TRAFFIC CIRCLES IN SOUTH AFRICA: TRAFFIC PERFORMANCE AND DRIVER BEHAVIOUR

by

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SOLI DEO GLORIA

ABSTRACT

This thesis presents the results of an investigation into traffic operations and driver behaviour at traffic circles under South African conditions. The scarcity of local traffic circles necessitated the development of a simulation program (TRACSIM) to assist in the research process. This microscopic program for single lane circles is based on event updates and was calibrated and validated based on local data. Because the acceptance of gaps is such a vital part of the operation of a traffic circle, it was examined in detail. Specific attention was given to the possible use of a gap acceptance model based on variables other than time. Since the gap acceptance process also depends on the gap distribution in the circulating stream, the effect of the origin-destination pattern was also investigated. Two existing analysis techniques are evaluated and verified for local conditions, improving them where possible. Generally these techniques under-estimate traffic delay at local circles.

Observations indicate a difference between the acceptance of gaps/lags in the entering and circulating stream of conflicting traffic as well as a difference between critical gaps and critical lags. The mean observed critical gaps/lags are larger than in other countries, which indicates that delays at local circles will be greater. Gap/lag acceptance based on critical distances rather than critical times was applied successfully in the simulation program TRACSIM. A method is proposed to estimate critical distances from the geometric layout of the circle. Critical gaps are not fixed, but should vary with at least the conflicting flows. The investigation of the effect of unbalanced flows on delay, showed that the variability in drivers' critical gaps is more a function of delay than of conflicting flow. Entry delays increase because of an increase in conflicting flows or because of an unfavourable imbalance of conflicting flows. In both instances the drivers' critical gaps will decrease. A variable critical gap model only based on conflicting flows will show no change in the drivers' critical gaps if the conflicting volumes remain constant, even though the actual average delay might increase because of an unfavourable imbalance in conflicting flows.

PREFACE

I, Johann Christoff Krogscheepers, declare that this dissertation is my own work, carried out under the supervision of Professor C.S. Roebuck of the University of Natal, and is in accordance with the requirements of the University for the reward of the Phd degree in the Department of Civil Engineering at the University of Natal.

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Date Signature

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α	Weaving angle	degrees
α_1	Factor taking into account the disturbance of entry flow due to exiting flow	
α_2	Proportion of free vehicles	
β	Coefficient for Swiss capacity formula based on number of circulating lanes	
β_1	Minimum headway between vehicles	seconds
β_2	Move up time of minor road vehicles	seconds
β_{2D}	Move up time of minor road vehicles (seconds) - Dominant stream.	seconds
$\beta`_{2D}$	Dominant lane move-up time adjusted for entry flow to circulating flow ratio	seconds
$\beta`_o$	Minimum value of adjusted move-up time set for zero circulating flow	seconds
β_{2S}	Move up time of minor road vehicles (seconds) - Sub-dominant stream	seconds
δ	Minimum delay for full stop	seconds
κ	Coefficient for Swiss capacity formula based on number of entry lanes	
λ	Decay rate - Bunched exponential headway distribution	
ρ	Ratio: used for various ratios as defined in text	
τ	Critical gap	seconds
ϕ	Entry angle	
A_w	Total area of widening outside the area of basic cross-roads	$(\text{metre})^2$
c	Width of approach island parallel to circulating carriageway	metres
c_c	Critical values for Kolmogorov-Smirnov test	
d	Central island diameter (CID)	metres
D	Inscribed circle diameter (ICD)	metres
d_g	Geometric delay	seconds per vehicle
d_{min}	Delay to isolated minor road vehicles - Adam's delay	seconds per vehicle
d_s	Geometric delay to vehicles having to stop	seconds per vehicle
d_t	Traffic delay per vehicle	seconds per vehicle
D_2	Kolmogorov-Smirnov test statistic	
D_t	Total delay	seconds
D_T	Total vehicular delay	seconds
d_u	Geometric delay to vehicles not having to stop	seconds per vehicle
e	entry width	metres
$\bar{e}$	Average entry width	metres
e_1	Width of entry approach	metres

e_2	Width of circulating carriageway just upstream of entry	metres
ED	Extra distance involved in negotiating the circle	metres
F	Intercept for capacity equation	vehicles per hour
f_{c-}	Slope for capacity equation	
f_{HV}	Heavy vehicle adjustment factor	
f_{od}	factor to adjust for O-D pattern	
F_w	Weaving factor	
h_w	Time headway between successive vehicles	seconds
K_b	Efficiency coefficient in Blackmore's formula	
L	Length of the weaving section	metres
L	Also used for queue length	vehicles
L_0	Initial queue length	vehicles
l'	Effective length over which flare to approach develops	metres
L_t	Lost time	seconds
n	Number of stops	seconds
n_c	Number of circulating lanes	
n_e	Number of entering lanes	
n_m	Minimum number of entering vehicles under heavy circulating flows	vehicles per minute
N_e	Number of entering vehicles in a specific time period T	
N_L	Number of Lanes in the section under consideration for weaving capacity	
Q	Total capacity or full capacity of intersection	vehicles per hour
q_a	Arrival flow	vehicles per second
q_c	Circulating flow	vehicles per second
Q_c	Circulating flow	vehicles per hour
Q_{ca}	Circulating flow after adjustment for heavies	vehicles per hour
q_d	Departure flow	vehicles per second
Q_D	Entering flow in dominant lane	vehicles per hour
q_e	Entering Flow	vehicles per second
Q_e	Entering flow	vehicles per hour
Q_{ea}	Adjusted entry lane capacity	vehicles per hour
Q_i	Impeding flow	vehicles per hour
Q_l	Circulating flow exiting circle at the approach	vehicles per hour
Q_{max}	Maximum entry capacity	vehicles per hour
q_{max}	Maximum entry capacity	vehicles per second
Q_{min}	Minimum entry capacity	vehicles per hour

q_s	Saturation flow of the approach	vehicles per second
Q_S	Entering flow in sub-dominant lane	vehicles per hour
q_w	Capacity of weaving section	vehicles per second
Q_w	Capacity of weaving section	vehicles per hour
r	radius	metres
R	Ratio of smaller weaving stream to larger weaving stream S1/S2	
s	Average of squared residuals.	
s_x	Standard deviation of the independent variable	
S_d	Saturation Density of a lane	vehicles per hour
S	Sharpness of the flare	
S_1	Smaller weaving stream volume	vehicles per hour
S_2	Larger weaving stream volume	vehicles per hour
S_t	Stop-delay factor	
S_{nt}	Number of stops per 15 minutes	
T	Time period	
$t_{\alpha/2}$	value from t-distribution at α level of confidence.	
u	Width of the circulating roadway	metres
v	Approach road half width	metres
V_a	Approach speed	km/h
V_d	Departure speed	km/h
V_{min}	Minimum speed	km/h
w	Width of weaving section	metres
W	Width of approach road	metres
x	Degree of saturation	
x_2	Effective entry width	metres
z	Uniform random variable	

CHAPTER 1 : INTRODUCTION

1.1 Background and motivation

Unsignalised or priority control is one of the most common types of intersection control in urban and rural areas (Green, 1996). Traffic circles as an important type of priority control device are also finding increased usage, especially in urban areas abroad (Chung, 1993), but also to a large degree in South Africa. South Africa is presently experiencing an incomparable growth in the development of new townships in and around major urban centres. It has been found by Krogscheepers & Roebuck (1993) however, that traffic control in these developing urban areas is often impeded by various factors which include inter alia:

i) The lack of satisfactory street furniture, arising because of theft, damage and vandalism.
ii) Driver behaviour - drivers not adhering to regulations at for instance stop streets and traffic lights.
iii) Inadequate maintenance of street furniture.
iv) Lack of technical support to maintain facilities.
v) Limited budgets.

Inadequate traffic control gives rise to uncontrolled movement of traffic through dangerous conflict situations which could not only result in serious accidents with the associated loss of life and injuries, but also in a loss of confidence in the road transport system and the responsible authorities. Krogscheepers and Roebuck (1993) argue that because of the following advantages of traffic circle control their use should be encouraged:

i) **Improvement of intersection safety**. There are fewer points of conflict (Seim, 1991) than at other priority controlled intersections. Due to a reduction in vehicles speeds, forced by the geometric layout of circles (Seim, 1991) they are the safest type of intersection for vehicles and pedestrians, and personal injuries are much less frequent than at other at-grade intersections (Alphand et al, 1991). They are also self-enforcing and thus simplify the driver decision at the point of entry (Schermers, 1987). Circles are reliable as they function at all times (Seim, 1991). Circles can be effective in reducing speeds on for instance an arterial, but the road must be treated globally and not with one single traffic circle as a spot measure (Alphand et al, 1991).

ii) **Maintenance and operating cost** (Short & Van As, 1992). The maintenance cost (Seim, 1991) is low with hardly any follow-up required. The operating costs are less than at signalised intersections. Other than at traffic circles, theft and damage to street furniture at priority controlled intersections could leave these intersections uncontrolled and dangerous. For instance, in time the road markings at a stop controlled intersection can fade and if the stop sign is removed then there are no physical signs to warn motorists to stop.

iii) **Improvement of traffic flow** (Seim, 1991). The capacity of traffic circles is usually greater than for similar at-grade intersections except for possibly signalised control. Traffic flows are simplified and improved with less traffic delays, especially outside peak hours. It allows for better throughput of right turning traffic, since there is no hierarchy of flow (Alphand et al, 1991). Circles have the ability to handle fluctuating demand in daily as well as seasonal traffic (Alphand et al). In developing countries where expansion can be rapid and unpredictable, the traffic circle, with its self-policing nature, is flexible enough to control varying demand without unnecessary delay to the motorist (Sutcliffe, 1990). Circles are also adaptable to a range of sites (Schermers, 1987). Short distances between intersections can be allowed because little space is required for separate left and right turning lanes (Seim, 1991). The possibility of U-turns at traffic circles (Seim, 1991) is not only advantageous to unfamiliar drivers but it could help in simplifying traffic flow at intersections between traffic circles by preventing right turning movements.

iv) **Environment**. Fewer delays and stops reduce energy consumption and pollution (Seim, 1991). Central islands can be landscaped as a positive, aesthetic element thus improving the urban quality of the public space (Simon, 1991). It could mark the beginning of a street with priority to the environment or create a transitional zone between two districts of different nature (Alphand et al, 1991). Traffic circles could also restrict through traffic and hence restrain large traffic volumes and excessive speeds along local streets, i.e. traffic calming measure (Short & Van As, 1987). "...use of roundabouts in the road network could lead to the fluent traffic regulation at low speed thereby achieving safety and environmental standards at high capacity,..." (Simon, 1991). A 16% reduction in fuel consumption for the total traffic at circles has been estimated and hence a reduction in poisonous gases. The reduction in stops and starts would result in lower noise levels (Simon, 1991; Schermers, 1987).

Most of the above advantages tend to be tangible. However, Sutcliffe (1990) states that from site observations there seem to be further, less tangible advantages. "It appears that give-way facilities operate effectively in less developed communities where policing is often absent and strict rules of the road are rarely observed. This informal attitude to driving on the road seems to lead to an unwritten code of behaviour that is however, appropriate to the inherent style of driving. A typical example of this would be the reversal of the 'priority to circulating traffic' rule. It is quite acceptable in developing communities for the entering vehicles to force those circulating to wait until the entering queue has reduced sufficiently before reverting to the standard operation."

However, there are also some disadvantages (Krogscheepers & Roebuck, 1993):

i) The larger traffic circles require large land reserves
ii) Traffic circles cannot give priority to traffic on any primary road, neither to public transport vehicles if required.
iii) They may be inappropriate if there is an unequal distribution of traffic with high turning volumes.
iv) The successful geometric design to reduce speeds effectively is difficult to achieve (Seim, 1991)
v) The location of pedestrian crossings is not simple and straightforward (Seim, 1991).
vi) Steep gradients are to be avoided (Alphand et al, 1991).
vii) According to Alphand et al (1991) circles are not suitable for traffic streams with a high proportion of heavy- and two-wheeled vehicles.

Therefore, the use of a traffic circle does not necessarily guarantee the above advantages, but it is imperative that the circle is correctly designed and placed in the correct position. Designers need to be able to predict the performance of traffic circles and they should have the ability to estimate the useful design life of such an intersection. This research is an effort to improve the apparent lack of detailed design criteria for South African conditions (Schermers, 1987).

1.2 Goal and study approach

From the motivation in Section 1.1 the following goal was identified for this research:

To study traffic operations at traffic circles under South African conditions and to verify and improve where possible the existing models used for traffic circle analysis. Two important aspects to be investigated are the gap acceptance process and the effect of unbalanced flows at circles.

To reach the above goal the following study approach was defined:

i) From a detailed literature survey, identify the important aspects regarding traffic circles and the various analysis models that are being used internationally

ii) Verify the accuracy of the analysis models which have found the greatest local application

iii) With the scarcity of traffic circles in South Africa and hence the unavailability of a large data source in mind, develop a simulation program that could assist in the research process

iv) Calibrate and validate the simulation program with data obtained from detailed field studies at suitable local traffic circles

v) Because the gap acceptance process is such a vital part of the operation of a traffic circle, investigate the gap acceptance process, by examining the possible use of gap acceptance models based on variables other than time

vi) Compare any new methods or models developed during this research with internationally accepted models

vii) Use the simulation model to investigate the effect of unbalanced flows on entry delays

Due to the the few circles in South Africa which are operating under saturated conditions with vehicles queuing on any one approach for more than twenty minutes it is very difficult to comprehensively study the capacity of these facilities. Therefore, a comparison of the methods of capacity estimates developed in other countries with South African conditions is very intricate. Hence, the strategy for this research was to concentrate on delays rather on capacities.

1.3 Structure of the thesis

This thesis can be divided into five distinct steps or phases:

i) Literature survey to identify available models and methods for traffic circle analysis
ii) Evaluation of the available analysis models for South African conditions

iii) Development of a simulation model for traffic circles

iv) Calibration and validation of the model for South African conditions

v) Application of the model by first investigating the gap acceptance process in more detail, then comparing the model with other analysis models and finally looking at the effect of unbalanced flow on entry delays

These five different phases are discussed in the ten chapters included in this thesis. This chapter covers the background and motivation for the research, the goal and objectives, the structure of the thesis and some definitions.

Chapter 2 covers the literature investigation and looks at the historical background of circles abroad and locally as well as the development of the different analysis techniques to estimate capacities, delays and stops at circles.

The comparison of the available analysis models with observations at local traffic circles is discussed in *Chapter 3*, including various statistical tests to verify whether the differences in the observed and estimated results are significant or not.

Chapter 4 presents the development of the simulation program. This chapter covers the problem definition, objectives and criteria for the program, the system analysis and synthesis, the different modules and their interaction, the verification of the program and an analysis of the sensitivity of the program to some of the input variables.

The data collection process to calibrate the input parameters is discussed in *Chapter 5*. Included are different surveys to estimate traffic delays, critical gap and lag distributions, move-up times, speeds and turning volumes.

Chapter 6 covers the validation of the simulation program and the methodology followed to compare the simulated traffic delays with observed traffic delays at three local traffic circles. The comparisons are made individually for the different approaches and in combination. First, all the data for one circle were combined and compared, and then the data for all the circles were combined and compared.

Chapter 7 is a detailed examination of gap acceptance at traffic circles and it investigates gap acceptance based on distance rather than the traditional approach where it is based on times. This approach is tested with the simulation program in comparison with the observed delays. A simple method to establish critical distances from the geometric layout of a circle is proposed and tested with the simulation program.

Chapter 8 compares the delay estimates from the simulation program using the critical distance concept with delay estimates from the program SIDRA to place the work completed in this thesis in context with other work.
The effect of unbalanced flows on entry delays and indirectly also on approach capacities are compared and discussed in Chapter 9.

Chapter 10 summarizes and concludes the research contained in this thesis, also offering some recommendations for consideration in future research.

1.4 Definitions

Across the world different terms are used to describe a traffic circle. The term "**rotary**" is used mostly in the United States of America while in Europe and Australia the term "**roundabout**" is more commonly used. For the purposes of this study the term "**traffic circle**" will be used as this is the more common usage in South Africa.

Armitage and McDonald (1974) defined a traffic circle as a number of T-intersections following one another with the traffic on the major road (in the circle) having priority and the minor road traffic (on the approaches) having to yield. However, these intersections are not isolated because the entry flow from one approach affects the entry flows from the next. Therefore the Australian definition (Austroads, 1993) of a circle as a channelised intersection at which all traffic moves clockwise (right-hand driving) around a central traffic island, might be more appropriate.

Traffic circles exist in various sizes and shapes (Gorton, 1977) and the various **types of traffic circles** can be defined as follows:

i) **Conventional Traffic Circle**. A traffic circle having a one-way carriageway comprising a series of weaving sections around a circular or asymmetrical raised central island and normally without flared approaches.

ii) **Small Traffic Circle**. A traffic circle having a one-way, circulatory carriageway around a raised central island four metres or more in diameter with flared approaches to allow multiple vehicle entry. The inscribed diameter of the circle should be between 26 and 30 metres.

iii) **Mini Traffic Circle**. A traffic circle having a one-way circulatory carriageway around a flush or slightly raised circular marking less than four metres in diameter and with or without flared approaches and with the inscribed diameter less than 25 metres.

iv) **Double Traffic Circle**. An individual intersection with two small or mini traffic circles either contiguous or connected by a short link road.

v) **Multiple Traffic Circle**. An individual intersection with three or more small or mini traffic circles either contiguous or connected by short link roads.

vi) **Ring Junction**. An intersection having a two-way circulatory carriageway around a central island linking mini traffic circles at the mouth of each entry to the intersection.

vii) **Gyratory System**. A system where four or more roads are joined by a large traffic circle. The central island diameter is usually in the region of 90 metres or more, but it is not necessarily circular.

The following concepts/words which require defining were used in this text. Most of the geometric elements of a traffic circle are defined in Figure 1.1.

Circulating diameter: The diameter of the circulating pathway along which circulating vehicles travel

Degree of Saturation: The ratio of the number of vehicles entering an intersection in a specific period to the maximum number of vehicles which could enter during that period

Distance headway: The distance interval between the arrival at a point of one vehicle and the position of the next relevant vehicle

Move-up times: The time between successive queuing vehicles entering the circle with no conflicting traffic

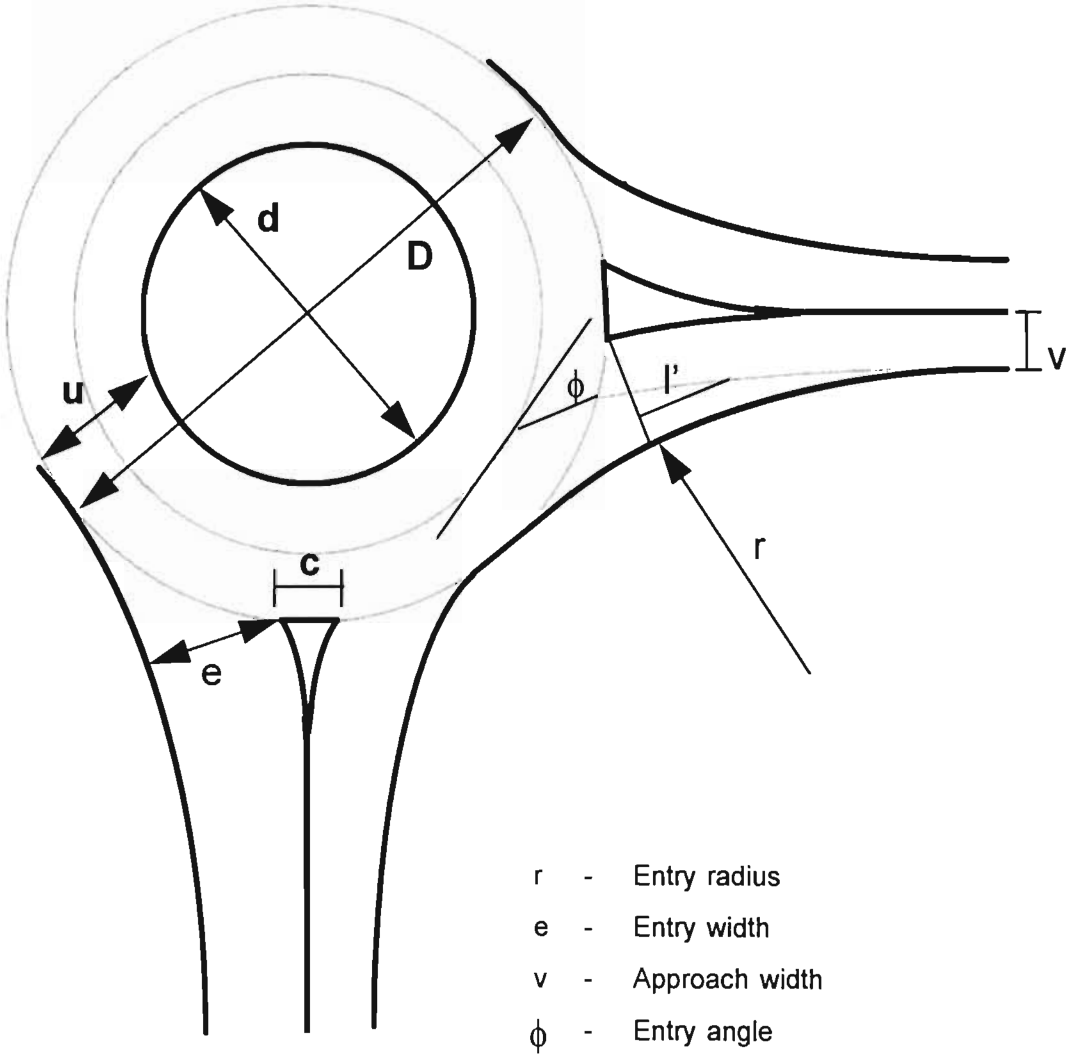
Nearside priority rule: Traffic on the circulating roadway of a traffic circle has to yield to entering traffic

Offside priority rule: Traffic on an approach entering the traffic circle has to yield to circulating traffic

Passenger car units : A measure to convert vehicles other than passenger cars, and turning movements other than through vehicles to equivalent passenger cars units (pcu), which is also expressed as passenger car units per hour (pcu/h).

Time headway: The time interval between the arrival at a point of one vehicle and the arrival at the same point of the next vehicle.

Traffic: Unless otherwise indicated traffic in this context refers to wheeled vehicles which are commonly found on roads.



r - Entry radius

e - Entry width

v - Approach width

ϕ - Entry angle

d - Central island diameter

D - Inscribed circle diameter

l' - Effective flare length

c - Island width

u - Circulation width

s - Sharpness of flare 1,6(e-v)/l'

See (Kimber, 1980) for determination of l' & ϕ

Figure 1.1: Important geometric elements of a traffic circle

CHAPTER 2: REVIEW OF TRAFFIC CIRCLE DEVELOPMENT

One of the fundamental characteristics of a traffic control facility is its ability to accommodate traffic. The ultimate ability of a control facility to accommodate traffic is also referred to as its capacity, which evidently is a key design parameter. Other design parameters which are frequently used to describe the operation of a control facility are the delay per vehicle and the number of stops per vehicle. The number of accidents occurring in the intersection give an indication of the operation of the circle in terms of safety requirements. Since the introduction of traffic circles at the turn of the century, traffic engineers have been researching different techniques and methods to assist in estimating these parameters. After discussing the history and development of traffic circles abroad and locally, this chapter will highlight the most important events and studies which played a major role in the development of techniques to estimate: capacity, delay, queues, and stops at traffic circles.

2.1 History of traffic circles abroad.

Places and sites of circular shape have been part of city and town planning layouts since the Middle ages, especially during the Renaissance and were used where it was worth emphasizing the significance of the place (Stuwe, 1991). Traditionally, traffic circles were not only associated with centres of architectural importance and where adequate space and numerous approaches lent themselves to a traffic circle, but also where they satisfied a military need by serving as focal points for radial routes on which the army could be swiftly deployed. These focal points were not always circular, but sometimes also square shaped. The traffic travelling around them travelled in all directions and not in a one-way gyratory manner. The idea of a one-way system with traffic rotating (rotary) around the central island seemed to be conceived independently in Britain, France and the United States at the turn of the century (Todd, 1988).

The conception of the rotary system is attributed to William Philips Eno (Eno, 1939), an American architect, Eugene Henard, a French architect for the city of Paris and Holroyd Smith who first presented this concept to the London City Council in 1897 (Todd, 1991). Eno (1858 - 1945) the "father of traffic control and of one-way traffic" was not only responsible for introducing rotary systems in the United States but was also instrumental in advocating their use in Paris and London. Eno even advocated the use of small circles with only painted central islands, but Henard rejected this idea and proposed a minimum central island diameter of 8 metres.

The first formal installation of the rotary system in the United states was in New York in 1905 at the Columbus Circle (Chin, 1983), but it was poorly installed and did not give satisfactory results. In France the first introduction of a rotary system was in 1907 at the Place de l'Etoile, in Paris (see Plates 2.1 and 2.1). This circle, where 12 streets converged into a 38m wide roadway, was reported to handle close to 20 000 vehicles per hour in 1956 (Stuwe, 1991), but operating under the nearside priority rule (circulating vehicles to give way to entering vehicles) it often locked solidly, so that no vehicles could enter or leave for several hours. After initial attempts by Holroyd Smith in 1897 (Todd, 1991) and Hollier in 1914 to introduce the use of rotaries to the United Kingdom (UK) it was not until 1925 and with the help of William Eno, that the first partial gyratory system - a half circle was introduced at Aldwych (Manton, 1958). The first complete gyratory system belongs to Parliament Square, where it was put into operation on 4 January 1926.

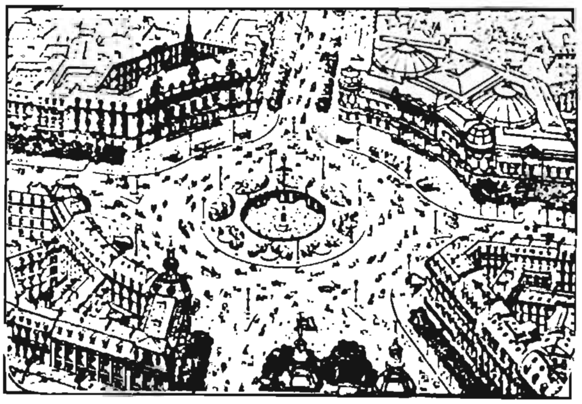


Plate 2-1: Place de l'Etoile, in Paris (1907)
Source: Bovy (1991)

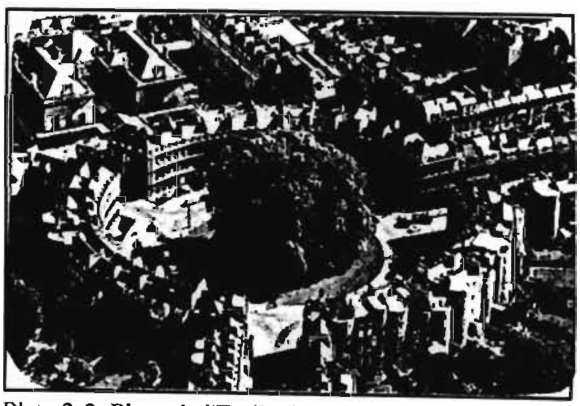


Plate 2-2: Place de l'Etoile, in Paris (1991)
Source: Bovy (1991)

In 1929 the UK Ministry of Transport recommended that traffic circles should be constructed with straight sides and rounded corners, i.e. a square central island for intersections with four approaches (see Figure 2.1) and a polygon island of equal sides and angles for intersections with more than four approaches. The length of any side was not to be less than 110 feet (33.5 metres) to facilitate weaving. However, it was realised that circular islands would improve the performance by not only reducing "dead areas", but also improving safety and efficiency (Bapat, 1969; Knight and Beddington, 1936; Royal Dawson, 1936). The recommendation on geometric design of roundabouts by the UK Ministry of Transport was amended in 1939 in favour of circular islands ranging from 100 feet (30.3 metres) to 180 feet (54.5 metres) diameter with the radius of the entry kerb a minimum of 60 feet (18.3 metres) to facilitate smooth transition on entry. (See Figure 2.2).

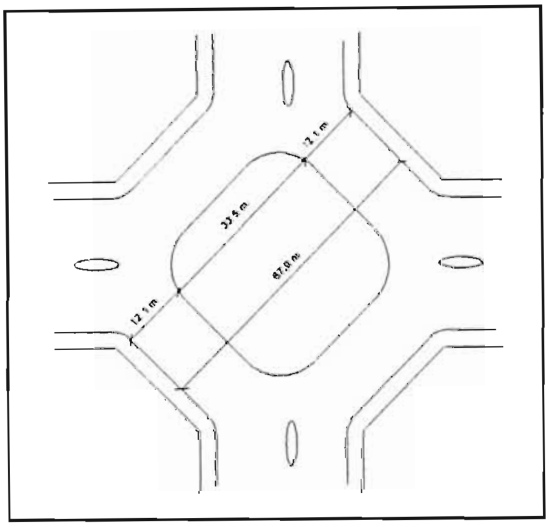


Figure 2.1: Recommended roundabout design in 1929 (Ministry of Transport, 1929)

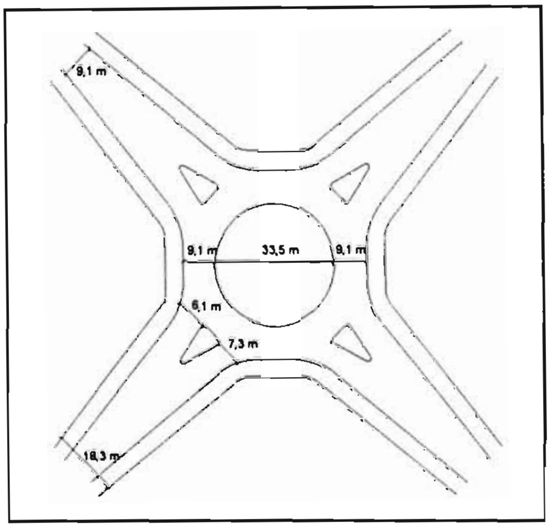


Figure 2.2: Recommended roundabout design in 1937 (Ministry of Transport, 1937)

Correspondence to 'The Times' criticized the use of the " uncouth, Latinise word 'gyratory' ... Why not use the simple, English word 'roundabout', which is understood by the people." Almost with immediate effect the new term 'roundabout' was accepted to describe this new form of traffic control (Todd, 1991). Since it seems that the term "traffic circle" is more commonly used in South Africa it will be used in this thesis. In the United States of America (USA), Australia and in most countries in Europe, after the First World War, this form of control was used more extensively at points of major traffic conflicts. In most countries this new form of control was met with public resistance mostly because of the motorists' unfamiliarity with the new device. However, after the initial resistance it was accepted in most countries as a positive step towards effective traffic control (Schermers 1987).

Initially all traffic circles operated under the nearside priority rule, i.e. circulating traffic yields to entering traffic. This meant that for traffic travelling on the lefthand-side of the road, drivers had to yield to the left and for traffic travelling on the right-hand-side of the road drivers had to yield to the right. This rule originated in France, where Charles Gariel proposed it in 1896 as a rule for cyclists and again in 1904 for inclusion in a highway code and it was later accepted internationally. The nearside priority rule also became the rule in France at all intersections where two roads of equal width met (Todd, 1988). Since Massard proposed the nearside priority rule for the city of Paris in 1910 there have been unsuccessful efforts to change this ruling as it was likened to a bus at a bus stop where passengers are permitted to board the bus before the others are allowed off. With the rapid increase in vehicle ownership in Europe and especially in the UK, problems at traffic circles operating under nearside priority became evident as they had a tendency to lock-up (Schermers, 1987), i.e. circle is jammed and no vehicles can move in.

Where traffic volumes were low, the weaving manoeuvres between entering and circulating vehicles took place without any problems, but as the volumes increased (especially right turning), entering vehicles with the right of way started forcing their way into the circle. Under these conditions the vehicles inside the circle could not exit, creating a "lock up" situation (Chin, 1983). In countries such as Germany, the initial interest in traffic circles was lost because of the lack of suitable capacity estimations, bad accident experience and because of the then priority ruling resulting in circles locking up. Consequently, many traffic circles built between 1930 and 1950 were converted to signalised intersections (Stuwe, 1991). Many different measures such as pointsmen and traffic signals (Webster, 1960) were tried to prevent locking, but the most successful was the introduction of the offside priority rule (Chin, 1983).

