Heterogeneous Motorised Traffic Flow Modelling using Cellular Automata

by

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Declaration

I hereby certify that this material, which I now submit for assessment on the programme of study leading to the award of PhD is entirely my own work, that I have exercised reasonable care to ensure that the work is original, and does not to the best of my knowledge breach any law of copyright, and has not been taken from the work of others save and to the extent that such work has been cited and acknowledged within the text of my work.

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I dedicate this study to...

My parents and elder sister

Who have supported me all the way since the beginning of my studies and have been a great source of motivation and inspiration

&

My siblings, whom I love and respect

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Abstract

Traffic congestion is a major problem in most major cities around the world with few signs that this is diminishing, despite management efforts. In planning traffic management and control strategies at urban and inter urban level, understanding the factors involved in vehicular progression is vital. Most work to date has, however, been restricted to single vehicle-type traffic. Study of *heterogeneous* traffic movements for urban single and multi-lane roads has been limited, even for developed countries and motorised traffic mix, (with a broader spectrum of vehicle type applicable for cities in the developing world). The aim of the research, presented in this thesis, was thus to propose and develop a model for *heterogeneous motorised* traffic, applicable to situations, involving common urban and interurban road features in the western or developed world. A further aim of the work was to provide a basis for comparison with current models for *homogeneous* vehicle type.

A *two-component cellular automata* (2-CA) methodology is used to examine traffic patterns for single-lane, multi-lane controlled and uncontrolled intersections and roundabouts. In this heterogeneous model (binary mix), space mapping rules are used for each vehicle type, namely *long (double-unit length)* and *short (single-unit length)* vehicles. Vehicle type is randomly categorised as long (LV) or short (SV) with different fractions considered. Update rules are defined based on given and neighbouring cell states at each time step, on manoeuvre complexity and on acceptable space criteria for different vehicle types. Inclusion of heterogeneous traffic units increases the algorithm complexity as different criteria apply to different cellular elements, but mixed traffic is clearly more reflective of the real-world situation.

The impact of vehicle mix on the overall performance of an intersection and roundabout (one-lane one-way, one-lane two-way and two-lane two-way) has been examined. The model for mixed traffic was also compared to similar models for homogeneous vehicle type, with throughput, queue length and other metrics explored. The relationship between arrival rates on the entrance roads and throughput for mixed traffic was studied and it was found that, as for the homogeneous case, critical arrival rates can be

identified for various traffic conditions. Investigation of performance metrics for *heterogeneous* traffic (short and long vehicles), can be shown to reproduce main aspects of real-world configuration performance. This has been validated, using local Dublin traffic data.

The 2-CA model can be shown to simulate successfully both homogeneous and heterogeneous traffic over a range of parameter values for arrival, turning rates, different urban configurations and a distribution of vehicle types. The developed model has potential to extend its use to linked transport network elements and can also incorporate further motorised and non-motorised vehicle diversity for various road configurations. It is anticipated that detailed studies, such as those presented here, can support efforts on *traffic management* and aid in the design of *optimisation strategies* for traffic flow.

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Chapter 1: Introduction

1 Introduction

Heterogeneous traffic flow characteristics on urban roadways are intrinsic to understanding urban traffic problems. In order to improve traffic conditions on large urban roadways, given the highly complex interplay of the multiple units and imposed control mechanisms, as well as to gain insight into response of heterogeneous traffic to different control strategies, modelling is an important tool, (Hoogendoorn, 1999). Behaviour is diverse and complex due to different vehicle length, (buses, cars, trucks and motorised two wheelers), heterogeneous drivers, multiple road configurations, alternative control measures and so on. These are just some of the key features that require attention in traffic simulation procedures to study real effects. Modelling of heterogeneous traffic flow to date is very limited; the analysis, findings and suggestion of this study provide a basis for future improvements.

Traffic in Western or *developed countries* is characterised by a variety of motorised vehicles such as cars, buses, trucks and motorcycles. This differs from the situation in *developing countries*, which have a diverse mix of motorised and non-motorised vehicles including bicycles, cargo tricycles and human and animal-drawn carts (see Figure 1.1 a and b). Control method can vary widely (Figure 1.1 c) and, together with the traffic mix, strongly influence urban network flows.

Figure 1. 1- Typical traffic in the developing and developed worlds. For example, from left, (a) India; (b) Ireland; (c) a local roundabout.

Traffic growth forecasts for *Ireland* for the period 2002-2040, made by the National Road Authority (NRA), (NRA, 2003) predict dramatic increases in both LGVs (cars and Light Good Vehicles combined) and HGVs (Heavy Goods Vehicles). With large traffic volumes, breakdown of traffic flow is likely to occur, particularly on single-lane roads where both turning and straight ahead vehicles wait in a single queue.

Nonetheless, most European cities still rely to some extent on single-lane connections to major arterial routes.

This thesis contributes to the traffic modelling field, through use of cellular automata (CA) techniques for heterogeneous microscopic traffic flow. It seeks to address questions relating to the effect of long vehicles on the operation of urban configurations and, thus, to the development of appropriate traffic simulation models for Western European roads.

For Western Europe, the simplified heterogeneous model excludes multiple and shared occupation, (which more nearly reflect India and other eastern road-usage patterns). Field observations on traffic patterns can be difficult and time consuming to obtain, meaning computer simulation models offer a viable alternative for in-depth study and can aid understanding of traffic dynamics (Arasan and Koshy, 2002).

Several methods have been used for traffic flow modelling and these are reviewed in the next chapter. In this context, CA has emerged as a particularly efficient tool, in various applications with the help of computer simulations e.g., large traffic network may be simulated in multiple real time on a standard PC (Brockfeld et al., 2001). CA defines a grid of cells using discrete variables for time, space and system states (Chopard and Droz, 1998 and Wolfram, 2002). Cells are updated synchronously and in parallel for every cell in the space, according to a local rule using a finite set of neighbouring cells. CA techniques, used to model complex traffic interaction based on simple rules (Nagel and Schreckenberg, 1992), have been studied over a decade, a very promising alternative to traditional models in describing traffic flow. Nevertheless, work has mostly focused on homogeneous situations, which are highly idealised. In this thesis, simulation of heterogeneous motorised traffic is the primary focus: new CA models are developed in what follows to study controlled and un-controlled junctions and roundabouts. Specifically, we propose and develop a simplified and novel heterogeneous *two-component cellular automata* model that allows, in the first instance, for two classes of vehicle, long (LVs) and short (SVs). The model was designed to describe stochastic interaction between individual vehicles and is independent of headway distribution, as in Wang and Ruskin (2002). Space mapping rules are used in this heterogeneous model for each vehicle type.

1.1 Research Objective

The goal of this research is to develop, analyse and, validate the system, and to ascertain the effect of LVs in the traffic mix on the metrics which reflect road performance and freedom of flow; these include throughput, flow capacity, total delay time and queue length (discussed Section 4.5.3). Amongst other factors we also wish to compare how different configurations and control set-ups influence a traffic system. Focus here has mainly been given to fixed time signalised and stop control (or yield control) intersections and roundabouts.

The main objectives of this research effort are summarized as follows:

- To determine the effect on throughput and capacity of a range of *arrival rate*s (Section 4.5) and to contrast homogeneous and heterogeneous cases.
- To determine the effect on throughput and capacity of a range of *turning rates* (Section 4.4) and to contrast homogeneous and heterogeneous cases.
- To determine the effect on performance of increasing the proportion of *long vehicles*, (Section 4.2).
- To determine the effect on throughput of increasing the proportion of *aggressive drivers* (Section 4.3),
- To determine the effect on throughput when control is varied for major roads.
- To determine additional delays caused by long vehicle manoeuvres
- To calibrate the heterogeneous model using real data.
- To provide a simulation interface to view traffic manoeuvres and flows.

1.2 Research contributions

The research effort provides three main contributions. First, it provides the two-component cellular automaton (2-CA) methodology for studying heterogeneous (binary) motorised traffic mix. This is new, as previous models focus on the assumption that all vehicle lengths are equal, which is not realistic. This new 2-CA model has been developed based on Minimum Acceptable sPace (MAP) criteria (Chapter-2), which enables categorization of vehicle types i.e. short (SV) and long (LV) vehicle, by space requirement on progression and manoeuvring. Secondly it permits systematic comparison of different configurations and control set-ups. Finally, the dissertation demonstrates the validity of the model in terms of its ability to reflect the behaviour of real local traffic data, (single-lane two-way controlled and uncontrolled junctions and roundabout), provided by Dublin City Council (DCC).These data are currently collected manually (Chapter-3). Sensitivity analysis can then be used to evaluate alternative conditions for the real world situation.

1.3 Research layout

This thesis is organised in seven chapters, as follows:

Chapter 2 describes the state of the art of road-traffic flow modelling, and how behaviour is affected by road type heterogeneity etc. The basis for cellular automata models and others is given in this chapter. Chapter 3 focuses on sources of real local traffic data and Visualisation of Real traffic situations. Chapters 4 to 6 describes the research methodology used, with details given on vehicle progression and movement, using CA and MAP rules at fixed time, signalised and stop controlled intersections and roundabouts. The particular focus in Chapter 4 is on performance measures at a controlled single-lane one-way cross intersection. In Chapter 5, we deal with single-lane two-way, X and T-intersection and provide estimates of throughput and capacity for the controlled and un-controlled intersection under different input conditions. Finally, in Chapter 6 the methodology is applied to a single-lane roundabout and studies the effect of heterogeneous traffic. Chapter 7 presents the summary of the research work, findings and future research directions.

Chapter 2: State-of-the-art of road-traffic flow modelling

2 State-of-the-art of road-traffic flow modelling

Vehicular traffic is increasing rapidly throughout the world, particularly in large urban areas in the past several years. It is become necessary to obtain the mathematical and theoretical description of the process in order to understand the dynamics of the traffic flow. This is specifically true for extremely heavy traffic when a roadway is forced to perform at its peak (Greenberg, 1958). The theories that describe traffic are in general derived from statistical study of the flows. Early scientific studies date from (Greenshields, 1935), while in 1955, Lighthill and Whitham presented a popular macroscopic traffic model, based on fluid-dynamic theory (Lighthill and Whitham, 1955). The authors studied the traffic jam as a shock wave by treating traffic as an effectively one-dimensional compressible fluid. Prigogine and Herman (1971) also presented the gas-kinetic model, based on space-time evolution of the velocity distribution of cars, (Prigogine, 1961). Further Newell (1961) proposed the microscopic, optimal velocity model based on the assumption of a delayed adaptation of velocity. Several concepts and techniques of physics were subsequently applied to such complex systems as transportation systems, including (Nagel and Schreckenberg, 1992; Nagel et al. 2000; Chowdhury et al 2000; Helbing 2001; Hoogendoorn, and Bovy, 2001 and Nagatani 2002). The complexity of vehicles motion along a highway or in urban roads has attracted the attention of many researchers due to its challenging nature, from the practical point of view (Velasco and Marques, 2005). In this context, cellular automaton (CA) models, which focus on microscopic evolution, are a well-established method to model, analyse, understand, and even forecast the real road traffic behaviour. Nagel and Schreckenberg (1992) proposed a "minimal" CA model in 1992 which has become the basic model of this field; the Nagel- Shreckenberg (NS), model (Nagel and Schreckenberg, 1992). The CA models are particularly interesting due to their computationally simplicity and flexibility and the ability to represent the complexity of real world traffic behaviour through detailed specification. Where as mathematical models have a closed form solution, which describes properties of the traffic flow in general. CA models are capable of producing clear physical patterns that are similar to those we see in everyday life.

Simulation of traffic flow models classified according to the level of detail with which they represent the system to be studied (Hoogendoorn, and Bovy, 2001):

- Microscopic (high-detail description where individual entities are distinguished and traced).
- Mesoscopic models (medium detail).
- Macroscopic models (low detail).

Here we discuss two different traffic flow perspectives i.e. macroscopic and microscopic modelling approaches. In the *macroscopic models* (Lighthill and Whitham, 1955), the whole traffic stream is treated as a continuous fluid, with behaviour described in terms of suitable aggregate variables such as traffic volume, density, and mean speed. These are concerned only with the group of vehicles, but individual vehicles are not considered. In contrast, *microscopic models* of vehicular traffic focus on individual vehicles, each of which represents a "particle"; the nature of the interaction among these particles is determined by the way the vehicles influence each other's movement, (Chowdhury et al. 2000). Since our interest is on models of CA type, due to the additional layers of complexity, which heterogeneity imposes, our main emphasis in what follows is on microscopic modelling approaches.

Some important model milestones are summarised and then we go on to describe the modelling approach that has been chosen, in this thesis, to represent heterogeneous motorised traffic flow. For this we elaborate on the cellular automata models, which have been recently extensively employed in the homogeneous case (Nagel and Schreckenberg 1992, Fukui and Ishimashi 1996, Nishinari and Takahashi 2000, Kerner and Klenov 2002 and others, before considering heterogeneous requirements).

2.1 Macroscopic models

Other macroscopic models describing traffic as a continuous flow of liquid in terms of aggregate quantities like density or flow followed that of Lighthill and Whitham, (1955) and include Kerner and Konha,(1994) and Lee et al., (1998). The first order fluid approximation of traffic flow dynamics proposed by Lighthill and Whitham (1955) and subsequently modified by Richards (1956) is known as the LWR model, and provides a description of traffic behaviour for a single one-way road using three variables that vary in time (t) and space (x): these are flow (q), density (k), speed (u). The First order models have some obvious deficiencies, such as failure to describe *platoon diffusion* (a phenomenon that takes place over long distances and times when traffic is light,(Newell, 1960)) and inability to explain the instability of heavy traffic. In attempt to correct some of these problems higher order theories were developed by Payne (1971), although the model was based on a few similar variables and assumptions. The state variables are the flow q(x,t), the density k(x,t) and the mean flow speed u(x,t), defined as, $u(x,t) = \frac{q(x,t)}{k(x,t)}$, where this is usually written in terms of $q(x,t) = u(x,t).k(x,t)$.

This equation gives the traffic stream is represented in an aggregate manner using characterstics as flow-rate, density, and velocity. Individual vehicle manoeuvre, such as a lane-change, are usually not explicitly represented. Cremer and May (1985) interestingly find that the results of such high-order models are unacceptable without numerous "engineering fixes". That is, the unadjusted models cannot produce acceptable results for simple situations, such as the onset and dissipation of congestion near a lane drop.

Higher order refinements of the LWR model do not correct the deficiencies (i.e. instability of heavy traffic, which exhibits oscillatory phenomena) in the proper way. Additional evidence in this respect has been presented by Cassidy and Windover (1995) who addressed the problem using a high resolution graphical technique for analyzing freeway data. These authors show that disturbances of higher flow/ increased density move along with traffic flow without diffusing, presumably because the traffic stream is composed of heterogeneous drivers who like to follow at different headways for the same speed. Holland and Woods (1997) and Greenberg et al. (2003) divided the homogeneous road into traffic lanes. These non-homogeneous road models assume that a slightly different fundamental diagram[1] applies to *each traffic lane*. On the basis of different densities between the lanes, lane change flows are then taken into consideration. These models have some similarities to the idea we have developed for traffic manoeuvring, but in this case unidirectional flow is all that is considered.

Logghe (2003) also extended the original homogeneous LWR model to a heterogeneous version. The author divided the heterogeneous traffic population into homogeneous classes. Vehicles of a particular class are described by the original LWR model, if the road is free of other vehicles. This allowed formulation of a fundamental diagram for each class separately. The characteristic property of each class is described by its fundamental diagram $q_i = Q_{ei}^h(k_i)$, where the superscript "h" in the formula indicates

[1] The Fundamental diagram of traffic flow is a diagram that gives a relation between the traffic flux (cars/hour or flow rate q) and the traffic density (cars/km),(Zhang and Lin, 2001).

the homogeneous flow of vehicles. It is in addition, assumed that flow "q" is related to density k. This equilibrium relation $Q_e(k)$ is better known as the fundamental diagram. When the various diagrams are similar in shape a scaling is carried out, related to the passenger-car equivalents. Speed in the mixed traffic flow is homogeneous over all classes. Interactions between the various vehicles do not affect the fundamental diagrams of the various classes. However, the extended LWR model cannot describe the following characteristics:

- Different maximum speed limits for different vehicle characteristics; (e.g heavy vehicles versus private cars), road characteristics; (e.g.slope) and driver characteristics; (fast drivers)
- Difference in speed of flow. Homogeneous traffic speed is assumed. Therefore overtaking and multilane highways are insufficiently modelled.

Critique on LWR-type models

Some of the following list of critique on LWR models given (see, Hoogendoorn, and Bovy, 2001)

- The LWR-models contain stationary speed-density relations, implying that the mean velocity adapts instantaneously to the traffic density rather than considering some delay. That is, the kinematic theory does not allow fluctuations around the equilibrium speed-density relationship.
- The kinematic wave-theory of Lighthill and Whitham shows shock wave formation by steeping velocity jumps to infinite sharp discontinuities in the density. These are in contradiction with smooth shocks observed in real-life traffic that can be described by Payne-type models.
- The LWR-theory is not able to describe regular start-stop waves with amplitude-dependent oscillation times that are observed in real-life traffic (e.g. Verweij, 1985).
- Similarly, the LWR-models are not able to predict the occurrence of localised structures and phantom-jams, i.e. the LWR-theory does not describe the amplification of small disturbances in heavy traffic.

Macroscopic flow models describe traffic at a high level of aggregation as a flow without distinguishing its constituent parts. For instance, the traffic stream is represented in an aggregate manner using characteristics as flow-rate, density, and velocity. Individual vehicle manoeuvres, such as a lane change, are usually not explicitly represented. A macroscopic model may assume that the traffic stream is properly allocated to the roadway lanes, and employ an approximation to this end.

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

2.2 Microscopic model

A model in which objects and events are represented individually is defined to be microscopic (Law and Kelton, 1991). Models of this kind allow for the characteristics and behaviour of individuals to be distinctly represented. Some examples of applications in the transportation field of this approach to modelling are queueing and gap acceptance processes (see Brilon, 1988; 1991), vehicle following and lane-changing (Gipps, 1981; 1986). These are also collectively known as "headway models," because the individual car movement relates to the headway distance between the two cars (Hammad 1998).

2.2.1 Car-following models

Car-following models were developed to model the motion of vehicles following each other on a single-lane without overtaking (Pipes 1953). These deal with the inter-vehicle relationship and from this basis build up a description of the total vehicular flow. Car-following models form the basis of microscopic simulation models and explain the behaviour of drivers in a platoon of vehicles.

According to Herman (1959) "follow the leader" type of problems, which form the basis of car following theory are the result of psychological behaviour of the drivers as they respond to certain stimuli. Car-following theory tries to mathematically describe, this behaviour. The general form of the car following models can be written as

$$\mathrm{Re}sponse(t+T) = SensitivityCoefficient * Stimulus(t) \qquad (2.1)$$

The nature of the response is acceleration or deceleration of the following car and the stimulus is the difference in velocity between the leader car and the follower. The Sensitivity coefficient can be considered as a constant value for a certain road with a certain traffic stream.

Classical car-following theory, Gazi et al (1961), is represented by the following key equation.

$$a_n(t+T) = \alpha \frac{(v_n(t))^m * \Delta v(t)}{(\Delta x(t))^l} \tag{2.2}$$

Where a_n is the acceleration of vehicle n at time t+T, $\Delta v(t)$ is the speed difference between the leading car and the following car, $\Delta x(t)$ is the gap between the leading vehicle and the following vehicle, $v_n(t)$ is a relative speed and α, m ,l are empirical constants. Equation (2.2) is better known as the GHR model because Gazis, Herman, and Rothery (1961) introduced the parameters *m* and *l* into the sensitivity term of the car-following model. For this equation, they also classified traffic flow conditions into congested and non-congested in order to investigate variable relationship. Several similar investigations followed subsequently in an attempt to define the `best' combination of m and l.

May and Keller (1967) has examined for non integral values for m and l. Fitting data obtained on the Eisenhower Expressway in Chicago they proposed a model with $m = 0.8$ and $l = 2.8$. Various values for *m* and l can be identified in the early work on steady-state flow and car following.

The case $m = 0$, $l = 0$ equates to the "simple" linear car following model. The case $m = 0$, $l = 2$ can be identified with a model developed from photographic observations of traffic flow made in 1934 (Greenshields 1935).

Heyes and Ashworth (1972) introduced a new model to look into the relationship between stimuli and response. In their model, the stimuli and the sensitivity term were denoted as $\frac{\Delta v}{\Delta x^2}$ and Δt^p, respectively, and the sensitivity term was estimated as 0.8 based on the survey field data of the Mersey tunnel in UK.

Ceder and May (1976) classified the traffic flow conditions into two groups, congested and non-congested condition, and for each group, they estimated the parameters, *m l*, from a large data base of real data.

Aron (1988), tested drivers' response in various traffic conditions, classified the drivers' responses into 3 types: deceleration, constant speed, and acceleration responses.

Ozaki (1993) investigated the sensitivity term with the data filmed from a 32 story building, and he analysed the time series data less than ten seconds due to the technical limitation of video film. From this analysis, he showed that the driver has a different sensitivity for acceleration and deceleration.

Bando et al. (1995) argued that there are two types of theories on the regulations of car-following. The first type is based on the assumption that the driver of each vehicle seeks a *safe following distance* from its leading vehicle, which depends on the relative velocity of the two successive vehicles. The second type of theories assumes that the driver seeks a *safe velocity* determined by the distance from the leading vehicle. Based on the latter assumption, these authors proposed a car-following model, called the optimal velocity model (OVM), as follows:

$$\frac{dv_{n+1}(t)}{dt} = k[V(\Delta x) - v_{n+1}(t)] \qquad (2.3)$$

Where k is reaction coefficient, $V(\Delta x)$ represents the legal velocity of the following car. In Bando et al.'s model, the time lag of any response is not included.

Helbing and Tilch (1998) carried out a calibration of the OVM with respect to empirical follow-the-leader data. They found that an extremely short relaxation time, T=1/k, could result in a very high value of acceleration, which led to an overshooting of the vehicle velocity. To improve the OVM, they developed a generalized force model (GFM). They believed that when $\Delta v < 0$, it is necessary to consider the acceleration caused by the relative speed of the successive cars.

According to Treiber et al. (1999), there exists in the real world common driver behaviour that none of the existing car-following models can explain. That is, when the distance between two vehicles is shorter than the safe distance, the driver of the following vehicle may not decelerate if the preceding vehicle travels faster than the following vehicle because the headway between the two vehicles will become larger.

Jiang et al (2002) thus more recently proposed an improved car following model as follows:

$$\frac{dv_{n+1}(t)}{dt} = k[V(\Delta x) - v_{n+1}(t)] + \lambda \Delta v \qquad (2.4)$$

This model considered the effects of *both* the distance between the leading and the following vehicles, and relative speed of two successive vehicles, which is more realistic and exact than those previous ones. Notice that this improved model combines both the classical car-following model in Eq. (2.2) and the OVM in Eq. (2.3).

2.2.2 Queueing theory

Queueing theory can be considered in terms of microscopic models although the target of the theory is not specifically individual car movement but the waiting line. Queueing theory involves use of mathematical models to study properties such as delay time or length of the queue and hence provides a framework for assessing *efficiency of road management features and their performance.* Tanner (1962) first modelled the un-signalised intersection using queueing theory and the method has since been intensively used to study traffic behaviour at intersections with and without traffic light. His approach has been improved subsequently, with many important contributions due to Yeo and Weesakul (1964), who noticed that Tanner's model was a special case of a queueing model called M/G2/1[ii]. Hawkes (1968), generalised Tanner's model in a different way, avoiding the somewhat unrealistic assumptions of the Yeo and Weesakul paper. Cowan (1987) introduced a generalisation of Tanner's model to yield the only model with an arrival process on the minor road which is not Poissonian.

Further, Heidemann and Wegmann (1997) studied the traffic phenomena at un-signalised intersections by means of an M/G2/1-queueing model which takes into account the different service time distributions for minor stream vehicles which come

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[ii] M/G2/1- queue elaborated in on page-23

into the first waiting position immediately, (non-queuers), and those which have to join the queue (queuers). Both consistent and inconsistent driver behaviour is treated. Moreover, it is shown how several streams of higher priority can be incorporated via the determination of busy periods of an M/G2/1-queue. Distributions of delays and queue lengths for the minor road vehicles are obtained for all these situations.

Moreover, Vandaele et al. (2000) showed that queueing models can also be used to explain uninterrupted traffic flows and thus offer a more practical approach, useful for sensitivity analysis and forecasts. In this paper, queueing models are referred to using the Kendall notation, consisting of several symbols, e.g. GI/G/z. The first symbol is the distribution of inter-arrival times, the second for the distribution of service times and the last one indicates the number of servers in the system. The *M/M/*1 queueing model (exponential arrival and service rates) is considered as a base line case, but due to its specific assumptions regarding the arrival and service processes, it is not useful to describe real-life situations. Relaxing the specifications for the service process of the *M/M/*1 queueing model, leads to the *M/G/*1 queueing model (generally distributed service rates). Relaxing both assumptions for the arrival and service processes results in the *GI/G/z* queueing model (with *z* being the number of servers). More-over, following Jain and Smith (1997), a state dependent *GI/G/z* queueing model is considered. This model assumes that the service rate is a linear exponential function of the traffic flow. In this case vehicles are "served" at a certain rate, which depends upon the number of vehicles already on the road.

