Modelling Unsignalised Traffic Flow with Reference to Urban and Interurban Networks

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Abstract

A new variant of cellular automata (CA) models is proposed, based on Minimum Acceptable Space (MAP) rules, to study unsignalised traffic flow at two-way stop-controlled (TWSC) intersections and roundabouts in urban and interurban networks.

Categorisation of different driver behaviour is possible, based on different space requirements (MAPs), which allow a variety of conditions to be considered. Driver behaviour may be randomly categorised as *rational*, (when optimum conditions of entry are realised), *conservative, urgent* and *radical,* with specified probabilities at each time step.

The model can successfully simulate both heterogeneous and inconsistent driver behaviour and interactions at the different road features. The impact of driver behaviour on the overall performance of intersections and roundabouts can be quantified and conditions for gridlock determined.

Theorems on roundabout size and throughput are given. The relationship between these measures is clearly non-monotonic.

Whereas previous models consider these road features in terms of T-intersections, our approach is new in that each is a unified system. Hence, the relationship between arrival rates on entrance roads can be studied and critical arrival rates can be identified under varied traffic and geometric conditions. The potential for extending the model to entire urban and interurban networks is discussed.

Declaration

I hereby certify that this material, which I now submit for assessment on the programme of study leading to the award of Doctor of Philosophy is entirely my own work and has not been taken from the work of others save and to the extend that such work has been cited and acknowledged within the text of my work.

Signed:________________________________

Ruili Wang

Date:__________________________________

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Chapter 1

Introduction and Scope

1.1 Introduction

Mathematical modelling of traffic flow has a long history. The heterogeneous nature of human behaviour, the random interactions between drivers, the complicated geometric features of the roads, the highly non-linear group dynamics and the large dimensions of the system under investigation combine together to create considerable complexity. To date, modelling has not reached a satisfactory level, but we hope to offer further analysis and suggestions for future improvements.

Modelling traffic flow at unsignalised intersections and roundabouts has focused on two different approaches in recent years. Essar et al. (1997) Chopard et al.(1998) Wang and Ruskin (2001 and 2002), and Ruskin and Wang (2002 a and b) simulate unsignalised intersections and roundabouts using cellular automata (CA) models. Brilon and Wu (1999), Bonneson and Fitts (1999), Harwood et al. (1999), Tian et al. (1999), Troubeck and Kako (1999), Wu (1999), Chodur (2000), Hargring (2000), Tracz and Gondek (2000), Tian et al. (2000), Tanyel and Yayla (2003) have concentrated on gap-acceptance models.

A common deficiency of all models until fairly recently is the assumption that drivers are consistent and homogenous. In reality, driver behaviour is heterogeneous and inconsistent. Therefore, it is necessary to develop new models to overcome previous drawbacks and this has a principle focus in much of the work described.

In order to simulate heterogeneous and inconsistent driver behaviour and interactions at the different road features, new CA models are developed based on the **M**inimum **A**cceptable s**P**ace (MAP) method. Basically, the MAP method enables us to categorise the driver behaviour into four groups (rational, conservative, urgent and radical).

In each group, driver behaviour has its own special space criteria. If the criteria are met, the vehicle can then move onto intersections or roundabouts. Each driver is randomly assigned to one of four categories in each time step according to a distribution of driver behaviour. In this way, we can successfully introduce both heterogeneity and inconsistency for the drivers and their interactions at the different road features.

Previous models (e.g., gap-acceptance models referenced earlier) considered road features (e.g., roundabouts) in terms of T-intersections. Thus, the models could only be used to investigate individual entrance operational properties. Our approach is new in that each intersection or roundabout is a unified system. Hence, the relationship between arrival rates on entrance roads can be studied and critical arrival rates can be identified under varied traffic and geometric conditions.

Furthermore, our models can be applied to situations for which headway distributions are insufficient to describe traffic flow (Ruskin and Wang 2002a), and where the gap-acceptance model is not readily applicable, such as traffic flow in an urban area. Additionally, our models do not have any restriction on speed either, i.e., they can be applied to either high or low speed vehicles, and are thus applicable to both urban and interurban networks.

1.2 Scope of this Thesis

This thesis is organised as follows:

Chapter 2: Approaches to Traffic Modelling. In this chapter, we comprehensively review microscopic and macroscopic traffic flow models including car-following theory, CA models etc.

Chapter 3: Modelling Traffic Flow at Single-lane Two-way Stop-controlled (TWSC) Intersections. In this chapter, we propose a new model to study single-lane TWSC intersections. The Minimum Acceptable sPace method (MAP) is proposed for the first time. Four driver behaviour groups are defined. The processes of vehicle arrivals on entrance roads and the intersection interactions are modelled. We also introduce a

Stop Sign Delay Time (SSDT) to simulate the delay due to the pause at stop signs of TWSC intersections. The operational properties (such as throughput, entry capacity, etc) of single-lane TWSC intersections are also investigated. (*Throughput* is defined as the total number of vehicles, which navigate the intersection or roundabout in a given time and *capacity* as the number of vehicles that can enter an intersection or roundabout from an individual entry road).

Chapter 4: Modelling Traffic Flow at Multilane TWSC Intersections. Two-lane TWSC models are proposed with different lane-allocation patterns. The processes of vehicle lane allocation are simulated. The operational properties of different lane-allocation patterns are investigated and compared. In order to contrast intersections with and without traffic lights, intersections with traffic lights are also considered briefly.

Chapter 5: Modelling Traffic Flow at Single-lane Roundabouts. Single-lane roundabouts, as an alternative to single-lane TWSC intersections, have been investigated. In this context, a new CA ring model is developed, which can be applied to any roundabout topology, (such as different numbers of entrance roads). Theorems on roundabout size and throughput are given. The operational properties of single-lane roundabouts are also investigated.

Chapter 6: Modelling Traffic Flow at Multilane Roundabouts. A two-lane roundabout model is developed based on our MAP method. Position Delay Time (PDT) is introduced to simulate the delay due to a vehicle's relative position on the entry road (lane choice). The operational properties are also discussed and extension to three-lane roundabout model is also considered.

Chapter 7: Summary. In this chapter, we present a summary of the main findings and conclusions, followed by a comprehensive review on the contributions of the research and some suggestions for future developments derived from the work to date. In this context, the key question of modelling heterogeneous driver vehicle units is addressed and can readily be incorporated in the models described.

Chapter 2

Approaches to Traffic Flow Modelling

2.1 Review of Microscopic Modelling

Basically there are three types of approaches in modelling the traffic flow, microscopic, mecroscopic (between micro- and macro-) and macroscopic modelling. Microscopic modelling generally starts with and focuses on individual car movement. Most microscopic models are known as “Headway models” because the individual car movement relates to the headway between the two cars (Hammad 1998). Others may be called “Interacting models”, since for intersections or roundabouts, for example, individual car movements may be inter-dependent.

2.1.1 Car–following models

The classical car-following models were developed to model the motions of vehicles following each other on the single lane without any overtaking (Pipes 1953). Despite fifty years of history, however, the very many relationships involved are frequently deficient in description and often not completely understood (as discussed below). The car-following process remains an important one, however, which is considered in all microscopic simulation models as well as in modern traffic flow theory (Brackstone and McDonald 1999).

2.1.1.1 Car-following theory

The first and most basic microscopic models was that based on follow-the-leader theory (Herman et al. 1959, Gazis et al. 1961), (also called “car-following” theory). In this theory, individual motion is essentially a reaction to the behaviour of the vehicle (the leading vehicle) in front and car motion is also restrained by other conditions such as engine power, delay times and traffic rules. The model gives a stimulus-response function of the headway distance between the leading vehicle and the following vehicle, the speed of the following vehicle and their relative speed. The model can be written as

$$a\,(\,t + T) = \alpha \cdot v(t)^{m} \cdot \Delta v(t) / \,gap(t)^{l} \qquad (2.1)$$

where: $a = dv / dt$ is the acceleration , Δv is the speed difference to the vehicle ahead, $v(t)$ is the relative speed, $gap(t)$ is the gap between the leading vehicle and following vehicle, and the α, m and l are empirical constants.

In order to determined the combination of constant α, m and l, many similar experiments have been conducted over the past 40 years (Gazis et al. 1961, May and Keller 1967, Heys and Ashworth 1972, Hoefs 1972, Treiterer and Myers 1974, Ceder and May 1976, Aron 1988, Ozaki 1993). Unfortunately values observed spread over a wide range and appear to reflect in part specific conditions of set-up .

In particular, true car-following behaviour has several other important features, which have not been explored by car following theory (Chakroborty and Kikuchi 2000). These include:

- Car following behaviour is "human behaviour", a process characterised by "vagueness" rather than determinism.
- Response to stimuli in car following is asymmetric.

This contrasts the theory, which requires acceleration and deceleration to be symmetric. Leutzbach (1988) suggests that this is because "drivers pay closer attention to decrease in spacing (decrements) than to increase in spacing (increments) simply on the basis of their own safety."

The theory also assumes that the desired speed depends on the gap between vehicles. Accordingly only one platoon (or set of cars with no intermediate spaces) will exist if the time of consideration is long enough. This assumption is only correct when the speeds of the cars are less than the desired speeds of the drivers. The desired driving speeds vary greatly between the drivers, and depend not only on gaps between the vehicle in front, but also on personal preferences, motivations for the journey, weather, car performance and road conditions, etc. In real road situations, the existence of several platoons is normal. On a single-lane motorway, for example, there are always several leader cars that have the lowest preferred speed in this platoon.

2.1.1.2 Safety distance models

A further type of car following model is the safety distance model, where the purpose initially was to specify a safe following distance to avoid collision. The first such model was suggested by Kometani and Sasaki (1959), and given as:

$$\Delta x\ (t-T)= \alpha\, v^2_{n-1}(t-T) + \beta_l\, v^2_n(t) + \beta\, v_n(t) + b_0 \qquad (2.2)$$

where T, α, β_l, β and b_0 are empirical constants, Δx is the distance between the leading and following vehicles, t is time, and v_{n-1} and v_n are speeds of the following and leading vehicles respectively.

The authors described two experiments. The first allowed average speeds < 45km/h and the second allowed average speeds between 40km/h and 60 km/h, and two set of constant values were observed.

The approach was subsequently developed by considering human factors by Gipps (1981). His model included a basic assumption of common sense, which is that the drivers will use the maximum braking rates only when they think that they should and/or estimate that the other drivers will do so. This requires inclusion of further constants in the formula, namely b_n, which is the largest braking rate that the driver of the n^{th} vehicle wishes to use and b^* the predicted braking rate that the $(n-1)^{th}$ car in front will use, namely $b_{n-1}= b^*$. The deceleration (braking process) uses the formula

$$v_n\,(\,T+t\,) \leq b_n\, T\ +(\, b_n\, T- b_n\ \{2[\, x_{\,n-1}(t)-s_{\,n-1}-x_{\,n}\,(t)]-v_{\,n}(t)T-v_{\,n-1}{}^{2}(t)/\, b^*\,]\}^{\frac{1}{2}} \qquad (2.3)$$

and the process of acceleration is given by

$$v_n(T+t) \leq v_n(t) + 2.5\, a_n T\, [1 - v_n(t)/V_n] \cdot (0.0025 + v_n(t)/v_n)^{1/2} \qquad (2.4)$$

where $v_n(T+t)$ is the maximum speed for the n^{th} vehicle with respect to vehicle $n-1;T$ is the constant time interval; s_n V_n and a_n are the effective size (the physical length plus some margin); the desired speed and the maximum acceleration for the n^{th} car.

This approach has been widely used in simulation models such as INTRAS and CARSIM in USA (Benekohal and Treiterer 1989), PROMETHEUS in France (Broqua et al. 1991), DoTs SISTM model in UK (McDonald et al. 1994) and more recently by Kumamoto et al. (1995) in Japan.

2.1.1.3 Psychophysical spacing models

Michaels (1963) was the first author to discuss the underlying psychological factors, which would eventually dictate driver behaviour. The underlying concept of his model is that drivers would know the gap size and be able to perceive changes in relative speed due to changes in the apparent size of the vehicle in front. This perception of relative speed through changes in the visual angle subtended by the leading vehicle would induce the drive to make the decision to decelerate or accelerate.

The threshold for perception of speed changes was given by Michaels, who stated that only when the threshold is exceeded, will drivers choose to take action. The need for action depends on driver perception, so that inability to perceive any changes in relative speed implies that these are no longer above the threshold. The gap threshold is more important for small headway distance where speed differences are normally below the threshold. The threshold for this “just noticeable” distance is given by 10% - 12 % changes in visual angle.

A series of experiments were conducted by Evans and Rothery (1973) to define the thresholds suggested by Michaels. The experiments were set up by asking the passengers in a test vehicle to judge the gap between the vehicle in front and the test vehicles that they are in with a set time given to make assessment. The experimental conclusions were that the chance of a correct judgement is a function of $v/\Delta x$ and the observation time. The results also indicated that the thresholds are subject to a negative response bias which increase the Δx. In other words, the passengers believe they are close to the vehicle in front when this is not really true (Eveans and Rothery 1977).

Wiedemann (1974) was the first to combine all these thresholds together. This model integrates the three main thresholds, i.e. relative speed and distance perceptions as follows:

- A relative speed threshold for perception of closing, ~ $-3.1 \bullet 10^{-4} \Delta X$, ΔX is relative distance.
- For small relative speeds the perception thresholds for closing and opining are $-5.2 \bullet 10^{-4} \Delta X$ and $6.9 \bullet 10^{-4} \Delta X$ respectively.
- Thresholds for perceiving increases and decreases are $2.5+2.5v^{1/2}$ and $2.5+3.8v^{1/2}$ respectively, v is the relative speed.

More recently experiments were conducted by Reiter (1994), who used an instrumented vehicle to measure the action points and amended the second threshold above to $0.05+41.5 \bullet 10^{-4} \Delta X$ and $-0.15 +8.5 \bullet 10^{-4} \Delta X$ respectively.

The arguments regarding this model are intensive and many, in recent years, stem from psychologists. Hancock (2000) argued against the fundamental basis of the model, which is a perceptual signal to trigger avoidance behaviour. Basically, his argument comes from scepticism of whether psychological response is a deviation from reality. He also doubts the way in which the thresholds are calibrated, which is normally done in static, non-reactive, laboratory conditions.

There are probably at least two important factors that have not been included in this model. Firstly, cognisance that the thresholds are different for different individual drivers. Secondly, the possible bias which may be caused by environmental or other factors. Brackstone and McDonald (1999) indicated that not enough specific research work has been done on these concepts in order to compile a coherent model of driver behaviour. In consequence, they claim that model validation is hard to accept or to reject.

Nevertheless Wiedemann's ideas have recently been incorporated in PARAMICS-CM model in the UK by Cameron (1995).

2.1.1.4 Fuzzy logic-based models

The latest distinct development in car-following models is to use fuzzy logic. The original use of this method was published, Kikuchi and Chakroborty (1992) and subsequent developments in 1999 and 2002 (Chakroborty and Kikuchi).

Basically this approach tries to incorporate "fuzzy" rules to reflect the stimuli conditions of classical car following theory, namely relative speed, distance headway and acceleration / deceleration of the leading vehicle. A set of fuzzy inference rules has the following form:

If (at time *t*)

- Distance Headway (DS) is A_i AND
- Relative Speed (RS) is B_j AND
- Acceleration of Leading Vehicle (ALV) is C_k

Then (at time $t+1$)

Acceleration / Deceleration of following vehicle should be D_l.

The above rule consists of three fuzzy propositions consisting of fuzzy sets A_i, B_j and C_k. They refer to certain linguistically described conditions in a fuzzy set of concepts ADEQUATE, LARGE POSITIVE, NONE, VERY MILD, etc. Consequently in "**Then**", D_l is also a fuzzy set for concept NONE, MILD etc. The fuzzy sets A_i B_j C_k and D_l are represented by using the triangular or trapezoidal shape membership functions (Kikuchi and Chakroborty 1992). Since RS, DS and ALV are grouped into six, six and eleven linguistic classes respectively, the entire rule base has 396 (6 x 6 x 11) rules.

Researchers who support fuzzy models believe that they help to combine the psychological and physical perspective (Brackstone and McDonald 1999), but this viewpoint is not universally shared (Hancock 2000). Anyway this method has been used to formulate the MIcroscopic model for analysis of TRAffic jaM (MITRAM) modelled by Henn (1995) and investigated through road tests by Brackstone et al. (1997).

All car-following models have a common weak point, which is that they try to describe a pair of vehicles only. In reality, a driver's action comes not only from observing the leading vehicle, but also watching out for at least several cars in front. A real world driver will use a braking rate based on the premonitory comprehensive information of several cars in front, rather than on information on only one vehicle in front. A corollary to this is that a driver would be more cautious and allow more space if driving behind a huge vehicle (i.e. typically unable to get any information about other vehicles in front).

Very recent work on car-following theory, Boer (2000), has specifically noted three issues that contribute to behavioural variance of drivers.

- Car following is only one of many tasks that drivers perform simultaneously
- Drivers are satisfied with a range of conditions that extend beyond the boundaries imposed by perceptual and control limitation (i.e. tolerance is board)
- In each driving task, drivers use a set of highly informative perceptual variables to guide decision-making and control

Thus, car-following theory has been intensively studied in the past half century, where current focus is on attempts to understand the interaction between phenomena at the individual driver level and global behaviour on a more macroscopic scale (Krauss 1997, Brackstone and McDonald 1999).

One reason for this refocus is that car-following models is that may also be helpful for developing cruise control for automated highway system (AHS) and other automatic traffic control systems. (Chakroborty and Kikuchi 1999).

2.1.2 Queueing theory

Queueing theory can be catalogued in terms of microscopic models although the target of the theory is not 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. The first use of queueing theory for unsignalised intersection modelling is due to Tanner (1962). Queueing theory has since been intensively used to study traffic behaviour at intersections with and without traffic lights by Heidemann (1991, 1994 and 1997).

A queue occurs when instantaneous demand exceeds instantaneous capacity of the road. The queue length depends on the inter-arrival times and service processes. The service processes here mean all stages of the vehicle arriving at the end of the queue and crossing the intersections, (hence leaving the queue). Queueing models are characterised by the distribution of inter-arrival times and the distribution of the service times. Two distributions are normally used, Poisson or general distribution.

Using a standard notation for classifying queueing systems proposed by D. G. Kendal, *M/G/1* for example, the first symbol is the distribution of inter-arrival times, the second is the distribution of the service times and the last one indicates the number of servers in the system. It is equal to number of lanes of a road. A single lane is therefore equivalent to one server. *M* refers to Poisson distribution and *G* means a general distribution (Vandaele 2000). The speed-flow-density relations will closely be different depending on the queueing model and distributions.

More recent work, based on queueing theory, is the studying of these relations through speed-flow-density (SFD) diagrams of motorway traffic flow (Vandaele 2000). The basic traffic flow-density-speed equation is written

$$q = Es \tag{2.5}$$

where q and E mean traffic flow and density and s is speed. And effective speed

$$S = V_n (D_{max} - D)/D_{max} \tag{2.6}$$

where V_n is the normal speed, D_{max} is maximum density, and D is the density. Vandaele (2000) obtained

$$q_{max} = V_n \cdot D_{max}/4 \tag{2.7}$$

for M/M/1 model . Similarly for M/G/1 model

$$q_{max} = 2V_n \cdot D_{max}\{ [(\beta^2+1)^{1/2} - 2^{1/2}]/(\beta-1)\}^2 \ , \quad \beta \geq 0 \tag{2.8}$$

where traffic flow q is a function of the variation parameter β and β is the coefficient of service time variation. The formula for *G/G/1* model is similarly derived. Several applications of these expressions have been attempted e.g. in highway E19 from St-Job to Merksem (Antwerp) in Belgium (Vandaele 2000).

As the shape of the SFD is determined by the model parameters, the real world situation can be simulated by adjusting these suitably (Vandael 2000).

2.1.3 Cellular automata (CA)

Modern science is challenged by the need to understand complexity and its origins in problems such as traffic flow. When scientists analyse such systems, one traditional way is to break them down into simple constituent parts (Wolfram 1986). In traffic flow models, each part of the problem such as car size, car speed, driver's age and personality can, in theory, be analysed separately. While some interesting results on individual aspects of the problem, may be obtained, the overall way in which those parts act and react together may still not be known, since traditional models can not cope with many degree of freedom.

Cellular automata simulation methods have thus become increasingly popular in modelling complex behaviour, such as traffic flow, since exact mathematical formulae is not available for these problems.

Cellular automata are dynamical systems—defined by Toffoli and Margolus (1987), with space, time and system states are discrete. A cellular automaton of traffic flow can be divided into uniform sites on a finite uniform lattice defining the road that vehicles drive on. The variables describing each phase of each site are updated for each time step. The variables may be the speeds of the vehicles, indications of whether the cells of the lattice are occupied or empty or any other parameters, which describe an aspect of traffic flow. The state of a cellular automaton depends on the value of discrete variable(s) at each site. Each site may have a finite number of discrete variables, but only one value of one variable in any single discrete time step.

2.1.3.1 One-dimensional deterministic cellular automata (1DDCA)

The simplest models are one-dimensional deterministic cellular automata (1DDCA). The basic idea of 1DDCA is to equally divide a road into adjacent cells along which the vehicles will move. The cells can be either vacant or occupied. One vehicle only can occupy one cell at a given time. Simple update rules may be defined, e.g. (Yukawa et al. 1994 and Chopard et al. 1995 and 1998) to model 1-D traffic flow.

The update rules are as the following:

- Query whether the cell in front is vacant,
- If yes, the vehicle can move forward one cell in this time step. Otherwise the vehicle does not move in this time step.

The maximum speed of a vehicle is 1, as it can advance only one cell in a single time step and cars only have two possible speeds, 0 and 1. As the update rule is similar to the rule-184 elemental CA (according to the Wolfram's (1986) labelling scheme), this kind of CA model is also called CA-184.

2.1.3.2 The asymmetric stochastic exclusion process

Given the issue of asymmetry, method with regard to Car-following theory (Section 2.1.1.1), it is worth noting 1DDCA the variation, which allows for asymmetric stochastic exclusion update rules are:

- Randomly pick a cell ,
- If the cell in front of the picked-up cell is vacant, the vehicle moves one cell.

This approach has been extensively used by Derrida et al. (1992 and 1993) and developed further by Nagatani (1993). The latter proposed that the speed should depend on the gap between the leading and following vehicle. The one-dimensional asymmetric exclusion model has been used to simulate highway traffic jams and also been extended for a two-dimensional traffic flow model (Nagatani 1994a).

2.1.3.3 Stochastic Traffic CA (STCA)

A further CA model, the Nagel-Schreckenberg Model (NSM) (Nagel and Schreckenberg 1992) has the principal feature that the speeds of vehicles have numerical expressions, based on the following two assumptions:

- Each time step is 1second, which links the time step of the model to real time.
- The length of each cell is 7.5 m, which represents the real road in terms of the number of unit cells.

According to Nagel's (1996) paper, the model can be described for cars can with integer velocity between 0 and v_{max}, where $v_{max} \leq 5$. Based on the above two assumptions, one unit of velocity = 7.5m/sec, (which = 27km/h). Thus v_{max} is 135km/h. For each vehicle, the following steps are carried out in *parallel*:

Find number of empty sites ahead *(= gap*) at time *t*,

- If $v > gap$ (too fast), then slow down $v = gap$. (NSM-rule-I),
- Else if $v < gap$ (enough head way) and $v < v_{max}$, then accelerate by one.$v=v+1$. (NSM- rule-II],

- Randomisation: if after the above steps, $v > 0$ and $v \leq v_{max}$, with probability p, reduce v by one [NSM-rule-III], and allow each vehicle to move sites ahead. The gap is the number of empty sites, the headway is equal to v/gap.

Nagel (1996) indicated that the randomisation (NSM rule-III) 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 retard (noisy acceleration).

Some improvements have been added by Ricker et al. (1996), Esser and Schreckenberg (1997) and Wagner et al. (1997). Ricker et al. indicated that the maximum speeds differ between cars and used $v_{d(i)}$ instead of v_{max} to allow for different desired velocities in a fleet of cars $i = 1, 2, ...$ Richer modified NSM-rule-II and NSM-rule-III by using $v_{d(i)}$ instead of v_{max} . We refer to these as NSM-rule-IIa and NSM-rule-IIIa respectively. This has also been used in two-lane traffic simulation. Different vehicle types are also considered by allowing a long vehicle to occupy more than one cell for urban traffic simulation (Esser and Schreckenberg 1997).

Wagner et al. (1997) further modified the NSM by suggesting a breaking probability P_{break} instead of NSM-rule-III, (refer here as NMS-rule-IIIb). He considered

- $v_{n+1} = Max(0, v-1)$ with probability P_{break}
- $v_{n+1} = v$ with probability of $1-P_{break}$

This STCA model has described qualitatively some known facts about traffic flow, e.g. the spontaneous occurrence of congestion, the relation between traffic flow and traffic density and the back travelling stop-and-go wave, (which propagates in the opposite direction to traffic flow (Wagner et al. 1997).

There are, however, several points on the assumptions and rules that benefit from reconsideration.

Firstly, rule 1 ($v = gap$) means that the headway of the following car is only 1 second, because headway is equal to v/gap. This assumption means the following car does not observe the 2-second rule, where safe headway is observed to be 2 seconds on average. It means that a further $gap = 2v$ would be more suitable.

Secondly, NSM-rule-III and NSM-rule-IIIa are questionable to some extent. To model a realistic system random speeds are necessary in highway traffic. However, random decrease of 1 unit in speed of a given car may be inappropriate because 1 unit in speed means 27km/h. It is not realistic to decrease speed by 27km/h in one second without leading to collision. A driver will normally decrease speed on a freeway only when the gap between him and car in front is smaller than the safe gap. And decrease will usually be gradual rather than dramatic for safe driving.

This model has also been used in urban networks (Esser and Schreckenberg 1997, Emmerich 1998). This aspect is considered further in Section 2.4 (Review on the urban networks). In summary, the STCA is a multi-speed model, but may be less necessary in modelling in urban context (Chopard et al. 1998).

2.2 Review of Macro- and Mecro- scopic Models

Macroscopic models, based on fluid dynamic equations, were originally proposed by Lighthill and Whitham (1955). Since then, dynamic macroscopic traffic flow modelling has become a central focus for both theoretical and application-oriented research. Second-order models were developed by Payne (1971) and others and overcame some deficiencies of first-order models in terms of improving accuracy.

The approach of most of the macroscopic mathematical model structures suggested so far are derived from microscopic considerations within a string of identical vehicles. This approach has been criticised and questioned. Papageorgiou (1998) argued against this approach. He indicated that in traffic modelling the number of individual particles, which are vehicles here, does not exceed a few hundreds per km. By contrast, when we proceed from microscopic to macroscopic equations in thermodynamics, this number is 10^{23} particles per cm^3.

When the number of cars is large enough, traffic flowing on a highway can be modelled in the term of a one-dimensional compressible gas (Nagatani 1995). Such a hydrodynamic approach predicts the appearance of traffic situations, shock waves and traffic jams. However the hydrodynamic approach does not naturally describe the behaviour of traffic flow in the low-density limit where there are large heterogeneities in the traffic density (Ben-Naim et al. 1994, Nagitani 1995).

Klar and Wegener (1999) proposed mecro-models, which have a frameworks close to kinetic theory of gases and Boltzmann-like models. These models may be classified as an intermediate step between microscopic and macroscopic models. They can be derived from microscopic considerations, while simultaneously, fluid dynamic models can be related to traffic kinetic.

Basically the method derives a Boltzman type evaluation equation for the statistical distribution function on the position and velocity of a vehicle along the road. However, the main controversy about this method is that gas is three-dimensional and symmetric, whereas traffic flow is only one-dimensional.

Recently, a multiple Bolzmann equation approach has been further developed by Hoogendoorn and Bovy (2000) and Helbing (2001), using the second-order movement ($\mathbf{v}^2$) method originally due to Payne (1971). However, this method is being challenged by Cho and Lo (2002). They argue that $\mathbf{v}^2$ does not have any physical sense in traffic flow and the velocity variance equation that is obtained by multiplying the Boltzmann equation by $\mathbf{v}^2$ is also a meaningless term in traffic flow. Cho and Lo (2002) suggest the use of $\|\mathbf{v}-\mathbf{u}_e\|^2$ (individual velocity variance) to modify the second order Boltzmann equation.

2.3 Review on Multilane Traffic Flow Modelling

Single-lane models, e.g. car-following models, fluid-dynamical models (Prigogine and Herman 1971), and single-lane CA models (Nagel 1996), can not represent realistic traffic flow features for one main reason. The situation of a single lane freeway (single-lane on a freeway) only seldom applies, and if it does, is probably just for a short part of a road. A passing lane is commonly available, so that other vehicles, which have been delayed by the leader car, can pass. In the presence of a passing lane the whole configuration of traffic flow changes to that of multilane flow.

The design of lane-changing rules is one of the main tasks for multilane traffic modelling. A common approach to date for building a multilane model is to try to modify single-lane models and upgrade with lane-changing rules. However lane-changing involves not only vehicle movement, but also the whole process of driver decision making. The effectiveness of lane-changing rules determines how well the model describes the real world.

2.3.1 Freeway multilane traffic flow

There are many differences between multilane traffic flow for freeways and urban networks. The difference in motivation for changing lanes is just one of them. For a freeway, the main motivation for lane change is reaching a desired speed, which may be by acceleration or by deceleration. Speed is thus the major concern. On urban streets, the motivation for lane-changing is not only to maintain the speed or to avoid being obstructed by e.g. bus or delivery vehicles, but also to access the proper lane, which will enable turning in the direction desired. In fact to access desired direction and to avoid obstructions are the main motivations for lane-changing on city streets.

Most work on multilane models in the literature deals with freeway traffic (Gipps 1986; Biham et al. 1992, Nagatani 1994b, Ricker et al. 1996; Wagner et al. 1997, Klar and Wegener 1999) rather than with urban networks. Two-lane CA traffic simulation models used to simulate freeway situations are due to Nagitani(1993), Ricker et al.(1996) and Wagner et al. (1997).

The two-lane model of Ricker et al. (1976) was built from two parallel single-lane models, which is based on the Stochastic Traffic CA (STCA) (detailed in Section 2.1.3.3). The three rules of Nagel-Schreckenberg Model (Nagel 1996) have been modified by Ricker et al. With four additional rules defining the exchange of vehicle between the lanes, the model contains 7 rules in total, given below.

In the following rules, the index i is the number (or label) of the vehicle, $x(i)$ is the position, $v(i)$ is its current velocity, $v_d(i)$ is the its maximum speed, $pred(i)$ is the number of the preceding vehicle, $gap(i)= x(pred(i))-x(i)-1$, the width of the gap to the predecessor. Using $gap(i)$ for the number of empty sites ahead in the same lane and $gap_o(i)$ to nearest the empty site ahead on the other lane, and $gap_{o,back}(i)$ for backward gap on the other lane. L, L_o and $L_{o,back}(i)$are the parameters which define the distances immediately ahead, ahead and behind on other lane respectively. A vehicle i changes to the other lane if the last four conditions below are met. All vehicles update simultaneously.

- if $v(i) \neq v_d(i)$, then $v(i) = v(i) + 1$ (2.9)
- if $v(i) > gap(i)$,then $v(i) = gap(i)$ (2.10)
- if $v(i) > 0$ and random number $< p_d(i)$, then $v(i) = v(i) - 1$ (2.11)
- $gap(i) < L$ (2.12)
- $gap_o(i) > L_o$ (2.13)
- $gap_{o,back}(i) > L_{o,back}(i)$ (2.14)
- generated random number $< p_{change.}$ (2.15)

This model has several defects, which are caused by the given nature of the rules. One is that the model considers different desired velocity, but does not consider that the driver who prefers a lower velocity is likely to stay in the slow lane rather than in the fast lane. This defect leads to an over estimate of the number of vehicles changing lanes.

Wagner et al. (1997) also simulated two-lane traffic by using a CA approach. His model is based on NSM, but is modified by the introduction of NSM-rule-IIIb (Section 2.1.3.3). The aim of this model is to reproduce the “density inversion” observed two-lane traffic. More restrictive rules are therefore used but velocity differences have not been considered.

2.3.2 Urban multilane traffic flow

In Hammad's two-lane model (Hammad 1998), the urgent, minimal and maximal conditions for lane-change are used to suit different situations requiring movement with three probability parameters governing (lane-changing probability, obstruction probability and lane obstruction probability) and four case rules. The model attained some insight in the relation between lane-usage and density. Inparticully, the model successfully simulates and has been validated for the macroscopic phenomenon called "lane-usage inversion" or "density inversion", which occurs long before maximum flow. The model has been shown to be robust to adjustment of the three lane-changing probability parameters.

Nevertheless, some unrealistic situations still exist, which can be further explored. Firstly, an obstruction, such as that due to delivery and bus stops, is normally in the left lane only, with some exception of breakdown. It is somewhat unrealistic, therefore, to allow equal probability of obstruction probability in both lanes, unless the street is one-way. We note this point, since our focus on urban features and driver behaviour will draw upon a distribution of event/decision probabilities for various modelling aspects.

Secondly, according to Hammad's rules, the vehicles may move to the other lane for turning, change back for speeding, and change lane again for turning, (also known as "Ping-Pong" changes). This oscillation process may continue without as long as the criteria are met. In reality, if a vehicle is turning at the next intersection on the urban road, particularly already in the proper lane, the possibility of lane-changing for gaining speed or avoiding a slow platoon is very low.

Thirdly, lane-change is intrinsically a stochastic process, so that even when all conditions have been met, where some drivers still do not change lane. Consequently the basis for driver decision as it related to the probability of changing lanes is not considered in the Hammad model.

2.3.3 Issues in multilane traffic flow modelling

A very common unrealistic feature of two-lane models is oscillation. There are two types of oscillation indicated by Ricker et al. (1996). The first one occurs if all vehicles start in one lane with a higher density in consequence, so that every driver decides to change lanes. Thus, all vehicles attempt to change to another lane. As a result, they will all change back again. This collective lane-change effect has been observed by Nagatani (1993 and 1994b). One way to overcome the problem is to change the symmetric model into an asymmetric one by randomising the lane-changing decision by using of a probability $P_{change,}$ (starting from random initial conditions), which has the effect of diluting this oscillation effect.

Another type of oscillation occurs when the vehicle changes to and fro or between lanes several times due to the following vehicle meeting the criterion of lane-change. In Hammad's model (1998) the following vehicle may change lane for speed and change back for turning. Similarly, in the model of Ricker et al. (1996), the following vehicle may change lane to increase speed, then may change back after the leading vehicle increases its speed to provide sufficient following space. This effect is caused by allowing random speeds, but is likely to be limited importance for urban networks, for reasons of speed-range and turning positioning (as noted previously).

Another common issue relates to the criterion of looking backward on nearest other lanes. Both of Ricker et al. and Hammad's models consider a look-backwards-rule in terms of to guarantee the following car in the other lane would not be blocked in the *next* time step. However, a block may occur for following cars in the other lane in the subsequent time step. This can happen if the speed difference is very large i.e. if the lane-changing vehicle's speed is slow and the following vehicle is travelling at high speed. The following vehicle has to stop or decrease the speed in the second time step to avoid to closing on the lane-changing vehicle. In the real world, the driver normally considers not only space but also speed difference and car accelerating capability before changing lane.

In a recent paper on multilane work, due to Klar and Wegener (1999), a microscopic multilane model, based on reaction thresholds, is developed. There are seven thresholds to be considered for a vehicle to change the lane and speed. Once the distances of a vehicle between the leading or following vehicle become larger or smaller than any threshold distance, the vehicle will change speed or change lane instantaneously. The threshold depends on the speed and reaction time. The reaction times are based on empirical experiments, e.g. Klar et al. (1996).

The thresholds of the vehicle in the given lane are:

- $H_L(v)=H_O+ vT_L$ (2.16)
- $H_R(v)=H_O+ vT_R$ (2.17)
- $H_B(v)=H_O+ vT_B$ (2.18)
- $H_A(v)=H_O+ \delta + vT_A$ (2.19)
- $H_F=H_O+ \delta + wT_F$ (2.20)

and the thresholds on the left lane and right neighbouring lane are

- $H_L^S=H_O+ vT_L^S$ (2.21)
- $H_R^S=H_O+ \delta + vT_R^S$ (2.22)

where H_L , H_R , H_B , H_A , H_F , H_L^S and H_R^S are thresholds for lane-changing to left, to right , breaking, accelerating and free driving on the given lane, and the thresholds on the left lane and right lane respectively; T_L , T_R ,T_B ,T_A and T_F are constants of the reaction times , which are determined by experiment; H_O *is* the minimal distance between the vehicles. δ is a constant related to acceleration delay.

These lane-changing rules are more sophisticated than those for other models, since speed of the vehicle that will change lane is also being considered. One basic assumption of the model is that the left lane is faster than right lane (in Germany). This is true only when there is no congestion on any lane. No consideration has been given to speed of cars in the neighbouring lanes.

2.4 Review of Urban Networks

The above methods are for general traffic modelling. Our research in this thesis will focus however on urban networks and more specifically on road features, which are intrinsic to urban and inter-urban systems, which closely reflect daily experience and which congestion is a daily hazard.

2.4.1 The context of urban networks

Traffic modelling on freeways as opposed to urban networks requires a different context. Firstly, traffic flow dynamics are 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 car manoeuvring and queuing, traffic lights, driver behaviour and the operation of business. Turns are inevitable in driving on urban streets. In contrast, turning on freeway often follow the geometrical shape of the freeway and this tuning does not change the components of traffic flow. In a freeway model, speeds may be considered up to 165km/ h, which clearly does not apply to urban areas.

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

Both in freeways or urban networks the group, or “collective behaviour”, is normally targeted. However, because of the difference in dimension of the systems and difference in the targeted traffic flow phenomena to be reproduced, different levels of compromises must be applied (Esser and Schreckenberg 1997).

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 street network. Hydrodynamic models are hard to apply in urban networks because of the many differently directed currents of traffic involved and intersections and traffic lights (roundabout as well) are difficult to translate into hydrodynamic language (Lehmann 1996).

Car-following theory may only be used separately on each road of an urban network, it does not help much for networks as whole, as the dynamic relies on traffic lights intersections rather than the gap between vehicles.

2.4.2 Urban networks details

The aim of urban networks modelling is to explore congestion on urban roads. Since the intersections are the "bottlenecks" of the whole network, the modelling of urban networks has focused on the intersections. Various types of intersections with traffic lights 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 and traffic lights have been investigated by topological methods (Cremer and Landenfeld 1998). The paths of traffic in urban networks is one of the basic problems to be met in modelling and has been addressed in by Nagel (1998), van Laak and Toorner (1998) etc. These papers on urban networks represent the state of art on this topic to some extent and put forward some interesting ideas to be investigated further.

2.4.2.1 Intersections with and without traffic lights

Chopard et al. (1998) first suggested the use of "a rotary" to simulate a junction. The rotary can be thought of as a CA ring allowing several one-dimensional CA to be interconnected. This is a generic way to represent intersection without traffic lights. The intersection can have any number of branches and numerical implementation is relatively simple. The rotary acts as a connection, which connects all branches to form a system, and a separation, which also separates those branches into individual sub-systems. A common rule is that the car in the rotary has priority over any entering car. The simple CA update rules have been used, such as CA-184 (Section 2.1.3.1).

Manhattan-like grid street networks have also been studied with and without traffic lights (Chopard et al. 1998). The situation without traffic lights corresponds to equally likely behaviour at each rotary junction. The interesting result is that queues are more likely to form and the global mobility is less in the situation with traffic lights.

The model of Chopard et al. has had some success in exploring some features of intersection without traffic lights. The problem is that all cars from different branches have equal priority to move in the rotary. In reality, however, on a junction without traffic lights, traffic flow is governed by yield rules (or priority regulations or "give-way" rules). The vehicles from the major roads have priority over cars from minor roads. The Chopard method may be applied to all-way stop controlled (AWSC) interactions, but it cannot be applied to two-way stop-controlled (TWSC) intersections.