Unlike many other countries, uncontrolled intersections on British roads had no directional priority rule, the only rule being to exercise due care. Typical to the nearside priority rule in other countries the absence of a rule in Britain allowed drivers on the approaches to a traffic circle to enter and impede the free flow of traffic on the circulating roadway. Under heavy traffic this behaviour caused the circle to lock-up and hence some authorities installed signs on the approaches instructing drivers to give way to circulating traffic. The Ministry of Transport however, called for an investigation into this "illegal" practice, which lead to the introduction of the offside priority rule (Todd, 1991). The offside priority rule was officially introduced in November 1966 (Troutbeck, 1984). As a result of the success of this change which negated the use of large circles to accommodate the long weaving areas required for the nearside priority rule, progress was rapid and many signalised intersections were replaced by smaller traffic circles (Schermers, 1987). Many developments in this field have originated from research by the Transport and Road Research Laboratory in the UK.

Following this change in the UK, many other countries followed suit with the events in **Australia** closely following those of the UK. As early as 1974 Australian traffic engineers were investigating gap acceptance behaviour at roundabouts (Horman and Turnbull, 1974). In **New Zealand** in 1980 Edgar researched the conversion of a conventional to a small traffic circle at Riccarton. He concludes that although approximately two dozen small traffic circles exist in New Zealand, they have been installed for reasons other than to meet traffic volume and traffic engineering demands. **France**, in 1972, was one of the first countries in mainland Europe to introduce traffic circles in their modern form. Again this was met with mistrust by the motorists. However, observations show that traffic circles are currently meeting increasing success and can be found almost anywhere in town centres, suburbs, peri-urban areas, industrial and commercial estates (Alphand, Noelle and Guichet, 1991). The number of traffic circles built in France in 1990, is estimated to be approximately ten thousand.

In **Germany**, the use of traffic circles declined in the 1950's because of the lack of suitable capacity predictions, bad accident experience and the misinterpretation of the priority rule. Recently however, traffic circles have once again attracted attention. This was mainly due to the change in the priority rule and the subsequent capacity improvements as well as the good experience regarding traffic safety. This experience was confirmed in the neighbouring countries of France and Switzerland. Although traffic circles were introduced rather hesitantly and mainly as a traffic calming measure, its excellent qualities regarding traffic operations were soon realized (Stuwe, 1991).

Simon (1991) asserts that the **Swiss** people are renowned for their skills, accuracy and technical abilities. These attributes lead them to aim to perfect road traffic management with modern sophisticated traffic signal systems optimized with computer software packages which allow for every traffic demand (Simon, 1991). This was the reason why until the late 1970's, hardly any traffic circles were present in Switzerland. However, when they became aware of urban quality and environmental problems increased, the need for more compatible traffic solutions grew. Subsequently, the first traffic circles appeared in the French-speaking part of Switzerland in the late 1970's. The early 1980's saw a large change in perceptions which were mainly due to the developments in France where increasing numbers of circles were constructed. The famous "right of way" rule was abandoned and replaced by the "give way"-rule in 1983. This evolution was adopted by the Swiss traffic engineers and in 1985 the first modern traffic circles were introduced in Switzerland (Simon, 1991).

In **Norway** small traffic circles have become popular with the total number of circles increasing from 15 in 1980 to approximately 400 in 1992. Prior to 1980, as in many other countries, there was little interest shown in traffic circles. It was thought that this type of intersection took up too much space while the driver population were accustomed to the nearside priority rule. In the mid 1970's the priority rule at some existing traffic circles was changed. This change led to a substantial improvement in traffic flow and a reduction in accidents. This experience, supported by the positive feed-back from Britain, led to the construction of many small traffic circles at the beginning of the 1980's (Seim, 1991). Again, there was a certain amount of scepticism and mistrust both among traffic engineers and the public at large. It was generally thought that British road users showed greater tolerance, flexibility and caution than the Norwegians, and that traffic circles would be more suitable in the UK than in Norway. It was assumed that priority rules and traffic signals at intersections were the better option for Norwegian drivers. However, several follow-up studies of the newly implemented traffic circles have shown that Norwegian drivers also cope extremely well with this type of intersection control. A remarkably low incidence of accidents and personal injury was recorded, while capacity parameters such as critical lags, gaps and follow-up time seem to be just as low as the British values. From this experience, Seim (1991) concluded that road users are probably as flexible and considerate as the system allows.

The experience in **Sweden** follows almost the same pattern as in other European countries. Many traffic circles here originated in the nearside priority era. During that time until the change to offside priority in 1967, circulating traffic had to yield to entering traffic (nearside priority rule), which often caused traffic jams so that many traffic circles were converted to signalised control intersections. After the change to offside priority, traffic circles gained renewed popularity as a type of intersection with high capacity and with comparatively low construction and operational cost (Cedersund, 1988).

In Czechoslovakia (Jirava and Karlicky, 1988), Israel (Hakkert et al, 1991), Poland (Tracz, 1991), and the countries in North America (Yagar, 1992) the use of small diameter traffic circles and the offside priority rule has not been so successful. In Poland, the attractiveness of traffic circles has decreased since the introduction of the offside priority rule in 1984. A recent international survey by the Institute of Transportation Engineers (ITE) indicated that traffic circles have failed to a large extent in the United States and Canada (Yagar, 1992).

The first priority ruling at traffic circles in the **United States** was under the Common Law which ruled that he who was there first had the Right of Way (Todd, 1988). As this caused much debate in court, a definite priority ruling was sought which resulted in the nearside priority rule. The increase in traffic volumes during the early parts of this century caused many traffic circles on high volume roads to lock during peak hours. The solution to these problems was to cut through the central island and to install traffic lights. However, this caused all sizes of traffic circles to fall out of favour. The introduction of the offside rule during 1954 to 1977 made no difference to the attitude towards circles in the United States. Moreover, highway engineers showed no interest in introducing more advanced designs to utilize the properties of this new type of control that was leading to a revival of the small circle around the world (Todd, 1988). Since the 1950's, traffic circles have often been used as speed control measures in residential areas around the US, but generally the highway engineering profession has shown a degree of hostility towards their construction (Todd, 1988).

2.2 Local history of traffic circles

Conventional traffic circles - which could be classified as gyratory systems - can be found in many rural towns in South Africa. The circles are usually in the main street and often encircle a major church; the encircling serves to highlight and emphasize the location. Because of the low traffic volumes in most of these towns, the circle is underutilised and its ability as a traffic control measure is probably underestimated and unappreciated (Krogscheepers & Roebuck, 1993).

Small circles controlling substantial traffic volumes close to their approach capacities have been viewed with mistrust and suspicion both by authorities and road users, so that this form of control has not been fully exploited (Schermers 1987). Various studies have been conducted by among others the CSIR (Earey, 1985; Schermers, 1987), which have highlighted the advantages of traffic circles in terms of safety and user costs. Nonetheless, traffic circles are still only used to a limited extent. The reasons for this situation are probably as different and diverse as they are subjective and could include some of the following:

i) The negative attitudes adopted by many decision makers in various authorities across the country (Schermers, 1987).

ii) The perception that "traffic circles will not work in South Africa" (Schermers, 1987), which corresponds with the view held in Norway; "*... our drivers are not as tolerant, flexible and cautious as the drivers in the UK and in Europe.*"

iii) The opinion that traffic circles require a large amount of land, such as those in Welkom (Sutcliffe, 1990).

iv) Because the traffic flow on rural roads in the RSA is much less than in the UK, it is probably less likely to justify traffic circles in terms of a benefit/cost analysis, especially for typical traffic circles which require larger tracts of land (Van As, 1991).

v) Drivers not educated sufficiently to the correct use of a traffic circle. The lack of use of turn indicators by drivers on traffic circles reduces the operational efficiencies of the control (Schermers, 1987).

vi) A lack of detailed design criteria (Schermers, 1987)

Although research by Gorton (1978), Earey (1985) and Schermers (1987) has indicated the potential benefits that could be realised by using traffic circles, it was only recently (Short and Van As, 1992) that a new interest in this form of control developed. In his study, probably the most comprehensive to date in the RSA on this subject, Schermers (1987) concludes that traffic circles significantly reduce the stopped delay and number of stops at intersections where the intersection volume is less than 4000 vehicles per hour, when compared with other forms of control.

Recently, mini circles have been installed in Johannesburg in the suburbs of Melville, Greymont, Northcliff and Fairland. The introduction of one circle led to the installation of several more after positive response from the residents. Subsequently the intention was to replace 4-way stops with traffic circles. Most of these circles have been raised by approximately 100 mm above the adjacent roadway - using semi-mountable kerbs (Short and Van As, 1992). To shed some light on statements such as "mini circles will not work in South Africa" a mini circle pilot study was set up in 1989 in Pretoria with the aim of determining acceptability to the motorist and the motorists' reactions to this new type of control (Van As, 1989). The positive results of this study led to the installation of more mini traffic circles in Pretoria. Most of these circles were of the painted island type with no raised central island (Jordaan and Joubert, 1991).

2.3 Research on design and operating parameters for traffic circles: Capacity

The capacity of a traffic circle can be defined as the sum of the maximum hourly traffic inflow from all approaches to the circle when the demand flow on these approaches is sufficient to cause steady queuing on each approach. Prior to the introduction of the offside priority rule the entering traffic had priority and had to weave into gaps in the circulating traffic. With the introduction of the offside priority rule, traffic waiting to enter the traffic circle has to give way to traffic already on the circulating roadway and consequently the entry capacity will decrease as the circulating flow increases, since there are fewer opportunities for entry. The dependence of entry capacity on circulating flow is known as the entry/circulating flow relationship, which in turn depends on the traffic circle geometry (Kimber, 1980). The basic task in capacity estimating is to define how this flow relationship may be predicted from a knowledge of the geometric layout and opposing circulating traffic flow. In this section the developments in capacity estimating for traffic circles is discussed under two headings, firstly developments prior to the introduction of the offside priority rule and secondly developments after the introduction of the offside priority rule.

2.3.1 Developments prior to the introduction of the offside priority rule

Most of the early traffic circle design was not based on any capacity considerations although two analytical models existed as proposed by Watson (1933) and Royal Dawson (1936). These models were based on the assumption that the capacity of a weaving section depends on its width, the so-called "throat capacity". Assuming a safe speed of rotation and fixed vehicle dimensions, Watson showed that for each intersecting point of two converging vehicle paths, the practical capacity for small vehicles was 1500 vehicles per hour and 1000 vehicles per hour for larger cars. Royal Dawson pursued the same approach, but found that the merging capacity of two streams was to be 800 vehicles per hour. Due to a lack of experimental evidence neither of these models were validated.

Clayton (1945) expanded on this concept of the "throat capacity" by considering the weaving capacity (Q_w in vehicles per hour) of a section of road and he expressed it as follows:

$$Q_w = F_w \, N_L \, S_d \tag{2-1}$$

where F_w (weaving factor) was empirically related to the geometry as follows:

$$F_W = 1 - \left(\frac{\alpha}{90}\right)\left(1 - \frac{4}{3N_L}\right) \tag{2-2}$$

and where,
- S_d - Saturation Density of a lane (vehicles per hour)
- N_L - Number of Lanes in the section under consideration
- α - Weaving angle in degrees

The weaving factor was later modified by Clayton (1955) to incorporate the effect of the proportion of weaving traffic (ρ = ratio of smaller weaving volume to total traffic volume) and he replaced N_L with w, the width of the weaving section:

$$F_W = 1 - \left(\frac{\frac{8\alpha}{9}(1 + 2\rho)}{90} \right)\left(1 - \frac{40}{3w}\right) \qquad (2\text{-}3)$$

The saturation density (S_d) is obviously not straightforward to measure and hence conflicting results could be expected from Clayton's expression. In order to study the capacity of a single weaving section the then Road and Research Laboratory (RRL) commissioned an experiment at Northhold Airport in 1955 (Wardrop, 1957) of which the results indicated the following factors to be important in predicting capacity (See Figure 2.3):

i) Weaving width, w, in metres
ii) Average entry width, $\bar{e}$, in metres where $\bar{e} = \frac{1}{2}(e_1 + e_2)$
iii) Weaving length, L, in metres
iv) Proportion of weaving traffic, ρ_w
v) Proportion of heavy and medium vehicles, ρ_{hv}

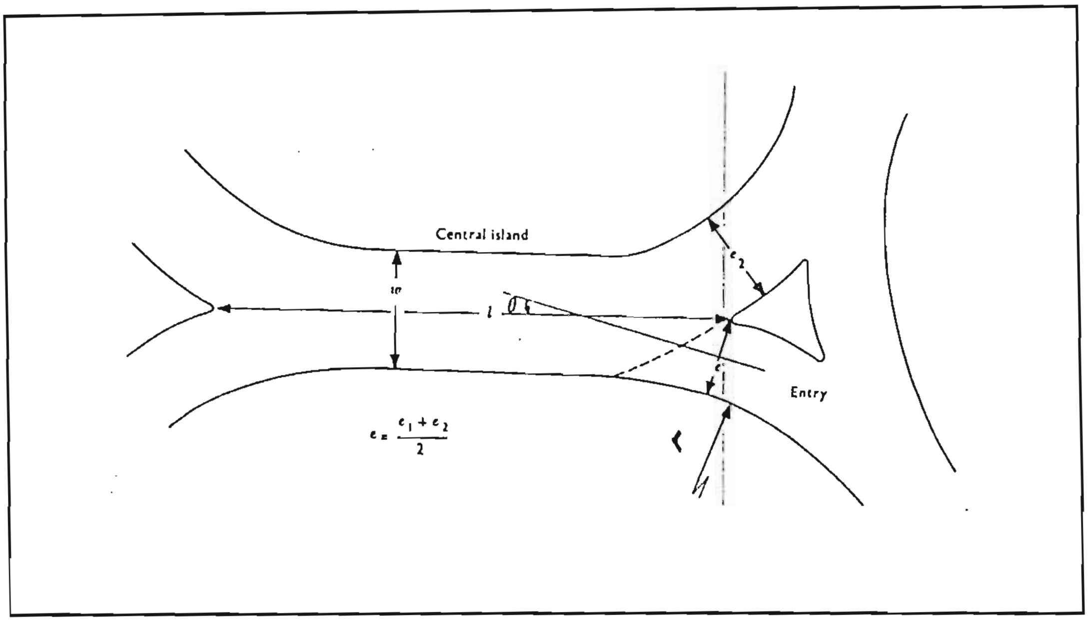


Figure 2.3: Traffic circle section notation (Ashworth & Laurence, 1978)

The initial regression equation fitted to the data observed during the Northhold Airport experiment included variables for the above factors. Wardrop (1957) modified the initial regression equation, considering traffic as passenger car units (pcu) and thus excluded the variable ρ_{hv} (proportion of heavy and medium vehicles) from the initial equation. His modified regression equation subsequently referred to as Wardrop's formula, gave the total capacity (Q_w) of the weaving section in pcu/h as follows:

$$Q_w = \frac{354\, w\, (1 + \frac{\bar{e}}{w})\,(1 - \frac{p_w}{3})}{(1 + \frac{w}{L})} \qquad (2\text{-}4)$$

Subsequent road trials indicated that Q_w was not reached at a number of sites and that the average capacity was in the order of 0,85 Q_w. Some authors argued that because of non ideal conditions the design capacity should be about 80 percent of the total weaving capacity (Philbrick, 1977). Since the traffic stream does not consist entirely of passenger cars, the practical vehicular capacity should be further reduced according to the number of heavy vehicles in the stream. Wardrop used a pcu value of 2,8 for buses and heavy vehicles and 0,5 for two wheeled vehicles.

Equation (2-4) enabled traffic engineers to estimate the capacity of a traffic circle from a knowledge of a series of simple geometric characteristics which were relatively simple to obtain. Consequently, this equation was used and researched widely (McDonald & Armitage, 1978). Asproth (1961), Ogland (1962) and Reid (1961) showed that Wardrop's formula is useful although adjustments for local conditions had to be made in some cases.

2.3.2 Developments after the introduction of the offside priority rule.

The introduction of offside priority in 1966 resulted in fundamental changes in the mode of operation and consequently capacity was not being affected (Schermers, 1987) by weaving manoeuvres anymore but by:

i) drivers using gaps in the circulating traffic stream;
ii) the number of entry and circulating lanes; and
iii) approach entry widths.

Together with changes in geometric design principles to exploit the priority-to-the-right rule (offside rule) and changes in driver and vehicle characteristics implicit in Wardrop's formula, the validity of the formula was questioned. This also prompted a number of investigations and research projects to improve the capacity predictions.

Various experiments were conducted to validate the effectiveness of the change to the offside rule. A number of these were completed at sites where the rule already applied even prior to its official introduction. Asproth (1961), Faulkner (1967) and Reid (1961) indicated an increase in flow and a reduction in journey times even though at some of the locations the drivers already followed the priority rule before the signage to affect that was officially erected. Blackmore (1963) concluded that the offside priority rule could be expected to relieve the congestion at traffic circles by increasing capacity by some ten percent, reducing traffic delays and accidents by 40 percent.

The geometric changes as a result of the introduction of the offside rule prompted the use of smaller circles with flared approaches which in turn resulted in the classification of the various circle types; Conventional, small-, mini- and double circles (see Section 1.4) The size of traffic circles constructed in Australia and the priority rule decreased the tendency of drivers to weave at traffic circles (Troutbeck, 1984c). Horman and Turnbull (1974) found that less than 1 percent of circulating drivers gave way to entering traffic when the circulating flows were greater than the entry flows. This situation led to the use of smaller traffic circles with flared approaches and smaller central island diameters for improved capacity. Blackmore (1970) indicated that the capacity of a small central island traffic circle was about a quarter greater than that of a traffic circle with a larger central island. This was confirmed by Edgar(1980) in New Zealand. Although the mode of operation at all these circles was the same, the tendency was to develop unique methods of estimating capacity for the different types of traffic circles. The different capacity models which have evolved are discussed in the remainder of this section.

In the search for models to estimate capacity at traffic circles, two main modelling approaches have emerged: **analytical modelling** and **empirical modelling**. Analytical modelling aims to represent the actual behaviour of drivers and their vehicles and is based on gap acceptance behaviour, trying to relate entry flow to circulating traffic and geometric characteristics. Empirical modelling aims to relate the same variables but purely from regression on observed data sets. Both methods have advantages and disadvantages and have been criticised. Recently, with the advent of the personal computer, **simulation models** have also become an attractive and viable alternative for modelling traffic flow through traffic circles. In the remainder of this section the capacity models are discussed in terms of:

i) empirical models,
ii) analytical models, and
iii) simulation models.

2.3.2.1 Empirical models

Most capacity models developed prior to 1966 were of an empirical nature relating entry volume to geometric and traffic parameters with the help of regression on experimental data. Although the validity of Wardrop's formula was doubtful, efforts were made to improve it to apply especially at conventional circles with long weaving lengths even after the introduction of the priority rule. Cherry (1968) reports a change which included not only the proportion of weaving traffic as before but also the proportion of the two weaving movements:

$$Q_w = \frac{354\, w\, (1 + \frac{\bar{e}}{w})\,(1 - \frac{2R\rho}{3(1 + R)})}{(1 + \frac{w}{L})} \qquad (2\text{-}5)$$

where :	Q_w	-	Theoretical capacity of weaving section in pcu per hour
	w	-	Weaving width in metres
	$\bar{e}$	-	Average entry width in metres [$\bar{e} = ½(e_1 + e_2)$]
	L	-	Weaving length in metres
	ρ_w	-	Proportion of weaving traffic, [weaving / total traffic]
	R	-	Ratio of smaller weaving stream to larger weaving stream S_1/S_2
	S_1	-	Smaller weaving stream volume
	S_2	-	Larger weaving stream volume

It was subsequently found that the weaving capacity was insensitive to changes in the proportion (ρ_w) of weaving traffic (Ashworth and Field, 1973), thus casting further doubt on the total validity of Wardrop's formula. However, until the 1970's, the lack of any substantiated design formula encouraged engineers to revert to Wardrop's formula to estimate the capacity of traffic circles (Troutbeck, 1984c). From empirical studies done by Murgatroyd (1973), Freeman, Fox and Associates (1974), and Wooton and Jeffreys (1975), the Department of Environment (UK) consequently introduced a modified version of Wardrop's formula to be used for conventional traffic circles. This formula excluded the proportion of weaving traffic (ρ_w), allowed for 15% heavy vehicles, and the constant (354) was adjusted to reflect the weaving traffic more accurately:

$$Q_w = \frac{160\, w\, (1 + \frac{\bar{e}}{w})}{(1 + \frac{w}{L})} \quad veh/h \qquad (2\text{-}6)$$

with the symbols as defined before (Philbrick, 1977) and applicable to the following ranges (see Figure 2.3):

$$9{,}10 < w < 18{,}00$$

$$0{,}63 < \bar{e}/w < 0{,}95$$

$$0{,}16 < w/L < 0{,}38$$

$$0{,}34 < e_1/e_2 < 1{,}14$$

A further recommendation was made that 85% of Q_w be used for design purposes and that corrections were required for more than 15% heavy vehicles. This formula was not necessarily chosen for being the most accurate capacity predictor with least residual scatter, but as an interim replacement for Wardrop's formula.

While Wardrop's formula for estimating traffic circle capacity was still under review, the Transport and Road Research Laboratory (TRRL) in the UK investigated possible alternatives using test track and public road experiments to find ways of making better use of the area available at intersections. It was found that the capacity of relatively large traffic circles (large central island with a narrow circulating roadway) could be improved by a reduction in the size of the island and by deflecting the entering traffic to encourage circular movement (Blackmore, 1970). The optimum size for the diameter of the central island was recommended as a third of the inscribed circle's diameter. A simple empirical formula (Blackmore formula) developed at the TRRL for mini traffic circles was as follows:

$$Q = K_b \left(\sum W + \sqrt{A_w} \right) \tag{2-7}$$

where

Q	=	total capacity or full capacity (vehicles per hour)	
$\sum W$	=	sum of all approach widths (m)	
A_w	=	total area of widening (m^2) outside the area of basic cross roads	
K_b	=	Efficiency coefficient	(See figure 2.4)

In a number of studies by Sawers and Blackmore (1973), Halshall and Blackmore (1975) and Marlow and Blackmore (1973), Blackmore's formula was tested and the values for the efficiency factor (K_b) evaluated. Edgar (1980) verified that in New Zealand this total capacity formula for small traffic circles appears to be a useful guide.

The apparent success of the Blackmore formula prompted the UK Department of Transport in 1971 (Highway Directorate) to issue recommended values for the factor K_b to be used for mini- and small circles:

3 way intersection	K_b = 80 pcu/h
4 way intersection	K_b = 70 pcu/h
5 way intersection	K_b = 60 pcu/h

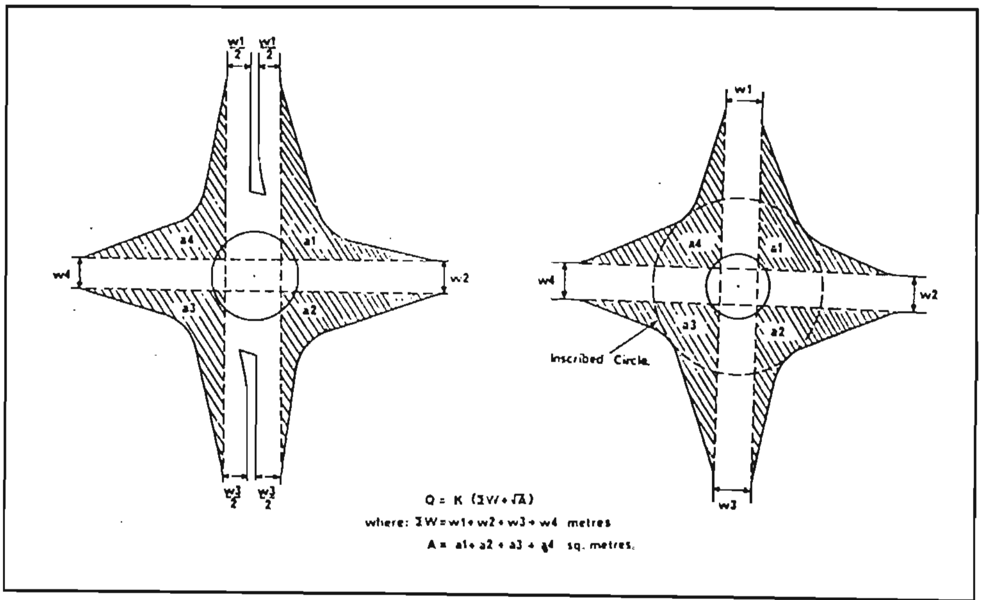


Figure 2.4: Basic geometric dimensions in Blackmore's capacity equation.
Source: Highway Directorate (1973)

In 1975 (Highway Directorate) these values were adjusted with different values for mini- and small circles:

Small circles	3 way intersection	K_b = 70 pcu/h
	4 way intersection	K_b = 50 pcu/h
	5 way intersection	K_b = 45 pcu/h
Mini circles	3 way intersection	K_b = 60 pcu/h
	4 way intersection	K_b = 45 pcu/h
	5 way intersection	K_b = 40 pcu/h

Blackmore's formula, which only considers geometric factors, calculates the full capacity of the circle assuming saturation on all approaches. This situation occurs seldom and when it does it is improbable that all the weaving sections will be fully utilised. As Blackmore's formula is based on the total area available to be used by vehicles, it can then certainly be prone to errors especially when all the approaches are not saturated. The efficiency factor recommended by the UK DOT reflects this as the intersection with more arms is less likely to operate at full capacity. Chin (1983) argues that if this is true then the value of K_b should also depend on the traffic characteristics.

Poole (1973) suggested a formula similar to that of Blackmore, but instead of using the road width (w) and area of widening (A) he suggested $Q = K_d D$ where D is the inscribed circle diameter and K_d a constant equal to 150 for three way intersections and equal to 140 for four way intersections.

Although most of the subsequent research done in the UK in search of a new capacity formula was based on analytical methods the formula that was eventually accepted and which is used today is based on empirical studies. In order to simplify the computational procedure of Tanner's (1962) theoretical gap-acceptance model, Maycock (1974) suggested that a linear approximation of the curve be used. A straight line fitted to data of observed circulating and entering traffic provides at least as good a fit as a curvilinear relationship predicted by the gap-acceptance theory (Philbrick, 1977). Both these methods relate entry capacity to circulating flow with the following straight line relationship as first suggested by Maycock (1974):

$$Q_e = F - f_c Q_c \tag{2-8}$$

where

Q_e	-	Entering flow (pcu/h)
Q_c	-	Circulating flow (pcu/h)
F	-	Intercept
f_c	-	Slope

and with the parameters F and f_c depending on the geometric characteristics of the traffic circle. Philbrick (1977) established a pcu-value of 2,00 for heavy vehicles and continued using stepwise regression analysis to relate the terms F and f_c to ten geometric parameters. He concluded that the optimal equation for the intercept involved only the entry width (e_1)and the radius (r_1) (see Figure 2.5):

$$F = 233e_1 \ (1{,}5 - \frac{1}{\sqrt{r_1}}) - 255 \tag{2-9}$$

Similarly, the optimal equation for the slope (f_c) involved only two variables, the one being the entry width (e_1) and the other the weaving width (w) (see Figure 2.5):

$$f_c = 0{,}0049\ (2e_1 - w) + 0{,}282 \tag{2-10}$$

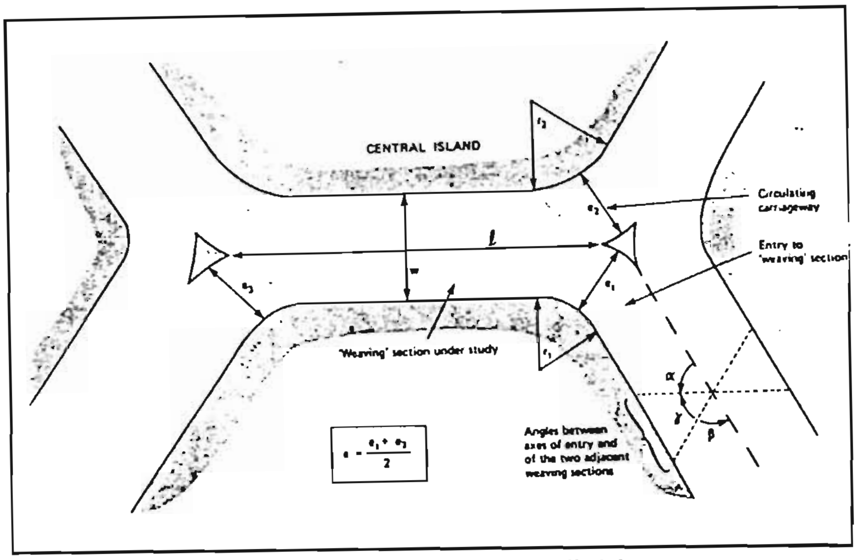


Figure 2.5: Geometric parameters studied by Philbrick

Source: Philbrick (1977)

The circulating traffic in Philbrick's data set was between 580 and 3890 vehicles per hour with the other variables in the following ranges:

$$4{,}00 < e_1 < 12{,}50$$

$$-2{,}50 < 2e_1 - w < 9{,}50$$

$$0{,}74 < \frac{e_1}{\sqrt{r_1}} < 3{,}30$$

Subsequently, numerous studies were done to develop relationships which could predict the values of F and f_c. The three reports by Kimber and Semmens (1977), Glen, Summer and Kimber (1978) and Kimber (1980) all described slightly different relationships. Kimber (1980) developed equations for large and small traffic circles using the data from Philbrick (1977), Kimber and Semmens (1977), Glen et al. (1978) and Ashworth and Laurance (1977;1978). The optimal predictive equation using a pcu-value of 2,00 for heavy vehicles as recommended by Kimber (1980) was as follows:

$$Q_e = k\ (F - f_c\ Q_c) \tag{2-11}$$

where:

$k = 1 - 0,00347\,(\phi - 30) - 0,978\,(r^{-1} - 0,05))$

$F = 303\,x_2$

$f_c = 0,210\,t_D\,(1 + 0,2\,x_2)$

$t_D = 1 + 0,5\,(1 + \exp(10^{-1}\,(D - 60))^{-1}$

$x_2 = v + (e - v)\,(1 + 2S)^{-1}$

$S = (e - v)\,l^{-1}$ or $1,6\,(e - v)\,(l')^{-1}$ (See Figure 2.6 for explanation of terms)

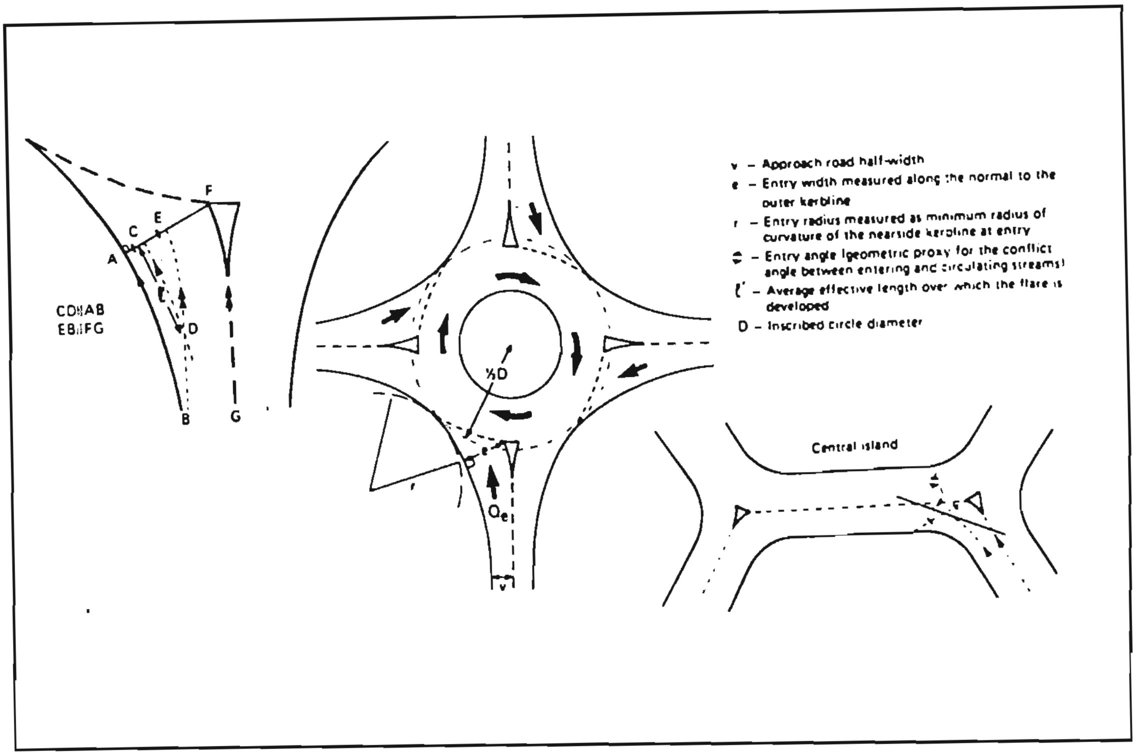


Figure 2.6: Definitions of Geometric Parameters

Source: Webb and Pearce (1990)

The equations were derived from the following ranges for the different parameters:

16,5	>	D	>	3,6 m
12,5	>	v	>	1,9 m
2,9	>	S	>	0,0 m
171,6	>	D	>	13,5 m
		r	>	3,4 m
		l	>	1,0 m

The above equations are very involved and it is possible to simplify them with some reasonable assumptions (Troutbeck, 1984c). For small diameter traffic circles the term t_D can be set to 1,48 and the k term can be ignored as it produces only small changes. The results of Kimber's regression studies showed the main determinant of entry capacity to be the effective width (x_2) of the approach entry (see Figure 2.7), with factors like angle of entry, kerb radius, overall size of traffic circle and time of operation having a smaller influence.