Recently Flannery et al. (2005) studied single-lane roundabout, based on analytical models to estimate the mean and variance of service time for a driver and was able to accommodate any distribution of headway times in the circulating stream. The model can easily compute the performance metrics on a personal computer. These analytical models can subsequently be applied in an M/G/1 queuing model of the roundabout approach to compute the desired performance measures, namely the average delay experienced by an arbitrary driver arriving at the roundabout attempting to enter the circulating stream.

2.2.3 Review of cellular automata models

Cellular automata are usually defined to be *mathematical idealisations of physical systems in which space and time are discrete, and physical quantities take on a finite set of discrete values* (Wolfram 1986). A cellular automaton consists of a regular uniform lattice (or "array"), usually finite in extent, with a discrete variable at each site ("cell"). The states of a cellular automaton are completely specified by the values of the variables at each site. The variables at each site are updated simultaneously, based on the values of the variables in their neighbourhood and the previous time step, and according to a definite set of "local rules".

Rule 184 is a one-dimensional binary automaton rule of Wolfram code and can be used as a simple model for traffic flow in a single-lane road. The name for this rule, Rule 184, is the Wolfram code describing the state table above: the bottom row of the table, 10111000, when viewed as a binary number, is equal to the decimal number 184 are shown in Table 2.1. Rule 184 automaton applies the following rule to determine the new state of each cell, in a one-dimensional array of cells:

Table 2. 1- Wolfram code describing the state of the table

Current pattern	111	110	101	100	011	010	001	000
New state for centre cell	1	0	1	1	1	0	0	0

According to Wolfram's 184 rule (Figure 2.1), if we interpret each 1 cell in rule 184 as containing a particle, these particles behaves in many ways similar to automobiles in a single-lane of traffic: they move forwards at constant speed if there is open space in front of them, and otherwise they stop. Traffic models such as 184 and its generalisations that discretize both space and time are commonly called *particle-hopping* models (Nagel, 1996; Chowdhury et al., 2000). Cellular automata models are a type of particle hopping models, and hence are of microscopic nature while being quite efficient in computational time, (Wagner, 1996).

...	1	1	1	0	1	0	1	1	...

Figure 2. 1-Rule 184 interpreted as a simulation of traffic flow. Each 1 cell corresponds to a vehicle, and each vehicle moves forwards only if it has free cell in front of it

2.2.3.1 Single-lane cellular automata models

The first cellular automata model for traffic simulation was proposed by Nagel and Schreckenberg (1992) (referred to as the NaSch model or *stochastic* cellular automata model). The model used a cellular automaton for a microscopic *homogeneous* traffic flow for single-lane high-way traffic. The rules, which contain stochastic elements, were developed to update the movement of the vehicles. The authors showed that CA concepts had good potential to model microscopic traffic flow; as these were models which are capable of representing individual vehicle interactions. In the basic model, a car has a non-negative velocity (No. cells/ unit time). The model is defined as a one-dimensional array of L sites, each of length 7.5m (representing the length of the car plus distance between cars in a jam) under periodic boundary conditions. This means that the total number of vehicles N in the system is maintained at constant level. The density of a road can be calculated as $(\rho) = \frac{N}{L}$. Each site may either be occupied by one vehicle, or it may be empty. Each vehicle has an integer velocity with values between zero and v_{max}, where $v_{max} = 5$ in general. The velocity thus corresponds to the number of sites that a vehicle advances in one iteration. The movement of vehicles through the cell is determined by a set of updating rules, which are applied in a parallel fashion (i.e. simultaneously) to each vehicle at each iteration.

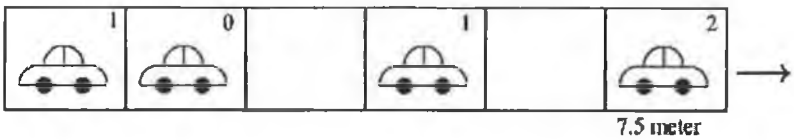

Figure 2. 2- A typical configuration in the NaSch model. The number in the upper right corner is the speed v_n of the vehicle.

The update system consists of four steps, which are:

Step1: Acceleration

For all cars that have not already reached the maximal velocity v_{max}, the speed is advanced by one unit i.e., $v_n \to v_n+1$

Step2: Deceleration (due to other cars)

If a car has d empty cells in front of it and its velocity v (after step 1) is larger then d, then the velocity reduces to d i.e., $v_n \to \min(d_n, v_n)$

Step3: Randomization

If after the above steps the velocity is larger than zero (v>0), then with probability p, the velocity is reduced by one unit i.e., $v_n \to v_n-1$

Step4: Vehicle movement

Each vehicle is moved forward according to its new velocity, determined in steps 1-3, i.e. $x_n \to x_n + v_n$.

where, v_n and x_n denote the velocity and position of the nth vehicle.

The NaSch model is a minimal model in the sense that any simplification of the rules no longer produces realistic results; additional rules are needed to capture more complex situations. Step 1 reflects the general tendency of the drivers to drive as fast as possible without crossing the maximum speed limit. Step 2 is intended to avoid collision between the cars. The randomization in Step 3 takes into account the different behavioural patterns of the individual drivers, especially nondeterministic acceleration as well as overreaction while slowing down; this is crucially important for the spontaneous formation of traffic jams.

The randomisation step takes into account three different properties of human driving In particular Nagel (1996) indicated that the randomisation (step-3) condenses three different properties of human driving into one computational operation. The three different properties are "fluctuation" at maximum speed, over reaction at breaking and retardations during acceleration. It turns out that overreactions in braking are crucial for spontaneous jam formation (Schadschneider, 2000). The order of the steps is important, too. If, for example, the order of steps 2 and 3 is changed, no overreactions can occur and consequently no spontaneous formation of jams. The parallel updating (instead of random sequential updating) takes into account the reaction time of drivers and may lead to a chain of overreactions. The jam formation mechanism cannot be modelled with the random sequential updating.

The NaSch model is a probabilistic cellular automaton, which in the case of vmax = 1 and the *deterministic* limit (randomisation probability p = 0) is equivalent to CA rule 184 in the Wolfram notation (Wolfram, 1983).

We start in this thesis with the deterministic CA model for *heterogeneous* traffic flow in which these two possible speeds i.e 0 or 1. Multi speed models are crucial when modelling free way traffic (Nagel et al., 1992).

Despite the simplicity of the NaSch model, the model is capable of capturing some essential features such as the basic structure of the flow-density relation (fundamental diagram), and start-stop waves do appears in the congested traffic jams. However, the NaSch model does not try to describe traffic flow very accurately on a *microscopic* level, (Barlovic et al., 1998). It also does not exhibit metastable states (slow-to-start rules) of high flow and synchronized[iii] traffic.

Barlovic et al.(1998) found metastable states in their velocity dependent randomization (VDR) which is an extension of the NaSch model. The one-lane VDR model belongs to the class of CA models with "slow-to-start" rules. The slow-to-start models differ somewhat from each other, but they share the use of most of the NaSch model steps. Their acceleration steps differ from each other (and from the NaSch model): the braking percentage for standing vehicles is different (larger) than for other vehicles. This usually causes the standing vehicles to start moving later than in the NaSch model.

2.2.3.2 Multilane traffic flow models

The first approaches to modelling lane-changing behaviour for two-lane road traffic using CA model, is due to Nagatani, where his work was based on the deterministic CA-184 model (Nagatani, 1993). One of the artifacts of his lane-changing rules was the existence of states in which blocks of vehicles alternated from one lane to another, without moving at all. To circumvent this problem, Nagatani randomised the lane-changing behaviour (Nagatani,1994). Rickert et al (1996) later applied this lane-changing methodology, where the update step is divided into two sub-steps, viz:

- the exchange of vehicles between two lanes is checked, and the vehicles wanting lane-change are moved sideways, (but not advanced), this sub-step is performed in parallel for all vehicles
- independent one-lane updates for forward movement are made using normal one-lane update rules.

The sideways lane-changing is supposed to happen within one time step, which is

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[iii] Kerner et al., (2003) introduced a three phase traffic flow theory. The three phases are **1. Free flow:** In the free flow phase the interaction between the vehicles can be neglected. Every car can move with its desired velocity. Therefore the flow increases linearly with the density of cars . **2.Synchronized flow:** Forms of congested traffic which cannot be classified as wide jams constitute synchronized flow. Here the average velocity is significantly lower than in free flow. Nevertheless the flow can be much larger than in wide moving jams, 3. **Wide moving jam**: Jams can form spontaneously, i.e., without any obvious external reason like an accident or road construction. Wide (moving) jams are regions with a very high density and negligible average velocity and flow.

usually one second in CA models. In reality the time is a bit longer. If there are more than two lanes (in one direction), the conflict of vehicles trying to change from left and right to the same cell in the middle lane can be avoided by determining, for example, that all changes from left to right are allowed to be made during odd time steps and from right to left during even time steps.

There are several reasons for lane-changing but also different prerequisites that must be fulfilled before the change can be initiated (Rickert et al., 1996; Nagel et al., 1998). These can be expressed as

- there must be incentive for lane change
- legal constraints must be fulfilled
- security issues have to fulfilled

Legal constraints usually vary in different countries. In many European countries, lane usage is governed mainly by two laws: the right lane has to be used by default and passing has to be done on the left. In the United States, on the other hand, the second of these laws is considerably relaxed; passing on the right is not explicitly forbidden (Nagel et al., 1998). This assumes right side of road driving in some European and US cases. Security issues concerning lane-changing include the need to have for having enough space on the target lane relative to the position of the vehicle wanting the lane-change. The lane-changing incentive of more 'European type' consists of changes from left to right and right to left and are handled somewhat differently. A vehicle reaching a slower vehicle on the same lane has to change to the left lane for passing. Also, when passing is not allowed from the right and there is a slower vehicle on the left lane, the vehicle on the right lane has to change lanes and get behind the slower vehicle.

Cellular automata models have been employed to represent several traffic scenarios from rather simple ones, such as representation of highway traffic, to more complex ones, such as the simulation of lane reduction scenarios, (Nassab et al., 2006), aggressive lane changing vehicle behaviour (Li et al., 2005) as well as the simulation of mixed traffic, (namely passenger cars and motorcycles), by introducing a cell width parameter (Lan and Chang, 2003, 2005) as motorcycles use less lateral space than passenger cars (Nagaraj et al., 1990; Singh, 1999; Lan and Hsu, 2006). Cellular automata rules have also been used to simulate pedestrian (Blue and Adler, 2001; Burstedde et al., 2001) and railway traffic (Li et al., 2005). A different development on cellular automata models is their employment into designing simulation models using multi-agent technique. Such simulation models have been constructed for vehicular traffic simulation, (Wahle and Schreckenberg, 2001; Wahle et al., 2002) but also for pedestrian simulation (Dijkstra et al., 2000).

2.3 Urban traffic flow modelling

Traffic modelling on freeways as opposed to urban networks requires a different context. Firstly, traffic flow dynamics are clearly different, since the normal situations on urban roads, such as stopping and turning, are not allowed on freeways. Stops belong to special events that only happen when a crash or traffic jam occurs. However in urban networks, crashes and jams are not the main reason for stopping. In an urban area, this is typically due to vehicle manoeuvring and queuing, traffic lights, and driver behaviour. Turns are inevitable in driving on urban roads. In contrast, turns on freeways often follow the geometrical shape of the freeway and this turning does not change the components of traffic flow.

Secondly, the geometrical configurations of freeway and urban networks are different, with that of the freeway much simpler. There are entrances and exits, mostly to one side and only one road direction. For urban networks in contrast, there are junctions with or without traffic lights, single, double and multi-lanes, single and multi-directions on urban roads, roundabouts of different size and so on.

Basis: The urban network level of traffic modelling was originally based on the two fluid theory of town traffic (Herman and Prigogine 1979, Herman and Ardekani 1984). The theory relates the average speed of moving cars to fraction of running cars in a road network. Hydrodynamics models are, however, hard to apply in urban networks because of the many differently directed currents of traffic involved and intersections and traffic lights are difficult to translate into hydrodynamic language, (Lehmann 1996).

Further, car-following theory may only be used separately on each road of an urban network, so it does not help much for networks as whole, as the dynamic for a network relies on traffic light intersections rather than the gap between vehicles. Car-following, like the intersection analysis, is one of the basic questions of traffic flow theory and simulation, and still under active analysis after almost 40 years from the first trials (Wu et al. 1998).

The aim of urban network modelling is, typically, to explore congestion on urban roads. Intersections are the "bottlenecks" of the whole network, so modelling has focused on the intersections. Various types of intersections with or without traffic lights have been studied, (Esser and Schreckenberg 1997, Chopard et al. 1998), but a full consideration of these and several other road features is also needed. One of the main efforts in relieving congestion is to improve traffic control strategy. Traffic light strategies e.g. have been investigated by topological methods, (Cremer and Landenfeld 1998).

Recognising the importance of urban road configuration in determining flows, a CA "ring" was firstly proposed for unsignalised intersections (Chopard et al. 1998). All entry roads are "connected" on ring. The car "on the ring" has priority over any new entry. However, there is no differentiation between the major and minor entry roads and all vehicles have equal priority to move into the ring (intersection), which two-way stop-controlled intersection (TWSC) usually has rules. A further CA model variant for intersections is described by Esser, and Schreckenberg (1997).

2.3.1 Un-signalised (Stop controlled) intersection

Un-signalised or stop controlled intersections are the most common type of intersection in an urban area. They give no positive indication to or control of the driver. The driver alone must decide when it is safe to enter the intersection; typically they look for safe opportunity or a gap in the conflicting traffic. This model of driver behaviour is called "gap acceptance". The "gap" is measured in units of time and corresponds to *headway*, (defined as distance divided by speed). Critical gap and follow-up time are two major parameters used in various gap acceptance models. The *critical gap* is defined as the minimum time interval between two major-stream vehicles required by one minor stream vehicles to pass through. By a critical gap we shall mean that the minimum gap a driver can accept when crossing a traffic stream .The *follow-up time* is the time span between two departing vehicles, under the condition of continuous queueing.

The hypothesis of the car-following model is that a driver reacts to the stimuli from the surrounding traffic by choosing an acceleration (positive or negative), which is proportional to two components: the difference between the preferred time gap and the actual time gap, and the difference between the preferred speed and the actual speed.

Gap-acceptance models are unrealistic in general, assuming that driver are consistent and homogeneous (Tanner, 1962). A *consistent driver* would be expected to behave in the same way in all similar situations, while in a *homogeneous* population, all drivers have the same critical gap and are expected to behave uniformly, (Plank and Catchpole, 1984). In any simulation, however, driver type may differ and critical gap for a particular driver should be represented by a stochastic distribution such as that initially suggested by Bottom and Ashworth (1978).

Shortcomings of gap acceptance and average gap distance and critical gap have been addressed by **MAP** (Minimum Acceptable sPace) and **MMAS** (Multi-stream Minimum Acceptable Space) type models. Initially developed, Wang and Ruskin (2003), these new cellular automata model dealt with un-signalised intersections with manoeuvres based on minimum acceptable space (MAP) rules, (which allow both spatial and temporal updating). MAP also permits different driver behaviour to be accounted for, assigning probabilities to driver types (non-uniformity), distinguishing between e.g. conservative, urgent, radical and rational, with specified probabilities associated with making a move at each time step. These models, extended to include MMAS,(Wang and Ruskin, 2006) also allow for *inconsistency* associated with each driver types, with random changes of behaviour possible over time. The models can successfully simulate both *heterogeneous* and *inconsistent* driver behaviour and interaction between drivers for different traffic conditions and a variety of urban and inter-urban road features.

The MAP and MMAS approach are reasonable for a stop controlled configuration, as these are based on the notion that different drivers require different space allowances to enter an intersection and roundabout. Configurations controlled by traffic light are less directly influenced by driver behaviour although some add-on to traffic light control time is likely. The additional factor such as aggressive behaviour at control intersection has been examined in detail in this thesis. In general, MAP method, considered in this thesis is assume that a rational driver for a two-unit long vehicle requires the same space as a conservative car driver in the homogeneous traffic.

2.3.2 Traffic flow at signalised intersections

The operation of signalised intersection is often complex, involving competing vehicular and pedestrian movements (Roess et al., 2004). Traffic signal controls are implemented for the purpose of reducing or eliminating conflicts at intersections. These conflicts exist because an intersection is an area shared among multiple traffic streams, and the role of the signal system is to manage the shared usage of the area. Signals accomplish this by controlling access to the intersection, allocating usage time among the various users. The logic for this allocation can vary from simple time-based methods to complex algorithms which calculate the allocation in real time based on traffic demand (Davol, 2001).

There are essentially two distinct methods of specifying basic signal control logic. The method that is standard in the United States is based on "phases," while the method standard in much of Europe is based on "signal groups." (FHWA, 1996; EB Traffic, 1990). In traffic signal operation, specified combinations of movements receive right-of-way simultaneously. A "phase" is the portion of the signal timing cycle that is allocated to one of these sets of movements. Each phase is divided into "intervals," which are durations in which all signal indications remain unchanged. In most European countries, specifically Ireland, a phase is typically made up of three intervals: *green, yellow*, and all *red*. A phase will progress through all its intervals before moving to the next phase in the cycle.

2.3.2.1 Control type

There is wide range of logic by which signal phasing and timings can be controlled. This logic can be *pretimed*, *actuated*, or *adaptiv*e.

Pretimed control is the most basic type of control logic that can be implemented. In pretimed control, the cycle length and the phase splits are set at fixed values, as are the durations of each interval within each phase. Historical flow data is typically used to determine appropriate values for these parameters. The key attribute of pretimed control is that the logic is not demand-responsive, meaning that the signals operate without regard to fluctuations in traffic demand, (Davol, 2001).

Actuated control uses demand-responsive logic to control signal timings, with phase durations based on traffic demand as registered by detectors on the intersection approaches. The most common feature of actuated control is the ability to extend the length of the green interval for a particular phase, (Davol, 2001). The interval might be extended, for example, when a vehicle is approaching a signal that is about to change to yellow, allowing that vehicle to pass through the intersection without stopping.

Adaptive control, like actuated control, responds to traffic demand in real time, but its logic can change more parameters than just interval length. The most common adjustments made are to the cycle time and to the phase splits, which determine the allocation of the cycle time to the various phases. These strategies rely on traffic data collected for each approach upstream of the intersection, and this data is used by the controller to estimate conditions at the intersections and to respond to them in real-time.

This logic is often optimization-based, allocating green time to maximize measures such as vehicle throughput or to minimize measures such as vehicle delays or stops. Adaptive logic can also be predictive, projecting future conditions based on detector inputs and historical trends and adjusting signal settings accordingly. Adaptive traffic control systems are becoming more widespread, both in application and in development, (Davol, 2001). Urban traffic control systems such as SCOOT are implemented widely (Bretherton, 1996), and applications of systems such as OPAC and UTOPIA are also becoming more prevalent (Gartner et al., 1991; Peek Traffic, 2000).

2.4 Heterogeneity in traffic

The traffic in *developing countries* is mixed, with a variety of motorised vehicles, using the same right of the way, (Khan and Maini, 1999). The *motorised* or fast-moving vehicles include passenger cars, buses, trucks, auto-rickshaws, scooters, and motorcycles; *non-motorised* or slow-moving vehicles include bicycles, cycle-rickshaws, and animal-drawn carts. Many *industrialised countries* face problems that concern congestion caused not so much by diversity of traffic type but by the increasing volume of passenger cars and lorries (NRA, 2003) and there is a need to anticipate future requirements of the infrastructure. In developing countries, motorisation is also on the rise. Larger-sized motorised vehicles may still be a smaller proportion of the overall traffic, but have effects in excess of volume, for off arterial flow in urban roads. The differences that characterise mixed traffic systems, otherwise known as *heterogeneous traffic*, are mainly due to the wide variation in size, and manoeuvrability, as well as static and dynamic properties.

Heterogeneity models for developed world: Traffic in the developed world, is normally a mix of several kinds of motorised vehicles, e.g., cars, buses, vans and trucks. These vehicles, with different dimensional and dynamic properties form heterogeneous traffic flow, which has been studied to some degree, although not in detail, (Evans, 1997; Treiber et al., 1999).It can be shown that mixed traffic flow can lead to platoon[iv]

[iv]A platoon is a number of vehicles travelling together as a group, either voluntarily or

formations at low densities in single-lane traffic. The effects of mixed vehicle lengths on traffic flow have also been studied in an asymmetric exclusion model (Ez-Zahraou et al, 2004). Based on this model, the authors concluded that the maximal flux decreases when increasing the number of long vehicles. Treiber and Helbing (1999) study heterogeneous traffic flow in congested states. The actual proportions of cars and trucks known from real traffic data were used in the macroscopic traffic model developed by these authors. The simulation results show a high level of agreement with Dutch highway data.

Most of the above models were, however, studied from the macroscopic viewpoints and did not emphasise the effect of vehicle length on the urban road configurations.

Heterogeneity models for developing world: Heterogeneous urban traffic models have already been reported by some authors for Asian regions and include: the first-order second moment method, Arasan and Jagadeesh (1995), used to estimate the saturation flow and the delay caused to traffic at signalised intersections under heterogeneous traffic conditions i.e. with vehicles of wide-ranging static and dynamic characteristics. Moreover, the size of vehicles varies widely, and the lateral and longitudinal placement of vehicles on carriageway is complex, with no discernible lane discipline.

Further, Marwah and Singh (2000) developed a simulation model for heterogeneous urban traffic, (mostly those extended types of traffic flow observed in the Asian-Pacific region), using field data from different roads in the Kanpur metropolitan area in India. Through this study, the Level of service (LOS) experienced by different categories of vehicles was determined when the traffic stream contained 65 percent non-motorised vehicles. LOS as a metric of performance is a composite of several operating characteristics that are supposed to measure the quality of service as perceived by the user at different flow levels. The operating characteristics, considered to define the LOS are: journey speeds of car and motorised two-wheelers; concentration; and road occupancy. The authors evolved the LOS classification to identify deficiencies of urban road system and to plan for alternative improvement measures to attain a desired level

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involuntarily because of signal control, geometrics or other factors (HCM, 2000).

of service.

Additionally, Gundaliya et al.,(2004) and Mallikarjuna and Rao (2005) have studied the heterogeneous traffic flow for Indian roadways using cellular automata. Their models have essentially a multi-cell structure, but now the multi-cell concept is extended in the lateral direction (i.e. to multiple occupation of a single cell). So cells not only get smaller, but also 'thinner', allowing *variable-width vehicles*, e.g., motor cycles that can more easily pass other vehicles in the same lane. Maerivoet and Moor, (2005) believe that such a scheme directly opposes the idea behind a CA model. They strongly feel that heterogeneity in a traffic flow, using a CA model should *only* be incorporated by means of *different lengths*, *maximum speeds*, *acceleration characteristics, anticipation levels, and stochastic noise* for distinct classes of vehicles and/or drivers. Any other approach, they state, would be better off with a continuous microscopic model. Here we use different length but not width of the vehicles to model heterogeneous traffic flow. The evolution rules, set up for heterogeneous traffic, were explained in detail in Chapter 4.

2.5 Summary

In this chapter we presented a review of main traffic flow models and common methods used to represent macroscopic and microscopic behaviour. In particular, for the class of macroscopic models we discussed the LWR and other analytical models. For microscopic models we discussed the car following, queueing theory and Cellular automata models (in more detail). Due to the stochastic nature of traffic flow and its nonlinear characteristics, the cellular automata approach has received much attention since 1990 and various versions have been applied.

Furthermore, in this chapter some heterogeneous traffic flow models are discussed which include heterogeneous models for the developing world and heterogeneous models for the developed world (for highway traffic in general). The basis for the work on heterogeneous traffic, under Western world assumptions, which is developed here, is thus the cellular automata approach. The focus is binary traffic mix and details are discussed in the next chapter.

Chapter 3: Dublin traffic data

3 Dublin traffic data

Historical justification for road design and control mechanisms was the continuous free-flow of traffic. However, as urban areas continue to grow, together with levels of car ownership and increase in public transport and goods vehicle usage, the road network has become increasingly congested. *Intelligent Ttransportation Systems* (ITS) vary in technologies applied, from basic management systems such as *traffic light* control systems, variable message signs or *speed cameras* to monitoring applications such as security CCTV systems, used in obtaining additional capacity for new and existing roads and to improve safety and transportation time. There are to date several ITS projects implemented on road networks in both developed and developing countries (based in *major cities*). Countries which have implemented ITS systems include Ireland. The SCATS (Sydney Coordinated Adaptive Traffic System) is used in Dublin City. Nevertheless, the online live data collected by Dublin City Council (DCC) through its SCATS system is only the total volume data. In order to monitor traffic composition on the road network, DCC captures data *manually* for some randomly chosen signalised intersections and un-signalised roundabout over 15-minutes intervals for 10-hours.

Traffic counts conducted by DCC manually these are conducted typically to gather data for determination of vehicle classification as a proportion of traffic volume, turning movements (percentage of left turning (LT), straight through (ST) and right turning (RT) vehicles) and numbers of vehicles on the selected road (total volume) for the given period. These data provide the baseline figures for use in a traffic impact analysis or traffic control device evaluation. For example, measurements are being made by NRA (National Roads Authority) Ireland in cooperation with DCC, of daily average directional volume at the end of each hour for heavy goods vehicles as well as the total volume. Further, the TCD group (TRIP centre- see e.g O'Mahony, 2004) have analysed the percentage of urban freight for the Greater Dublin area. Other methods of data collection, also include for example image analysis,(used to detect the movement of vehicles by virtual sensor placed in the computer images)

According to information of NRA and DCC, automatic vehicle counters provide information on the volume of traffic such as those above, by the hour or minute of the day and also on vehicle class i.e. motorcycle, car, goods vehicles, (distinguished usually by number of axles). Unfortunately, there is only a small amount of manual data available on road configurations. This is because the automated collection process (i) deals with straight roads i.e. no turning, (ii) deals with signalised road features only (iii) is limited to current control patterns and (iv) does not account for driver types.