Esser and Schreckenberg (1997) have also tackled the intersection *without* traffic lights. The method used is to set a flag (a variable) to control leaving the minor road. The switching on of the leaving flag (i.e. change of the state of the variable) depends on the number of vacant cells at the intersections of the major road. When the leaving flag is on, vehicles on the minor-street can go. This method has also been used in intersections *with* traffic lights. There is one flag on each road of intersection. The switching on of the leaving flag corresponds to the green light and traffic light sequence is determined by a predefined switch matrix.

2.4.2.2 Model of traffic light strategy

In the paper of Cremer and Landenfeld (1998), a mesoscopic model for saturated urban road networks has been developed. Basically, the model neglects any dynamic details of the vehicles and any other elements that are not important for flow control in over saturated networks, i.e. those where traffic is stopped or jammed. The model describes the dynamic state of individual vehicles in a simplified manner. They have only two speeds; zero if the vehicle in a waiting queue, the other equal to 50km/h if moving. The details of acceleration, deceleration and lane-change have been omitted. When a green light shows, the vehicle in the queue will move directly to the next point downstream in one time step. The number of vehicles that move depends on the duration of the green light. The author also used a topology editor to synthesise a large variety of network topology.

The model simplifies the network as a whole and can be used to test, evaluate and compare control strategies for congested road networks. Traffic control strategy is seen as the only method to relieve congestion. Cremer and Landenfeld (1998) also indicated that the following performance criteria may be used to test, evaluate and compare signal strategies for congested networks, e.g. travel time, number vehicles in the network, utilisation of green time.

The underlying message of the Cremer and Landenfeld (1998) is similar to that of Chopard et al. (1998) in that the details of dynamics are seen to be often irrelevant at the level of the whole urban networks.

2.4.2.3 Paths of vehicles

A fundamental problem with urban network modelling is also how to determine the path of the vehicles in the networks. If a realistic traffic simulation is attempted, the knowledge of the time-dependent path of each vehicle is crucial (Chopard et al. 1998). Normally that information is both unknown and extremely difficult to collect.

When information on each turning operation is unknown, percentages are assigned to different directions. For example 50% straight, 20% left and 30% right (Nagel 1998). Modelling is, de facto, based on the fact that an individual vehicle does not have any predetermined destination, but randomly moves through networks. From the network level, the model is only concerned with collective behaviour is what the model concerned. For the Duisburg network, an attempt was made to obtain accurate information by using 51 checkpoints for turn counts, which could be updated every minute. The turning count could be thus be directly derived for 56 directions (Esser and Schreckenberg 1997). However it was still not found possible to obtain complete overall traffic information even at the current checkpoint positions, since the number of checking points on the network border was insufficient.

An alternative approach to path determination is to use origin-destination (OD) matrices, but this information is also not available for most cities. Trips for people going to work may not change form Monday to Friday and demographic data on working patterns may be available, but information on non-working trips is not available. A micro-simulation project on drive activities (i.e. sleeping, work, shopping) has been piloted by Beckman et al. (1996) and others. Nagel (1998) noted that it has so far little insight into what is “driving” the type of micro-simulation.

Even given a time-dependent OD matrix and a traffic network, the allocation of paths is still a problem, since assuming that all drivers are perfect rational decision-makers and have full information about current traffic states, there still be different criteria for them to decide which paths they take. The optimal routes are different based on different criteria, such as travel time, route length, traffic density, route simplicity (van Laak and Toorner 1998) and “preferences”.

Some work (Nagel 1998, Chopard et al. 1998) have chosen only one criterion of travel time. The basic idea is that for any route from A to B (any two locations in a network) time taken is the same. Otherwise, if a trip takes less time because it is less congested than another, some driver will find it, and balance the respective traffic loads (Chopard et al. 1998).

This criterion may be over-simple. Suppose one path from A to B needs T minutes and the length of this path is L kilometres, another one is $(T+\alpha)$ minutes but the length is $(L-\beta)$ kilometres, the path taken will depend on the trade-off time α and length β. That decision will almost certainly be different for different individuals.

2.5 Summary

In this chapter, we reviewed micro-, mecro- and macro- scopic traffic flow models. In particular, car-following models, multilane traffic flow models, CA models and traffic flow in urban networks are examined and provide fundamental building blocks for our research topic.

As car-following behaviour is a process characterised by "vagueness" rather than determinism, any rigid formula may fail to describe the nature of driver behaviour. Consequently fuzzy-logic based models and stochastic traffic CA models are becoming increasingly popular in modelling traffic flow.

In our review, we focus on car-following theory as it is considered in all microscopic simulation models as well as traffic theory. Clearly all models (e.g. PARAMICS-CM, MITRAM, INTRAS, CARSIM, etc. as in Section 2.1), which belong to car-following theory, only simulate the reaction of the following vehicle to the headway. Therefore, there is no direct link between car following theory and our research, which is to simulate driver behaviour and interaction between vehicles from two or more streams. However, the review on car-following theory is essential to understand state-of-art technique to simulate driver behaviour and interactions.

The design of lane-changing rules is a main task for multilane traffic modelling. Many criteria for lane-changing have been defined in terms of speeds and spaces. Stochastic processes are also introduced, but, as mentioned previously, some issues still remain unsolved.

As urban networks have a special context (Section 2.4), CA models have been intensively used to simulate traffic flow in urban networks. The advantages of using CA models are obvious, as both microscopic features and macroscopic properties can be investigated. Therefore, we will use CA models to simulate unsignalised traffic flow at urban and inter-urban networks.

Chapter 3

Single-lane TWSC Intersections

3.1 Introduction

Two types of unsignalised intersections have been the main focus in modelling uncontrolled flow. These are the two-way stop-controlled intersection (**TWSC**) and all-way stop-controlled intersection (AWSC). Because AWSC is mostly used in North America, the focus of this chapter is TWSC intersections, which are close to the UK and Ireland situation.

Traffic flow at a TWSC intersection has to observe both priority and stop rules. Priority rules are applied in the following ways:

- All entering vehicles give way to all vehicles on the intersection
- A right-turning (RT) vehicle from a major-stream gives way to the oncoming straight-through (ST) or a left-turning (LT) vehicle from another major-stream in Ireland and the UK (however, a LT vehicle gives way to the RT vehicles in New Zealand, for example, so there are national variants with in broader groups)
- A vehicle from a minor-street gives way to all vehicles on the major roads
- A RT vehicle from a minor-street gives way to the oncoming ST or a LT vehicle from another minor-street

Stop-rule ("stop sign" rule) is that a vehicle from a minor-street must *stop* before entering the intersection (even there is no vehicle on the major-street).

American engineers use a ranking system to describe the above rules, which is given by the Highway Capacity Manual (Transport Research Board, 2000).

The research on traffic flow at TWSC intersections has focused on performance measurements, such as capacity, queue-length and delay. The **entry capacity** (or **capacity)** of an intersection is the number of **v**ehicles passing through *an entrance* road

per unit of time (normally an **h**our—**vph**), which is *different from* throughput. **Throughput** is the number of vehicles, which navigate through the intersection in a given time.

Both empirical and analytical methods have been used. Kimber's capacity model (Kimber 1980) and the linear capacity model (Brilon et al. 1997) belong to the empirical method. The most common analytical method is that of the gap-acceptance model and most TWSC intersection capacity models are based on gap-acceptance (Tian et al. 1999).

Cellular automata (CA) models provide an efficient way to model traffic flow on highway and urban networks, (Nagel and Schreckenberg 1992, Chopard et al. 1998 and Wahle et al. 2001 and references, Section 2.1.3). The CA model is designed to describe stochastic interaction between individual vehicles, independently of headway distribution. It can then be applied to most features of traffic flow, whether or not these can be described by a theoretical distribution. Features modelled may include multi-streams on the major road, heterogeneous vehicles (passenger and heavy vehicles), and intersections with or without flaring.

3.2 Background

3.2.1 Gap-acceptance models

Gap acceptance models widely used in calculating capacity of a TWSC intersection. Basically, there are based on the notion that a driver will take the opportunity to move onto the intersection when the gap is larger a particular size (Troutbeck and Brilon 1997).

The basic assumption of gap-acceptance models is that the driver will enter the intersection when a safe opportunity or "gap" occurs in the traffic. 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 the 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 vehicle to pass through. The **follow-up time** is the time span between two departing vehicles, under the condition of continuous queueing. The values of *critical gap = 3-5.2* seconds, *follow-up time = 2-3* seconds, and *minimum headway = 1 or 2* seconds were recommended (Troutbeck 1984, Flannery and Datta 1997).

In order to use the gap acceptance model, the distribution of gap sizes has to be known first. Several distributions have been proposed, such as exponential, displaced exponential and dichotomised (Schuhl 1955) distributions. However, the M3 distribution model proposed by Cowan (1975) has been widely accepted (Troutbeck and Brilon 1997, Hagring 1998 and 2000, Tian et al. 1999)

Cowan's M3 distribution assumes that a proportion α, of all vehicles are free to interact, travel at headways greater than t_m and have displaced exponential headway distributions, while the remaining $1-\alpha$ bunched vehicles have the same headway of only $t_{m.}$

Gap-acceptance models are, however, unrealistic in general assuming that drivers are consistent and homogenous (Tanner 1962, Robin and Tian 1997). A *consistent driver* would be expected to behave in the same way in all similar situations, while in a *homogenous 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 the critical gap for *a particular driver* should be represented by a stochastic distribution initially suggested by Bottom and Ashworth (1978), but ignored until relative recently.

Estimation of the *critical gap* has attracted much attention, with use of a mean critical gap also proposed (Harwood et al. 1999, Tian et al. 2000, and Troutbeck and Brilon 2001). Maximum likelihood estimation of the mean critical gap has been widely accepted (Harwoood et al. 1999, Tian et al. 1999 and 2000, Troutbeck and Kako 1999, Tracz and Gondek 2000), but has not influenced the basic assumption, which is still that all drivers are consistent.

The critical gap is clearly a key parameter for various gap-acceptance capacity models and significantly affects the final results. However, the critical gap distribution and its parameters can not be directly observed in the field (Kyte et al. 1996). Only rejected and accepted gaps can be directly measured, and the critical gap estimated, based on the largest of these.

Determination of the critical gap distribution has been the focus of much effort. Over 30 methods have been published and all produce different results for the critical gap (Brilon et al. 1997). A comprehensive review and simulations has been made by Brilon et al. (1997). The maximum likelihood (Troutbeck 1992) and Hewitt's method (1983, 1985 and 1988) are recommended by the authors based on their criteria and simulations. The maximum likelihood method has also be recommend in the Highway Capacity Manual (1990) and also by Tian et al. (2000) and Hagring (2000) in very recent work.

While, the original maximum likelihood approach can be braced back to Miller and Pretty (1968). More explicit procedures are described by Troutbeck (1992). Basically, the maximum likelihood approach assumes that all drivers are consistent and calculated as given above, the mean critical gap has been found to be a reasonable quantity for the representation of average driver behaviour (Troutbeck 1992).

3.2.2 Critical review of gap-acceptance models

There are, however, several phenomena that gap acceptance fails to take into account, most notably inconsistency and heterogeneity of driver behaviour, priority sharing, give-way between two vehicles from the opposite major streams.

It seems clearly that in any real situation, c***ritical gap is not a constant value*** for different drivers or for each individual driver over time (Tanner 1962), since driver behaviour is both an intrinsic characteristic of individual experience, as well as a response to the current environment. A *consistent driver* is expected to behave in the same way in all similar situations, while in a *homogenous population*, all drivers have the same critical gap and are expected to behave similarly (Plank 1984). It is unreasonable to consider drivers to be homogenous and consistent in the real world (Troutbeck and Brilon 1997), thus in any model, the critical gap for *a particular driver* should be represented by a stochastic distribution. Also, *a group of drivers* will have different values of the critical gap or different stochastic distributions of the critical gap. Bottom and Ashworth (1978) further indicated that permitting inconsistent *drivers* was more realistic than permitting heterogeneity in the driver *group*, since the major source of variability in gap acceptance was likely to be due to individual drivers and not variation between them.

Tian et al. (2000) investigated the factors affecting critical gap and follow-up time, concluding that drivers use shorter critical gap at higher flow and delay conditions. Many other factors have also been found to affect the value of critical gap, such as intersection geometry, traffic movement, vehicle type, speed limits, gender, age, time of day etc. (Harwood et al. 1999, Tian et al. 2000). Thus, a critical value, obtained for any given situation, is unlikely to be generally applicable.

Priority sharing: According to the priority rules, the vehicles from major streams have absolutely priority over the vehicles from minor stream. However in reality, *priority sharing always occurs*. **Priority sharing** is a phenomenon, which allows for non-absolute priority of the major-stream vehicles. This phenomenon is usually believed to be caused by the high volume of traffic flow (Troutbeck and Kako 1999) and saturation on the minor stream (Harwood et al. 1999).

It may be generated by aggressive behaviour of the driver from a minor stream. It may also be the result of courtesy of a driver from one of the major streams. Harwood et al. (1999) believe it is most often caused by the minor-stream driver compel a major stream driver to give way by using a gap so tiny that the latter has to reduce speed. Based on field observations, Troutbeck and Kako (*1999*) indicated that major-stream vehicles could be slightly delayed to accommodate a minor vehicle. Harwood et al. (1999) described the phenomenon in terms of speed reduction to 85% for a major-stream vehicle. No matter what the triggers, the facts are that drivers from minor streams will use technically too small gap and the drivers from major streams will experience consequent delay.

Traditional gap-acceptance models have failed to take this phenomenon into account, but more recently research (Troutbeck and Kako 1999) has tried to overcome it by adding an additional factor "*C"* value in the capacity formula to justify the priority sharing effects. This *C* value ranges from 0 to 1 and depends on the headway distribution. Although this modification can improve the accuracy of previous gap acceptance models, it has provided little help in analysing the TWSC operation unless there is evidence or conclusion that priority sharing is directly related to the headway distribution.

Give-way: The phenomena of "give-way" also occurs between vehicles from two different major stream directions as one RT vehicle needs to give way to a ST or LT vehicle from the opposing direction stream. The effects depend on the turning and flow rates. When the vehicle from a major stream is waiting for a suitable gap, no ST or RT vehicle from a minor stream can drive into the intersection. Therefore, the capacity of an entrance does not depend solely on the gap distributions of the major streams, but also on the delay that the vehicle from the major- stream will experience.

Conflicts*:* Gap-acceptance models have also failed to consider conflicts between the two major-streams, which *change the headway distributions*. When RT vehicles (for left-side driving) in the major-stream give way to ST vehicles from the opposing street, a queue will form on the major-stream behind the subject vehicle, if the road is narrow (i.e. turning-left and going-straight vehicles share the same lane). The headway distributions affected so that the original gap-acceptance criteria no longer apply. No vehicle from a minor stream may drive onto the intersection unless it turns left.

Platoons: It is difficult to apply the gap-acceptance model on an unsignalised intersection in an urban network, since adjacent intersections with traffic lights will have grouped the vehicles into a queue (or queues) during the red signal phases, and there will thus be platoons present, (i.e. a filtering effect). The filtering of traffic flow by traffic signals has a significant impact on capacity and performance of an unsignalised intersection (Tracz and Gondek 2000).

Robinson et al. (1999) indicated that the gap-acceptance model could be applied only when *no* platoon is present. Otherwise, no minor-stream vehicle can enter the intersection, as the mean headway inside a platoon is supposed to be less than the critical gap. If the traffic signal cycles are known and are co-ordinated, the platoon pattern may be predictable. If the lengths of signal cycles are different and independent, the pattern is less predictable (Robinson et al. 1999), and traditional gap-acceptance is impossible to apply.

Even without traffic lights present, platoon formation in traffic flows is unavoidable, as the speeds of vehicles are different. At the same time, the critical gap is not easy to define and implement when several traffic streams are involved (Tian et al. 1999). Hence, using the critical gap and headway distribution may be too simple alone for the complexity of the interaction at many intersections.

In gap-acceptance models, the effect of directional flow is also not specifically modelled, with the driver of a vehicle travelling straight through facing a different decision based on whether major street vehicles approach from the left or from the right (Tian et al. 2000).

Our approach: New CA model proposed in this chapter uses an analogous but more flexible methodology compared to that of gap-acceptance. It not only facilities understanding of the interaction between the drivers, but can also be applied to situations for which headway distributions are insufficient to describe traffic flow.

A CA ring was first proposed for unsignalised intersections (Chopard et al. 1995 and 1998). All entry roads are “connected” on the ring. The car “on the ring” has priority over any new entry. However, there is no differentiation between the major and minor entry roads. All entry roads have equal priority and all vehicles have equal priority to move into the ring (intersection), which compromises usual TWSC rules (for details see Section 2.4.2.1). A further CA model variant for intersections described (Esser and

Schreckenberg 1997) has also not considered detailed interactions between drivers. The approach described below seeks to remedy these shortcomings.

3.3 Methodology

A *two-speed* one-dimensional deterministic CA model (1DDCA), (Yukawa et al. 1994, Chopard et al. 1998, Wang and Ruskin 2001, Ruskin and Wang 2002) is used to simulate the interaction between the drivers, in which the speed of vehicle is either 0 or 1(v_{max}=1)*, **on intersections only***. A vehicle can move only one cell in a given time-step.

While multi-speed CA models, (Nagel and Schreckenberg 1992), critical to successful modelling of freeway traffic are somewhat similar, these have many features, which are superfluous for urban traffic such as intersections and roundabouts or to representation of driver behaviour (Chopard et al. 1996 and 1998). Moreover, vehicle dynamics are often less important than driver interactions in simulating queue formation in urban networks (Queloz 1995 and Chopard et al. 1998).

One time-step is equal to 1 second throughout this thesis. The length of each cell corresponds to the average speed on a given section of the road. For example, *for two-speed 1DDCA*, if average speed of passing the intersection is 32.4 *km/h,* the length of *1 cell = 9 m,* while if average speed is around *50km/h,* then the length in each cell = 13.89 m.

A *three-speed* 1DDCA (Nagel and Schreckenberg 1992, Nagle 1996), is used to model the traffic flow on ***straight roads only*** (two-speeds 1DDCA is applied to intersection area only) in urban networks. For a three-speed 1DDCA, speed of vehicles is 0, 1 or 2 (v_{max}=2)*,* corresponding to speed of 0, 25 km/h and 50 km/h. Length of *1 cell = ~7 m* in *three-speed 1DDCA*.

We can either increase the length of each cell or increase the number of speeds if we want to apply our model to interurban networks. In other words, our models do not have any limitation on speeds and can be applied over a wide range. Therefore, they can be applied to either urban or inter-urban networks.

3.3.1 Up-date on single-lane roads

There are three actual speeds on the single-lane road: 0, 1 or 2, but two possible speeds for the next time step: 1 and 2. If a vehicle does not move in this time step (actual speed = 0), the maximum speed will be 1 for the next time step (the possible speed = 1). If the vehicle moves one or two cells (actual speeds = 1 and 2) in *current* time step, the maximum possible speed will be 2 (the possible speed = 2). These rules on speeds are for simulating the acceleration and deceleration process.

The update rules are show as follows. C^{t}_{n} designates the state of the n^{th} cell at time-step t. If $C^{t}_{n} > 0$, there is a vehicle in n^{th} cell at t'th time-step and the possible speed is C^{t}_{n}. The algorithm will be:

- If $C^{t}_{n} = 1$ and $C^{t}_{(n+1)} = 0$, then $C^{(t+1)}_{(n+1)} = C^{t}_{n} + 1$and $C^{(t+1)}_{n} = 0$ (If the possible speed is 1 and the cell in front is vacant, then the vehicle will move one cell and also increase its speed to 2 in next time step).
- If $C^{t}_{n} = 1$ and $C^{t}_{(n+1)} > 0$, then $C^{(t+1)}_{n} = C^{t}_{n}$ (If the possible speed is 1 and the cell in front is occupied, then the vehicle will not move and the speed is unchanged (=1) in next time step).
- If $C^{t}_{n} = 2$ and $C^{t}_{(n+1)} > 0$, then $C^{(t+1)}_{n+1} = C^{t}_{n} - 1$and $C^{(t+1)}_{n} = 0$ (If the possible speed is 2 and the cell in front is occupied, then the vehicle will not move and the speed decreases to 1 in next time step).
- If $C^{t}_{n} = 2$ and $C^{t}_{(n+1)} = C^{t}_{(n+2)} = 0$, then $C^{(t+1)}_{(n+2)} = C^{t}_{n}$ *and* $C^{(t+1)}_{(n+1)} = C^{(t+1)}_{n} = 0$ (If the possible speed is 2 and both two cells in front are vacant, then the vehicle will move two cells forward and the speed is unchanged(=2) in next time step).
- If $C^{t}_{n} = 2$, $C^{t}_{(n+1)} = 0$ and $C^{t}_{(n+2)} > 0$, then $C^{(t+1)}_{(n+1)} = C^{t}_{n}$ and $C^{(t+1)}_{n} = 0$ (If the possible speed is 2 and only one cell in front is vacant, than the vehicle will move one cell forward and the speed is unchanged in next time step).

3.3.2 Driver behaviour categories

A two-stream intersection (Figure 3.1) is used to illustrate the driver interaction. Theoretically, a vehicle at the stop-line of a minor-stream can drive onto the intersection, without interrupting the major flow, when the space between two vehicles on the major-stream is three cells or more. Thus, three cells give the minimum theoretical acceptable space.

Driver behaviour is categorised into four groups: *radical, urgent, rational* and *conservative*. If a driver accepts a 3-cell space as the ***M**inimum **A**cceptable s**P**ace* (**MAP**) and enters the intersection, behaviour is rational. One cell space is required by *radical* behaviour. The driver will take any space on the intersection without any consideration of safety. The consequence is that the vehicle may generate gridlock (see Chapter 4,5 and 6). A 2-cell space corresponds to *urgent* behaviour, which may be the result of e.g. misjudgement, over confidence in the vehicle acceleration, bad driving habits, urgency of travel or the phenomenon of *priority sharing*. The effect is the blocking of the vehicle that has priority by the sub-rank vehicle. *Conservative* behaviour corresponds to MAP $\geq$ 4cells.

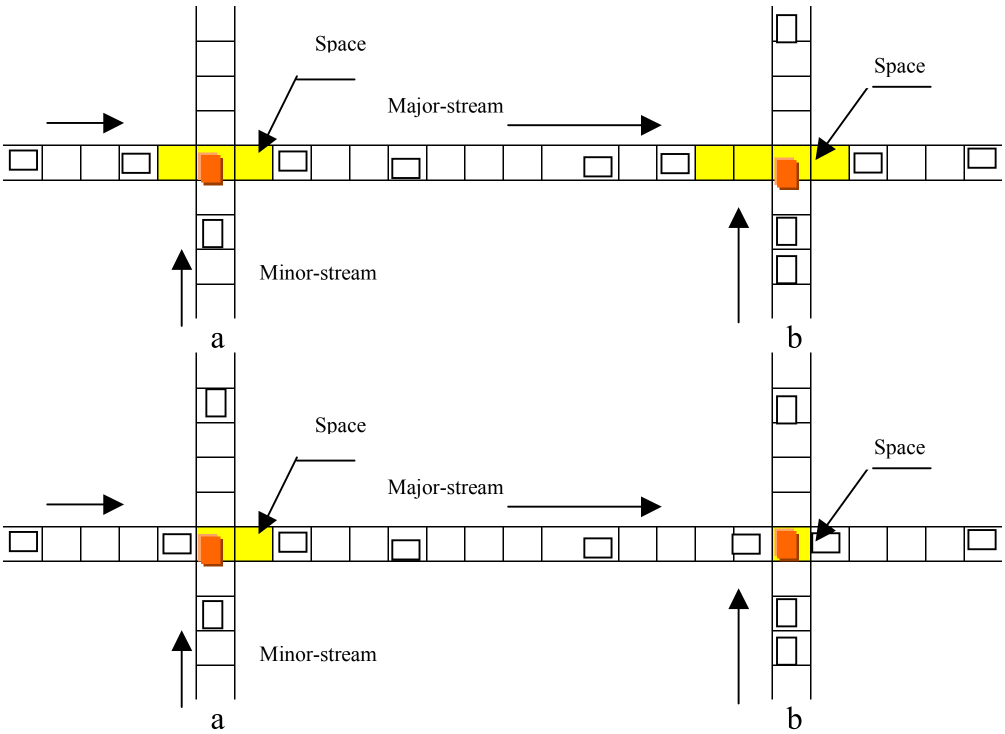

Figure 3. 1 Two-stream intersections: (a) rational, (b) conservative, (c) urgent, and (d) radical.

Harwood et al. (1999) indicated that drivers are likely to prefer longer gaps for the more complex decision involved in turning, even though longer gaps are not required theoretically. We thus expect that most driver behaviour can be classified as rational or conservative.

The distribution of driver behaviour is expressed as four probabilities for conservative, rational, urgent and radical behaviour denoted P_{co}, P_{ra} P_{ur} *and* P_{rad} respectively. Clearly

$$P_{ur} + P_{ra} + P_{co} + P_{rad} = 1 \qquad (3.1)$$

According to the above driver behaviour distribution, each driver from a minor-street at a stop-line of an intersection or right-turning from a major-street is randomly assigned to one of four driver behaviour categories at each time-step. In this way, heterogeneous and inconsistent driver behaviour is simulated. In other words, if a driver is assigned to one category in this time-step and its space conditions are not met, the vehicle is stationary in this time-step. The driver may be re-assigned randomly to any of the four categories according to the behaviour distribution in the next time-step. If he/she is assigned to a new category, his/her space requirements are thus changed.

3.3.3 Stop Sign Delay Time (SSDT)

According to the rules of the road, a vehicle from a minor-street has to obey a stop sign before it can enter an intersection. Our simulation ensures that all vehicles from the minor-street will stop for at least one time-step (equal to 1 second). For minor-street vehicles travelling ST or RT, a two time-step delay is allowed, in order to make a decision, (two major-streams are checked). We denoted the time required as stop-sign-delay-time (SSDT). Thus, the follow-up time for a minor-street in the simulation will be from 2 to 3 seconds, which agrees with the recommended follow-up time from observed data (Troutbeck 1984, Flannery and Datta 1997).

3.3.4 Comparing MAP method with gap-acceptance models

The main difference between our CA model, which is referred to as the MAP method in this thesis, and gap-acceptance models in general, is that the MAP in our model and the critical gap in the gap-acceptance model have different temporal and spatial content, although both provide criteria for a driver to take action.

For the gap-acceptance model, where the conflicting flow includes more than two streams, the gap is normally defined as the time taken for two vehicles from conflicting streams to pass through the path of the subject vehicle. Without distinguishing the direction that each vehicle comes from, the critical gap then has strong temporal meaning but is weak in spatial detail.

However, in our model, the space required (in terms of different number of vacant cells required in *each* conflicting stream) is clearly specified so that temporal and spatial details are known for each different movement (e.g. RT, LT or ST). The temporal details are derived from the speed conditions; the vehicle moves no more than one cell in one time step, so time can be measured in terms of number of cells. The spatial meaning is expressed precisely for different streams (details below), and the driver decision process is thus fully specified.

3.3.5 Interaction at intersection entrance

Before we describe how to apply our MAP method to intersections, we address the issue of time. In the CA model described, the states of all cells update simultaneously. This means that the states of all cells have been updated in *this time-step* and the vehicle moves onto the intersection *in next time-step* when the conditions have been met. Figures 3.2 – 3.5 represent the current situation for available spaces and to follow through on the movement, we consider the situation at the *next time-step*. (This will be revised slightly in Chapter 6, for reason explained there).

For single-lane TWSC intersections, the minimal acceptable space conditions for a vehicle from a minor-street to move onto the intersections in the *next time step* are shown in the following figures (the shaded cells). The conditions depend on the direction of movement and driver behaviour. The detailed space criteria contains the requirements for each marked cell, which is labelled with 0, a, b or c, having the following meanings:

- "0" means that the cell is vacant
- "a" means that the cell is either vacant or occupied by a vehicle that will turn left
- "b" means that the cell is *not* occupied by a RT vehicle
- "c" means that the cell is either occupied by a RT vehicle or vacant

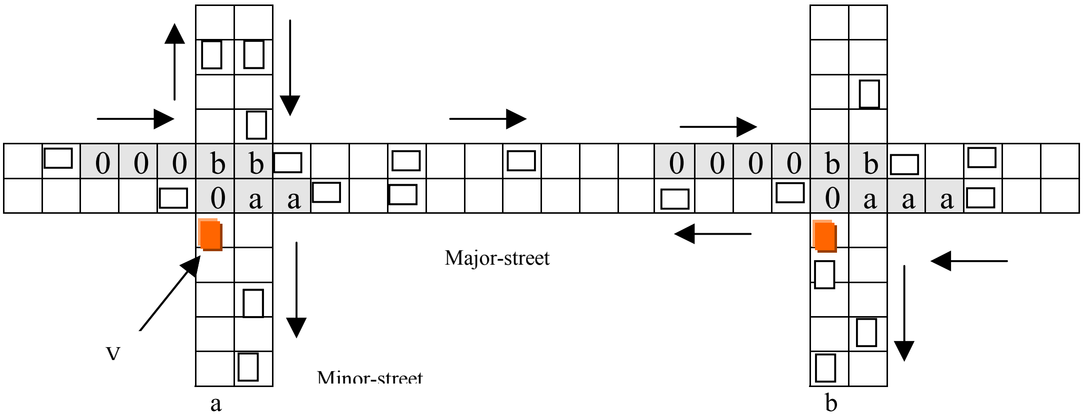

Figure 3. 2 A ST vehicle from a minor-street: (a) rational behaviour, (b) conservative behaviour.

The space conditions for the ST vehicle V to move into the intersection are illustrated in Figure 3. 2. A rational driver needs to observe the 7 marked cells before s/he can drive onto the intersection. In contrast, a conservative driver needs to check 9 marked cells (Figure 3.2 b).

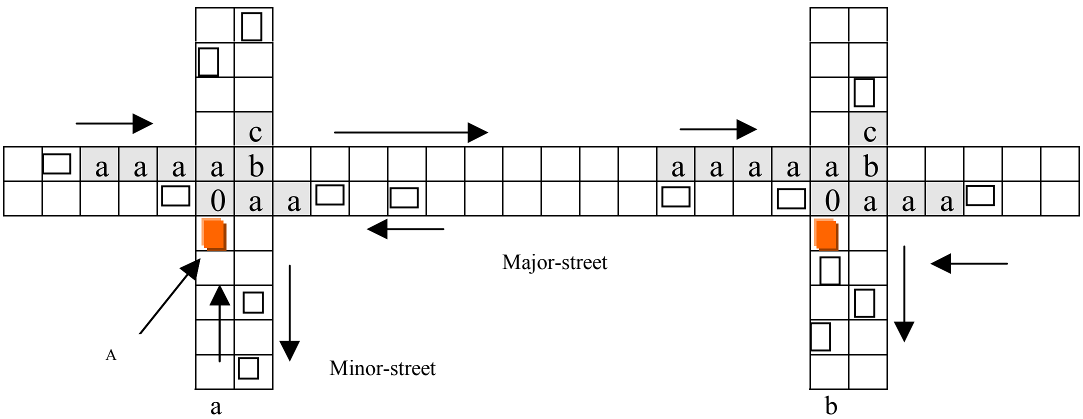

Figure 3.3 A RT vehicle from a minor-street: (a) rational, (b) conservative.

Figure 3.3 indicates the conditions for a rational or conservative RT vehicle driver to enter the intersection from a minor-street. A vehicle from the opposing minor-street, ST or LT has priority over a RT vehicle from the given minor-street according to the rules of the road. However, Tian et al. (2000) indicated that the priorities between minor-street vehicles were not distinct. They indicated that drivers were observed to enter the intersection on a first-come, first-served basis. The movement of a RT vehicle from a minor-street does not need to consider opposing vehicles if one of the following conditions is met.

- The first cell in the opposing minor-street is vacant
- A RT vehicle is the first vehicle in the opposing minor-street
- The first vehicle in the opposing minor-street arrives at a stop-line in less than SSDT (Section 3.3.3)

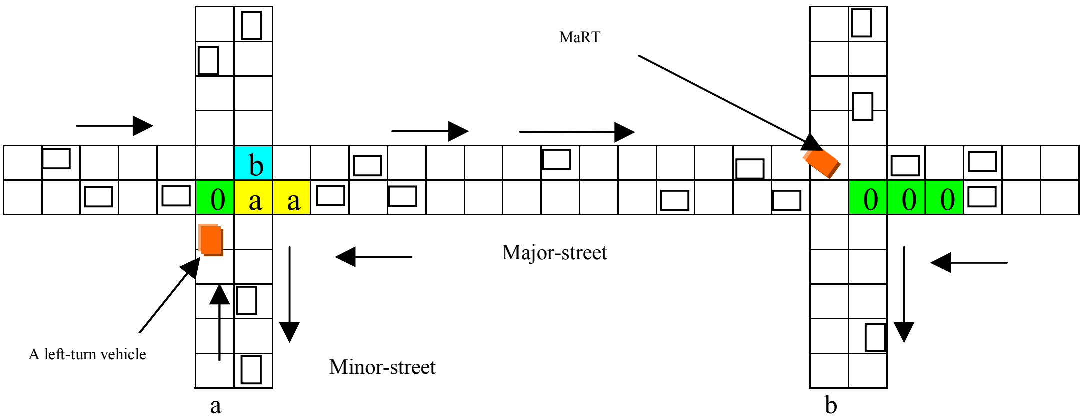

Figure 3.4 All rational: (a) a LT vehicle from a minor-street
(b) a RT vehicle from a major-street (MaRT).

For rational drivers, the space conditions for a LT vehicle from a minor-street (MiLT) are shown in Figure 3.4a and the space conditions for a RT vehicle from a major-street (MaRT) are shown in Figure 3.4.

***Heterogeneity of Vehicles*:** The case for a long vehicle can be considered briefly based on occupation of more than one cell (e.g. two cells, see Figure 3.5). An additional cell space is needed for a long vehicle to move onto the intersection. Rational movement through the intersection requires a check on the same number of cells that a conservative car driver in the simple model will perform.

***Intersection Variants*:** A flared minor-street increases the capacity of an intersection. Two vehicles can stop and depart from the stop line simultaneously as a result of a large curb radius or a parking prohibition. These conditions transfer a single-lane into a limited two-lane street. In a recent study (Robinson et al. 1999), the authors have indicated that the magnitude of this effect depends, in part, on factors such as the turning-movement volume and the length of the second lane etc.

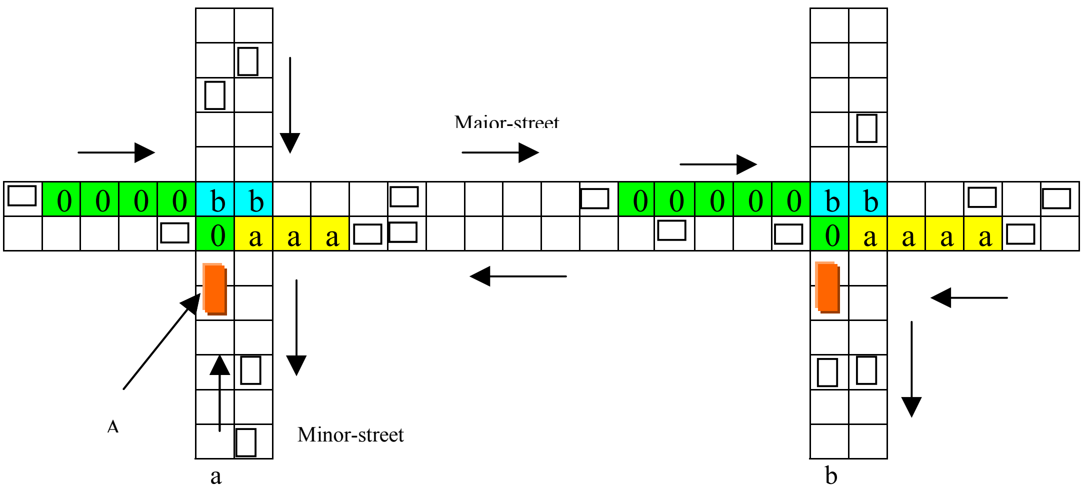

Figure 3.5 A long going-straight vehicle from a minor-street: (a) rational (b) conservative.

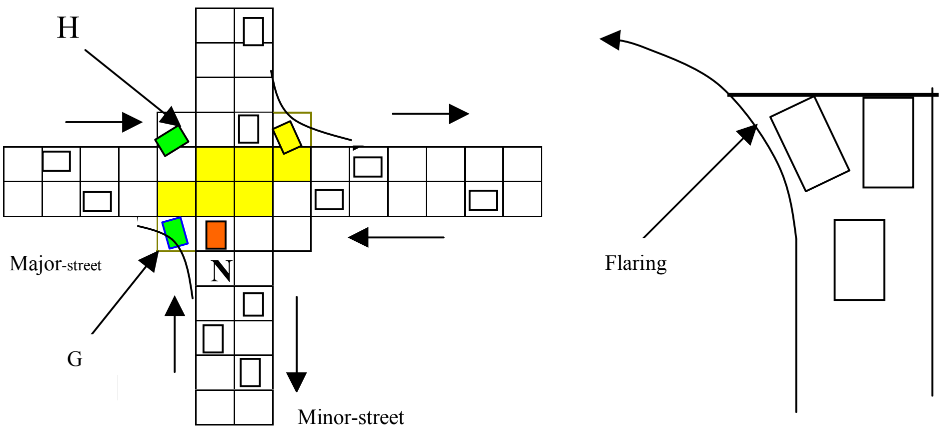

Figure 3.6 Intersection with flaring.

In the case of just one space in the second lane, which is very common, the intersection can be simulated as in Figure 3.6. One extra cell G is located on the corner of the intersection. If cell G is free and the vehicle in cell N is a LT vehicle, the vehicle can move into cell G. If RT or SA, the vehicle will not move into cell G and will continue straight ahead. A rational driver in cell G needs three vacant-cells to move onto the intersection in the next time step. Cell H also can be used by a LT vehicle from a major-street.

3.4. Single-lane Intersection Simulation

Based on the assumptions described, we studied performance measures (capacity, time delay and queue-length) of a TWSC intersection under different values of traffic flow parameters, such as arrival rate (traffic volumes) and turning rate (turning proportions). In order to obtain the maximum capacity, the given street must be fully saturated. Experiments were carried out for 36,000 time-steps (equivalent to 10 hours) for a street-length of 100 cells for all approaches. All driver behaviour is assumed rational unless otherwise specified. The **arrival rate** (AR) is the probability that vehicles arrive at one end of a road in a given time period. Vehicles arrive at random with Poisson distribution, (where AR = $\lambda \leq 0.5$ (1800vph) in general for free flow). If all arriving vehicles pass the intersection without queueing, the flow rate corresponds to AR, (for AR=0.1, 0.2, the flow rate is equivalent to 360vph, 720vph respectively).

3.4.1 Capacity of a minor-street

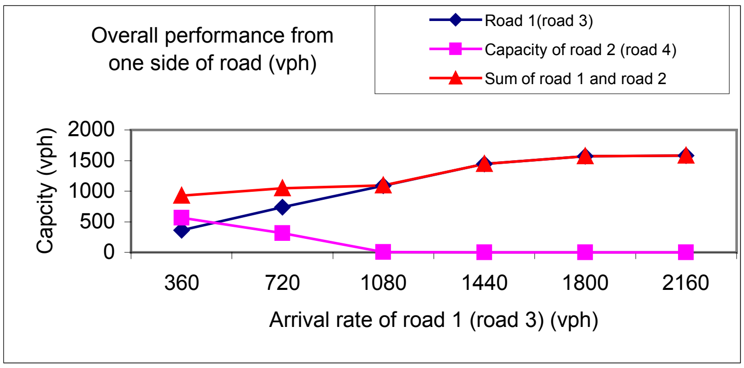

Figure 3.7. Overall performance of intersection

Vehicles are assumed to converge from all directions. Arrival rates of the two major-streets are taken to be equal. Arrival rates of minor-streets are set to the maximum flow rate (1800 vph) that the single-lane road can manage. On both major-streets, LT rate (LTR): ST rate (STR): RT rate (RTR) = 0.2:0.6:0.2. On both minor-streets, LTR: STR: RTR = 0.4:0.2:0.4. Figure 3.7 shows the entry capacity of the minor-street (roads 2 or 4). The entry capacity is nearly zero when arrival rates of the major-streets > 1080 vph.