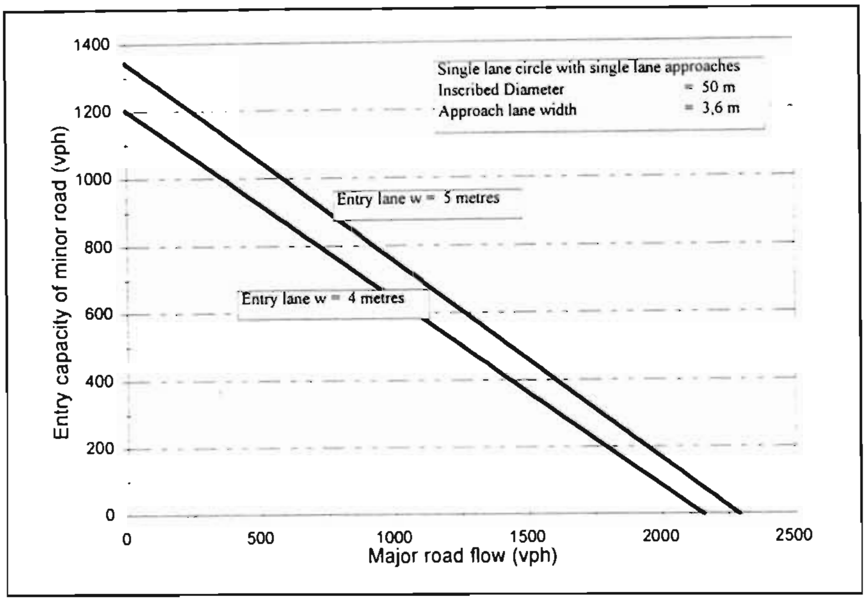


Figure 2.7: Effect of entry width on Kimber's equation

The computer program ARCADY developed by the TRRL (Webb & Pearce, 1990) incorporates Kimber's model in estimating capacities for traffic circles. It is not a simulation program, but basically uses the above linear relationship together with the appropriate geometric characteristics to balance inflows into the circle and to estimate capacities entry by entry, as well as queues, delay and accident risk. This program is also incorporated in the DOT's (UK) advice and standards for traffic circle design, which is used extensively in the UK. Other countries in Europe also make use of the TRRL method to estimate capacities for traffic circles - such as Czechoslovakia (Jirava and Karlicky, 1988).

Kimber also tested the validity of the linear form of the equation by fitting a second order empirical model to the data. He concluded, however, that the parabolic function added no significant improvement to the predictive ability of the linear model and hence accepted the linear model.

The **French** (Louah, 1988) also used statistical analysis (regression) for estimating capacity. The following capacity formula for rural areas was established by linear regression:

$$Q_e = (1330 - 0{,}7Q_i)\ (1 + 0{,}1\ (\nu - 3{,}5)) \qquad (2\text{-}12)$$

where Q_i is the impeding flow defined as:

$$Q_i = (Q_c + \frac{2}{3} Q_l (1 - \frac{c}{15}))\ (1 - 0{,}085(u - 8)) \qquad (2\text{-}13)$$

and where

Q_e	=	entering flow on approach
Q_c	=	circulating flow passing the approach
Q_i	=	impeding flow
Q_l	=	circulating flow exiting circle at the approach
v	=	approach road half width
u	=	width of circulating carriageway
c	=	width of approach island parallel to circulating carriageway

All widths are in metres and flows in pcu per hour with a pcu value of 2,0 for heavy goods vehicles. Louah (1988) stressed that the above formula is applicable to rural areas and that much higher capacities could be expected in urban areas.

In **Germany**, traffic engineers also reverted to the use of statistical techniques as preliminary studies (Brilon and Ohadi, 1988) indicated that gap-acceptance theory could not easily be applied to traffic circle entries. Regression techniques were applied to data obtained from observations made at 10 different traffic circles in Germany (Stuwe, 1991). This resulted in the following exponential regression equation to describe the relationship between entering and circulating traffic:

$$Q_e = A\, e^{-\frac{B\, Q_c}{10000}} \qquad (2\text{-}14)$$

where	Q_e	=	entering flow on approach (pcu/h)
	Q_c	=	circulating flow passing the approach (pcu/h)
	A,B	=	parameters depending on geometrics (see Table 2.1).

The above flows are in pcu's per hour with a pcu value of 2,0 for heavy vehicles and 1,5 for light trucks. An exponential expression was used because of the similarity to gap-acceptance theory. Equation (2.14) proved to be marginally better (larger correlation coefficient) than the linear approximations which were used in the UK by Kimber (1980), in France by Louah (1988) and in Switzerland by Simon (1988) and have been included in a computer program KREISEL which is also able to calculate capacities according to the Australian, French, Swiss and TRRL formulae.

Table 2.1: Values for the geometric parameters A and B (Brilon and Stuwe, 1992)

Number of lanes on		Parameters		Line in Fig. 2.8
Circle	Approach	A	B	
3	2	2018	6,68	a
2	2	1577	6,61	b
2 - 3	1	1300	8,60	c
1	1	1226	10,77	d

A comparison of the German capacity results (shown in Figure 2.8) with those found in the UK (see Figure 2.7) indicates that the German results are between 70 and 80 percent of the English values for single lane circles, whereas the French and Swiss results correspond more favourably with the German results. The differences can only be explained if it is assumed that drivers in the UK are more accustomed to using traffic circles than the German drivers.

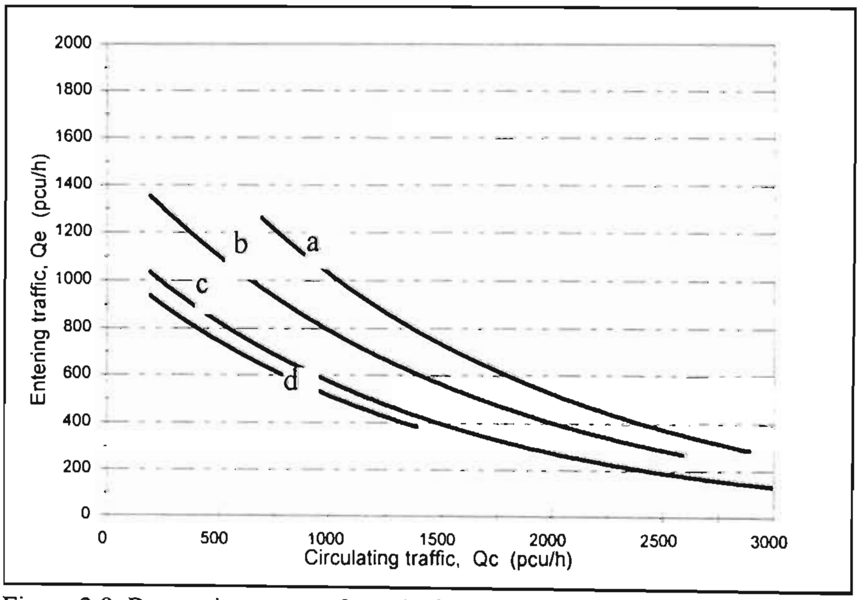


Figure 2.8: Regression curves for calculating capacity of circles in Germany
Source: Brilon et al. (1991)

In Germany, traffic circles are still viewed as being "exotic solutions" (Stuwe, 1988). A possible reason for the difference could be the behaviour of German drivers to orientate by lanes, leaving the left lane (inside lane for driving on the right hand side) of a multilane approach unused, which according to Brilon, Grossmann and Stuwe (1991) could also be the reason why road widths seemingly have little influence on the approach capacity in Germany. Brilon et al. argue that because of the difference in driver behaviour, capacity equations should not be transferred internationally. *"Instead, each country has to find a solution of its own"*.

To follow the trend in other countries where the entry capacity is also related to other geometric parameters (Kimber, 1980; Troutbeck, 1989; Louah, 1988) and not only the number of entry and circulating lanes as in (2-14) Brilon and Stuwe (1992) investigated larger databases of traffic circle flows. They conclude that the inscribed circle diameter (D), the number of approaches to the traffic circle (n_e) and the distance (c) between exit and entry of the observed arm have a significant influence on the capacity of the entry as shown in the following preliminary regression equation:

$$Q_e = Q_{eb} + f_n(D) + f_n(c) \qquad (2\text{-}15)$$

where:

$$\begin{aligned} Q_{eb} &= 1548{,}71e^{\,0{,}00084Q_c} + 208{,}43n_e + 48{,}02n_c \\ f_n(D) &= 60{,}03D - 7{,}38D^2 + 0{,}152D^3 \\ f_n(c) &= -99{,}21c + 4{,}37c^2 - 0{,}0477c^3 \end{aligned} \qquad (2\text{-}16)$$

with n_c the number of circulating lanes and all other symbols as defined before.

A recent comparative study in **Israel** (Hakkert, et al, 1991) showed that gap-acceptance methods give a good estimation of capacity for only "medium capacity" traffic circles. This study also showed that the UK method (2-11) using the empirical model ARCADY, tends to over-estimate capacity while the French model (2-12) under-estimates capacity. The over-estimation by the UK model may be expected since traffic circles are not as common in Israel as they are in the UK.

Two parallel research programmes in **Switzerland** resulted in the development of two different formulae to estimate traffic circle entry capacity. One of these methods did not take the exiting traffic into account while the other one did. These two methods, however, give similar results (Simon, 1991). The method developed by Emch and Berger (Simon, 1991) uses two formulae to describe entry capacity (Q_e - pcu/h) of single lane entries in terms of circulating flow (Q_c - pcu/h) for urban traffic circles (25m<D>40m). The first formula:

$$Q_e = 1300 - 0{,}75Q_c \qquad (2\text{-}17)$$

is applied for urban traffic circles with one circulating lane (25<CID<30m) where there are no special geometric or traffic conditions and the second formula :

$$Q_e = 1450 - 0{,}95Q_c \qquad (2\text{-}18)$$

is used for entries with one lane and a bus lane, for a widened entry lane, or where the entry flow is more than 1000 pcu/h. For any double lane entries, the capacity is taken to be 1,4 times that of a single lane entry. To calculate pcu values, a coefficient of 2,0 is used for heavy trucks.

The second method was developed by Bovy (1991), Tan (1993) and others at the Federal Institute of Technology in Lausanne (Simon, 1991). This method takes into account the exiting flow (Q_l - pcu/h) and as a result the entry capacity is defined by a linear relationship in terms of the disturbing (impeding) traffic flow (Q_i - pcu/h). For a four-legged traffic circle the regression equation is as follows:

$$q_e = \kappa \left(1500 - \frac{8}{9} q_i\right) \qquad (2\text{-}19)$$

with

$$q_i = \beta q_c + \alpha_1 q_l \qquad (2\text{-}20)$$

Where α_1 takes into account the disturbance of the entry flow due to the exiting flow, and was determined as a function of the distance between the conflicting points of exit and entry as shown in Figure 2. 9. β takes into account the number of circulating lanes (Simon, 1991) while κ takes into account the number of entering lanes. Suggested values for these are shown in Table 2.2.

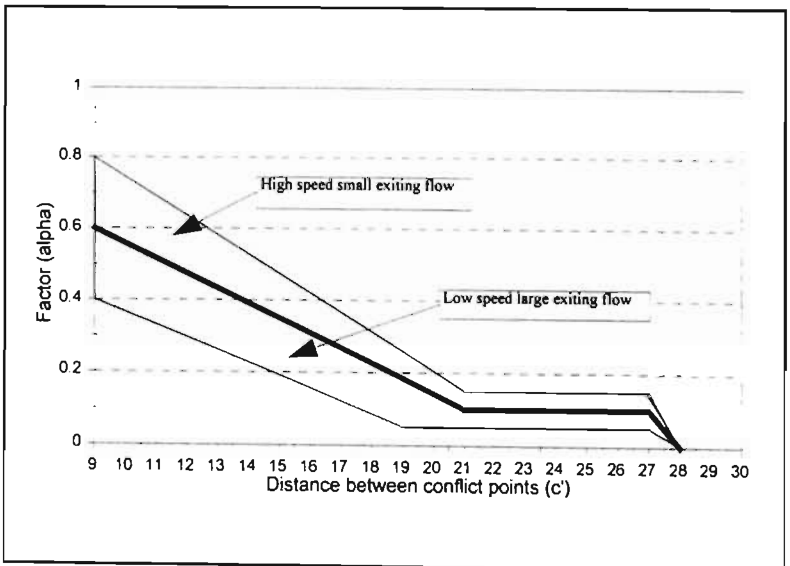


Figure 2.9: Estimation of α_1 (Simon, 1991)

Table 2.2: Suggested values of β and κ (Tan, 1993)

No of Entry Lanes (n_e)	κ	No of circulating lanes (n_c)	β
1 2 3	1,0 1,4 - 1,6 2,0	1 2 3	0,9 - 1,0 0,6 - 0,8 0,5 - 0,6

In **South Africa**, apart from the work done by Short and Van As (1992) - which is on simulation and is discussed in section 2.3.2.3 - little research has been carried out on capacity prediction of traffic circles. The most common method to date has been the application of the TRRL method (eq. 2-11), although lately the usage of the Australian software program, SIDRA (Akçelik and Besley, 1992) based on gap acceptance techniques (see Section 2.3.2.2) has increased considerably (Schermers, 1987 and Green, 1996). Schermers (1987) found that the TRRL method was the most suitable one for capacity prediction in South Africa at the time, while previous work by Earey (1985) and Skutil (1986) suggests that this method overestimates capacities by up to 10%. Sutcliffe (1988) investigated ARCADY2 (which is based on the TRRL method) as a tool for predicting capacities under South African conditions. He concludes that "Reduction of the intercept in ARCADY2 is clearly the most significant factor in achieving results closer to the observed delays". The intercept (F) in (2-11) is a function of the effective width (x_2) of the approach, which in turn is a function of the entry width (e) and the approach road half width (v). The entry width parameter is supplied to the program as the actual width in metres. Sutcliffe recommends that if the entry width and the half-road width used, are calculated in terms of the number of lanes multiplied with a factor of 3,00 - 3,40 more realistic results will be obtained from Arcady2. He recommends no change to the effective length of the flare (l').

2.3.2.2 Analytical Models

With the introduction of the offside priority rule the weaving manoeuvre was no longer the factor determining the capacity of a section but instead it is determined by the gap acceptance behaviour of the driver. Gap-acceptance models constitute an analytical approach and are based on drivers entering the traffic circle only when the gap in the circulating traffic is large enough. Each approach is then considered to be a T-intersection and it is assumed that all minor stream drivers entering the circle will accept gaps in the major stream - circulating traffic - greater than the critical gap, τ, and will follow other minor stream vehicles through a large gap at headways (move-up time) of β_2 (Troutbeck, 1984c). Gap acceptance theory is based on two basic elements. One is the availability of gaps in the main stream of traffic into which entrance is sought and their usefulness to the entering driver, and the other, the size of these gaps and their expected arrival pattern. To deal with the first element it is usually assumed that drivers are consistent and homogeneous, which will ensure that once a gap greater than the critical gap (τ) becomes available, it will be used and if it is much larger it will be used by several minor road vehicles following each other at a constant move-up headway (β_2) (Troutbeck, 1991). To deal with the second element, a theoretical stochastic distribution describing the actual arrival pattern of gaps, needs to be identified. The common ground in most of the analytical models is the assumption of a constant critical gap and move-up time while the proposal for the gap distribution varies.

The analytical capacity estimating methods based on gap acceptance have received much criticism in the past (see next paragraph). Consequently, traffic engineers in many countries - Australia being a notable exception - reverted to regression/empirical techniques on existing data. Troutbeck (1984c) however, argues that gap acceptance techniques offer a logical basis for the evaluation of capacity and that it is easy to appreciate the meaning of the parameters and to make adjustments for unusual conditions. Although driver behaviour has been simplified, for instance by using a single critical-gap, this gap acceptance method still gave better results than that predicted by any of the empirical models developed in the United Kingdom. Troutbeck further argues that it is relatively simple to estimate capacity, but more difficult to estimate delays. Moreover, since it seems logical to use the same mathematical model to estimate delays and capacities, models other than a gap acceptance approach are eliminated because they cannot be extended easily to estimate delays.

The basic criticisms of the gap acceptance method in general have been that it is intrinsically passive in the sense that circulating traffic is assumed not to react to the presence of entering traffic. In addition, the gap-acceptance parameters are assumed to be unaffected by any traffic entering the intersection and also independent of the magnitude of the circulating flow. These assumptions do not hold true in practice since entering traffic often forces entry into the circulating streams and also under high circulating flow conditions some merging takes place during the entry process (Kimber, 1980). Other criticisms have been related to the assumptions used in the theory developments. It has been shown that large variations exist in the critical-gaps accepted by the same drivers whereas in the gap acceptance models a single average critical-gap is used. To support the assumption of consistent and homogenous drivers, Troutbeck (1988b) argues that if drivers were heterogenous, gap acceptance models would tend to over-predict the entry capacity. On the other hand if drivers' decisions were inconsistent, the models would tend to under-predict the entry capacity. Consequently, Troutbeck (1988b) argues that the error in capacity estimates will be small with the assumption of homogenous and consistent drivers.

Most gap-acceptance models are based on Tanner's (1962) model for the prediction of capacity of a single lane, minor road entry onto a major road under priority control, i.e. major road has priority over a minor road and traffic on the latter has to give way and accept suitable gaps in the traffic flow on the major road. Tanner's formula, based on random vehicle arrivals that are distributed negative exponentially is as follows:

$$q_e = \frac{q_c\,(1 - \beta_1 q_c)\,e^{-q_c(\tau - \beta_1)}}{1 - e^{(-q_c\beta_2)}} \qquad (2\text{-}21)$$

where, q_e = maximum entry flow on an approach (veh/second)

q_c = circulating flow at the approach entry (veh/second)

τ = critical gap (seconds)

β_1 = minimum headway of circulating vehicles (seconds)

β_2 = move-up time of minor road vehicles (seconds)

This equation was based on a shifted negative exponential distribution of headways where the shift was based on the minimum possible time gap between two successive major road vehicles, allowing for the fact that vehicles have a finite length and that they do not follow at headways equal to zero seconds. Grant (1969) studied the entry capacities of a number of traffic circles in Aberdeen (Australia) so that they could be compared with other types of intersections. He measured the capacities of the entering lanes as well as the headways between the circulating vehicles. From plots of entry capacity versus circulating flow he estimated the gap acceptance parameters as follows: critical gap τ = 3.8 seconds, move-up times β_2 = 2,7 seconds and minimum gap in circulating flow β_1=1,8 seconds. Assuming a critical gap (τ) of 3,8 seconds, a minimum headway (β_1) of circulating vehicles of 1,8 seconds and a move-up time for minor road vehicles (β_2) of 2,7 seconds, Tanner's equation estimates the entering capacity as shown in Figure 2.10.

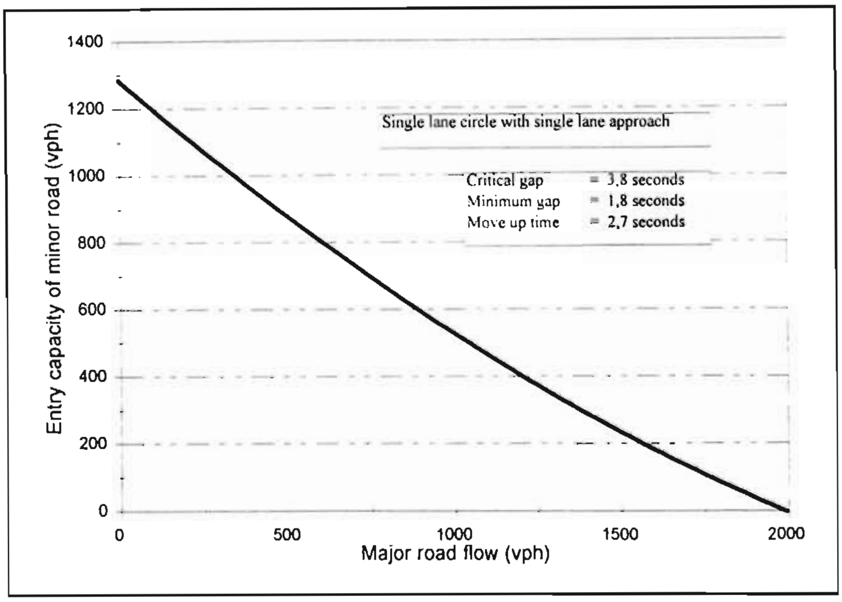


Figure 2.10: Capacity of minor approaches according to Tanner (1962)

Tanner's equation is based on random arrivals of conflicting vehicles and does not allow for platooning which can considerably influence the availability of gaps in the conflicting stream . The original equation was extended by Fisk (1989) to allow for multiple conflicting traffic streams. If it is assumed that Tanner's equation applies, there are two possible approaches (Troutbeck, 1984c). In the first approach, field data can be used to relate gap acceptance parameters to site geometry, which would be an analytical approach using field data to verify the gap acceptance parameters that are once again used in a theoretical model to estimate capacity. The second approach is to approximate a linear equation for Tanner's equation and to then use field data to relate the intercept and the slope of the line to site geometry. The second approach as discussed in Section 2.3.2.1 was adopted by the TRRL.

In 1967, Wohl and Martin did a theoretical analysis of the weaving section of a traffic circle, basing their argument on the gap acceptance behaviour of the minor stream attempting entry into the major stream by means of a weaving manoeuvre. Assuming a random distribution of traffic in the major stream (circulating traffic) and a critical gap of τ seconds for the minor road traffic (entering traffic) the number of vehicles (N_e) over a period (T) that could merge from the minor stream into the major stream - with a flow of q_c(veh/second) - was given by:

$$N_e = \frac{q_c \, e^{-\frac{q_c \tau}{T}}}{1 - e^{(-\frac{q_c \tau}{T})}} \tag{2-22}$$

The weaving capacity (q_w) of the section for the period T is then as follows:

$$q_w = q_c + N_e = \frac{q_c}{(1 - e^{-\frac{q_c \tau}{T}})} \tag{2-23}$$

which could be reduced to:

$$q_w = \frac{(\rho_w + 1)T \ \log_e (\rho_w + 1)}{\rho_w} \tag{2-24}$$

if the weaving ratio of $\rho_w = q_c/q_e$ is introduced, where q_e is the entering flow.

Pursuing Wohl and Martin's (1967) gap acceptance approach to model the capacity of weaving sections, Ashworth and Field (1973) employed a simple M/G/1 (random arrivals, general service distribution and one service channel) queuing model to analyse traffic circle capacities. It was mostly because of their work that it was first realised that the weaving ratio does not affect the capacity of an approach. Using Tanner's (1962) equation, setting the critical gap (τ) equal to the move-up time (β_2) and the minimum gap in the circulating traffic (β_1) equal to zero they derived the following regression equation relating entry capacity (q_e) to circulating flow (q_c):

$$q_e = \frac{2q_c}{(e^{\frac{q_c}{1100}} - 1)} \tag{2-25}$$

Assuming random arrivals with a minimum headway (β_1) of zero, Watson (1974) pursued a similar approach reducing Tanner's equation (Chin, 1983) to:

$$q_e = (\frac{1}{\beta_2}) \ e^{-(\alpha - \beta_2)} \ q_c \tag{2-26}$$

(The symbols as before). Aiming to produce a method of finding the geometric design parameters needed to construct a circle knowing the entry and circulating flow, Watson (1933) produced a cumbersome method with functional deficiencies. His method implied that by increasing the entry width (e) the move-up time (β_2) of the entry vehicles will also increase (Chin, 1983).

According to Chin (1983), Ferguson and Papathassiou (1974) found that Tanner's equation gave a better estimate of mini circle capacities than the equation proposed by Wohl and Martin. Chin argues that while the assumption of a minimum gap of zero in the circulating traffic (β_1) is reasonable for mini traffic circles, the further assumption that the critical gap (τ) is equal to the move-up time (β_2) is restrictive. It can hardly be argued that a minimum circulating gap of zero seconds is acceptable. They subsequently proposed an intermediate range of flows in which Wohl and Martin's equation could be applied to retain the simplicity of their formula, but in the process introduced two extra variables that made the model more cumbersome (Chin, 1983).

Armitage and McDonald (1974) proposed a revision to Tanner's equation to allow for flared entries:

$$q_e = \frac{q_c\,(1 - \beta_1 q_c)}{e^{q_c(\tau - \beta_1)}\,(1 - e^{-q_c\beta_2})} * [n_e - (e^{-q_c\beta_2})^{c_1} - (e^{-q_c\beta_2})^{c_2} - \dots - (e^{-q_c\beta_2})^{c_n}] \tag{2-27}$$

where n_e - Number of lanes at entry

c_i - Number of car lengths that could be stored in the flared lane

q_c, q_e, β_1, β_2 and τ are as before

Armitage and McDonald (1974) also tried to evaluate the gap acceptance parameters used in Tanner's equation by first measuring them and secondly by fitting a capacity formula employing these parameters to the observed flow data using regression analysis (see Table 2.3).

Table 2.3: Gap acceptance parameters (Armitage and McDonald (1974)

	Measured (sec)	Regression (sec)
Critical gap (τ)	3.37	3.56
Minimum Headway (β_1)	1.16	1.17
Move-up time (β_2)	2.32	2.34

Ashworth and Laurence (1978), arguing that driver behaviour is similar to that at priority intersections and assuming as before (Ashworth and Field, 1973) that the critical gap (τ) equals the move-up time (β_2), proposed the following change to Tanner's equation:

$$q_e = \frac{n_e\, q_c}{(e^{q_c t} - 1)} \qquad \textit{vehicles/second} \tag{2-28}$$

where n_e is the number of standard entry lanes (entry width/3,65) and the other variables as before. Letting $A = (3600/\tau)$ the above was rewritten as:

$$Q_e = \frac{n_e\, Q_c}{(e^{\frac{Q_c}{A}} - 1)} \qquad \textit{vehicles/hour} \qquad (2\text{-}29)$$

and with A = 1120 the best fit to the observed data sets was found with $n_e = 1$ it follows line c in Figure 2.8 closely. They also showed that a simplification of the above equation to incorporate a simple linear relationship does not result in a significant difference:

$$Q_e = n_e\,(868 - 0{,}200\, Q_c) \qquad (2\text{-}30)$$

with the variables as before. They recommended that the adapted DOT equation (eq. 2-6) be replaced by one that reflects more precisely the actual mode of operation of traffic circles, and that without conclusive evidence regarding the influence of geometric parameters other than entry width, the simple exponential formula (eq. 2-28) should be used.

Armitage and McDonald (1978) also used Tanner's gap acceptance model as a basis to develop formulae to predict capacity at traffic circles with and without flared entries. They developed the concepts of saturation flow and lost time, assuming that each circulating vehicle has a certain lost time associated with it (time not available for entry) and that vehicles enter the traffic circle at a constant saturation flow rate. A series of theoretical formulae was developed of which the most useful one was:

$$q_e = q_s\,(1 - \beta_1 q_c)\, e^{-q_c (L_t - \beta_1)} \qquad (2\text{-}31)$$

where,

- q_e = entering flow at the approach entry (veh/second)
- q_c = circulating flow at the approach entry (veh/second)
- q_s = saturation flow of the approach (veh/second)
- β_1 = minimum headway of circulating vehicles (second)
- L_t = lost time

Armitage and McDonald (1978) also developed four linear regression models to relate q_s (flared and parallel entries), L_t and β_1 to the specific geometric parameters:

$q_s = 0{,}12v + 0{,}04(e_1 - v)$ vehicles per hour (for parallel entries)

$q_s = 0{,}12(v + F_1(e_1 - v)/(F_1 + 69)$ vehicles per hour (for flared approaches)

$L_t = 2{,}3 + 0{,}006\, r^{-1} - 0{,}04w'$ seconds

$\beta_1 = (o{,}12v' + 0{,}04(e_1' - v'))^{-1}$ seconds

where e_1, v, r and w are as before and measured in metres while the prime represents the geometry of the previous entry.

Gap acceptance techniques for evaluating the capacity and delays at unsignalised intersections have been used by many Australian researchers. Horman and Turnbull (1974) and Avent and Taylor (1979) investigated traffic circle capacity by estimating gap acceptance parameters for use in Tanner's equation. Other Australian researches investigating traffic circle capacity using gap acceptance techniques were Major and Buckley (1962), Pretty and Blunden (1964), Allan (1968), Fry and Buckley (1970), Dune and Buckley (1972) and Uber (1978). Because of all this research Troutbeck (1984c) argues that it is reasonable for Australian design guides to continue to use the gap acceptance method.

Initially Australian traffic circle guidelines used Tanner's equation and modelled traffic circles as a series of T-intersections, assuming homogeneous and consistent driver behaviour, i.e. constant critical gap (τ) and follow-up headway (β_2). Horman and Turnbull (1974) and Avent and Taylor (1979) studied traffic flow at a number of traffic circles in Australia and recommended gap acceptance parameters to be used in Tanner's capacity model. The National Association of Australian State Road Authority (NAASRA, 1986) published the report 'Roundabouts - A design guide' based on the research done on gap acceptance parameters and recommended the use of Tanner's model with the new parameters.

This design guide was superseded in 1993 with Part 6 of AUSTROADS (1993) 'Guide to Traffic Engineering Practice - Roundabouts'. The new guideline incorporated extensive research by Troutbeck which is reported in summary form in the Special Report No. 45 commonly referred to as SR45 (Troutbeck, 1989). The models proposed in SR45 are dynamic where the gap acceptance parameters are not constant but are affected by geometry, circulating flows and entry lane flows.

As the modelling of all aspects of traffic, but specifically the entry capacity of an approach to a traffic circle, depend on the estimate of arrival headways, the Australians concentrated on finding an improved headway distribution model to improve Tanner's model. The research showed that leaving/exiting vehicles (q_l) have only a negligible influence on the entering traffic and are thus not considered when estimating entry capacity. The Australian experience (Troutbeck, 1991) agrees with that in the UK (Kimber and Semmens,1977), which suggests that entering vehicles give way to all circulating vehicles, regardless of the circulating lane occupied by the conflicting vehicle.

Kimber and Semmens (1977) concluded that the traffic leaving at the previous exit, the proportion of vehicles exiting at the next exit and the proportion of vehicles turning left did not have a 'discernible effect on capacity'. Therefore, a single lane model was used to represent the headways in all circulating streams.