The focus is thus usually to provide information on the effectiveness in handling volume of the controls already in operation, rather than as a means of assessing or predicting overall impact of signalised verses un-signalised features. We have taken manually collected data, a base line in our models and as on empirical basis for subsequent sensitivity analysis. This real data and the model representations of the flows are presented in the following section.

3.1 Urban configurations: Dublin examples

The following are examples of configurations, proposed for modelling in the thesis. We focus "symmetrical configurations", as for field data collected for particular local intersections and roundabouts. In this instance "symmetrical" means a configuration with equal number of entry and exit roads and with similar control rules on each. Entry roads are labelled road 1 to 4, with major and minor indicated for both controlled and stop-controlled intersections and roundabouts. The operation of these configurations is described in detail in the following chapters but flows are shown below. The shaded area in Figures 3.1, 3.2 and 3.3 are the intersection and roundabout operational areas for manoeuvres. In the case of a signalised junction and roundabout, these are controlled by traffic lights, with a pre-determined cycle of green, yellow and red lights, (with the yellow light occurring twice per cycle). This is common to most European countries including Ireland, (Roess et al. 2004).

In the case of the TWSC (two-way stop controlled) intersection and "Yield sign" roundabout, these follow the "give-way" rule operations. We have also constructed signalised and un-signalised intersections and roundabouts and also looked at multi-lane situations. The intersections and roundabouts included in this study were examined with regard to performance measure such as throughput, capacity, and total delay time and queue length characteristics for a range of arrival rates, turning rates, traffic proportions and other factors.

Real data:

The following data, used to determine baseline parameter values, distributional characteristics and flows was collected at by Dublin City Council (DCC). The data were typically captured during 8.15 to 18.30. Different vehicles types were captured i.e. (1) cycles (pedal and motor); (2) cars; (3) lorries and (4) buses. This study was performed for short (cars) and long (bus or equivalent) vehicles only. Short and long vehicle composition is given in Table 3.1, 3.2, 3.3

3.1.1 Single-lane two-way controlled cross-road or four-way intersection

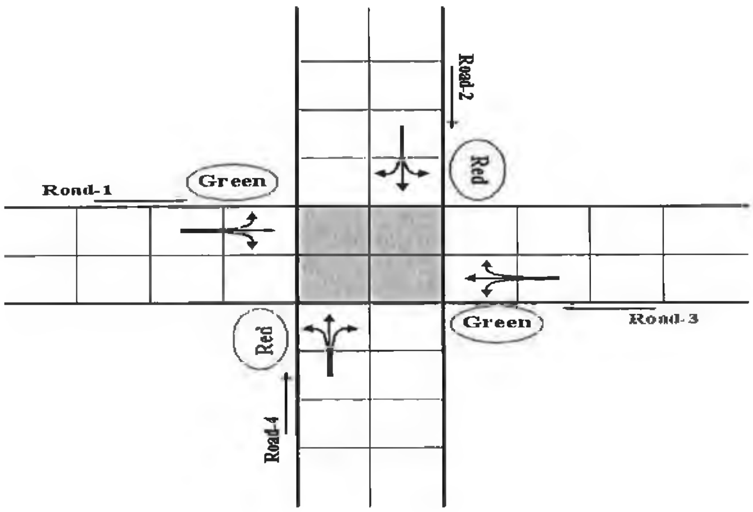


Figure 3. 1 Schematic of traffic flow at a single-lane two-way signalised intersection

A typical "local" single-lane two-way controlled intersection is depicted in Figure 3.1, which shows the actual intersection traffic flow and the model representation. Typical manual data for such a configuration are given in Table 3.1, (in this case for Rathgar road- Frankfort Avenue). For all data collected by DCC, specific weather and road conditions are recorded, as well as start and finish time of the collection period. Also noted is the number of large vehicles in the total count, where these are defined as those exceeding "standard length", (roughly taken to be single-cell occupancy of standard car or van for our purposes). The traffic composition, turning percentages and total number of vehicles at the study location is shown in the table for each entry road.

Table 3. 1- Single-lane two-way traffic light controlled cross-road (Rathgar road-Frankfort Avenue). Weather and road conditions fair on date recorded (Dec 17th 1997).

	Road-1	Road-2	Road-3	Road-4	Overall
Totals (SV+LV) for 10 hours	4937	2428	4941	2138	14444
Averages (SV+LV) per hours	494	243	494	214	1444
Averages (SV+LV) per seconds	0.14	0.07	0.14	0.06	0.4
Total SV	4703	2391	4678	2111	13883
% SV	95	98	95	99	96
Total LV	234	37	263	27	561
% LV	5	2	5	1	4
Total Left Turning (LT) vehicles	532	387	140	481	
% LT vehicles	11	16	3	23	13
Total Straight Through (ST) vehicles	4160	1569	4427	1526	
% ST vehicles	84	65	90	71	78
Total Right Turning (RT) vehicles	245	472	374	131	
% RT vehicles	5	19	7	6	9

The intersection is a four-way intersection (as for Figure 3.1). Overall, long vehicles (LV) account for about 4 percent of all vehicles. Overall, left turning (LT): straight through (ST): right turning (RT) vehicles account for 13%:78%:9% of all vehicles monitored for this intersection and the average overall "volume[v]" is 1444 vehicle per hours (vph) in this data capture. As such, these provide a baseline for turning rate distributions, volume of the traffic on the road and traffic composition, selected for our models.

3.1.2 Single-lane two-way controlled T-intersection

Similarly, for a T-intersection traffic flows are shown in Figure 3.2 and typical data, collected over a ten hour period under conditions, as described above, are given in Table 3.2

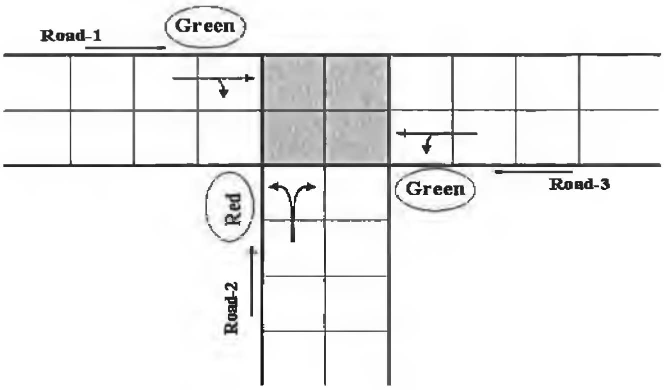


Figure 3. 2 - Schematic of traffic flows at a single-lane two-way signalised T- intersection

The schematic in Figure 3.2 assumes that the vertical bar of the T corresponds to the minor road. T-intersections are simpler than four-way intersections in several ways. A typical four-way intersection for example contains twelve vehicular movements, while there are only six for T-intersection, (as illustrated by arrows in Figure 3.2). Further in UK/Ireland LT manoeuvres are less "costly" in terms of cross traffic, while an extended intersection will inevitably include a more complex light cycle and hence additional delays. In particular terms and for the signalised T-intersection illustrated, RT vehicular movements from road-2 experience no opposed movement, as in four-way intersection. The four-way intersection overall traffic for road-2 LT: RT account for16 %: 19 % of total movement reported in Table 3.1. The T-intersection for road-2 LT: RT account for 85%: 15 % of 2956 of all traffic.

[v] Volume is typically used to describe the rate in vph

Table 3.2 data were collected manually on 30th April 2003. The overall percentage of LV and SV are 6% and 94% respectively. The percentage of RT vehicles in road-1 is lower than ST vehicles and similarly for LT and ST. Again in road-2 the percentage of LT vehicles is higher than that of RT vehicles.

Table 3. 2- Single-lane two-way traffic light control T-intersection (Oscar Traynor road-Dundaniel road). Weather and road conditions fair date recorded (30th April 2003).

	Road-1	Road-2	Road-3	Overall volume
Totals (SV+LV) for 10 hours	10117	2956	7196	20269
Averages (SV+LV) per hours	1012	296	720	2027
Averages (SV+LV) per seconds	0.3	0.08	0.2	0.6
Total SV	9501	2876	6652	19029
% SV	94	97	92	94
Total LV	616	80	544	1240
% LV	6	3	8	6
Total Left Turning (LT) vehicles		2502	456	
% LT vehicles		85	6	
Total Straight Through (ST) vehicles	7256		6740	
% ST vehicles	72		94	
Total Right Turning (RT) vehicles	2861	454		
% RT vehicles	28	15		

3.1.3 Single-lane yield sign controlled roundabout

In the schematic, Figure 3.3 depicts flows on a four entry/exit roundabout for single-lane road with Yield sign control. The roads intersect at right angles to each other. Yield lines mark the borders between entry lanes and the circular roadway with all four roads feeding and receiving entry/ exit flows respectively. Vehicles enter the roundabout at the left side of the road from all approaches. Before entry a vehicle must wait for a gap in the roundabout flow and then merge into the traffic on the roundabout, in a clockwise direction. The details of operation are discussed in Chapter-6. A vehicle continues around the Central Island until reaching its predetermined, but randomly assigned, exit point.

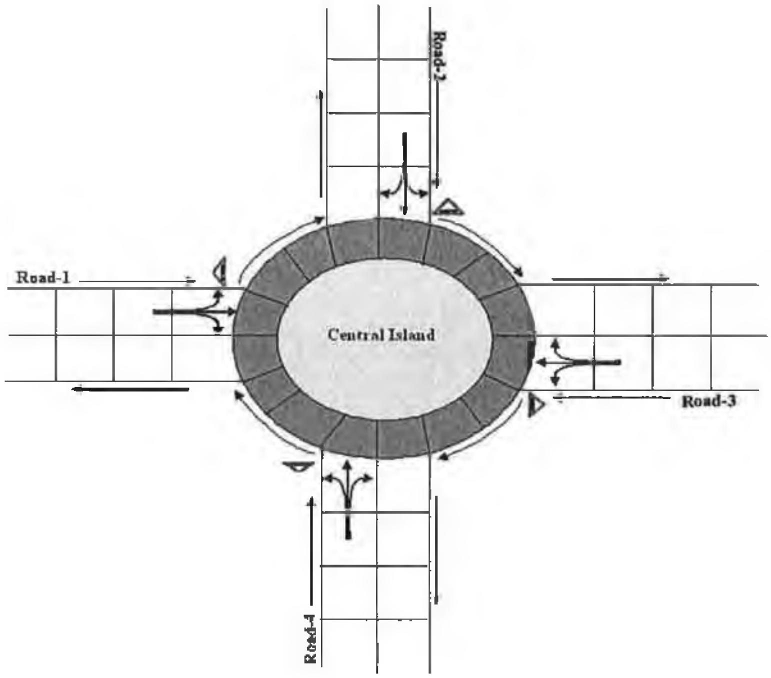


Figure 3. 3 Schematic traffic flow at a single-lane two-way yield sign roundabout

The data, illustrated were collected by DCC on 5th March 2003 at Kilmore road-Skellys lane roundabout. The percentages of LVs on the four entry/exit roads are similar, with the exception of road-3, with roads 1, 2, 3 and 4 shown to be 4%, 4%, 1% and 4% of total volume. Overall LVs account for 3 % of all vehicles and SVs for 97% of all vehicles passing through this configuration. ST vehicles account for the majority of manoeuvres when compared to RT and LT taken separately. The overall percentage of ST: LT: RT= 37: 34: 29. In the period mentioned a total of 17079 vehicles were observed to enter the roundabout from all four approaches. Data collected were limited to:

- Vehicle classification data
- Number of vehicle turning and moving straight through (i.e. LT, ST and RT)

It should be noted that, since the roundabout is yield sign controlled, there are no automatic traffic recorder volume data available. This is common to roundabouts of this size in Dublin City and serves to emphasise the point on management of traffic for urban conurbations where in 80% of such road configurations are "priority" or "yield" rather than signal controlled. This, despite traffic mixed traffic and relatively large total volume over the given period.

Table 3. 3- Single-lane two-way yield sign controlled roundabout (Kilmore road- Skellys lane). Weather and road conditions fair date recorded (5th March 2003).

	Road-1	Road-2	Road-3	Road-4	Overall volume
Totals (SV+LV) for 10 hours	5553	4561	2765	4200	17079
Averages (SV+LV) per hours	555	456	277	420	1708
Averages (SV+LV) per seconds	0.15	0.13	0.08	0.12	0.5
Total SV	5342	4376	2756	4037	16511
% SV	96	96	99	96	97
Total LV	211	185	9	163	568
% LV	4	4	1	4	3
Total left Turning (LT) vehicles	2038	1046	972	1740	5796
% LT vehicles	37	23	35	41	34
Total Straight Through (ST) vehicles	1510	1777	1172	1710	6169
% ST vehicles	27	39	42	41	37
Total Right Turning (RT) vehicles	2005	1738	621	750	5114
% RT vehicles	36	38	23	18	29

3.2 Conclusions

The percentage of LV and SV volume for particular urban configurations collected by DCC provided baseline information for example junctions and roundabout for testing. These data served the purpose of validating model forms for the configurations chosen, and also provided a basis for looking at sensitivity analysis of the flows to parameter changes , and hence to inform prediction. Sensitivity analysis here we mean as the systematic investigation of the reaction of the simulation output (or response) to change in the input values of the model. For example, what happen to the "overall throughput" of the intersection or roundabouts when change in arrival rate, traffic composition and turning rate; what happens if the right turning rate is very high in all roads approaches to the intersection? If there will be the change in input there will change in output so the model sensitive to the input data and the model reproduce the real world situation.

The limitation of this study is that only one data set available in the particular configurations. Therefore further experimental or observational studied needed.

Chapter 4: Research methodology and Single-lane one-way cross intersection

4 Research methodology and single-lane one-way cross intersection

The objective is to provide a basis to quantify the effect of inclusion of long vehicles in manoeuvres through intersections and roundabouts. Our basic model is the simplest version of the homogeneous CA model Wang (2003), adapted for heterogeneous vehicle mix, (limited to binary here). In this model we address relevant questions including: the basis for comparison with homogeneous flow, the effect of varying traffic light cycles and other means of control e.g. through priority sharing and the impact of different driver types. Specific issues, for example, relate to cross-traffic flows at intersections, particularly when the crossing streams are a mix of heavy and light flows, since these have major control implications.

4.1 Road configurations

Road configurations considered include: *single-lane one-way*, *single-lane two-way*, *multi-lane* controlled and uncontrolled *T* and *cross-intersections* and roundabouts. To examine traffic patterns for these, we consider intersections with a prescribed number of entrance/exit roads in these instances, though these can be of course varied. Thus configurations, as in the example above, are variously defined in terms of two, three or four entrance/exit roads, (see Figures, in Chapter 3). An "intersection" in its broadest interpretation of course, can connect many roads of varying sizes together, with or without benefit of traffic light control, (the multiple cross geometry is also commonly designated a "cross-road"). Roads are numbered, for convenience, in the Figure 3.1, 3.2, 3.3, 4.4 and 5.6 referred to, but this implies no additional ranking to that designated by major/ minor roads. Approach road lengths are also considered fixed for convenience in all model variants discussed.Total length of each approach road is taken to be 100 cells, excluding all intersection areas, (dark shaded areas manoeuvre in Figures mentioned above). At the beginning of each entry road, vehicles arrive randomly for given fixed arrival rate in a given simulation. In cases discussed in the thesis, we do not consider the possibility of varying arrival rates over a given run, although this has obvious relevance to the real-world situation. Each incoming vehicle approaching an intersection or roundabout (i.e. at 100^{th} cell of an entry road) is randomly categorised as left-turning (LT), straight through (ST) or right turning (RT) with different turning rates considered.

We model the incoming and outgoing roads; as soon as the vehicles leave the outgoing road, i.e. pass the 100th cell of the exit road they have left our simulation world. The output road is included in order to simulate the "complete system" flows.

The model discussed here is based on the assumption that exit lanes have infinite capacity and that there is no congestion on these exit lanes. Once a vehicle enters an exit road it will progress (using the model's rules i.e. the vehicles move only when the cell in front of them is free) and exit the model. An infinite number of vehicles can travel down the exit road with no consequence to the model's behaviour. When the model connect two junctions together as the exit of the 1st junction no longer has infinite capacity and a delay at the second junction may cause a build-up of traffic that will affect the behaviour at the 1st junction. Therefore passing out of the linked configuration could be investigated for further study.

4.2 General model specification and capability for road feature representation

Vehicle movement is defined by a cellular automaton for road progression and similarly, for roundabout and other intersection passage. Various adaptations apply specifically signalised and un-signalised intersection/roundabouts. The general technique partitions space into cells, with occupation by a vehicle updated via simple rules. To describe the states of a road using a CA, the road is first divided into cells of length 7.5m, (based on Nagel et al., 1992, 2002 and Section 2.4.). This corresponds to the typical space (car length + distance to the preceding car) occupied by a car in dense road packing. Each cell can either be empty or occupied by exactly one car. A speed of say, v=5 (Chapter 2, Section 2.2.3) means that the vehicle travels five cells per time step or 37.5 m/s (135 km/h).

In a homogeneous traffic model, each cell is occupied by one particle per cell corresponding to a standard car of length less then or equal to 7.5metres (Wang, 2003). In our model, long vehicles (LV) are taken, for simplicity, to be double the length of a standard car i.e. two cells are required for one LV (a short vehicle (SV) is thus of normalised length 1, while a LV is of length 2). Both SV and LV will move through exactly one cell in the next time step if the cell in front is vacant. The system uses parallel updating (Figure 4.1): All vehicles that have an empty cell in front of them at time t can move one cell; the result is the configuration for time t+1. Vehicles may traverse the road according to these simple principle, but on arriving at the configuration, can only initiate a manoeuvre :(right/ left turns etc.), when the traffic light is green, and/or when there is space on the intersection or roundabout.

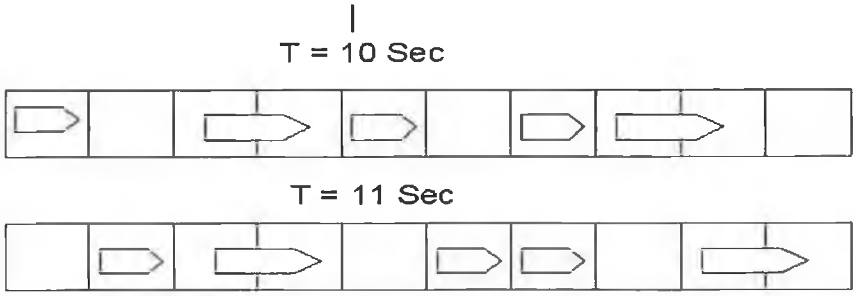


Figure 4. 1– Cellular automaton driving logic

This methodology and approach to studying various road configurations is presented in terms of description of its main features (following section).

4.2.1 Update rule: progression along entry road

The update rules defining progression along single-lane roads are as follows:

The state of a cell "n" on the road, at time't' is designated C^t_n. If C^t_n >0, there is a vehicle in n[th] cell at time step t. The updates of the cells are on a vehicle by vehicle basis i.e. if the C^t_n =1 (SV) in this time step and the cell in front is vacant (C^t_{n+1}=0) then the SV will move one cell, otherwise the SV will stay in the same cell in the next time step (t-> t+1). Similarly, if $C^t_n = C^t_{n+1} = 2$ (LV) in this time step and the cell in front is vacant (C^t_{n+2}=0) then the LV will move one cell in next time step. If C^t_{n+2} ! =0 (not equal to zero), no movement of the LV occurs.

The algorithm is:

- If $C^t_n = 1$ and $C^t_{(n+1)} = 0$, then $C^{(t+1)}_{(n+1)} = C^t_n$ and $C^{(t+1)}_n = 0$
- If $C^t_n = 1$ and $C^t_{(n+1)} > 0$, then $C^{(t+1)}_n = C^t_n$
- If $C^t_n = C^t_{(n-1)} = 2$ and $C^t_{(n+1)} = 0$, then

  $C^{(t+1)}_{(n+1)} = C^t_n$ and $C^{(t+1)}_n = C^t_{(n-1)}$ and $C^{t+1}_{(n-1)} = 0$
- If $C^t_n = C^t_{(n-1)} = 2$ and $C^t_{(n+1)} > 0$, then $C^{(t+1)}_n = C^{(t+1)}_{(n-1)} = 2$

All cells are viewed at time t before update and these update rules are applied to all entry roads simultaneously. In the following section we explained how priority is determined at configuration.

4.2.2 Traffic control scheme

At different types of urban configurations (e.g. one-way, two-way intersections and roundabouts) different forms of traffic control (e.g. yield or stop-signs, traffic-lights) apply.

***Traffic light signals: fixed time scheme*:** In our model we consider *two phases* for controlling different signalized configurations. In these schemes, traffic flow is controlled by a set of traffic lights, which are operated in a *fixed cycle* manner. Fixed-cycle intersections (or roundabouts) operate with a constant period of time T= 100 seconds for the full light cycle, where this is divided into a green, yellow and red periods for each phase, with proportionate period roughly as illustrated for all approaches.

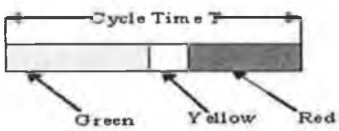


Figure 4. 2- Breakdown of a single fixed cycle

For example, we consider a typical four entry road intersection (or cross-road). In *phase-1* the traffic light is green for 55seconds for major road-1 and road-3 (simultaneously red for road-2 and road-4). In the second part, the lights change colour to yellow for 5seconds for major road-1 and road-3 and simultaneously change to red for road-2 and road-4. In *phase-2* the cycle repeats i.e. road-2 and road-4 become green for 35seconds with road-1 and road-3 red (for the same period).The lights then go to yellow for 5seconds for road-2 and road-4 and are simultaneously red for road-1 and road-3. The cycle then repeats. In more sophisticated traffic management schemes, it is also the case of course that the cycle is not fixed but varies according to the information fed-back on traffic volume through, e.g. sensor systems on the road. In principle incorporating such variability is well within the capability of the models developed. For example, the cycle can be programmed to change after fixing start times of the simulation to reflect a "day" with known congested periods. It can also be adapted to respond to random or quasi-random signals, indicating increased arrival rate, e.g. for one cross-stream. Implications for these variants and incorporation are also discussed in Chapter-2, Section 2.3.2.1.

***Stop control rules (Stop sign rule)*:** According to the stop control rule of the road, a vehicle from a minor road must stop before entering the intersection, no matter how quiet it might appear, Road Safety Authority (RSA) (RSA, 2007). In our simulation we assume a fixed two time-step delay, (equivalent to 2 seconds), for a two-way stop controlled (TWSC) intersection in order to permit checking of the two-way major stream flow. When a vehicle arrives at a stop-line from a minor road, the driver is assumed to look along interfacing vehicle streams for an acceptable break in the flow specified by gap acceptance or number of cells(in model terms). Before crossing such a stream, he/she should also, theoretically take into account any additional time required so as to not interrupt the major road flow.

***Yield control rules (give right of way rule)*:** A yield sign is also typical near a junction and roundabout and a driver is expected to give way to any traffic on a major road ahead RSA (2007). Movement onto the main road is expected to take place only when there is enough time to complete the manoeuvre.

4.3 Driver behaviour

Risser (1985) initially studied different types of errors in driving behaviour that were related to conflicting situations in the traffic. He concluded that most errors were the result of a lack or misunderstanding (or communication) in the interactions between different road users. Included in the behaviour types that he found to be related to conflict probability among drivers were: risky passing manoeuvres, badly adapted speed, following too closely, *unlawful behaviour at traffic lights*, risky lane-changing, lack of precaution at intersection through, insisting on(or taking) others right of way. Aggressive driving as typical of risk-taking behaviour behind the wheel has been variously labelled and includes (amongst others) speeding, tailgating, weaving dangerously through traffic, and ignoring signs or red lights (Shuster, 1997). Miles and Johnson (2003) proved that parameters such as personality characteristics and attributes of people who are likely to commit aggressive driving behaviour are of great interest to researchers for the purpose of understanding, predicting and correcting or preventing such dangerous behaviour. Liu and Lee (2005) have also recently studied the effect of car-phone use and aggressive disposition in braking response at signalised intersection. These can result in increased accident risk due to the multi- tasking combination of decision making and car-phone communication. Wang and Ruskin (2006) studied heterogeneous and inconsistent driver behaviour at multilane urban roundabout using MMAS (multi-stream minimum acceptable space) rule. The authors concluded that driver behaviour has an impact on the overall performance of the roundabout and individual roads. Kaysi and Abbany (2007) studied aggressive behaviour of minor street vehicles at un-signalised intersections. The model can predict the probability of a driver performing an aggressive manoeuvre as a function of a set of driver and driver attributes. Parameters tested and modelled were: driver characteristic (gender and age), car characteristics (performance and model year) and driver attributes (number of rejected gaps, total waiting time, at head of queue and major-traffic speed). They concluded that age, car performance, and average speed on the major road are the major determinants of aggressive behaviour and also that total waiting time of the driver while waiting for an acceptable gap is of significance in incurring "forcing" behaviour.