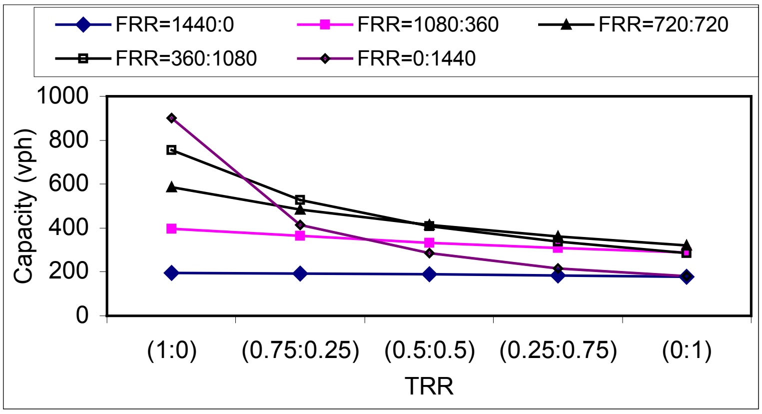

Figure 3.8 Capacity of minor-street of T-intersection with TRR of minor-street and FRR of major-streams.

When a RT or ST vehicle from a minor-street involves two major-streams, the capacity depends on their flow rates and configurations. In order to determine impact of different turning rates and different major-stream combinations, a T-intersection is studied, which contains only RT and LT vehicles in the minor-stream. All major-streams are assumed to have only ST vehicles. The total number of vehicles per hour in major-streams is assumed to be 1440 vph, which is split between the near-lane stream, (vehicles coming from the right), and far-lane stream, (vehicles coming from the left). Both left-turning-rate (**LTR**) and right-turning-rate (**RTR**) are varied. The differences in turning rates of the minor-stream can be expressed in terms of *turning rate ratio* (***TRR** =LT rate: RT rate*)**.** The difference in flow rates of the two major-streams can be expressed in terms of *flow rate ratio* (***FRR**= flow rate of near lane: flow rate of far lane*).

Table 3.1 and Figure 3.8 indicate that both TRR and FRR affect capacity and both ratios should therefore be considered. In our simulation, TRR has been varied by increasing the number of RT vehicles in the minor-street. We find that the capacity of the minor-stream decreases in general when TRR decreases. However, this effect differs as FRR varies. When FRR increases (by increasing flow rate of near lane), the decrease in capacity is less marked, and vice versa.

Table 3.1 Capacity of Minor-street vs. TRR and FRR

TRR (=LT rate: RT rate)	Capacity (vph)				
	FRR(=Flow rate of near lane : Flow rate of far lane)				
	1440:0	1080:360	720:720	360:1080	0:1440
1:0	196	397	585	755	900
0.75:0.25	193	363	483	527	415
0.5:0.5	190	331	413	408	286
0.25:0.75	183	308	361	337	217
0:1	177	288	321	286	180

3.4.2 Capacity of a major-street

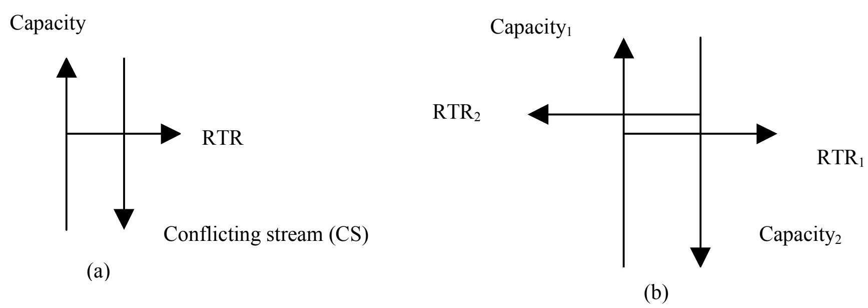

Figure 3.9 Traffic configurations of shared lane on the major-streets

Right-turning vehicles from major-street (MaRT) in a shared major-street, where RT, ST and LT vehicles are on the one lane, can block ST and LT vehicles behind and in the same stream. RT rates (RTR) of major-streams thus have great impact on capacities of major-streams. Two configurations have been studied (Figure 3.9), with the analysis of major-street capacity similar to that of Chodur (2000).

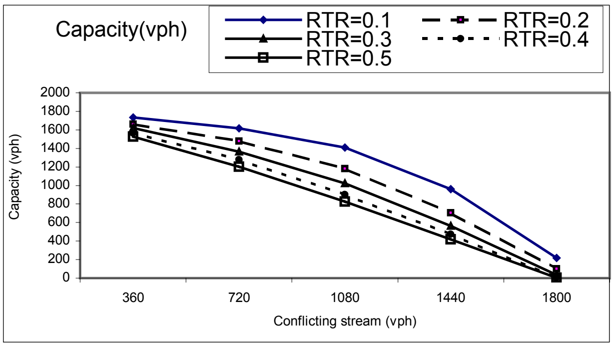

Figure 3.10 Capacity of a major-street in situation of Figure 3.9(a) for rational driver behaviour

Figure 3.10 shows unsurprisingly that the capacity of the major-stream declines rapidly with RTR and flow rate of conflicting major-stream increase (Figure 3.9(a)), where only one major-stream has RT vehicles. Table 3.2 for major-stream capacities, (both with RT vehicles in Figure 3.9(b)), yields a similar relationship (Expression 3.2) to that found from empirical study by as found also by Chodur (2000).

$$Capacity_1 : Capacity_2 = RTR_2 : RTR_1 \qquad (3.2)$$

Table 3.2 Capacities and capacity ratio vs. right-turning rate ratio

	$RTR_1:RTR_2$					
	0.4:0.1	0.3:0.1	0.2:0.1	0.2:0.2	0.2:0.3	0.2:0.4
$Capacity_1:Capacity_2$	~1:4	~1:3	~2:1	1:1	~3:2	2:1
$Capacity_1$(vph)	413	541	758	1164	1373	1480
$Capacity_2$(vph)	1659	1616	1508	1164	911	740

3.4.3 Queue-length and Delay

The **length of a queue** on a road is defined as follows: the queue starts to form at the intersection, and will grow along the road opposite to the direction of movement of vehicles. The furthest two adjacent cells, which are occupied by two vehicles, indicate the end of queue. This definition means that a vehicle is in the queue if the vehicle can not move in next time step because the cell in front is occupied.

The following assumptions are made in the simulations:

- Major streets (approaches 1 and 3): LTR: STR: RTR = 0.2:0.7:0.1 and FRR=0.15:0.15 (AR_1 =AR_3 =0.15)
- Minor-streets (approaches 2 and 4):LTR: STR: RTR= 0.4:0.2:0.4 , AR_2 = 0.05

In this case, the capacity of approach 4 is 518 vph. When arrival rate > 518 vph (i.e. AR_4> 0.144), the queue grows rapidly to maximum (= length of approach).

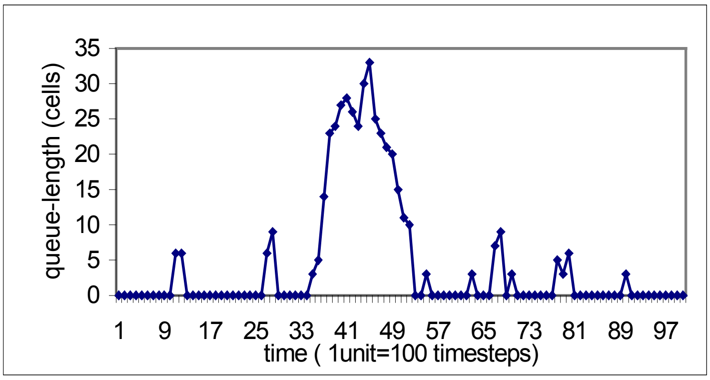

Figure 3.11. Queue-length of approach 4.

Figure 3.11 gives some of the results for minor street queue-length for the degree of saturation (= arrival rate /capacity) 0.90 (i.e. AR=~0.13). The maximum queue-length reached on approach 4 was 42 cells, but was <27 cells for 95% of the time queue-lengths < 27 cells. The corresponding maximum delay time was found to be 227 time-steps (seconds) and 95% of the drivers experienced less than 113 time-steps delay. 50 % of drivers could expect a delay of less than 18.5 time-steps.

3.4.4 Driver behaviour

Table 3 illustrates the effects of different driver behaviour populations. In each scenario, turning rates and arrival rates are fixed, with AR of three approaches < 0.5, AR_4 = 0.8 is much large than 0.5 for approach 4 (a minor-street) only.

An approximate linear relationship is observed between the capacities and driver behaviour ratio. Hence we could use the capacities to roughly calibrate the driver behaviour distribution.

Table 3.3 Capacity vs. diver behaviour

Modelled Scenarios	Driver Populations (Rational :Conservative)				
	1:0	0.75:0.25	0.5:0.5	0.25:0.75	0:1
Scenario 1	518	492	464	435	406
Scenario 2	412	377	343	308	269
Scenario 3	527	504	482	461	437

An approximate linear relationship is observed between the capacities and driver behaviour ratio. Hence we could use the capacities to roughly calibrate the driver behaviour distribution.

3.5. Summary

A new cellular automata model is proposed to simulate directly the interactions between drivers at TWSC in urban networks using space considerations. The heterogeneity and inconsistency of driver behaviour is also investigated. The method can be easily applied to many features of urban traffic, where gap-acceptance models are less amenable to study.

The capacity of the minor-street in a T-intersection not only changes with the flow rates of major-streams, but also changes with flow rate ratio. Flow rates corresponding to each stream must be distinguished. The capacity of a minor-stream decreases when LTR decreases, but this is again dependent on FRR. When FRR increases (flow rate of near-lane increases), the decrease in capacity is less marked.

The major-street capacities depend on the flow rate of the opposing stream and RT rates of both major streets.

The queue-length and delay time of each street in each time step can be directly obtained from the model. Also, the relationship between the performance measurements of intersections and parameters of traffic flow are easily derived from the simulation.

Lacking real data, the distribution of driver behaviour is arbitrarily decided in the experiments, but the model can be used to investigate various assumptions and conditions of performance for TWSC intersections.

Chapter 4

Multilane TWSC Intersections

4.1. Introduction

Much research has been done on traffic-light controlled intersections, but rather less on unsignalised urban intersections. In particular, research is very rare on multilane unsignalised intersections. The unwritten perception seems to be that such research is largely unnecessary, since most traffic is controlled. Unfortunately this is not in fact the case and where inter-urban sprawl is considered, is even less likely to be universal. Considering the number of unsignalised intersections and comparing it with the number of signalised intersection in a traffic network (e.g. Dublin, the aerial photographs provided by Mapflow http://www.mapflow.com/webdemo/demomap.asp), modelling intersections with several lanes is still well founded. The area shown in the map (Appendix E) is less than 3% of the area of Dublin. It is close to the city centre. In this area, the number of intersections without traffic lights is over 30, but the number of intersections with traffic lights is less than 16. This can give us an approximate picture, for this example of part of an urban network, of the percentages which apply.

In particular, situations which cause problems at unsignalised intersections need to be documented and a number of "what if" questions need to be asked about the changes in flow likely to be caused by the introduction of traffic lights or other control options.

In addition, intersection manoeuvres provide information on use of other urban and inter-urban road features, not least those of roundabouts. These attempts to reduce at least some of cross-traffic problems, but also introduce others, not least because these are frequently unsignalised (or wrongly signalised) also. Consequently, an understanding of the natural flow dynamics for these configurations is extremely important in any attempt at planning. Furthermore, it is now widely accept that the influence of human behaviour on traffic system operation is of great significance and has been the focus of a number of studies, e.g. Lajunen et al. (1999), Norris et al. (2000) and Hakkert et al. (2001).

In this chapter, we concentrate on the two-lane two-way stop-controlled (2-TWSC) intersection. In particular, we introduce a new process of lane allocation. At the end of the chapter, modelling traffic flow at a two-lane traffic-light controlled (2-TLC) intersection is also reported in order to compare 2-TWSC and 2-TLC intersections.

4.2 Background

Two types of multilane TWSC intersections are commonly used in urban areas. The difference between them, in general, is whether there is a "bay" between the two different major stream directions. There are two major functions of the bay area: (a) to allow a RT vehicle to wait there for an opportunity to progress, (b) to allow a straight-going or RT vehicle from a minor street to drive onto the bay area first and stop there to wait an opportunity to progress to the second step. This type of intersection therefore is also known as "two-stage priority" (Brilon and Wu 1999), and has been studied, Bonneson and McCoy (1997), Brilon and Wu (1999), and Bonneson and Fitts (1999).

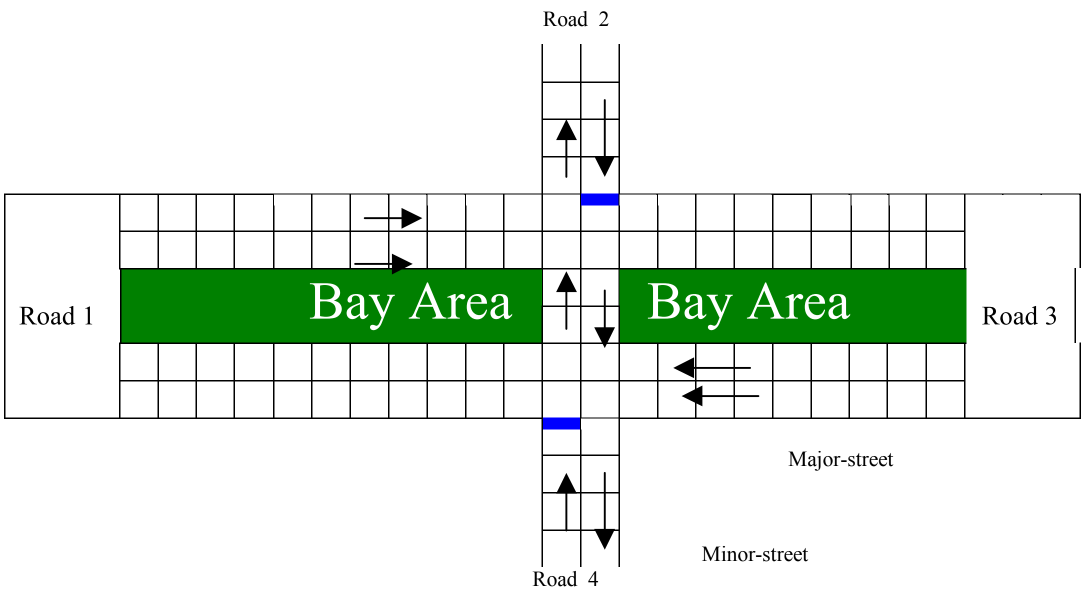

Figure 4.1. Two-stage priority intersection

A TWSC multilane intersection with bay area (Figure 4.1) has many similarities to a multilane roundabout (details in Chapter 6). Basically, an entering vehicle only needs to check a single direction of traffic flow. Hence, in this chapter, we mainly concentrate on a TWSC multilane intersection *without* a bay area (Figure 4.2), which has two lanes in each major directional flow and is thus the more general and complicated case. In this case, the ST/ RT vehicle from a minor street needs to check four/three traffic streams and two directions of traffic flow before entering the intersection. These situations arise in urban and (even more commonly) interurban areas. One example in Dublin is Gardiner Street (Upper, Middle and Lower), located close to the city centre and with two lanes in each major direction flow. Over five intersections on this street do not have traffic lights. These can also be readily located on aerial photographs provided by Mapflow (ref. previous section).

Basically, vehicles at a TWSC multilane intersection observe the same priority rules and stop rule as at a single-lane TWSC intersection (Chapter 3). The model of traffic flow at a TWSC multilane intersection includes the following processes:

- Vehicle arrivals at the beginning of an entrance road (e.g. 100 cells away from the intersection)
- Lane allocation for vehicles in major flow
- Halts induced by stop-sign, i.e. Stop Sign Delay Time (**SSDT**), see Chapter 3.
- Vehicle movement along roads
- Interaction between drivers on the intersection

Vehicles from a minor street must obey the stop rule, i.e. vehicles must stop before passing the stop-line. The delay experienced is then defined as Stop Sign Delay-Time (**SSDT**)(as in Chapter 3). Our simulation ensures that all vehicles from the minor-street will stop for at least two time-steps (equal to 2 seconds). In this chapter, we assume that the SSDT times for a LT, ST and RT vehicle from a minor street are 2, 4 and 4 seconds respectively. Basically, the SSDT is the time needed for a vehicle to stop and check the traffic situation *once*. If the situation meets the driver's requirements, he/she will begin to pass the stop-line, otherwise he/she will wait. The duration of SSDT thus depends on the number of lanes needing to be checked and the complexity of the manoeuvre.

For a vehicle feeding into a major street, the vehicle needs to be allocated to a lane before it can enter, (as there are two lanes for each direction). There is no need to consider lane allocation for vehicles entering a minor entrance road, (considered to be single-lane only). Given the requirements, imposed by movement through the major road streams, we feel that this is justified, since major road traffic alignment is crucial to successful negotiation of the interaction. The only type of vehicle, which needs to negotiate one major stream only, is one, which turns left from the minor road and its movement is essentially unchanged from that of a single-lane major/minor intersection. In this chapter, we mainly focus on the second and the last two of the five processes (see Page 53) identified above, as the first and the third processes are similar to those of Chapter 3.

Hagring (1998) indicated that the process of lane allocation for a vehicle in the major flow needs to be considered in modelling multilane traffic flow. He also indicated that the lane allocation of major flow had a considerable effect on capacity. For example, a vehicle from a minor-street enters an intersection depending not only on the flow rates of major roads but also the distribution of vehicles on the major roads, (i.e. the traffic situation on the intersection is dictated by lane-allocation process).

Likewise, RT vehicles on a major street need to give way to ST vehicles from the opposite direction, queues may form on the right-lanes of major road (stream 3 and 4 in Figure 4.2). The queues are then obstacles to progress of subsequent ST vehicles. Consequently the delays of ST and RT vehicles from minor streets are also increased. As a result, the throughput of the intersection decreases and capacity of each minor road decreases.

4.3 Methodology

Figure 4.2 is an illustration of 2-TWSC intersection with the major roads (1 and 3), the minor roads (2 and 4), streams and key movement all highlighted. We define each lane as a ***stream***. For instance, if a major street has four lanes (there are two lanes in each direction of traffic flow), each direction has two streams.

The action of a 2-TWSC-intersection system begins with vehicle arrivals on the major or minor entry roads, where these are assumed to follow a Poisson distribution with parameter λ. The λ (equivalent to arrival rate (AR)) can be expressed either in the range of 0~1 (one time step = 1 second) or in the range of 0~3600 vph (in terms of vehicles per hour). The two expressions are interchangeable. Before each vehicle arrives on the major or minor entry roads, it has been randomly assigned to a destination based on a probability distribution of directions. For example, if *a %* of vehicles arriving by road 1 are assumed to turn right (i.e. turn into road 4), then these will be assigned a particular number in order to guarantee that these vehicles will eventually turn into road 4.

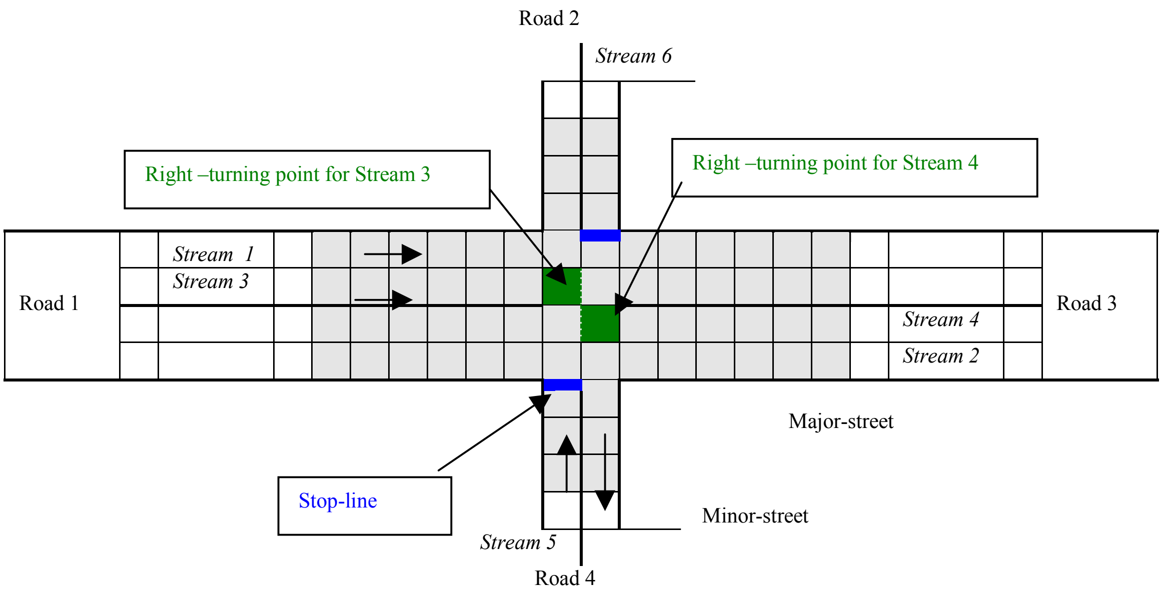

Figure 4.2. The intersection area where interactions between the vehicles occur

Arrival of a vehicle at the beginning of a feeder road (major- or minor- street) of an intersection does not necessarily mean that the vehicle can immediately progress. This also depends on the level of congestion on the road (for minor roads) or the particular stream of the road (for major roads). When vehicles move along an entry road, they retain their attributes, such as destination.

When a vehicle arrives at a stop-line from a minor street, it needs to check if there is enough space for it to drive onto the intersection. The space criteria (update rules) are defined as MAP (as in Chapter 3). When RT vehicles from major streets arrive at the RT point (Figure 4.2), they also need to check if the space meets the MAP. Further details on permitted movements are described in Section 4.3.3 below.

4.3.1 Lane allocation processes

We assume that a vehicle will *stay in* the lane after the vehicle is allocated to a lane of a major road, although in reality, some lane-changing may take place. The reason for this simplification is justified by the consideration that the intersection manoeuvre requires correct lane-allocation. Previous lane-changing is thus assumed to be minimal and can be disregarded.

We note, however, that a vehicle changes lane, normally, for the following reasons:

- To access a predetermined direction
- To escape from a queue in front
- To avoid a stopping service vehicle in front, such as bus or delivering vehicle
- To gain additional speed

Vehicles that change lane because of the last two reasons are more likely to change lane again, while the others imply that vehicles tend to stay in the lane after changing. If we overlook the process of changing forward and backward, LT and RT vehicles will finally end up in lanes that lead to their destinations, i.e. RT vehicles in the right-lane and LT vehicles in the left-lane. Clearly, straight though vehicles can end up on both lanes, unless specified by road signs. It is logically less likely for a driver in the absence of these to choose a lane where delays will occur, so that they will end up in a lane with a shorter queue.

Lane allocation types: There are three types of lane-allocation process on major roads, but only the last two are common for intersections and for driving on the left-hand side. (For roundabouts, the first and third types usually apply (details: see Chapter 6)).

- Left-lane used by LT vehicles only. ST and RT vehicles use right-lane.
- Right-lane used by RT vehicles only. ST and LT vehicles use left-lane.
- LT vehicles use left-lane and RT vehicles use right-lane only. ST vehicles can use both lanes.

In the first two situations, the lane-allocation process is relatively simple and clear, but the last situation is more complicated. If the major road has a high percentage of RT vehicles, it is necessary to specify that the right-lane is used by RT vehicles only (the second type), so that the delay for ST vehicles can be minimised.

For the last type, ST drivers choose a lane based on their perception of the delay expected. Normally, ST drivers would avoid using a right-lane in order to avoid delay behind RT vehicles, particularly when the driver can see the queue forming. If there is no queue on the right-lane or the queue is very short, then even if they are behind RT vehicles, the delay will not be significant and they may just remain in the right-lane. However if the queue is relatively long, ST drivers will tend to change lane as soon as they can. Thus, the queue length on the right-lane is the main factor in deciding ST vehicle choice of lane. Clearly, vehicles will not therefore be equally distributed between both lanes. In our model, the lane-allocation process is modelled by considering which lane a vehicle will end up in, rather than considering the intermediate lane-changing process.

Lane allocation assumptions: We model the lane-allocation process in a two-lane road, based on the following assumptions:

- All LT vehicles use the left-lane only. All RT vehicles use the right-lane only.

- Most straight-going vehicles will tend to use the left-lane to avoid the possible delay on the right-lane. We assume that over 50% of the vehicles (e.g. 60% vehicles arbitrarily) will use the left-lane unless they are RT vehicles.
- If there is a long queue (over three vehicles) on the right-lane and no queue on the left-lane, all ST vehicles will use the left-lane, unless there is no vacancy on the left-lane.
- If queues on both lanes, all ST vehicles will use left-lane except the RT vehicles, unless the queue on the left-lane is much longer than the queue on the right-lane (e.g. over ten-vehicles longer say).

Under the above assumptions, we can observe lane-allocation patterns, which will be used to model the 2-TWSC intersection.

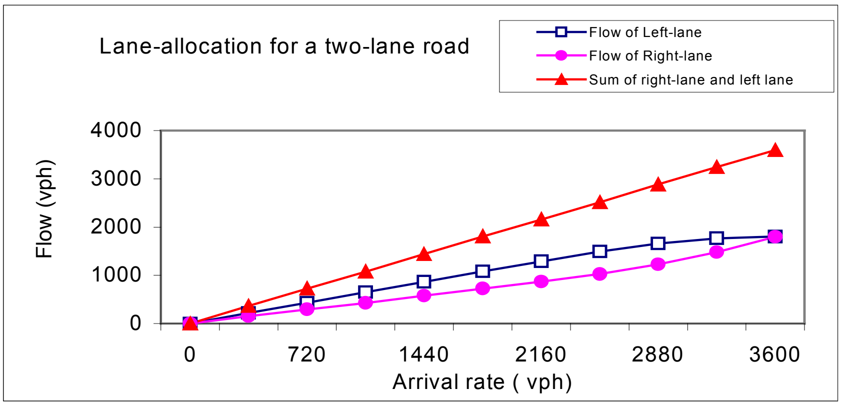

Figure 4.3 A lane-allocation pattern of RTR < 0.4 and LTR ≤ 0.5 if there is no queue on the right-lane.

Several realistic values are considered. For example, vehicles from a major road, a right-turning rate (RTR) < 0.4 and left-turning rate (LTR) ≤ 0.5 would be considered reasonable for a TWSC intersection in an urban area (Kyte et al. 1986). Figure 4.3 shows a lane-allocation pattern under these considerations. Obviously when neither lane develops a queue, flow of the road (sum flow of right-lane and left-lane) is equal to arrival rate. Approximately 60 % of vehicles use the left-lane when arrival rate ≤ 2520 vph (vehicles per hour). Further the difference between the two lanes reaches a maximum at the arrival rate of 2520 vph and decreases for arrival rate > 2520 vph. This pattern is caused by the left-lane becoming saturated. Some ST vehicles may move to the right-lane to avoid delay. Finally, vehicle numbers in the two lanes are equal when the arrival rate is equal to 3600 vph, i.e. the volume of vehicles arriving at the intersection-feeder road reaches the maximum capacity of the major road.

Figure 4.4 shows another lane-allocation pattern, when we now assume that RTR is increased and = 0.6. In this case the number of left-lane vehicles arriving constantly increases with the arrival rate increase, but the number of right-lane arrivals gradually reaches the maximum flow rate 1800 vph and remains at the maximum. So some RT vehicles arrive at the road, but can not progress until the right-lane has a vacancy. Consequently, the capacity of the road is around 3240 vph (RTR = 0.6), which is less than the maximum flow rate (3600 vph) of a two-lane road.

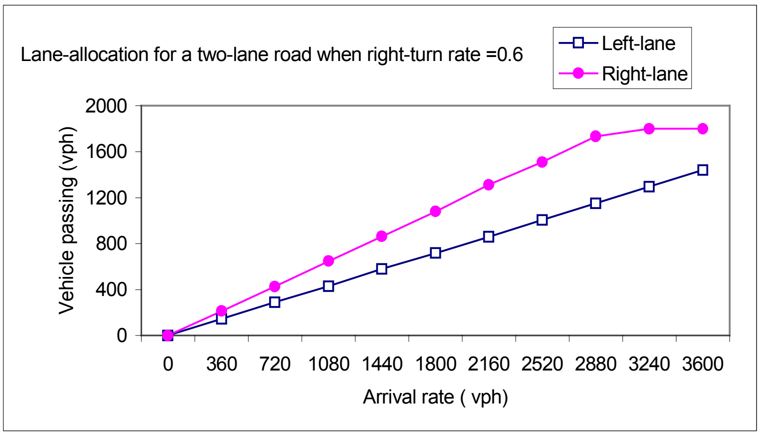

Figure 4.4. A lane-allocation pattern, when the RTR =0.6.

Figure 4.3 and 4.4 only show two relatively static situations. In our model, the lane-allocation process is a dynamic process, which means the model will check queue lengths at each time step, and allocate newly arriving vehicles to lanes accordingly.

4.3.2 Updates for two-lane roads

A two-digit number has been used to indicate a vehicle, where the first digit indicates the direction that the vehicle will take: value 1, 2 or 3 corresponded to LT, ST or RT respectively. The value of the second digit corresponds to the maximum speed that the vehicle can use in the next time step. A "1" or "2" thus means that the maximum number of cells which may be traversed in next time step = one or two respectively. However, actual movement depends on number of vacant cells immediately ahead, as does speed acquired (see Chapter 3). Speeds attend in consecutive time steps are governed by the conditions for acceleration and deceleration processes described in Section 3.3.1.

C^{t}_{n} means the state of n^{th} cell at time-step t. If $C^{t}_{n} > 0$, there is a vehicle in n^{th} cell at t'th time-step. C^{t}_{n} here refers to the second digit number of C^{t}_{n},, which is the possible speed of the vehicle. Thus, the algorithm only addresses the speed component and the direction of the vehicle keeps unchanged. The algorithm (additional details see Section 3.3.1) will be:

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

4.3.3 Interaction on intersections

The MAP method used here is similar to that of the last chapter (Chapter 3), but some further complexity is required. The shaded area (in Figure 4.2) is defined as *intersection area*. Roads 1 and 3 are two-lane major roads. The interaction area contains 68 cells (Appendix C), which represent two-dimensional cellular automata in the sense that vehicles may turn right or left on the intersection.

In the interaction area, update rules of the cellular automata are not universal, as vehicles that come from different streams and/or move in different directions observe different rules. Clearly the rules depend on the position and state of the given or occupied cell.

A vehicle will be given a *new* number before it enters the intersection area. The new number normally contains the following information.

- The state of a cell, i.e. if it is 0, the cell is vacant. If it non-zero, there is a vehicle in the cell.
- The direction that the vehicle will take (LT, ST or RT)
- The number of *cells* which need to be traversed before vehicle is out of the intersection area

The new number is different from the number that a vehicle acquires when it arrives at an entry road (Section 4.3.2).

For a vehicle from a major street arriving at the interaction area (see Appendix C), the new number will be *11*,*13* or *15* i.e. LT, ST or RT respectively. In other words, a vehicle from a major road needs to travel *11,13* or *15* cells to turn left, go straight through or turn right respectively. For a vehicle from a minor street, the new number will be *11,21* or *23*, i.e. LT, ST or RT respectively. For the number *21* and *23*, the numbers of cells that vehicles need to travel are only *11* and *13* cells respectively. The extra *10* just indicate that the vehicle from a minor-street needs to pass the central line, i.e. it is either ST or LT.

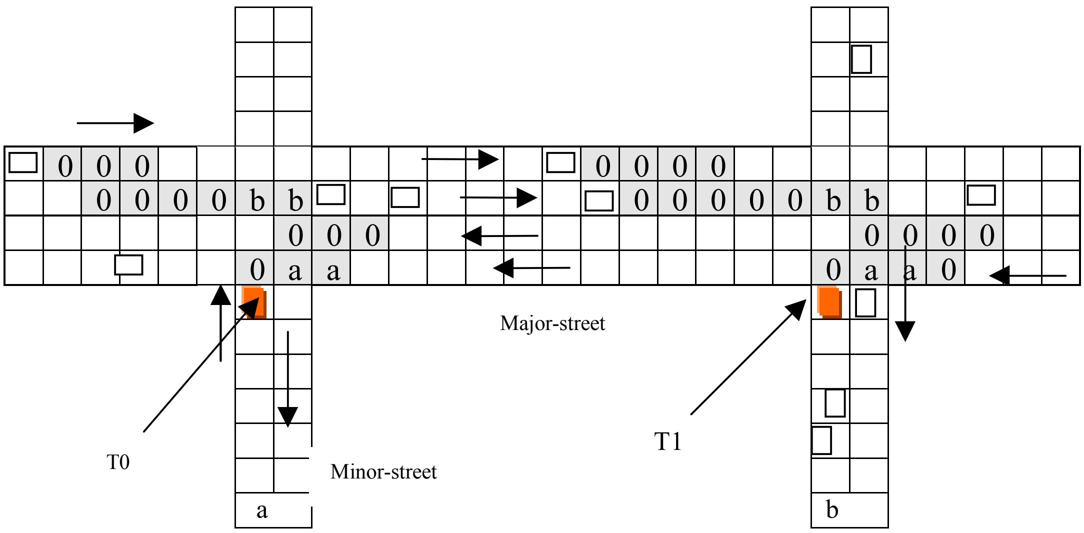

Figure 4.5. ST vehicle from a minor street: (a) rational behaviour and (b) conservative behaviour

Figures 4.5 to 4.10 indicate the conditions, under which a target vehicle (shaded) can move forward in the ***next*** time step, (except Figure 4.5 (d), which indicates the requirement for this time step, otherwise it is impossible to clearly indicate the conditions required for radical behaviour). The requirement for each shaded cell is indicated by 0, a, b or c. The notation of 0, a, b and c has the following meaning, and is as for Chapter 3 (Section 3.3.5).

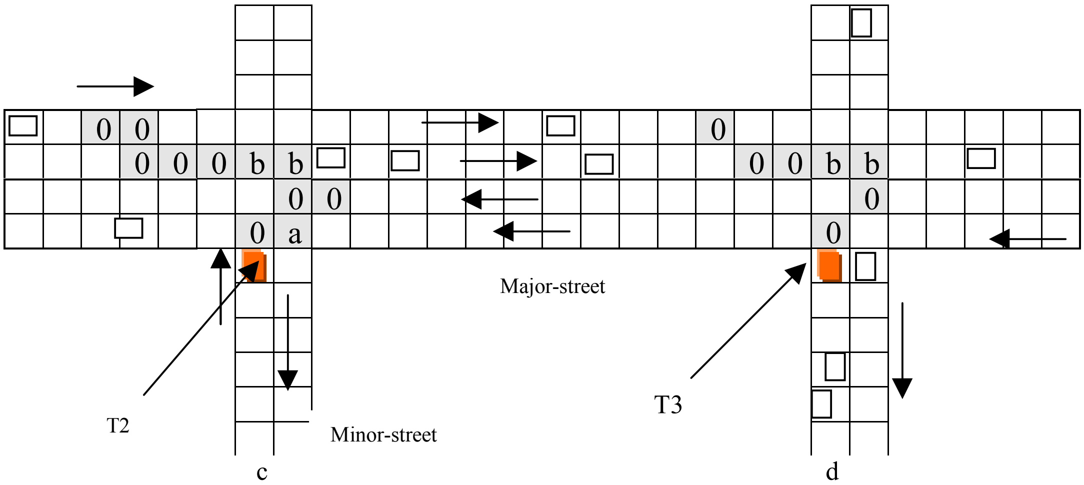

Figure 4.5. ST vehicle from a minor street: (c) urgent behaviour, and (d) radical behaviour

Figures 4.5 (a), (b) and (c) show the conditions (*MAPs*) required by a rational ST vehicle T0, conservative ST vehicle T1 and urgent ST vehicle T2 from a minor street to move forward in the ***next time step*** respectively. Figure 4.5 (d) shows the MAPs required by a radical ST vehicle T3 from a minor street to move forward in ***this time step*** respectively.

Comparing Figures 4.5 (a) to (d), the MAPs for 4 different categories of driver behaviour are clearly shown. Basically, the notion behind the figures is to describe the spatial conditions required. For conservative driver behaviour, MAP requires the largest space as shown in Figure 4.5 (b). MAP becomes one cell smaller in each stream when the driver behaviour category changes from conservative to rational, urgent and radical.

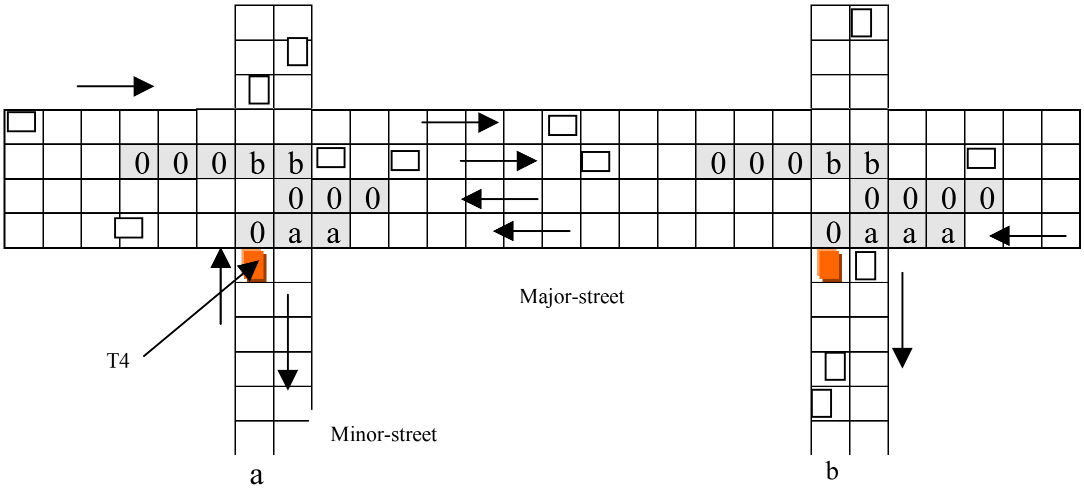

Figure 4.6. RT vehicle from a minor street: (a) rational behaviour and (b) conservative behaviour.

For Figures 4.6 - 4.10, only MAPs for rational and conservative behaviour are shown, but the MAPs for other driver categories can be obtained in the same way as for Figures 4.5 (c) and (d).

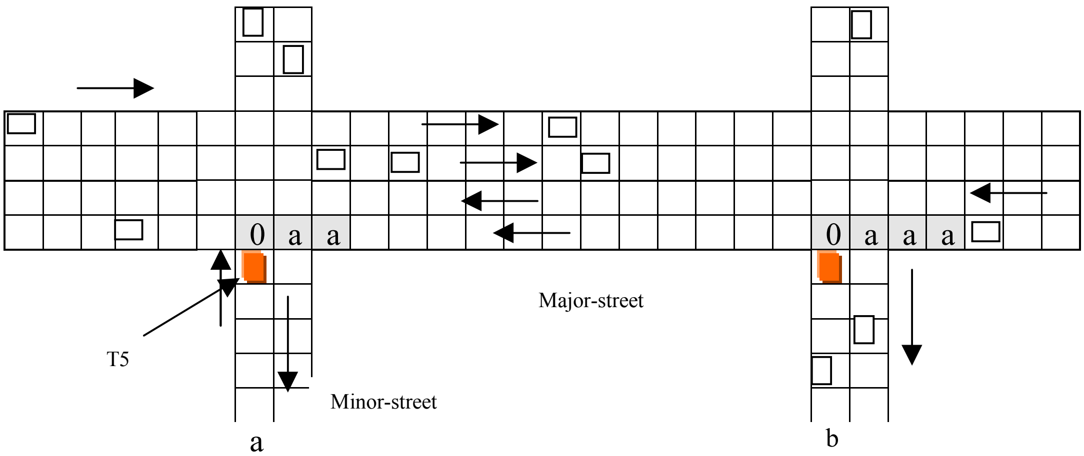

Figure 4.7. LT vehicle from a minor street: (a) rational behaviour and (b) conservative behaviour.