The headway model for traffic which has had the greatest application to the problem to date is the bunched exponential distribution first proposed by Cowan in 1975. This will be referred to in this thesis as Cowan's model. Cowan's model (Troutbeck, 1989; Akcelik and Chung, 1994) assumes that a proportion (1 - α_2) vehicles are in platoons or following with a constant inter-platoon headway of β_1 seconds, and α_2 vehicles are free or arriving at random with headways exponentially distributed. The cumulative probability distribtion function for the bunched exponential distribution is thus:

$$\begin{aligned} F(t) &= 1 - \alpha_2 e^{-\lambda(t - \beta_1)} && t > \beta_1 \\ F(t) &= 0 && t < \beta_1 \end{aligned} \qquad (2\text{-}32)$$

where

t - time (headway)

α_2 - Proportion of free vehicles (not in platoons)

β_1 - inter-platoon headway - minimum gap in circulating traffic

λ - decay rate which is related to the circulating flow q_c as follows:

$$\lambda = \frac{\alpha_2 q_c}{(1 - \beta_1 q_c)} \qquad (2\text{-}33)$$

and which gives the entering capacity of a minor approach in vehicles per second as follows (Troutbeck, 1989):

$$q_e = \frac{q_c \alpha_2 e^{-\lambda(\tau - \beta_1)}}{1 - e^{-\lambda \beta_2}} \qquad (2\text{-}34)$$

where q_c is the sum of the flow in all circulating lanes, τ = critical gap, β_2 = average move up time and the other symbols as before. In Figure 2.11 Cowan's headway distribution model based on equation (2-32) is shown for two hypothetical cases.

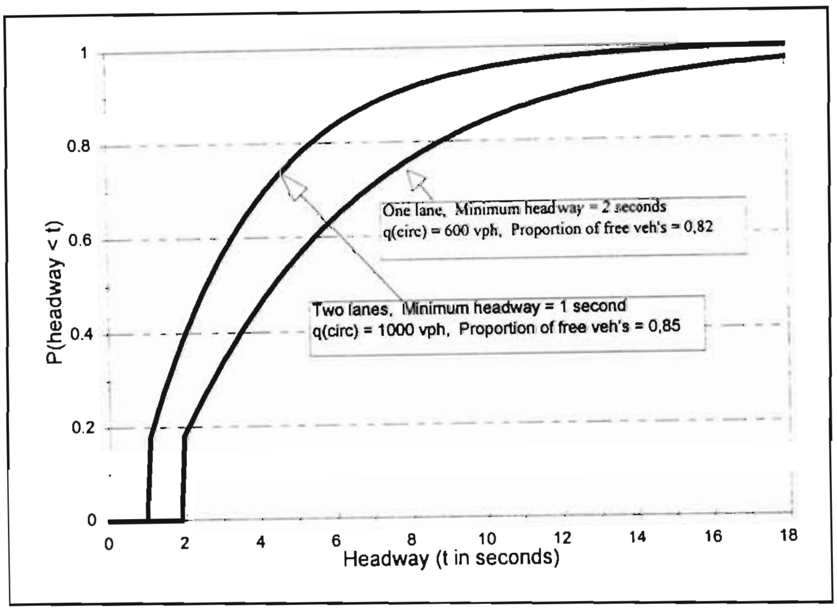


Figure 2.11: Cowan's headway model

The knee of the curve occurs at the point (β_1;1-α_2). The headways smaller than the minimum headway (β_1) are negligible for estimating entry capacity. Figure 2.12, illustrates a comparison of the entry capacities of a single lane approach as predicted by Tanner's equation (2-21) and Troutbeck's (2-34) for a single lane circle. The 'S' shape of Troutbeck's curve is unlike the exponential curve and compares favourably with the estimates of Tanner's equation.

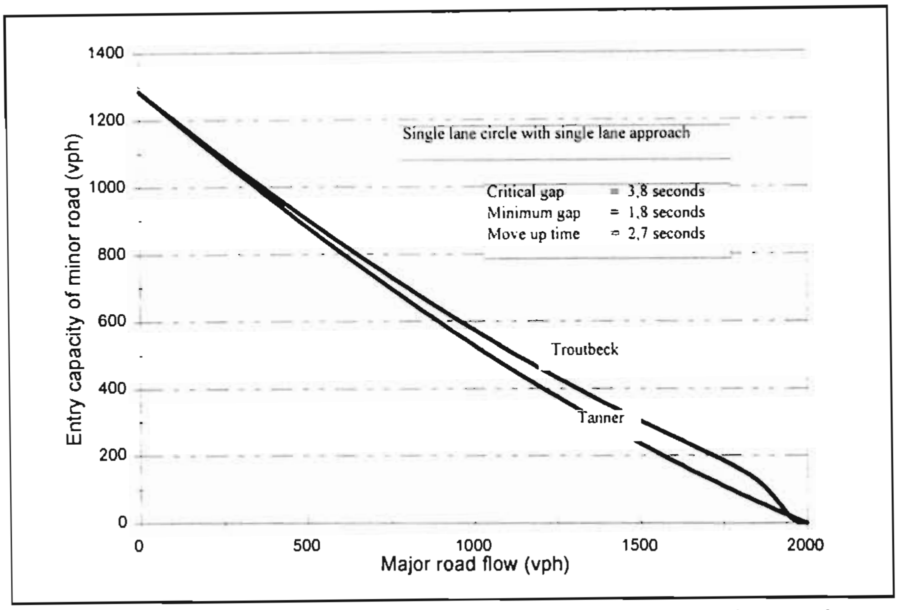


Figure 2.12: Comparing estimates of entry capacity - Tannerr's equation vs Troutbeck's.

Troutbeck's equation includes no reference to any geometric parameters. The influence of the geometrics should be reflected in the gap acceptance and move-up time characteristics of the drivers.

The critical gap (τ), minimum gap in circulating stream (β_1), average move-up time (β_2) and percentage of free vehicles (α_2) in the above equations have all been well researched in Australia and are discussed briefly in the following paragraphs. Of these parameters, the critical gap and the move-up times of minor road vehicles are strongly influenced by the geometrics of the circle (Troutbeck, 1989). The Australian guide for traffic circles (Austroads, 1993) incorporates most of this research and where available provides tables to simplify the calculations.

Troutbeck (1989), reports that there is a difference in **move-up times** (β_2) between vehicles in the dominant stream and vehicles in the sub-dominant stream, where these two streams could be in any of the approach lanes and depends on the lane with the greatest flow. The following regression equation is proposed for the move-up time for the dominant stream (β_{2D}):

$$\beta_{2D} = 3,37 - 0,000394Q_c - 0,0208D + 0,0000889D^2 - 0,395n_e + 0,388n_c \qquad (2\text{-}35)$$

where D is the inscribed diameter, n_c and n_e the number of circulating and entering lanes respectively and subject to $20 \leq D \geq 80$ and $\beta_{2D} \geq \beta_{2DMin}$ ($\beta_{2DMin} = 0,8$ seconds). If outside these ranges, β_{2D} is to be set equal to the maximum if greater than or to the minimum if smaller than. The proposed equation for the move-up time for the sub-dominant stream (β_{2S}) is:

$$\beta_{2S} = 2{,}149 + 0{,}5135\beta_{2D}\frac{Q_D}{Q_S} - 0{,}8735\frac{Q_D}{Q_S} \qquad (2\text{-}36)$$

where Q_D and Q_S are the entry flows in the dominant and sub-dominant streams respectively and subject to $\beta_{2S} \geq \beta_{2D}$. If the average move-up time for the sub-dominant stream (β_{2S}) does exceed that of the dominant stream (β_{2D}) it is set equal to the move-up time for the dominant stream (β_{2D}). The distribution of **critical gaps** (τ) - assumed to be of such a nature that all gaps greater than this gap will be accepted by the driver - has been shown to be log-normally distributed (Troutbeck, 1984a and b). A significant correlation was found between the mean critical gap (τ) and the expected move-up time (β_2), the circulating flow (Q_c), average entry width ($\bar{e}$) and the number of entry lanes n_e (Troutbeck, 1989). The subsequent regression equation relating the critical gap to the other variables is as follows:

$$\tau = (3{,}6135 - 0{,}0003137Q_c - 0{,}339\bar{e} - 0{,}2775n_c)\ \beta_2 \qquad (2\text{-}37)$$

subject to $\tau/\beta_2 \geq 1.1$ and $\tau \geq \tau_{min}$. Again if outside these ranges the critical gap (τ) is set equal to these minimum values. The critical gap can be calculated for either of the dominant or sub-dominant traffic streams by using either β_{2D} or β_{2S}. Note that the ratio $\frac{\tau}{\beta_2}$ will be the same for the dominant and sub-dominant streams. Figure 2.13 shows the critical gap for the dominant stream as a function of the circulating flow.

The **inter-platoon headways** or **minimum gaps** in circulating traffic (Troutbeck ,1989) for circulating traffic of one and two seconds for multi- and single circulating lanes respectively have given suitable results. Troutbeck (1989) proposes the following approximate regression equations for the **proportion of free vehicles** (α_2):

$$\alpha_2 = 0{,}8 - 0{,}0005Q_c \qquad (n_c = 1)$$
$$\alpha_2 = 0{,}8 - 0{,}00025Q_c \qquad (n_c > 1) \qquad (2\text{-}38)$$

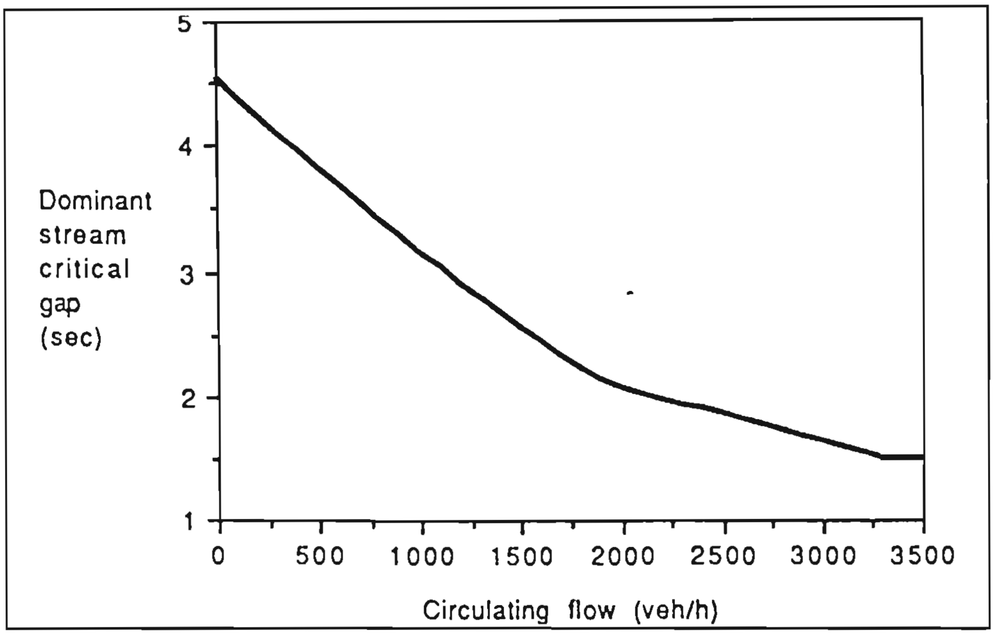


Figure 2.13: Critical gap τ vs Q_c for ($\bar{e}$ = 4,0 m, D = 40 m, $n_e = n_c = 2$)
Source: Akçelik (1992)

The computer program SIDRA 4.07 (Akçelik & Besley, 1992) incorporates most of the above theory with only minor changes to the prediction of the proportion of free vehicles (α_2) (eq. 2-38):

$$\alpha_2 = 0{,}75\ (1 - \beta_1\ q_c)$$

$$\textit{Condition:}\quad q_c \le \frac{0{,}98}{\beta_1} \quad (\textit{if no then set } q_c = \frac{0{,}98}{\beta_1}) \qquad (2\text{-}39)$$

where q_c is the circulating flow in (veh/second) and β_1 = 1 or 2 seconds for multi- or single lane circulating roads respectively. For estimating approach capacity (eq. 2-34) a minimum capacity concept was introduced:

$$Q_e = \max\ (Q_g\ ,\ Q_{min})$$

$$Q_g = \frac{3600\ q_c \alpha_2 e^{-\lambda(\tau - \beta_1)}}{1 - e^{-\lambda\beta_2}} \qquad for\ q_c > 0 \qquad (2\text{-}40)$$

$$= Q_{max} \qquad for\ q_c = 0$$

where	Q_e	-	Capacity of approach (vehicles per hour)
	Q_g	-	Capacity estimate using gap acceptance theory (vehicles per hour)
	Q_{max}	-	Maximum capacity = 3600/β_1 (vehicles per hour) when q_c= 0.
	Q_{min}	-	Minimum capacity = min(Q_{ea}, 60n_m) in vehicles per hour.

β_1	-	Minimum gap in circulating stream
β_2	-	Move-up time for entering vehicles
Q_{ea}	-	Actual approach flow (vehicles per hour)
n_m	-	Minimum number of vehicles per minute which can enter circulating stream under heavy circulating flow - Default is taken as 1 vehicle per minute.

Other symbols as before.

Figure 2.14 shows the estimates - using (2-40) - of the capacity of the dominant lane of the approach as a function of circulating flow (Q_c).

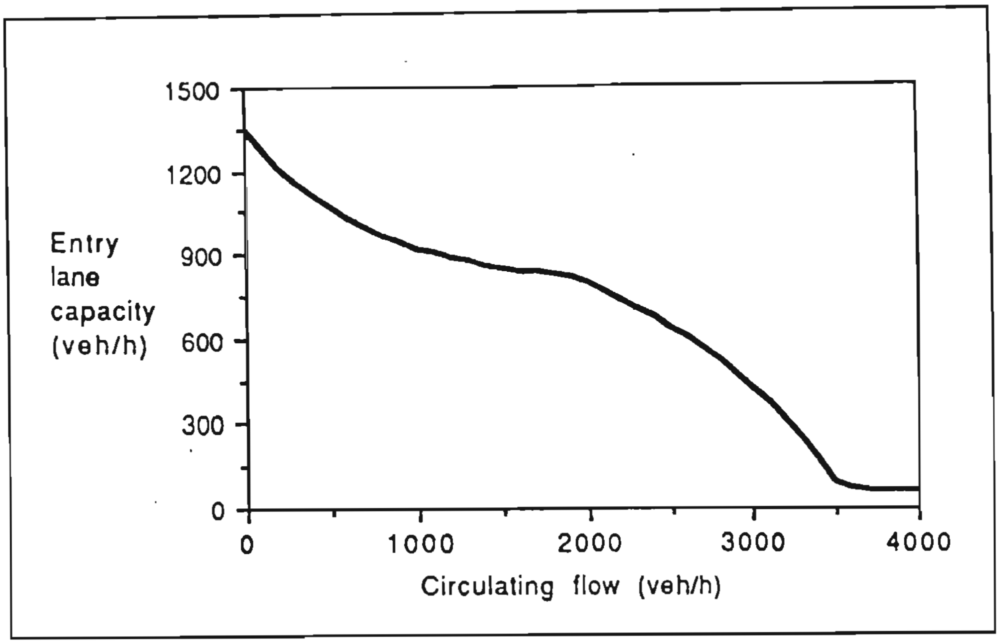


Figure 2.14: Dominant lane capacity, Q_e vs Q_c (D = 40 m, $n_e = n_c = 2$, $\bar{e}$ = 4,0 m)
Source: Akçelik (1992)

Two important sub-models included in the SIDRA (Akcelik, 1992) entry capacity model are used for estimating the effects of heavy vehicles and lane utilization. As the traditional approach of converting heavy vehicles to passenger car units by using a constant factor may not accurately reflect gap acceptance by heavy vehicles and in front of heavy vehicles, SIDRA makes use of 'incremental corrections' for heavy vehicles. NAASRA (1986) recommended that the traffic flow only be adjusted for heavy vehicles when the percentage of heavies exceeds 5% and the adjustment should be 2 pcu's for single trucks and 3 pcu's for articulated trucks. The incremental heavy vehicle factor f_{HV} as used in SIDRA is as follows:

$$f_{HV} = \frac{1,0}{0,95 + \rho_{HV}} \qquad for \quad \rho_{HV} > 0,05$$
$$= 1,0 \qquad for \quad \rho_{HV} \leq 0,05 \qquad (2\text{-}41)$$

where ρ_{HV} is the proportion of heavy vehicles and f_{HV} is used as follows:

Circulating flow: $$Q_{ca} = \frac{Q_c}{f_{HV}}$$

Entry flow: $$Q_{ea} = f_{HV}\, Q_e$$

where

Q_c	-	Circulating flow in vehicles per hour.
Q_{ca}	-	Circulating flow after adjustment for heavies in pcu/h, which should be used for calculating critical gap, entry capacity, etc.
Q_{ea}	-	Adjusted entry lane capacity in vehicles per hour
Q_e	-	Entry lane capacity based on Q_{ca} in vehicles per hour.

As indicated by Troutbeck (1989) the capacity estimating techniques for a traffic circle rely on defining dominant and sub-dominant lanes as the capacities of these are not the same (see (2-35) and 2-36)) SIDRA defines a 'lane utilisation ratio, ρ_i for the *ith* entry lane of a lane group':

$$\rho_i = \frac{x_i}{x_c} \qquad (2\text{-}42)$$

where x_i is the degree of saturation of the *ith* lane and x_c is the degree of saturation of the critical lane which will give the largest degree of saturation for the lane group. The degree of saturation is defined as the arrival flow (demand) in the lane divided by the capacity (supply) of the lane in question. SIDRA uses an iterative method to find an equal degree of saturation for all lanes.

Akçelik (1992) stresses two important factors which should be taken account of when determining circulating flows; firstly the effect of over-saturation, and secondly the effect of traffic leaving at the approach from where the opposed stream enters the circle. If any circulating stream is over-saturated, the capacity of that stream should be used in the capacity model of the entering stream rather than the arrival flow. The behaviour of traffic at a traffic circle is such that repetition in the estimation process is required to determine whether opposing streams are at capacity and if so, then a different model should be applied. This then calls for an iterative approach, which was not included in SIDRA 4.07, but has been included in later versions (Akçelik, 1995). Previous Australian research (NAASRA, 1986; Troutbeck, 1989) suggested that exiting flow has little influence on the capacity of the entering stream. Akçelik (1992) however, argued that there may be some instances where capacity models might be improved by including this parameter.

SIDRA 4.1, the latest version of the program, incorporates a number of enhancements to the estimating techniques for traffic circles which are based on recent research conducted at ARRB (Akcelik, 1994; Akcelik and Chung, 1994a; Akcelik and Chung, 1994b; Akcelik et al, 1995; Chung et al, 1992). The major changes have been the introduction of new formulae for estimating capacity and performance in terms of delay, queue length and stops, as well as the introduction of factors to reflect the effect of Origin-Destination (O-D) pattern and approach queuing on capacity. The change in capacity formulae is based on the theory of gap-acceptance modelling using traffic signal analogy, and the new capacity formula is as follows:

$$\begin{aligned} Q_e &= \max(f_{od}Q_g, Q_m) \\ Q_g &= \frac{3600}{\beta_2}\left(1 - \beta_1 q_c + 0{,}5\beta_2\alpha_2 q_c\right)e^{-\lambda(\tau-\beta_1)} \\ Q_m &= \min(q_e,\ 60n_m) \end{aligned} \qquad (2\text{-}43)$$

where f_{od} - factor to adjust the basic gap-acceptance capacity for O-D pattern and approach queuing effects

α_2 - proportion of free vehicles = $e^{-b\beta_1 q_c}$ (Akcelik & Chung, 1994)

b - calibration parameter

and all other symbols as before.

According to Akcelik et al (1995) the values for the minimum headway (β_1) in platoons are equal to 2,0 seconds and 1,2 seconds for a single- and multi-lane circulating stream respectively. The value for the calibration parameter (b) for both a single- and multi-lane circulating stream is 2,5.

To avoid over-estimation of capacity at high flows, the lower limits introduced in SIDRA 4.07 for critical gaps and move-up headways for approaching traffic have been increased in SIDRA 4.1 from 2,1 to 2,2 and 0,8 to 1,2 respectively. Another significant change in SIDRA 4.1 is in the formula for calculating critical gaps. Equation (2.37) to calculate the critical gap for the dominant and subdominant lanes was changed as follows:

$$\begin{aligned} \tau &= (3{,}6135 - 0{,}0003137Q_c - 0{,}339\bar{e} - 0{,}2775n_c)\,\beta_2 && for\ Q_c \leq 1200 \\ &= (3{,}2371 - 0{,}339\bar{e} - 0{,}2775n_c) && for\ Q_c \geq 1200 \\ & subject\ to\ 3{,}0 \geq \frac{\tau}{\beta_2} \geq 1{,}1 \quad and \quad \tau \geq \tau_{min} \end{aligned} \tag{2-44}$$

where all symbols are as before.

To avoid capacity under-estimation at low flows the maximum critical gap was reduced from 10 seconds to 8 seconds and the maximum move-up headway which only applied to dominant lanes was applied to all lanes. A further major change was in the estimating of move-up headways to allow for the effect of heavy entry flows against low circulating flows. The estimation of the dominant lane move-up headway (β_{2D}) as given in (2-35) was also further adjusted with the ratio of entry flow to circulating flow as follows:

$$\begin{aligned} \beta'_{2D} &= \beta_{2D} - \frac{q_e/q_c}{(q_e/q_c)_{max}}[\beta_{2D} - \beta'_o - \frac{q_c}{q_{cm}}(\beta_{Lm} - \beta'_o)] && for\ q_c \leq q_{cm} \\ &= \beta_2 D && for\ q_c > q_{cm} \\ & subject\ to\ \beta_{Lm} \geq \beta'_o \quad and \quad \frac{q_e}{q_c} \leq (\frac{q_e}{q_c})_{max} \end{aligned} \tag{2-45}$$

where
- β'_{2D} - Dominant lane move-up time adjusted for entry flow: circulating flow ratio
- β'_o - Minimum value of adjusted move-up time set for zero circulating flow
- β_{Lm} - Move-up time where circulating flow equal the limit value for adjustments ($q_c = q_{cm}$)
- q_e/q_c - Ratio of entry flow to circulating flow
- $(q_e/q_c)_{max}$ - Limit on the ratio of entry flow to circulating flow
- q_{cm} - Limit on circulating flow rate above which the move-up time is not adjusted

In **Sweden** the lack of empirical data on how different intersection types and designs affect capacity resulted in the National Swedish Road Administration initiating two long-term research programs. The final results (a design manual and a computer program CAPCAL) are based on gap-acceptance theory (Bergh, 1991) and incorporate most unsignalised intersection control types. For the headway distribution at traffic circles, use is made of a combination of the exponential and the Pearson type III distribution.

As mentioned previously, in **South Africa** little research has been conducted on traffic circles. A recent study by Short and Van As (1992) investigated traffic flow characteristics of mini circles in Pretoria, which is probably the first major study of its kind in South Africa. The results of the study were used for simulation purposes in the program SIMTRA and are discussed in more detail in Section 2.3.2.3. However, in Table 2.4 their observed critical gaps are compared with those of other studies. They concluded that the "*observed gap acceptance parameters are similar to observations in other countries*".

Table 2.4: Comparison of critical gaps and minimum headways

Study/Reference	Critical gap (seconds)	Minimum headway (seconds)
Grant (1969)	3,8	2,7
Armitage & McDonald (1974)	3,4	2,3
Ashworth & Field (1973)	3,3	3,3
Ashworth & Laurance (1978)	3,2	3,2
McDonald and Armitage (1978)	3,8	2,4
Horman and Turnbull (1974)	4,0	2,0
Avent and Taylor (1979)	3,5	2,1
NAASRA (1982)	4,0	2,0
NAASRA (1993)	Variable	Variable
Short and Van As (1992) - mini circles		
Left Turn	2,9	
Straight	3,4	
Right Turn	3,8	

The lack of research on circles in South Africa, specifically on the capacity of these facilities can be attributed to the few circles in operation which actually operate under saturated conditions with vehicles queuing on any one approach for more than twenty minutes. Therefore, a comparison between the methods of capacity estimates as discussed in this section and South African conditions is rather intricate. Hence, the strategy for this research was to concentrate on delays rather on capacities.

2.3.2.3 Simulation

One of the criticisms of analytical models is the assumption that some parameters such as gap acceptance are constant, while it has been shown that they could vary dramatically, not only among different drivers but also for the same driver in different situations. To incorporate this aspect of driver behaviour into a mathematical model becomes exceedingly complex. Therefore, the limitations of the mathematical/analytical solutions for the problems of unsignalised intersections in general become ever more apparent. The coming of the computer age has simplified the use of simulation techniques which enable the behaviour of an individual vehicle/driver at an intersection or through a network to be simulated. Most simulation programs for intersections are microscopic, i.e. they simulate individual vehicles. According to Chin (1983) the following considerations support the use of simulation:

(1) Closed form mathematical solutions are either too limited in scope or they can incorporate simplifying assumptions which can compromise the realism of the results.

(2) Steady state representations are limited in scope and cannot describe temporal variation in the traffic environment.

(3) The traffic flow process is highly complex and stochastic in nature reflecting the decision process of individual motorists. While the rules governing each of these decision processes are fairly understood; there is a multiplicity of interactions possible between any given vehicle and its immediate environment - other vehicles, applied control, geometric constraints and planned manoeuvres. The traffic stream is composed of such vehicles, each corresponding to stimuli provided by these interactions. Furthermore, these stimuli are constantly changing with time.

(4) The simulation methodology is particularly effective in describing such time varying, complex and stochastic processes.

May (1990) however, is more circumspect and concludes that simulation is not a 'cure-all' solution and should be viewed as one of many techniques available to the traffic engineer. Acknowledging the usefulness of simulation when analytical approaches are inappropriate and its power to vary demand over time and space, to simulate unusual arrival patterns and to handle interactive queuing processes, May (1990) warns that there may be easier, less time and data consuming methods to solve a problem. Simulation models require extensive verification, calibration and simulation while users might still apply them as "black boxes" without appreciating the model limits and the underlying assumptions.

Hoffen (1964) was one of the first people to attempt to simulate traffic behaviour at a traffic circle, employing gap acceptance techniques at small circles while the nearside priority rule still applied. Al-Salman (1976) separated conventional and mini circles, simulating headways and car following parameters for the circulating flow and gap acceptance parameters for the entering flow. His efforts seem adequate for predicting capacity, but his delay model performed poorly. Dawson (1979) describes the simulation of traffic at traffic circles in the program TRAFFICQ, also employing gap acceptance techniques.

Many of the more recent widely used computer programs such as ARCADY2 (UK - Semmens, 1985), ROAP (Switzerland - Tan, 1993), RODEL (UK - Crown, 1987) and SIDRA (Australia - Akçelik, 1992), have been and are being used to estimate traffic circle entry capacities. These are not simulation programs as such but are merely making use of analytical or empirical techniques to calculate capacities from given input values (Semmens, 1985). Most simulation programs have been designed to simulate traffic flow at priority intersections and only a few provide for simulation of traffic flow at traffic circles. SIMRO (UK - Chin, 1983), OCTAVE (France - Louah, 1988), INSECT (Australia - Tudge, 1988), MODELC (Chung, 1993) and a model developed by Tan (1993) at the University of Lucerne (TAN's model), are some of the recent microscopic programs with the capability to simulate traffic flow at traffic circles. .

SIMRO (Simulation Model for Roundabout Operation) was developed by Chin (1983) at the University of Southampton and is a detailed microscopic simulation model of traffic operations at a traffic circle based on periodic scanning or time updates. Driver behaviour was assumed to be consistent for the same driver but variable between drivers. A driver would thus be assigned a critical gap which would not change during the search for a gap in the conflicting traffic stream. The assignment of critical gaps was based on a log-normal distribution of gaps which was not affected by geometric layout or opposing flows. Chin verified the model against observations made at a number of circles in the United Kingdom and he concludes that:

i) "*the model was found to be an appropriate tool for the study and design of roundabouts*",
ii) the entry-circulating flow relationship tends not to be linear as assumed in the Kimber (1980) model (see eq. 2-11),
iii) reducing the size of the central island could increase the capacity of a circle,
iv) the practical limit for design should be 85% of the capacity, and
v) capacity and therefore also delays at a traffic circle are not only determined by the geometric layout, but also by traffic factors such as turning proportions.

INSECT (Tudge, 1988) is also a microscopic simulation model developed by R.J. Nairn and Partners Pty. Ltd for the Roads and Traffic Authority of New South Wales in Australia. Critical gaps are specified as input and modified by the program as a function of the time a driver has been in the queue. According to Chung (1993) this is an improvement to using a fixed critical gap, but it does not take into account the effects of geometric layouts on the critical gap. Chung et al (1992) showed that the capacity estimates of the simulation model are over-sensitive to the effects of origin-destination patterns. When balanced origin-destination flows are used, the INSECT capacity and delay estimates are sufficiently close to estimates from other models (Chung, 1993).

TAN's model is another microscopic simulation model developed at the Institute of Transport and Planning in Switzerland (Tan, 1993). This model is different from the previous models in that it only simulates traffic on one approach to the circle. Vehicles entered the circle from this one approach using a gap acceptance approach. Tan's research concentrated on defining the role of pedestrians conflicting with entering traffic and also to identify the effect of exiting traffic impeding the entry flow. As this model only simulates one approach to the circle, it is not effective in evaluating the total operation at a traffic circle. The conflicting headway distribution might not be an accurate representation of what really happens when traffic entering at different approaches are allowed to influence the distribution of gaps presented to entering traffic at the other approaches.

MODELC, the most recent of the simulation models, was developed by Chung (1993) as part of his studies towards a PhD at the Monash University in Australia. This microscopic model is based on time updates and incorporates both a fixed gap acceptance model and a variable gap acceptance model which are based on the geometric layout and conflicting flows as described in SR45 (Troutbeck, 1989; see eqs. 2-45 to 2-47). The program can only model single lane traffic circles. Chung (1993) concludes that:

i) a variable gap acceptance model improves the prediction of delay estimates
ii) MODELC is an accurate and useful tool for further analysis of analytical methods
iii) traffic circles perform poorly under unbalanced traffic flows, mainly because of the effects of entering traffic from the previous approach.

In South Africa, the intersection simulation program **SIMTRA** (Van As, 1985) was upgraded to include mini circles as an alternative intersection control option (Short and Van As, 1991). The validations undertaken in the study by Short and Van As (1991) showed good correlation between observed and simulated results for mini circles. However, no work has been done locally to simulate any other traffic circles apart from mini circles. In previous research done by Kiln (1988), it has been shown that SIMTRA is a good simulator of other forms of intersection control. SIMTRA thus provides an effective means of evaluating different intersection control strategies including mini circles, but not larger circles.

2.4 Research on Design and Operating Parameters for Traffic Circles: Delay and Queue lengths

The delay incurred at a traffic control facility is a direct measure of the performance of the facility while the calculation of queue lengths is often required to estimate the influence of a facility on the neighbouring intersections or other facilities. A facility should be designed to accommodate queues for most of the operating time. These two parameters are discussed together in this section because queue lengths and delays are closely related. Queue length observations (area under the queue length graph) are often used to estimate traffic delay at traffic control facilities.

Delay is defined as the difference between the *theoretical arrival* and *departure* times (Van As and Joubert, 1993) and can be classified into two types; geometric delay and traffic delay. Geometric delay is caused by the presence of the traffic circle and is the additional travel time needed to negotiate the intersection - with no other traffic present - compared with the time required to travel along a straight road (Schermers, 1987).

Queuing delay is the time minor stream vehicles are required to wait (in a queue or at stop/yield line) before entering the circulating stream and is caused by the presence of other vehicles. *Total delay* is *geometric* plus *queuing* delay (D_{ta} and D_{tb} in Figure 2.15). It is measured from the time that a vehicle's speed is reduced to the time when the vehicle has accelerated back to its cruising speed and thus includes time spent decelerating and accelerating, time spent in the queue, plus time spent waiting to be serviced.

Stopped delay is also sometimes used as a measure of effectiveness at an intersection (Van As & Joubert, 1993; Chung, 1993), but conflicting definitions seem to exist. Chung (1993) defines queuing delay (D_{qb} in Figure 2.15) as the time a vehicle spends waiting in the queue and at the head of the queue, i.e. all traffic delay excluding the major acceleration and deceleration time. This definition is similar to that for stopped delay in the HCM (1985) and the Manual of Traffic Engineering Studies (ITE, 1976). In these documents the point sampling method (see Section 5.3.1) is recommended for observation of stopped delay. This method includes all vehicles in a queue, whether stationary or moving-up (D_{qb} in Figure 2.15). To convert stopped delay to traffic delay the HCM (1985) recommends that it be increased by 30%. However, in SIDRA (Akçelik, 1995) the definition of stopped delay excludes any queue move-up times and includes only the time the vehicle is stationary (D_{sa} and D_{sb} in Figure 2.15).