4.3.1 Driver variants considered

In this research, we have limited our study to two behavioural types of driver. These are broadly designated as rational (normal) while the other is designated aggressive (prone to risky or *unlawful behaviour e.g. at traffic lights* or reasonably respectful of the rules). The impacts of these alternative behaviours are considered at road configuration examples including the controlled single-*lane one-way intersection* in order to examine the effect on throughput for heterogeneous traffic. As an illustration of aggressive versus rational behaviour, a driver entering an intersection during the yellow signal, is *aggressive* as opposed to the rational driver who recognises that the time to complete the manoeuvre is sufficient.

Driver distribution: The distribution of driver behaviour is expressed probabilistically for rational and aggressive denoted as P_{ra} and P_{ag} respectively. Clearly $P_{ra} + P_{ag} = 1$ and, for the more general case of a range of driver types, it is obvious that

$$\sum_{\substack{all \\ drivers}} P_{type} = 1 \tag{4.1}$$

According to the binary driver behaviour distribution, each driver on each approach road before entering an intersection is randomly assigned to one of two driver behaviour categories. This is revised at each time step for which he/she is subjected to non-progression stage of the fixed light cycle; in this case yellow light period. In this way both heterogeneous and inconsistent driver behaviour is simulated. In other words, if a driver is assigned as aggressive in any time step during the yellow of the traffic light cycle then that driver will enter the intersection, whereas a rational driver remains stationary in this time step. The driver behaviour may be re-assigned randomly during yellow period to either category, but the intersection manoeuvre only commences during this period if the waiting driver receives the designation "aggressive". The effect of this modification is, effectively to reduce the safety part of the light cycle at the same time as increasing the driver's opportunity of movement. Given the restriction to the "yellow" part of the cycle and binary categorisation, the expected impact for a simple intersection is likely to be small. Implications for entry to, or manoeuvre at, more complicated intersections or at roundabout entry for a more extended range of drivers types is considerable. This has been explored recently, for the homogeneous traffic case by Wang and Ruskin (2006).

4.3.2 Limitation of driver behaviour study

For purposes of illustration, in this work we have applied the case of different driver behaviour to both types of vehicle SV, LV and to the single-lane one-way intersection model only. In all other configurations considered, driver behaviour is taken to be rational only (P_{ra}=1) for simplicity. Nevertheless, the implications of irrational or reckless driver behaviour are also exacerbated for LV: SV mix as initiating a LV vehicle manoeuvre is inherently more risky. There is an argument, therefore, for further weighting the impact of LV manoeuvres, but we have not specifically considered this here.

4.4 The intersection or roundabout: General principle

A further cellular automaton (ring CA) is defined for the activity of the intersection or roundabout itself. These are constructed from a mosaic of 7.5-meter squares, which are equivalent to normal cells of the roadways, and it is expected that a vehicle can enter and leave a cell from any of the directions, according to the corresponding geometry of the intersection or roundabout; (detailed in the following Chapter). Each vehicle on an input roadway has possible exit routes i.e. LT, ST, RT to follow across the surface of the junction, and dependent on number of output roadways.

The turning probability is expressed as P_{LT}, P_{ST} and P_{RT} respectively. Clearly $P_{LT} + P_{ST} + P_{RT} = 1$. In general case for different road configurations and possible exit route "j".

We can write also

$$\sum_{\substack{all\\turning\\options\\j}} P_j = 1 \qquad (4.2)$$

A vehicle initiates a manoeuvre, (enters the intersection), when the entry cell and its upstream neighbour are empty and the traffic light is green. The vehicle leaves the intersection (or roundabout) when the outgoing cell in the designated direction for movement is free. In the case of a roundabout entry is of course, in one direction only, since a vehicle must enter the circulatory movement according to the designated direction of roundabout flow. Further, given random population of the system, results are averaged over several runs.

4.5 Implementation

4.5.1 Structure

The simulation has been developed in two parts with the first handling data input using file (input.cpp). Data inputs include the following

- Number of roads connected to the intersection or roundabout
- Length of each road (should be same each time i.e. a constant)
- Turning rates for each entry road
- Mean arrival rates (λ) or fixed arrival distribution(AR) of each entry road
- Proportion of SV and LV vehicles arrivals for each road.
- Simulation time (period for which the system configurations are considered are long enough to represent extended periods of real time)
- Distribution of driver behaviour

The configuration file (input.dat) is set up for use by the second programme. The second is the main file (output.cpp) which contains (road.h) and (intersection.h) or (roundabout.h) classes. When we run the main program, the main program automatically read the data file. Statistics are gathered over periods of updates at each 1 second time step and result for measures such as throughput, capacity, queue length, density, total delay (defined in Section 4.5.3) are obtained.

An overall perspective of the simulation scope is given by the schematic representation (Flow chart) Figure 4.3.

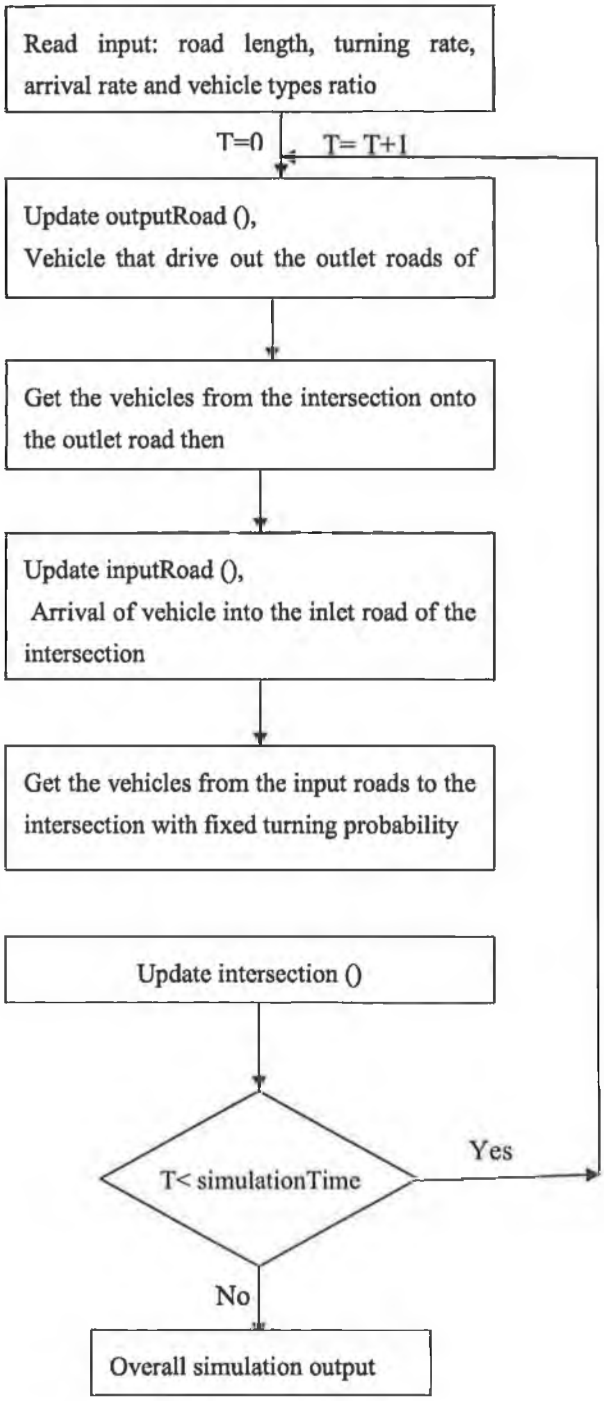


Figure 4. 3- Overall simulation structure for heterogeneous traffic flow model

4.5.2 Distributions

4.5.2.1 Vehicle generation algorithm

In terms of populating the road cells in our models, the arrival rate (or more precisely average arrival rate, λ) is the number of vehicles arriving (on average) at one of the entry roads to the configurations in unit time step, (one time step = 1 second).

For the work presented for the single-lane two-way intersection, the maximum overall average arrival rate is taken to be λ=0.4 (equivalent to 1440vph) with maximum SV: LV proportion=0.96:0.04, based on observations from *real data* (supplied DCC). At a T-intersection the maximum overall average arrival rate used is λ=0.6 (2160 vph) with maximum SV: LV proportion=0.94:0.06. At a roundabout values used are λ = 0.5 (1800 vph) and maximum SV: LV proportion=0.97:0.03. These observed data sets are quite similar and represent an instance. One of the reasons for simulation is that it permits sensitivity analyses i.e. the variation of particular values to which parameters are set, in order to see how robust the system is to different such values.

Random or quasi-random vehicle arrivals are generated using Poisson distribution, as a basis, but adapted in two ways. For the TWSCI and two-way traffic light controlled intersection (TWTLCI) and T-intersection we used effectively a *truncated Poisson* in order to control vehicle proportions more precisely and for single-lane one-way intersection and roundabout, are used a Poisson distribution was used with parameter (λ) varied from 0.1 to 0.8 (equivalent to360 vph to 2880 vph). The truncated Poisson arrival under specific conditions is discussed in (Chapter-5). For typical Poisson arrivals vehicle options are as follows.

Vehicles (both SV and LV) arrive at random on a given entry road with a probability (for each road) derived from the Poisson distribution. Once a vehicle arrives, vehicles are denoted SV or LV with associated probabilities, P_{SV} and P_{LV} respectively. Subsequent progression of a given vehicle is as described earlier. The random arrivals may be SV or LV in any time steps and next time steps arrival is independently SV or LV. Also if road cannot be entered, assume initial LV or SV arrival does not enter.

$$(P_{SV} + P_{LV}) + P_o = 1 \tag{4.3}$$

$$P_r\{\text{of one arrival only}\} = e^{-\lambda} * \lambda \tag{4.4}$$

λ = Total average arrival rate in arrivals per sec

P_r = probability of single arrival in unit time (or arrival probability of a single vehicle of either type i.e. single event in one second of time)

$P_{SV} * P_r =$ P { vehicle arrive and type SV}

$P_{LV} * P_r =$ P { vehicle arrive and type LV}

$P_o =$ P { No arrival of vehicle type SV or LV} = 1- P_r

The algorithm and decision rule criteria for incoming vehicle are as follows: Suppose the random number 'b' and 'c' are generated between 0 and 1, not including 1 then

Step1: Vehicle of either type will arrive if $(b \leq P_r)$

Step2: 1. SV will arrive if ((c$\leq P_{SV}$) and the first cell incoming road empty)

else

2. LV will arrive if ((c $> P_{SV}$) and the first two cell incoming road empty)

else

3. no vehicle will arrive.

4.5.3 Performance measures

Based on the assumptions described, various performance measures have been studied. These include:

- **Flow capacity (C) or Capacity:** Defined as the number of vehicles that can enter from entry road to the intersection or roundabout per unit of time (vph). Clearly, this is the number of vehicles moving over a link or an entry road per unit time,(different from throughput)

- **Throughput:** The numbers of vehicles passing through the intersection or roundabout per unit of time (vph). The *overall throughput* is then computed as follows:

Overall throughput(vph) $= \sum_{i=1}^{n} C_i$ (vph) -
(number of vehicles on the intersection or roundabout per unit of time (vph))

C_i : Capacity of entry road "i"

i: number of entry roads connected to the configuration(i.e. intersection or roundabout).

- **Road density(D):** The road density for a given road defined and written or interpreted in the following form:

$$D = \frac{n_{CellSV} * n_{SV} + n_{CellLV} * n_{LV}}{l}$$

n_{CellSV} : n cells occupied by one short vehicle(SV)

n_{CellLV} : n cells occupied by one long vehicle(LV)

n_{SV} : n number of SV

n_{LV} : n number of LV

l : Length of the road

- **Queue length:** Queue length is calculated from the intersection back along on entry road. If two consecutive cells are vacant, this indicates the end of the queue. This definition means that a queue ends when vehicle can freely move, because the cells in front are vacant.

- **Total delay:** Delay experienced by a vehicle includes summation of all time steps for which it cannot move when approaching the intersection, plus time spent waiting to enter the intersection or roundabout, (that is start its manoeuvre).

4.6 Single-lane one-way cross intersection

Single-lane one-way crossed intersections are fundamental operating units of the sophisticated and connected urban networks. Analysis of this operation is thus advantageous in terms of the ultimate aim to globally optimise any city network. This type of intersection is also known as "one-way to one-way" intersection and has been studied Fouladvand et al (2004) for the homogeneous traffic case. Wang (2003) also theoretically indicated different driver behaviours for "two-stream" intersections for homogeneous traffic. Our objective is to analyse the traffic state for heterogeneous driver for the mixed traffic case, in order to obtain insight on the impact even on the simple flows.

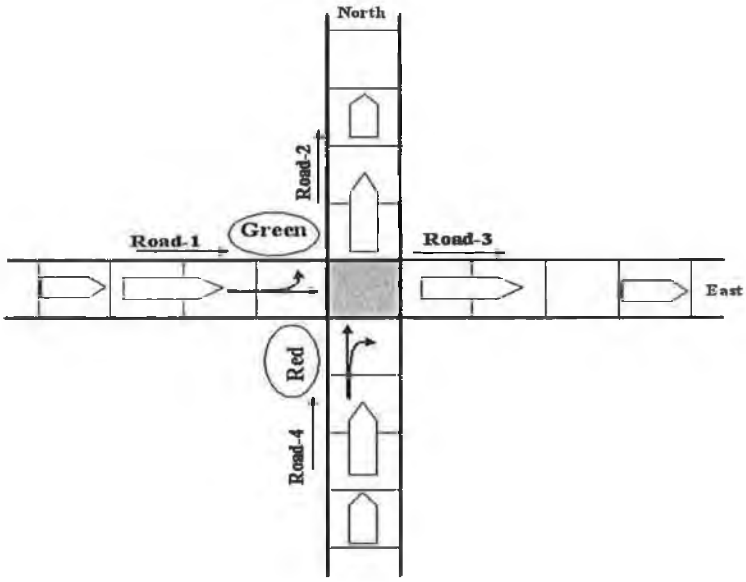


Figure 4. 4- Vehicle manoeuvres at single-lane one-way cross intersection

Permitted manoeuvres**:** Figure 4.4 is an illustration of single-lane one-way intersection. The shaded area is the intersection area. The junction is controlled by traffic lights with a pre-determined cycle of green, yellow and red lights. Vehicles arrive are taken at south and west entrance of the intersection only: Here referred to as road-1 and road-4 with no loss of generality. From road-1 a vehicle can either make a LT or go ST and, for road-4, vehicles can go ST or make a RT manoeuvre. On both entrance roads of the intersection, traffic lights are employed. For this simple intersection, light cycle length is taken to be 100 seconds, of which the green light phase on road-1 or road-4 is 45 seconds, followed by 5 seconds of yellow light and 50 second of red light.

This limits cross traffic flows considerably, since we never have the situation of a RT vehicles waiting to cross opposing traffic as in single-lane two-way cross intersections.

Methodology: Again, a CA is the basis for the model. Vehicles typically enter the intersection when the traffic light is green,(rational driver), or when the traffic light is yellow,(aggressive driver).

The intersection is an area of one cell. A SV vehicle on the intersection can leave the intersection in 2 second whereas an LV takes 3 seconds to complete the manoeuvre. In other words the yellow light phase is taken to be 5 seconds to reflect the complete movement for any vehicle that is in the intersection.

4.6.1 Simulation results

Simulation was carried out for 3600 time steps (equivalent to 1 hour) for a road length of 100 cells for both approaches and under different values of traffic parameters, such as arrival rate, turning rate and proportion of short and long vehicle in each of the two roads. Based on the assumptions described we studied *throughput, delay time, density and queue length.*

4.6.1.1 Vehicle type

Table 4.1 and Figure 4.5 show throughput changes with arrival rates both for homogeneous (SV or LV) and different heterogeneous (SV+LV) traffic flows. Arrival rates of the two roads are taken to be same $\lambda_1 = \lambda_4$ and range from 0.2 to 0.8 (equivalent to 720vph to 2880vph) while turning ratio of road-1 is LT: ST=0.3:0.7 and road-4 is ST: RT=0.7:0.3. The simulation was run for 1 hour and averaged over 50 runs.

Table 4. 1 Average throughputs of single-lane one-way intersection for λ1= λ4 and LV ratio varies from 0 to 1

	Vehicle type ratio (SV: LV)		
$\lambda_1 = \lambda_4$	1:0	0.5:0.5	0:1
0.2	1136	1130	1090
0.4	1631	1448	1116
0.6	**1636**	**1436**	**1116**
0.8	**1637**	**1461**	**1118**

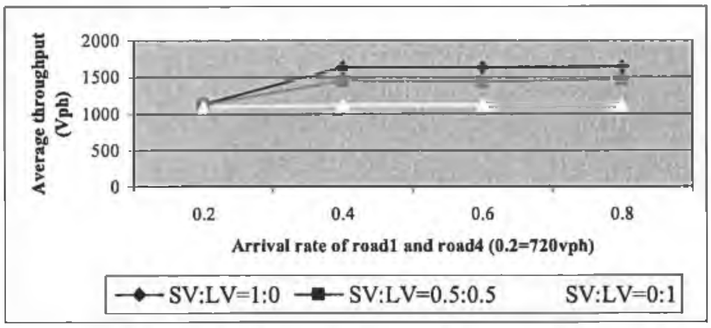


Figure 4. 5- Average throughput vs arrival rate

For increasing proportion of LV throughput decreases and, for increase in arrival rates rapidly saturates for all traffic types although at a level which is higher on average for SV than LV mix traffic.

4.6.1.2 Driver behaviour

Table 4.2 illustrates the impact on throughput of different driver behaviour populations. For each arrival rate considered ($\lambda 1 = \lambda 4 = 0.1$ to 0.4 equivalent to 360 vph to 1440 vph), the turning rates of road-1, LTR: STR=0.4:0.6 and for road-4 are STR: RTR=0.6:0.4. The proportion of SV: LV is taken to be 0.8:0.2. The simulation was run 1 hours and averaged over 1000 times (the very large average time is taken to see the convergence value for a small simulation time). For increasing arrival rate throughput increases in all cases of driver distribution. For saturation values are again indicated in the Table in bold. When queue length has reached the length of the road we said saturation.

Table 4. 2 Overall average throughput vs. driver behaviour for heterogeneous traffic

$\lambda 1 = \lambda 4$	Driver proportions (rational: aggressive)				
	1:0	0.75:0.25	0.5:0.5	0.25:0.75	0:1
0.1	625	627	**629**	**628**	**629**
0.2	1134	1138	**1138**	**1138**	**1137**
0.3	1498	1524	**1530**	**1536**	**1539**
0.4	1579	1636	1676	**1701**	**1711**

For this artificial intersection, we found that aggressive driver behaviour leads to same increases in the throughput, compared to that for rational driver behaviour. The reason is that the aggressive drivers do not obey traffic rules and attempt to enter the intersection at any time during the "non-red" part of the light cycle. However, this is an extreme example, where no driver has to compete for intersection space with oncoming or opposing traffic flows. In consequence only one-directional cross traffic and opposing traffic flows serve to constrain or penalise out-of-time manoeuvres adapted by aggressive drivers.

Throughput is much the same for given arrival rates and increases slightly with increased proportion of aggressive drivers.

4.6.1.3 Individual Road per performance: expected total delay

Figure 4.6 and Table 4.3 indicate the total delay. The arrival rates of two roads are taken to be equal, $\lambda_1 = \lambda_4 =$ 0.1 to 0.6 (equivalent to 360 vph to 2160 vph). The turning rates are: Road-1, LT: ST=0.5:0.5 and Road-4, ST: RT= 0.5:0.5. The traffic light cycle is modified to be green: yellow: red for road-1=55:5:40 seconds in order to provide major flow from road-1(i.e. asymmetrical flow of the system). In this thesis all notation SV, LV and SV+LV means 100% SV, %LV and (50% SV+50% LV) traffic unless otherwise specified.

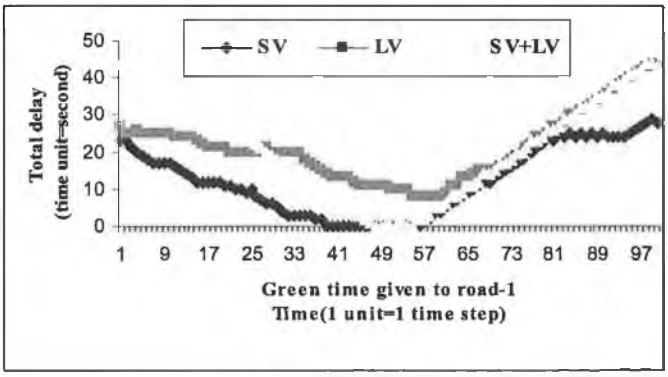


Figure 4. 6- Total delay versus the green time of road-1 is sketched for homogeneous and heterogeneous traffic flow when ($\lambda_1 = \lambda_4 =$ 0.5). The cycle length is taken 100seconds

Figure 4.6 depict the total delay curves as a function of green time allocated to road-1 for a value of cycle length is taken 100 seconds for arrival rate =0.5 illustrated. The middle area, where the total delay is minimized for all cases (mix (SV+LV), 100% SV and 100% LV traffic) corresponds to the situation in which most vehicles traverse the intersection in the green period and the queue may be dissolved during one green period and consequently the waiting cars will be able to go through the intersection in upcoming green period.

Figure 4.6 is just one realisation the light cycle, but this might vary, depending strongly upon the arrival rates of the vehicle types and hence the availability of space on the intersection during the green time phase.

We find that for 100% LV total delay curve in the middle area is less minimised as compared to (SV+LV)=(50:50) and 100 % SV curve.

Table 4. 3 Total delay for road-1 versus Arrival rate

λ1= λ4	Total delay in seconds over all vehicles		
	SV	**LV**	**SV+LV**
0.1	3876	4286	3787
0.2	8038	14647	9852
0.3	13930	68857	24943
0.4	25764	**73617**	102328
0.5	100698	**74391**	**112747**
0.6	128450	74561	113985

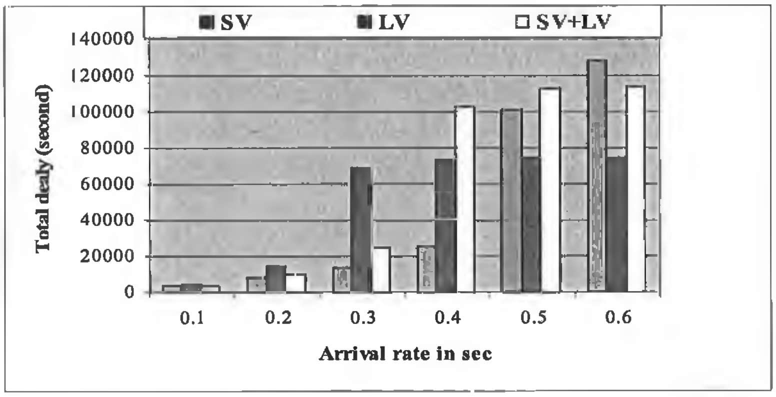


Figure 4. 7- Total delay for road-1 versus Arrival rate

Table 4.3 and Figure 4.7 show the total delay for road-1 over the period of the simulation and over all vehicles occupying the road during period (see definition total delay: Section 4.5.3).We evaluate the total delay during 3600 time steps which is equal to a real time period of 1hour and average the results of 100 independent runs of the program. When arrival rates increase the total delay increases for both homogeneous and heterogeneous traffic on average. In order to see the absolute change in total delay from arrival rate 0.1 to 0.2 and from 0.2 to 0.3 and so on for homogeneous and heterogeneous traffic we calculated absolute change and average change in total delay and present the results on Table 4.4.

One way of looking at average delay is normalised by each vehicle. In practice no comparable data are available from known sources. Consequently one elected to look at total amount of delay for vehicles of given types and differences between theses over a period. Table 4.4 should indicate that this is looking at total amount of delay. In general, this gives some idea of the efficiency of operation at the configuration as experienced by vehicles of different types.

Table 4. 4 Overall total delay in seconds over 1 hour for both vehicle types.

	$\lambda 1 = \lambda 4$	SV	Change in Delay SV	LV	Change in Delay LV	SV+LV	Change in Delay (SV+ LV)
	0.1	3876	4162	4286	10361	3787	6065
	0.2	8038	5892	14647	54210	9852	15091
	0.3	13930	11834	68857	4760	24943	77385
	0.4	25764	74934	73617	774	102328	10419
	0.5	100698	27752	74391	170	112747	1238
	0.6	128450		74561		113985	
Average change in total delay=			**24914.8**		**14055**		**22039.6**

The average change in total delay of SV traffic is higher than for mixed traffic and for mixed traffic is higher than that for LV traffic i.e. in the order SV> (SV+LV)>LV. The reason SV traffic has higher delay is that the more vehicles are on the road, more delay. For example in a 100 cell length of the road 100 SV can be accommodated as opposed to 50 LV. So, in the case of SV traffic sum of all vehicles delay will be more. This is as expected for the model.

4.6.1.4 Individual road performance: density and queue length for mix traffic

Figures 4.8 and 4.9 indicate the expected density and queue length of road-4, when λ1= λ4= 0.1 to 0.5 and SV: LV=0.5:0.5.The turning proportion of road1= LT: ST=0.5:0.5 and for road 4 ST: RT=0.5:0.5. The simulation is run for 10,000 time steps and with densities and queue length determined in each case over the last 100 time steps. The reason is that we are looking in each case for the situation on road-4 after the green period is over. So in the 100 time steps, the road-4 green periods will finish. For that reason we take the last 100 time steps.