Figure 4.6 (a) shows the conditions required by a rational RT vehicle T4 from a minor street to move forward. Both RT and ST vehicles require the same conditions in the near side two lanes (streams 2 and 4), but require different conditions for streams 1 and 3.

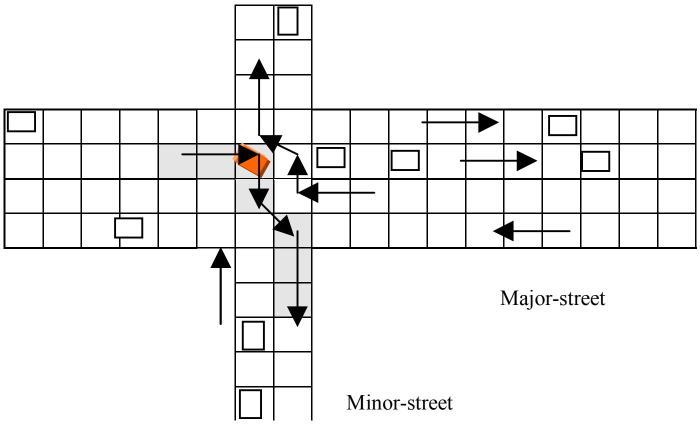

Figure 4.8. The path of a RT vehicle from a major street indicated by arrow

Figure 4.7 (a) shows the conditions required by a LT vehicle T5 from a minor street to move forward. Clearly the driver needs only to check the first lane, i.e. for vehicles from the left.

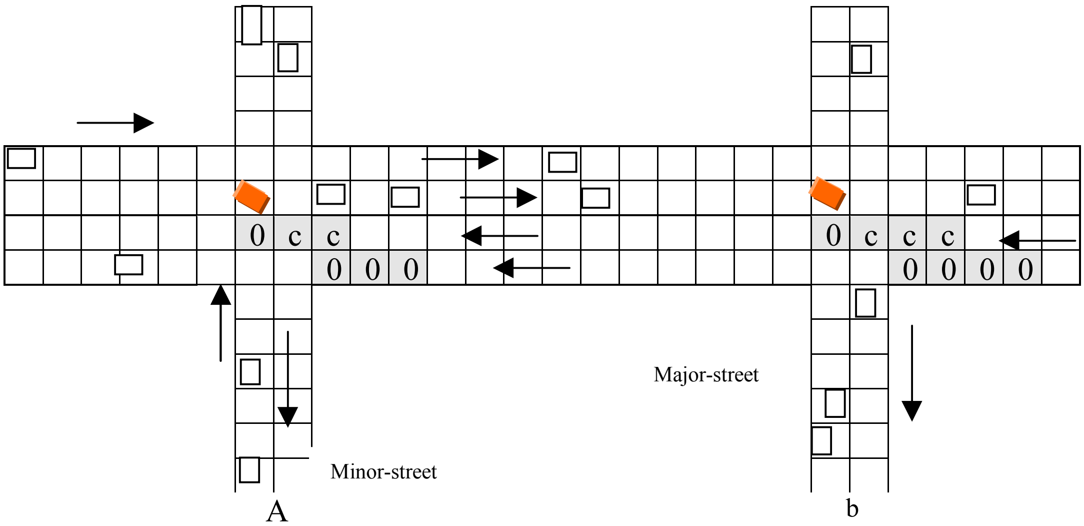

Figure 4.9. RT vehicle from a major street: (a) rational behaviour and (b) conservative behaviour.

Theoretically, a RT vehicle from the major road outer stream should not be blocked by the RT vehicle from the opposing major road outer stream. Therefore, the path that a right- turning vehicle uses is as shown in Figure 4.8. Also a RT vehicle should not be blocked by any vehicle from a minor street, as it has priority over vehicles from minor streets. The conditions for a major stream vehicle to turn right are shown in Figure 4.9.

4.4 Multilane Intersection Simulation

In this section, the capacity under different traffic conditions and operational properties of a 2-lane TWSC intersection will be investigate.

4.4.1 Major road right–turning capacity

The capacity for RT from a major road (road 1) has been studied for two situations. ***Situation 1:*** In the first situation, no vehicle interposes from left-lane of road 1, i.e. no vehicle comes from stream 1 (see Figure 4.2). Further the ST and LT vehicles from road 3 are on the left-lane (stream 2) only. RT vehicles from road 3 occupy the right-lane (stream 4) only. As no vehicle comes from stream 1 of road 1, the traffic flow of all RT vehicles from the right-lane of road 3 (stream 4) is free flowing. Based on the road rules, a RT vehicle from a major street should not be delayed by any vehicle from minor streets. Thus, the RT vehicles from the right-lane of road 1 (stream 3) can be possibly delayed only by ST and LT vehicles from road 3 (vehicles on the left-lane of road 3).

The capacity of RT from road 1 varies from the maximum of 1800 vph to 0 vph as the flow rate of the left-lane of road 3 changes from 0 vph to 1800 vph. The negative relationship between flow of RT vehicles from road 1 (stream 3) and flow of vehicles from the left-lane of road 3 is shown in Figure 4.10. Also, the sum of the two flows declines to the minimum when both flow rates are equal, i.e. where allowing vehicles from two conflicting direction-flows the opportunity to pass an intersection at the same time does not result in an increase in the total number of vehicles passing the intersection.

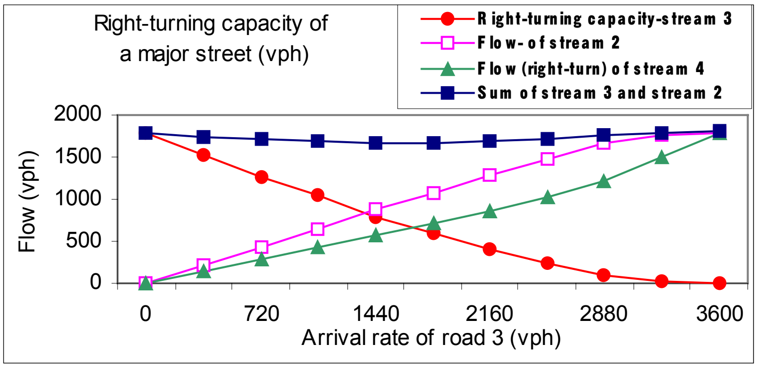

Fig 4.10.The relationship between the capacity of RT from road 1 and the flow rate of left-lane of road 3 (stream 2)

This provides useful insight on intersection operation performance, namely that throughput of an intersection can reach the maximum when no crossing flow is allowed (e.g. only straight vehicles from major roads). No matter how well different flows are scheduled (e.g. using different traffic light time schemes), throughput will always be less than the maximum once flows cross. While self-evident to some extent, this result provides internal validation of the model form and assumptions.

Although the RT capacity of road 1 (stream 3) decreases when the arrival rate of the left-lane of road 3 increases, the RT capacity of road 1 has not been found to vary when the percentage of LT vehicles on the left-lane of road 3 changes. The reason for this is that a RT vehicle from stream 3 needs to give way for both LT and ST vehicles of stream 2.

For a single lane TWSC intersection, the RT capacity of the major street depends on the arrival rate of the traffic from the opposing direction (see Chapter 3), while for the 2-lane TWSC intersection, the RT capacity depends only on *the non-RT proportion* from the opposing direction.

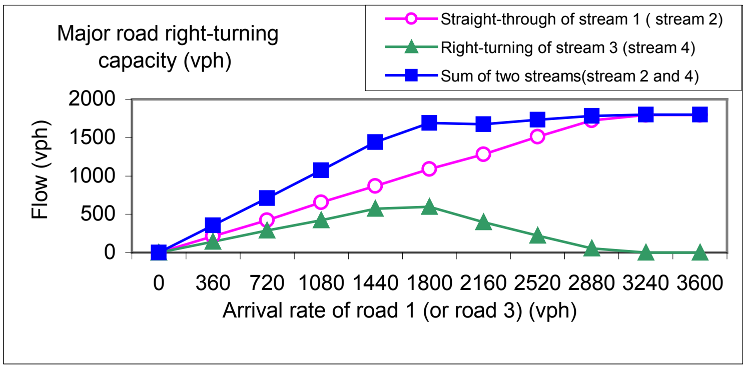

Fig 4.11.Major road RT capacity when both major roads have the same arrival rates and turning rates

Situation 2: We assume that both major roads (roads 1 and 3) have the same arrival rates (changing in the range of 0 to 3600 vph) and the same percentage of RT and ST rate (0.4:0.6). The flows of ST (and LT) vehicles from major street (streams 1 and 2) are not affected by RT vehicles from road 1 and 3 (streams 3 and 4). However, flows of streams 3 and 4 decrease dramatically when the arrival rates of roads 1 and 3 are above 1800vph. The relationship is shown in Fig 4.11, with the maximum flow of each road around 1800 vph.

4.4.2 Minor road left-turning capacity

Assuming that all vehicles from the minor street (road 4) are LT vehicles, the capacity of LT will be examined in this section. The interaction between the vehicles from road 4 (stream 5) and the left-lane of road 3 (stream 2) is similar to the interaction between the RT vehicles of road 1 (stream 3) and left-lane of road 3 (stream 2) (studied in section 4.4.1). The only difference is that vehicles from road 4 (stream 5) need to stop at the stop-line for at least 2 seconds before progressing.

The relationship between flow of road 4 (stream 5) and flow of the left-lane of road 3 (stream 2) is shown in Figure 4.12. The maximum capacity for LT from road 4 is less than 900 vph, which is only half of the maximum capacity of RT of road 1. The SSDT has a big impact on the capacity of the minor street (also see Section 4.4.5). In

Figure 4.12, there is one line (sum of stream 2 and 5) to indicate the sum of flow of the left-lane of road 3 and flow of road 4. The right-lane of road 3 (stream 4) is not included, as the LT vehicles from road 4 do not interact with it.

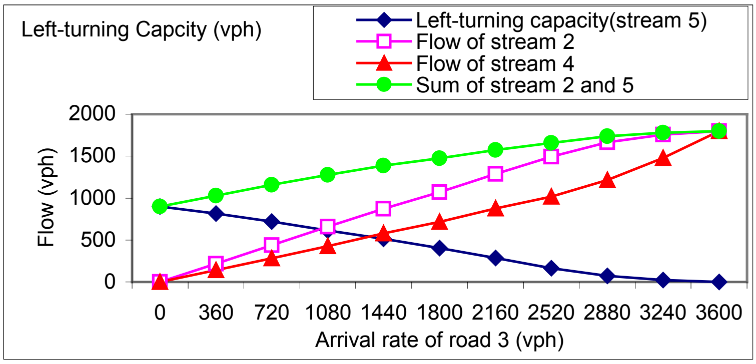

Figure 4.12. LT capacity for minor street

4.4.3 Minor road right-turning and straight-through capacity

In this section, the ability to RT and ST for vehicles from the minor street (road 4) is tested. Firstly, all vehicles from road 4 are assumed to be all RT vehicles. In order to show the relationship between the RT capacity of the minor street, the arrival rates of major roads (road 1 and 3) are assumed to be equal and both have STR: RTR = 0.6:0.4 (no LT vehicle).

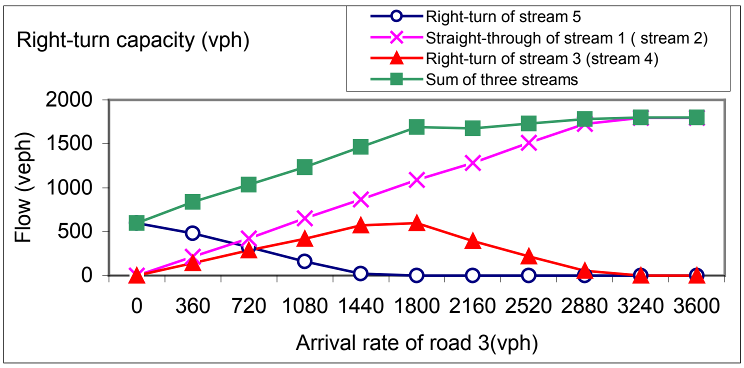

Fig 4.13 Minor street RT capacity

Figure 4.13 indicates that the RT capacity of the minor street varies with the arrival rates of roads 1 and 3. When arrival rates of roads 1 and 3 are greater than 1440vph, the RT capacity of road 4 is approximately zero, i.e. it is nearly impossible for a vehicle from minor street to move. Therefore, the RT movement from the minor street is effectively blocked even if the flow rates of the major streets are as low as 1440 vph.

Secondly, all such vehicles (from road 4) are assumed to be ST. Arrival rates of roads 1 and 3 are equal and both have STR: RTR = 0.6:0.4 (no left-turning vehicle), so that the relationship between arrival rates of major roads and ST capacity of the minor street can be assessed.

Figure 4.14 shows the capacity of ST vehicles from the minor street. Because ST vehicles from the minor streets require all four major streams to meet specific conditions (see Figure 4.5), whereas RT vehicles requires conditions only on three major streams (see Figure 4.6), the capacity of ST is further reduced when arrival rates of the major streams are the same. Again, the RT movement from the minor street is effectively blocked even if the arrival rates of major streets are as low as 1440 vph.

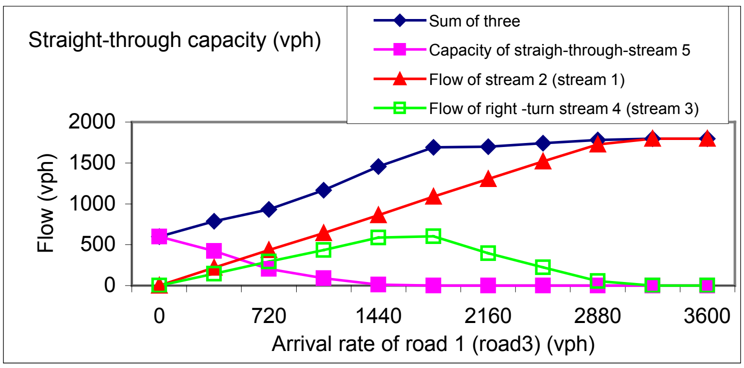

Figure 4.14. Minor street ST capacity

4.4.4 Turning rates and minor road capacity

In order to investigate the relationship between turning rates and minor road capacity, we assume that arrival rates and turning rates of the major roads are fixed.

Turning-rats of major roads both have STR: RTR = 0.6:0.4 (no LT vehicle). STR of minor roads (roads 2 and 4) are assumed to be equal and taken to be 0.2.

Table 4.1: Turning rates vs. minor rod capacity (vph)

LTR: RTR of minor roads	Flow rates of major roads (roads 1 and 3) (vph)			
	0	360	720	1080
0.2:0.6	666	522	340	162
0.4:0.4	753	585	408	203
0.6:0.2	858	689	503	270

In Table 4.1, flow rates of both major roads increase from 0 to 1080 vph, while LTR: RTR of minor roads change from 0.2:0.6, to 0.4: 0.4 and 0.6:0.2. The results indicate that capacity decreases when flow rates of major roads increase, for all flow rates and turning rates considered. Increase in LTR on minor roads obviously leads to capacity increase as we can see in each column.

4.4.5 The effects of stop-sign-delay-time (SSDT)

Stopping at a stop sign is a legal requirement, as well as good driving practice. The stop-sign-delay-time is thus the minimum delay that a vehicle may expect in order to follow road rules and make sure that it is safe to pass the stop line. Different drivers take different amounts of time to check all conflicting streams, to stop and then progress, so that a distribution for SSDT is realistic. Furthermore, individual driver SSDT is also likely to vary with road, traffic and weather conditions.

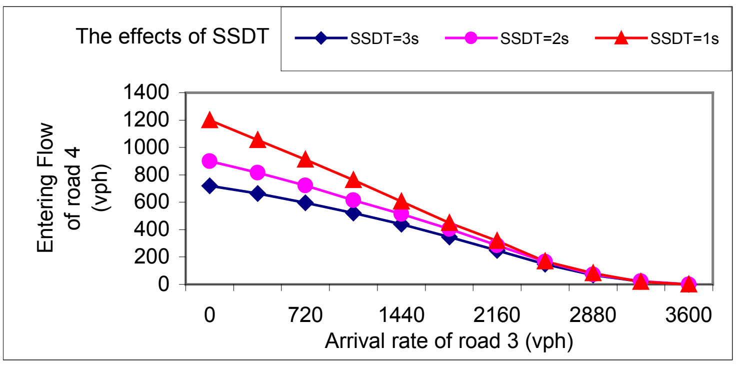

Figure 4.15. The effects of SSDT

Clearly SSTD also depends on the direction that the vehicle will take, as the numbers of streams to negotiate varies with different directions of movement. In this chapter, the SSDTs have been arbitrarily set as 2, 4 and 4 time-steps (seconds) for LT, ST and RT, but we have considered some modification in this section, for the following reasons. Original values are based on the same notion of SSDT as in Section 3.3.3, where SSDTs are assumed to be 2, 3 and 3 seconds for LT, ST and RT respectively for single-lane intersections. Comparing with single-lane intersections, the number of streams for RT and ST to negotiate increase in two-lane intersections, so that we assume that SSDT also increases one second. In this section, we observe the effect of changing SSDT on the capacity, (using the SSDT of LT vehicles from minor streets as an example).

From Figure 4.15, the shorter the SSDT, the higher the capacity of road 4. This effect decreases, however, as the arrival rate of road 3 increases, since vehicles on road 4 need to wait longer for a sufficient space to move. Thus the delay due to the stop sign becomes less significant. Similar results are observed for other directions of movements.

4.4.6 Overall operation of 2-lane TWSC intersection

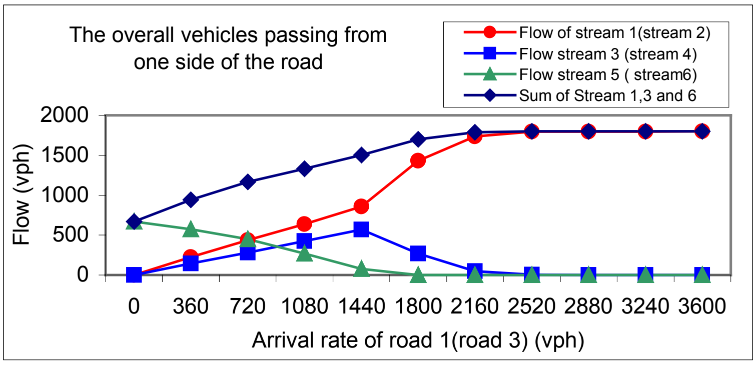

Figure 4.16. Vehicles from all directions. Flows change with the arrival rates of the major roads.

In the previous sections, several isolated scenarios have been considered. In this section, a somewhat more realistic scenario is studied. Vehicles are assumed to converge from all directions. Arrival rates of roads 1 and 3 are equal. Arrival rate of roads 2 and 4 are set to the maximum flow rate (1800 vph) that the single-lane road can manage. On both major roads, LTR: STR: RTR = 0.2:0.6:0.2. On both minor roads, LTR: STR: RTR = 0.4:0.2:0.4.

In Figure 4.16, the flow of minor roads (roads 2 and 4) are close to zero once arrival rates of roads 1 and 3 ≥ 1440 vph. When arrival rates of roads 1 and 3 are larger than 1440 vph, the flow rates of streams 1 and 2 increase drastically. At the same time, flow rates of streams 3 and 5 decrease, when arrival rates of major roads increase. These results are caused by the number of ST vehicles use the left-lane increase, as queue formation occur on the right-lanes of major roads when the flow rates are larger than 1440 vph.

Comparing to Figure 3.7, the two-lane TWSC intersection *does* improve the performance of the single-lane intersection in the sense of the mobility of minor road vehicles. However, the entry capacity is still very low when arrival rates of the major roads > 1080 vph. Also the RT capacity of the major road streams is approximately zero when arrival rates of major roads ≥ 2160 vph, i.e. the intersection actually only allow ST and LT vehicles to pass.

The throughput of 2-TWSC intersection reaches a maximum 3600vph (= the maximum of stream 1 + the maximum of stream 3) when ST and LT vehicles arrive at major street reach 1800 vph (= the maximum capacity of left-lane) on each major road.

4.4.7 Queue formation on major and minor roads

Under the more realistic conditions that vehicles come from all roads, LTR: STR: RTR = 0.2:0.6:0.2 on both major roads, and LTR: STR: RTR = 0.4:0.6:0.4 on both minor roads, the following is observed. For arrival rates of major roads taken to be 720 vph, the capacities of minor roads (roads 2 and 4) are around 448 vph. When the arrival rate of road 4 is slightly smaller (e.g. 432 vph) than the capacity, queue-length over a three-hour period can be observed to follow typical behaviour as shown in Figure 4.17. The queue-length can be in a very wide range (e.g. from 5 to 50 cells).

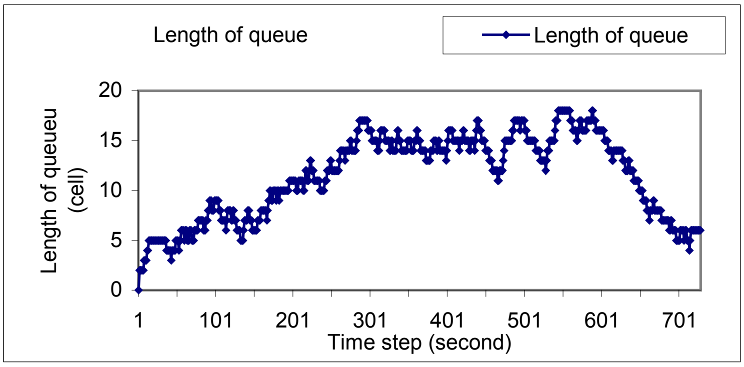

Figure 4.17. Queue-length of road 4

The reason for this is that the number of arrival vehicles is smaller than the capacity, so that the queue may form only temporarily due to the random process of arrival and MAP availability. Similar results are also found in short running time (e.g.7200 seconds) when arrival rate is slightly larger than the capacity (e.g. 464 vph), but in long running time (e.g. over 2 hours) the queue-length will eventually reach the maximum length of the road. Again, the reason for this fluctuation of queue length is due to the random process of arrival and MAP availability. However, because the number of arrival vehicles is greater than the capacity, the queue will eventually reach the maximum road length situation.

When a wide range of arrival rates is studied, the queue formation on major (RT lanes) or minor roads can normally be summarised as follows:

- If arrival rate is much larger than (>>) the capacity, queue-length increases drastically and rapidly reaches the maximum length of the road.
- If the arrival rate is much less than (<<) the capacity, queuing is rare.
- If the arrival rate = the capacity, the queue will reach the maximum length of road sooner or later.
- If the arrival rate is slightly lower than the maximum capacity, queue-length will fluctuate from 0 to some length. It may reach the maximum length of the road if the arrival rate is relatively close to the capacity, but usually only after a relatively long period of time.

4.4.8 Driver behaviour

We now consider the effect of driver behaviour for vehicles coming from all directions. Arrival rates of roads 1 and 3 are taken to be equal, while arrival rate of roads 2 and 4 are taken to be equal to the maximum flow rate (1800 vph) that the single-lane road can manage. On both major roads, LTR: STR: RTR = 0.2:0.6:0.2. On both minor roads, LTR: STR: RTR = 0.4:0.6:0.4.

The following deterministic situation is considered: we assume that all drivers are in one of the following four categories: conservative, rational, urgent or radical. This will result in four different road capacities for one set of arrival rates of major roads. If the arrival rates of major roads change, the four capacities will change as a consequence.

If arrival rates of major roads = 0, it is obviously that the road 4 capacities of the four categories are equal (see the second column of Table 4.2), as no vehicle is on the major road and the vehicles from road 4 do not need to give way. When the arrival rates of the major roads > 0, for the first three categories with the same arrival rates, the capacities will increase *pro rata*, i.e. capacity of conservative < capacity of rational <capacity of urgent. Clearly the more impatient drivers will exploit the more opportunities.

However, the most extreme case is that of the radical driver. The capacity of road 4 is highest when the arrival rates of major roads (roads 1 and 3) are 360 vph, but radical driver behaviour can causes gridlock on the intersection. If all drivers from minor roads seize any space onto the intersection (not observing give-way rules), blockage of vehicles from major roads will occur. Particularly, when the arrival rates of major roads are high, the vehicles from major roads also block the vehicles from minor roads. Gridlock is unavoidable.

Our model illustrates possible causes of gridlock. We find that occurrence of gridlock may need two conditions: (i) traffic flow on major roads heavy and (ii) drivers on minor roads failing to observe the rules.

Table 4.2. Entry capacity of road 4 (vph)

Driver Behaviour	Arrival rate of road 1 (and 3)						
	0	360	720	1080	1440	1800	2160
Conservative	750	560	47	1	0	0	0
Rational	753	574	448	269	78	0	0
Urgent	749	660	552	399	193	7	0
Radical	748	698	Gridlock	Gridlock	gridlock	gridlock	gridlock

Clearly, a comprehensive sensitivity analysis is needed to determine what combinations can trigger gridlock. Initial analysis has shown that for a combination of driver behaviour with high probability of radical drivers gridlock may occur very quickly (within one hour), or take a relatively long time (e.g. over 5 hours. This is estimated, as the model can not show exactly when gridlock exactly occurs. However, the time can be estimated based on how many vehicles passed through, over 10 hours say) for the same flow rate of major roads. Consequently, the capacity can be varied in a wide range depending on when gridlock occurs. When gridlock does not occur, an approximate linear relationship, which is similar to Section 3.4.4, is observed between the capacities and driver behaviour distribution. However, further sensitivity analysis is also needed in order to investigate the effect of combination of different types of driver behaviour, which has not as yet been done in this research.

In the real world, drivers stopped on the intersection can co-operate and free gridlock by self-organisation (not by road rules). Our model does not currently include the process of releasing gridlock. It can, however, be incorporated into the present model in the future. One possible way to release gridlock is to force one vehicle to change direction (e.g. a straight-through or right-turning vehicle is forced to turn left) and release one cell on the intersection. It nevertheless typically takes a very long time for traffic flow to recover, so that estimates of time needed to release gridlock can be very variable.

4.5 Signalised Intersections

4.5.1 Background

Traffic light controlled intersections are an alternative to TWSC intersections. In a city, traffic lights have been considered as the main method of traffic control, despite many other methods, such as signs (free-standing or on the road surface), radio broadcasting and manual control-points, etc.

Traffic light control systems have developed from fixed time control systems into real-time adaptive control systems. State-of-the-art traffic signal-control systems are capable of dynamically modifying signal timings in response to changing traffic demand (Mirchandani and Head 2001, Brockfeld, et al. 2002). Two centralised adaptive control systems (SCATS and SCOOT) are used in Ireland. SCATS (Sydney Coordinated Adaptive Traffic System) is used in Dublin City, while SCOOT (Split, Cycle, Offset Optimisation Technique) is currently used in Cork (Traffic Information 2003).

There are some common methods used in traffic light design at the network level, such as traffic light co-ordination, interconnection and synchronisation (Traffic Control Systems Handbook 1996, Office of Technology Applications (OTA) 2001). Traffic lights in the city are normally closely spaced, typically < 1km apart. They are often co-ordinated in order to minimise delays and to move large volumes or "platoons" of traffic in one movement along the main road (Seattle Department of Transportation 2002). However this co-ordination is not easy to achieve due to the differences in distance between traffic lights, volume of traffic, speeds, and amounts of green time required for each intersection. Consequently, it is very difficult to obtain perfect co-ordination for all directions.

Co-ordination is achieved by connecting all traffic lights to form a communication network (Office of Technology Applications (OTA) 2001). Interconnection allows traffic lights to share traffic control information and to be simultaneously programmed and consistently work together. Once co-ordination is established the traffic lights can be synchronised.

In order to establish a common green and red light cycle length along a major road, traffic light synchronisation is used to activate signals together (Seattle Department of Transportation 2002). All intersections in the co-ordinated system have the same cycle length. Traffic lights may also be synchronised over the entire traffic control system (whole urban network), and, it is believed, permit more efficient mobility (Leonard and Rodegerdts 1998).

Furthermore, in order to achieve better performance over whole networks, the following facilities are normally also put into place: Traffic Detection Devices, Intelligent Transportation Systems (ITS) and Traffic Management Centre (details see Seattle Department of Transportation 2002).

In order to compare 2-lane traffic light controlled intersections with the 2-lane TWSC intersections, we primarily focus on modelling with the traffic light controlled intersection in Section 4.5. Although modelling signalised controlled traffic flow is not the main task of our research, it can help us to a further understanding of unsignalised traffic flow and especially can offer insight on aspects of traffic control at local level. In particular, we are interested in the comparison between the function of controlled and uncontrolled intersections and the effect on flow dynamics.

Traffic light control is based on understanding how the cycle of traffic lights affect the mobility of traffic flow, i.e. the relationship between the volume of vehicles and time-cycle setting. This is usually an inexact task, as road users know. We wish to explore the nature of the signalised intersection improvement and /or dis-improvement on mobility of traffic flow.

In an intersection, the duration of green and red lights for each direction is different and depends on the signal timing policies (including minimizing delay, minimizing stops, minimizing fuel consumption, maximizing coordination band width, a "baseline" policy) as well as the traffic flow patterns, (Leonard and Rodegerdts 1998).

When traffic loads are relatively balanced in each direction, the duration of “reds" and "greens" are balanced in all directions. When the traffic flow is heavier in one direction, the traffic lights are co-ordinated to favour the highest volume of vehicles. Many other policies may of course be considered, such as a favoured policy for public transportation. Settings may also change at different times of day. In the morning e.g. traffic flow is relatively heavy towards the central business district (CBD) in a city, such as Dublin, while the situation is reversed in the evening. The settings need to reflect the differences.

In our model the duration of yellow (amber) light is three seconds. In the U.K. and Ireland, the duration of yellow light is legally required to be three seconds' duration. However, the duration of yellow (amber) in U.S. is 3-6 seconds depending on speed limits according to the new version of the *Manual on Uniform Traffic Control Devices* or MUTCD (Federal Highway Admission, U.S. Department of Transportation) recommendation. An historical review of the investigation and practice with respect to the United States process of selecting an interval of yellow light is presented by Liu et al. (1996). Differences stem from regulations of how yellow lights are to be used. In the U.S., drivers can drive onto the intersection when lights are yellow, if they can clear the intersection before the light turns to red. In Ireland and the U.K., drivers should stop on yellow if they can stop behind the stop line safely.

The above review is not a comprehensive review on traffic-light controlled intersection. It would be needed if our research was on traffic light timing strategies or optimising traffic lights. As our research is primarily on unsignalised traffic, we present here just a preliminary examination of signalised control. The purpose is to flag some of the wider issues, (Hounsell and Salter 1996).

4.5.2 Methodology

We use our basic CA model and enhance this to incorporate traffic light conditions. Vehicles can not enter the intersection unless the traffic light is green in their direction. The intersection is an area of 4 x 4 (cells) square, so that a ST or RT vehicle on the intersection can leave the intersection in 3 seconds if traffic lights turn to yellow.

In other words, the area is exactly defined to reflect complete movement for any vehicle that enters the intersection before the lights change to yellow and before the next change.

Time settings and throughput: On each entrance road of the intersection, three traffic lights are employed, LT, ST and RT, and all lights have three colours (the RT or LT lights may be just an arrow). In reality of course, there may only be lights for RT and ST. Some intersections do not control LT vehicles (i.e. LT controlled by priority rules), so that more LT vehicles may pass the intersection compared to using LT lights (LT vehicles may be required to give-way to pedestrians).

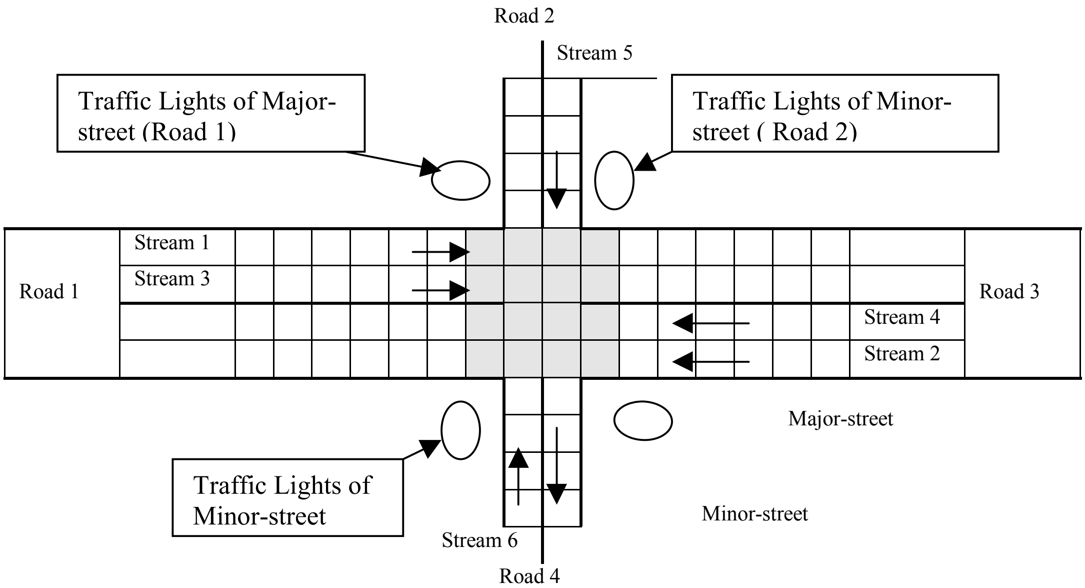

Figure 4.18 An intersection with traffic lights

Theoretically, there is no interaction between different directions of the traffic flow, as traffic lights are designed to avoid conflict between different directions. Therefore, the capacities are directly related to the traffic light settings. We can assume that the cycle is around 2 minutes, (cycles generally range from about one minute to two minutes, where a 2-minute cycle is slightly longer than normal), in order to study the effects of different green and red settings. At a simple intersection, a cycle might include 48 seconds of green light for traffic on major roads (1 or 3), followed by 3 seconds of yellow light and 61 seconds of red light. One time-setting example can be seen in Appendix B.

Table 4.3. The intervals of green lights (in seconds) vs. throughputs

Major street	66	63	60	57	54	51	48	45	42	39	36	33
Minor street	3	6	9	12	15	18	21	24	27	30	33	36
Throughput (vph)	3444	3330	3300	3199	3192	3074	3051	2953	2919	2814	2795	2685

During the 2 minutes, the intervals of green light for a major and minor road are listed in Table 4.3. The relationship between different settings and throughputs is also shown. Basically, throughput of the intersection increases as the green light periods increase for major roads. The reason for this is that the green lights for major roads allow traffic flow of two lanes to pass through the intersection at the same time, whereas green lights for minor roads only allow single lane traffic flow to pass through. However, this increase in throughput is achieved by sacrificing the mobility of minor roads.

4.5.3 Signalised Intersection Simulation

In Figure 4.19, flows of left-lane (stream 1) and right-lane (stream 3) of the major road are shown. The flows are approximately linearly decreasing as the duration of green lights for minor road increase. Furthermore, the flow of the minor road (stream 6) increases linearly.

As long as the duration of green lights for the minor road >0, the maximum throughput of 2-TLC intersection < 3500 vph, whereas throughput for 2-TWSC intersection is around 3600 vph (Section 4.4.6). Traffic lights clearly cause throughput decrease for two reasons, i.e.

- The yellow light period, for which no vehicle moves onto the intersection
- The green lights not being fully utilised, as there are too few vehicles in the permitted direction. This is the reason why adaptive controls systems are the focus of recent efforts on control.

Comparing Figures 4.16 and 4.19 (the two traffic configurations-signalised and unsignalised), we can see that traffic lights increase throughput when the same number of vehicles from the minor roads are able to pass through the intersection. For the TWSC intersection, when the arrival rates of major roads ≥ 1440 vph, the entry capacities of the minor roads are nearly zero. For signalised intersections, when similar numbers of vehicles, ~1440 vph pass through the intersection from each major road, the capacities of minor roads are around 195 vph.

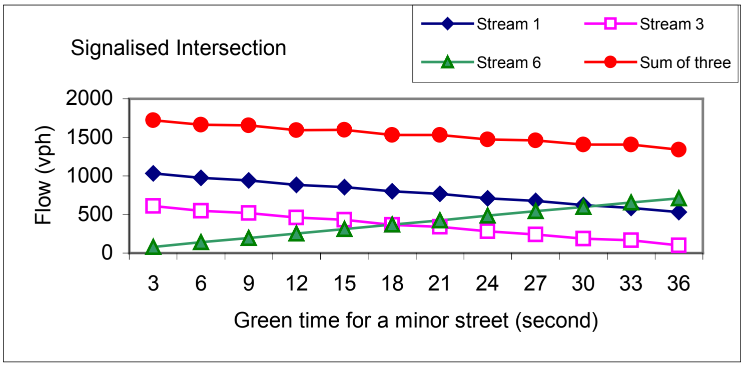

Figure 4.19. Flow of different streams varies with the duration of green light of minor-street

With the same capacity of the minor-road, signalised intersections have better performance compared to TWSC intersections, providing that there are enough vehicles on all roads to utilise the green light periods. Obviously, if there are not enough vehicles to fully utilise the green light periods on one or more roads, traffic lights can not enhance the intersection performance. In this case, a TWSC intersection may have better performance than a traffic-light controlled intersection. Clearly a signalised intersection can give a chance to streams that have been blocked under the priority rules, but can also block vehicles on the major streams when there is no need to do so (for example, when there are not enough vehicles from the minor roads). Therefore, whether an intersection should be controlled depends on traffic situations.

4.6 Summary

In this chapter, we proposed a new model to study traffic flow at a two-lane two-way stop-controlled (2-TWSC) intersection. A model of dynamic lane-allocation process has also been developed. The vehicle allocation process depends on the direction of the vehicle and queue length on each lane of the major road. An algorithm to update position on a two-lane road was built, in which a vehicle can have multiple speeds and retain its destination attributes.

Interaction on the intersection has been modelled using the MAP method, but further complexity is required. Using the MAP approach, capacities have been obtained for different road manoeuvres.

The two-lane TWSC intersection *does* improve the performance of single-lane intersection in the sense of mobility on the minor roads. However, the capacity of the minor roads is close to zero when arrival rates of the major roads ≥ 1440 vph. Also the RT capacity of the major street is approximately zero when arrival rates on the major roads ≥ 2160 vph.

The effects of SSDT also are analysed. Not surprisingly, the shorter the SSDT, the higher the capacity of the minor road. This effect decreases, however, as the arrival rates of the major road increase.

When a wide range of arrival rates are studied, the queue formation on major (RT only) or minor streets can be summarised as in Section 4.4.7. The key is clearly that if arrival rate ≥ the capacity, queue-length will reaches the maximum length of the road.

Four categories (conservative, rational, urgent and radical) of driver behaviour have also been studied and significantly affect results. We find that the occurrence of gridlock requires that that major road traffic is heavy and that drivers on the minor roads "bend" the rules.

Finally, a traffic flow model at a signalised intersection has been built in order to compare with 2-TWSC intersection. With different green light settings for the major and minor roads, different flow patterns were observed. Providing that there are enough vehicles on all roads to utilise the green light periods, and for the same minor-road capacity, traffic lights are found to improve overall performance measured in throughput. Signalised intersections facilitate cross flow in that they are able to provide the chance for the streams that are blocked under the priority rules, but at the expense of blocking vehicles on the major streams.

Chapter 5

Single-lane Roundabouts

5.1 Introduction

Roundabouts are an important part of urban networks, with some controlled by traffic lights and some controlled by rules of the road. Roundabout operation is also governed by the **offside-priority rule**, by which a vehicle entering gives way to one already on the roundabout. Roundabouts connect urban (and inter-urban) streets, with some having four or more entrance/exit points.

The main feature of roundabouts is that they transfer a complicated intersection into several simple T-intersections and consequently improve road safety. According to the Norwegian Road Safety Handbook (Elvik et al. 1997, Hyden et al. 2000), there are five main advantages in using roundabouts to improve traffic safety:

- The number of conflict points in the traffic flow is decreased;
- The "Give way rules" are imposed;
- All traffic inside the roundabout comes *from one direction*;
- Right turns for opposing traffic are excluded (for left-side driving);
- The speed is reduced.

Studies also indicate that acceptable safety levels can be fully reached only if vehicle speed is lower than approximately 40 km/h (DIB 1998).