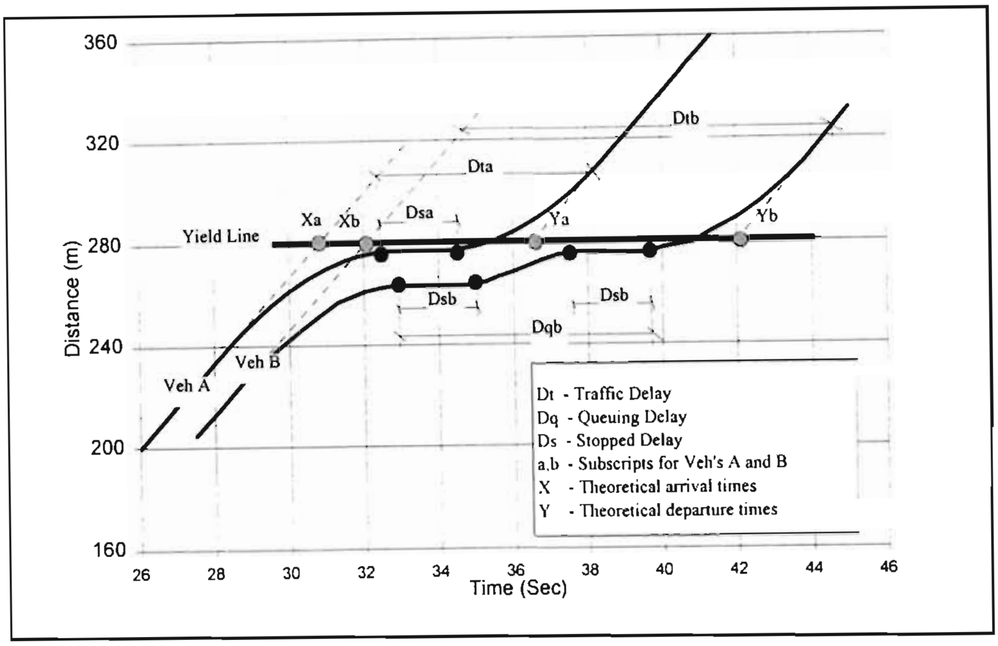


Figure 2.15: Time-space diagram for definition of delay

In SIDRA the following four methods of calculating delay are allowed for:

(i) *The overall delay with geometric delays: the overall delay includes deceleration and acceleration delays for the major stop experienced by queued vehicles, as well as the geometric delays experienced by all vehicles in negotiating the intersection,*

(ii) *The overall delay without geometric delays: this is equivalent to the delay predicted by the analytical models listed in Appendix A of the SIDRA Output Guide (includes the queueing delay and the major stop-start delay),*

(iii) *The stopped delay (idling time): this is the delay excluding all deceleration and acceleration delays (i.e. not including any geometric, major stop-start and queue move-up delays), and can be estimated by:*

(a) *the SIDRA method which calculates the stopped delay by subtracting the deceleration and acceleration delays associated with major stop-start and queue move-ups from the analytical model delay, or*

(b) *the HCM method which uses the simple formula: stopped delay = overall (analytical model) delay / 1.3.*

The definition under (iii) for stopped delay is not consistent with that of the HCM as quoted under point (iii)-(b). For this research, stopped delay is defined as equal to traffic delay less the time for the major acceleration and deceleration.

In simulation models vehicles are frequently assumed to queue in a vertical queue and the theoretical arrival time of a vehicle at the stop line is assumed to be at the intersection of the extension of the vehicle trajectory before starting to decelerate, and the yield line (Point X in Figure 2.15). Similarly the theoretical departure time is shown by the point Y in Figure 2.15. It can be seen from Figure 2.15 that the error when estimating delay using the theoretical arrival and departure times, even for vehicles stopping in a queue, is negligible. The time difference between successive theoretical arrival times is similar to the arrival headway (h_w) between vehicles arriving on the approach (Van As & Joubert, 1993).

The different elements of geometric and traffic delay apparently cannot be separated completely as there are some commonalities. However, for reasons of simplicity they are often separated specifically to simplify mathematical delay models based on queuing theory. A number of the early research endeavours on capacity estimation at traffic circles did not attempt to estimate traffic delay on the approaches (Ashworth & Field, 1973; Ashworth & Laurance, 1978). Instead they suggested that the traffic flow on an approach should not be greater than 80% of the predicted capacity to ensure that the delays would be acceptable. This could be a simple way of overcoming the problem, but according to Kimber and Hollis (1979) there are three basic reasons why traffic delay estimates are essential:

- Delay savings constitute the main benefit in an economic analysis to justify expenditure
- In assigning traffic to a network the route choice is largely dependent on the traffic delay along the route.
- Queue length estimates are needed for detailed intersection design, i.e. turning lane and storage lane lengths.

Subsequently the complex mathematical analysis of delays at priority controlled intersections has attracted much attention. Traditionally queuing analysis has been approached in two ways of which the first, based on classical queuing theory and stochastic steady state systems, probably had the widest application. The second approach is based on deterministic queuing theory assuming constant arrival and departure patterns.

Geometric delay depends on the geometric layout - which is fixed - and the approach and departure speeds which are measurable and vary little with time. When approximate results are required and especially when traffic circles are compared with other forms of give way control then geometric delay is often ignored. However, when comparisons are made with traffic signals, the total delay (geometric plus traffic delay) must be considered. Therefore, this section mostly deals with traffic delay, while geometric delay is only discussed briefly.

2.4.1 Traffic delay

Most stochastic models, attempting to establish exact solutions, came up with complicated equations which seldom showed satisfying correlation with actual observations, although some simplifying assumptions were made (Van As & Joubert, 1993). Although stochastic queuing analysis takes into account the probabilistic nature of traffic arrival and departures, it is only useful when the demand is less than the capacity, i.e. $\rho < 1$ where $\rho = \frac{Q_{ea}}{Q_e}$ with Q_{ea} the arrival flow and Q_e the entry capacity. Tanner's (1962) well-known model was one of the first which tried to provide a simple equation for delay estimation. Though based on the simplifying assumptions of random arrivals and constant service times, it still has the following complicated form:

$$d_t = \frac{\frac{1}{2}\frac{E(y^2)}{Y} + q_e\, Y\, e^{-\beta_2 q_c}\, \frac{(e^{\beta_2 q_c} - \beta_2 q_c - 1)}{q_c}}{1 - q_e\, Y\,(1 - e^{-\beta_2 q_c})} \qquad (2\text{-}46)$$

with

$$Y = \frac{e^{q_c}(\tau - \beta_1)}{q_c(1 - \beta_1 q_c)}$$

$$E(y^2) = \frac{2Y}{q_c}\left(e^{q_c(\tau - \beta_1)} - \tau\, q_c(1 - \beta_1 q_c) - 1 + \beta_1 q_c - \beta_1^2\, q_c^2 - \frac{\frac{1}{2}\beta_1^2 q_c^2}{1 - \beta_1 q_c}\right) \qquad (2\text{-}47)$$

where:
- d_t - average delay on approach (seconds/vehicle)
- q_{ae} - arrival rate on entering approach (vehicles/second)
- q_c - arrival rate on major road/circulating flow (vehicles/second)
- τ, β_1 and β_2 as before

Chin (1983) argues that, as Tanner's model considers single turning movements, its application to and constant service times limits its application. Fisk and Tan (1989) showed that simplifying Tanner's equation by assuming minimum gaps equal to zero in the circulating traffic ($\beta_1 = 0$) results in the underestimation of traffic delay. Again assuming random arrivals, Tanner showed that the average delay for isolated minor road vehicles also termed *Adams' delay* (for the first vehicle only, i.e. no queues) is as follows:

$$d_{min} = \frac{e^{q_c(\tau - \beta_1)}}{q_c(1 - \beta_1 q_c)} - \tau - \frac{1}{q_c} + \frac{q_c \tau^2 (2\tau q_c - 1)}{2(1 - \tau q_c)^2} \qquad (2\text{-}48)$$

If the headway distribution of the major stream is assumed to be distributed according to the bunched negative exponential distribution, then the delay to an isolated vehicle can be calculated as follows (Troutbeck, 1984):

$$d_{min} = \frac{e^{\lambda(\tau - \beta_1)}}{\alpha_2 \, q_1} - \tau - \frac{1}{\lambda} + \frac{\lambda \beta_1^2 + 2\alpha_2 \beta_1 - 2\beta_1}{2(\lambda \beta_1 + \alpha_2)} \qquad (2\text{-}49)$$

If the proportion of free vehicles (α_2) is set equal to one and the minimum gap in the circulating traffic (β_2) set to zero, then (2-49) reduces to Adams' delay equation which is also termed by Troutbeck (1984) the 'pseudo Tanner's model':

$$d_{min} = \frac{e^{q_c(\tau - \beta_1)}}{q_c \, (1 - \beta_1 q_c)} - \tau - \frac{1}{q_c} - \frac{\beta_2^2 \, q_c}{2} \qquad (2\text{-}50)$$

Equations (2-48) and (2-50) are similar except for the last term, which is attributable to the distribution of gaps in the major stream - the one being random arrivals and the other according to the bunched exponential distribution. Figure 2.16 shows Adams' delay (delay to an isolated minor road vehicle - d_{min}) as estimated by four different circulating stream headway models. It is obvious that there are appreciable differences in the estimates which deteriorate with an increase in the major stream flow.

The total delay which includes the queuing delay will obviously increase as the flows on the minor road increase and move closer to capacity. Under these conditions Adams' delay (eq. 2-48) will underestimate the mean delay. NAASRA (1982) proposed the use of a negative exponential model for delay estimates, but Troutbeck (1984) showed (See Figure 2.17) that it could underestimate delays significantly. Tanner's equation gave better results and might be the preferred starting point.

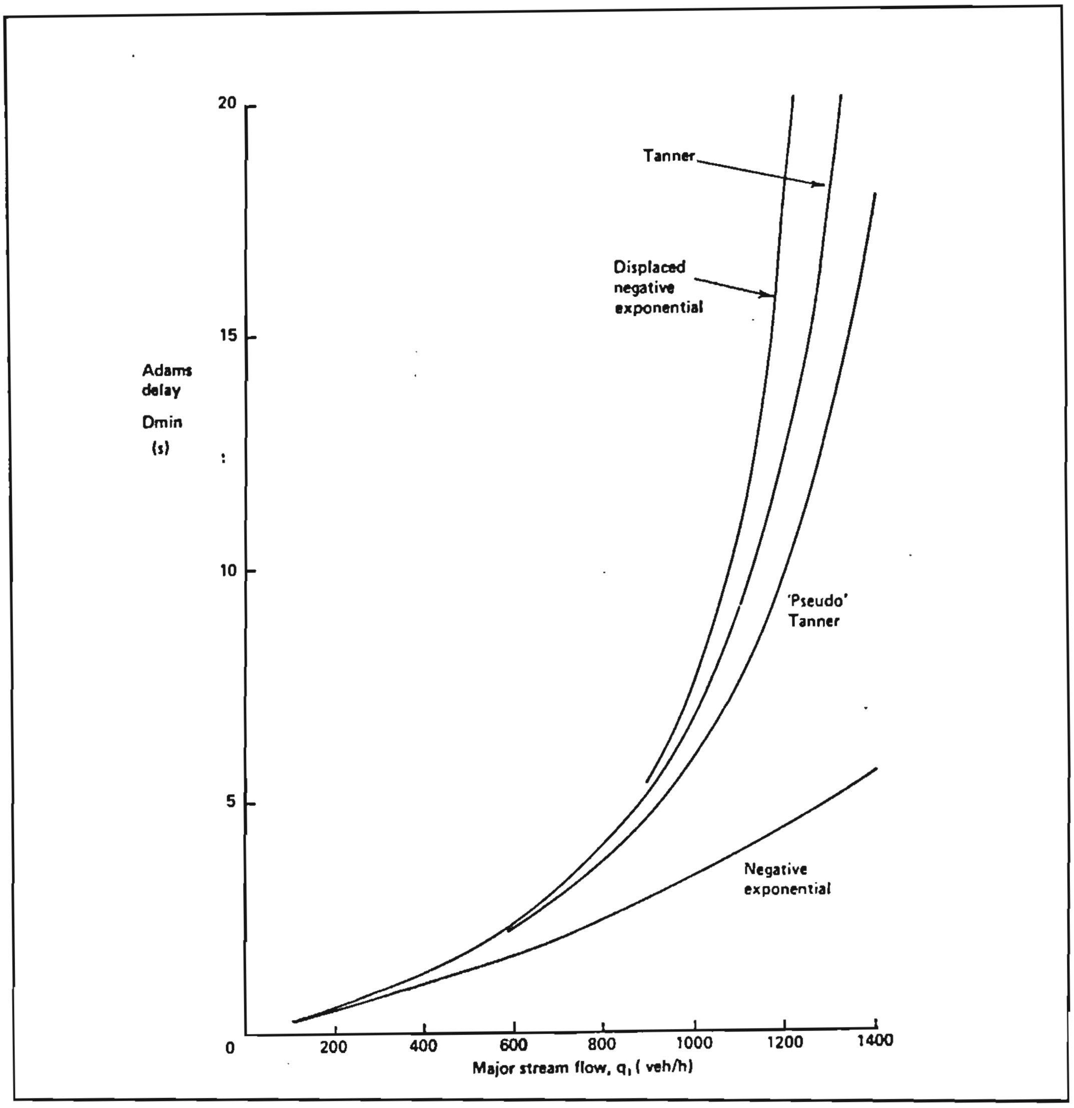


Figure 2.16: Adams' delay for various circulating stream headway models.

Source: Troutbeck (1984)

Troutbeck (1989) recommends that the following steady state model be used for estimating average delay (d_t):

$$d_t = d_{min} \left[1 + \frac{ex}{1 - x}\right] \tag{2-51}$$

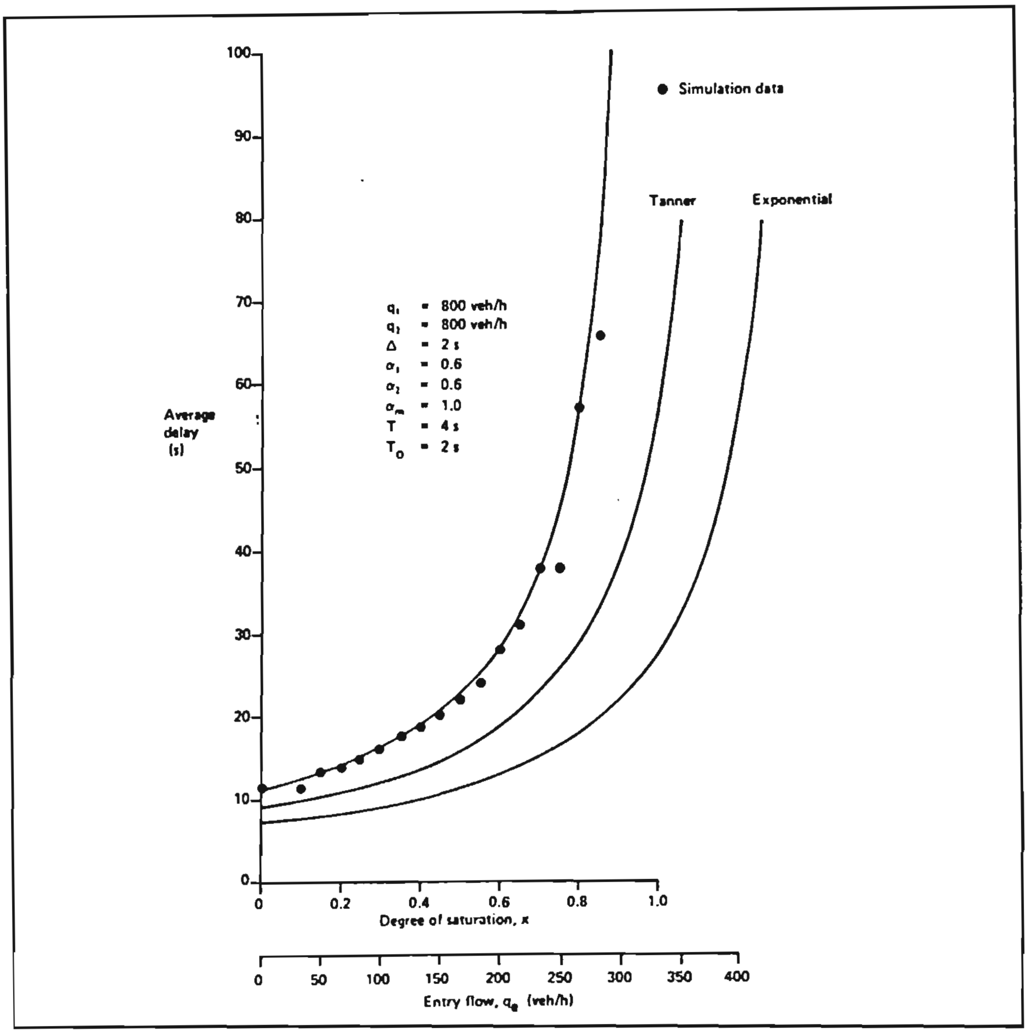


Figure 2.17: Average delay estimate for the specified conditions.

Source: Troutbeck (1984)

where d_{min} is calculated using (2-49), x is the degree of saturation and e a form factor which can be set to 1,0 if no other value is available. As an improvement to (2-50) and to allow for variation over time the following model is proposed by AUSTROADS (1993) and is also included in the program SIDRA 4.07 (Akçelik, 1992):

$$d_t = d_{min} + 900T \left[(x - 1) + \sqrt{(x - 1)^2 + \frac{8kx}{Q_e T}}\right]$$

$$k = \frac{d_{min} Q_e}{3600} \qquad (2\text{-}52)$$

where T is the duration in hours, x the degree of saturation and Q_e the entry lane capacity in vehicles per hour. In both (2-51) and (2-52) the second term accounts for the queuing delay as a result of the presence of a queue. The duration T becomes important at high degrees of saturation, but relatively unimportant at low flows, i.e. the delays are insensitive to the flow period under low flow conditions.

unimportant at low flows, i.e. the delays are insensitive to the flow period under low flow conditions. SIDRA 4.07 also calculates average queue lengths by multiplying the average delay (d_t), calculated using (2-52), with the arrival rate of traffic on the approach Q_{ea}.

The second queuing theory often used to estimate delay is the deterministic theory which allows for unsteady flows over time, i.e. demand and capacity vary over time, but it ignores the statistical nature of the arrivals, thus no queue or delay is encountered until demand exceeds capacity. May (1990) describes several deterministic queuing models varying from constant arrival and service rates to varying arrival and service rates (see Figure 2.18). The hashed areas in Figure 2.18 show the excess number of vehicles when demand (arrivals - q_a) exceeds supply (departures - q_d). The total delay (D_T) for all vehicles can be calculated by subtracting the two areas under the curves from each other for the time period t_1 to t_2 during which arrivals exceed departures.

$$D_T = \int_{t_1}^{t_2} [q_a(t) - q_d(t)]\, dt \qquad (2\text{-}53)$$

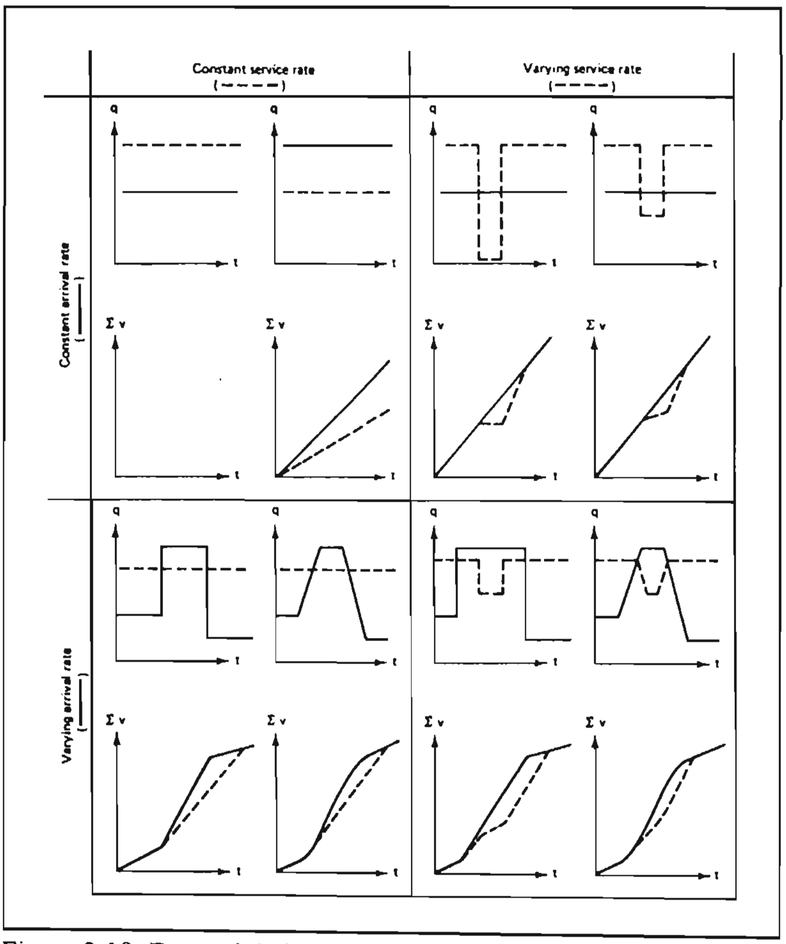


Figure 2.18: Deterministic queuing patterns

Source: May (1990)

Theoretical models attempting to solve the problem of estimating queue lengths under over-saturated and deterministic queuing theories to allow for under- and over-saturated conditions with stochastic arrivals, Kimber & Hollis (1979) proposed a coordinate transformation technique. Figure 2.19 shows the two curves obtained by the two queue length models of which the first one is in deterministic terms:

$$L = (\rho - 1)q_e t + L_0 \qquad (2\text{-}54)$$

and the second one for steady state conditions:

$$L = \rho + C\rho^2(1 - \rho)$$

$$L = \frac{\rho}{(1 - \rho)} \qquad for\ C = 1 \qquad (2\text{-}55)$$

where L_0 - initial queue length

t - any time interval

ρ - the traffic intensity (q_a/q_{max})

q_a - arrival (demand) flow in vehicles per time unit

q_{max} - capacity of approach in vehicles per time unit

C - constant to describe arrival and service patterns. For regular arrivals and service C=0 and for random arrivals and service C=1.

Obviously the steady state only applies for $\rho < 1$ while the deterministic theory applies for $\rho > 1$. For simplicity and as shown in Figure 2.19 the case where L_0=0 at time t=0 with C=1 is considered. When applying the coordinate transformation technique then instead of the steady state queue estimates becoming infinite as the approach flow reaches capacity (ρ = 1) it approaches the deterministic queue estimates as shown in Figure 2.19. Kimber & Hollis (1979) suggest that for a given queue length L the steady state intensity ρ_s should be transformed to the new value ρ_n such that X = Y.

Using this method, the average queue length including the vehicle being serviced is given by:

$$L = ½(\sqrt{A^2 + B} - A) \tag{2-56}$$

where

$$A = \frac{(1 - \rho)(q_{max}t)^2 + (1 - L_0)q_{max}t - 2(1 - C)(L_0 + \rho q_{max}t)}{q_{max}t + (1 - C)}$$

$$B = \frac{4(L_0 + \rho q_{max}t)\,[q_{max}t - (1 - C)(L_0 + \rho q_{max}t)]}{q_{max}t + (1 - C)}$$

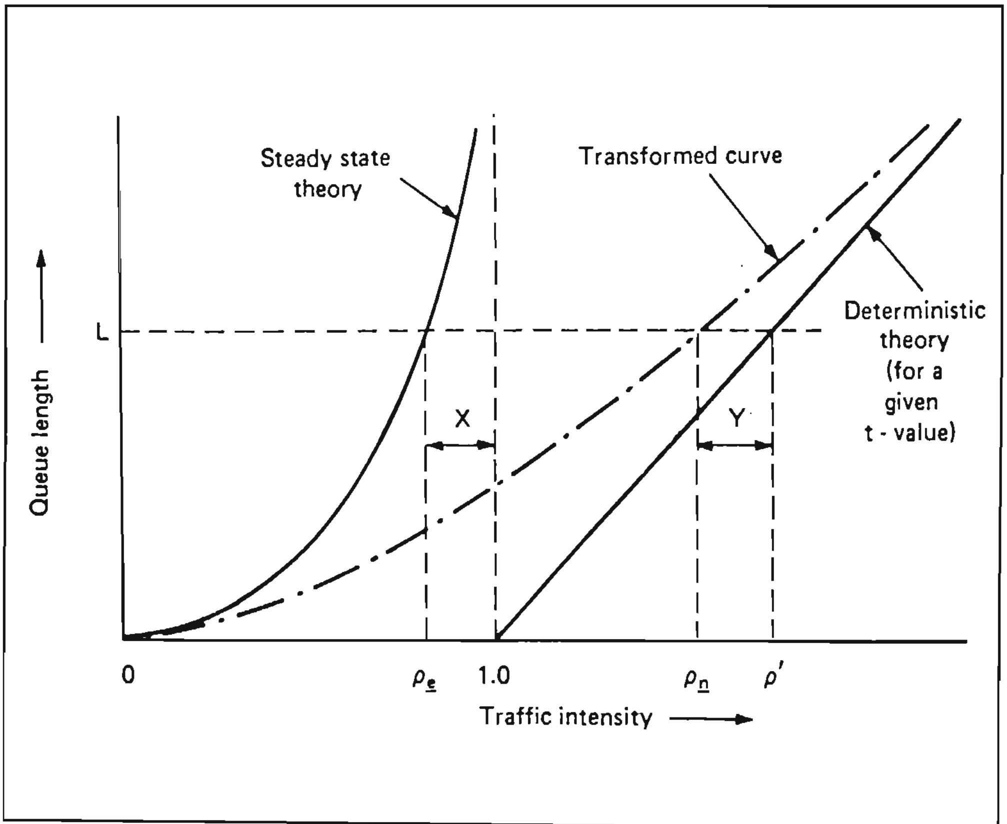


Figure 2.19: Principle of the Co-ordinate Transformation method to estimate delay
Source: Kimber & Hollis (1979)

The average delay per minor road vehicle (d_t), including queueing and service time, for a specific time period (t) is given by:

$$d_t = \tfrac{1}{2} \left(\sqrt{J^2 + K} - J\right) \tag{2-57}$$

where

$$J = \frac{t}{2}(1 - \rho) + \frac{L_0 - C + 2}{q_e}$$

$$K = \frac{4}{q_e}\left[\frac{t}{2}(1 - \rho) + \tfrac{1}{2}\rho t C - \frac{L_0 + 1}{q_e}(1 - C)\right]$$

With C = 1 for random arrivals (2-55), (2-56) and (2-57) are simplified. Due to the dependance of the entry flow on the entry flow from the previous entry to the circle, the application of any delay formula to traffic circles is not straightforward and a more cumbersome iterative procedure must be used. The software program ARCADY (Semmens, 1985) estimates queues and delays as described above by Kimber and Hollis (1979).

Based on steady state arrivals and stochastic flow CETUR (1988) and Harders (modified by CETE, 1989) developed different equations for the estimation of average queue lengths (L) and delay per vehicle (d_t) for traffic circles in France. CETUR's equations are as follows:

$$d_t = \frac{2000 + 2Q_c}{Q_e - Q_{max}}$$

$$L = d_t \frac{Q_{max}}{3600} \tag{2-58}$$

and Harder's proposed equations are the following:

$$d_t = 3600 \frac{1 - e^{\left(\frac{\tau Q_c + B_2 Q_e}{3600}\right)}}{Q_{max} - Q_e}$$

$$L = d_t \frac{Q_e}{3600} \tag{2-59}$$

where Q_e - entering flow (vehicles per hour)

Q_{max} - capacity (vehicles per hour)

Q_c - circulating flow (vehicles per hour)

Both the above formulae for queue length (L) reduce to the following equation which is similar to the steady state equation for estimating average queue length (May, 1990) except for the constant k:

$$L = k \frac{\rho}{1 - \rho} \tag{2-60}$$

where

$$k = 1 \qquad \textit{Kimber}$$

$$k = \frac{2000 + Q_i}{3600} \qquad \textit{CETUR} \tag{2-61}$$

$$k = 1 - e^{\frac{-(\tau Q_c + B_2 Q_e)}{3600}} \qquad \textit{Harders}$$

Tan (1993) compared these formulae and concluded that the CETUR (2-58), and Harders (2-59) formulae are only to be used for general studies where the degree of saturation is less than one ($\rho < 1$), while Kimber's formula (2-57) can be used for more detailed studies.

Surprisingly, little has been published on the verification of all the theoretical delay prediction models with actual observations. Tan (1993) however, compared the performance of the delay models proposed by CETUR (2-58), Harders (2-59) and Kimber (2-57) with observations at a traffic circle in the small town of Moutier in Switzerland. He concludes that under low flows ($\rho < 0{,}8$) the theoretical models give 'good predictions', but under high flows there are significant differences. The theoretical models tend to overestimate delays especially at over-saturated conditions although the theoretical capacity predictions are 'relatively accurate' and the greater the variance in the arrival flow the less accurate the delay models are, even at average flows.

Tan (1993) continued to investigate Kimber's formulae ((2-56) and (2-57)) suggesting that to overcome the over-estimation of average queue lengths, especially under over-saturated conditions, the coordinate transformation method should be adjusted such that the general formula's approach to the deterministic line be accelerated. From Figure 2.19 it can be concluded that $Y < X$ or $X = \xi Y$ where $\xi < 1$. ξ can then be used to regulate the approach speed of the general formula to the deterministic curve. With the help of simulation studies Tan proposed the following improved formula for estimating average queue lengths (L):

$$L = ½(\sqrt{A^2 + B} - A) \qquad (2\text{-}62)$$

where

$$A = (1 - \rho)\lambda q_{max}t + (1 - \lambda L_0)$$
$$B = 4\lambda[L_0 + (\frac{1}{\xi} + \rho - 1)q_{max}t]$$
$$\lambda = \frac{1}{1 + 0{,}2\ [\rho;1{,}5]_{min}}$$
$$\xi = 1 + \rho^3$$

- *other symbols as before*

and the following for estimating average delay per vehicle (d_t):

$$d_t = ½(\sqrt{C^2 + E} - C \qquad (2\text{-}63)$$

where

$$C = \lambda[\frac{q_{max}t(1 - \rho)}{2q_e} - \frac{L_0}{q_e}]$$
$$E = \frac{2\lambda t}{\xi q_e}$$
$$\lambda = \frac{1}{1 + 0{,}2[\rho;\ 1{,}5]_{min}}$$
$$\xi = 1 + \rho^3$$

- *other symbols as before*

The average delay could also be calculated by simply dividing the average queue length (L in (2-62)) with the arrival flow on the approach (q_e), and in fact Tan suggests that it might be even more accurate than (2-63)

In South Africa, little research has been done to verify or substitute any of the above traffic delay models - which can be expected, given the few circles operating at high degrees of saturation. Schermers (1987) conducted surveys at three mini circles and five conventional traffic circles in several cities and towns around South Africa. He fitted the following regression curve through the observed data points:

$$Delay = 0{,}0217Q^2 - 6{,}042Q \qquad (2\text{-}64)$$

and the delay in vehicle-seconds per 15 minutes and Q the total flow through the intersection in vehicles per 15 minutes. Figure 2.20 shows a comparison of his traffic circle delay observations with observations at signalized and 4-way stop controlled intersections with similar flows. From

Figure 2.20 traffic circles are evidently the most effective forms of control in terms of traffic delay for intersection volumes up to 3800 vehicles per hour. Priority control has not been included in this figure because the vehicles on the main road through this intersection do not incur any delay and normally represent the higher proportion of the total traffic volume. A more realistic comparison would be to compare delay per stopped vehicle at a priority intersection with that at a traffic circle. From such a comparison, Schermers (1987) shows - as expected - that only at low volumes does priority control result in less stopped delay than at traffic circles.

To validate the changes to the simulation program SIMTRA to allow for simulation of mini traffic circles, Short & Van As (1992) studied a number of mini circles in Pretoria. They concluded that the program was successfully calibrated with a high correlation between observed and simulated delay results.