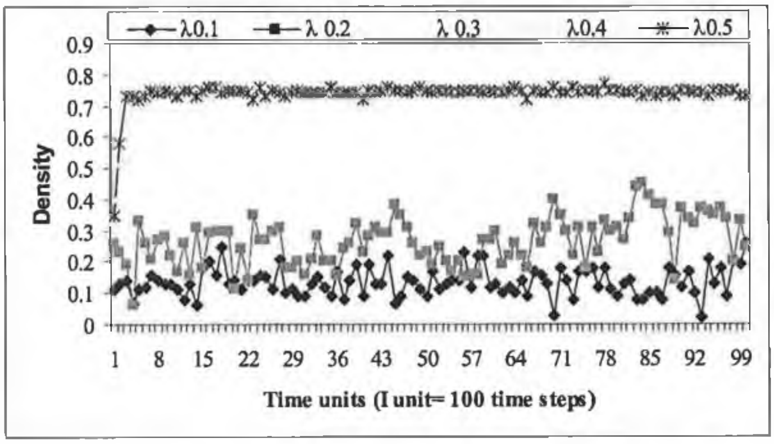


Figure 4. 8- Traffic density on road-4 for mixed with SV:LV=0.5:0.5 over time for a range of arrival rates

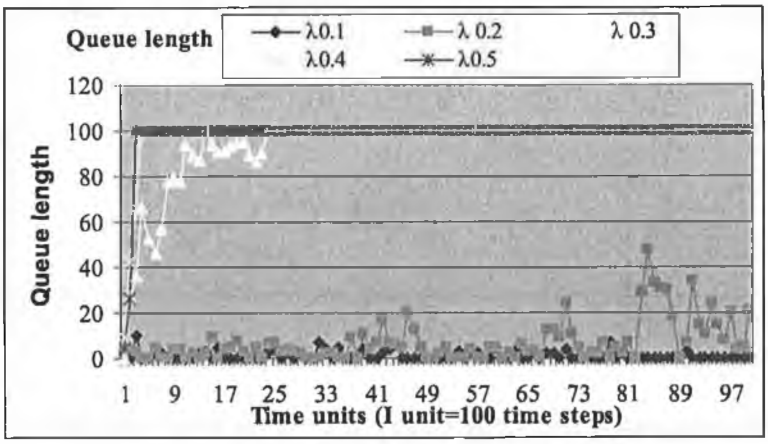


Figure 4. 9- Queue length of road-4 for mixed traffic (with SV:LV=0.5:0.5)vs time for a range of arrival rates

In Figures 4.8 and 4.9, densities and hence queue lengths change on road-4 when λ>0.3 for mixed traffic in the ratio SV: LV =0.5:0.5. When λ < 0.3 the queue is very small or non existent, the density of the entrance road is 0.36. However, the speed of formation of the queue and density increases as λ increases. When queue length reaches maximum length of the road (100 cells) the density of the entrance is 0.75. Therefore, the maximum density and maximum queue length do not necessary happen simultaneously.

4.7 Summary

In this chapter, we described the *research methodology* and a model to study traffic flow at *single-lane one way* intersection. Vehicular traffic flow is highly dynamic and constituent factors affecting flow through urban configurations are many and difficult to quantify. Included are driver behaviour, vehicle type, arrival rates, and cross-traffic rules and so on. In this spatially and temporally dynamic environment, homogeneous CA methods for motorised traffic flow are limited in predicating throughput, capacity, queue length, delay and density for both *homogeneous* and *heterogeneous* traffic.

A new two-component cellular automata (2-CA) model is proposed to simulate directly different drivers (limited) and vehicle types (binary to date) at different urban configurations using space considerations to govern manoeuvres. Heterogeneous and inconsistent driver behaviour can also be investigated by the model formulated.

The queue length, density and delay time of each of the road in each time step can be directly obtain from the model for different arrival rates and traffic mix.

The throughput of the single-lane one-way intersection not only changes with the increase in arrival rate but also changes to same extent with driver behaviour and LV proportion. Lacking real data,(on what proportion of Irish drivers are aggressive and, in particular whether they are consistently so), proportions have been arbitrarily assigned in this simple experiment.

The most important conclusion is that this single-lane one-way controlled intersection can only be considered as a first step on the way to understanding and modelling the aggressive behaviour of drivers at signalised intersections. More extensive data collection processes are needed to validate the obtained model, as well as further tests to investigate the significance of other parameters in explaining the aggressiveness of drivers.

Finally, the problem of aggressive driving behaviour may be a worldwide problem. It has been suggested that cross-cultural research should be conducted to explore whether a profile of an aggressive driver (and/or a "road rager") developed in one country or culture would be consistent with a similar profile developed in another country or culture, (Miles and Johnson, 2003). These authors suggest that more effort should be brought to bear on changing the attitude of road users, aiming at changing aggressive and risky driving behaviours.

In general, we have explored the benefits of modeling driver behavior at a signalised intersection and we believe that this model and the general approach to modeling driver behavior at signalised intersection will be an extremely fruitful avenue for future research in modeling a wide range of driver behavior phenomena.

Chapter 5: Controlled and uncontrolled single-lane two-way X and T-intersections

5 Controlled and uncontrolled single-lane two-way X and T-intersections

In this Chapter, we investigate traffic behaviour at controlled and uncontrolled single-lane two-way X and T-intersections for heterogeneous motorised traffic and incorporate detailed interactions at individual vehicle level for a (binary) mix of vehicle type (SV and LV). While models are developed for left-lane driving (Ireland/UK), the principles apply directly to the alternative case. Single-lane X and T intersections are the most commonly used intersection in urban areas. The study detail is thus targeted to ascertain the effect of long vehicles in the traffic mix on throughput and capacity at controlled and un-controlled intersections.

5.1 Model basis (distribution assumptions)

The operation of a X and T- intersection systems begins with vehicle arrivals on the major or minor entry roads to the configuration, (where arrivals are assumed to follow a truncated Poisson distribution in this instance). The reason for this choice of distribution is that even in urban traffic only 1 or 0 vehicle arrives in a unit of time(one unit time is quall to one second) if taken to be small enough (equal to 1 second here). From our previously published works, (Deo and Ruskin, 2005, 2006) relating to these configurations, we also discuss implications of adapting distribution assumptions. The movement of arrival vehicles at the entry road depends on the level of congestion on the road. When a vehicle (i) arrives at the end of the feeder roads i.e. at the configuration entrance=100th cell and the signal is green or (ii) arrives at a stop-line in a free flow major and minor road, it has already been randomly assigned to a directions. For example if v % of vehicles arriving by road-1 are assumed to turn left (e.g. turn into road -2), then these will be assigned a particular number in order to guarantee that these vehicles will eventually turn into road-2. Before the manoeuvre towards the assigned destination is started, on reaching the configuration, it is clearly necessary to check that there is enough space for vehicle to drive onto the intersection. In this chapter we have assumed all drivers are rational i.e. driver distribution is single-valued, $P_{ra}=1$ for all intersections discussed. Fixing the arrival rate (AR) distribution to be 0.05, 0.1, 0.15, 0.2., 0.25, 0.3, 0.35, 0.4, 0.45, etc with parameter λ=0.053, 0.12, 0.18, 0.26, 0.36, 0.49, 0.72, 0.8, 1.0, etc vps (in terms of vehicle per second) or 190 to 3600 vph (expressed in terms vehicle per hour).

The organisation of the chapter is as follows. In Section 5.2 we present the traffic light controlled (TLC) X-intersection and investigate performance measure such as *throughput* and *capacity* and discuss these results in relation to the Dublin locality. In Section 5.3 we present the TWSC intersection and define the minimum acceptable space (MAP) rule at a stop-line from minor road for SV and LV to start intersection manoeuvres and compare the result with TLC X-intersection. In Section 5.4 we present the TLC T-intersection and contrast results with both the TLC X-intersection and with real data of a T-shaped intersection. Finally, in Section 5.5, we discuss the findings generally for a binary traffic mix and develop ideas for future work.

5.2 Traffic light controlled single-lane two-way X-intersection (TLC X-intersection)

Intersections where traffic flows in different directions converge play an important role in a road network. The schematic diagram of a controlled single-lane two-way X-intersection is presented and its operation described in Chapter-3. The junction is controlled by traffic lights with pre-determined cycle (as in Chapter-4, section, 4.2.2). A study of traffic flow characteristics at signalised intersections arguably is one of most effective modelling measures to pursue with a view to improving the understanding on how to enhance the capacity of road networks and relieve congestion in cities. In this chapter, we focus our attention on signalised single-lane two-way intersection for heterogeneous traffic mix based on our previous model single-lane one-way intersection (see Chapter 4, Section 4.6), in which there are four entry road approaches to the intersection and all entry roads have the option for vehicles to turn LT, ST and RT. We then look at TWSC and signalised T-intersections and compare the result with TLC X-intersection both for homogeneous and mixed traffic flow. In the final section we also discussed the option of the flared approach (defined in Section 5.5.1) and its importance to the single-lane mix traffic flow for future research.

5.2.1 Arrival / progression on an entry road

We consider an intersection of single-lane two-way x-road (see Figure 5.1). There are four entries of the intersection road-1, road-2, road-3 and road-4 with minor and major road as indicated. At each corner of the intersection there is a traffic light (as illustrated Chapter-3, Section 3.1.1). A vehicle arrive at an entry road, where rules for generation of either type of vehicle (SV or LV) are adapted slightly differently (as for step1 Section 4.5.2).In this chapter we provide the value "P_r" directly to the model, i.e. fix the arrival distribution (AR).Then we compare AR with b (i.e (if (b$\leq$ AR))), a vehicle of either type is said to arrive, else no vehicle, (the "b" is a randomly generated in the range 0 to 1). Step 2, Section 4.5.2 then applies for SV or LV arrivals. For updating of vehicles on the single-lane roads, we use our basic general 2-CA update rule, given in the previous chapter.

5.2.2Manoeuvres at the intersection

When vehicles approach an intersection (i.e. when they get to 100th cell of the entry road) and the signal is green, they may proceed to LT, ST or RT according to fixed turning rates for a given simulation. The pattern flow at a junction is called the turning movement flow matrix for the junction, and it can be regarded as small origin-destination matrix (Taylor et al., 2000). A schematic representation of turning movement flow matrix is shown in Table 5.1. The rows represent the approach road flow direction to the intersection, the columns represents the departure road flow direction.

Table 5. 1- Schematic turning movement flow matrix

Approach road to the junction(i)	Departure road flow direction (j)			
	1	2	3	4
1	$\mathbf{V_{11}}$	V_{12}	V_{13}	V_{14}
2	V_{21}	$\mathbf{V_{22}}$	V_{23}	V_{24}
3	V_{31}	V_{32}	$\mathbf{V_{33}}$	V_{34}
4	V_{41}	V_{42}	V_{43}	$\mathbf{V_{44}}$

The elements in each cell of the table (e.g. V_{ij}) represent vehicles (V) from approach "i" turn into departure "j". U-turns are generally not considered in turning movement at intersection studied (Taylor and et al, 2000). So the exception V_{ij} is taken to be zero when i equal to j and denoted bold in the Table. Only at a roundabout are u-turns made with ease if they are permitted at all. Each vehicle makes its turning decision, corresponding to the random individual route planes, which are derived from origin-destination information.

For a vehicles from all approaches (roads 1, 2, 3, and 4) at the 100th cell of the entry road approach to the intersection assigned (randomly with fixed turning distribution) to a new number to turn left, go straight through or turn right. For example, for short vehicle we assigned LT=7, ST=11, RT=17 and for long vehicle LT=27, ST=31, RT=37.

These arbitrary numbers we used for our convenience and in order to make sure the vehicles are moving to the assigned direction during traversal.

The intersection connected to four entries and four exit road, as shown in Figure 5.1. Figure 5.1 shows the requirements in terms of free cells for right turning vehicles both major and minor roads at a controlled four-way intersection. Right turning (RT) short vehicles and long vehicles require 2 marked free cells for manoeuvring. A "0" means that the cell is free or vacant. Left turning (LT) and straight through (ST) vehicles need one free cell before entry into the intersection. To complete a manoeuvre from entry road to the intersection without causing interruption to flow in case of right turning, a minimum of 4 time steps for LV and a minimum of 3 time steps for an SV are required.

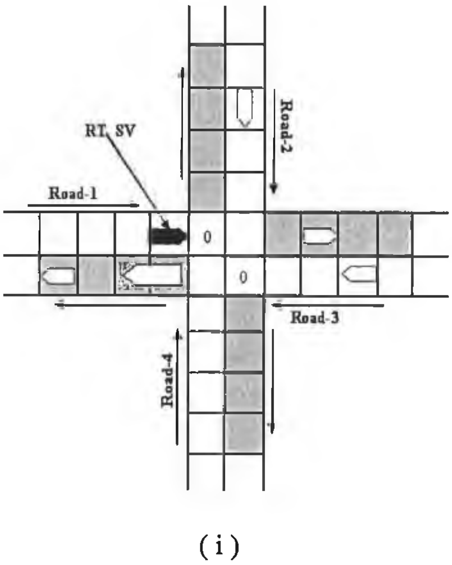


Figure 5. 1- A right turning (RT) vehicle from major road (i) SV (ii) LV. The four central cells represent the intersection area. Dark gray roads indicate outgoing roads. White roads show incoming roads.

The traffic light phase scheme of four roads is shown in Figure 5.2. The total time (with a constant period of time T=100) for signal to complete one sequence of indication i.e. (Green+ Yellow +Red) time.

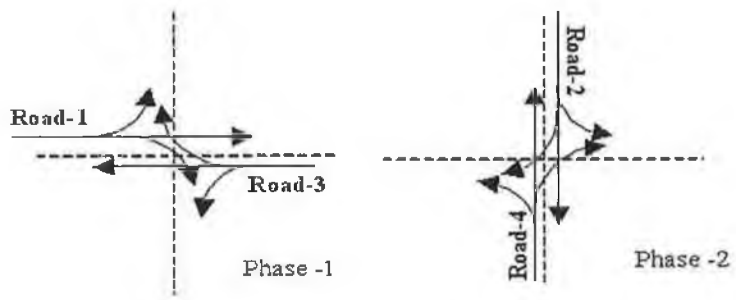


Figure 5. 2- Four approaches with two phases

Manoeuvres are dealt with four approaches with two phases i.e. in phase-1 vehicle moving from road-1 and road-3 and for phase-2 from road-2 and road-4. The fixed light signal scheme is detailed in Section 4.2.2.

5.2.2.1 The turning rules at the turning points

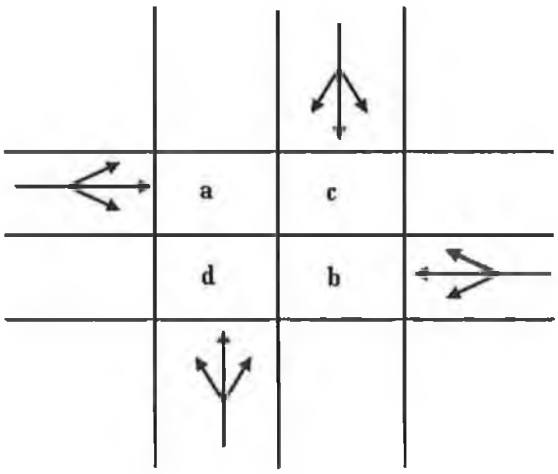


Figure 5. 3- Traffic light controlled single-lane two-way X-intersection with cell numbers

The intersection area is an area of (2x2) cells, i.e. those denote cell a, b, c and d as shown in Figure 5.3. The vehicles from all approaches after assignment of the new number they get to the first cell of the intersection from each direction (i.e. the turning vehicle from road-1 arrives at cell "a", road-2 at cell "c", road-3 at cell "b" and road-4 at cell "d"). In general when LV vehicles are given a new number, the front enters at the first time step then at the second time step the full length would be entered into the intersection area (e.g. A ST, LV from road-1 in time step 2 it will be in cell a and c).

In the intersection area the update rules depend on the position and state of the occupied cell as vehicles that come from different roads moves in different directions observes different rules.

5.2.3 Simulation results

As a simple model in previous chapter, we simulated the effect of LV proportion on mixed traffic flow when changing the input parameter λ on single-lane one-way X-intersection without influence of other roads. Here we simulate the model of single-lane two-way X-intersection and consider of influence of the other roads. Specifically, the turning vehicle acts to form bottleneck for road because it slows down near its turning point. We show the simulation results of overall average throughput of the intersection and the RT effect on the capacity of road-1. The simulations were carried out for longer period (equivalent to 36000 time steps) and result have been averaged over 10 independent runs. Empirical inputs used to underpin and validate the simulation are given in Section 3.1.1, and provide a base line for sensitivity analysis.

5.2.3.1 Validation of the model with real data

The real data, on which the sensitivity analyses were based, were obtained from a local (Dublin) single-lane two-way intersection: (Rathgar Road/ Frankfort Avenue, Chapter 3). This intersection is controlled by traffic light: basic characteristics and composition of the intersection are given (Section 3.1.1, Table3.1).

The *developed model clearly needs to be validated against real life situations, (field conditions*). Accordingly, while the simulation may attempt to replicate directly the mixed traffic flow on a given single lane two-way control intersection, (for which we have observed data), this is clearly only one possible realisation of all such intersections, just as the recorded data are only a snap shot of activity at the given intersection in real terms. The field data represents an average day's traffic, so it makes sense to validate our model using a run over one "day" or "daily period" where this represents duration of principal activity. It should be noted that this is based on the period for which real data are recorded. Further, each such daily period is replicated a number of times and an average figure obtained. For 50 runs of ten hours, and parameter values fixed, average results for the simulation are presented in Tables 5.2, 5.3 and Figure5.4. The average value was used to validate model performance against "real data" where the latter was considered applicable to a "typical" day. It should be noted that minimal metadata are recorded by DCC on the test data obtained, apart from the location. Such additional data are available usually relate to start, finish time, traffic composition and weather conditions. Other data possibly available are e.g. 24 hours volume for junction in 15 minute interval.

A comparison of observed and simulated turning rates is shown in Figure 5.4 and Table 5.2 respectively. From the original simulation runs, the comparison of the observed (from the generated experiment) to the expected (from DCC) frequencies was not particularly good (χ_6^2 for SV and LV e.g. significant at better then 0.001% level). This, partly because only one data collection set (realisation) was available on this configuration and partly because our model simplifies some of the configuration features. For example, from Table 5.2, it is noticeable that RT frequencies are most affected for several roads. The real configuration has features which are not reproduced in our model exactly e.g. no filters or no RT extra lane and pedestrian crossing is allowed for. These will affect RT rates. However as a crude fit this agreement provide a foundation for further sensitivity analysis, where parameters were varied to allow for circumstances with increased RT rates.

Only limited real data are available therefore real local features are approximated and some simplification in the configurations modelled. Hence, a range of parameters values has been considered in each case to allow for under/ over compensation of the real road configurations and which applies to all single-lane models in this thesis.

Table 5. 2 - Comparison results of our model and field data.

Road Number	Turn	Short vehicle (SV)		Long vehicle (LV)	
		Obs Data	Avg. Sim. Data	Obs . Data	Avg. Sim. Data
Road-1	LT	523	478.12	9	21.42
	ST	3941	4057.68	219	184.9
	RT	293	240.8	6	10.34
Road-2	LT	378	395.04	9	7.74
	ST	1545	1596.74	24	29.66
	RT	468	464.34	4	9.32
Road-3	LT	137	149.32	3	6.48
	ST	4173	4458.44	254	203.98
	RT	368	163.02	6	7.7
Road-4	LT	497	510.1	2	5.06
	ST	1504	1578.06	22	14.68
	RT	128	49.24	3	0.32

Obs.= Observed, Avg. Sim= Average simulated

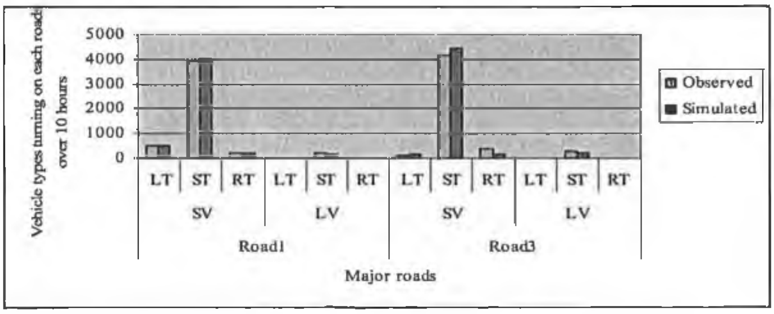


Figure 5. 4- Model validation (comparison of observed and simulated turning data of SV and LV from two major roads)

Further, Table 5.3 presents validate of the model by comparison of observed and average simulated flow capacity of each approach of the study intersection. Simulated capacity matches the corresponding observed value with low % error. The highest relates to road-2, possibly due to variation of cycle time allowed for by the model compared to actual flows and e.g. filters permitted in the real-world situation.

Table 5. 3 Comparison of observed and simulated entry capacity or capacity of the intersection over a 10 hour period.

Intersection entry capacity (vehicle in 10 hours)			
Road number	Obs. Data	Avg. sim. Data	% Error
Road-1	4937	4993.26	+1.13
Road-2	2428	2502.84	+3.08
Road-3	4941	4988.92	+0.96
Road-4	2138	2157.46	+0.91

We also performed the statistical test of Table 5.3 to test the *goodness of fit* (See Appendices B). The testing at the 0.05 level of significance for entry capacity, we have χ^2=3.5897 and $\chi^2_{0.05,3}$=7.817. The computed test statistic 3.5897 is smaller than 7.817. So, the observed and expected values are close and the model is a good fit to the data.

5.2.3.2 Sensitivity analysis

Overall Throughput of the intersection:

Table 5.4 illustrates the effects of different SV: LV proportions on overall throughputs. In each scenario, the turning rates of all approaches are based on analysis of the field data (see Section3.1.1). The fixed arrival rate (AR) distribution of the two major roads and minor roads are taken to be equal and allowed to vary from 0.1 to 0.3. The choice of arrival rate 0.1 to 0.3 is that, it is interesting to use simulation to analyse the effect on the throughput on mix traffic flow. In addition, it is useful to investigate how LV variations in the mix traffic can affect the overall average throughput of the intersection when arrival rate increases.

Table 5. 4- Avg. throughput Vs. arrival rate and long-vehicle proportion

AR(1=2=3=4)	Vehicle types proportion (SV: LV)					
	1:0	**0.9:0.1**	**0.8:0.2**	**0.7:0.3**	**0.6:0.4**	**0.5:0.5**
0.1	14405	14249	14031	13984	13793	13707
0.2	28744	28143	27352	26171	24905	23855
0.3	38111	36308	34509	32865	31327	29624

Clearly, we find that the average throughput of the intersection increases as AR increases both homogeneous (100 percent SV or LV) and heterogeneous (SV+LV) traffic. In contrast, heterogeneous traffic throughput decreases with increased proportion of LV in the traffic mix and become saturated even for low fixed arrival rate $AR_{1=2=3=4}$ =0.1(roughly eequivalent to a truncated Poisson with $\lambda_{1=2=3=4}$= 0.12 (equivalent to 432 vph)).

Capacity of Major Road:

Right-turning vehicles from the major-road in a shared lane, (where RT, ST and LT vehicles are on the one lane), can block ST and LT vehicles behind and on the same road. RT rates (RTR) of the major-roads thus have great impact on capacity of the major-roads. In order to examine this, we vary major road-1 right turning rate (RTR1) from 0.1 to 0.2, with major road-3 RTR3=0. Fixed Arrival rate distributions of AR1 = AR2 =AR4 = 0.15 were used initially, with AR3 varied from 0.05 to 0.55.

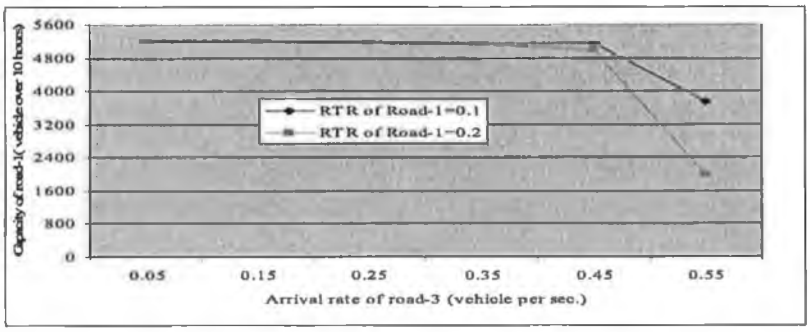


Figure 5. 5 Capacity of major road-1, RTR, road-3 taken to be zero

Figure 5.5 shows unsurprisingly that the capacity of the major-road 1 declines as RTR of road-1 and arrival rate of opposing traffic on the major-road-3 increases.

5.3 TWSC X- intersection

The TWSC intersection is essentially a special case of a traffic light controlled intersection with stop condition based on priority sharing rather than external agency. Figure 5.6 present schematic representation of two-way stop controlled intersection. The TWSC-intersection consists of two-approaches controlled by stop signs and other two major approaches are free flow. In an attempt to describe, as nearly as possible, the real-life situation at an intersection, the simulation models have been constructed from a detailed microscopic view point.