The mobility of vehicles at roundabouts is an important issue related to the global mobility and capacity of urban networks. One experimental study (Hyden et al. 2000) has indicated that time taken to pass the network point (i) increases when roundabouts are used to replace unsignalised intersections and (ii) decreases when they replace signalised intersections. In the light of our discussion on the findings of the previous chapter, this is an issue of particular interest. Consequently, the focus of the work presented in this and the following chapter related to the theoretical analysis of mobility and time-delay in different geometrical road features, specifically symmetric and asymmetric roundabouts.

Time taken at intersections and roundabouts contributes significantly to travel time and route choices in urban networks. Road users at roundabouts also interact with each other, and time delays are different for different individuals. Even at the aggregate level, network flow patterns depend on delays at intersections and roundabouts. In particular, for urban networks under saturated conditions, a large part of total travel time is due to queueing delays (Queloz 1995).

In the following, a cellular automata (CA) model is used to simulate a single-lane roundabout operating under the offside-priority rule. Three aspects of roundabout performance in particular have been studied. The first looks at overall throughput, (the number of vehicles, which navigate the roundabout in a given time), for different geometries, arrival and turning rates. The second investigates changes in queue-length, delay-time and vehicle density for an individual road. The third considers the impact of driver behaviour on throughput and the performance of the roundabout.

5.2 Background

Several attempts to simulate roundabout operations exist, mostly based on entry capacity models. The **entry capacity (capacity)** of a roundabout is the number of **v**ehicles pass through *an entrance* **p**er unit of time (normally an **h**our—**vph**), which is *different from* throughput. **Throughput** is the number of vehicles that pass through the *roundabout* in a given time. Hagring (1996) refers to ten different models, which can generally be classified into two groups. The first consists of linear regression models developed by empirical methods. The second group that of gap-acceptance models developed using analytical methods has been discussed earlier in relation to intersection movements (Chapter 3). Gap-acceptance models have also been used, not only to model the entry capacity but also in studying the queue-length and delays at a roundabout.

Liner regression models use linear approximations to determine the relationship between entry capacity and **circulating flow** (the total volume in a given period of time on the roundabout immediately prior to an entrance) for a single-lane roundabout, (Kimber 1980, Guichet 1997 and Brilon et al.1997). Kimber's equation (see Equation 5.1 at the end of this section) assumes that capacity has no relationship with the size of the central island (Chin 1983). Therefore, Kimber's equation could be used for both large and small roundabouts of any shape.

Kimber's model has been used both in the software RODEL (developed for the evaluation and design of roundabouts and **RO**undabout **DEL**ay) and ARCADY (**A**ssessment of **R**oundabout **CA**pacity and **D**ela**Y**) (Semmens 1985 and Brown 1995). It is widely used in the UK and Germany. The latest research on roundabout entry capacity in Germany shows that a linear rather than an exponential function (developed by Siegloch 1973) is also in better agreement with the observed data (Brilon et al. 1997).

Empirical capacity models have some advantages, one of which is clearly that there is no need to describe or to understand driver behaviour, as the data are from the real world, which has already taken many such factors that influence capacity into account. There are some obvious drawbacks, e.g. the significant amount of data that has to be collected to ensure reliability of results. Entry data have to be collected at saturation (or at capacity) level.

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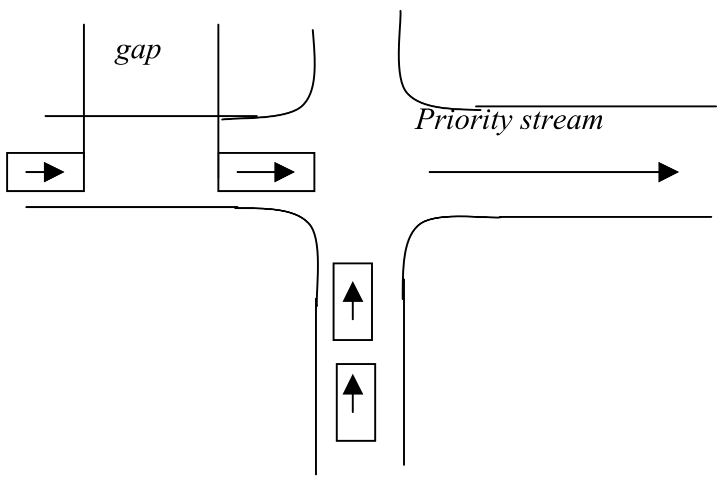


Figure 5.1: Illustration of two-stream intersection

The gap-acceptance model (shown in Figure 5.1) was developed originally for "priority rule" intersection (i.e. without traffic lights) and was based on Tanner's capacity model (1962, see Equation 5.2 at the end of this section). The basic assumption of this model is that the driver will enter the intersection when a safe opportunity or "gap" in the traffic presents itself; (for further discussion, see Chapter 3).

Troutbeck (1988 and 1991) proposed a two-stage theory to modified Tanner's model. He indicated that assumptions that T and T_0 were constant (and that this headway distribution of priority stream was random) were not realistic. He believed that vehicles travel in two stages, the "bunched vehicle" stage and free vehicle stage. In the bunched vehicle stage, vehicles are "following" leading-vehicles. In the free vehicle stage, however, these vehicles travel without interaction with the vehicle ahead. Taekratok (1998) modified Tanner's equation using Cowan's M3 headway distribution model (Cowan 1975 and Section 3.2).

Troutbeck (1990) conducted a study of driver interactions at roundabouts in Australia. His study supports his assumption that traffic streams influence each other. Two critical points are:

- Priority sharing occurs at the entrance of the roundabout. Circulating vehicles may give way to entering vehicles deliberately. This appeared to lead both to a reduced critical gap and average follow-on time for entering vehicles.
- In general, entering vehicles give way to all circulating vehicles. Entering drivers were often unsure whether a circulating driver on their left intended to leave at the exit before theirs or travel across their paths.

Additionally, Taekratok's model (Taekratok 1998) has been adjusted based on data observed in Australia and software developed as aaSIDRA (**aa**Traffic **S**ignalised & unsignalised **I**ntersection **D**esign and **R**esearch **A**id, Akcelik 1997 and 1998). The gap-acceptance model is also extensively used in the USA, recent examples include (Kyte 1997).

In brief, the models revised above have the form:

Kimber's capacity model is:

$$Q_e = F - f_c\, Q_c \qquad (5.1)$$

where Q_e =entering capacity (vph).

Q_c = circulating flow (vph), (flow coming from the left).

F, f_c parameters defined by roundabout geometry.

Tanner's equation is

$$Q_e = Q_c\,(1- \Delta\, Q_c)\, e^{Qc\,(T- \Delta)} /(1- e^{Qc\,T}\ ) \qquad (5.2)$$

where Q_e = Entering capacity (veh/sec)

Q_c = Circulating flow (veh/sec)

T = Critical gap

T_0 = Follow-up time

Δ = The minimum headway

The modified gap-acceptance model is

$$Q_e = 3600Q_c\,(1- \theta)\, e^{-\lambda\,(T- \Delta)} /(1- e^{-\lambda\, T_0}\ ) \qquad (5.3)$$

where

Q_e = Entering Capacity (veh/h)

Q_c = Circulating Flow (veh/sec)

θ = The proportion of bunched vehicles

Δ = The minimum headway in the circulating streams, and these are 1 second for multilane and 2 second for single lane

T = The critical gap

T_o = The follow-up time

λ = Decay parameters $=(1- \theta\,)\, Q_c\, /(1- \Delta\, Q_c\,)$

These essentially represent hierarchical model development, with complexly increasing. We now discuss our approach to modelling roundabout manoeuvres, which relies on multi-valued space criteria (as described in MAP models in previous chapters).

5.3 Methodology

We use a CA ring to represent a single-lane roundabout, stimulated by the work of Chopard (1998), who used this idea to simulate intersections without traffic lights.

We develop *a multi-state CA ring* in order to characterise vehicle destinations. The state in each cell has three meanings. If zero ($C = 0$), this means that there is no vehicle in this cell. If larger than zero ($C > 0$), it means that there is a vehicle in this cell. The actual value indicates how many cells the car needs to traverse to arrive at the destined exit. This approach extends to multilane roundabouts in Chapter 6.

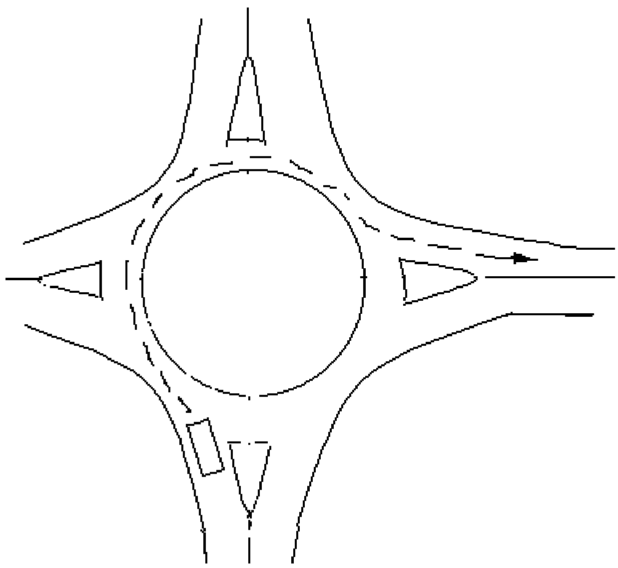


Figure 5.2: A single-lane roundabout: flow pattern

The number of cells in the ring is determined by the real dimension of the roundabout, which, if known, gives the number of cells in each ring. The overall requirement for the model (or program) is obviously to be flexible enough to allow size to be varied.

A typical roundabout is shown in Figure 5.2. Four roads connect to the roundabout, where each road has two directions for traffic flow. The roundabout has four single-lane entrances/exits for this example. Movement for each lane is handled by one three-speed deterministic CA model (as in Chapter 3). The roundabout is represented also by a multi-state CA ring. The roundabout system contains eight three-speed deterministic CA and one multi-state CA ring shown in Figure 5.3.

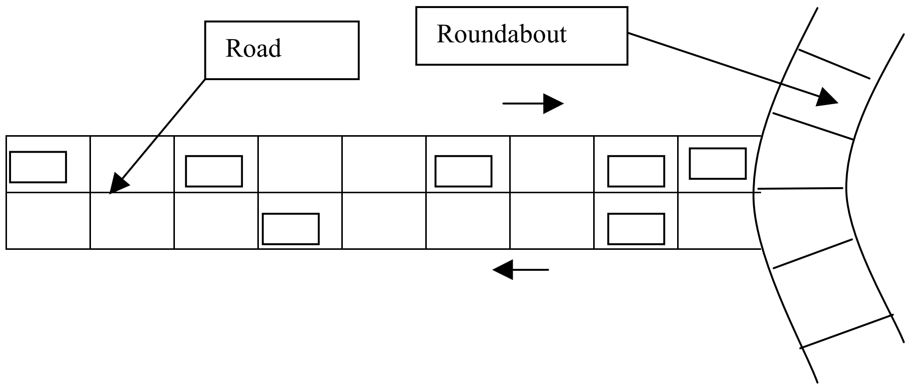


Figure 5. 3: A road and its entrance to a roundabout

5.3.1 Driver behaviour at entrance

Under the offside-priority rule, the vehicles waiting at a roundabout entrance need to give way to the vehicles on the roundabout. Drivers need to determine how much space on the roundabout is sufficient for them to drive to the required position and to gain enough speed so that their car will not obstruct an oncoming vehicle. Determination of the opportunity to drive onto the roundabout is a complicated decision-making process.

Factors that influence the driver's decision include driver skills, the weather, the car performance, motivation of travel etc and may vary for each individual driver. However the important common factor is the space available on the roundabout.

In this model, we use the space available on a roundabout as the only parameter to describe driver behaviour. Similar to intersection models, the **optimum condition** for a vehicle to move onto the roundabout is that this space is just enough for the vehicle to enter the roundabout without interrupting an oncoming vehicle. However, an individual driver's own space criterion of entry to a roundabout may differ from the optimum condition. Thus, driver behaviour can be categorised as *conservative*, *rational, urgent* and *radical* and considered in addition to space conditions.

Rational decision-making is that which is based on the optimum condition being met, whereas **conservative behaviour** implies delayed entry, even when the space available on a roundabout is larger than optimum. **Urgent behaviour** is a rushed entry, when the space is just smaller than the optimum. The action of an urgent driver will slightly block the oncoming vehicle (to pause for one time-step), but the entry vehicle itself can move on smoothly. By contrast, **radical behaviour** occurs when the driver will squeeze onto the roundabout, even when the space is far less than optimum. The result is that the entering vehicle not only blocks the oncoming vehicle (causing a pause for two time-steps), but also a further pause for one time-step (to avoid running into the vehicle in front).

Both urgent and radical behaviour may result in blocking an oncoming car, which should not happen according to the offside-priority rule. Consequently, radical behaviour may lead to congestion and a breakdown of free flow.

The distribution of driver behaviour is therefore expressed in four probabilities as previous chapters (Chapters 3 and 4), i.e. probability of conservative entry (P_{co}), probability of rational (P_{ra}), probability of urgent (P_{ur}) and probability of radical (P_{rad}) behaviours (with sum =1).

5.3.2 Entering the Roundabout

The optimum condition for a vehicle to drive onto a roundabout is that there are three sequential vacant cells available on the roundabout. If the condition is met, we can put an extra vehicle between two vehicles without causing interruption of flow.

A driver observing the optimum condition is behaving rationally. A driver who takes *four* vacant cells or more is favouring conservative behaviour. By contrast, moving onto the roundabout when only *one* or *two* cells are vacant displays radical or urgent behaviour respectively.

Simulation conditions for *rational behaviour* are as follows:

- Find the number of vacant cells of the CA ring, which is to the right of an entrance.
- If the number of free cells ≥ 3, the vehicle waiting at the entrance may drive onto the roundabout.
- If there are *two* vacant cells in *three* sequential cells and *the third one* is occupied by a vehicle that will exit from the roundabout before this entrance, the waiting vehicle can also enter roundabout.

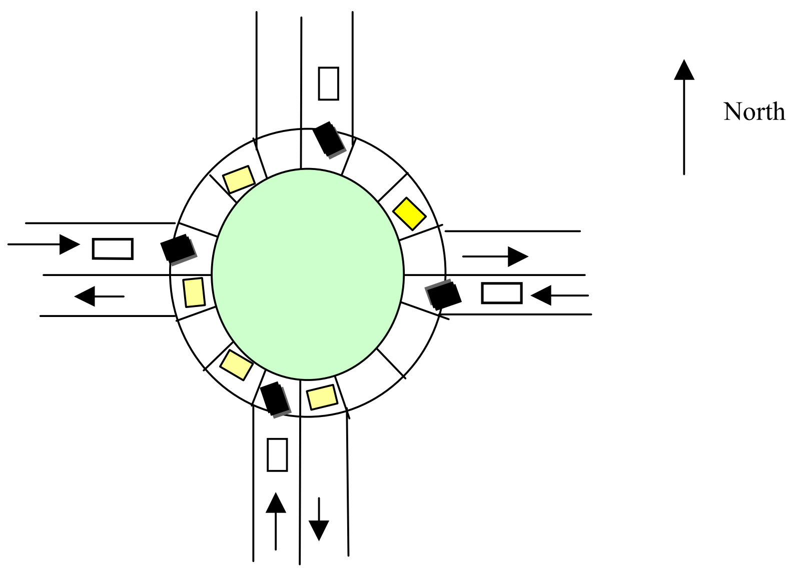


Figure 5.4: An illustration of driver behaviour

Figure 5.4 illustrates four different behaviours. The dark vehicle from *north* is entering the roundabout with rational behaviour, as there are *three* vacant cells between two light colour vehicles. The vehicle from the *east* is entering the roundabout with conservative behaviour, as there are *four* vacant cells between two light colour vehicles. By contrast, the vehicles from the west and south are entering the roundabout with urgent and radical behaviour respectively.

5.3.3 Predetermined exit before entering the roundabout

Drivers clearly have their own destination in mind, so that they would make decisions on which exit is appropriate before entering. Characterising a given exit for each vehicle before entry is clearly more realistic, than assuming that such a decision is made once entry is effected.

The approach used is to characterise each car by randomly giving each car a different number. The number is equal to the number of the cells that a car needs to pass to arrive at its destination exit. For instance, a four-road single-lane roundabout CA model has in total $(a+b+c+d)$ cells, where a, b, c and d are the cells between arms *1* and *2*, *2* and *3*, *3* and *4*, *4* and *1* respectively. The vehicles entering from arm *1* are signed a, $(a+b)$, $(a+b+c)$ and $(a+b+c+d)$ randomly as shown in Table 1. The four numbers represent the number of cells to pass prior to exiting at the respective points. If the exit distribution of vehicles entering is known, i.e. that LT, ST, RT and back exit vehicles are $m_1\%$, $n_1\%$, $o_1\%$ and $p_1\%$ respectively, we then randomly assign $m_1\%$ of the vehicles with a, $n_1\%$ with $(a+b)$, $o_1\%$ with $(a+b+c)$ and $p_1\%$ with $(a+b+c+d)$ as shown in Table 5.1. Also, clearly

$$m_i\% + n_i\% + o_i\% + p_i\% = 100\% \qquad (i=1, 2, 3, 4) \qquad (5.4)$$

Table 5.1: The numbers that will be assigned to the vehicles

Arm1	Arm2	Arm3	Arm4
• $m_1\%$ a;	• $m_2\%$ b;	• $m_3\%$ c;	• $m_4\%$ d;
• $n_1\%$ $(a+b)$;	• $n_2\%$ $(b+c)$;	• $n_3\%$ $(c+d)$;	• $n_4\%$ $(d+a)$;
• $o_1\%$ $(a+b+c)$	• $o_2\%$ $(b+c+d)$	• $o_3\%$ $(c+d+a)$;	• $o_4\%$ $(d+a+b)$;
• $p_1\%$ $(a+b+c+d)$	• $p_2\%$ $(a+b+c+d)$	• $p_3\%$ $(a+b+c+d)$	• $p_4\%$ $(a+b+c+d)$

5.3.4 Up-date rules on the roundabout

The update rule for the roundabout is as follows. If the state in cell n at time-step t is larger than one ($C^t_n > 1$), which shows that there is a vehicle in cell n, the state ($C^t_{(n+1)}$) in cell $(n+1)$ in front must be checked to see if it is vacant. If it is ($C^t_{(n+1)} = 0$), the number will decrease by one when it moves forward into the cell $(n+1)$ in front ($C^{(t+1)}_{n+1} = C^t_n - 1$) and cell n will become zero ($C^{(t+1)}_n = 0$) in time step ($t+1$) (Expression 5.5). If the state in cell $(n+1)$ is not zero ($C^t_{(n+1)} \geq 1$), the number in cell n ($C^{(t+1)}_n$) will be unchanged ($C^{(t+1)}_n = C^t_n$) (Expression 5.6). As the car moves, its number finally becomes equal to one ($C^t_n = 1$), indicating that the car will leave the roundabout in the next time step if the exit is free, and there will be no car in this cell in the next time step ($C^{(t+1)}_n = 0$) (Expression 5.8). If the exit is not free, the car must remain in the current cell (Expression 5.9). The update rule on the roundabout is shown in Figure 5.5.

The update rules can thus be summarised:

- If $C^t_n > 1$ and $C^t_{(n+1)} = 0$, then $C^{(t+1)}_n = 0$ and $C^{(t+1)}_{(n+1)} = C^{(t+1)}_n - 1$ (5.5)
- If $C^t_n > 1$ and $C^t_{(n+1)} \geq 1$, then $C^{(t+1)}_n = C^t_n$ (5.6)
- If $C^t_n = 1$, then $C^{(t+1)}_n = 0$, if it is able to exit, otherwise, $C^{(t+1)}_n = C^t_n$ (5.7)

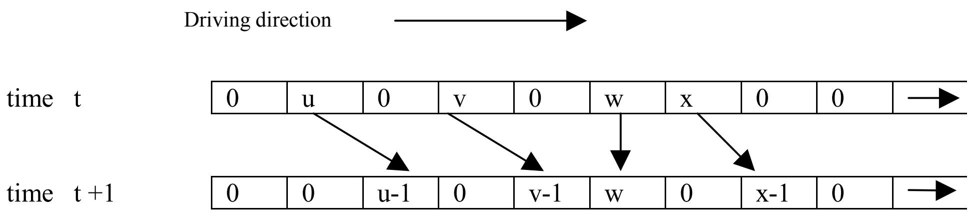


Figure 5.5: The update rules on the roundabout

There are several advantages of using this notation instead of just 0 and 1 as in 1DDCA. Firstly, this notation puts three meanings into a single integer number, so that the update rule becomes uniform for the roundabout, and is also simple and easy to program. The number not only provides information on whether a cell is occupied or vacant, but also indicates where its occupant will go. The update rule is as simple as for a normal road and it is the same for any cell on the roundabout. When a car drives out of the roundabout, the number automatically becomes zero.

Secondly, if we want to visualise the car on the roundabout in the future, its directional indicators may also be noted. Driving on the left lane in UK and Ireland for example, we can simply define it like this: if the number in the cell $\geq (a+b+c+d)/2$, its RT indicator is on; if the number $\leq (a+b+c+d)/4$ the left indicator on.

Thirdly, this method makes multiple entrance/exit programming possible and it can be applied to simulation of traffic flow where origin and destination are known, by assigning the vehicle a number corresponding to the steps needed to arrive at its destination exit.

5.3.5 Theorems of optimum density, throughput and size

The following theorems are developed to indicate the relationship between size density (the **density** is defined to be the number of vehicles on a road or a roundabout divided by the number of cells of the road or roundabout) and throughput of the roundabout. Theoretical deductions are given in this section and empirical proofs are given in Section 5.4.

Theorem 1: If the number of cells in a roundabout is even, assuming all the vehicles are evenly distributed on the roundabout (gaps between all vehicles are equal), the optimum density is 0.5 and the maximum throughput (see Section 5.2) is not related to the size (= number of cells) of the roundabout. If the density is smaller or larger than 0.5, the throughput observed will be smaller than the maximum throughput.

Theorem 2: If the number of cells of a roundabout is odd, equal to $(2n+1)$ cells, two local optimum densities are $n/(2n+1)$ or $(n+1)/(2n+1)$. Both have the same throughputs, which are maximum for the given size of the roundabout. The throughput is smaller than the maximum if the density is smaller than $n/(2n+1)$ or larger than $(n+1)/(2n+1)$. The maximum throughput increases slightly with the size of the roundabout.

These theorems can be proved based on **average speed** (*av*) from *density* (ρ) in a queue or in free flow. For a queue, $av = (1-\rho)/\rho$. For free flow, $av=1$, (Chopard 1998).

We may assume that all vehicles on average travel through Ω cells on the roundabout. If the total number of cells (N) is even or odd, that is $N=2n$ or $N=2n+1$ respectively, where n is an integer and Q is the number of vehicles on the roundabout, then **density**

$$\rho = \text{Number of vehicle /Number of cells} = Q / N \qquad (5.8)$$

Proof of Theorem 1, ($N=2n$)

Case 1:

When the density is 0.5, i.e. $\boldsymbol{\rho_1 = 0.5}$ and $N = 2n$, Q_1 computed from Equation 5.8, $Q_1 = \rho_1 N = n$. Hence there are n vehicles on the roundabout. It is free flow, therefore $av_1 = 1$. Time-steps (t) needed for Q vehicles to pass through the roundabout is given by $t_1 = \Omega / av_1 = \Omega$. The **passing rate of roundabout** (q) is the number of vehicles (Q) passing through a roundabout in one time step. Thus, $q_1 = Q_1 / t_1 = n / \Omega$.

When turning rates are fixed, Ω is related to the total number of cells of the roundabout, i.e. $\Omega = \delta (2n)$, where δ is a constant related to the turning rates. We can now assume that $\Omega = \beta n$, where β is just the constant redefined. Therefore, $\boldsymbol{q_1 = 1/\beta}$, i.e. the passing rate of roundabout has no relationship to the size of the roundabout when the density is 0.5.

Since **throughput** is the number of vehicles, which pass through the roundabout in a given time, this equals passing rate times the number of time steps. Thus, throughput has no relationship to roundabout size when the density is 0.5.

Case 2:

For m vehicles on the roundabout ($Q_2 = m$) and $m < n$, thus $\boldsymbol{\rho_2 < 0.5}$ and traffic is free flow. We get $av_2 = 1$, $t_2 = \Omega / av_2 = \Omega$, $q_2 = Q_2 / t_2 = m / \Omega$.

Since $\Omega = \beta n$, $q_2 = m/(\beta n)$. As $m < n$, $q_2 < q_1$, then if the density on roundabout is less than 0.5, the passing rate is less than q_1.

Case 3:

When vehicle number (Q_3) on the roundabout is k and $k > n$, we get $\boldsymbol{\rho_3 = k/2n > 0.5}$. There is now a queue on the roundabout, so $av_3 < 1$ and $av_3 = (1-\rho)/\rho = (2n-k)/k$. Therefore, $t_3 = \Omega/av = \Omega k/(2n-k)$, $q_3 = Q_3 / t = k/(\Omega k/(2n-k)) = (2n-k)/\Omega$.

As $\Omega = \beta n$, $q_3 = (2n-k)/\beta n$. Since $k > n$, $(2n-k)/\beta n < (2n-n)/\beta n = 1/\beta$. Therefore $q_3 < q_1$, i.e. if the density on roundabout is *larger* than 0.5, the passing rate is also less than q_1. Thus, 0.5 is the *optimum* density. Also, q_1 is the maximum passing rate. Therefore *maximum throughput* has no relationship with size of the roundabout, for number of cells even.

Proof of Theorem 2, ($N=2n+1$)

We also assume here that all vehicles on average pass Ω cells to traverse the roundabout.

Case 1:

When the vehicle number $Q_4 = n$, $\rho_4 = n/(2n+1)$ and $\boldsymbol{\rho_4 < 0.5}$. Thus, $av_4=1$. Consequently $t_4 = \Omega/av = \Omega$, $q_4 = Q_4 / t_4 = n/\Omega$. . Similarly, $q_4 = \beta(2n+1)$. Therefore,

$$q_4 = n/(\beta(2n+1)) \tag{5.9}$$

Assuming $\eta = 1/\beta$, then $q_4 = \eta n/(2n+1)$. For two roundabouts (**a** and **b**), if size of **a >** size of **b,** i.e. $(2n_a+1) > (2n_b+1)$, then $n_a > n_b$, thus we can get $n_a/(2n_a+1) > n_b/(2n_b+1)$. Therefore $q_a > q_b$, where q_a and q_b are the passing rates of two roundabouts, i.e. the passing rate increases with the size of the roundabout.

Case 2:

When $Q_5 = n+1$, $\rho_5=(n+1)/(2n+1)$, ($\boldsymbol{\rho_5 > 0.5}$) and there is a queue on the roundabout. Thus, $av_5 = (1-\rho)/\rho = n/(n+1)$, $t_5 = \Omega / av_5 = \Omega (n+1)/n$, $q_5 = Q_5 / t_5 = (n+1) / (\Omega (n+1)/n) = n/ \Omega$. As $\Omega = \beta (2n+1)$,

$$q_5= n/ (\beta (2n+1)) \qquad (5.12)$$

According to Equations 5.9 and 5.10, $q_4 = q_5$. It follows that both densities $n/(2n+1)$ and $(n+1)/(2n+1)$ have the same passing rate.

Case 3:

When $\rho_6= m /(2n+1)$ and $m< n$, $Q_6 = m$ and $\boldsymbol{\rho < 0.5}$, so there is free flow on the roundabout. $q_6= m/ (\beta (2n+1))$. Since $m < n$, $q_6 < q_4$, i.e. if the density on the roundabout is smaller than $n/(2n+1)$, the passing rate is less than q_1.

Similarly, we can show that if density is larger than $(n+1)/ (2n+1)$, the passing rate is less than q_4. Therefore, when the density is $n/ (2n+1)$ *or* $(n+1)/ (2n+1)$, throughput reaches a maximum for given roundabout size. The maximum throughput increases with the size of the roundabout.

5.3.6 Implementation

The program has been developed in two parts. The first part handles data input. It produces a configuration file for use by the second program. The second is the main program. In the former, data entered include the number of roads, the length of each road, the number of cells between entrances and the length of time for the simulation. Also, further information is provided on mean arrival and turning rates of each road, and driver behaviour probabilities are adjustable. Hence, the program can simulate different traffic configurations and different geometric sizes and shapes of roundabouts.

The main program contains two classes: road and roundabout classes. Both classes contain the following functions: driving-in, driving-out, update and information output. The information output functions give us all the information about the operation of the roundabout for each time unit, (1 time unit = 60 time-steps) and also in total. This includes details such as:

- Number of cars entering the roundabout
- Numbers of vehicles that have passed through and remain on each road
- Number of vehicles that have passed through roundabout (throughput)
- Density and queue length of each road, expected delay times at entry etc.

5.4 Single-lane Roundabout Simulation

In order to study the three aspects of roundabout performance, which have been mentioned at the beginning of Section 5.3, the following experiments have been carried out. In each experiment, the length of each entrance road is 100 cells. If the throughput is printed in **bold**, (as in Table 5.2 for instance), it means that the queue length has reached the length of the road on at least one road, which we denote **saturated.** All experiments are carried out very long periods (equivalent to 30 hours or 10800 time-steps).

5.4.1 Relationship between the size (or shape) and the overall throughput

In Section 5.3.5, this relation has been deduced mathematically. The experiments are set up to investigate the theorems of optimum density and throughput on the roundabout. The first series of experiments seek to determine the relationship between size and throughput of the roundabout based on the same topology, i.e. a four-arm roundabout (four entrances/exits), but of different sizes (i.e. number of cells). Over 100 paired experiments have been performed. In each pair of experiments, the topologies, arrival rates and turning rates are the same, and driver behaviour is taken to be same but sizes of roundabouts are varied. One contained 16 cells, the other 32 cells for example. In Table 2, the means of turning rates for left-turn, straight and right-turn are taken to be 0.25, 0.5 and 0.25 respectively and all use optimal entry conditions. Different shapes of roundabouts are also explored, (i.e. distances between the entrances taken to be different).

Five sets of experimental results are shown in Table 5.2, in which the numbers of cells are even, i.e. *16, 32* and *50*. In the first two experiments, the distances between entrances are equal (**equal-spacing**), but sizes are different. In the third and fourth experiments, the distances between entrances are varied (**non-equal-spacing**) and sizes are also different. In the fifth experiment, the size is *50* cells and again non-equal spacing applied.

Table 5.2: Throughputs (vph) for the numbers of cells of roundabouts are even and the topologies and turning rates are the same

Size (cells)	A1 to A2 (cells)	A2 to A3 (cells)	A3 to A4 (cells)	A4 to A1 (cells)	Throughput1 AR =0.15	Throughput2 AR=0.20	Throughput3 AR =0.25	Throughput4 AR=0.30
16	4	4	4	4	2149	2883	**3578**	**3600**
32	8	8	8	8	2157	2884	**3578**	**3597**
16	3	4	3	6	2159	2876	**3575**	**3595**
32	13	5	3	11	2158	2886	**3581**	**3599**
50	5	15	10	20	2159	2889	**3577**	**3598**

A1 to A2 is the distance between the first and the second entrance of the roundabout. Throughput1 is the throughput when the means of all arrival rates (AR) are 0.15.

We find that throughput values in each column (with the same arrival rates) are very similar, although sizes and shapes are different. The throughputs change when arrival rates increase (for the first three columns). Throughputs do not appear to depend on whether the distances between the entrances are equal or unequal, as long as turning rates and the topologies are the same. The same results are also found for other topologies, i.e. 3-arm roundabouts. The results indicate that the overall throughput is not related to size for roundabouts, given that numbers of cells are even and for topologies, arrival rates and turning rates otherwise the same.

In Table 5.3, the number of cells are odd, 17, 21, 41 and 51. Non-equal-spacing applies throughout. We also find that throughput values in the first two columns (with arrival rates of 0.15 and 0.20) are similar, although sizes and shape differ. However, throughputs in the last two columns (with arrival rates of 0.25 and 0.30) increase with the size of the roundabout.

When arrival rates are lower, there are no queues on the entrances and no saturated situations. Throughput values are equal to the number of vehicles that arrived at the roundabouts, so values in each column are the same, (see e.g. arrival rates = 0.15 and 0.20 in Table 5.3). By contrast, when arrival rates are higher, e.g. 0.25 and 0.30, the throughputs increase with the size of the roundabout.

Table 5.3: Throughputs (vph) for numbers of cells of roundabouts are odd and the topologies and turning rates are the same

Size (cells)	A1 to A2 (cells)	A2 to A3 (cells)	A3 to A4 (cells)	A4 to A1 (cells)	Throughput1 AR=0.15	Throughput2 AR=0.20	Throughput3 AR =0.25	Throughput4 AR=0.30
17	3	5	4	5	2166	2874	**3457**	**3481**
31	5	**7**	11	8	2157	2867	**3514**	**3514**
41	5	**17**	11	8	2146	2873	**3527**	**3524**
51	5	**27**	11	8	2154	2877	**3531**	**3538**

The experimental results support the theorems of optimum density on the roundabout. Whether a vehicle can or can not drive onto a roundabout depends on the situation at the entrance, where these are the bottlenecks.

For an individual vehicle passing through a large sized roundabout, more time steps are needed compared to the requirement for a small one. However, considering the roundabout as a whole, the number of vehicles passing through relies on how many opportunities there are for vehicles to enter. The size and geometry of a roundabout have therefore no direct influence on throughputs of single-lane roundabouts when the number of cells is even.

However, the phenomenon of maximum throughputs increase with size, when the number of cells is odd, is caused by the free flow requirement, i.e. there is one space and one vehicle alternatively on the road. One extra space is not enough to add an extra vehicle and also keep traffic in free flow, as any extra vehicle will block the vehicle behind. If no extra vehicle enters the roundabout, this extra space only increases the travelling distance of vehicles (conclusions apply for an ideal situation, i.e. uniform size and speed of vehicles), hence non-optimum spacing (or size) is a factor.

5.4.2 Relationship between throughput and arrival rates

Table 5.4 and Figure 5.6 show that throughputs change with arrival rates. Arrival rates of three roads, i.e. (AR_1, AR_2 and AR_3) are the same and increase from 0.05 to 0.45. Arrival Rate of road 4 (AR_4) also increases from 0.05 to 0.55.

Table 5.4: Throughputs vs. arrival rates

AR_1,= AR_2 =AR_3	*AR_4 (0.10=360vph)*										
	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55
0.05	722	932	1075	1259	1448	1618	1807	1976	**2113**	**2117**	**2121**
0.10	1262	1409	1619	1804	1971	2185	2352	**2452**	**2441**	**2442**	**2455**
0.15	1808	2002	2151	2337	2520	2691	**2786**	**2799**	**2797**	**2793**	**2790**
0.20	2352	2545	2697	2884	3064	**3155**	**3164**	**3170**	**3151**	**3166**	**3166**
0.25	**2794**	**2983**	**3178**	**3371**	**3570**	**3576**	**3582**	**3574**	**3584**	**3589**	**3600**
0.30	**2932**	**3124**	**3310**	**3443**	**3591**	**3599**	**3591**	**3599**	**3580**	**3589**	**3600**
0.35	**3051**	**3252**	**3331**	**3444**	**3591**	**3595**	**3600**	**3595**	**3599**	**3591**	**3601**
0.40	**3175**	**3250**	**3339**	**3463**	**3586**	**3598**	**3596**	**3600**	**3599**	**3599**	**3596**
0.45	**3177**	**3251**	**3338**	**3439**	**3599**	**3600**	**3589**	**3598**	**3596**	**3591**	**3599**

For AR_1=AR_2=AR_3 < 0.25, we find that throughput increases linearly as arrival rate of road 4 (AR_4) increases, (for no entrance saturated). For example, for AR_1=AR_2=AR_3= 0.10, and when $AR_4 \geq 0.40$, road 4 is saturated and throughputs are constant. The maximum throughput is achieved when road 4 saturates. Thus throughputs increase as arrival-rates increase to a saturation level.

When an arrival rate for an entry road ≥ **critical arrival rate** *(CAR),* saturation occurs on the entry road. For these conditions, $CAR = 0.4$ for road 4, which is indicated in shading in the table. Critical arrival rates varied with the other three ARs. The relationship between CAR and arrival rates of the other three roads is:

- *If* $AR_1=AR_2=AR_3 < 0.25$, *then* $CAR_4=0.5 - AR_i$ (5.11)
- *If* $AR_1=AR_2=AR_3 \geq 0.25$, *then* $CAR_4=0.25$ (5.12)

where $i = 1, 2$ *or* 3.

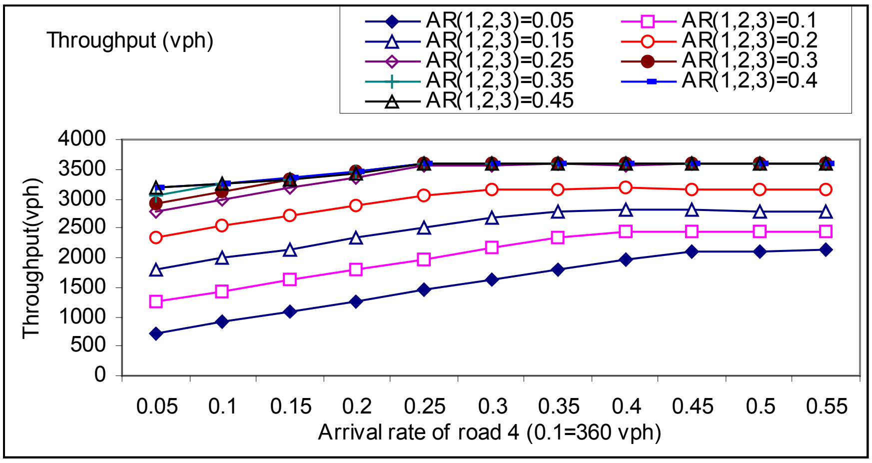


Figure 5.6: Throughputs vs. arrival rate.

When $AR_1 = AR_2 = AR_3 \geq 0.25$, even though $AR_4 < 0.05$, one entrance road of the roundabout is over saturated. For any AR > 0.25 saturation will happen in at least one entrance road.

We also find that the value of critical arrival rate is constant and $CAR = 0.25$ (Expression 5.12), when $AR_1 = AR_2 = AR_3 \geq 0.25$. Throughputs reach a maximum rapidly and remain constant at this saturation level on all four roads.

We also find that by balancing arrival rates ($AR_1 = AR_2 = AR_3 = AR_4$), the operational performance of a roundabout can be improved. If we define the **effective throughput** as the throughput when no entrance road is saturated, *the maximum effective throughput* that we find is *3458* vph and it is achieved when $AR_1 = AR_2 = AR_3 = AR_4 < 0.25$.

5.4.3 Relationship between throughput and turning rates

For situations when cars are driving on the left-hand side of the road, such as in the UK and Ireland, the relationship between throughput of a roundabout and turning rates can be observed from Table 5.5. The data generated are based on a 32-cell 4-road-single-lane roundabout. $AR_1 = AR_2 = AR_3 = AR_4$. The mean of straight-through rates

(STR) remains constant = *0.5*. The mean right-turning rate (RTR) increases from *0.15* to *0.35* and left-turning rates (LTR) vary from *0.35* to *0.15* respectively.

Table 5.5: Throughputs of the roundabout for $AR_1=AR_2=AR_3=AR_4$ and Right-turning rate is from *0.15* to *0.35*. Straight going rates are *0.5*

$AR_1=AR_2=AR_3=AR_4$	Right Turning Rate		
	0.15	0.25	0.35
0.15	2160	2158	2160
0.20	2898	2885	2881
0.25	3615	**3570**	**3267**
0.30	**3999**	**3599**	**3273**
0.35	**3996**	**3600**	**3270**

In Table 5.5, when $AR_1=AR_2=AR_3=AR_4= 0.15$ and *0.20* , traffic is in free flow. Turning rates have no impact on throughput. When $AR_1=AR_2=AR_3=AR_4= 0.25$ and RTR is *0.15*, traffic is still in free flow. However, when RTRs are equal to *0.25* and *0.35*, entrance roads are saturated and turning rates do affect throughputs, by about *10%* (see difference of 3570 to 3267 vph). When $AR_1 = AR_2 = AR_3 = AR_4 > 0.25$, turning rates also affect throughputs: *5%* increase in RTR, gives around *10%* decrease in throughput. Throughputs thus decrease as right-turning rates increase when entrance roads are saturated.