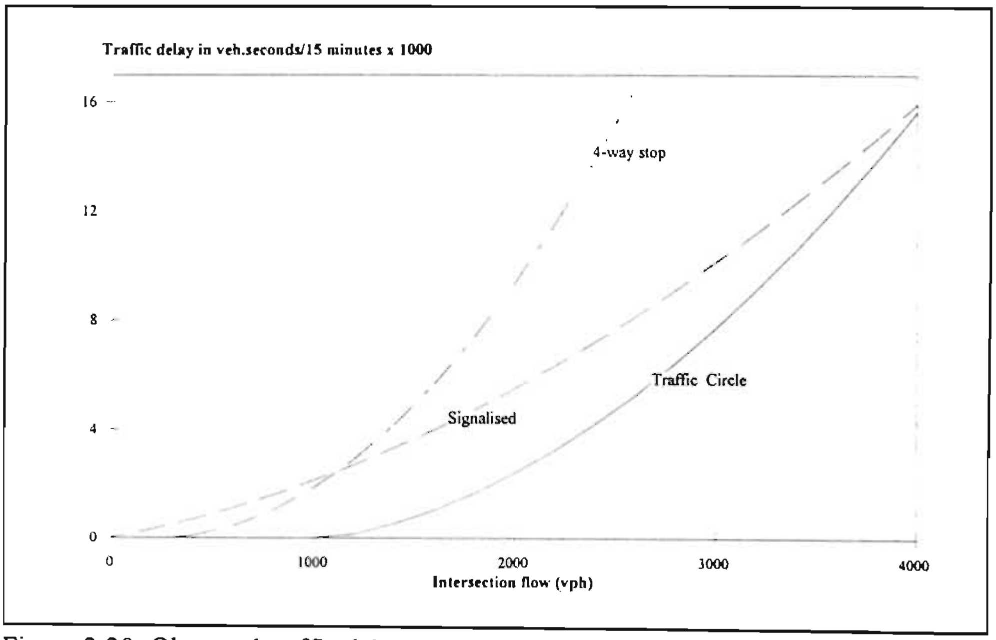


Figure 2.20: Observed traffic delays at intersections.

Source: Schermers (1987)

2.4.2 Geometric delay

The geometric delays for vehicles depend on the extra distance which has to be travelled and the speed changes necessary while moving through the intersection compared with the no-intersection alternative. A paramount study in this regard was that by McDonald, Hounsell and Kimber (1984) which describes the geometric delay at a range of unsignalised intersections by means of a synthetic model and a regression model. They developed relationships for parameters such as acceleration, deceleration and minimum speeds which are based on measurable geometric and dynamic variables.

The regression analysis resulted in the following regression equation, among others, for at-grade traffic circles:

$$d_g = 0{,}11ED + 0{,}72(Y - V_{avg}) + 3{,}06 \tag{2-65}$$

where Y is the average of the entry (V_a) and departure (V_d) speeds in metres per second, V_{avg} the average speed within the circle, and ED the extra distance (metres) involved in negotiating the circle. In their synthetic model, a minimum speed is calculated, based on entry and exit curb radii, and the delays for the different turning movements were shown to be different. They conclude that both the regression and synthetic models provided satisfactory results recommending that the delay be adjusted if high flows are experienced.

Austroads (1993) suggests, based on research by George (1982), that the geometric delay depends on whether vehicles have to stop or not with average geometric delay given by:

$$d_g = \rho_s d_s + (1 - \rho_s) d_u \tag{2-66}$$

where ρ_s is the proportion of vehicles having to stop and d_s and d_u the geometric delay to vehicles having to stop and not to stop respectively. The proportion of vehicles on an approach that have to stop obviously depend on the approach and circulating flows. Austroads(1991) contains two graphs (reproduced here as Figures 2.21 and 2.22) from which the proportion of vehicles that must stop can be estimated.

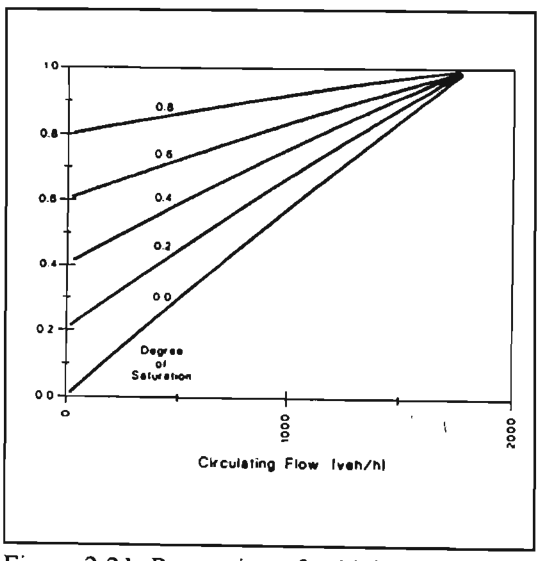


Figure 2.21: Proportion of vehicles stopped on a single lane circle. Source: Austroads (1993)

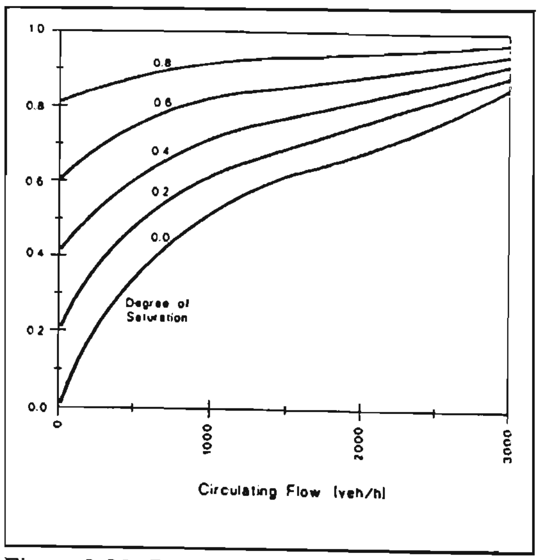


Figure 2.22: Proportion of vehicles stopped on a multi-lane circle. Source: Austroads (1993)

For estimating the geometric delay for stopped and unstopped vehicles Austroads (1993) suggests the values as listed in Table 2.5.

Table 2.5: Geometric delay (sec/veh) for stopped vehicles and (non-stopping vehicles) Akçelik (1993)

Approach speed V_a (km/h)	Distance around circle (m)	Negotiation speed through traffic circle V_n (km/h)							
		15	20	25	30	35	40	45	50
40	20	10 97)	8 (4)	7 (2)	7 (1)	7 (0)			
40	60	19 (17)	15 (11)	9 (7)	9 (4)	7 (0)			
40	100		22 (19)	17 (13)	13 (8)	10 (4)			
40	140				18 (13)	14 (8)			
40	180					18 (12)			
60	20	13 (11)	11 (8)	10 (5)	10 (4)	10 (3)	10 (2)	10 (1)	10 (1)
60	60	23 (20)	18 (15)	15 (11)	13 (8)	10 (4)	10 (2)	10 (1)	10 (1)
60	100		26 (22)	21 (17)	18 (13)	15 (9)	12 (5)	10 (1)	10 (1)
60	140				22 (17)	19 (13)	15 (8)	12 (4)	10 (1)
60	180					23 (17)	19 (12)	15 (7)	10 (2)
80	20	17 (14)	15 (11)	13 (9)	13 (7)	13 (6)	13 (5)	13 (4)	13 (3)
80	60	26 (24)	22 (19)	19 (15)	17 (11)	14 (8)	13 (5)	13 (4)	13 (3)
80	100		29 (26)	25 (20)	21 (16)	19 (13)	16 (9)	13 (5)	13 (3)
80	140				26 (21)	23 (17)	19 (13)	16 (9)	13 (4)
80	180					27 (21)	23 (16)	19 (12)	16 (7)
100	20	20 (18)	18 (15)	17 (12)	17 (10)	17 (9)	17 (8)	17 (7)	17 (6)
100	60	30 (27)	25 (22)	22 (18)	20 (15)	18 (12)	17 (9)	17 (7)	17 (6)
100	100		33 (29)	28 (24)	25 (20)	22 (16)	20 13)	17 (10)	17 (6)
100	140				30 (25)	26 (20)	23 (17)	20 (13)	17 (12)
100	180					30 (25)	27 (20)	24 (16)	20

Short and Van As (1992) and Schermers (1987) measured geometric delay at South African circles. All the mini circles of Short & Van As were in Pretoria while those of Schermers were measured at larger conventional circles. These results are summarized in Table 2.6.

Table 2.6 Summary of geometric delay values. Short & Van AS (1992)

	Turning Movement		
	Left	Straight	Right
Mini Circle			
Short and Van As (1992)			
Painted island	7.2	5.8	8.3
Raised island	8.9	9.2	9.2
Schermers (1987)	3.8	5.4	10.2
Conventional Circle			
Schermers (1987)	9.0	13.6	15.4

Short & Van As (1992) conclude that the geometric delays measured at mini circles in Pretoria 'are similar to other studies, both locally and abroad.' They attribute the higher geometric delay values at raised island circles to the 'restrictions imposed by the physical obstruction'.

2.5 Research on Design and Operating Parameters for Traffic Circles: Stops

The number of vehicular stops is often used in an economic evaluation of intersection control strategies, so a method to estimate stops on an approach is essential. Although geometric delay includes the time lost as a result of decelerating and accelerating, compared with the no-intersection alternative, the value of this time can only be calculated in terms of fuel consumed while idling and a monetary time value for all occupants. The cost of a stop is much more as it includes the total amount of fuel used to accelerate back to the original speed. The Australian model (Austroads, 1993) used to calculate geometric delay includes an estimate of the number of stops as the geometric delay for vehicles stopping are greater than the geometric delay for vehicles not stopping (See Table 2.5).

According to Austroads (1993) the proportion of vehicles having to stop can be estimated by using the graphs in Figures 2.21 and 2.22. SIDRA (Akçelik, 1992) uses a model based on gap acceptance theory:

$$p_s = p_{sm} + (1 - p_{sm})\, x^n \qquad p_s \leq 1{,}0 \tag{2-67}$$

where p_{sm} is the minimum proportion stopped as proposed by Troutbeck (1984):

$$p_{sm} = 1 - \alpha_2\, e^{-\lambda(\tau - \beta_1)} \tag{2-68}$$

x is the entry lane degree of saturation and n is a calibration parameter calculated as follows with

$$n = 5{,}0 - 7{,}0\beta_2 q_c + 2{,}5(\beta_2 q_c)^2 \tag{2-69}$$

and all other symbols as before. Van As and Joubert (1993) define a vehicle as having stopped when it has incurred delay. This will obviously overestimate stops as not all vehicles being delayed, actually stop. To allow for this discrepancy a stop-delay factor (S_t) was introduced to adjust the estimated number of stops. Taking only fuel consumption as the cost of a stop, Joubert (1988) developed the following linear relationship to determine partial stops for different initial speeds:

$$S_t = \frac{d_t}{\delta} \qquad S_t \leq 1 \tag{2-70}$$

where δ is the minimum delay for a full stop (about 9 seconds for a speed of 60 km/h). Under heavy flow conditions a vehicle may however stop more than once in a queuing situation and this model will underestimate the number of stops. The software program SIMTRA uses this model to estimate the number of stops.

Schermers (1987) compared the observed number of stops per vehicle at four intersection control types, i.e. priority control, 4-way stop control, traffic circle control and signal control. Figure 2.23 shows the result of a regression analysis on his observations at the different types of control, with the equation for circles as follows:

$$S_{nt} = 0{,}072Q + 0{,}00056Q^2 \tag{2-71}$$

where S_{nt} is the number of stops per 15 minutes and Q the total intersection flow in vehicles per 15 minutes.

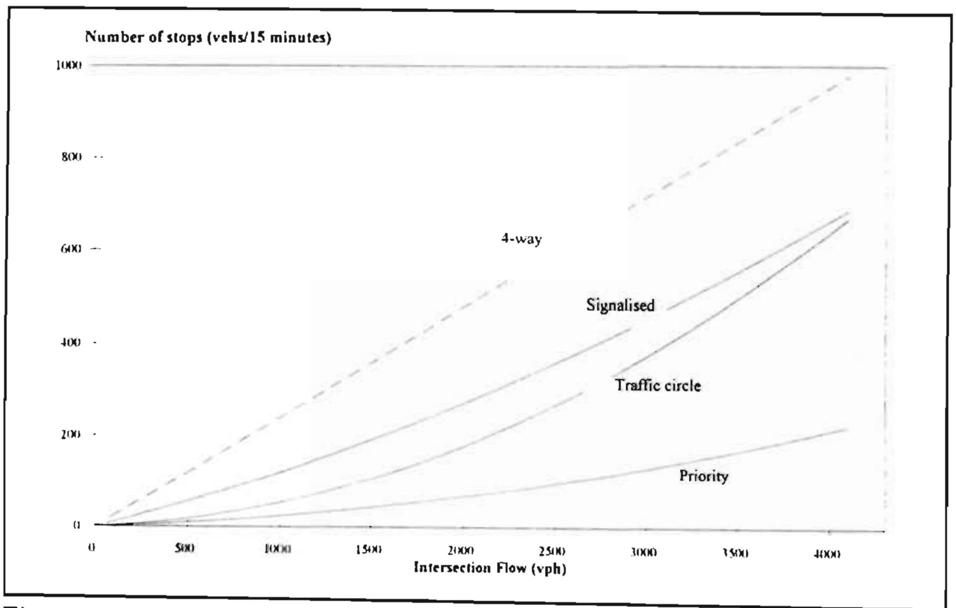


Figure 2.23: Observed number of stops

Source: Schermers (1987)

It can be seen from Figure 2.23 that traffic circles result in a much lower number of stops when compared with 4-way stop control and signalised control. Priority control, however, shows the lowest number of stops in the surveyed volume ranges. This could be ascribed to the difference in major/minor volume split since an increase in the minor street volume will result in an immediate increase in the number of stops. It must be kept in mind that the major stream traffic, in the case of priority controlled intersections can move freely through the intersection with no additional stops or delay to the major stream traffic. The effectiveness of a priority controlled intersection depends mostly on the distribution of traffic between the minor and major approaches (Jordaan and Joubert, 1991).

2.6 Summary

In this chapter the results of an extensive literature survey on traffic circles are recorded and summarized. This forms the basis of all the later work discussed in this thesis. It covers the historical development of traffic circles and also describes the endeavours through the years to quantify the effect of traffic operations at these control facilities in terms of entry capacities, delays, queues and stops.

Before 1966 traffic negotiated a traffic circle giving way to entering vehicles weaving into the circulating stream (nearside priority). Since then the priority rule changed to offside priority where entering traffic has to yield to circulating traffic and is in effect accepting gaps in the circulating traffic stream. Three major approaches in the analysis of traffic circles have emerged: empirical methods, analytical methods and simulation models. The empirical methods which are used locally are mostly based on models developed in the United Kingdom which are incorporated in the software program ARCADY. On the other hand the important analytical models which are based on gap acceptance theories have been developed in AUSTRALIA and have been incorporated in the software program SIDRA. Other models developed in France, Germany and Switzerland are also discussed. Because of the difference in driver behaviour in the different countries, Brilon et al (1991) argue that capacity equations should not be transferred internationally, but "*Instead, each country has to find a solution of its own.*"

A number of microscopic traffic circle simulation programs are also discussed (SIMRO, INSECT and MODELC). These programs were all developed and validated in the country of origin and because of the general unavailability of traffic data at traffic circles and the costs of collecting such data, were written to assist in finding improved analytical techniques. The most important aspect of the simulation process is that of gap acceptance. The initial models were based on either a fixed critical gap (SIMRO) or a variable gap, which was independent of the geometric layout of the circle (INSECT). The gap acceptance process in MODELC is the only model which incorporates a variable critical gap as a function of both conflicting traffic and geometric layout of the circle.

As it has been shown that there is evidence that driver behaviour, especially at traffic circles, varies among drivers in different countries, it was decided to investigate traffic operations at local traffic circles. However, because of the sparse occurrence of these facilities in South Africa it was decided to use a simulation program to assist in the process. Although obtaining copies of some of the simulation programs is possible, in general the source codes of these programs are not freely available. These programs have generally been developed for research purposes and consequently are user-unfriendly and some of the earlier ones have been written in languages not commonly used today. Moreover, they often incorporate certain defaults and models which cannot be changed without the source code. For this reason together with the reasons discussed in Chapter 3, the decision was made to develop TRACSIM, a traffic circle simulation program to study driver behaviour at local traffic circles.

CHAPTER 3: APPLYING AVAILABLE ANALYSIS MODELS TO LOCAL CONDITIONS

The traffic circle analysis models developed in the United Kingdom (ARCADY) and Australia (SIDRA), as incorporated in the software programs ARCADY and SIDRA respectively, are among those being used in the analysis and design of intersections and specifically traffic circles in South Africa (Sutcliffe, 1988; Green, 1996). When this research, to investigate traffic operations at local traffic circles was initiated, one of the first questions to be answered was whether analysis models developed in other countries would be applicable to local drivers and road environments. An affirmative result would suggest that the possibility could exist that the local situation is similar to that in another country. The analysis models developed in such a country could then be used locally with reasonable confidence. It was decided to evaluate two of the available traffic circle analysis models (ARCADY and SIDRA) and to verify the accuracy of these models under the local conditions. In this chapter, delay estimates from the two software programs ARCADY and SIDRA are compared with observations at four local traffic circles. Initially the data collection process at the four circles is discussed after which comparisons are made between the observed and estimated results with the assistance of statistical techniques such as regression analysis and the Kolmogorov-Smirnov test.

3.1 Data Collection

For the validation of the two analysis models, four circles in three different cities were identified for data collection and analysis. Two of these traffic circles are situated in Durban, one in Chatsworth (Chatsworth circle) and the other one in Pinetown (Pinetown circle). The other two circles are situated in Pietermaritzburg (Chatterton circle) and Parow (Parow circle). Parow is situated in the Cape Town metropolitan area. The two circles in Durban have single circulating carriageways while the two other circles have double circulating carriageways. Table 3.1 summarizes the geometric features of each of these circles.

Table 3.1: Summary of circle geometrics

Circle	Geometry					
	No of entrances	No of exits	No of effect. circulating lanes	Central Island. Diameter	Circulating Lane width	Inscribed Diameter
Chatsworth	4	4	2	36.2	6.9	50.0
Chatterton	4	4	1	40.0	8.0	56.0
Parow	4	4	2	47.8	9.5	65.6
Pinetown	4	3	1	20.0	6.7	33.4

Chatsworth circle (See Figure 3.1) is situated on two major arterials (Arena Park and Moorton Roads) in Chatsworth, a suburb of Durban. Most of the peak traffic generated in the eastern parts of Chatsworth travels through this circle. The approach roads flare only slightly, so that there is only one entry lane into the circle at each of the four yield lines. The circulating carriageway is wide enough to accommodate both a stationary and circulating vehicle. Consequently, drivers occasionally stop in the circle to pick up or drop passengers in the circle. Nonetheless, this does not have a significant effect on the circulating flow.

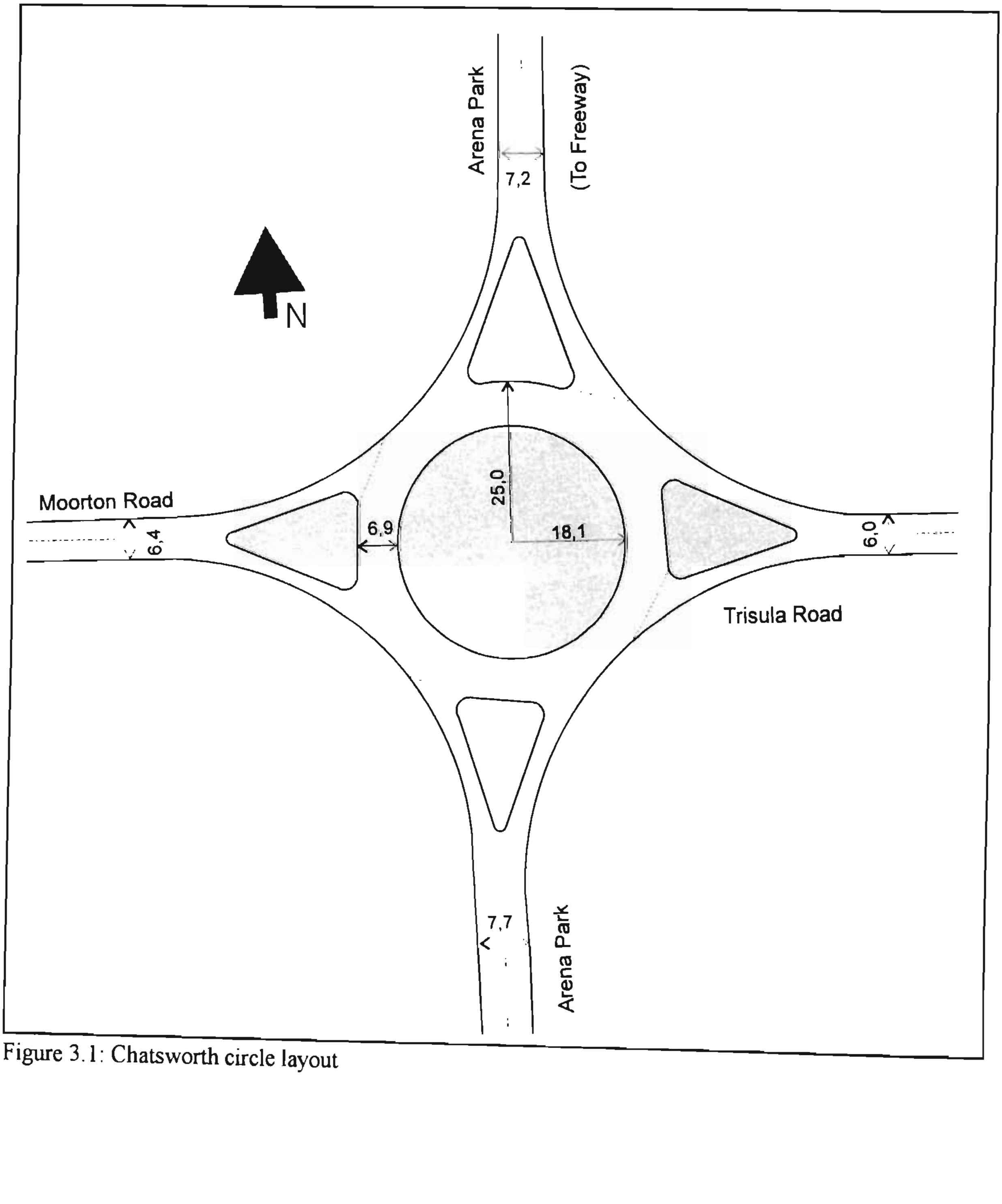


Figure 3.1: Chatsworth circle layout

The Chatterton circle (see Figure 3.2) is situated slightly north of the Central Business District (CBD) of Pietermaritzburg at the intersection of Chatterton (one of the important north-south arterial routes) and Armitage Roads. Both roads provide access to the N3 freeway situated to the east of the circle. The four-lane, two-way approaches from the north and south on Chatterton Road, flare very little and terminate with two lanes at the yield lines to the circle. The two-lane, two-way approaches from east and west along Armitage road flare into two lanes at the yield lines before entering the double circulating lanes with a central island diameter of 40,0 metres, inscribed within a diameter of 56 metres.

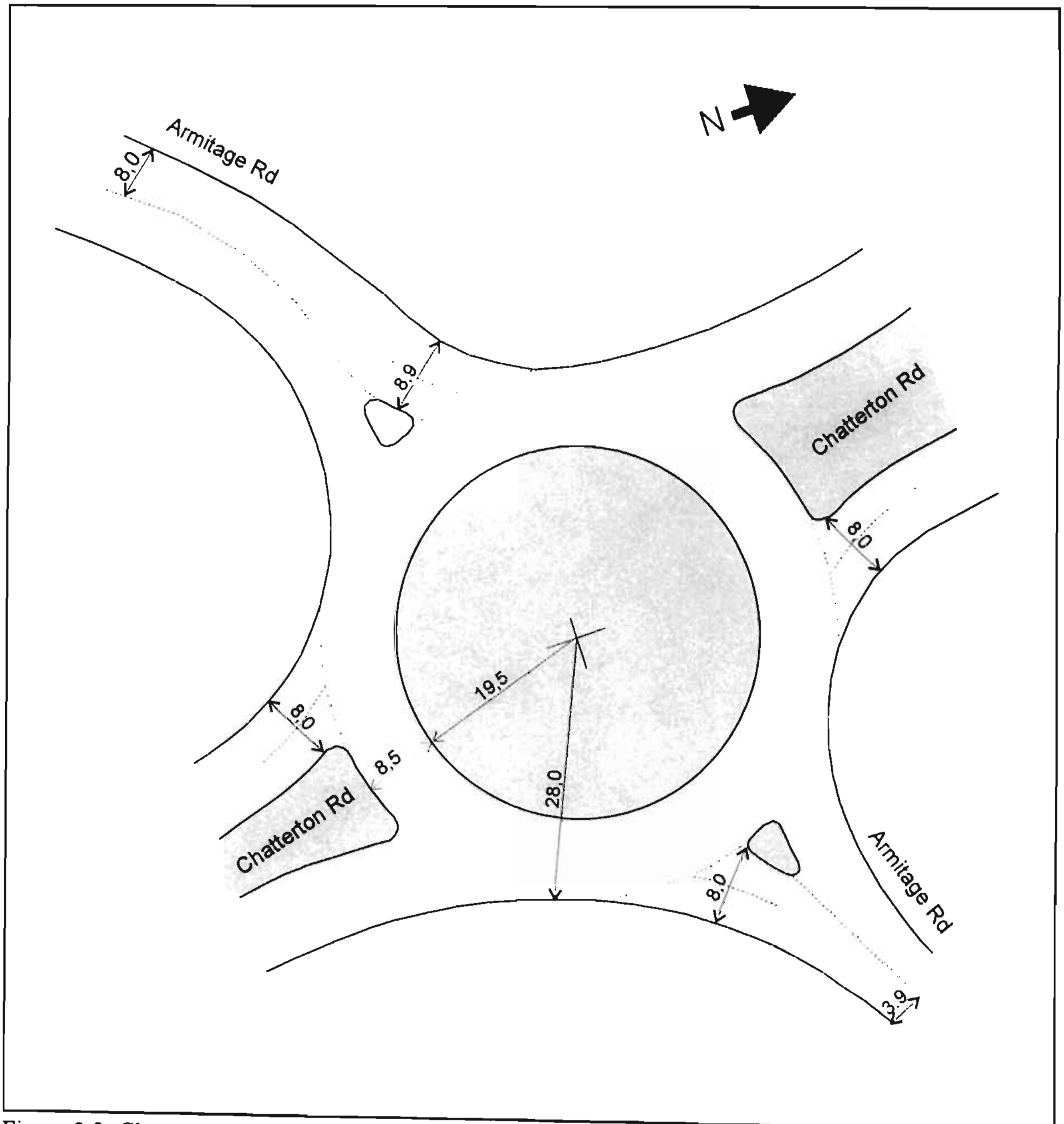


Figure 3.2: Chatterton circle layout

The third circle is the largest one of the four surveyed and has a central island diameter of 47.8 metres. It is situated in Parow, a suburb of Cape Town. The four, four-lane, two-way approach roads are major arterials providing north-south (McIntyre Road) and east-west (Frans Conradie Road) movement. McIntyre Road links the Parow Central Business District with the N1 Freeway. None of the approach roads flare significantly at the yield lines before they enter the double lane circulating carriageway (See Figure 3.3).

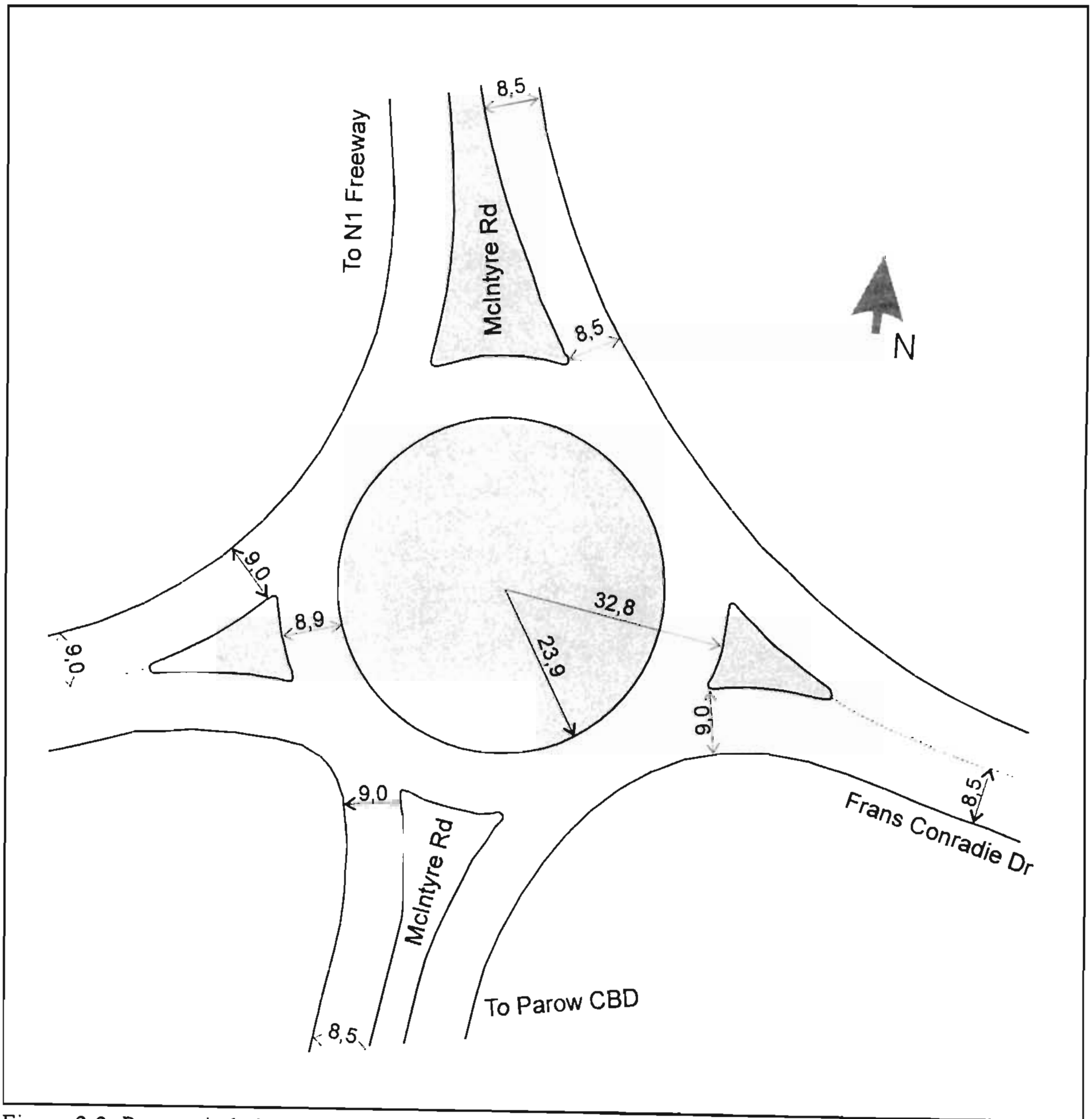


Figure 3.3: Parow circle layout

The fourth circle is situated in Pinetown, a municipality within the Durban metropolitan area, and at the intersection of Underwood, Maurice Nicols Roads and an off-ramp from the M13 freeway to Pinetown. Of the four circles this circle has been in operation for the shortest time (± 2 years). It is a small circle (central island diameter of 20 metres), with a single circulating carriageway and four asymmetrical approach roads, but only three exiting roads. The approach from the M13 is a one-way only off-ramp (See Figure 3.4).