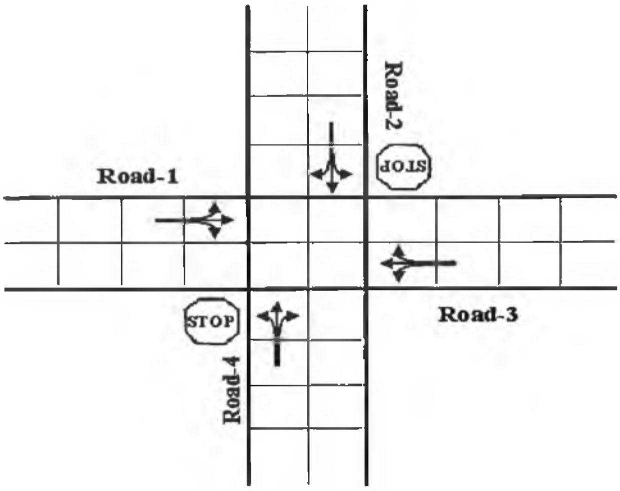


Figure 5. 6- Schematic traffic flow at two-way stop controlled intersection

Basic assumptions created in this investigation are

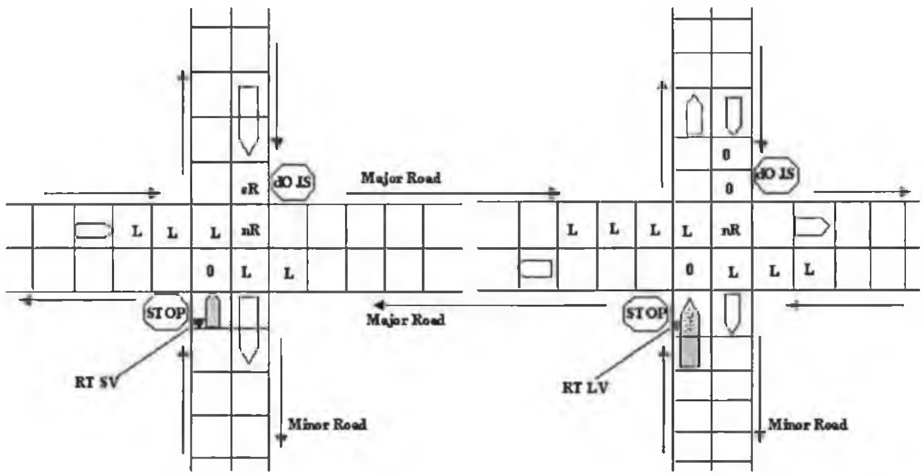
- All drivers are cautious and law-binding (i.e. rational)
- Pedestrians are not considered in the models.
- Two types of vehicles are considered i.e. SV and LV
- To generate the arrival of vehicles at the entry road of all approaches and turning matrix approaching to the intersection are same as TLC X-intersection.

5.3.1 Vehicle manoeuvre at TWSC intersection

A vehicle from the opposing minor road at the TWSC, which intends to move straight-ahead or turn left (LT), has priority over a RT vehicle from the given minor-road according to the rules of the road. However, priorities between minor-road vehicles may be less distinct, Tian et al. (2000). In some systems, as the authors indicate, drivers were observed to enter the intersection on a first come, first-served basis. In other systems, RT vehicles must wait for a clear path.

Minor-road vehicles will move on to the junction only when the required numbers of empty cells are available. In the CA model described (Section 4.2.1), the states of all cells update simultaneously. Figure 5.7 represents the current situation for available spaces and, to follow through on the movement, we consider the situation at the next time-step. We assume that all driver behaviour is rational and that, for our CA model, the space required for SV and LV is determined by the different number of vacant cells in crossing the opposing directions of flow. The number of vacant cells required for an SV and an LV manoeuvre at TWSC intersection is taken to be the same as the number of vacant cells required for a rational and conservative driver for *homogeneous* traffic as specified in Wang and Ruskin (2002).

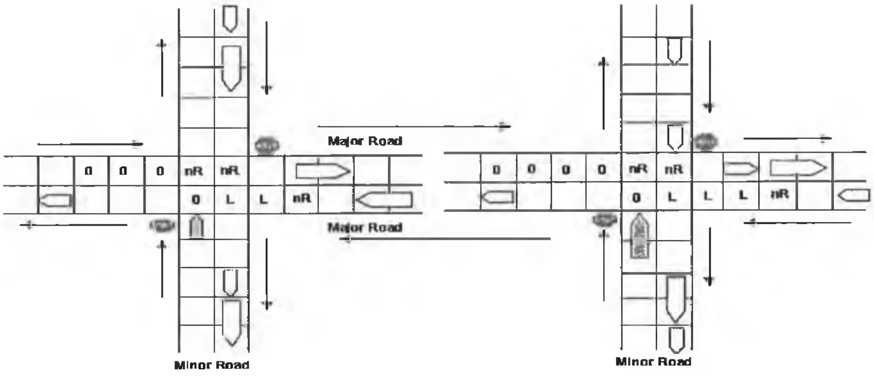
Figure 5.7 indicates the conditions for a RT vehicle (SV or LV) driver to enter the intersection from a minor road. A SV and LV need to check 8 and 11 marked cells respectively. Marked cells are denoted as 0, L, nR, sR. A "0" means that the cell needs to be vacant," L" means that the cell needs to be either vacant or occupied by vehicle that will turn left, "nR" means that the cell must not be occupied by right-turning vehicle, "sR" means the cell needs to be either occupied by a right turning short vehicle or vacant.



(i) (ii)

Figure 5. 7 Right turning vehicles from minor road TWSC Intersection (i) SV (ii) LV

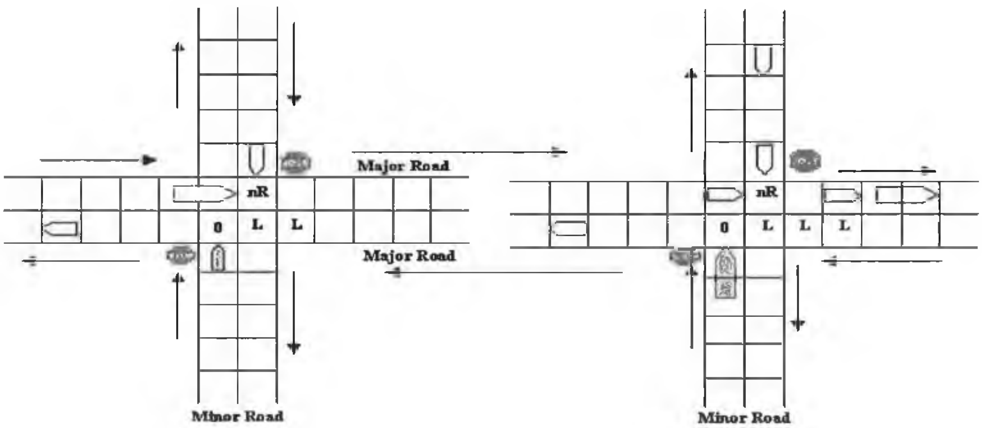
The movement of a RT SV or LV vehicle from a minor road does not need to consider opposing vehicles if either one of several conditions is met: (a) for an SV vehicle, the first cell in the opposing minor road is vacant, while for an LV, two cells in the opposing minor road should be vacant, or (b) the RT vehicle is the first vehicle in the opposing minor-road, or (c) the first vehicle in the opposing minor-road arrives at a stop-line in less than the stop time delay time. A minor road vehicle has to wait for 2-time steps prior to entering the intersection to check available space (see Section 4.2.2).



(i) (ii)

Figure 5. 8 A straight through vehicle from minor road (i) SV (ii) LV

The space conditions for ST vehicle to move into the intersection are illustrated in Figure 5.8. A SV from the minor road needs to check 9 marked cells before it can drive onto the intersection. In contrast, a LV needs to check 11 marked cells



(i) (ii)

Figure 5. 9 A left turning vehicle from minor road (i) SV (ii) LV

Similarly the shaded vehicles performing manoeuvres from the minor road are shown in Figure 5.9. A SV turning left needs to check 4 marked cells and a LV performing the same manoeuvre needs to check 5 marked cells.

5.3.2 Result from Computer Simulations

The simulation time and length of the road is taken to be the same as for the TLC X-intersection and for a single run. Single run here we mean that there was no average value taken over the several runs. Table 5.5 show results for a series of simulations with fixed and equal arrival rates for the four roads, (AR_1=AR_2=AR_3=AR_4), varied from 0.05 to 0.4. Here the focus is again on the effects of the LV on the throughput in order to compare the result with TWSC intersection. The arrival rate considered as low 0.05 to high 0.4 and effects of variation in increments of 0.05 are examined in a sensitivity analysis framework. To guide the range for turning rate and traffic composition for all approaches, real traffic data (Table 3.1, Section 3.1.1) values were used. We found that throughput of homogeneous traffic (i.e. 100 percent passenger cars or SVs in our model) increases linearly for arrival rate in this range and for all approaches (roads 1, 2, 3 and 4) increasing simultaneously. It is also clear that the throughput of 100 percent SV in a TLC intersection is higher than that of TWSC intersection. In this case arrival rates were not high enough to produce saturation of throughput but were designed to assess impact of vehicle mix on the flow.

Table 5. 5 -Comparison: Overall Throughput of Homogeneous and Heterogeneous Traffic at TWSC and TLC Intersections

AR(1,2,3&4)		Throughput (vph)		
		100 % SV	(50% SV+ 50% LV)	100% LV
0.05	TLC	866	871	52
	TWSC	863	872	415
0.1	TLC	1593	1481	17
	TWSC	1568	998	99
0.15	TLC	2303	1221	10
	TWSC	1773	747	95
0.2	TLC	3020	807	5
	TWSC	1859	664	61
0.25	TLC	3559	424	5
	TWSC	1944	339	18
0.3	TLC	4039	343	5
	TWSC	2147	250	17
0.35	TLC	4330	248	4
	TWSC	2293	175	13
0.4	TLC	4683	139	4
	TWSC	2429	121	12

In the case of heterogeneous traffic (i.e. SV+LV), when the fixed arrival rate (AR1=AR2=AR3=AR4) increases, throughput also increases for low AR and than decreases when AR> =0.2 both for TWSC and TLC intersection. The throughput of the heterogeneous traffic in a TLC intersection is higher again than that for TWSC intersection.

For both TLC and TWSC intersection, 100% LV traffic throughput obtain is very low, as compared to that for 100% SV traffic. However, homogeneous long vehicle traffic clearly does better at a TWSC intersection, but in reality no city traffic is 100 per cent long vehicles. Clearly, while conditions are extreme and artificial for some of the percentages of LV traffic in these tests, our model can be used to predict the impact of traffic mix on intersection performance.

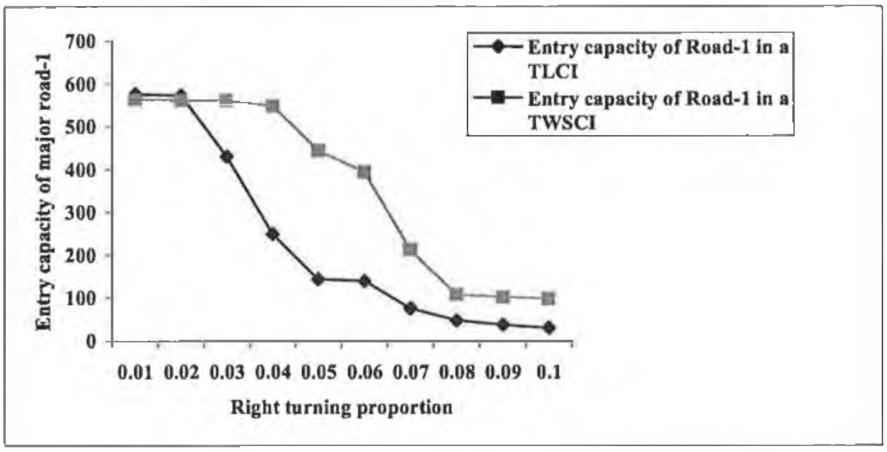


Figure 5. 10 Entry capacity of major road1 Vs right turning rate for mix traffic flow

For the single lane intersection, right-turning vehicles from a major road, (where RT, ST and LT vehicles share road space), generate manoeuvres which can block the ST and LT vehicles behind them. RT rates (RTR) of single-lane major roads thus have great impact on major-road capacity. In order to examine this for road entry, we varied right turning rate (RTR) of the roads from 0.01 to 0.1. Arrival rates were fixed at $AR_1=AR_2=AR_3=AR_4=0.15$, for this test case.

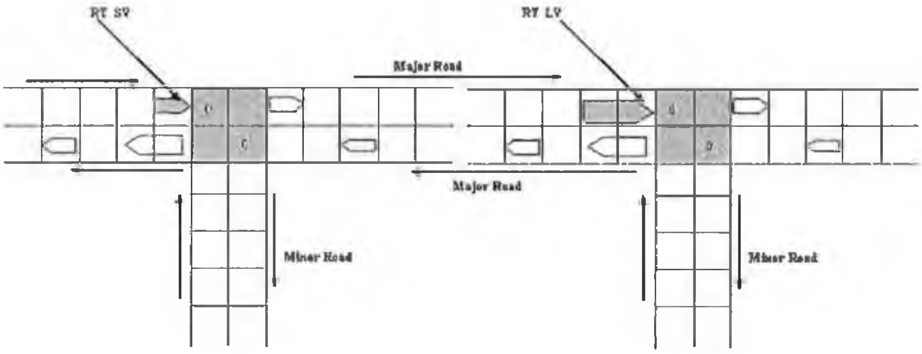
Figure 5.10 shows, unsurprisingly, that the entry capacity of the major road intersection declines with RTR increase. What is surprising perhaps is the low small RT %s for which this occurs for a TLC-intersection, and also to a lesser, but still marked extent for TWSC- intersection. For less than 10% of RT vehicles in the direction of flow overall, there is a decline in entry capacity of 80% plus. Yet these control features are common in urban networks. We conclude that capacity for mixed traffic of the major road declines drastically when the percentage of RTR increases for both TLC and TWSC intersections. TLC entry capacity curve is lower than that for the TWSC intersection, since the TWSC intersection permits free flow of traffic when the opportunity arises.

5.4 TLC T-intersection

In an urban system, the majority of the intersections are T-shaped and un-signalised. Recently Ceder and Eldar (2002) suggested that splitting an uncontrolled X-shaped intersection into two adjacent uncontrolled T- shaped intersections can improve both the movement and safety of traffic. Li et al. (2004) studied uncontrolled T- shaped intersection using cellular automata and investigated the phase diagram of the system and the effect of the turning car on the whole traffic situation. They focused on the traffic situations in the congested region. Their result suggested that when the ratio of the turning cars is small, straight through cars should give way to turning cars, while when the turning car rates is high, the turning cars should give way to the through cars, in order to optimize the system. Later Wu et al. (2005) again studied interactions between vehicles on different lanes and effects of traffic flow states of different roads on the capacity of T-shaped intersection based on the Li and et al. model. Both models were studied for homogeneous cases, but did not study complete traffic flow from minor T-road. Li et al. did not consider the flow from T-minor road and Wu et al., did not consider left-turning vehicles on the T-minor road for right hand driving. In this study, therefore our particular focus has been heterogeneous traffic flow at a controlled T-shaped intersection with complete flow from all direction. Here we considered left hand side driving, which is common to UK, Ireland, New Zealand and India. However the rules translate to right hand side driving directly. The schematic traffic flow of a T-shaped TLC intersection was presented in Section 3.1.2. The T-shaped intersection has 3 entry/exit roads. The vehicle from road-1 can either go straight or right turn. The vehicles from road-3 can go straight or left turn. Similarly from minor road-2 vehicles can turn either left or right. Early work on this CA model is reported in our publication, Deo and Ruskin (2006). Here, we give sufficient detail again for convenience to motivate the discussion and support interpretation of the results. Principal expression and turning manoeuvre rules are given in Chapter 4 and 5. Model features and vehicle manoeuvre at T-intersection are described below.

5.4.1 Model

Vehicle manoeuvre at TLC intersection are shown in Figure 5.11 with major roads and minor road as indicated. Both roads are single-lane with traffic flow in two opposing direction. We consider two phase light cycle for controlling the T-intersection (as in chapter 4, Section 4.2.2). In *phase-1* the traffic light is green for major road-1 and road-3 (and simultaneously red for road-2). The light then changes to yellow for major road-1 and road-3 and simultaneously to red for road-2. In *phase-2* the cycle repeats for road-2 green and vice versa.



(i) (ii)

Figure 5. 11- A right turning (RT) vehicle manouevres from major road at a single-lane two-way signalised T- intersection, (i) SV (ii) LV

Figure 5.11 shows a RT vehicle waiting to enter a T-shaped intersection. The space required is two cells for an SV or LV either from a major or minor road. The T-shaped junction is effectively half of the X-intersection, so manoeuvering rules are similar; detail expressions are given in Section 5.2.2.

5.4.1.1 Model validation

For the validation of our model to real data, a local T-intersection (Oscar Traynor road-Dundaniel road, Chapter 3, Section 3.2, Table 3.2) was considered. Arrival/ turning rates and ratio of vehicle types are based on data, collected by *Dublin City Council (April 2003)*. Here, the model was validated with respect to turning ratio and capacity per approach of the intersection. Figure 5.12 and Table 5.6 shows the simulated turning ratio and capacity from the model for an average of 10 runs of 10 hours each, this to explain variability such that seen as day-to-day.

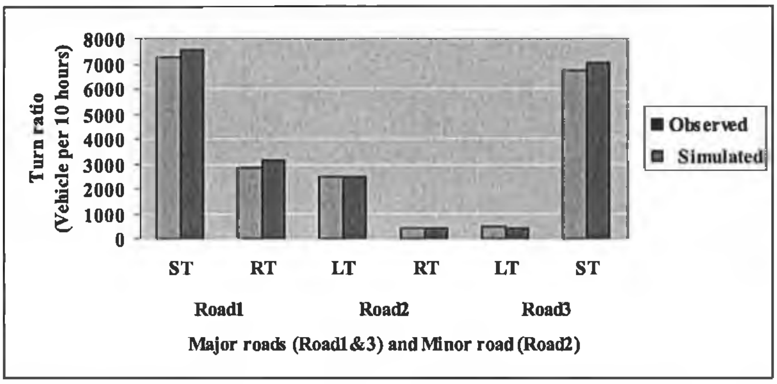


Figure 5. 12 Turning rates on major/ minor road

Table 5. 6- Comparison of observed and simulated capacity

Intersection entry capacity (vehicle per 10 hours)			
Road number	Observed Data	Simulated Data	% Error
Road-1	10117	10739	6.148
Road-2	2956	2893	2.131
Road-3	7196	7424	3.168

Figure 5.12 illustrate the observed and simulated turning ratio. The percentage error in the estimation of capacity was obtained from all approaches of the T-shaped intersection and the results are summarised in Table 5.6.This is tested statically and found that χ_2^2 for capacity e.g. significant at better then 0.001% level.

5.4.1.2 Model sensitivity analysis

This section is dedicated to verifying the effects of the different input parameters for the simulation. Elements explored in the sensitivity analysis included interval of green lights, traffic composition and right turning rate and arrival rate.

Table 5. 7 -Overall throughput with green light interval in seconds

Interval of green lights in seconds		
Road-1,3	Road-2	Throughput (vehicle per 10 hours)
70	20	21332
65	25	21324
60	30	21291
55	35	21257
50	40	21239
45	**45**	**20470**
40	50	18532
35	55	16556
30	60	14563
25	65	12705
20	70	11006

During a 5 minute period, the intervals of green light for major road-1 and road-3 and minor road-2 are listed in Table 5.7. The relationship between different settings and throughputs is also shown. Basically, throughput of the intersection increases as the green light period increases for the major roads. The reason for this is that green lights for major roads allow traffic of two roads to pass through the intersection at the same time, where as a green light for a minor road allows only one flow direction. From the table, it can also be seen that when both minor road and major road have symmetrical green lights, the throughput closely approximates the field value (bolded in the table).

Throughput increases when green light period increases in the major roads. Similar values obtained Wang (2003) for multilane control intersection for homogeneous traffic.

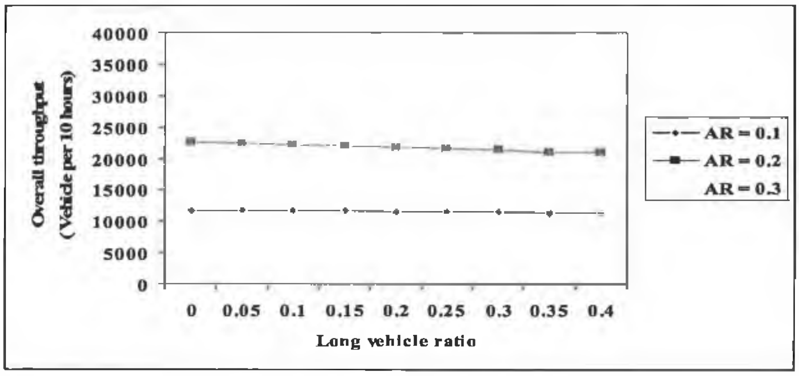


Figure 5. 13- Overall throughput vs LV ratio

Figure 5.13 Shows that overall average throughput was reduced in all cases for a significant proportion of LV in the traffic mix. All result here are based on the assumption that drivers are rational, (e.g. obey rules).

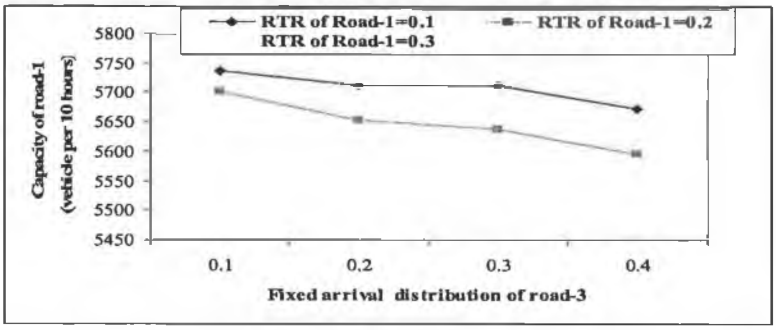


Figure 5. 14- Capacity of Major road-1

Figure 5.14 shows that the capacity of major road-1 declines with arrival distribution of the conflicting major-road-3 increases, where only one major road has RT vehicles in this model as it is a T-shaped intersection. TLC X-intersection for major road capacities yield similar relationships

5.5 Summary of results: comments on further developments

In urban roads, network flow will be determined by the intersection. Intersections are of the greatest important in traffic networks because of their effect on the movement and safety of vehicular traffic flow. At an intersection route decisions are implemented and crossing of other traffic streams occurs. To perform this manoeuvre a vehicle may diverge from, merge with, or cross the paths of other vehicles, so it is easy to incur jams and accidents at the intersection. For this reason we pay particular attention to the study traffic flow characteristics at different configurations.

We have described two-component cellular automaton methodology for modelling *heterogeneous motorised* traffic flow at a single lane two-way signalised and un-signalised T and X-intersection and effect of mixed traffic flow on performance reproduced.

The most important finding is that, in the case of traffic light controls, capacity and throughput of the intersection is increased provided that there are enough vehicles on all roads to utilize the green light period (i.e. flows). The advantage of a signalised intersection compared to the TWSC case is that the former overcomes the blockages caused by priority rules operating independently of deviation of heavy flows.

The model has been validated in part against observed data, (from a local single-lane two-way intersection in Dublin, Ireland) on long/short vehicles ratios passing through the intersection. The model can be shown to satisfactorily replicate heterogeneous motorised traffic flow on a single-lane two-way road.

The model, in its present form, has potential for use in evaluating traffic management measures, such as banning certain categories of vehicles from exclusively single-lane roads, with peak time predictions of throughputs and similarly. The model can also be used to predicate the effects of the single-lane two-way control and uncontrolled intersection for both homogeneous and heterogeneous traffic.

It is clear that investigations into the nature and impact of long vehicles in exclusively single-lane traffic are vital to understanding urban flows. Such shared roads constitute bottlenecks and dictate feeder traffic flow to larger arterial routes; the proportion of

LVs in the local traffic has a significant impact on this.

Further work on other kinds of performance measure such as delay time, queue length and so on, is also describable and possible using the model forms developed.

5.5.1 Further research

A flared approach to a configuration is one that allows vehicles performing different manoeuvres to start/ complete these, while other vehicles occupy the "single-lane" road. Figure 5.15 presents an example of a flared approach allowing space for 6 additional SV, 3 each side of single lane or 4 additional vehicles, 2 each side (one SV + one LV) at the stop bar or at the red signal. It is proposed to further develop the model to incorporate the effects of variation in roadway geometry e.g. an additional lane for RT and LT traffic. If there is no minimum space or vacant cell for RT and LT vehicle manoeuvring, both RT and LT vehicles must stay in the additional lane so ST vehicles in parallel spaces will be less affected. However, this alleviation is limited to the flare space. Introduction of a flare on a single-lane road improves realism for these models developed for heterogeneous urban traffic flow in restricted inner-city routes.

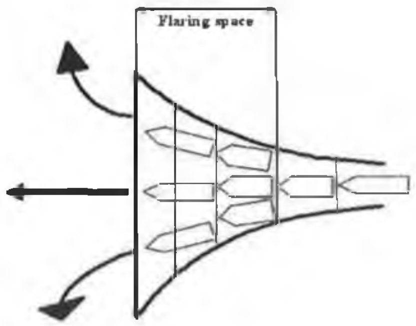


Figure 5. 15- Intersection with flaring right and left-turn road

While the flared entry provides additional capacity at the intersection, it is a limited solution to multiple entries to a road configuration.