5.4.4 Individual road performance—queue length

In Figure 5.7, the queue-lengths change with AR_4, which gives us a clear picture of how critical arrival rate corresponds to the maximum throughput and saturation. In Figure 7, $AR_1=AR_2=AR_3=AR_4 = 0.20$, but AR_4 increases from 0.2 to 0.4.

When AR_4 is below the critical arrival rate (*CAR=0.30*, Section 5.4.2), the queues will usually be short and frequently no car is waiting to enter so that throughput will be less than the maximum.

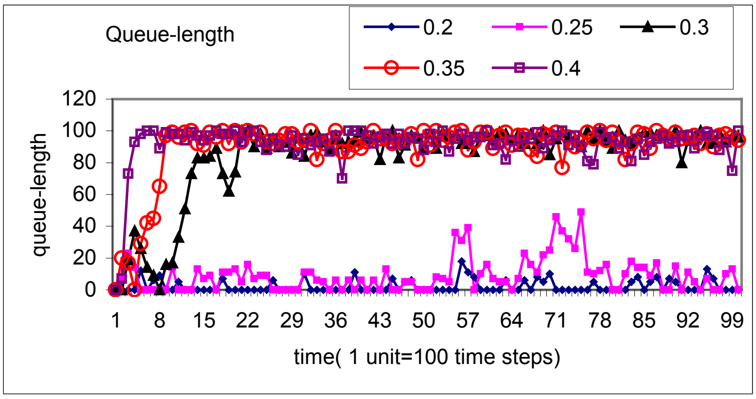
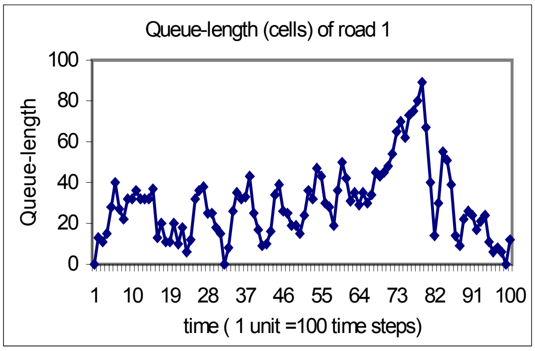


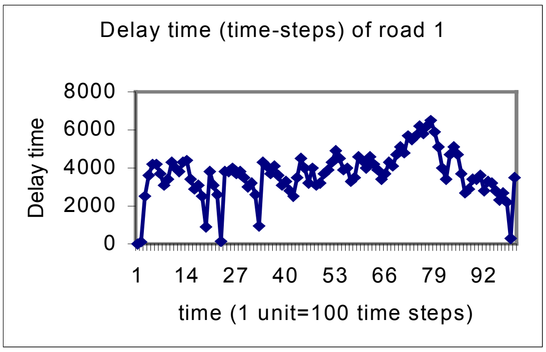
Figure 5.7: Queue-lengths on road 4 change with AR_4 from 0.20 to 0.4, for AR_1=AR_2=AR_3= 0.20.

When $AR_4 \geq CAR$, we find that the queues build up very quickly (Figure 5.7). It takes about 900 time-steps (15 minutes) for AR_4=0.35 to result in a queue up 100 cells. It takes about 600 time-steps (10 minutes) for AR_4=0.40 to match the same length. Basically, the speed of formation of the queue increases as AR_4 increases. The queue reaches the maximum length very rapidly for any arrival rate larger than CAR (0.3 under these conditions).

5.4.5 Individual road performance—expected delay time



(a)

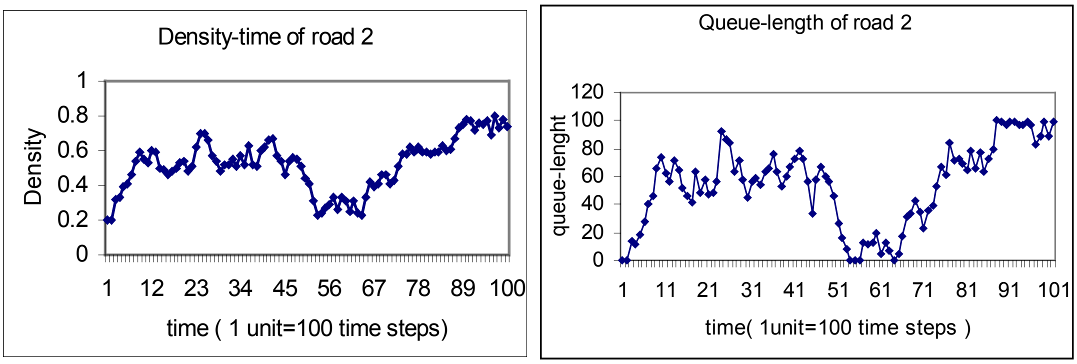


(b)

Figure 5.8: Queue-length and delay-time of road 1 for 10,000 time steps

Expected delay time is determined based on the available opportunities for vehicles to drive onto the roundabout in the last 100 time-steps and the number of vehicles on the given road. Figure 5.8 indicates the expected delay time and queue length of road 1, when $AR_1=AR_2=0.23$, $AR_3=0.24$ and $AR_4 = 0.25$. Figure 5.9 (a and b) give the details of the first 3000 time steps of Figure 5.8 (a and b). Queue length is clearly a general indicator of delay time.

5.4.6 Individual road performance—average densities



(a) (b)

Figure 5.9: Density and queue length of road 2.

In Figure 5.9 (a) and (b), the densities (ρ) and queue-lengths change on road 2 when $AR_1=AR_2=AR_3=AR_4=0.25$. When density is 0.23 at time-step 6300 in Figure 5.9 (a), a queue forms, even though density is 0.23 is much less than 0.5. 0.5 is the maximum density for free flow. Thus, queue may form even the density is much less than the maximum free flow density.

When the queue reaches the maximum length of the road (100 cells), the density of the entrance road is 0.73. In other experiments, we also found similar results with queue formation occurring at densities in the range of 0.2 to 0.8 (similar to the relationship between density and queue formation for an unsignalised intersection road, Chopard (1998)). However, $\rho_{max}=0.8$ for a queue of just 97 cells. Therefore, the maximum density and maximum queue length do not necessarily happen simultaneously.

5.4.7 Driver Behaviour

The impact of driver behaviour on throughputs can be shown in the following experiments. As explained in Section 5.3, a simplification for each experiment is to assume one of conservative (P_{co}), rational (P_{ra}), urgent (P_{ur}) and radical (P_{rad}) to have probability equal to 1, the other three equal to 0, i.e. all driver behaviour is taken to be similar for a given special case. Although the model enables us to deal with all possible combinations of driver behaviour, we use these four special situations to give us some indication of how this behaviour impacts on roundabout performance.

In Table 5.6, $AR_1=AR_2=AR_3=AR_4$ in each row. When $AR_1=AR_2=AR_3=AR_4=0.10$ in row 1, all throughputs are the same. When $P_{co}=1$ and $AR_1=AR_2=AR_3=AR_4 \geq 0.20$, throughput reaches the maximum and a saturated situation occurs on entrance roads, but traffic flow on the roundabout remains in free-flow at all times. When $P_{ra}=1$ or $P_{ur}=1$, throughputs are the same and are larger than throughputs for $P_{co}=1$. Traffic flow on the roundabout again remaines in free-flow all times. When $P_{rad}=1$ and ARs increases, throughput decreases, as when $AR_1=AR_2=AR_3=AR_4\geq$ 0.15, congestion occurs *on the roundabout.*

Table 5.6: Throughput of roundabout when driver behaviour at four special situations. Arrival rates are the same for all roads.

Row *No*	AR	Driver behaviour			
		$P_{co}=1$	$P_{ra}=1$	$P_{ur}=1$	$P_{rad}=1$
1	0.10	1440	1439	1439	1443
2	0.15	2169	2169	2169	**20**
3	0.20	**2446**	2862	2873	**3**
4	0.25	**2445**	**3575**	**3573**	**4**
5	0.30	**2436**	**3590**	**3599**	**2**

Similar results are also found for other turning rates. As might be expected, the collective conservative behaviour decreases throughput. Urgent behaviour, however, will not increase or decrease throughput, compared to rational behaviour. In contrast, collective radical behaviour will cause congestion on the roundabout and radically decrease throughput compared to rational behaviour. Driver behaviour is clearly not universally good or bad in the real world, so that a distribution is clearly more appropriate.

5.5 Calibration and Validation

The validation of this as for all the models discussed in this work involves two aspects. The first is checking the model itself, which includes checking assumptions and rules, where these represent a compromised view of the world reality plus the process debugging.

Including this single-lane roundabout model, all our models have been tested by the above methods. Update rules are tested by observing content in each cell in each time-step. Details of the entry process are checked by observing interaction between vehicles when vehicles enter the roundabout according to different driving behaviour. Checking of the total number of vehicles entering the roundabout is also performed and the sums of vehicles on the roundabout and passing through the roundabout reconciled.

The second aspect to consider was the reality of the model. Three levels of validation are suggested by Shannon (1975), namely are:

- Validation of each subsystem
- Validation at the interfaces
- Validation of entire model

Ideally, this should be performed at both the macroscopic level to ensure that overall performance of the model matches the observable reality, but also at a microscopic level (calibration), with regard to individual vehicle-vehicle interactions (Brackstone and McDonald 1996).

Individual vehicle-vehicle interactions are essentially confined to entrances. Probabilities of different driver behaviour are assigned subjectively in our experiments, which would benefit from calibration on further real data, as our field observations have been necessarily limited.

As we do not have much real data, we have calibrated our model by comparing it with previous models. All previous roundabout capacity models (Section 5.2) mainly analysed the relationship between the entry capacity and circulating flow rate. In order to compare our model with previous models on the same basis, the circulating and entrance situation have been simulated.

The original of Figure 5.10 was presented in an analysis paper of the Transportation Planing Analysis Unit (TPAU 1998), which had the responsibility for selecting the methodology for Oregon Department of Transportation (US) to analyse roundabouts. We have added two results (curve CA and CA1) from our model to the figure. These are compared with the SIDRA5.1 program, (Traffic Signalised & unsignalised Intersection Design and Research Aid), two Highway Capacity Manual methodologies (HCM (upper, lower) (US), an Australian methodology (AUSTROADS), two German methodologies and the UK methodology (G1 and G2).

SIDRA5.1, HCM (upper and lower) and AUSTROADS are based on gap-acceptance models. Models G1, G2 and UK are empirical models. Variables of analytical models can be modified to match the driver behaviour of a target area, while the empirical models are not ready to be modified (TPAU 1998). Some models (e.g. SIDRA5.1 and G2) are more conservative than others, when the circulating flow is heavy. However, among all methods mentioned above, a combination of SIDRA 5.1 and the German ‘G2’ methodology was recommended by TPAU (1998). Details of comparison of all these models can be found in TPAU (1998).

Assuming that all driver behaviour is rational, we observed the relationship between circulating flow rate and entering capacity shown in the curve CA1, which is slightly below that for UK methodology and above the SIDRA5.1 result.

According to the investigation on critical gap and follow-up time by Tian et al. (1999), we have a further relationship between circulating flow rate and entering capacity, shown in the curve CA. Tian et al. (1999) indicated that there were many factors that might influence the critical gap and follow-up time, such as delays, vehicle types, traffic movements and speed limits. They found that drivers use shorter critical gaps at high circulating rates due to the effect of longer delays. Drivers may use longer critical gaps when they do not need to wait so long to get a longer gap.

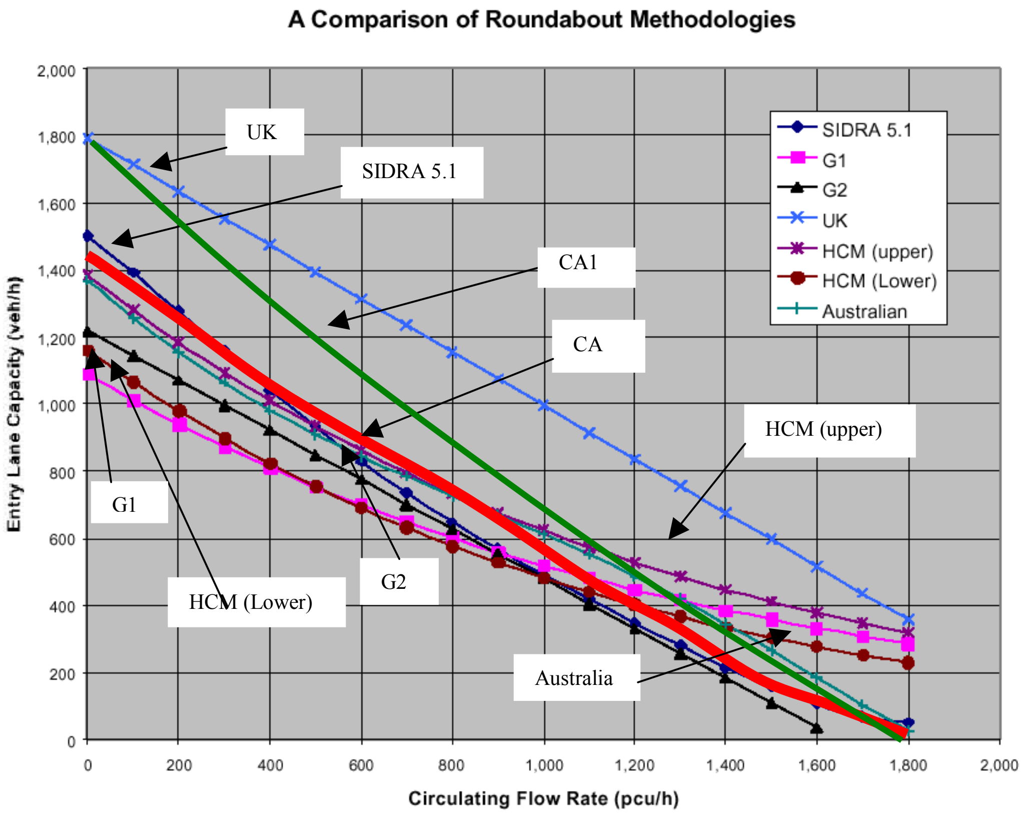


Figure 5.10: A comparison of roundabout methodologies

This finding had already been used in the Australian capacity formula, which incorporated variations of critical gaps and follow-up times with different volumes of traffic in order to over come the shortcomings of the gap-acceptance technique (Taekratok 1998). Based on Tian's theory and recommendation of TPAU (i.e. a combination of SIDRA 5.1 and the German 'G2' methodology is recommended), probabilities of different driver behaviour can be therefore approximately calibrated. We allowed the probability of conservative behaviour to change from 0.5 to 0 when the circulating rates changed from 1800 to 0 vph (see Equation 5.13) and probabilities of rational, urgent and radical behaviour change correspondingly according to Equations 5.14 and 5.15. We got the curve CA. We found that curve CA agreed well with most methodologies.

$$P_{co}= 0.05\ Q_c /180 \tag{5.13}$$

$$P_{ra}= 1- P_{co} \tag{5.14}$$

$$P_{rad} = P_{ur}= 0 \tag{5.15}$$

where Q_c = ($0\sim1800$) circulating flow (vph)

P_{co}= probability of conservative behaviour

P_{ra}= probability of rational behaviour

P_{ur}= probability of urgent behaviour

P_{rad} = probability of radical behaviour

The moderating effect of this additional flexibility is clearly seen in Figure 5.10. Overall, our model seems both flexibility and compatible with findings for other countries and systems.

5.6 Summary

CA models have been used effectively to simulate traffic flow at urban roundabouts. Various properties of roundabout operations have been explored, including time delay, critical arrival rates, throughputs and queue formation, together with variations of queue lengths, time delay and congestion on the roundabout itself.

Theoretical analysis has show that if the number of cells is even on the roundabouts, then throughput does not depend on roundabout size, equal-spacing or non-equal-spacing, given similar topology and other parameters held constant. If the number of cells of the roundabouts is odd, throughput increases when the size of the roundabout increases. Throughput levels in general are different across different topologies. Clearly, the entrances are bottlenecks in terms of smooth operation.

In general, throughput increases with arrival rate linearly when no entrance road is saturated. Throughput reaches a maximum when the arrival rate reaches a critical value on one or more roads. When the arrival rate is larger than the critical value, saturation occurs on one or more roads. Critical arrival rates also depend on other road arrival rates (e.g. Expression 5.9) and depend for all roads on roundabout topology and turning rates. The operational performance of a roundabout is clearly improved when arrival rates are balanced, ($AR_1=AR_2=AR_3=AR_4$).

Throughput decreases as right-turning rate increases, as vehicles on average need to travel longer distances on the roundabout.

When arrival rate is less than the critical value, queue-length of an individual road is low, but for arrival rate greater than critical, the queue length rapidly achieves maximum. The speed of formation of the queue increases as arrival rates increase. Over 100,000 time-steps, the maximum queue-length or saturation of a given entry road was observed to occur within a few hundred time-steps if arrival rates ≥ the critical arrival rate.

Queue-length is an important indicator of delay. Queue formation occurs at densities in the truncated range 0.2-0.8, which is similar to the result obtained by Chopard (1998). The queue forms at a density far below the maximum for free flow, which is *0.5*. The maximum density and the maximum queue-length do not necessarily occur simultaneously.

Driver behaviour has an impact on the overall performance of the roundabout and individual roads. Rational, urgent and conservative behaviour leads to free-flow on the roundabout for all arrival/turning rates considered, whereas radical behaviour can lead rapidly to congestion.

Assigned probabilities are clearly subjective and would benefit from calibration on real data, but equally are unlikely to be standard for real traffic systems.

Chapter 6
Multilane Roundabouts

6.1 Introduction

When traffic flow is too large for a single-lane roundabout to cope with, multilane roundabouts are an alternative. Two-lane roundabouts, for example, are heavily used in the UK and Ireland, whereas three-lane roundabouts are common in other parts of the world, even in some parts of Europe. One of the reasons that two-lane roundabouts are more commonly used than three-lane roundabouts clearly relates to space required. In this chapter, we mainly model traffic flow at two-lane roundabouts, since we are interested in the effects of lane-allocation on entry roads and lane-changing on roundabouts, while the three-lane roundabout model is briefly considered in Section 6.5, as extension of the two-lane case.

6.2 Background

Two-lane roundabouts have previously been studied using gap-acceptance models. Research has thus focused on the estimation of critical gaps in two (or multi-) major-streams (Golias 1981, McDowell et al. 1983, Catchpole and Plank 1986, Hagring 2000). Golias (1981) used the EM algorithm, (Dempster et al. 1977), for estimating the critical gaps in T-junctions (with two major streams) and McDowell et al. (1983) used an edge distribution, which required observation of rejected and accepted gaps only in one major lane when gaps in other major lanes were so large that the driver on a minor-stream could not be influenced by them, (Hagring 2000).

Hagring (2000) presented a maximum likelihood method for estimating the different critical gaps in the case of two major lanes and confirmed that it was possible to estimate critical gaps separately for each major stream. The author also suggested that gaps in the two lanes might not be correlated. However, his result that the critical gap in the near lane was larger than that in far lane contradicted the result reported by Golias

(1981) and McDowell et al. (1983). Hagring suggested that the explanation was that the Golias (1981) and McDowell et al. (1983) studies were conducted on T-junctions or combinations thereof whereas his investigation was on roundabouts.

Hagring (2000) also indicated that minor-stream vehicles in the outer lane were also impeded by the far lane major-stream vehicles, even if these did not physically interact, although no quantification of this conclusion has been reported and it is hard to explain why this should be the cases. One explanation, given by Troutbeck (1990), was that it might be difficult for the minor-stream vehicles in the outer lane to judge if the far-lane major-stream vehicles would exit (or change lane). If this was the case, the minor-stream vehicles on both lanes should have nearly the same amount of opportunities to move onto the roundabout. However, according to our data recorded for a two-lane roundabout, the minor-stream vehicles in the outer lane actually have roughly double the amount of opportunities compared to minor-stream vehicles in the inner-lane.

Our explanation is that minor-stream vehicles on the outer lane are not impeded by the far lane major-stream vehicles, but by minor-stream vehicles on the inner lane. In other words, for a vehicle driving on the right-hand side in the UK and Ireland, *minor-stream* vehicles on the left-lane are impeded by minor-street vehicles on the right-lane, as the later block the view of the former. Therefore, even if the outer lane of a *major-stream* is free, minor-stream vehicles on the left-lane still need to firstly position properly and then to get a view and check whether the outer lane of major stream is free. Thus, the delay is due to the position of minor-stream vehicles on the outer lane, so that this suggests the need to introduce a position delay feature into modelling traffic flow at two-lane roundabouts to simulate this delay.

In previous chapters, (Chapter 3, 4 and 5), we have analysed why the gap-acceptance approach does not properly describe the driver behaviour and why it is not suitable for modelling urban drive behaviour. In this chapter, we apply our MAP method to modelling multilane roundabouts and investigate in the operational properties.

6.3 Methodology

Vehicles at a two-lane roundabout observe the same priority rules as at a single lane roundabout. Vehicles in the left-lane and right-lane of the entry roads move onto the corresponding lanes of the roundabout.

Vehicle navigation through a roundabout is subjected to the following processes:

- Vehicle arrival: vehicles arrive at the beginning of entrance road (e.g. 100 cells away from the roundabout)
- Pre-determined destination: each vehicle has its own pre-determined destination (before entering an entry road and being allocated to a lane of the entry road)
- Lane allocation: a vehicle is allocated to a lane on an entry road
- Vehicles move along entrance roads
- Position delay: vehicles on the left-lane of an entrance road may be halted for position delay time (PDT), in order to adjust their positions and check if they have opportunities to enter the roundabout, (details Section 6.3.2).
- Entering roundabout: interaction between drivers at the entrance and vehicles on the roundabout
- Navigation of roundabout
- Exit

In this chapter, we mainly focus on the third, fifth, sixth and eighth processes identified above, as the others are similar to previous chapters.

One factor that dictates which lane a vehicle is assigned to is the destination of the vehicle. For example, if the vehicle turns right on a roundabout, it will be allocated to the right-lane of the entry road. Logically, therefore, we assign the destination before lane-allocation. It is also more realistic to assume that the destination remains predetermined before navigation of the roundabout. Therefore, the destination remains unchanged, once assigned, for all vehicles throughout the manoeuvre.

6.3.1 Lane–allocation Process

The lane-allocation process at a roundabout is similar to for the major roads of a 2-lane TWSC intersection. However, criteria of lane-allocation differ slightly since traffic flow features are different. For example, the feature that only right-turning (RT) vehicles may use the right-lanes is common for 2-lane TWSC intersections, but is rare for two-lane roundabouts. We develop three possible systems for two-lane roundabouts:

- Left-turning (LT) vehicles using left-lane only, straight-through (ST) and right-turning (RT) vehicles using right-lane only (Model A)
- LT vehicles using left-lane only, RT vehicles using right-lane only, and ST vehicles can use both lanes (Model B)
- LT vehicles using left-lane only, unless the left-lane is full. RT vehicles using right-lane only, unless the right-lane is full. ST vehicles can use both lanes (Model C)

In the first scenario, the vehicles on the roundabout are free to exit and LT vehicles are free to enter the roundabout, as vehicles on the outer-lane (outside of inner-lane) of roundabouts are LT vehicles only. Another advantage of this system is that entry vehicles need to check the space on the inner-lane of the roundabout only, because there is no oncoming vehicle from the outer-lane. The interaction is only a merging process between the vehicles in the circulating flow on the inner lane of the roundabout and vehicles on the right-lanes of entry roads. No cross interaction occurs, as there is no oncoming vehicle from the outer-lane of the roundabout. Thus, this system is safer than others and is the most commonly used.

This system is implemented by putting traffic sign “arrow” marks on the surface of entrance roads, which all drivers should observe. Obviously these rules typically only be observed when the road is not saturated, as the ST vehicles will take the left-lane in reality if the right-lane is full!

The second scenario is used to give greater flexibility to the ST vehicles. Driver-lane selection might be based on personal preference, queue-length of each lane and perception of waiting time and so on. Based on our observations, we believe that queue-lengths are the major factor. Drivers normally tend to select the shortest queue. As ST vehicles can use the outer-lane, some passing interaction occurs when vehicles are entering the roundabout and when vehicles on the inner-lane of the roundabout are exiting. Thus, it is less safe than the first system.

The third scenario is a special case of Model B, when the left- or right- lane is full, LT or RT vehicles can use the less busy lane. It is possible that Model C is the one that most closely resembles reality in special situations, where there are high LT or RT rates, but is otherwise not common. These three scenarios are modelled, but the operational performances of Model A and B are particularly studied and compared in Section 6.4.

6.3.2 Position Delay Time

We have observed one particular phenomenon of driver behaviour (see example below), which to our knowledge is not reported by any other researchers, but which should be built into viable roundabout models in our view. It occurs commonly on two-lane minor roads of TWSC intersections and two-lane entry roads of roundabouts. The occasion is that of driver on the vehicle on the left-lane needing extra time to adjust his/her position to avoid sight-blocking caused by the vehicle and/or people sitting in the front seats of the vehicle on the right-lane of the road. This is designated "Position Delay Time" (PDT). Particularly for two-lane intersections and relatively large diameter roundabouts, it is more difficult for the left-lane vehicle to check if there is a vehicle oncoming from the right in these circumstances.

This phenomenon can also be found on the entry roads of three-lane roundabouts. The vehicle on the right-lane has no problem observing the circulating vehicle on the roundabout, but the vehicles in the middle-lane and left-lane have to adjust their positions to get a better view.

The above finding is supported by our observations conducted at rush hour in the afternoon from December 10 to January 10, 2002 at Panmure Roundabout (a three-lane roundabout, Auckland, New Zealand). Total tape-recorded observation hours were 6 hours. The observation results gave us only crude estimates and were used to form idea of how PDT time works. Clearly the need for extensive collection is obvious.

6.3.3 Interaction at entrance of roundabout

We use two CA-rings to simulate the two-lane roundabout (as in the figure shown in Appendix A). Both rings have the same number of cells, i.e. we assume that the vehicles in both lanes transverse the same number of radians in the same period of time. This is permitted by the assumption of an adjustment of the speed of the vehicle in the inner lane (which has a shorter radius). Thus speed in this lane is taken to be slower than the speed of the vehicle in the outer lane. Therefore, our assumption is closer to reality than if all vehicles have the same speeds.

In order to simplify the representation, the shape of the arc of the roundabout (Appendix B) with entry road can been changed to resemble Figure 6.1, which looks like a T-intersection. The paths of vehicles in the left-lane and right-lane of the entry road are shown in Figure 6.1 (d), while the paths of vehicles exiting from the roundabout (from the inner and outer-lane) are shown in Figure 6.1 (c). When the vehicle in the right-lane needs to change lane from the outer-lane to the inner-lane, it crosses the two cells diagonally. Likewise, this is true for the vehicles coming from the inner-lane to the outer-lane (see the curved arrow line). In other words, when the vehicle changes lane, it move ahead one cell at the same time.

Following the MAP method, used in previous chapters, we show similar figures to explain the conditions that are required by vehicles from entry roads. Again, driver behaviour is categorised into four groups: conservative, rational, urgent and radical. The distribution of driver behaviour can be expressed by four probabilities as before (see Section 3.3.2).

The required conditions for the target vehicle (shaded) to move onto the roundabout in ***this*** time step are indicated by the spaces required (shaded cells) in each of Figures 6.1(a) to (d) and Figures 6.2 (a) to (d) based on different driver behaviour.

Although the states in all cells in CA models are updated simultaneously, we show in figure that the states of cells *on the intersection* have been updated in this time step, but the cell that is occupied by the target vehicle (shaded) has not yet been updated (not move on to the roundabout). We do this alternative to explain in detail how the MAP method is used here (further details also see Section 3.3.5).

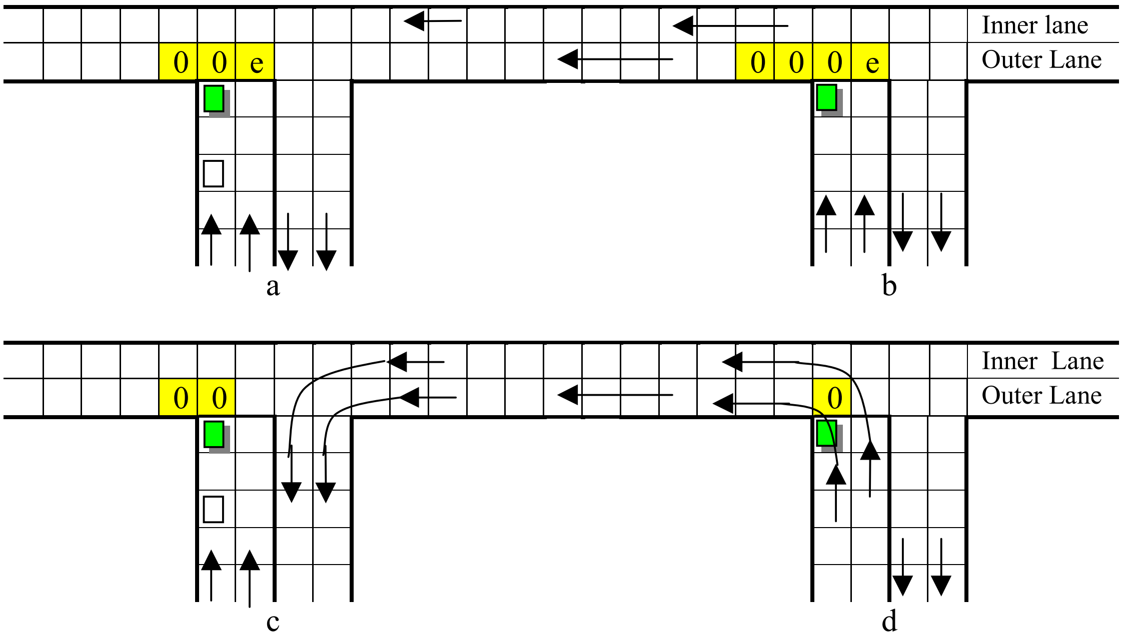


Figure 6.1. Vehicle on the left-lane of the entrance road with behaviour of (a) rational, (b) conservative, (c) urgent and (d) radical

The requirement for each cell is indicated by “0” or “e”, where “ 0” means that the cell must be vacant and “e” means that the cell is either vacant or occupied by a non-circulating vehicle. A non-circulating vehicle is a vehicle that is either just entering the roundabout from an entry road or going to leave the roundabout in next time step.

Figure 6.2 indicates the requirements for the vehicles on the right-lane of the entry road to enter the roundabout. Obviously, they need space in both lanes. All space requirements are indicated cell by cell (and with the same notation “0” or “e”).

We assume that drivers use similar space requirements for each lane in the figures, e.g. in Figure 6.2 (a). MAP covers 3 cells in both outer-lane and inner-lane. The similar space requirement for each lane is also simpler to model than rules that a driver requires different space for different lanes, because there are 4X4 combinations of rules.

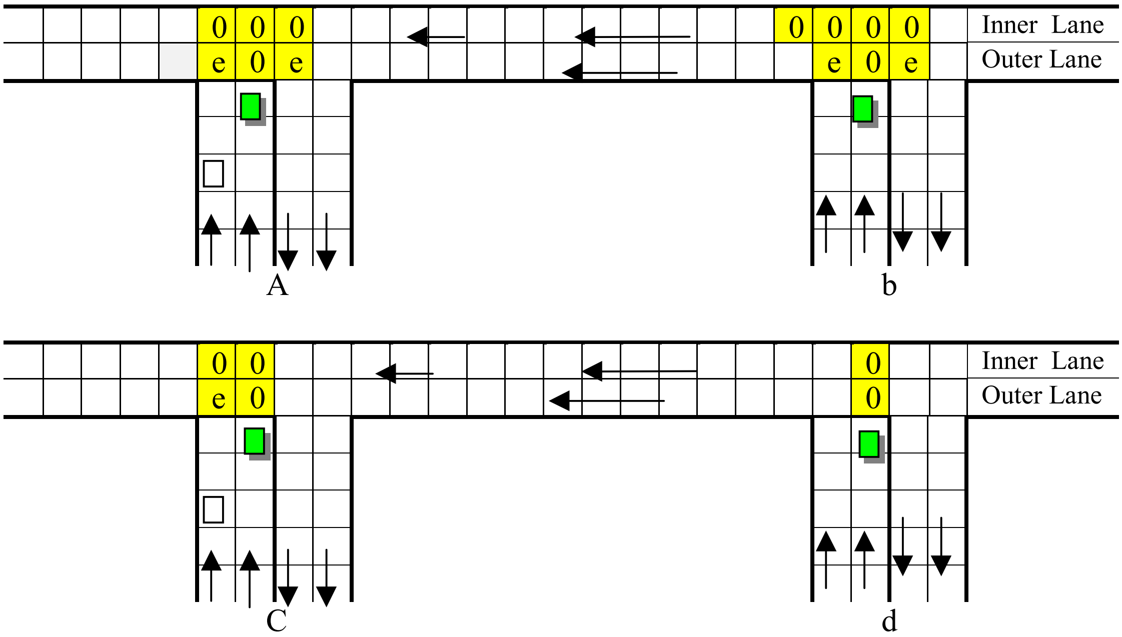


Figure 6.2. Vehicle on the right-lane of the entrance road with behaviour of (a) rational, (b) conservative, (c) urgent and (d) radical

The assumption of the similar space requirement for each lane is justified by the argument that drivers' heterogeneous behaviour is partially determined by their types and individual characteristics, such as sex, age and driving experience, amongst others (Teply et al. 1997), and not by their location -- different lanes. Thus, drivers who accept a small space in one lane are likely to accept a small space in another lane. The investigation of Nishida (1999) also supports Teply's (1997) argument that age is an important factor in determining not only driver reaction time but also driving behaviour. However, it may be still an open question whether drivers do use the same space criteria in each lane. Wilde (1982) suggested that a driver who accepts a small gap in one lane is more likely to use a larger gap in the other lane in order to compensate for the risk.

Hagring (1998 and 2000) also suggested that drivers use larger gaps in the near-lane and smaller gaps in the far-lane. One reason offered for this was that there are two different types of interactions involved: crossing and merging. The crossing interaction is difficult to perform and takes more time, but the merging interaction is easier and needs less time. Another explanation was that the speeds that the entering vehicle can reach to pass the near-lane and far-lane are different, i.e. when the vehicle merges with the far lane, the speed is higher than the speed of passing the near-lane. Golias (1981) and McDowell et al. (1983) however reported the opposite result namely that the critical gap in a far-lane was larger than that for the near-lane.

Our view is that all possibilities reflect the individual driver. A “risk-taker” takes the same amount of risk either way, no matter whether the risk is equally or unequally distributed between the two lanes (in agree with Wilde (1982)). On the other hand, a “risk-averse” decision implies equal caution in both lanes. As the gaps in the gap-acceptance models are not equal to our MAP (see Chapter 3), the assumption of equal space requirements in each lane can be seen as a compromise.

6.3.4 Interaction on roundabouts

Immediately after entering the roundabout (at the next time step), the vehicles from the right-lane of the entry roads move from the outer-lane into the inner-lane. In our model, they move along the inner-lane until they arrive at the destination (exit road). We assume that they do not change lane except for entering and exiting for simplicity. This assumption agrees with our observations conducted at the roundabouts on N2 road under M50 road and on N1 road and the North Street in April, June and September 2002, in Dublin. In a total of 15 hours (5 rush hour and 10 non-rush hour), we found that less than 3% vehicles change lane not for entering and exiting on the roundabout.

A few vehicles are found to move from the inner-lane to the outer-lane earlier then they need to, when approaching the exit road. In other words, the driver will change lane shortly before exiting the roundabout when s/he finds that the outer lane is vacant, but still earlier than s/he needs to. Since such lane-changing occurs shortly before the exit, (i.e. it does not confuse previous exit/entrance movements), the exiting vehicle should have no effect on any entering vehicle. Thus, this phenomenon does not violate the earlier assumption and the overall performance of the roundabout does not change.

When some straight-going vehicles can drive on the outer-lane of the roundabout, the vehicles driving on the inner-lane may be blocked by the vehicles driving on the outer-lane. Theoretically, this blockage should not occur since according to the rules of the road, vehicles on the outer-lane need to give way to vehicles driving on the inner lane. However, blockages is common. Particularly when the vehicle in the inner-lane is less than half a car ahead of the vehicle in the outer-lane. In this case, the vehicle in the outer-lane may or may not give way to the vehicle in the inner-lane depending on the interaction between them. In our model, we use a probability (**give-way rate**) to simulate this random result of driver interaction. The probability is either/or i.e. 0 (no driver gives way) and 1 (all drivers give way). Although we have not determined the value of this probability explicitly, we can use our model to analysis the likely effects of this interaction.

6.4 Two-lane Roundabout Simulation

In order to study the three aspects of roundabout performance, throughput, capacity and queue-length, the following experiments have been carried out. In each experiment, the length of each entrance road is 100 cells. If the **throughput** is printed in **bold**, (as in Table 6.2 for instance), it means that the queue length has reached the length of the road on all entrance roads, i.e. is **saturated**. <u>**Throughputs**</u> (see Table 6.5 and 6.6) in **bold** and <u>underlined</u> means that on at least one entrance road a **saturated situation occurs,** but *not* on all entrance roads. All experiments are carried out for 36,000(= 60 x 60 x 10 = 10 hours) time-steps.

6.4.1 Relationship between the size or shape and throughput

In Chapter 5, we give intuitive proofs of theorems on the relationship between optimum density, the size of a roundabout and throughput. In this section, we use our new models to test if the relationship is still valid for two-lane roundabouts. Since we have three models (Models A, B and C—see Section 6.3.1) for two-lane roundabouts, (each of which have different lane-allocation patterns), we test these individually.

The experiments are set up based on the same topology: we consider just a four-arm roundabout (four entrances/exits), but of different size measured in terms of the number of cells in column 1 of Tables 6.1 and 6.2. In Table 6.1 for Model A for example, the mean of left-turning rate (LTR) , straight-through rate (STR) and right-turn rate (RTR) are 0.25, 0.5 and 0.25 respectively, and all use optimal entry conditions. Also, the shapes of the roundabouts are different (asymmetric) in that the distances between the entrances are different. Arrival rates (AR) of all entrance roads are 0.2, 0.25, 0.30 and 0.35 (equivalent to 720, 900 and 1080, 1260 vph respectively).

Model A: Five (from over 100) sets of experimental results of Model A are shown in Table 6.1, in which the numbers of cells are even, i.e. 24, 28, 36, 38 and 50. In the first two experiments, the distances between entrances are equal (**equal-spacing**), but sizes are different. In the third and fourth experiments, the distances between entrances are varied (**non-equal-spacing**) and overall sizes are also different. In the fifth experiment, the size is 50 cells and again non-equal spacing applies.

Table 6.1: Throughput for Model A when the numbers of cells of roundabouts are even and the topologies and turning rates are the same

Size (cells)	A1 to A2 (cells)	A2 to A3 (cells)	A3 to A4 (cells)	A4 to A1 (cells)	Throughput1 AR =0.20	Throughput2 AR=0.25	Throughput3 AR =0.30	Throughput4 AR=0.35
24	6	6	6	6	2886	3594	**4149**	**4334**
36	9	9	9	9	2879	3591	**4184**	**4329**
28	6	5	7	10	2883	3591	**4161**	**4336**
38	7	5	8	18	2878	3616	**4162**	**4333**
50	5	15	10	20	2877	3602	**4156**	**4334**

A1 to A2 is the distance between the first and the second entrance of the roundabout. Throughput (vph) is the throughput when the means of all arrival rates (AR) = 0.20, 0.25,0.30 and 0.35.

We find that throughput in each column (with the same arrival rates) is very similar, despite different sizes and shapes. The throughput increases when arrival rates increase. Throughput in each column appears neither to depend on equal- or unequal-spacing, nor on the sizes of the roundabouts, providing turning rates and topologies are the same. Thus results for Model A are similar to the results for the single-lane roundabout in Chapter 5. When the arrival rates are lower (≤ 0.25), we see no saturated situations in any lane of any entrance road. When the arrival rates ≥ 0.30, queues appear on all right-lanes of all entrance roads.