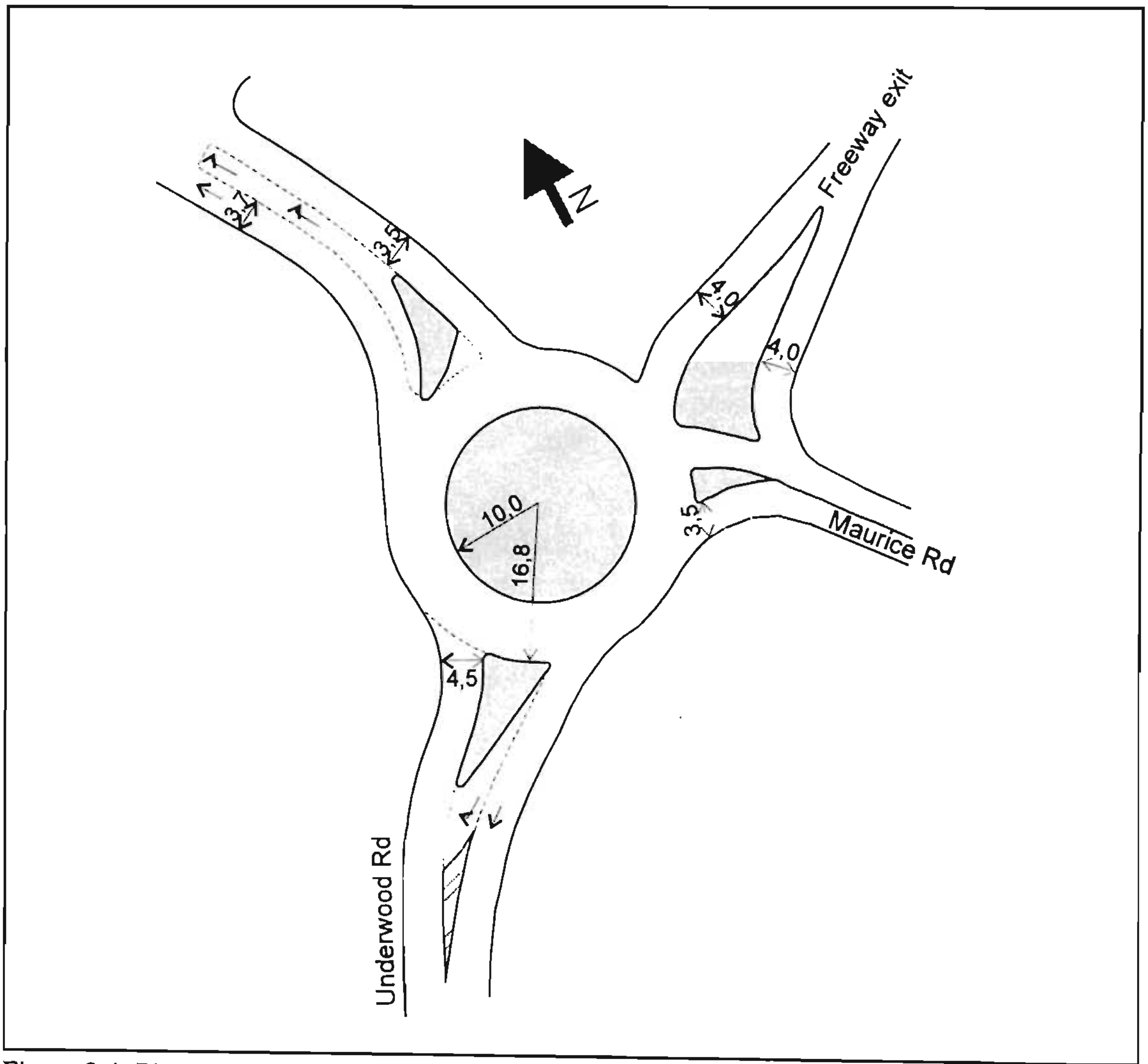


Figure 3.4: Pinetown circle layout

Table 3.2 summarizes the dates and times of the traffic surveys undertaken at the four circles as well as an average value of the traffic volumes moving through the circles during the peak hour, the peak hour factor and also the percentages of buses, taxis and heavy vehicles in the traffic streams. A detailed summary of the traffic count for each survey is included in Appendix A.

Table 3.2: Summary of traffic and delay surveys at the different circles

Circle	Date	Time	Peak hour volume (pcuph)	Peak hour factor	% of Buses	% of Combi-taxis	% of Heavies
Chatsworth	30/07/93 am	06:30 - 08:15	1751	0.89	3	10	2
	16/08/93 pm	15:30 - 17:30	1922	0.90	2	6	3
	17/08/93 pm	15:45 - 17:30	1961	0.94	2	7	2
	18/08/93 am	06:30 - 08:00	1537	0.98	3	11	2
Chatterton	31/01/94 pm	15:45 - 17:45	3089	0.90	0	3	3
	02/02/94 am	06:30 - 08:30	3261	0.92	0	6	3
Parow	07/02/94 pm	16:00 - 18:00	3677	0.95	0	3	2
	08/02/94 am	06:30 - 08:30	3583	0.89	0	4	2
Pinetown	15/08/95 am	06:30 - 08:15	1508	0.83	0	4	1
	15/08/95 pm	16:30 - 17:45	1579	0.93	0	6	1

The survey durations were generally at least one and three quarters of an hour and sometimes two hours where the traffic flows were still substantial. The high proportion of buses and minibus-taxis for Chatsworth indicate the greater use of public transport in this areas compared to the other areas. The peak hour factors are generally around 90%, which suggests a consistent arrival/demand pattern throughout the peak hour.

3.2 Delay observations

Few traffic circles in South Africa operate under capacity conditions, where queues on specific approaches exist continuously for longer than 20 minutes. Assessing the capacity estimates of certain models under South African conditions is thus difficult. Therefore it is easier to compare the delay estimates of a model with observed delays. The delay on an approach is a function of the degree of saturation, which in turn depends on the capacity.

A number of methods are available for field observations of delay (Van As & Joubert, 1993) and they can be classified into three basic approaches, i.e. the Point Sample Method, the Input-Output Method and the Path-trace Method. The three methods and their application are discussed in detail in Section 5.3.1. Due to its simplicity the point sampling method was used for recording queue lengths. As part of all the traffic surveys undertaken during the course of this study, the queue lengths on the different approaches to the circles were recorded at intervals of ten seconds. The observed queue lengths were used to determine the average stopped delay on each approach by assuming that the area under the queue length diagram represents the total stopped delay on each approach. The total stopped delay was calculated for consecutive 15 minute intervals. To convert the total observed stopped delay to delay per vehicle, the delay was divided by the total number of vehicles arriving during the specific 15 minute intervals.

According to Van As and Joubert (1993), there is a tendency to overestimate stopped delay, because observers tend to concentrate more on the upstream end of the queue. Observers are more likely to include a slow travelling vehicle, on the point of joining the queue, in the queue count. To take this overestimation of observed queue lengths into account Cohen and Reilly (1978) suggests a correction factor of 0,92. The queue length observations only result in an estimate of stopped delay and need to be adjusted to convert stopped delay to total traffic delay. To obtain total delay which includes deceleration and acceleration, the stopped delay should be increased by about 30% (ITE, 1976). Wherever stopped delay could not be used for comparison the stopped delay was converted to total delay by using this suggested factor.

3.3 ARCADY3 Comparisons

As explained in Section 2.3.2.1, capacity predictions in ARCADY3 are based on the straight line equation proposed by Maycock, and as finally formulated by Kimber (1980):

$$Q_e = F_i - f_i Q_c \qquad (3\text{-}1)$$

with Q_e - the entry capacity, Q_c - the circulating flow and F_i and f_i - constants determined by the geometric characteristics of the circle. Another feature of ARCADY is the linking of the individual arms of the traffic circle, where the conflicting circulating flow past any entry is related to the entry flows and turning proportions from upstream approaches. Time-dependent queuing theory is used for calculating queue lengths and delays (Kimber and Hollis, 1979).

Three of the circles (Chatsworth, Chatterton and Parow) were analysed with the software program ARCADY3. The existing circle geometrics and the observed traffic flows were used as input to the program, while the delay estimated by the program was compared with the observed delays on each approach. Figures 3.5, 3.6 and 3.7 present graphically the observed delays versus the ARCADY3 estimated delays for all the approaches of the Chatsworth, Chatterton and Parow circles respectively. Also shown on the graphs is the line where observed delays are equal to estimated delays. If the model is accurate, then data points should be on or close to this line, which is at 45 degrees if the scales of the two axes are equal. and will be refered to as the "ideal line".

All the observed stopped delays were multiplied by the factor 1,196 (1,3 x 0,92 - see section 3.2) to remove any overestimation due to observers' error and to increase the stopped delay to actual delay. ARCADY3 delay estimates are calculated for each time segment in terms of vehicle-minutes and the point is made that deriving the average delay per vehicle from this figure by dividing it with the number of vehicles in that time segment is not always possible. The reason for this is that vehicles arriving in one time segment may also suffer delay in other subsequent time segments (Webb & Peirce, 1990). An approximate method for calculating values of delay per arriving vehicle is suggested by Web and Peirce (1990). This approximate method was compared with delays obtained from dividing the vehicle-minutes by the number of vehicles, and little difference was found. The reason for this is the under-saturated conditions which existed at all the observed circles with few vehicles experiencing long delays and hence few vehicles were present in more than one time interval.

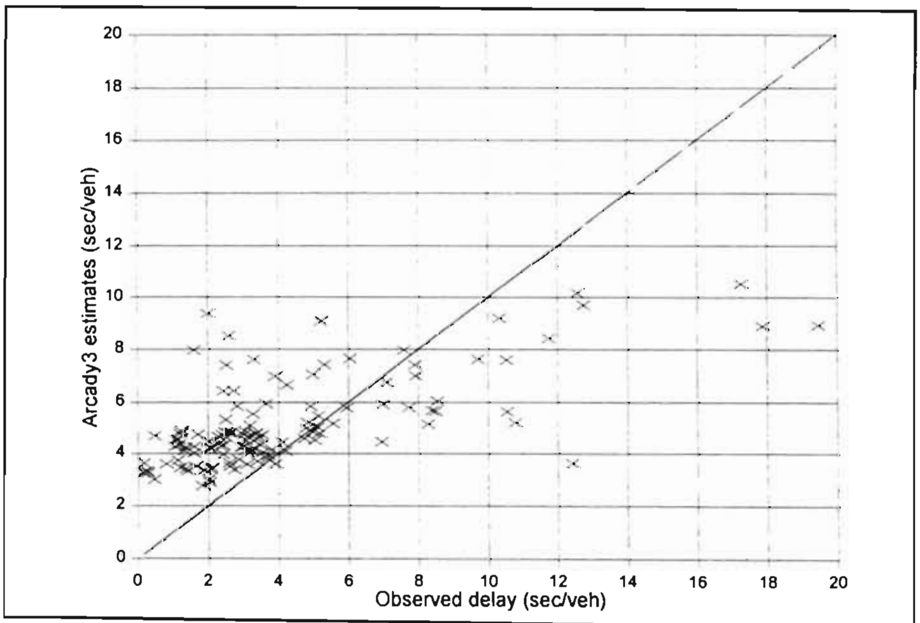


Figure 3.5: Chatsworth circle: Observed vs ARCADY3 delay estimates

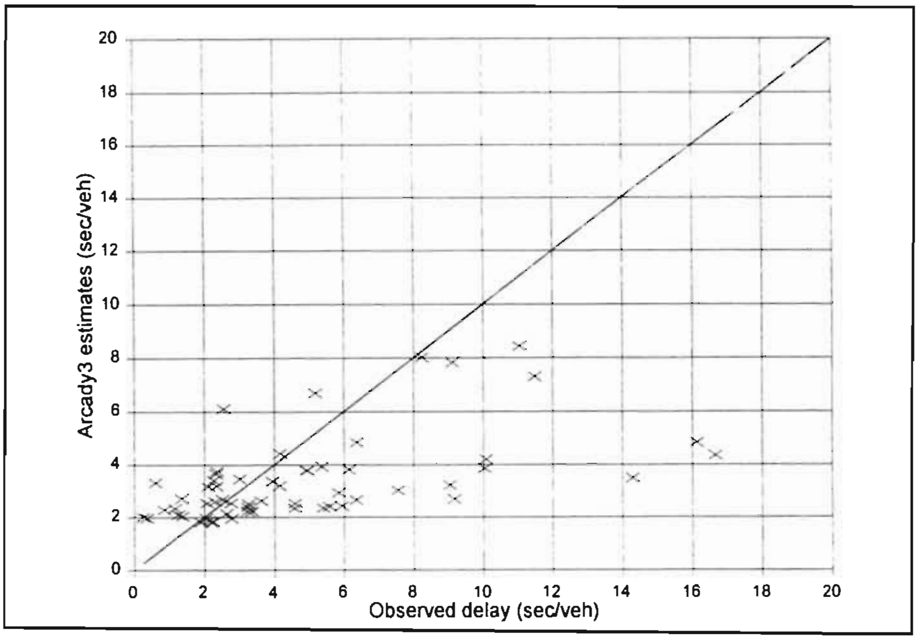


Figure 3.6: Chatterton circle: Observed vs ARCADY3 delay estimates

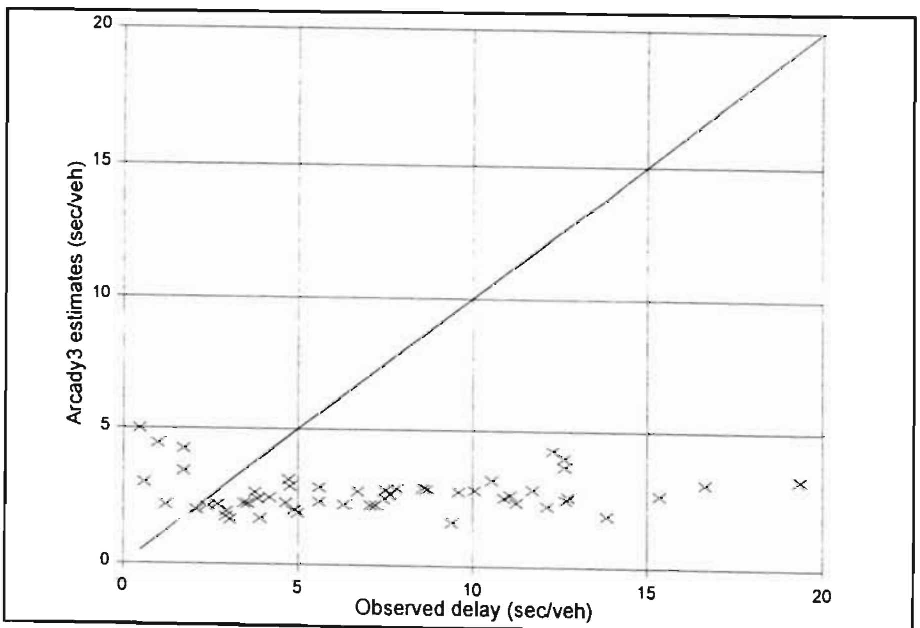


Figure 3.7: Parow circle: Observed vs ARCADY3 delay estimates

From these graphs it can be seen that there is clearly little correlation between the estimated data points and the ideal line where estimated delays are equal to observed delays. In most cases the observed delays are much greater than the delays estimated by ARCADY3. Only in the case of the Chatterton circle is there seemingly some correlation between the estimated delays and the ideal line.

Although it may seem from these graphs as if ARCADY3 does not predict stopped delay accurately under South African conditions, there is still a possibility that the two samples, i.e. observed and estimated, come from the same population. This can be explained by realizing that the observed delays are also only samples which will vary from day to day. On the other hand ARCADY3 predicts average values. To investigate the probability of two samples being from the same population, two types of statistical tests were conducted.

The first test involved fitting a straight line (regression analysis) through the observed versus estimated data points, and predicting confidence bounds for the intercept and gradient of this straight line. Two null hypotheses are then evaluated:

i) the intercept is equal to zero and
ii) the gradient is equal to 1.

The second test to confirm the results of the regression analysis was the Kolmogorov-Smirnov test

3.3.1 Regression Analysis

A straight line of the form $y = a + bx$ was fitted through the observed (x) versus ARCADY3 estimated (y) delay data, with a being the intercept (constant) and b the gradient. Based on the t-distribution, the confidence intervals for the intercept and the gradient are follows (Benjamin & Cornell, 1970):

$$\begin{aligned} &\underline{b} \pm t_{\alpha/2,(n-2)}\ s_b \\ &\underline{a} \pm t_{\alpha/2,(n-2)}\ s_a \end{aligned} \qquad (3\text{-}2)$$

where

$$s_a^2 = \frac{s^2}{n}(1 + \frac{x^2}{s_x^2})$$
$$s_b^2 = \frac{s^2}{ns_x} \qquad (3\text{-}3)$$

with $\underline{a}$ an estimator of the intercept,

$\underline{b}$ an estimator of the slope,

s_x the standard deviation of the independent variable

s the average of the squared residuals.

n number of observations

$t_{\alpha/2}$ value from t-distribution at α level of confidence.

The results of the regression analysis and confidence intervals for the three circles are given in Table 3.3. For a perfect fit, i.e. estimated delay equal to observed delay, the intercept (a) should be equal to zero and the gradient (b) equal to one, resulting in an equation of the form $y = x$.

Table 3.3: Results of Regression analysis - ARCADY3 comparison

Circle	No of observations	Correlation Coefficient	Intercept(a) Slope(b)	Standard error	95% Confidence Interval	
					Lower	Higher
Chatsworth	124	0,532	a: 4,30 b: 0,18	s_a: 0,191 s_b: 0,027	3,92 0,13	4,67 0,24
Chatterton	64	0,574	a: 1,86 b: 0,34	s_a: 0,793 s_b: 0,062	0,28 0,22	3,45 0,47
Parow	64	0,169	a: 2,68 b: 0,01	s_a: 0,137 s_b: 0,006	2,41 0,00	2,96 0,02

The regression analysis shows a moderate to poor correlation between the observed and estimated delay values. From the confidence intervals it can be stated with 95% confidence that none of the intercepts of the regressed data could be equal to zero. Similarly none of the gradients of the best fit straight lines through the data could be equal to one.

The null hypothesis

$$H_0: a = 0$$

can be rejected (except for Chatterton circle) and it can be stated that the estimates of the intercept are significantly different from zero at the 5% level of significance.

Similarly the null hypothesis

$$H_0: b = 1$$

can be rejected for all circles and it can be stated that the estimates of the slope of the regressed lines are significantly different from one at the 5% level of significance.

3.3.2 Kolmogorov-Smirnov test

To confirm the results of the regression analysis the Kolmogorov-Smirnov goodness-of-fit test was used. This test concentrates on the deviations between the hypothesized cumulative distribution function $F_x(x)$ and the observed cumulative histogram $F^*_x(x)$ (Benjamin & Cornell, 1970), where:

$$F^*(X^{(i)}) = \frac{i}{n} \qquad (3\text{-}4)$$

with $X^{(i)}$ the ith largest observed value in the random sample of size n. The test statistic D_2,

$$D_2 = \max \left[\left| \frac{i}{n} - F_x(X^{(i)}) \right| \right] \quad for\ i = 1\ to\ n \qquad (3\text{-}5)$$

the largest absolute difference between the hypothesized cumulative distribution and the observed cumulative histogram is then evaluated and compared with a computed critical value c_c. When comparing two samples to establish whether they come from the same population, the critical value c_c (Lindgren, 1965) can be calculated as follows:

$$c_c = \begin{cases} 1{,}22Y & for\ \alpha = 10\% \\ 1{,}36Y & for\ \alpha = 5\% \\ 1{,}63Y & for\ \alpha = 1\% \end{cases} \qquad (3\text{-}6)$$

$$where \quad Y = \sqrt{\frac{n_1 + n_2}{n_1 + n_2}} \quad with\ n_1\ and\ n_2\ the\ number\ of\ data\ points\ in\ the\ two\ samples$$

The following hypothesis can then be stated and the null hypothesis is accepted if $D_2 \leq c_c$:

H_0: The observed and estimated distributions are the same, i.e. come from same population.

H_1: The two distributions are different.

The test statistic D_2 was determined graphically as shown on Figures 3.8, 3.9 and 3.10 and is summarized with the critical values (c_c) and the conclusions of the hypothesis test in Table 3.4.

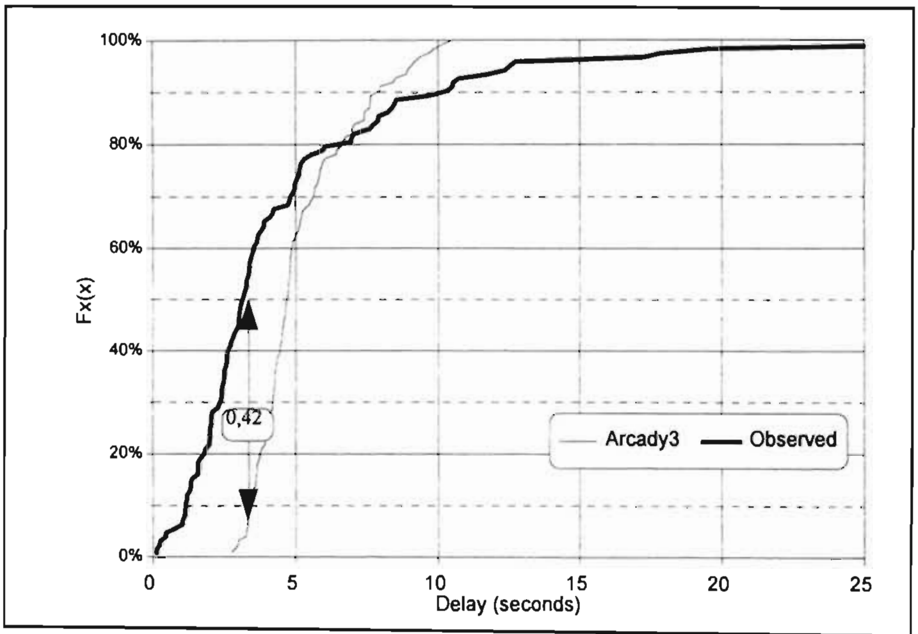


Figure 3.8: Cumulative distribution functions - Chatsworth circle

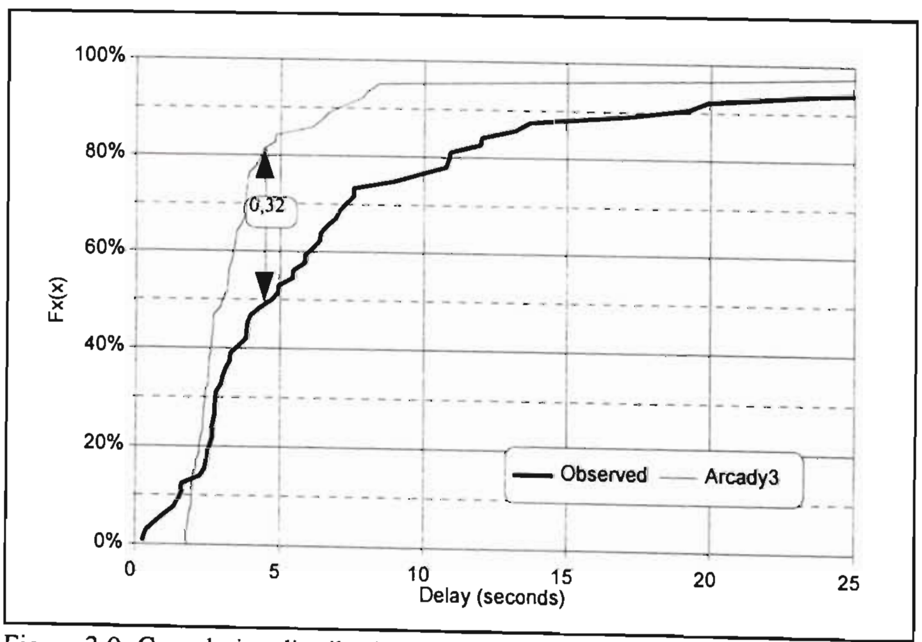


Figure 3.9: Cumulative distribution functions - Chatterton circle

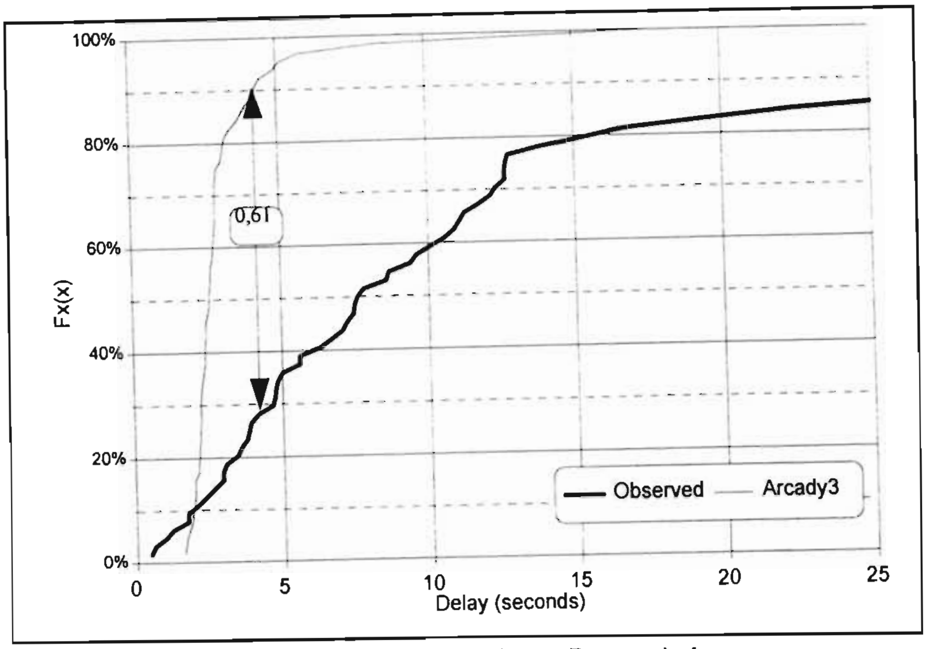


Figure 3.10: Cumulative distribution functions - Parow circle

Table 3.4: Kolmogorov-Smirnov test results - ARCADY3 comparison

Circle	No of observations	D_2	c_ϵ ($\alpha = 5$)	c_ϵ ($\alpha = 1$)	H_0 $D_2 \leq c$
Chatsworth	124	0,42	0,17	0,21	Reject/Reject
Chatterton	64	0,31	0,.24	0,29	Reject/Reject
Parow	64	0,61	0,24	0,29	Reject/Reject

These results suggest that at the 5% level of significance (5% likelihood that the null hypothesis is rejected in error) the null hypothesis should be rejected for all circles, i.e. the observed and estimated delays are not from the same population. At the 1% level of significance (1% likelihood that the null hypothesis is rejected in error) rejecting the null hypothesis for all the circles is also possible.

3.3.3 Other work on ARCADY3

Two other studies (Glass, 1995 and Sutcliffe, 1988) have investigated the suitability of ARCADY as an analysis tool under South African conditions.

Sutcliffe (1988) investigated the capacity predictions of ARCADY2 by comparing the program's estimates with observations at a number of local traffic circles. He concludes that capacity predictions in ARCADY2 are significantly higher than what was observed at the circles studied and that a reduction in the intercept (F_i) of (3-1) would achieve results closer to the observed delays. He suggests that the best way of reducing the intercept (which is a function of geometric characteristics of the circle) would be by reducing the entry width of each approach to the circle. This could be done by, instead of entering the actual entry width of the approach, calculating the width by multiplying the actual number of entry lanes with a factor of approximately 3,0 to 3,4.

Glass (1995) investigated delay predictions by ARCADY3 and compared it with observed delays at the Chatterton Circle in Pietermaritzburg. He also found that generally the ARCADY3 estimates were lower than the observed delay values, but that for some approaches the estimates were quite close. Based on Sutcliffe's recommendations, Glass then investigated the effect of changing the entry width of the approaches to the circle. For the Chatterton circle, Glass could not establish a single, reliable adjustment factor to reduce the entry widths of the approaches and in turn improve the delay estimates of ARCADY3 under local conditions.

3.4 SIDRA 4.1 Comparisons

This Australian-developed software program, based on gap acceptance theory, is becoming increasingly popular (Green, 1996) for the analysis of intersections in South Africa. The underlying theories are discussed in Section 2.2. The vast number of input parameters required makes the program extremely useful, but at the same time hazardous, in that wrong results may be obtained if the default values for these parameters are used without verifying them with local data. This section describes a comparison between observed delay data, obtained from three local circles (Chatsworth, Chatterton and Pinetown), and delays as estimated with SIDRA 4.1, the latest version of the program. SIDRA (Akçelik et al, 1995) allows for four different ways of calculating delay (see Section 2.3.3)

All the delay observations were made using the point or queue sampling method (see Section 5.3.1) The delays derived from the queue length sampling method are stopped delays and exclude the major stop-start deceleration and acceleration, but include all the time spent in the queue. Therefore, the only correct comparison with SIDRA delay estimates would be either to use method iii(b) (see Section 2.3.3 - stopped delay = overall delay / 1,3) or to multiply the observed delays with 1,3 to convert to total delay (excluding geometric delay) and to compare this with SIDRA's total delay obtained by using method (ii - see Section 2.3.3). Both methods would give similar results and it was decided to remain with the observed stopped delays and to use SIDRA's method iii(b) (stopped delay = overal delay / 1,3) to estimate stopped delay.

Initial comparisons indicated that SIDRA underestimated stopped delays significantly and so it was decided to increase the effect of exiting traffic (in terms of a percentage added to circulating traffic) from 0 to 50 percent. However, from observations at the three circles it does not appear that exiting traffic affects entry capacity. Increasing the effect of exiting traffic basically increases the circulating traffic. Another obvious variable, which affects the SIDRA stopped delay estimates, is the critical gap. Because of a lack of data on South African conditions at this stage of the research, the default values as calculated by the program (see (2-44) and (2-45)) were used. Figures 3.11 to 3.16 on the following two pages compare the observed versus estimated stopped delays for the three circles using both 0% and 50% exiting traffic.