Chapter 6: Yield sign and traffic light controlled single-lane roundabout

6 Yield sign and traffic light controlled single-lane roundabout

There are many crossings in typical urban traffic networks and these are prone to cause delays. One crossing configuration, which at least maintains same direction flow, is the roundabout. Safe and efficient operation of the roundabout is dependent on the effectiveness of measures to reduce vehicle speed, (Technical council committee 5B-17 of Institute of Transportation Engineering, USA. 1992). Slower traffic movement means a large roundabout radius is no longer needed, and thus use of much smaller roundabouts is now feasible. *Mini roundabouts* are frequently used in urban and inter-urban areas. For example, in housing estates, industrial estates and for linking small urban roads. The roundabout is a channelised intersection where traffic moves around a central island, clockwise for left-side driving as in Ireland, India, Japan, Britain, Australia and counter clock wise for right-side driving as in America and most of Europe.

Traffic flow around a central island is one-directional and also operates with controlled case. This may be *yield control* at the entry point, giving priority to vehicles within the roundabout. Yield-at-entry is one of the most important operational elements of the modern roundabout. It is still controversial whether control at an "intersection" under a signalised or un-signalised scheme via a roundabout is more efficient (Fouladvand et al, 2004). Wang and Ruskin (2002) have investigated a number of properties of single-lane roundabouts using a CA ring model under the offside priority rule for homogeneous traffic. Hyden and Varhely (2000) experimentally studied small roundabouts in a Swedish city for speed reducing measures. The results showed that the roundabouts reduced the speed considerably at the junctions and also on roads linking roundabouts. Al-Madani (2003) studied the vehicular delay comparison between a police-controlled roundabout and a traffic signal and found that for low-volume situations non-signalised methods seem to show better performance, while for high-volume situations, traffic light signalisation is better. Fouladvand et al (2004) investigated the characteristics of traffic at isolated roundabouts using a cellular automata and car following framework. They compare results with traffic-light signalised schemes. These developed models focus on homogeneous flow and ignore many important features of real heterogeneous traffic. We now discuss our approach to modelling *heterogeneous* traffic manoeuvres at controlled and uncontrolled single-lane roundabouts, using our basic CA rule and have investigated roundabout performance for various traffic flows at yield sign controlled and signalised roundabout.

6.1 Model

Yield sign controlled roundabout (YSCR): In our most general form, a roundabout connects four entry/ exit roads. The schematic for traffic flow is described in Chapter-3 (Section 3.1.3, Figure 3.3) and real data are presented in Table 3.3. The approaching vehicles *yield* to the circulating traffic flow in the roundabout, and are allowed to enter the roundabout, provided that a vehicle observers the optimum condition (for rational drivers) for vehicles onto driving on to roundabout. According to the rules of the road the *circulating vehicles* around the central island have higher priority than vehicles entering the roundabout, so this adds to delay time on entry. Figure 6.1 shows the requirements in terms of free cells for vehicles at a single-lane four-way roundabout. SV and LV require 3 and 4 free cells marked for manoeuvring. A "0" means that the cell is free or vacant and "L" means that the cell needs to be either vacant or occupied by vehicle that will turn left.

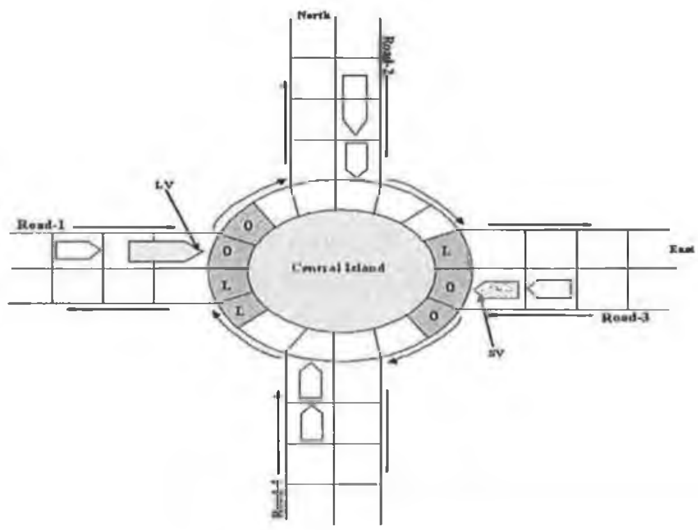


Figure 6. 1- Vehicle manoeuvring at YSCR

Traffic light controlled roundabout (TLCR): Although yield control of entries is the default at roundabout, roundabouts have also been signalised at each entry (FHWA). For model purposes, traffic signals are installed to control the traffic flow of entries on to the circulatory roadway. Two phases of traffic signal control are adopted, similar to that for the controlled intersection, (see Chapter 4).

6.1.1 Entering roundabout ring and problem description

In principle, each incoming vehicle approaching the roundabout can exit from each of four out-going directions making appropriate turning manouevers by moving around the central island. Here we consider the vehicles from all approaches to LT, ST or RT. Before entering the roundabout a vehicle is randomly assigned a particular number in order to guarantee that these vehicles will eventually turn into the destination road (see Chapter-4). Once a vehicle is permitted to enter the roundabout, it continues moving until it reaches the pre-determined exit direction. Each circulatory vehicle thus moves a portion of the way around the central island, but does not pass the entry point again, (exception in the case of rare confusion, which is discussed).

Roundabout ring and update rule: The sketch map of roundabout is shown in Figure 6.2. The roundabout is a sub-system containg16 cells. At the joining-up points marked as cell number (0, 4, 8, 12), vehicles drive in, and at the taking-off points marked with cell number (3, 7, 11, 15), vehicles drive out.

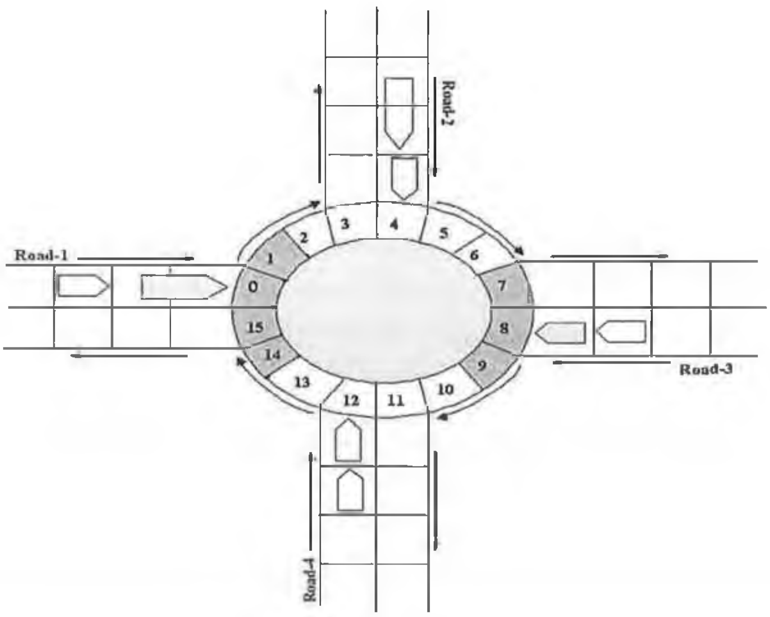


Figure 6. 2- The sketch map of inner lane roundabout

In view of the symmetry of the four entry roads (road-1, road-2, road-3 and road-4) in these cases and for the sake of clarity, we simply discuss and demonstrate the situation for a vehicle coming from road-1 to pass through the roundabout. There are three choices for the vehicle moving clockwise:

1. to LT (road-2), in this case it passes cells (0 to 3) then turns into road-2.
2. to ST , in this case it will pass through cells (from 0 to 7) and exits at road-3 (i.e. straight ahead) .
3. to RT, cells traversed on the ring are 0 to 11, then vehicle exit on road-4(i.e. right turning).

The updating regulation is based on the procedure as in single-lane road (See Chapter-4, Section 4.1.1).

Problem description: The entering vehicle can freely move around the roundabout until it reaches its exit. In the case of *yield sign* control, an approaching vehicle to the entry points of the roundabout should yield to the circulating traffic. This procedure is continuously applied to all approaching vehicles in all four directions. The above mentioned driving rules establish a mechanism responsible for controlling the traffic at conflicting points. This mechanism blocks any direction which is conflicting with a flowing one, therefore producing a queue in the blocked direction. In contrast *signalised roundabouts* are controlled via self organized mechanism of blocking. It is evident that in the case of a congested traffic situation, the probability of finding a space to enter to the roundabout is lower. This leads to global blocking of other directions and gives rise to the formation of a queue. In this situation, the yield sign roundabout is inefficient, and signalised roundabout gives a better performance.

6.2 Simulated results and discussion

We let the roundabout evolve for 36, 000 time steps (seconds) which is equivalent to a real time period of 10 hours and average the results of 10 independent runs unless other wise specified. We validate our proposed model based on real data (see Table 3.3) that were collected by DCC and used for baseline. Secondly, we perform sensitivity analysis and compare the result with fixed time traffic light controlled intersection.

The real world dataset is just one possibility in a range of such possibilities. It has a specific role in validation or trying to see how well the model reproduces the real-world, but in order to test system robustness, it is just a guideline or basis for "typical" values.

6.2.1 Result of YSCR

In the following, we consider validation of the model and sensitivity analysis of the yield controlled roundabout.

6.2.1.1 Validation with real data

Figure 6.3 and 6.4 are shown observed and simulated RT SV and LV capacity for each quarter hour intervals ending. We feed real input data such as arrival rate, turning rate and traffic composition into the simulation program. After running the program we compare the simulated output with real output.

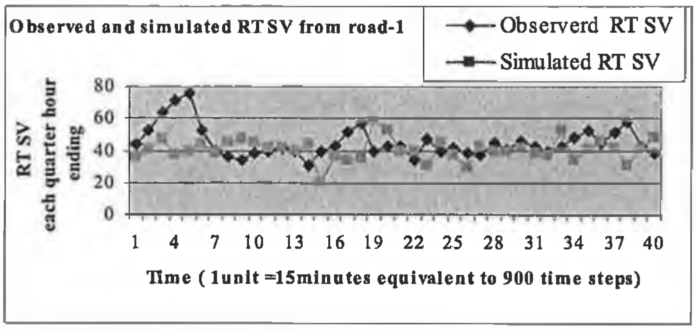


Figure 6. 3- Comparison of observed and simulated RT SV in quarter hour intervals

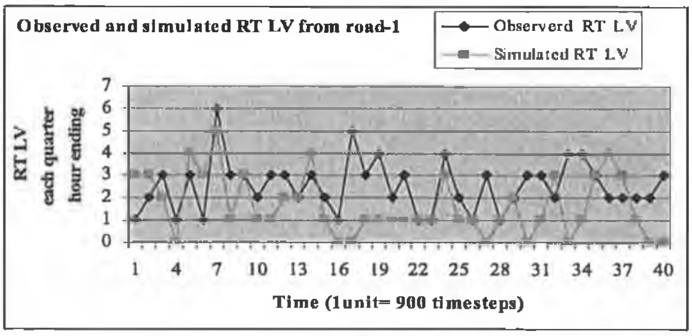


Figure 6. 4- Comparison of observed and simulated RT LV in quarter hour intervals

In Figure 6.3 one can show that at time unit 3 to 6 the observed values are much higher than simulated values. Similarly, in Figure 6.4 the observed values are higher from time unit 17 to 21. These observed data is just a single realisation so it is difficult to estimate how good the model reproduces the real world situation. Therefore more real data set is needed to validate the model against real data is a question for further research.

Table 6. 1 - Comparison of observed and simulated capacity of the roundabout

capacity of the roundabout (vehicle per 10 hours)			
Road number	Observed Data	Simulated Data	% Error
Road-1	5553	4653	-16.20
Road-2	4561	4133	-9.38
Road-3	2765	2539	-8.17
Road-4	4200	3829	-8.83

Table 6.1 present the comparison of observed and average simulated capacity of each road. Baseline attempt is to reproduce the "field data" based on the data from the collected set. We use the chi-square test to test the validity of the distribution. The test statistics χ_3^2 is with the chi-square distribution with 3 degrees of freedom for the capacity of the roundabout is better than the value corresponding to the 0.001 significance level. This is because the real configuration has features which are not reproduce our model e.g., there is space for RT vehicle and also the physical size of the roundabout is not equal.

6.2.1.2 Sensitivity analysis

Individual road performance - total delay

The following assumptions are made in the simulation:
Arrival rate from all approaches are taken to be equal ($\lambda_1= \lambda_2= \lambda_3= \lambda_4$), and are varied from 0.2 to 0.8 (equivalent to vehicle 720vph to 2880vph). For all road LT: ST: RT=0.4:0.4:0.2., SV: LV=0.8:0.2 for all scenarios considered. Total delay time is determined by time for which a vehicle cannot move when approaching the roundabout plus time spent waiting to enter the roundabout (i.e. start its manoeuvre) in the last 200 time steps.

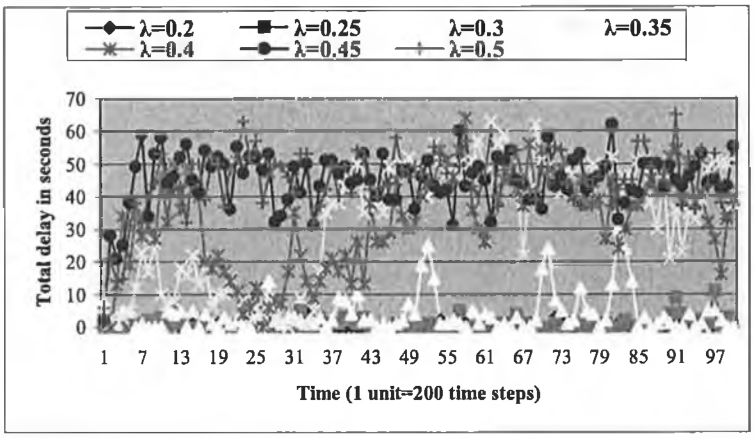


Figure 6. 5- Total delay of road-1 for heterogeneous traffic with $\lambda_1 = \lambda_2 = \lambda_3 = \lambda_4$

The result of total delay for heterogeneous traffic case is shown in Figure 6.5. As arrival rate increases, the total delay of the road increases. The delay is nearly zero when $\lambda \leq 0.3$. Additionally, delay increases after an initial period of 30 time steps, for $\lambda = 0.35$, while for $\lambda = 0.4$, it increases rapidly, drops after a short period, increases again, and, for the duration of the run, stays mostly high thereafter. For $\lambda \geq 0.4$, delay increases rapidly and thereafter stays consistently high, (fluctuating around 50 seconds – also for $\lambda = 0.45, 0.5$). This emphasises that low volume reflects the high efficiency of the roundabout. In heavy traffic ($\lambda \geq 0.4$), we see that delays rapidly increase. This is due to gridlock in road-1 which occurs as a result of no space available on the roundabout.

The result indicates that this mini roundabout flow is efficient when arrival rate is less than 0.4.

Relation between throughput and turning rates

The relationship between throughput of the roundabout and turning rates can be observed from Table 6.2 Arrival rate $\lambda_{1=2=3=4}$, which varies from 0.15 to 0.55 (equivalent to 540 vph to 1980 vph). The ST ratio remain constant=0.5. The RT ratio increases from 0.15 to 0.45 and LT ratio vary from 0.35 to 0.05 respectively. The traffic composition SV: LV= 0.8:0.2. In all scenarios the simulation is carried out over a long period (equivalent to 80 hours or 288000 time steps).

Table 6. 2 Throughput of the roundabout

$\lambda_{1=2=3=4}$	Right turning ratio			
	0.15	0.25	0.35	0.45
0.15	148757	148728	148725	148919
0.25	224989	224718	224383	223793
0.35	283466	274683	249892	228873
0.45	**305655**	**274826**	**249632**	**229134**
0.55	**305631**	**274866**	**249643**	**229112**

In Table 6.2 when arrival rate increases throughput increases up to certain extent. Turning ratio has less impact on throughputs. When $\lambda_{1=2=3=4}$ is greater than and equal to 0.45 and RT ratio varies from 0.15 to 0.45 entrance roads are saturated and turning rates do affect throughputs.

Overall roundabout performance - throughput

The impact of traffic composition on throughput can be shown in the following diagram. For each experiment $\lambda_1 = \lambda_2 = \lambda_3 = \lambda_4 = 0.25$ (equivalent to 900 vph), LT:ST:RT=0.4:0.4:0.2.

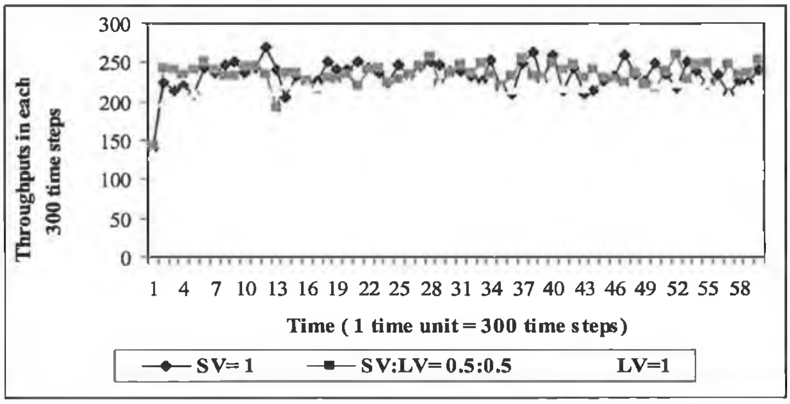


Figure 6. 6- Overall throughput of the roundabout for $\lambda_1 = \lambda_2 = \lambda_3 = \lambda_4 = 0.25$

Figure 6.6 shows that in the case of 100% SV, (50% SV + 50% LV) and 100% LV traffic roundabout remain free flow all the time. When LV percentage increases throughput decreases. In contrast, homogeneous SV traffic increases the throughput in comparison to mixed traffic and homogeneous LV traffic. In conclusion, LV proportion increase is clearly decreasing the throughput so it does affect the urban traffic flow.

6.2.2 Comparison to TLCR

Let us now compare the yield sign roundabout performance with signalised roundabout. This comparison is our main motivation for studying roundabout characteristics.

Let us replace the yield sign with a traffic light at roundabout. We consider here fixed time signalisation (as in chapter 4). Arrivals rates (λ) of four roads are taken to be equal and constant for a given run, but vary over the range of 0.1 to 0.3, (equivalent to 360vph to 1080 vph), for turning proportion of all approaches fixed to LT: ST: RT=0.25:0.25:0.5 and vehicle composition SV: LV=0.6:0.4. All parameters mentioned are the same for YSCR and TLCR. In case of TLCR traffic light phase is fixed to G: Y: R = 45:5:50.

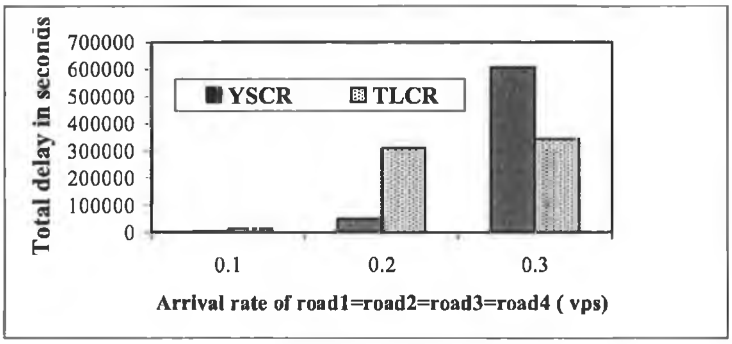


Figure 6. 7- Comparison of total delay with TLCR

In Figure 6.7 We compare the performance of YSCR with TLCR. According to the diagram for relatively low arrival rate, YSCR shows a better performance, and give rise to lower or closer-to-zero delay. Conversely, for high arrival rates, (a more congested traffic situation), controlling the roundabout by signalised traffic light leads to better results. For sufficiently low arrival rate, the approaching vehicles can easily fill the required space gap in the circulatory flow; hence, these can enter the roundabout without much time lost, whereas in a signalised scheme, most have to wait at the red light even if the flow is negligible on the roundabout. This shows that below a certain congestion level, the yield sign roundabout efficiency is higher than the fixed-time signalised one.

6.3 Summary

In this study, a microscopic simulation model was developed to replicate the flow of heterogeneous traffic on mini roundabouts for urban roads. The influence of proportion of LV, arrival rate change and % of RT vehicle on the traffic flow has been analysed. This analysis shows that throughput increases with arrival rate increase but in contrast throughput decreases when RT ratio increases.

Traffic composition impacts on the overall performance of the roundabout as well as on individual roads. Homogeneous (SV or LV) and mixed traffic remain free flow on the roundabout when arrival rate is less then or equal to 0.25.

The main conclusion is that delay is higher at yield sign control intersection in comparison to fixed time signalised intersection when traffic volume is high. The only delay in entering a roundabout is the wait for a gap in the circulating traffic. To this end, the sole purpose of a traffic signal is to allocate time between competing traffic streams.

Finally, this work serves as a stepping stone for the analysis of multi-lane roundabouts for mixed traffic, which we have studied in preliminary cases (Feng et al., 2007, Chapter 8). The complexity of the multilane problem appears to be significantly greater in the circulating roadway and gap acceptance behaviour of entering vehicles in mixed traffic.

In summary, we have presented the yield sign controlled and traffic light controlled single-lane roundabout operation for heterogeneous flows, and for various levels of input parameters. We have also discussed the problem of yield sign controlled roundabout at congested traffic situation and advantage of controlled roundabout. These models were studied with different performance measures using various arrivals rate both for homogeneous and heterogeneous traffic situation. Comparison of the total delay with traffic light controlled roundabout also presented. The study allows the traffic planner and management to gain better understanding of overall traffic flows and roundabout efficiency.

Chapter: 7 Visualisation and views of multilane traffic flow

7 Visualisation and views of multilane traffic flows

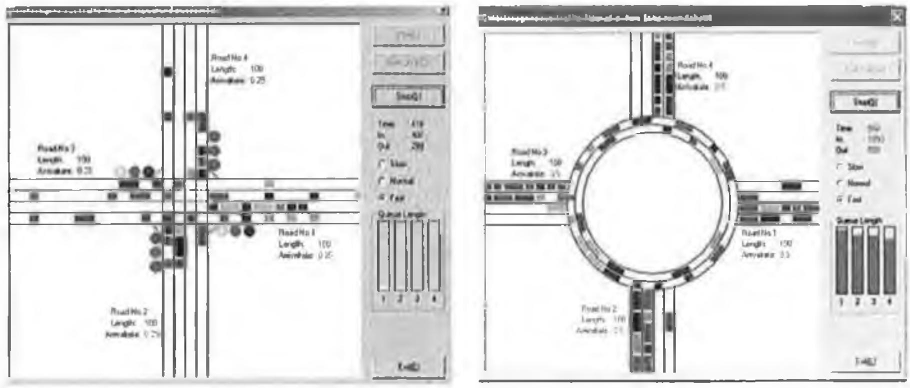
In terms of illustrating the complexity of mapping or modelling vehicle manoeuvres at different road features, it is useful to look at a typical multi-road, multi-way situation through e.g. a schematic with simple animation. The value of such a visualisation is not only to appreciate the way in which relatively small changes in the parameter values can radically affect performance, but also to provide valuable information in terms of knock-on effects when linking road features as well as impacts to be expected from radical changes such as

- Introducing different or more rigid controls, (such as lights replacing "yield" or stop control)
- Substituting the type of configuration, e.g replacing an intersection with a roundabout
- altering traffic mix proportions

An analytical model of the whole traffic system is frequently unrealistic, so that simulation with visualisation elements can be effective in assessing behaviour of large-scale and complex systems and their properties. For the purpose of simulating heterogeneous (binary mix of motorised vehicles), at microscopic level, traffic models discussed by Deo and Ruskin (2006) are described in this work, and some supplements in *visual form* are developed here for the multilane case,(as part of collaborative work with co-authors, published Feng et al (2007)[vi], Appendix B).

Figure 7.1 shows the visualisation interface of the developed simulator for two-lane, two-way signalised intersection and yield control roundabout. The visualisation system was developed in Visual basic (VB) and represents traffic flows on urban roads. The simulation of the urban traffic system at microscopic level was used to examine and explore turning movements at the intersections, as displayed by the animated vehicles. Allowing for different vehicle mix, light cycles and/or priorities or similarly, thus can provide a good picture of important flows. Simple checks on queue length are also stored, while traffic speed can also be varied on these choices.

[vi] China –Ireland grant supported visit of Prof. Yuefeng Liu (Integration of Spatial Information Lab., Remote Sensing Institute, Peking University, Beijing, China)



(i) (ii)

Figure 7.1- Simulated heterogeneous traffic flow snapshot at multilane (i) intersection and (ii) roundabout

Multilane intersection illustrated uses a fixed time signal control scheme. The traffic light cycle, for convenience, was chosen to be red: yellow: green = 50:3: 47 seconds respectively, which is within the normal traffic light range of 30-60 seconds for the green and red lights. Whereas roundabout controlled by Yield sign control rule or giving way rule. The giving way rules applicable here, are thus

1. vehicles entering must give way to those already on the roundabout and
2. vehicles on the roundabout outer ring must give way to vehicles moving from the inner ring, prior to the latter exiting

Intersection operation was simulated typically for around 6000 time steps for various parameter settings for a range of arrival rates and for different proportions SV: LV in the traffic mix.