In Table 6.2, the number of cells is odd, 25, 37, 45 and 55. Non-equal-spacing applies throughout. We also find that throughputs in the first two columns (with arrival rates of 0.20 and 0.25) are not different, although sizes and shape differ. However, throughputs in each column of the last two columns (with arrival rates of 0.30 and 0.35) increased with the size of the roundabout. When arrival rates ≥ 0.30, queues appear again on all right-lanes of all entrance roads.

Table 6.2: Throughput for Model A when the numbers of cells of roundabouts are odd and the topologies and turning rates are the same

Size (cells)	A1 to A2 (cells)	A2 to A3 (cells)	A3 to A4 (cells)	A4 to A1 (cells)	Throughput AR =0.20	Throughput AR=0.25	Throughput3 AR =0.30	Throughput4 AR=0.35
25	6	6	6	7	2885	3612	**4070**	**4241**
37	6	7	10	14	2879	3616	**4086**	**4246**
45	6	9	9	21	2881	3592	**4096**	**4257**
55	6	14	10	25	2873	3596	**4108**	**4271**

Comparing Table 6.1 and 6.2, all throughputs in the Throughput3 column in Table 6.1 are larger than those in the Throughput3 column in Table 6.2. A similar situation can be found for the columns of Throughput4. These results are expected according to the theorems of the optimum density on the roundabout, because the numbers of cells (= sizes) of roundabouts in Table 6.1 are even and thus the optimum density can be achieved.

The results could be explained as follows:

- When arrival rates (≤0.25) are low, there are no queues on any entrances and no saturated situations exist. Throughputs are thus equal to the number of vehicles that arrive at the roundabouts. Consequently, there will be no difference between throughputs, regardless of whether the size is even or odd, large or small.

- When arrival rates ≥ 0.3), the queues form on the right-lanes of all entrance roads (saturation). Traffic flow on the left-lanes of all entrance roads is, however, free flow. The throughput is thus dictated by the operation of the inner-lane of the roundabout. The inner-lane of the roundabout can be seen as a single-lane roundabout. The only different is that there are no LT vehicles on it.

It is not surprising, therefore, that our theorems are proved to be applicable in Model A, (again conclusions apply for an ideal situation, i.e. uniform size and speed of vehicles).

Model B: Ten sets of experimental results for Model B are shown in Table 6.3, in which the numbers of cells are even, i.e. 24, 28, 32,36, 40, 44 and 48. The distances between entrances are equally-spaced for the first seven rows, but sizes are different. The last three rows are for non-equally-spaced and different sizes.

Table 6.3: Throughput for Model B when the numbers of cells of roundabouts are even, equal spacing, same topologies and turning rates.

Size (cells)	A1 to A2 (cells)	A2 to A3 (cells)	A3 to A4 (cells)	A4 to A1 (cells)	Throughput1 AR=0.35	Throughput2 AR=0.40	Throughput3 AR=0.45	Throughput4 AR=0.50
24	6	6	6	6	5039	5757	**6005**	**6036**
28	7	7	6	8	5038	5757	**6007**	**6046**
32	8	8	8	8	5042	5056	**6010**	**6066**
36	6	9	9	12	5041	5756	**6029**	**6069**
38	10	10	10	8	5039	5756	**6037**	**6073**
40	11	11	11	11	5040	5761	**6042**	**6074**
48	7	10	7	14	5038	5758	**6048**	**6075**

We find that the throughput increases as the arrival rates increase for the same roundabout. We also find that throughput in the *first two throughput columns* (with arrival rates of 0.35 and 0.40) are not different, although sizes differ.

However, throughput in each column of the *last two throughput columns* (with arrival rates of 0.45 and 0.50) increased with the size of the roundabout for both equal and non-equal-spacing.

Four sets of experimental results of Model B are shown in Table 6.4, in which the numbers of cells are odd, i.e. 25, 37, 45, and 55. We also find that in the last two throughput columns (with arrival rates of 0.45 and 0.50) values increase with the size of the roundabout. We can see by comparing column 3 in Table 6.3 and Table 6.4, that throughputs with an even number of roundabout cells (N_{even}) may be larger than the throughputs with odd number of cells (N_{Odd}), even if the N_{even} <. $N_{Odd.}$

The relationship between the size and throughput of Model B is different from that of Model A. The explanation of this difference appears due to the difference of lane-allocation processes between two models. However, further theoretical analysis on why and how lane-allocation processes cause this difference are needed.

Table 6.4: Throughput (vph) for Model B when the numbers of cells of roundabouts are odd and the topologies and turning rates are the same

Size (cells)	A1 to A2 (cells)	A2 to A3 (cells)	A3 to A4 (cells)	A4 to A1 (cells)	Throughput AR=0.35	Throughput AR =0.40	Throughput AR=0.45	Throughput AR=0.50
25	6	6	6	7	5034	5758	**6003**	**6038**
37	6	7	10	14	5021	5733	**6017**	**6049**
45	6	9	9	21	5051	5737	**6028**	**6059**
55	6	14	10	25	5049	5742	**6036**	**6075**

Comparing Table 6.1 and 6.2 with Table 6.3 and 6.4, we find that Model B has a better operational performance with higher throughput when arrival rates ≥. 0.30. Particularly, when 0.45 ≥ arrival rates ≥ 0.30, saturation occurs for Model A, but not for Model B.

6.4.2 Relationship between throughput and arrival rates

Table 6.5 and Figure 6.3 show that throughputs vary with arrival rates for Model A for the numerical simulations reported. Arrival rates of three roads (AR_2, AR_3 and AR_4) are taken to be the same and allowed to range from 0.15 (540vph) to 0.45 (1620vph). Arrival Rate of road 1 (AR_1) also increases from 0.10 (360vph) to 0.55 (1980vph).

For AR_2=AR_3 =AR_4< 0.30, we find that throughput increases linearly as arrival rate of road 1 (AR_1) increases, when no entrance is saturated. For example, for AR_2=AR_3

$=AR_4= 0.15$, when $AR_1 \leq 0.45,$ road 1 is in free flow, but when $AR_1 \geq 0.45$, road 1 is saturated and throughputs continues to rise with a maximum at $AR_{1\ max.}$

Table 6.5: Throughputs vs. arrival rates for Model A.

$AR_2 =$	$AR1_1$									
$AR_{3,} = AR_4$	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55
0.10	1438	1621	1797	1977	2160	2339	2520	2697	**2877**	**2939**
0.15	1982	2165	2337	2519	2698	2886	3067	**3190**	**3238**	**3297**
0.20	2518	2702	2879	3058	3240	3394	**3503**	**3561**	**3613**	**3625**
0.25	3053	3233	3416	3596	**3778**	**3837**	**3891**	**3937**	**3972**	**4016**
0.30	**3422**	**3615**	**3818**	**3999**	**4156**	**4205**	**4242**	**4290**	**4350**	**4382**
0.35	**3654**	**3844**	**4033**	**4176**	**4307**	**4350**	**4391**	**4434**	**4466**	**4508**
0.40	**3903**	**4086**	**4202**	**4311**	**4430**	**4470**	**4523**	**4565**	**4612**	**4654**
0.45	**4145**	**4226**	**4334**	**4431**	**4565**	**4608**	**4649**	**4686**	**4724**	**4764**

For $AR_2=AR_3=AR_4 \geq 0.3,$ we find also that road 1 saturates only when $AR_1 \geq 0.3,$ whereas even for a negligible entry rate for road 1, increase in arrival rates on the other three roads ($AR_2=AR_3=AR_4 \geq 0.3$) means that at least one entrance road of the roundabout is saturated.

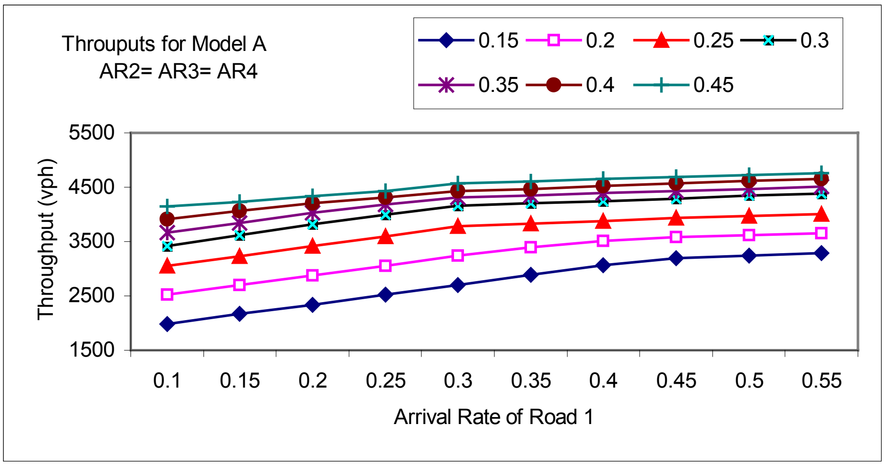


Figure 6.3 Throughputs vs. arrival rates for Model A.

The above findings for Model A can be summarised in the following three expressions. When the arrival rate of the entry road ≥ **critical arrival rate** *(CAR),* saturation occurs on the entry road. The empirical relationships between CAR_1 and arrival rates of other three roads is:

- If $AR\ i= 0.05$, then $CAR1 \geq 0.60$ (6.1)

- If $0.05<ARi\leq 0.20$ and ARi >0.05, then $CAR1=0.6 - ARi$ (6.2)
- If $ARi=\geq 0.25$, then $CAR1 =0.3$ (6.3)

where i = 2, 3 or 4.

The above CAR formulae also can be expressed in terms of vph.

- If AR_i = 180 vph, than CAR_1 =2160 vph (6.4)
- *If* AR_i < 1080 vph, then $CAR_1=2160 - AR_i$ (6.5)
- *If* $AR_i \geq 1080$ vph, then CAR_1 =1080 vph (6.6)

where i = 2, 3 or 4.

Throughput of the Model A two-lane roundabout continues to increase with arrival rate when all roads are *saturated* (i.e. arrival rate > CAR). The situation is *different* from that in single-lane roundabouts. Because Model A only allows LT vehicles to use the left-lane of entrance road, traffic on the left-lane is always free flow. Therefore, when arrival rates increase, the number of LT vehicles continues to increase. Consequently throughput also increases.

Table 6.6 and Figure 6.4 show that throughputs change with arrival rates for Model B. Again, arrival rates of three roads (AR_2, AR_3 and AR_4) are taken to be the same, but vary from 0.20 (720vph) to 0.65 (2340vph). The arrival rate of road 1 (AR_1) increases from 0.25 (900vph) to 0.65 (2340vph).

Table 6.6 Throughputs vs. arrival rates for Model B.

$AR_2 = AR_3$, $=AR_4$	$AR1_1$									
	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
0.25	3414	3608	3783	3943	4111	4297	4501	4685	**4832**	**4848**
0.30	3964	4141	4293	4497	4690	4861	5033	**5166**	**5163**	**5163**
0.35	4478	4699	4848	5038	5211	5384	**5484**	**5508**	**5507**	**5508**
0.40	5025	5224	5403	5572	5764	**5867**	**5872**	**5865**	**5874**	**5856**
0.45	**5357**	**5520**	**5685**	**5836**	**5962**	**6012**	**6035**	**6051**	**6034**	**6045**
0.50	**5460**	**5603**	**5753**	**5886**	**6009**	**6058**	**6053**	**6070**	**6077**	**6073**
0.55	**5553**	**5728**	**5832**	**5910**	**6023**	**6080**	**6088**	**6094**	**6105**	**6105**
0.60	**5673**	**5765**	**5851**	**5953**	**6036**	**6081**	**6100**	**6109**	**6112**	**6109**
0.65	**5703**	**5783**	**5851**	**5959**	**6042**	**6098**	**6112**	**6114**	**6127**	**6132**

For $AR_2=AR_3 =AR_4< 0.45$, we find that throughput increases linearly as arrival rate of road 1 (AR_1) increases, when no entrance road is saturated. For example, for

AR_2=AR_3 =AR_4= 0.25, $AR_1 \leq 0.60$, road 1 is in free flow, but when $AR_1 \geq 0.60$, road 1 is saturated and throughput is a constant (different from Model A) and maximum.

For AR_2=AR_3 =$AR_4 \geq 0.45$, we find also that road 1 is saturated only when $AR_1 \geq 0.45$. If $AR_1 < 0.45$, at least road 1 is free flow. When AR_2=AR_3 =$AR_4 \geq 0.45$, even though $AR_1 = 0.0$, at least one entrance road of the roundabout is in situated situation. If three road arrival rates are equal, they should then be less than 0.45. Otherwise saturation will occur on at least one entrance road.

For AR_2=AR_3 =$AR_4 \geq 0.45$, throughput increases with AR_1, when $AR_1 < 0.45$. Throughput is approximately constant when $AR_1 \geq 0.45$. Throughput shows little difference when all arrival rates ≥ 0.45.

The above findings for Model B can also be summarised in the following expressions. The empirical relationship between CAR and arrival rates of other three roads is:

- *If* $AR_i < 0.45$, *then* $CAR_1 = 0.8 - AR_i$ (6.7)
- *If* $AR_i \geq 0.45$, *then* $CAR_1 = 0.45$ (6.8)

where i = *2, 3 or 4.*

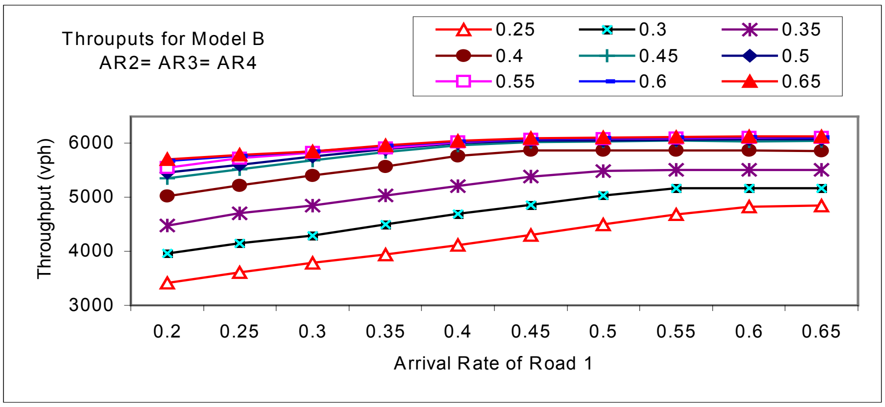


Figure 6.4 Throughputs vs. arrival rates for Model B.

The above CAR formulae also can be expressed in terms of vph.

- *If* $AR_i < 1620$ *vph, then* $CAR_1 = 2880 - AR_i$ (6.9)

- *If $AR_i \geq 1620$ vph, then $CAR_1 = 1620$ vph* (6.10)

  where i = 2, 3 or 4.

We also find that balanced arrival rates (AR_1=AR_2=AR_3=AR_4), lead to improvement in operational performance of the roundabout for both Model A and Model B. Again, if we define the **effective throughput** as the throughput when no entrance road is saturated, *the maximum effective throughput (MET)* that we find is 3665 vph for model A when AR_1=AR_2=AR_3=AR_4=*0.28,* and 5806 vph for model B when AR_1=AR_2=AR_3=AR_4=*0.43* . When arrival rates are *not* equal, the effective throughput is *less* than optimal.

6.4.3 Relationship between throughput and turning rates

For situations when cars are driving on the left-hand side of roads, such as in the UK and Ireland, the relationship between throughput and turning rates of a roundabout can be observed from the tables and figures below. The data generated are based on a 32-cell 4-road-two-lane roundabout with AR_1=AR_2=AR_3=AR_4.

Table 6.7: Throughputs of the roundabout for $AR_1=AR_2=AR_3=AR_4$ and a right turning rate between 0.05 to 0.45. Straight through rates are 0.5 for Model A

AR_1=AR_2=AR_3 =AR_4	Right Turning Rate				
	0.05	0.15	0.25	0.35	0.45
0.15	2161	2158	2165	2162	2160
0.20	2876	2877	2879	2881	2871
0.25	3604	3595	3596	**3515**	**3083**
0.30	4333	4307	**4156**	**3627**	**3121**
0.35	5040	**4954**	**4350**	**3728**	**3150**
0.40	5750	**5234**	**4523**	**3841**	**3194**
0.45	**6345**	**5478**	**4686**	**3961**	**3234**

Table 6.7 and Figure 6.5 show the relationship between throughput and turning rates for Model A. The mean ST rate (STR) is the same (= *0.5*). The mean right turning rates (RTR) is allowed to increase from *0.05* to *0.45* in increments of *0.05*, while LT rates (LTR) consequently vary from *0.45* to *0.05.* As the arrival rate increases from 0.15 to 0.45, we see the traffic on the entry road (to the roundabout) changes from free flow to saturation.

In Table 6.7, when $AR_1=AR_2=AR_3=AR_4 < 0.25$ (in row 1, 2 and 3), traffic flows freely and turning rates have no impact on throughput. When $AR_1=AR_2=AR_3=AR_4= 0.25$ (row 3) and RTR is *0.25*, traffic still flows freely. However, when RTRs are equal to *0.35* and higher, entrance roads are saturated and turning rates do have some effect on throughputs. When $AR_1=AR_2=AR_3=AR_4>0.25$ (in rows 5, 6 and 7), the turning rate also affects throughput: When RTR increases by *0.10*, this gives around a *15%* decrease in throughput when entrance roads are saturated.

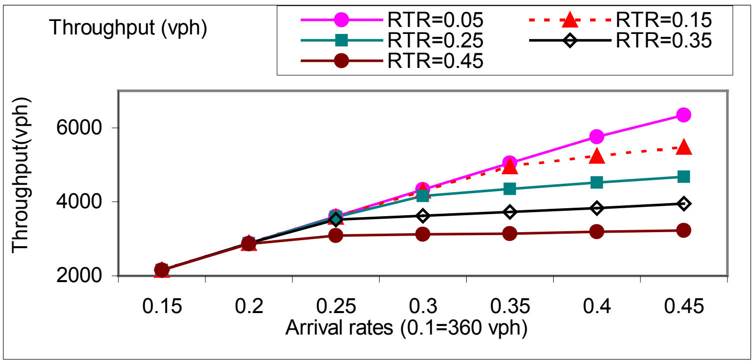


Figure 6.5 Throughputs change vs. right-turning rate for Model A

In Figure 6.5, the curve of RT=0.05 (RTR=0.05) increases linearly until $AR_1=AR_2=AR_3=AR_4$ = 0.45, and traffic flows freely ($AR_1=AR_2=AR_3=AR_4$ <0.45). Thus, 0.45 is the CAR for the turning rate. CARs for RTR=0.05, 0.15, 0.25, 0.35 and 0.45 are 0.45, 0.35, 0.3 0.25 and 0.25 respectively. All curves follow the same pattern of increase with arrival rates when $AR_1=AR_2=AR_3=AR_4$ > CAR.

Table 6.8: Throughputs of the roundabout for $AR_1=AR_2=AR_3=AR_4$ and RTR is between 0.05 to 0.45. STRs are 0.5 for Model B

$AR_1=AR_2=AR_3=AR_4$	Right Turning Rate				
	0.05	0.15	0.25	0.35	0.45
0.2	2879	2876	2878	2881	2878
0.25	3597	3600	3598	3596	3590
0.3	4317	4313	4318	4312	4327
0.35	5038	5044	5038	5047	**4765**
0.4	5758	5754	5764	**5408**	**5057**
0.45	6477	6476	**6054**	**5438**	**5070**
0.5	7196	**6729**	**6095**	**5476**	**5066**
0.55	**7653**	**6793**	**6113**	**5504**	**5067**
0.6	**7699**	**6809**	**6133**	**5503**	**5072**

Table 6.8 and Figure 6.6 show the relationship between throughput and turning rates for Model B. Again, the mean ST rate (STR) remains 0.5, and the mean of RTR and LTR change over a wider range compared to Table 6.7.

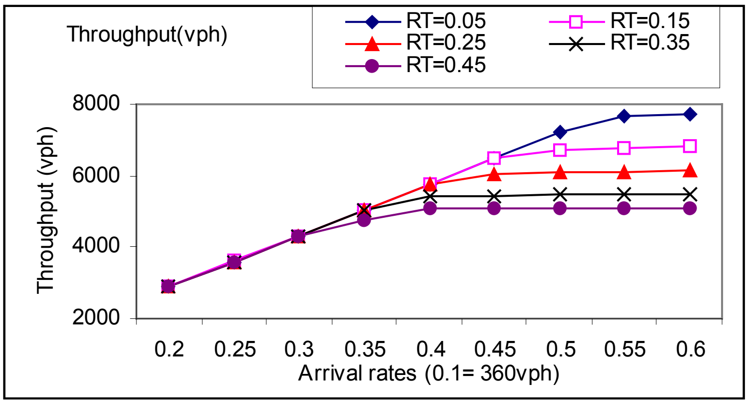


Figure 6.6 Throughput vs. RTR rate for Model B

In Table 6.7, when $AR_1=AR_2=AR_3=AR_4 < 0.3$ in rows 1 to 3, traffic flows freely, and turning rates have no impact on throughput. When $AR_1=AR_2=AR_3=AR_4= 0.3$ (in row 5) and RTR is 0.35, traffic still flows freely. However when RTRs are equal to 0.45, entrance roads are saturated and turning rates do have an effect on throughput. When $AR_1=AR_2=AR_3=AR_4>0.3$, turning rate also has an effect on throughput: When RTR is increased by 0.10 this gives around a 10% decrease in throughput when entrance roads are saturated. The relationship between RTR and its CAR can be roughly expressed by the following empirical relation.

$$\text{CAR (for } AR_1=AR_2=AR_3=AR_4) = 0.4 - 0.5(RTR - 0.35) \qquad (6.11)$$

6.4.4 Queue formation: roundabout and individual road

In the experiments in Section 6.4.2, the operational performance of throughput has been studied. The same experiments also reveal the relationship between the queue formation and arrival rates of a roundabout.

Table 6.9: Queue-formation vs. arrival rates for Model A.

$AR_2 = AR_3$, $=AR_4$	$AR1_1$									
	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55
0.10	Free	Free	Free	Free	Free	Free	Free	Free	**F234**	**F234**
0.15	Free	Free	Free	Free	Free	Free	Free	**F234**	**F(2)34**	**F(2)34**
0.20	Free	Free	Free	Free	Free	Free	**F234**	**F234**	**F(2)34**	**F34**
0.25	Free	Free	Free	Free	**F234**	**F34**	**F34**	**F34**	**F34**	**F34**
0.30	**F123**	**F12**	**F12**	**F12**	**S**	**S**	**S**	**S**	**S**	**S**
0.35	**F12**	**F12**	**F1**	**F1**	**S**	**S**	**S**	**S**	**S**	**S**
0.40	**F12**	**F1**	**F1**	**F1**	**S**	**S**	**S**	**S**	**S**	**S**
0.45	**F1**	**F1**	**F1**	**F1**	**S**	**S**	**S**	**S**	**S**	**S**

We name the four entrance roads of a roundabout clockwise, i.e. a RT vehicle from road 1 for example, enters the roundabout from road 1 and passes road 2 and road 3, and exits at road 4. When *all* entrance roads are flowing freely or not fully saturated (the length of queue is less than the length of road (100 cells)), we denote this situation as "*Free*" (see Table 6.9). If *only* road 3 and 4 are flowing freely or are not fully saturated, we denote this as F34. The designation F(2) means that road 2 has been saturated in some but not all experiments. "S" means all entrance roads are fully saturated for *all* roads.

Table 6.10 Queue formation vs. arrival rates for Model B.

$AR_2 =$ AR_3, $=AR_4$	$AR1_1$									
	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
0.25	Free	Free	Free	Free	Free	Free	Free	Free	**F234**	**F234**
0.30	Free	Free	Free	Free	Free	Free	Free	**F234**	**F234**	**F234**
0.35	Free	Free	Free	Free	Free	Free	**F234**	**F234**	**F234**	**F234**
0.40	Free	Free	Free	Free	Free	**F234**	**F234**	**F234**	**F234**	**F234**
0.45	**F12**	**F12**	**F12**	**F1(2)**	**F1**	**S**	**S**	**S**	**S**	**S**
0.50	**F12**	**F12**	**F1**	**F1**	**F1**	**S**	**S**	**S**	**S**	**S**
0.55	**F12**	**F1(2)**	**F1**	**F1**	**F1**	**S**	**S**	**S**	**S**	**S**
0.60	**F1(2)**	**F1**	**F1**	**F1**	**F1**	**S**	**S**	**S**	**S**	**S**
0.65	**F1**	**F1**	**F1**	**F1**	**F1**	**S**	**S**	**S**	**S**	**S**

Queue-formation for all entrance roads of Model A and Model B are shown in Table 6.9 and Table 6.10. We find that road 1 is *always* in free flow or not fully saturated when $AR_1 \leq 0.25$ for Model A, $AR_1 \leq 0.65$ for Model B and $AR_2=AR_3=AR_4$ (regardless of their value). Also road 2 may be free flowing as well (depending on the arrival rate of road 1). If $AR_2=AR_3=AR_4 \leq 0.25$ for Model A and $AR_2=AR_3=AR_4 \leq 0.40$, roads 3 and 4 are always in free flow or are not fully saturated (regardless of the arrival rate on road 1). With the same conditions, road 2 may be free flowing for Model A, but road 1 is always free.

6.4.5 Individual road performance—queue lengths

For individual roads (as in model A), queues can form only on the right-lane of an entrance road. For Model B, two lanes of an entrance road have to be studied, as queues can form on both lanes.

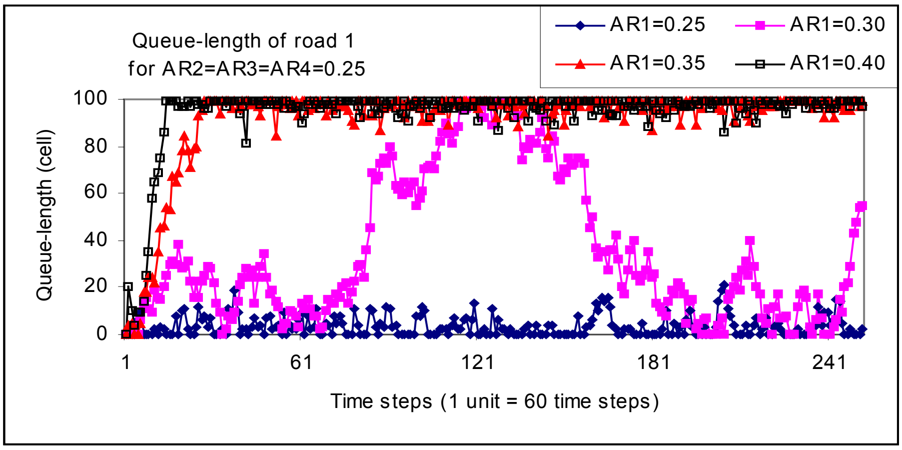


Figure 6.7. Queue-length of road 1 varied with AR_1 for Model A. AR_1 increases from 0.2 to 0.40 when $AR_2=AR_3=AR_4 = 0.25$

In Figure 6.7, queue-lengths of the right-lane of road 1 change with AR_1, (for Model A). This gives us a clear picture of how critical arrival rate corresponds to the saturation. Figure 6.7, $AR_2=AR_3=AR_4= 0.25$, AR_1 increases from 0.25 to 0.4. As $CAR= 0.30$, Figure 6.7 shows the queue formation pattern for AR_1 less than, equal to and greater than CAR.

When AR_1 is below the critical arrival rate, queues to enter the roundabout will usually be short and frequently there are no cars waiting to enter. This means that throughput will be less than the maximum. When AR_1 = CAR, we find that the queues build up slowly (Figure 6.7, AR_1=0.3). It takes about 120x60 time steps (=2 hours) for $AR_1=0.30$ to result in a queue of up to 100 cells. The queue can also disappear after another 120X60 time steps (=2 hours).

When $AR_1 \geq$ CAR, queues are built up very quickly. It takes less than 30x60 time-steps (= 1/2 hour) for $AR_1=0.35$ to reach 100 cells, whereas it takes about 15x60 time-steps (= 1/4 hour) for $AR_1=0.40$ to reach the same length. The speed of formation of the queue increases as AR_1 increases. The queue reaches the maximum length very rapidly for any AR >> CAR (similar to findings for single-lane roundabouts).

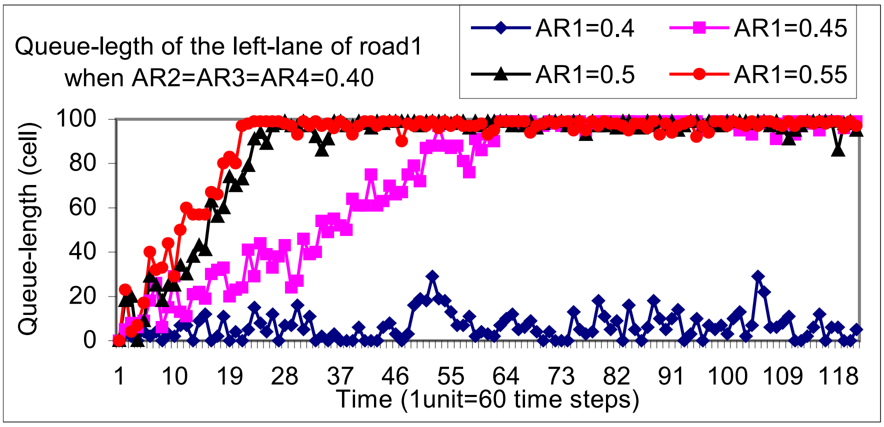


Figure 6.8 Queue-lengths of the left-lane of road 1 vs. AR_1 for Model B

Figure 6.8 and 6.9 show how the queue-lengths of the left- and right-lane of road 1 change with AR_1 for Model B. These give us a picture of how critical arrival rates correspond to the saturation on both lanes of the road. In Figure 6.8, $AR_2=AR_3=AR_4 = 0.40$, AR_1 increases from 0.40 to 0.55. As $CAR= 0.45$, Figures 6.8 and 6.9 show the queue formation patterns on both lanes when AR_1 less than, equal to and greater than CAR_1.

We find that both lanes have a similar queue-formation pattern. Since the lane-allocation process is based on the queue-length of each lane, ST vehicles are allocated to the lane with the shorter queue. (It is unsurprising that both lanes have the same pattern, but does show that the model is operating as intended).

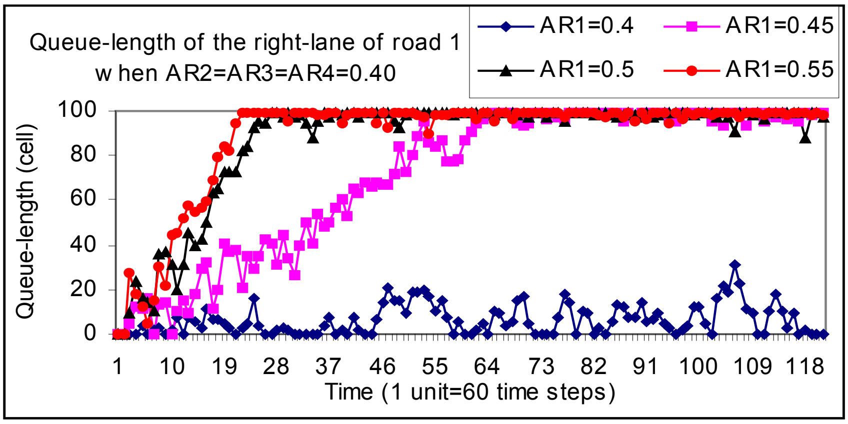


Figure 6.9. Queue-lengths of the right-lane of road 1 vs. AR_1 for Model B

Again when AR_1 is below the critical arrival rate, the queues are usually short or non-existent. When AR_1 = CAR, queues build up slowly (Figure 6.8 AR1=0.45). It takes about 60x60 time steps (=1 hour) for $AR_1=0.40$ to result in a queue of up to 100 cells. The queue will not vanish unless AR decreases.

When $AR_1 \geq$ CAR, queues again build quickly. It takes less than 30x60 time-steps (= 1/2 hour) for $AR_1=0.50$ to reach 100 cells. It also takes about 30x60 time-steps (= 1/2 hour) for $AR_1=0.55$ to equal the same length.

6.4.6 Position Delay Time (PDT) and give way on the roundabout

Table 6.11. Throughputs vs. PDT for Model A

PDT	Arrival Rates ($AR_1 = AR_2 = AR_3 = AR_4$)					
	0.35	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**
0	**4336**	**4534**	**4700**	**4884**	**5056**	**5236**
2s	**4342**	**4519**	**4705**	**4878**	**5057**	**5247**
4s	**4337**	**4516**	**4701**	**4877**	**5060**	**5246**
6s	**4341**	**4521**	**4710**	**4881**	**5066**	**5253**

For Model A, the PDT has little effect on the throughput (Table 6.11), as only LT vehicles occupy the left-lane of entrance roads. However, for Model B, nearly two thirds of vehicles that enter an entrance road finally pass through the right-lane (Table 6.12). Therefore, PDT is likely to have considerable effect on the entrance capacity of each road and overall performance-throughput for Model B.

Table 6.12. Throughputs vs. PDT for Model B

PDT	Arrival Rates (AR_1= AR_2=AR_3=AR_4)					
	0.35	**0.40**	**0.45**	**0.50**	**0.55**	**0.60**
0	5041	5754	**6273**	**6324**	**6377**	**6413**
2s	5038	5760	**6012**	**6053**	**6094**	**6112**
4s	5040	**5726**	**5789**	**5817**	**5840**	**5847**
6s	5042	**5643**	**5661**	**5671**	**5685**	**5700**

Table 6.12 and Figure 6.10 show the effects of PDT on throughput. When the arrival rates ≤0.35 and PDT ≤ 6s, there is no difference in throughput. When arrival rates ≥ 0.40, throughput decreases as PDT increases. In particular, when arrival rates = 0.40, the PDT can strongly influence the traffic situation from flowing freely to congestion.

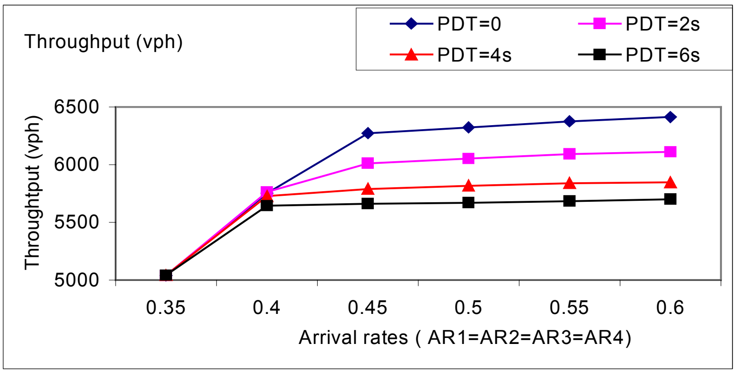


Figure 6.10. Throughputs vs. PDT form Model B

When entry roads are not crowded (arrival rates ≤ 0.35), the position delay of the entering vehicle does not result in blocking of following vehicles. There is also less opportunity for the vehicles in the left-lane of an entry road to experience such delay, as the number of vehicle in the right–lane is typically small. However, if the entry road is crowded, more vehicles or nearly every vehicle in the left-lane of an entry road will experience delay time, and such delay will cause the entering vehicles to further block the vehicles behind. The longer the PDT, the fewer vehicles enter the roundabout. As a result, throughput decreases as PDT increases.

6.4.7 Driver behaviour

The impact of driver behaviour on throughput of Models A and B can be shown in the following experiments. We assume that the sum of probabilities of conservative (P_{co}), rational (P_{ra}), urgent (P_{ur}) and radical (P_{rad}) is equal to 1 as usual. For simplicity, all drivers are of one type in the first instance. These are clearly special situations, which are examined to give us some indication of how extremes of driver behaviour impact on two-lane roundabout performance. A mixed driver set is also possible of course and easily tested with our models.

Table 6.13 Driver behaviour vs. throughput for Model A

Driver behavior	Arrival Rates ($AR_1= AR_2=AR_3=AR_4$)					
	0.20	**0.25**	**0.30**	**0.35**	**0.40**	**0.45**
$P_{co}=1$	2878	**2988**	**3168**	**3356**	**3532**	**3705**
$P_{ra}=1$	2880	3583	**4163**	**4332**	**4516**	**4704**
$P_{ur}=1$	2877	3598	**4161**	**4336**	**4525**	**4702**
$P_{rad}=1$	**765**	**950**	**1074**	**1253**	**1441**	**1646**

Tables 6.13 and 6.13 show the results for Model A and Model B, in which arrival rates are equal in each column. For all $AR = 0.20$ for Model A and all $AR = 0.30$ for Model B in column 1, all throughputs are the same except that of $P_{rad} =1$. When $P_{co} =1$ and $AR_1=AR_2=AR_3=AR_4 \geq 0.25$ for Model A and $AR_1=AR_2=AR_3=AR_4 \geq 0.35$ for Model B, throughput reaches the maximum and a saturated situation occurs on entrance *road*s, while traffic flow on the *roundabout* remains in free-flow at all times. When $P_{ra}=1$ or $P_{ur} =1$, throughputs are similar for Model A (different for Model B), but larger than those for $P_{co} =1$. Traffic flow on the *roundabout* again remains free at all times. When $P_{rad} =$, all AR >0.20 for Model A and all AR >0.30 for Model B, throughputs are reduced compared to others discussed, as congestion forms *on the roundabout.* Similar results are also found with other turning rates.

Table 6.14 Driver behaviour vs. throughput for Model B

Driver behavior	Arrival Rates ($AR_1= AR_2=AR_3=AR_4$)						
	0.30	0.35	0.40	0.45	0.50	0.55	0.60
$P_{co}=1$	4322	**4475**	**4512**	**4552**	**4582**	**4589**	**4620**
$P_{ra}=1$	4320	5038	5764	**6012**	**6053**	**6094**	**6112**
$P_{ur}=1$	4319	5061	5768	**6345**	**6398**	**6434**	**6494**
$P_{red}=1$	**83**	**95**	**62**	**19**	**26**	**19**	**33**

Thus, similar to the conclusion for single-lane roundabouts, collective conservative behaviour decreases throughput. Urgent and rational behaviours give similar performance. In contrast, collective radical behaviour can cause congestion on the roundabout and decrease in throughput compared to rational behaviour. Driver behaviour is clearly not the same for every driver in the real world, so that a distribution of driver behaviour is more appropriate, but our results do reproduce the phenomenon of congestion on a two-lane roundabouts due to too many drivers not observing the give-way rules.

6.5 Three-lane Roundabouts

A vehicle navigating a three-lane roundabout experiences much the same process as it does for a two-lane roundabout (Section 6.2) with the following differences:

- When a vehicle decides its destination, it also decides which lane it needs to takes, as LT, ST or RT vehicles go to the left-, middle- or right-lane respectively. Hence, strictly speaking, lane allocation is automatic.
- A vehicle in the left- or middle- lane experiences different Position Delay Time (PDT) (details see Section 6.3.2)
- Interaction between drivers at the entrances of roundabouts are as shown: (details see Figures 6.11 – 6.13)

Again, we use a similar method and similar figures to explain the conditions that are required by vehicles from entry roads. The required conditions for the target vehicle (shaded) on the left-lane on a three-lane entry road to move onto the roundabout in ***this*** time step are indicated by the space required (shaded cells) in each figure based on different driver behaviour (Figure 6.11). Figure 6.11(d) also shows the path of the vehicle as it enters the roundabout. The requirements for each cell are again indicated by “0” or “e”, where “0” means that the cell must be vacant and “e” means that the cell is either vacant or occupied by a non-circulating vehicle (Section 6.3.3).

Figure 6.12 indicates the requirements for the vehicles in the middle-lane of the entry road to enter the roundabout. Obviously, space is needed in two lanes. Figure 6.12

(b) indicates paths of vehicles leaving roundabouts and Figure (d) shows the path for an entering vehicle from the middle-lane of an entry road.

