most Extreme Differences	Absolute	.014
	Positive	.014
	Negative	-.008
Kolmogorov-Smirnov Z		.745
Asymp. Sig. (2-tailed)		.635

a Test distribution is Normal.

b Calculated from data.

Table C 9 One-Sample Kolmogorov-Smirnov Test for Vehicle Free-Flow Speed

		FFS_VEH
N		358
Normal Parameters(a,b)	Mean	10.8710
	Std. Deviation	1.21065
Most Extreme Differences	Absolute	.055
	Positive	.039
	Negative	-.055
Kolmogorov-Smirnov Z		1.037
Asymp. Sig. (2-tailed)		.232

a Test distribution is Normal.

b Calculated from data.

Table C 10 One-Sample Kolmogorov-Smirnov Test for Bicycle Free-Flow Speed

		FFS_BIC
N		550
Normal Parameters(a,b)	Mean	4.2636
	Std. Deviation	.78936
Most Extreme Differences	Absolute	.049
	Positive	.049
	Negative	-.048
Kolmogorov-Smirnov Z		1.158
Asymp. Sig. (2-tailed)		.137

a Test distribution is Normal.

b Calculated from data.

Table C 11 F-Test for Comparison of Bicycle Average Speed

F-Test Two-Sample for Variances		
	Variable 1	Variable 2
Mean	4.255101328	4.257780076
Variance	0.719095865	0.476010723
Observations	232	242
df	231	241
F	1.510671567	
P(F<=f) one-tail	0.000791019	
F Critical one-tail	1.239043534	

Table C 12 F-Test for Comparison of Vehicle Average Travel Time (Route A_B)

F-Test Two-Sample for Variances	Alpha	
	Variable 1	Variable 2
Mean	65.19548872	64.5210728
Variance	200.6736159	113.8546415
Observations	133	522
df	132	521
F	1.762542249	
P(F<=f) one-tail	6.52648E-06	
F Critical one-tail	1.244110592	

Table C 13 F-Test for Comparison of Vehicle Average Travel Time (Route C_A)

F-Test Two-Sample for Variances	Alpha	
	Variable 1	Variable 2
Mean	54.58974359	54.40168539
Variance	382.1750663	182.6640084
Observations	117	178
df	116	177
F	2.092229715	
P(F<=f) one-tail	4.54238E-06	
F Critical one-tail	1.315441089	