7.1 Methodology

Lane allocation assumptions: The lane allocation process of the four entry roads of the intersection and roundabout is as follows: for the *two-way, two lane intersection*; a left-turning/right-turning (LT/RT) vehicle uses only the left-lane/right-lane respectively, straight ahead (SA) vehicles can use both lanes. For the *two- lane roundabout:* (also with 4 entry/exit roads), this corresponds to the simplifying assumption that vehicles stay in the left lane of the entry road/outer ring of the roundabout if taking the first exit, the right lane of the entry road/inner ring of the roundabout if taking the third exit and either lane if taking the second (straight across) exit. Assignment to lanes is random on vehicle arrival to the entry road. A vehicle, entering the roundabout, is also randomly assigned an exit and its destination does not change during traversal.

For a vehicle waiting to enter an intersection or roundabout, the Multistream Minimum Acceptable Space (MMAS) is that space required to complete a manoeuvre before the cell(s), targeted for occupation during the manoeuvre, are populated by an oncoming vehicle, assuming a constant speed of one cell moved in one time step for a given driver distribution. The flow path for any vehicle entering the roundabout is taken to be the hypotenuse of the triangle formed from one step ahead and one to the left. Diagonal moves are taken, for simplicity, to be of one time step duration. Other possibilities and a detailed description of the model is described in the paper Feng et al. (2007).

Intersection and roundabout manouevres: For the case of a two-way unsignalised intersection, priority is usually given to those cars on the major road, with minor road traffic. In the case of a controlled intersection, priority-sharing or give-way rules are replaced by fixed waiting periods, determined by the traffic light cycle. Space requirements for left and right lane, flows and interactions at a roundabout entrance are illustrated in paper (Feng et al. , 2007, Appendix C).

Vehicle Update rules: In order to "progress" vehicles in our simulations and animation, we take the state of cell n at time step t to be denoted as C_n^t.Detailed expressions for the homogeneous case, i.e, short vehicles (SV) only, for single and two-lane intersection and roundabouts update rule are similar to those given in (Wang and Ruskin, 2002, 2006). For long vehicles (LV), the cell length twice that for SV, thus the update rules for LV at time t are the following.

If $C_n^t > 2, C_{n-1}^t > 2$ and $C_{n+1}^t = 0$, then, $C_{n+1}^{t+1} = C_n^t - 1, C_n^{t+1} = C_{n-1}^t - 1$, and $C_{n-1}^{t+1} = 0$

If $C_n^t \geq 2, C_{n-1}^t \geq 2$ and $C_{n+1}^{t+1} > 0$ then, $C_n^{t+1} = C_n^t$, and $C_{n-1}^{t+1} = C_{n-1}^t$

For movement along the road, then for cells n and n -1 (occupied by a LV at time step t), cell n + 1 at n must be checked. If vacant, then both cells n -1 and n + 1 must change simultaneously with cell n occupied by the LV and with tied cells (now n, n+1) retaining the same state value. If cell n+1 is occupied, however, no movement occurs and the state values for n and n - 1 do not change.

7.2 Illustrative results of multilane intersection

Arrival rates were initially taken to be the same for all directions, $AR_{1=2=3=4}$ and range of arrival rates considered was 0.10 to 0.65 and different proportions of SV: LV in traffic mix.

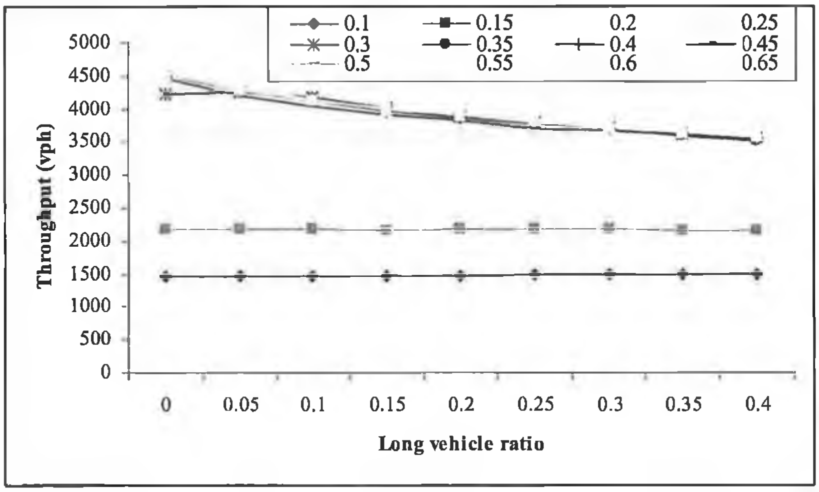


Figure 7. 2- Throughput (total No. vehicles) versus Long-Vehicle ratio for range of arrival rates (in vehicles per second) at signalised two lane intersection.

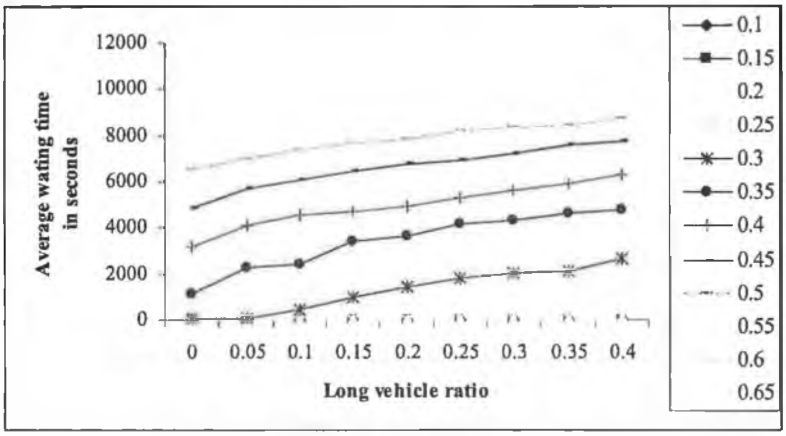


Figure 7. 3- Average waiting time versus Long-Vehicle ratio for range of arrival rates (in vehicles per second) at signalised two lane intersection.

Figure 7.5 and 7.6, which show that, while throughput is steady for low arrival rates (for all distributions of SV: LV considered), it cannot be sustained as arrival rates increase, and also declines with increasing LV proportion. For different arrival rates from each road or direction, for example for (AR1 = AR3) > (AR2 = AR4), the traffic light cycle can be adjusted to allow for shorter waiting times for roads 1 and 3, e.g., by reducing red to 40 seconds, while yellow, and green do not change. While average throughput is improved slightly, with corresponding reduction in waiting time, the overall pattern remains the same. With roads 1 and 3 treated as major, relative red and green periods (yellow unchanged), can typically be around 1:2, but the LV ratio remains a key factor in performance for this simple exapmle, with critical LV values around 35-40% of total traffic under given model assumptions. Right turning: left turning and straight through rates were taken to be 0.25: 0.25: 0.5 vps throughout.

7.3 Conclusions

We have outlined here, the development of a simple model for two-lane traffic for binary vehicle mix. While focus here is on the rational driver case, the model design allows for different driver type, through space-time considerations for manoeuvres. There is clear requirement for further investigation of the range of conditions applying for multiple-lane mixed traffic at urban control features. This visual simulation of mixed traffic flow offered a realistic view with moveable viewpoints for simultaneously realising the effect of changing behavioural elements of vehicles. This is developed in detail for configurations in the following chapters, but it was useful to see the effects of changing assumptions w.r.t. heterogeneous or homogeneous flows.

Chapter: 8 Conclusions, discussion and future work

8 Conclusions, discussion and future work

This thesis extends, to heterogeneous traffic the original homogeneous Minimum Acceptable sPace (MAP) to simulate single-lane one-way, single-lane two-way intersections and roundabout. It also extends Multistream Minimum Acceptable Space (MMAS) CA models to a multi-lane intersection and roundabout for the heterogeneous traffic version. This not only facilitates understanding of the interaction between the drivers, but can also be applied to situations for which headway distributions are insufficient to describe traffic flow (Wang and Ruskin, 2006).It is based on the MAP method to study the multi-lane case.

Real traffic is not homogeneous. The extension formulated in this work, incorporates both heterogeneous vehicle types and (limited) heterogeneous driver behaviour at a signalised intersection, specifically for rational and aggressive driver. The motivation behind this approach is two-fold.

First, the systematic enhanced development of MAP and MMAS models can other new insights, and possible explanations, of observed traffic phenomena, that originate in the heterogeneous character of the traffic flow. Second, to lay a foundation for linking intersections in network segments.

In this closing chapter, we summarise the *heterogeneous* model development in this thesis. A short summary will be followed by the most important finding and by proposing research directions.

8.1 Brief summary and main findings

The original MAP and MMAS model assumes traffic to be homogeneous but the "acceptable space criterion" can be extended to address this limitation and examine the effect of heterogeneity. In the first instant the work has dealt with binary heterogeneity of vehicle SV and LV and described a model for heterogeneous motorised traffic flow; a new approach based on *two-component cellular automaton* benchmarked methodology has been proposed. Nevertheless, the approach is readily generalisable to other unit multiples of traffic type. A more radical change, to accommodate small vehicles, would be to looked at multiple cell occupation i.e. SV, LV's as multiples of smaller cell units.

Road features, including TWSC intersections, YSCR (yield sign controlled roundabouts) and fixed time signalised intersection and roundabout have been studied. These choices contrast priority rules with fixed traffic light signals for common urban configurations. Small variations to some of these are also outlined. In addition, a preliminary study of driver type for heterogeneous traffic is also considered. To our knowledge, our heterogeneous model is the first to categories different types of vehicle, based on space requirement for roadways of the developed world. Both vehicles and driver differences can be allowed for as well as different control systems. A further interesting issue might be to combine driver behaviour with vehicle type under different "flashpoint" situations, such as blockages, broken light etc. To date, our 2-CA models has successfully simulated heterogeneous (SV+LV) and homogeneous (SV or LV) traffic (Deo and Ruskin, 2006, and Feng and et al, 2007) traffic flow at urban road.

We found aggressive driving leads to increased throughput at a signalised intersection even for the toy example studied. However, in future real world data should be require for validation of the model studied.

Capacity at single-lane TWSC,TLC and T-intersections for mixed traffic flows are found to decrease when percentage of RTR increases, while throughput increases when arrival rate increases both for single-lane TWSC and TLC intersection and then decrease when arrival rate (λ >0.26). The indications are, however that the TLC intersection improves cross flow in comparison to TWSC. In the case of *homogeneous* (100 percent SV) traffic performance is clearly improved compared to *homogeneous* LV traffic at a TLC intersection, whereas interestingly at TWSC 100 percent LV traffic performs better. Also we note that that throughput increases as green light period's increases for the major roads and compare with Wang (2003) broad finding.

For single-lane YSCR roundabout, we found that the efficiency of roundabouts depends on the low-volume of traffic. In the case of low arrival rate homogeneous (SV or LV) and heterogeneous (SV+LV) traffic remains in free flow with performance for 100 percent SV traffic better than for (SV+LV) traffic. In comparison to YSCR, TLCR roundabout has better operational performance in more congested traffic situation i.e. higher volume situations.

For two-lane two-way intersection and roundabout models developed as part of the collaborative project, the model developed with different lane-allocation patterns with binary mix traffic shows throughput decreases when the proportion of long vehicles increases for a given arrival rate, with waiting time and queue length correspondingly increasing (Feng and et al, 2007).

Results have been validated and sensitivity analysis supported by comparison with real local traffic data, collected by DCC.

8.2 Integrated picture and future direction

The 2-CA models developed are capable of simulating binary mixes, as well as homogeneous traffic (SV or LV) and also accounting for distribution of driver behaviour at various road features. Much of the work has been presented and published in conference proceedings/journals, (Deo and Ruskin, 2005, 2006, 2007 and Feng et al, 2007). Our models overcome some of the limitation of MAP and MMAS models for homogeneous vehicles and attempt to move close to the real traffic flow picture for the urban context.

It is clear that investigations into the nature and impact of LVs in single-lane traffic are vital to understanding urban congestion. Such shared roads constitute bottlenecks and dictate feeder traffic flow to larger arterial routes; the proportion of LVs in the local traffic has a significant impact on this.

Limitation of the model which has been suggested by the work to date is:

1. There is clearly need to source other more extensive real data sets from both Dublin and other location.
2. In the simple model describe here, additional factors such as longer start-up time are not specifically incorporated, but would also add to delays caused by LV manoeuvres.
3. Only one type of LV is considered here. Clearly, very large buses and trucks will have a different impact on road capacity. See comments on multi-cellular representations of SV and LV as part of an extended distribution.
4. Speed of the vehicle is constant (i.e., in the case of all single-lane work) and

this could be extended in several ways, not least to allow for driver type.

5. One major drawback of this model is that these models are built for specified intersection or roundabout geometries, but for different geometries the principles should be similar.

Strength of the model and / or Suggestions from this research:

The micro model implemented using cellular automata (CA), can simulate heterogeneous and homogeneous traffic in urban road features. The model has demonstrated performance measures which reproduce both heterogeneous and homogeneous traffic conditions well.

The results of this research suggest that in the case of busy single-lane junctions and roundabouts, restrictions on LV movement will improve throughput to some extent. In the situations studied here, the primary justification for banning LVs is capacity and throughput rather than safety. However, our sensitivity analysis would permit quantification of improvements that might be expected, in controlling for other factor.

Furthermore we got similar result when we study the sensitivity analysis for different geometries (intersection and roundabout) using similar input parameter values for heterogeneous traffic flow. We conclude that LV inclusion does affect urban traffic flow, so that building heterogeneous models is necessary for realistic management and control purposes.

8.3 Contributions

The objective of this research is to study the impact of inclusion of long vehicles on traffic performance in an urban environment.

This thesis contributes to in the following respects:

- A frame work of heterogeneous motorised traffic is proposed. It is based on a new two-component cellular automaton.
- Based on the concept, single-lane, multi-lane junctions and roundabouts are developed in order to study the heterogeneous and homogeneous flow in different configurations.
- Aggressive diver behaviour is introduced to study the aggressive behaviour at signalised intersection.
- Sensitivity analysis performed to evaluate alternative conditions for real world situation.

8.4 Future research direction

In this final section, we consider future research directions that follow naturally from the research described. Firstly, we consider improvements to the models as presented in the thesis in terms of accuracy or feature addition. Secondly, we would like to add to the model the potential to allow for temporary road /lane blockage for the same and different geometries. Thirdly, we would consider detail the notation of linking configurations towards network control of heterogeneous traffic flow operation. Finally, we describe the long term research goal.

8.4.1 Improvements of the models in terms of accuracy or feature addition

8.4.1.1 Investigation of the effectiveness of driver behaviours and model verification

At signalised intersections driving behaviour is one of the main factors contributing to the safety level. This behaviour, with respect to yellow signal violation or obedience, is examined at a signalised single-lane one-way intersection in Chapter-4. We suggest for further research to investigate the other performance measures for the same and/ or different configurations in order to create a better picture by relating intersection related factors to behaviour and safety effects.

Whichever the traffic flow model is chosen, the model should calibrated and validated with real data. In future, the effectiveness of aggressive driver on signalised intersection of this thesis will be studied with model parameters extracted from real data.

8.4.1.2 Investigation of Avg. change in delay and avg. waiting time per vehicle

Delay is a complex measure and is dependent on a number of variables, including quality of progression, traffic volumes, signal timing parameters and intersection capacity.

In this thesis we investigated the total delay (summation of all time steps for which it cannot move when approaching the intersection, plus time spent waiting to enter the intersection or roundabout) both for heterogeneous (SV+LV) and homogeneous (SV or LV) traffic.

However there are also options possible such as:

- Avg. change in delay per vehicle
- Avg. waiting time per vehicle

These performance measures need to be incorporated in the model for further research in future.

8.4.1.3 Ternary mix traffic

The main contribution of this thesis is the presentation of a heterogeneous binary mix traffic flow modelling using cellular automata methodology. The same method can be applied to *ternary mix* traffic flow model. For example, one cell is occupied by one particle corresponding to a standard car (SV), long vehicles (LV) two cells are required and three cells are taken for very long vehicle (VLV) for simplicity. Typical configuration of the road is shown in the following Figure 8.1.

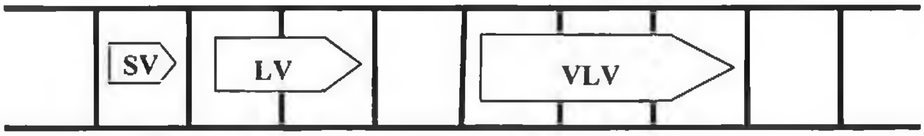


Figure 8. 1-Space representation of SV, LV and VLV vehicle

Incorporation of *ternary* mix traffic (cars + buses (equivalent vehicles) + very large trucks/ buses) i.e. SV, LV and VLV on urban roads: (recently, Northwestern University, Chicago has been looking at the possibility of this ternary heterogeneity in traffic. Specifically they are developing a computer simulation interface for American urban roads configurations. The vehicles movement rules for ternary mix traffic could be used like Deo and Ruskin (2006) or Feng et al, (2007) rules. The interpretation of the rules of Deo Ruskin rule is that: (i) the vehicle will move forward if there is vacant cell in front of it along the direction of movement (ii) vehicle will not move or stay in the same cell if the cell in front is occupied. For Feng et al. rules the interested reader please refer to their papers.

8.4.2 Bus stop impact on mixed traffic flow

Future investigation of the influence of road blockage (e.g., by bus stops in a cross road geometry) under heterogeneous traffic condition. These stops are a most common feature between urban control configurations. A stopped bus creates a temporary bottleneck, reducing road capacity, during its stationary period. One obvious question is whether bus lanes actually work.

8.4.3 Linked intersections

Another interesting topic for further research is of linked intersection. The focus is to mainly been on two neighbouring intersections and how knock-on effect occurs for heterogeneous and homogeneous condition.

For example, for X and Y two intersections as shown in Figure 8.2, where X is controlled by traffic light and Y is stop controlled flow traffic. The capacity of an exit road of intersection X (e.g. road-3), can be complex due to vehicles and groups of vehicles, or platoons, travelling at various rates, as well as by formation and dispersion of queues at the entry points to the intersection Y.

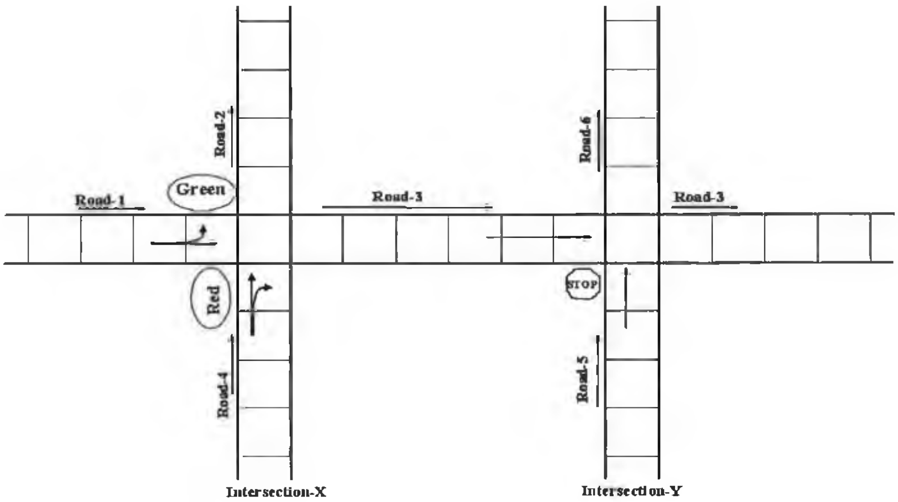


Figure 8. 2- Schematic of traffic flow at linked intersection

For example, a vehicle traveling from intersection X to intersection Y, within road-3. If vehicles from intersection-X flows into road-3, and if the flow on intersection is greater than the flow on intersection Y, then road-3's queue will grow. It also must be remembered that X is interfaced with two exit roads (e.g. road-2 and road-3) and its actual flow into Y depends on the probability that vehicles will turn in road-3's direction. That is, the probability that a vehicle on X will either turn left, or goes straight. Also, if road-3's queue reaches capacity it will affect X because of backup.

Hence, we wish to investigate the effect when the percentage of ST vehicle in X intersection increases suddenly, giving a cohort arriving at intersection Y. The theory here is that such linkages could then be indexed to feed into a longer network.

Again the simplifying assumption is that exit lanes (e.g. road-2 and road-6) have infinite capacity, with no congestion, and hence passing out of the linked configuration being investigated. In network terms, this means that roads at the boundary of the network, which serve as "exit routes" would be seen as free-flow, offering no barrier to vehicles departing.

8.4.4 Long-term research plans

In this research, heterogeneous traffic flow has been studied for the Western developed world. Our long term goal is to expand the model into every types of vehicle e.g. bicycle, motor cycles etc. Obliviously for developing countries, other non-motorised vehicles may also apply. A key difference here is multiple occupation of a cell. For this purpose we might use *grid base* traffic flow, which allows the movement of small size vehicles side by side. In this model a stretch of would be road divided by a grid of cells. Each vehicle could represented by one or more cell and the number of cells would be governed by the size of the vehicle. Clearly, this model is computationally more complex but would be closer to preliminary heterogeneous urban traffic flow models currently being investigated for Indian roadways (Gundaliya, and et al, 2005).

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Appendices

A Glossary of terms

CA- Cellular automata technique

DCC- Dublin City Council

FHWA- Federal Highway Administration

Flared approach- A flared approach to a configuration is one that allows vehicles performing different manoeuvres to start/ complete these, while other vehicles occupy the single-lane road.

HGVs- Heavy goods vehicles

ITS- Intelligent transportation systems

LGVs- Cars and light good vehicles combined

LT- Left turning

LV- Long vehicle (buses and equivalent length vehicles)

LWR- Lighhill-Whitham-Richards traffic flow model

MAP- Minimum acceptable space

MMAS- Multi-stream Minimum Acceptable Space

NRA- National road authority

OPAC- Optimisation Policies for Adaptive Control

RSA- Road Safety Authority

RT-Right turning

SCATS- Sydney Coordinated Adaptive Traffic System. It is an online, adaptive traffic system; it can identify and compensate for most types of traffic conditions and apply appropriate strategies. Traffic control room staff monitors the SCATS system, and can intervene to help clear congestion due to accidents, protests and other disruptions.

SCOOT- Split Cycle Offset Optimisation Technique. It is a tool for managing and controlling traffic signals in urban areas. It is an adaptive system that responds automatically to fluctuations in traffic flow through the use of on-street detectors embedded in the road.

ST- Straight through

SV- Short vehicle (cars and equivalent length vehicles)

TLCI-Traffic light controlled intersection

TLCR- Traffic light controlled roundabout

TWSCI- Two-way stop controlled intersection

TWTLCI-Two-way traffic light control intersection

UTOPIA- Urban Traffic Optimisation by Integrated Automation

YSCR- Yield sign controlled roundabout

B Chi-Square Goodness of Fit Test

When an analyst attempts to fit a statistical model to observed data, he or she may wonder how well the model actually reflects the data. How "close" are the observed values to those which would be expected under the fitted model? One statistical test that addresses this issue is the chi-square goodness of fit test.
In general, the chi-square test statistic is of the form

$$\chi^2 = \sum \frac{(\text{observed-expected})^2}{\text{expected}}$$

If the computed test statistic is large, then the observed and expected values are not close and the model is a poor fit to the data.

C List of publications from this work

1. Deo P. and Ruskin, H.J. (2007), *Performance evaluation of heterogeneous motorized traffic at urban single-lane junctions and roundabouts*, ERCIM News No. 68, January 2007, pp.28-29

2. Feng Y., Liu Y., Deo P. and Ruskin H.J. (2007), *Heterogeneous traffic flow model for a two lane roundabout and controlled intersection*, International journal of modern physics C (IJMPC) 18(1):107-118, January 2007

3. Deo, P. and Ruskin, H. J. (2006), *Heterogeneous traffic flow at a single lane T-Intersection: traffic responsive signalisation,* Econophysics Colloquium, Tokyo November Nov 23-25, 2006. (oral presentation)

4. Deo, P. and Ruskin, H. J., (2006), *Simulation of Heterogeneous Motorised Traffic at a Signalised Intersection,* Seventh International Conference on Cellular Automata for Resurch and Industry-ACRI, University of Perpignan, France Sept 20-23, LNCS 4173, pp.522-531 (oral presentation)

5. Deo, P. and Ruskin, H. J., (2006) *Comparison of Homogeneous and Heterogeneous Motorised Traffic at Signalised and Two-way Stop control Single lane Intersection,* International Conference on Computational Science and Its Applications - ICCSA, Glasgow, UK, May 8-11, LNCS 3980, pp.622-632 (oral presentation)

6. Deo, P. and Ruskin, H. J., (2005) *Space-requirement Implications for Heterogeneous Traffic Manoeuvres,* International Conference on Computational Science and Its Applications - ICCSA, Singapore, May 9-12,Proceedings PART V, pp 196, ISBN: 981-05-3498-1(poster presentation)

D Diskette and programs

The list of diskette folders are presented

All single-lane programs are written in C++ and all multi-lane programs are written in VB. Both can be compiled by Visual C++ 6.0.