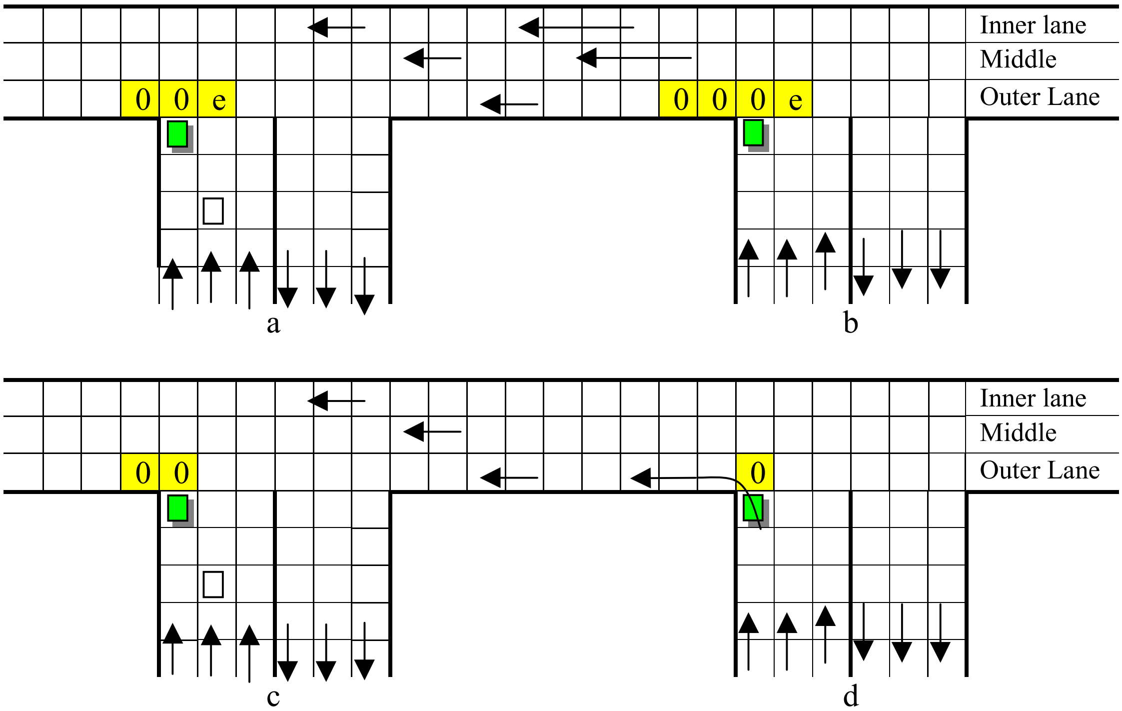


Figure 6.11. Vehicle on the left-lane of the entrance road with behaviour of (a) rational drivers, (b) conservative, (c) urgent and (d) radical

When a vehicle changes lane *on* a roundabout, it moves diagonally across two cells in two lanes. Vehicles change a lane and move forward for one cell at the same time. This applies to all *changing lane movements* on roundabouts. Consequently, LT vehicles, which do not change lanes on roundabouts, are not affected. Equally, vehicles driving in outer-lanes do not need to change lanes on the roundabout either, so are not involved. Therefore only ST and RT vehicles are involved, as they need to change lanes to enter or exit the roundabout.

Figure 6.13 indicates the requirements for the vehicles on the right-lane of an entry road to enter the roundabout. Obviously, space is need in three lanes. Figure 6.13 (d) also indicates a path for an entering vehicle from the right-lane of an entry road.

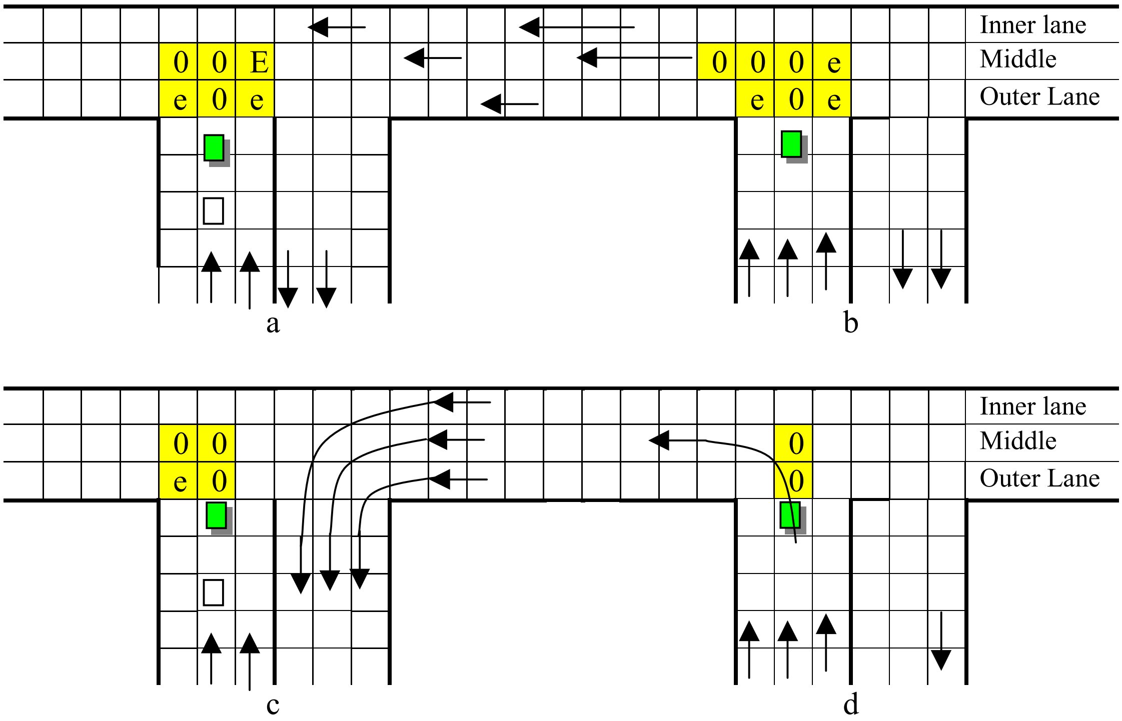


Figure 6.12. Vehicle on the meddle-lane of the entrance road with behaviour of (a) rational drivers, (b) conservative, (c) urgent and (d) radical

Clearly, vehicles on outer and middle- lanes of a roundabout move freely to the exit, (assuming outer-lane exits occur appropriately). However, when vehicles in inner-lanes exit, they have to interact with the vehicles in the middle lanes, (no interaction with the outer-lane is considered, if appropriate outer-lane behaviour is assumed, since movement to the outer-lane presumes that the desired exit is imminent). The interaction is similar to the interaction between vehicles in inner-lanes and outer-lanes for Model B and C (see Section 6.3.4); therefore, a similar give-way rate is applied.

This extension has not been presumed further to date, in part because of space restrictions on large roundabouts in urban areas, but the prototype model appears to perform reasonably.

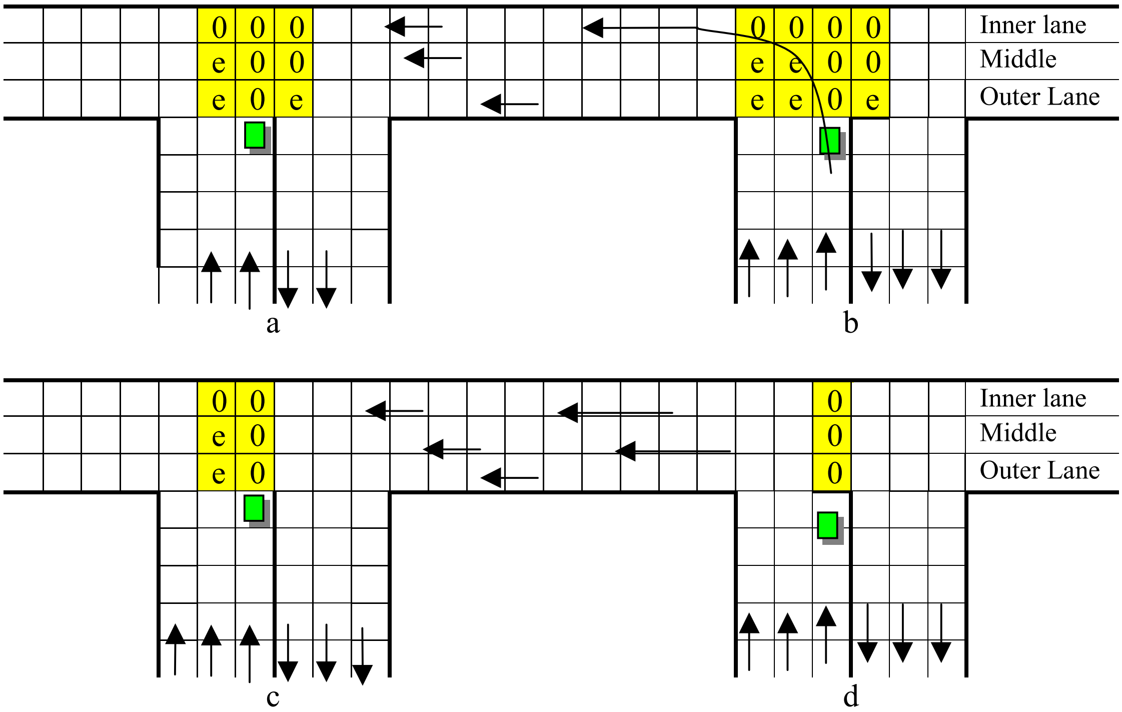


Figure 6.13. Vehicle on the right-lane of the entrance road with behaviour of (a) rational drivers, (b) conservative, (c) urgent and (d) radical

6.6 Summary

Three two-lane roundabout models are developed with different lane-allocation patterns. Various properties of two-lane roundabout operations have been explored including throughput, turning rates, critical arrival rates the queue formation process, together with variations of queue lengths and congestion on the roundabout itself.

For Model A, conclusions are similar for the relationship between throughput and size of a two-lane roundabout to those obtained for single-lane roundabouts. If the number of cells of the roundabouts is even, then throughput does not depend on roundabout size, equal-spacing or non-equal-spacing, given similar topology and other parameters held constant. If the number of cells of the roundabouts is odd, throughput then increases with size of roundabout. Theorems obtained (Chapter 5) thus apply to Model A.

The case is different for Model B, where choice of lane is possible. Throughput depends on size of the roundabout, regardless of the number of cells (even or odd) and given similar topology and other parameters held constant. In general, throughput increases with size if all numbers of cells are either all even or all odd.

For both Models A and B, throughput increases with arrival rate linearly when no entrance road is in a saturated situation. Throughput reaches a maximum when the arrival rates reach their maximum for Model A. Throughput reaches a maximum when the arrival rate reaches a critical value on *one or more* roads for Model B.

When the arrival rate is larger than the critical value, saturation occurs on one or more roads. Critical arrival rates (CAR) also depend on other road arrival rates and on roundabout topology and turning rates for all roads.

The operational performance of a roundabout is improved when arrival rates (AR_1=AR_2=AR_3=AR_4) are balanced. Throughput decreases as RT rate increases when one or more roads are saturated, as vehicles, on average, need to travel longer distances on the roundabout.

When arrival rate is less than the critical value, queue-length of an individual road is low, but for when arrival rate is greater than the critical rate, the queue length rapidly achieves maximum.

For Model B only, Position Delay Time (PDT) has an effect on throughput when the arrival rate is close to or larger than CAR. Throughput also decreases as PDT increases. PDT has little effect on throughput, as left-lanes of Model A are theoretically in free-flow at all time.

Driver behaviour has an impact on the overall performance of the roundabout and individual roads. Rational, urgent and conservative behaviour leads to free-flow on the roundabout for all arrival/turning rates considered, whereas reckless behaviour can lead rapidly to congestion for both Models A and B. For Model A, there is no difference between rational and urgent behaviour in respect of throughput, but for Model B, throughput is different for urgent and rational behaviour. Conservative behaviour leads to decreased throughput for both Models.

Compared to Model A, Model B has better operational performance with higher throughput when all arrival rates ≥. 0.30. Particularly, when 0.45 ≥ all arrival rates ≥ 0.30, saturation occurs for Model A, but not for Model B. However, Model A is safer than Model B, as there is theoretically, no cross traffic on the Model A roundabout.

Chapter 7
Summary and Future Research

7.1 Overview of Research Focus

In this chapter, we present the summarised our research first, its contribution and further comments. Finally, we also propose an extension of the approach to simulate heterogeneous driver and vehicle units, and discuss further research possibilities.

7.1.1 Summary of main findings

In this thesis, a new model to study unsignalised traffic flow in urban networks is proposed, which is based on **M**inimum **A**cceptable s**P**ace (**MAP**) method. Using MAP, the model, implemented using cellular automata (CA), can simulate heterogeneous and inconsistent drive behavior and interaction between drivers for different traffic conditions, and for a variety of urban and inter-urban road features.

Two types of road features have been focused on: (i) two-way stop-controlled (TWSC) intersections and (ii) roundabouts. A TWSC intersection is controlled by priority and stop rules. Priority rules require that vehicles from a minor street give way to vehicles from a major street, and RT vehicles from a major street give way to LT vehicles from a major street. The stop rule demands that a vehicle stops at the stop-line before entering the intersection. Priority rules are also known as offside priority rules (by which a vehicle entering gives way to one already on the roundabout). In addition to all vehicles being governed by the rules mentioned above, the process of passing through an intersection and/or roundabout depends on the process of drivers' self-organisation (Wang and Ruskin 2001, 2002), and e.g. the phenomenon of priority-sharing (Troutbeck and Kako 1999).

Driver interaction and behaviour are the main focus of the work effort so far. Our model has, for the first time (to our knowledge), attempted to categorise different driver behaviour based on different space requirements (MAP) and to detail the space conditions to the requirement of each cell inside the space required, in order to ascertain the effect on performance. Driver behaviour at intersection or roundabout entrances is randomly categorised as *rational*, (when optimum conditions of entry are realised), *conservative, urgent* and *radical*, with specified probabilities. Drivers are also randomly assigned to one of the above categories at each time step. In this way, our CA model has successfully simulated elements both of heterogeneous and inconsistent driver behaviour (Ruskin and Wang 2002b), whereas drivers in the previous gap-acceptance models are assumed to be homogeneous and consistent (Troutbeck and Brilon 1997).

Furthermore, our CA models successfully apply to TWSC intersections and roundabouts in networks, where the headway distributions are insufficient to describe traffic flow, (Ruskin and Wang 2002a).

Three aspects of intersection and roundabout performance in particular have been studied. The first, looks at overall *throughput* (the total number of vehicles, which navigate the intersection or roundabout in a given time) and *capacity* (the number of vehicles can enter intersection or roundabout from an individual entry road), for different geometric conditions, arrival and turning rates. The second investigates changes in queue-length, delay-time and vehicle density for an individual road and roundabout. The third considers the impact of driver choice on throughput and operation of the roundabout.

Driver behaviour clearly has an impact on the overall performance of intersections and roundabouts, as well as on flow in individual roads (Wang and Ruskin 2002). Rational, conservative and urgent behaviour leads to free-flow on the intersections and roundabouts for all arrival/turning rates considered, whereas radical behaviour can rapidly lead to gridlock.

The model has successfully reproduced, for the first time, the typical congestion phenomena in the operation of roundabouts and intersections (gridlock). Failure to obey the road rules is as crucial a factor in congestion as traffic density, according to our findings. Our model clearly shows how driver behaviour can cause traffic system failure (Wang and Ruskin 2002).

Capacity of minor streams in a single-lane TWSC intersections are found to depend on flow rates of major-streams, and this also changes with flow rate ratio (FRR= flow rate of near lane: flow rate of far lane). Hence the flow rates corresponding to each stream must be clearly differentiated (Ruskin and Wang 2002 a).

We have also noted that a two-lane TWSC intersection *does* improve mobility of minor steams, measured in capacity comparing to single-lane TWSC intersection. However, entry capacity of the minor road and the RT capacity of the major road are nearly zero when major-road arrival rates ≥1440 vph for a two-lane TWSC intersection.

Compared to 2-TWSC intersections, for the same minor-road capacity, traffic lights are found to positively effect throughput. However, it should be stated that this is conditional on there being enough vehicles on all roads to fully utilise the green light periods. In addition, signalised intersections improve cross flow, but at expense of vehicles on the major streams.

An additional feature of our approach is that, while previous models looked at roundabouts as a combination of many T-intersections, our model treats a roundabout as a *unified system* so that we can study how the arrival rate of one entrance can affect other entrances or be affected by other entrances.

We also develop theorems of optimum density, capacity and size of the roundabout based on our findings. The theorems are proved theoretically (Section 5.3.5) and shown empirically. We find that throughput does not appear to depend on single-lane roundabout size in some situations, if the number of cells of the roundabouts is even. If the number of cells of the roundabouts is odd, throughput then increases when the size of roundabout increases. Size of the roundabout is not an important in term of throughput. Clearly, the entrances are bottlenecks in terms of smooth operation (Wang and Ruskin 2002).

For the single-lane roundabout, we noted that throughput of roundabout increases with arrival rate linearly when no entrance road is in a saturated state. Throughput reaches a maximum when the arrival rate reaches *a critical value* on one or more roads. When the arrival rate is larger than the critical value, the state of saturation occurs on one or more roads. Critical arrival rates also depend on other road arrival rates, roundabout topology and turning rates. Throughput decreases as right-turning rate increases, as vehicles on average need to travel longer distances on the roundabout.

Three two-lane roundabout models, Model A, B and C are developed with different lane-allocation patterns. Model C can be seen as an extreme situation of model B. Therefore, Model A and Model B are the main focus of the results reported here.

For Model A, our conclusions are similar with respect to the relationship between throughput and size of a two-lane roundabout to those obtained for single-lane roundabouts. The theorems obtained in Chapter 5 thus apply to Model A but not for Model B.

For both Models A and B, throughput increases linearly with arrival rate when no entrance road is in a saturated condition. Referring to Model A, throughput reaches a maximum when all arrival rates reach their maximum. But, with regard to Model B, throughput reaches a maximum when arrival rate reaches a critical value on *one or more* roads.

Critical arrival rates (CAR) also depend on other road arrival rates and on roundabout topology and turning rates for all roads for both Models A and B.

For Model B *only*, Position Delay Time (PDT) has an effect on throughput when the arrival rate is close to or larger than CAR. Throughput also decreases as PDT increases. Referring to Model A, PDT has little effect on throughput, as left-lanes of are theoretically in free-flow at all time.

We have seen that driver behaviour has an impact on the overall performance of the roundabout and on individual roads. Rational, urgent and conservative behaviour leads to free-flow on the roundabout for all arrival/turning rates considered, whereas reckless behaviour can lead rapidly to congestion for both Models A and B. For Model A, there is no difference between rational and urgent behaviour in respect of throughput, but for Model B, throughput is different for urgent and rational behaviour.

Compared to Model A, Model B has better operational performance with higher throughput when all arrival rates ≥. 0.30. Particularly, when 0.45 ≥ all arrival rates ≥ 0.30, saturation occurs for Model A, but not for Model B. However, Model A is safer than Model B, as there is theoretically, no cross traffic on the Model A roundabout.

7.1.2 The integrated picture and future direction

In our research, we creatively use CA models to simulate heterogeneous and inconsistent driver behaviour and interaction in unsignalised different cross traffic flow situations for urban/interurban networks (see Section 3.3). Part of this work has been published in three papers and a conference presentation (published abstract), (Wang and Ruskin (2001, 2002), Ruskin and Wang (2002a and b)). While CA models have been used to model different aspects of vehicle movement, such as randomised vehicle speeds, interaction (such as over-taking, following) on highways etc. To our knowledge, this is the first work to attempt to simulate heterogeneous and inconsistent driver behaviour and interactions of **cross-traffic**. Pervious cross-traffic analytical models, i.e. gap-acceptance model, unrealistically assume that drivers do not vary in their behaviour. Gap-acceptance models fail to model the phenomenon that indicates interactions between drivers. Neither give-way between the major streams nor platoons are considered in gap-acceptance model.

Our model not only overcomes some of the drawbacks of gap-acceptance models (details see Section 3.2.2), but also develops the traffic flow picture for the urban context. MAPs of radical and urgent driver behaviour are less than the optimal space. When a driver from the minor road use one of these two MAPs, he/she may block the oncoming vehicle on the major road. In this way, our models simulate the phenomena of "priority sharing" and gridlock. This represents an important contribution to understanding of why traffic systems fail and we believe that the methodology has wide application.

The second important feature of this research is the focus on roundabout operation, implementation through a unified CA ring (or rings) to model traffic flow at a roundabout (Wang and Ruskin, 2001 and 2002). CA models have typically been applied to model straight-through flows (such as highways), but **never been** applied to **circulating flow**. Stimulated by work of Chopard (1998), we have developed a CA ring model that can be used for roundabouts of various types. The stochastic CA ring models (theoretically, the update rules that govern CA-ring are not stochastic, but aggregated behaviour has a stochastic nature) presented here have been used to address issues of heterogeneity and inconsistency in driver behaviour as well as driver-interactions on single-lane and multilane roundabouts. These issues of driver behaviour have been demonstrated to be lacking in models, such as those for gap-acceptance and, from our results, can be crucial in terms of influencing performance measures. Other quantification of performance measures is also discussed in detail for a number of complex road features.

We also suggest important features in traffic flow, which to our knowledge have not been considered before, which are Stop Sign Delay Time (SSTD) and Position Delay Time (PDT). These phenomena are important, as they are part of the driver interaction process, and should not be overlooked in modelling unsignalised traffic flow.

Limitations and possible further considerations, which have been suggested by the work to date, are as follows: Firstly, there is clearly need to gather more real data to validate and test current models and give these a basis and reinforcement for further development. We have recorded some tapes of traffic flow (including several of flow at three-lane roundabouts in New Zealand-also a left-hand driving country). Some data are also available from publications (e.g. Robin and Tian 1997) and research conducted by Dr Tian (Texas Transportation Institute, USA), although primarily collected to test e.g. gap-acceptance criteria. Clearly, collaboration with the relevant local government bodies would be useful in this regard.

Secondly, using CA models clearly offers one viable approach to future effort in understanding traffic flow, control and management, but there are limitations in the work to date. For example, speed or length of vehicle can be more accurately described by further partitioning roads (resulting in smaller size of cells), although the approach may drastically increase computational time and algorithm complexity. In this respect, we may need to look at efficient parallel computing, especially where overhead of data exchange between nodes can be minimised.

7.2 Modelling Heterogeneous Driver and Vehicle Units

In this section, we suggest how the MAP method might be applied to simulation of heterogeneous driver and vehicle units, which we see as a next obvious step.

In contemporary multi-class traffic flow modelling, the focus is mainly on using *macroscopic* traffic flow models to model *highway* multi-class traffic flow (Zhang 1999, Hoogendoom et al. 2000, Wong and Wong 2002). In particular, a great deal of effort has been spent to replicate three major traffic flow patterns, such as discontinuity and platoon-dispersion phenomena (which are observed from highway traffic data) (Wong and Wong 2002).

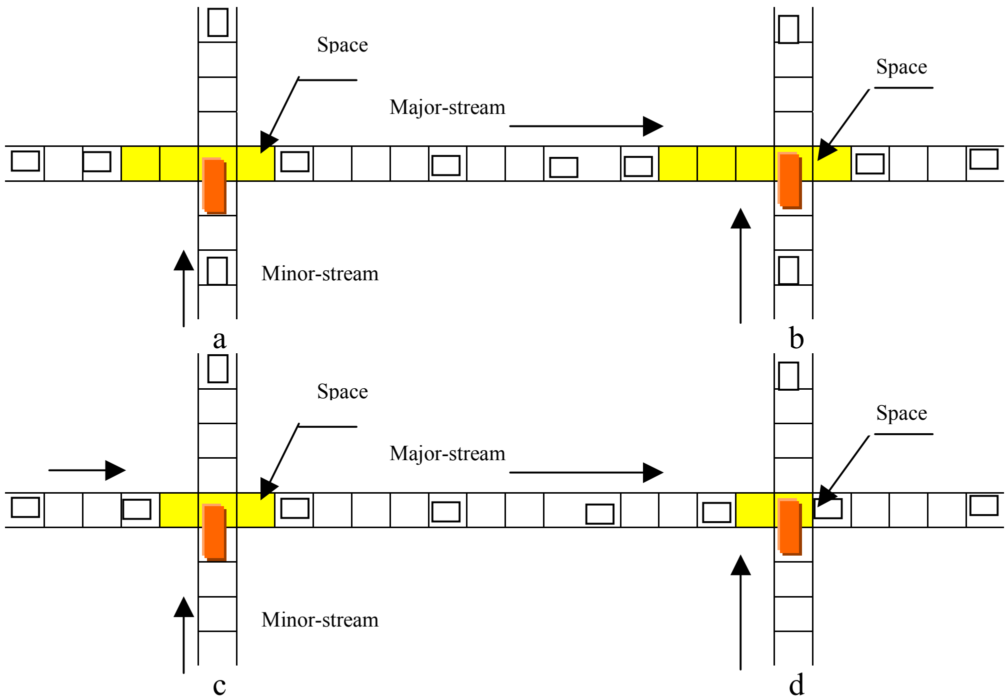


Figure 7. 1 Two-stream intersections for a long vehicle driver: (a) rational, (b) conservative (c) urgent and (d) radical

One main assumption in macroscopic models is that vehicles are treated as a particle, (where the lengths of the vehicles are neglected). Consequently, so called multiple user-classes (multi-classes) mean that dynamics (speeds, acceleration and deceleration capabilities) for these classes are different. In each class, vehicles have the same acceleration (up to a class-specific velocity) and deceleration capabilities. The differences are shown when vehicles have interactions, such as overtaking and lane-changing, (Hoogendoom et al. 2000).

However, in an urban network, when the speed limit is 50 km /h or under, all vehicles can reach and adopt this speed. The difference in acceleration abilities is not obvious when the speed limit is this low. Furthermore, the dynamics of vehicles have no obvious effect on queue formation and delay time (Chopard 1998). Therefore, in our heterogeneous driver and vehicle unit models, we assume that the differences between units are in vehicle length and driver behaviour, not in speed and acceleration and deceleration ability. The difference in lengths of vehicles is an important factor that effects the operation of urban networks (measured as usually by capacity, throughput etc.) Ruskin and Wang (2002a) indicate that a long vehicle can be considered based on occupation of more than one cell.

From aerial photographs, (which are generally flown at 5,000 feet at photo scale of 1:10,000 http://www.mapflow.com), we can see the differences of vehicle lengths in the city of Dublin. If we assume a normal car has a length of one unit, the length of long vehicles can be roughly categorised into either *two* units or *three* units. Therefore, we propose that the different vehicle lengths can be considered based on occupation of *two* or *three* cells.

As an analogy to our MAP method, we can consider that a rational driver driving a 2-unit long vehicle requires the same space as a conservative car driver. Figure 7.1 shows the space requirements for different drivers.

Figure 7.2 shows that an additional cell is required in each lane for the same category of driver behaviour. For a 3-unit long vehicle, two additional cells are needed.

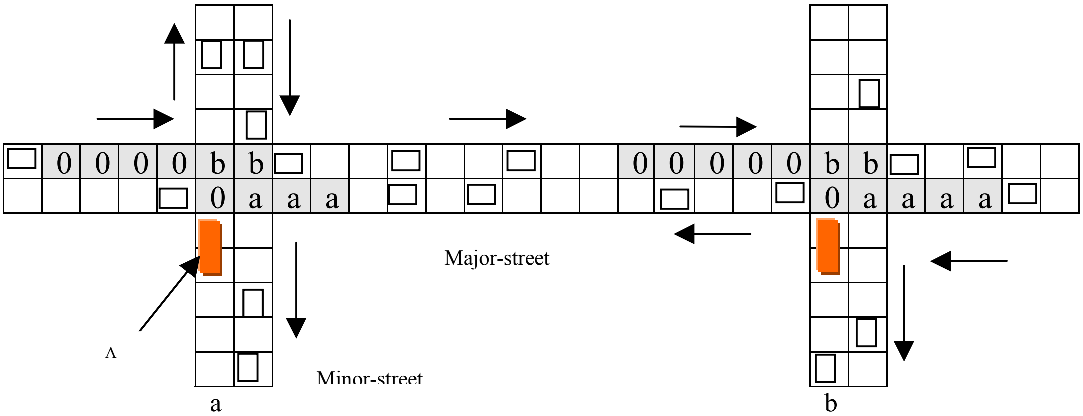


Figure 7. 2 A ST 2-unit-long vehicle from a minor-street: (a) rational behaviour (b) conservative behaviour.

A distribution of three different lengths of vehicles can be estimated from collection of real data e.g. from the aerial photographs or at streets. The distribution may of course vary with different time of a day, e.g. in rush hours, at delivery times/days etc.

The approach described in this thesis should therefore extended viably to further consideration of heterogeneity of vehicle lengths.

8 References

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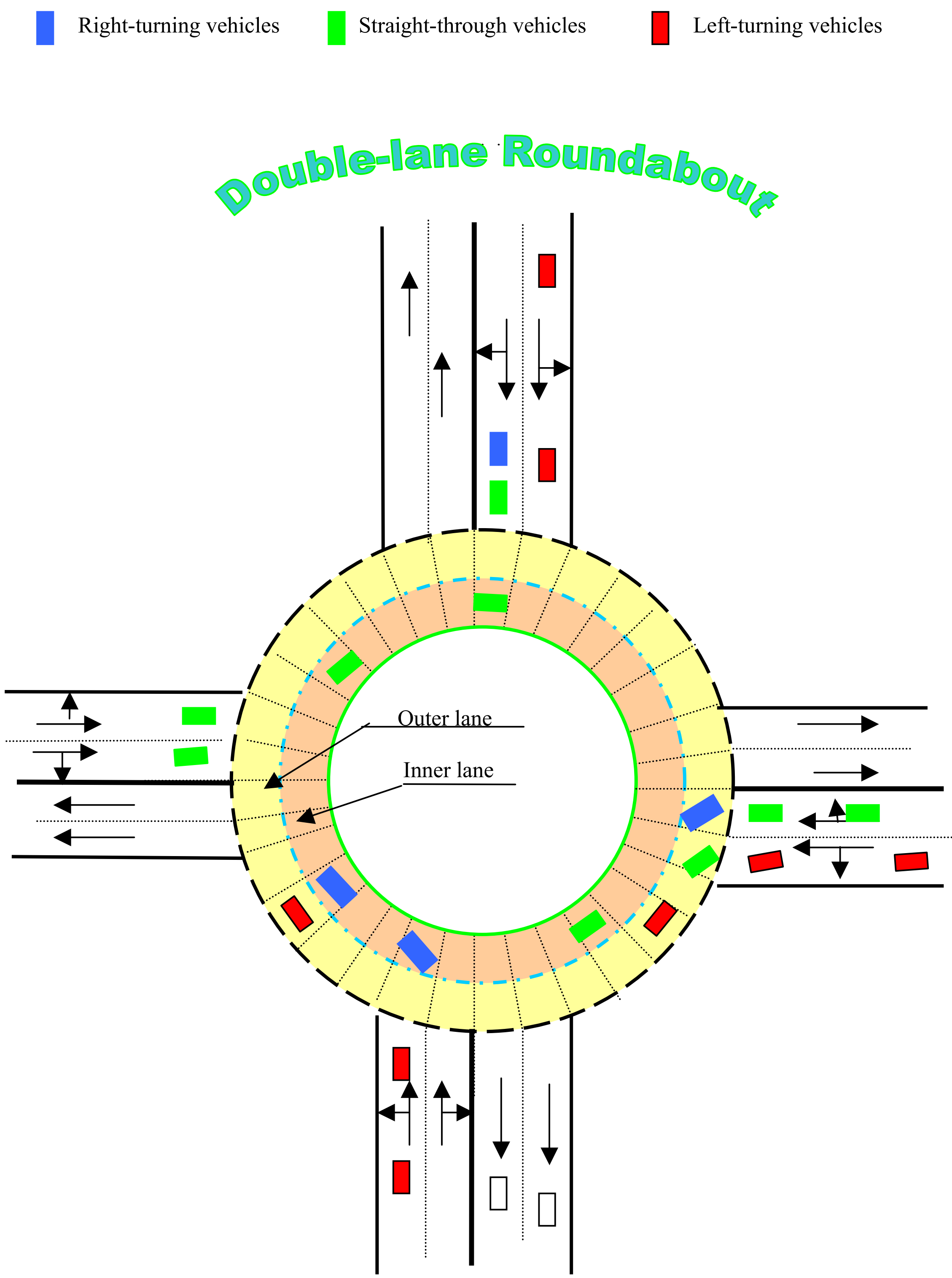
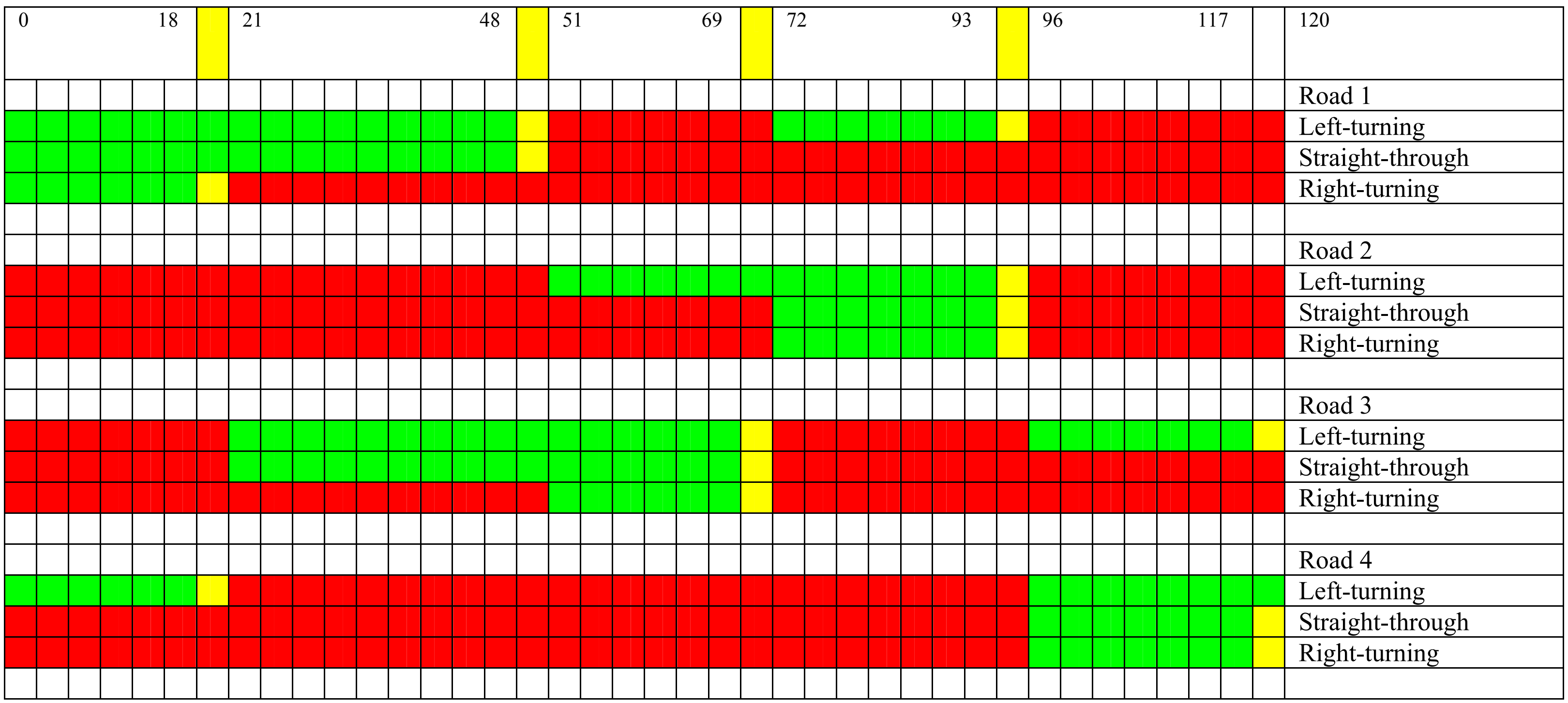
Appendix A Two-lane roundabout



Appendix B Traffic-light time setting

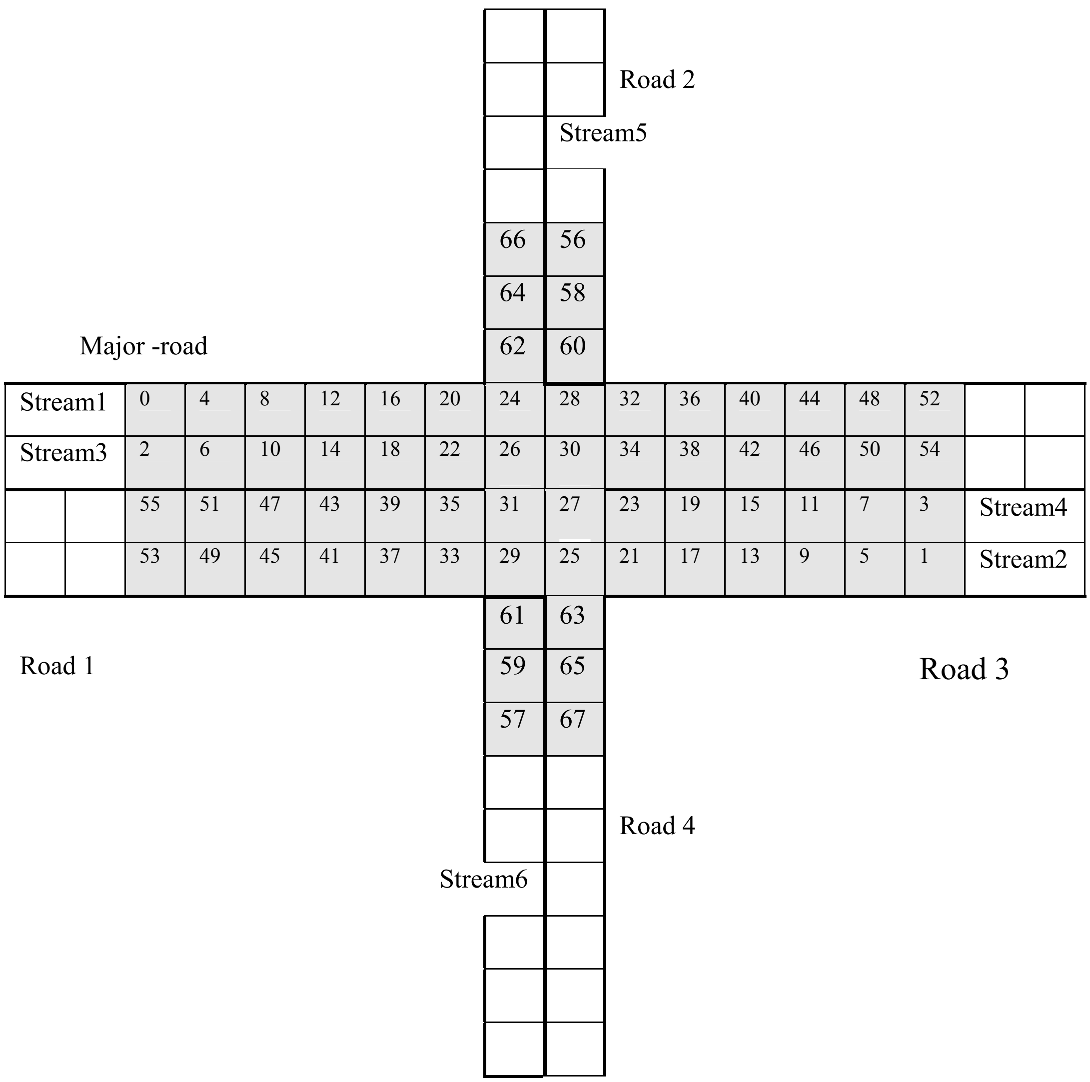
Traffic lights (each column = 3 seconds)



Appendix B Traffic-light time setting

Appendix C Multilane TWSC Intersection

This figure shows a 2-lane intersection. The cell numbers are corresponding to the program of intersection2 header file.



Appendix D   Published Papers

Appendix E Map

Appendix F Programs Disk