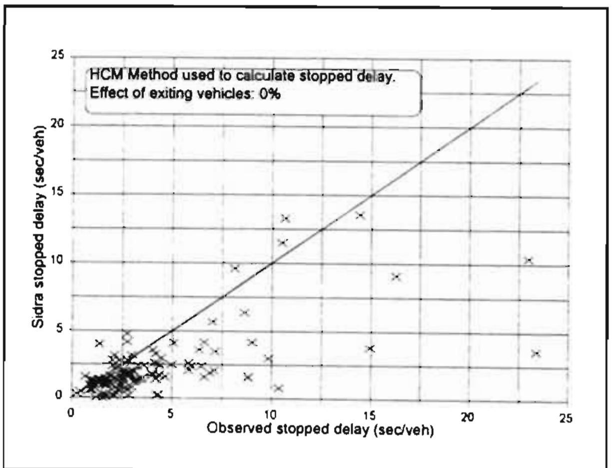


Figure 3.11: Stopped delay: Observed vs SIDRA estimates - Chatsworth circle (HCM Method, Exiting traffic: 0%)

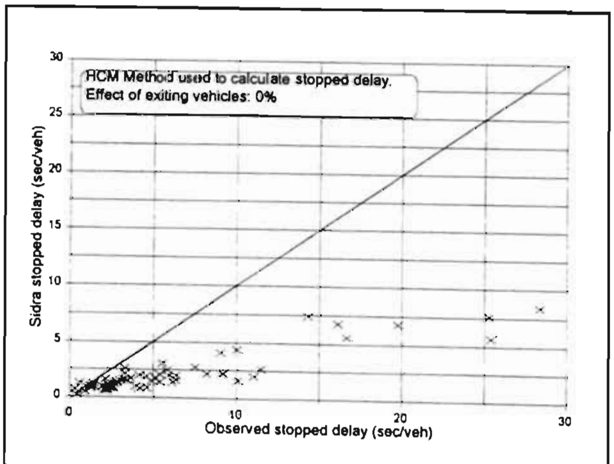


Figure 3.12: Stopped delay: Observed vs SIDRA estimates - Chatterton circle (HCM Method, Exiting traffic: 0%)

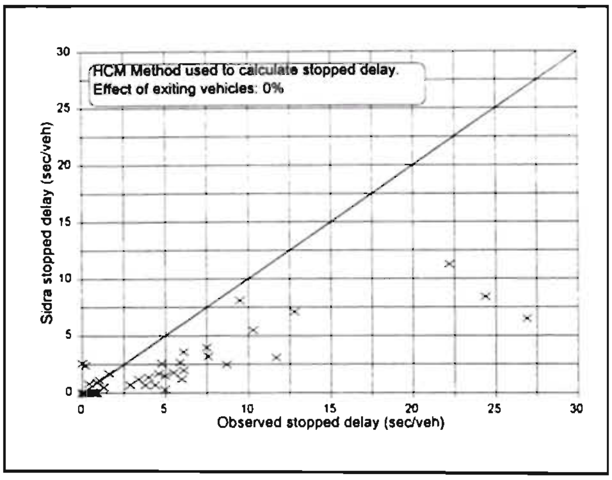


Figure 3.13: Stopped delay: Observed vs SIDRA estimates - Pinetown circle (HCM Method, Exiting traffic: 0%)

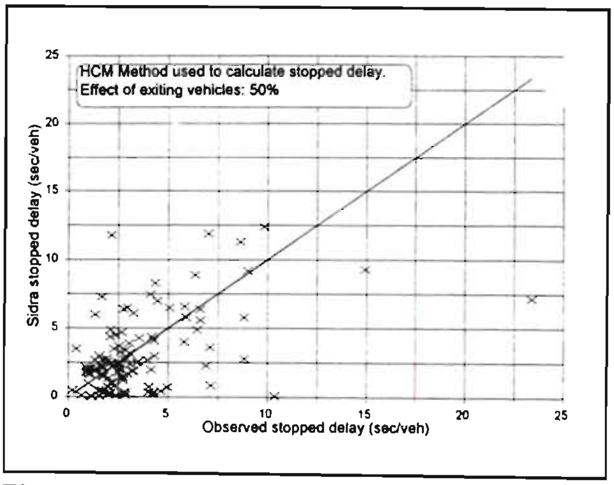


Figure 3.14: Stopped delay: Observed vs SIDRA estimates - Chatsworth circle (HCM Method, Exiting traffic: 50%)

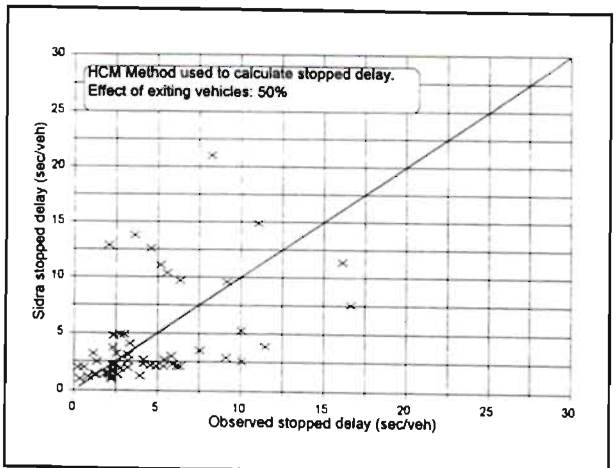


Figure 3.15: Stopped delay: Observed vs SIDRA estimates - Chatterton circle (HCM Method, Exiting traffic: 50%)

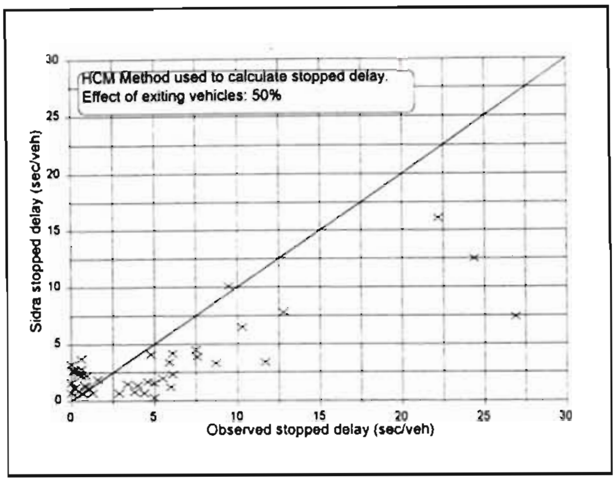


Figure 3.16: Stopped delay: Observed vs SIDRA estimates - Pinetown circle (HCM Method, Exiting traffic: 50%)

From the above graphs the estimated stopped delay values are apparently predominantly lower than the observed delays, i.e. most of the plotted points lie below the perfect fit line ($y = x$). By using the HCM Method to calculate stopped delay from total delays as estimated with SIDRA 4.1, the comparisons are even further improved if the effect of the exiting traffic is increased to 50%.

As for the ARCADY3 comparisons the same two tests (Regression analysis and Kolmogorov-Smirnov Test) were employed to evaluate the reliability of the SIDRA 4.1 estimates when compared with data at three local traffic circles. The results of the two tests are discussed in the following two sections.

3.4.1 Regression Analysis

Similar to the ARCADY3 comparison, a straight line of the form:

$$y = a + bx$$

was fitted through the observed (x) versus SIDRA 4.1 estimated (y) stopped delay data, where a is the intercept (constant) and b the gradient/slope. The results of the regression analysis are recorded in Table 3.3. Based on the t-distribution, the confidence intervals for the intercept and the gradient are calculated as shown in Table 3.5.

For a perfect fit the intercept should be equal to zero and the gradient be equal to one ($y = x$). The following two null hypotheses were defined to test this:

For the intercept: H_0: a = 0

For the gradient/slope: H_0: b = 1

These hypotheses were tested at the 5% level of significance and the results of the tests are shown in Table 3.5. When the hypothesis is accepted at the 5% level of significance, it means that there is a 5% likelihood that this is done in error.

Table 3.5: Results of Regression analysis - SIDRA 4.1 comparison

Circle	Method/ Effect of exiting traffic	No of obser-vations	Correlation Coefficient	Intercept(a) Slope(b)	Standard error	95% Confidence Interval		Accept H_0
						Lower	Higher	
Chatsworth	HCM Method Exiting: 0%	120	0,672	a: 0,61 b: 0,41	0,234 0,042	0,14 0,33	1,07 0,50	No No
	HCM Method Exiting: 50%	120	0,665	a: -0,71 b: 1,39	0,799 0,143	-2,29 1,10	0,87 1,67	Yes No
Chatterton	HCM Method Exiting: 0%	64	0,477	a: 1,15 b: 0,19	0,389 0,044	0,38 0,10	1,93 0,28	No No
	HCM Method Exiting: 50%	64	0,452	a: -0,66 b: 1,80	3,970 0,451	-8,59 0,90	7,28 2,70	Yes Yes
Pinetown	HCM Method Exiting: 0%	48	0,741	a: 0 ,65 b: 0,27	0,783 0,037	-0,93 0,20	2,23 0,35	Yes No
	HCM Method Exiting: 50%	48	0,723	a: 1,10 b: 0,39	1,188 0,056	-1,29 0,28	3,50 0,51	Yes No

The correlation between observed and estimated delays is not good and only in the case of the Pinetown circle are the correlation coefficients higher than 70%, which suggests some linearity of the data. In all but three cases the null hypothesis (intercept equal to zero) is accepted but in only two cases both hypotheses can be accepted at the 5% level of significance. The reliability of the model depends on both the intercept and gradient. The two cases where both hypotheses are accepted have the lowest correlation coefficients.

From the regression analysis it cannot be stated with confidence that SIDRA 4.1 is a good estimator of stopped delay under South African conditions. If the HCM Method is used and/or the effect of exiting traffic is increased, the estimates improve. However, too little information is available about the effect of exiting traffic at local traffic circles, and from observations at the circles it seems as if exiting traffic has only a minor effect on entering traffic.

3.4.2 Kolmogorov-Smirnov test

Similar to the ARCADY3 comparisons, the Kolmogorov-Smirnov test was used to verify the results of the regression analysis and to provide further insight into the reliability of SIDRA 4.1 under South African conditions. The test is explained in detail in section 3.3.2. The test statistic D_2 (see (3-5)) was obtained graphically from the cumulative distribution plots (see Figures 3.17 to 3.19) and the critical value (c_c) was calculated using (3-6) (see Table 3.6 for results).

The following hypothesis was again stated and the null hypothesis is accepted if $D_2 \leq c_c$:

H_0: The observed and estimated distributions are the same - come from same population.
H_1: The two distributions are different.

Table 3.6: Kolmogorov-Smirnov test results - SIDRA 4.1 comparison

Circle	Method	Effect of exiting traffic	No of obser-vations	D_2	c_c ($\alpha = 5$)	c_c ($\alpha = 1$)	H_0 $D_2 \leq c$
Chatsworth	HCM Method	0%	120	0,30	0,18	0,21	Reject/Reject
		50%	120	0,15	0,18	0,21	Accept/Accept
Chatterton	HCM Method	0%	64	0,54	0,24	0,29	Reject/Reject
		50%	64	0,20	0,24	0,29	Accept/Accept
Parow	HCM Method	0%	48	0,30	0,28	0,33	Reject/Accept
		50%	48	0,23	0,28	0,33	Accept/Accept

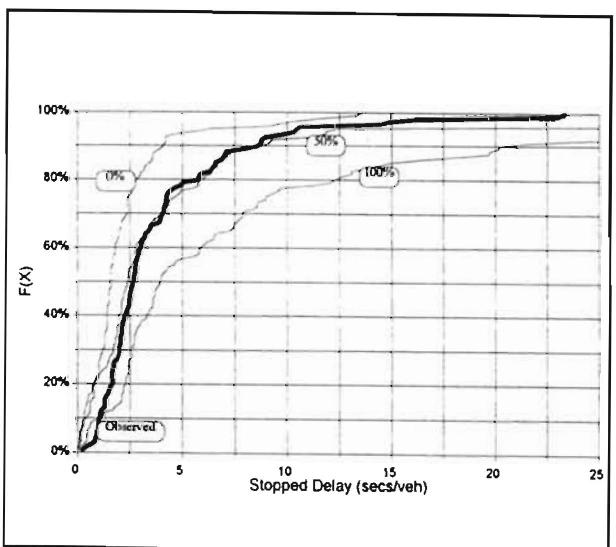


Figure 3.17: Cumulative distribution functions Chatsworth circle (HCM Method)

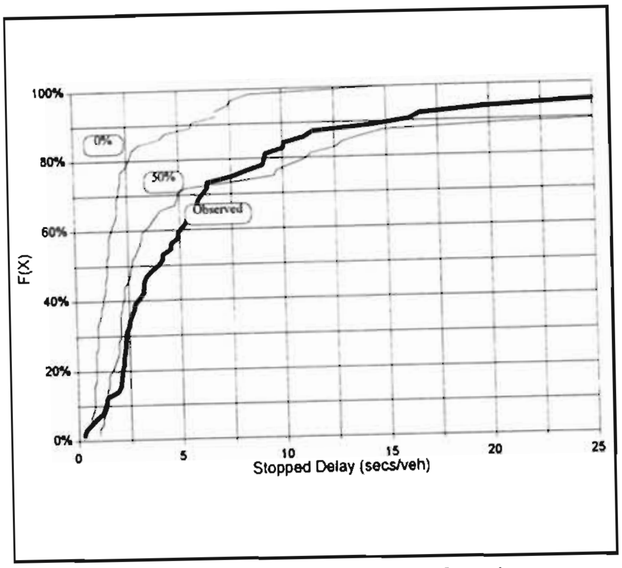


Figure 3.18: Cumulative distribution functions
Chatterton circle (HCM Method)

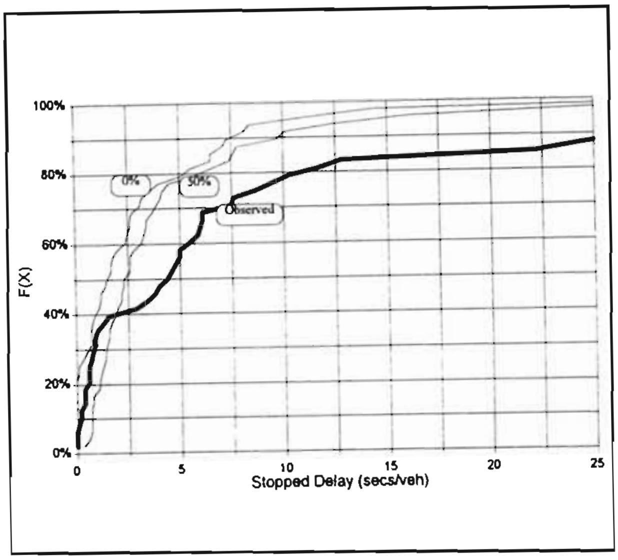


Figure 3.19: Cumulative distribution functions
Pinetown circle (HCM Method)

These results suggest that at the 5% level of significance (5% likelihood that the null hypothesis is rejected in error) and at the 1% level of significance (1% likelihood that the null hypothesis is rejected in error) rejecting the null hypothesis for all the circles is possible except for the cases where the HCM Method is used with 50% effect of exiting traffic.

The Kolmogorov-Smirnov test confirms the doubt about the reliability of SIDRA 4.1 to accurately estimate stopped delay under South African conditions unless the effect of exiting traffic is increased and the HCM Method is used to convert total delay to stopped delay.

3.5 Summary

In this chapter a comparison between the delay observations at local traffic circles and estimated delays is discussed. The delays were estimated using two software programs: ARCADY3 (a software program developed in the UK and based on regression analysis) and SIDRA 4.1 (a software program developed in Australia and based on analytical techniques).

Two statistical tests (Regression analysis and Kolmogorov-Smirnov test) were employed to test the reliability of the delay estimates by both ARCADY3 and SIDRA 4.1. Both tests suggest significant doubt regarding their accuracy and reliability. It is possible to improve the accuracy of ARCADY3 by reducing the entry widths (although not common to all cases). In the case of SIDRA 4.1, accuracy can be improved by increasing the effect of exiting traffic. However, the extent of the change in entry width, or of the increase in exiting traffic is not exactly known and does not always result in an increase in accuracy.

The increasing use of traffic circles in South Africa and the subsequent increase in use of analysis software such as ARCADY and SIDRA, prompt the need for more accurate methods of analysis. Reliable methods applicable to South African conditions must be developed or, in the light of the apparent inaccuracy of ARCADY and SIDRA, further research is required in finding ways of calibrating these methods for local conditions. However, the absence of local circles operating at or near capacity, complicates efforts to study and provide the necessary solutions.

This apparent lack of information on local traffic circles and the associated difficulties in obtaining such data, motivated the development of a simulation program. Such a program which has been calibrated for local conditions can be used extensively to generate not only the required data for developing methods of analysis, but also to investigate for instance, the effect of an imbalance in traffic on the approaches to a traffic circle.

CHAPTER 4: DEVELOPMENT OF SIMULATION MODEL

In this chapter the development of a traffic circle simulation model is discussed. Firstly, the problem and the reasons for developing a simulation model are stated. Secondly, the analysis of the operation at a traffic circle and the important processes which need to be included in a simulation model are discussed. Thirdly, the details of each stage of the development process of the traffic circle simulation program (TRACSIM which is used to model the system) are discussed. Finally, the results of a sensitivity analysis of how sensitive the model outputs are to a change in the different input variables to the system are presented. In the context of this thesis the term model is used to refer to the actual simulation program while system is used to refer to the physical intersection with its approaches and the traffic moving through it.

The flow diagram in Figure 4.1 and adapted for this thesis from Young, et al (1989) shows the general process which was used to analyse the system and to develop the program. In general this chapter has been structured according to the flow process illustrated in Figure 4.1, but often the discrete steps are not entirely independent or clearly differentiated from the steps preceding or following, and there is thus some overlapping. Therefore, the chronological order within which the different steps are discussed are not precisely in the same order as the arrow diagram in Figure 4.1.

4.1 Problem definition and motivation for simulation model

In Chapter 3 the evaluations of two existing intersection analysis models (ARCADY and SIDRA) are discussed. In South Africa, these models are generally used for intersection analysis and specifically for traffic circle analysis (Sutcliffe, 1989; Green, 1996). From the evaluation in Chapter 3 it follows that these traffic circle analysis models are not sufficiently accurate under local conditions. Furthermore, it is essential that the models should either be calibrated for local driver behaviour or that new models be developed for the estimation of traffic performance at circles. This confirms Brilon et al's (1991) argument that because of the difference in driver behaviour, capacity equations should not be transferred internationally. "*Instead, each country has to find a solution of its own*".

However, to find a local solution for traffic circle analysis is not simple, not only because of the rarity of such facilities but also because of the lack of high traffic volumes using them (see Chapter 2). As a result, it was decided to develop a simulation program to assist in the process of developing local solutions for traffic circle analysis. Although it is possible to obtain copies of a few of the available

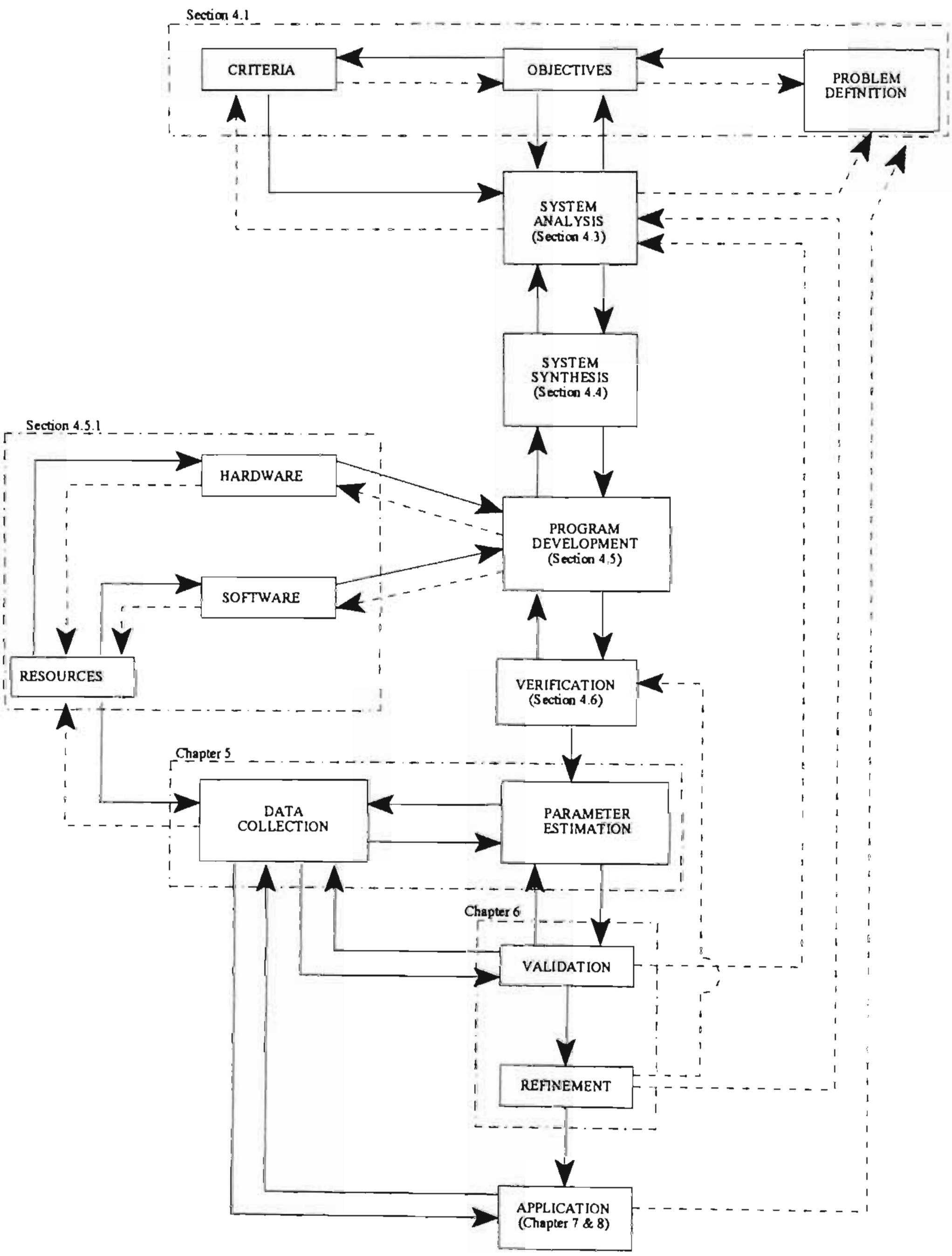


Figure 4.1: Development process (after Young, et al, 1989)

simulation programs, in general the source codes of these are not freely available. These programs were mostly developed for research purposes and are consequently user-unfriendly while some of the earlier ones were written in outdated programming languages. Moreover, they often incorporate certain defaults and models which cannot be changed without the source code.

MODELC (Chung, 1993) was made available to the author for research purposes and was evaluated by an undergraduate at the University of Natal (Bruton, 1996). This proved to be unsatisfactory because no user manual was available, nor any access to the source code to allow changes to the program if needed. Errors which occurred during the simulation could be neither interpreted nor corrected. Moreover, in the specific version made available, the variable gap acceptance model could not be used. This was one more reason to develop TRACSIM, a traffic circle simulation program to study driver behaviour at local traffic circles.

4.2 Objective and criteria for simulation model

The *objective* of this simulation model was to provide a tool which could assist in the research process by providing a means of generating local traffic circle data through simulation and to evaluate traffic operations at different circles. This study concentrated on single circulating and single approach lane circles. The following requirements were defined for the simulation model.

i) To evaluate different arrival volumes, the effect of arrival distribution and the origin-destination pattern, the model has to allow for different turning volumes and arrival distributions.
ii) To evaluated different sizes and shapes the model has to allow for alternative geometric layouts.
iii) It has to allow for accurate simulation of driver behaviour and vehicle characteristics.
iv) It must be relatively easy to use in terms of data requirements (simple input).
v) It must provide output statistics that can be compared and evaluated.

To meet requirement (v) as listed above, the *criteria* or outputs to be used to assess the efficiency of the system under evaluation were:

i) the delay per vehicle,
ii) the number of vehicles having to stop, and
iii) the average and maximum queue lengths.

These measures of performance are generally used in traffic engineering to define a level of service and are also relatively simple to observe in the field.

4.3 System Analysis

This section concentrates on not only defining the scope of the model, but breaking the model up into different sub-modules in an effort to simplify the system and its operation. System in this context refers to the physical intersection with its approaches and the traffic moving thereon. Only once the system and its components were understood could the development of the simulation program start.

A traffic circle is another type of priority intersection in which drivers approaching the intersection (stream A in Figure 4.2) have to give way to conflicting vehicles (stream B in Figure 4.2). To proceed on their journey drivers have to accept gaps (enter at X in Figure 4.2) in the conflicting traffic.

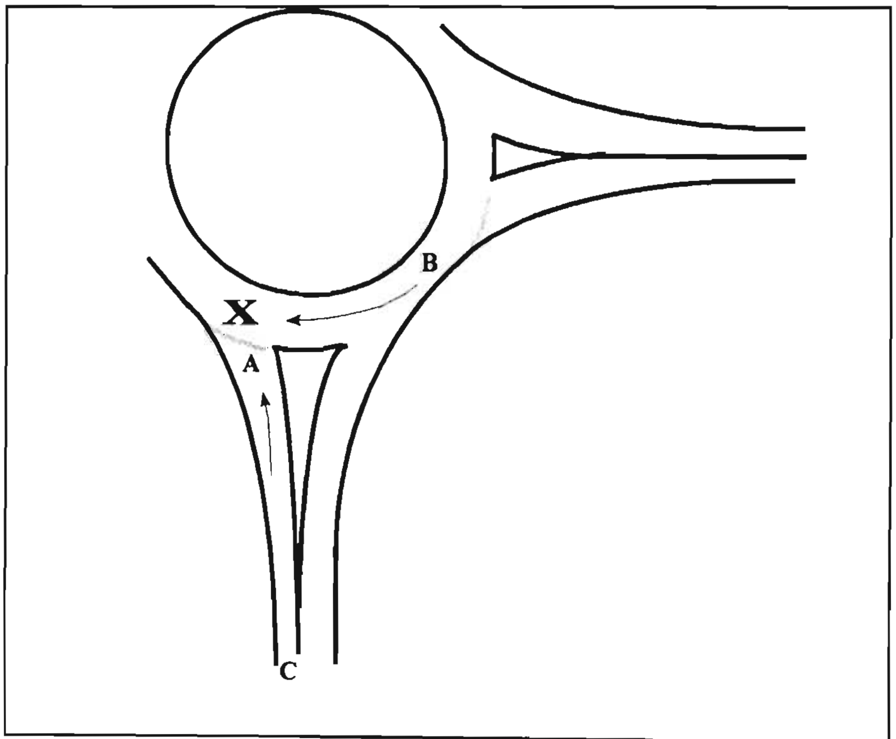


Figure 4.2: Traffic circle operation

There are three key elements that determine the rate at which stream A can enter the circle and hence the delays and stops experienced. These are (references are to Figure 4.2):

i) *When* the vehicle in stream A arrives at the yield line, i.e. the arrival pattern of traffic on the approach (position C).
ii) *The availability of a gap* in stream B at point X, which is a function of the arrival pattern of the circulating traffic.
iii) If an available gap in stream B is accepted or rejected by the driver of the stream A vehicle, i.e. the nature of the gap acceptance process (stream A merging with stream B).

The delay to any vehicle depends on its arrival time at the yield line, the driver's gap acceptance characteristics ((iii) above) and the arrival pattern of the circulating stream ((ii) above). The arrival pattern of the circulating stream at (X) in turn depends on the arrivals and gap acceptance processes at the other approaches to the circle. The arrival time at the yield line of any approaching vehicle depends on its arrival time on the approach (at point C in Figure 4.2) and the preceding queue present on the approach. Moreover, the preceding queue is a function of the rate and pattern of arrival of the preceding vehicles, the nature of the gap acceptance process and the pattern of conflicting flows.

It is clear from the inter-relationships between the various elements that the operation at a traffic circle is much more than just a series of T-intersections (see Section 1.4) and that a simple analytical or empirical model, even if it is iterative, cannot do justice to the actual behaviour at a circle. This complicated process lends itself extremely well to simulation where the conflicting flow (stream B in Figure 4.2) on which so many of the other processes depend, can be simulated and not estimated.

With the above analysis of the system in mind and to simplify the system and the simulation thereof, the operation at a circle was separated into five different sub-models or sub-processes, which are defined as follows:

i) approaching /arriving,
ii) queuing,
iii) entering,
iv) circulating, and
v) exiting.

The remainder of Section 4.3 describes the various sub-models which need to be incorporated into the simulation model.

4.3.1 Approaching

Vehicles arrive at an intersection either at the driver's desired speed and deceleration (non-following) or by following a slower vehicle. If there is a queue of vehicles waiting on the approach the vehicle will decelerate and join the back of the queue. If there is no queue, the vehicle will advance to the yield line from where it will either enter the intersection or remain queued. With no upstream control devices influencing the flow of the traffic stream and low traffic volumes, vehicles will be arriving at random. At higher volumes where significant platooning occurs, the first vehicle in a platoon will arrive at random.

The best way of describing this stochastic process of vehicle arrivals is by using the time headways between vehicles. As the estimation of time headways is important to a number of applications in traffic engineering, it has been researched and documented by a number of researchers (Akcelik & Chung, 1994a; Branston, 1976; Cowan, 1975; Tolle, 1976; Van As & Joubert, 1993 - among others) and will not be discussed in detail in this thesis.

Time headways vary considerably between two boundary conditions (May, 1990). At the one end of the scale, under low flow conditions, headways can be considered random, where arriving at any point in time is equally likely for a vehicle except for a minimum following headway (**random headway state**). On the contrary, when traffic flow is near capacity the time headways are nearly constant, except for driver error, which causes deviations around the mean. This defines the other boundary condition and can be classified as the **constant headway state**. The headway distribution most often found in practice, occurs between the two boundary conditions and is often referred to as the **intermediate headway state** (May, 1990).

On the one side of the spectrum of headway distributions, random headways can be modelled with the negative exponential distribution. On the other hand the constant headway state can be represented with the normal distribution (May, 1990). The intermediated headway state is the most difficult to analyse and basically two different approaches are employed to model these headways.

Firstly, the generalized mathematical model approach is used, where theoretical distributions such as the displaced negative exponential distribution, the gamma distribution and log-normal distribution are used to model the time headways. The second approach attempts to model a distribution $f(t)$ of all headways while treating the distribution of followers $g(t)$ and non-followers $h(t)$ separately, such that:

$$f(t) = \phi g(t) + (1 - \phi) h(t) \tag{4-1}$$

where ϕ represents the proportion of following vehicles (Van As & Joubert, 1993). Methods to deal with this mixed headway distribution are: combined distributions (composite negative exponential distribution), semi-Poisson distributions and the travelling queue distributions (constant headway queuing model or bunched exponential model and log-normal queuing model).

Although the negative and shifted negative exponential distributions have been used extensively in the study of traffic headways (Chung, 1993; Chin, 1983, Van As & Joubert, 1993), work by Akçelik and

Chung (1994a) indicates that the bunched exponential distribution is much more realistic than the other two distributions and they strongly recommend its usage. This distribution states that a proportion, $(1 - \alpha_2)$, of vehicles are following at a headway β_1, while a proportion, α_2, are moving freely at greater random headways (Cowan, 1975). The cumulative distribution function, *F(t)*, of the bunched exponential distribution, representing headways in a multi-lane traffic stream and the probability that a headway is less than or equal to t, is stated as (Akçelik & Chin, 1994a):

$$F(t) = 1 - \alpha_2 e^{-\lambda(t-\beta_1)} \quad for\ t \geq \beta_1$$
$$= 0 \quad for\ t \leq \beta_1 \qquad (4\text{-}2)$$

where λ is a parameter given by $\lambda = \alpha_2 q_t / (1 - \beta_1 q_t)$ and

q_t total arrival flows on all the lanes.

α_2 proportion of free vehicles = $e^{-bB_1 q_c}$

b bunching factor

β_1 minimum arrival headway

The bunched exponential distribution is a generalization of the exponential model and both the negative exponential and shifted negative exponential models can be derived from (4-2). The negative exponential model can be derived from (4-2) by setting $\beta_1 = 0$ and $\alpha_2 = 1$, which means that $\lambda = q_t$. The shifted negative exponential model can be found in (4-2) by setting $\alpha_2 = 1$.

Every vehicle arriving on an approach to an intersection is either turning left, going straight, turning right or turning back from where it came (u-turn) if all four of the movements are possible at the intersection. These turning movements can be observed and expressed as proportions of the arrival volumes. Turning movements observed in practice are mostly deterministic although there can be a slight daily variation. The arrival speeds of non-following vehicles are not uniform and observed free speed distributions are commonly described by using the normal distribution (McShane and Roess, 1990) with an observed mean speed and standard deviation.

4.3.2 Queuing and Entering

Vehicles arriving on an approach with no queue present will advance to the yield line and enter the circle if a large enough gap is available in the conflicting circulating traffic stream. If the available gap in the circulating stream is smaller than the minimum acceptable gap of the driver waiting at the yield line then the vehicle will wait at the yield line for the next gap. Successive gaps are then evaluated until a gap greater than the waiting driver's minimum acceptable gap is presented in the circulating stream. If there is a queue present on arrival, then the arriving vehicle will join the back of the queue and will move up in the queue as vehicles depart from the front of the queue.

The important aspects when modelling the queuing and entering process at an intersection are:

i) How the queue is formed and when vehicles join the queue.
ii) How vehicles move up in the queue.
iii) How the gap acceptance decision is made.

The *queuing process* affects the way in which delays are estimated, while the *move-up times* between successive vehicles and the *gap acceptance* decision of drivers, affect the capacity of an approach and hence the delay on that approach

Queuing

In a traffic model vehicles can be placed either in horizontal queues or vertical queues at the yield line (Van As, 1979). Horizontal queues are similar to the situation in practice where vehicles queue back in space and the actual time of arrival in the queue is earlier than the time of arrival at the yield line if there were no queue present. With vertical queues the vehicles are stacked vertically in a queue at the yield line and arriving vehicles only join the queue at the yield line. In Section 2.3.3 it is shown that traffic delay estimates when vehicles are placed in vertical queues are negligible and that the theoretical arrival and departure times are sufficient to describe headways between arriving vehicles and to estimate the traffic delay of a vehicle. However, when comparing stopped delay estimates using vertical queues in a simulation model with stopped delays as observed using the point sampling method (see Section 5.3.1) there can be a difference (Roebuck, 1996). Because the definition for stopped delay (see Section 2.3.3) includes all queue move-ups, this difference only occurs at the back of the queue, i.e. the arrival end and not at the front of the queue, the departure end. The difference is shown graphically in Figure 4.3 (Roebuck, 1996).

Figure 4.3 also shows the theoretical stopping headways for a model using vertical queues (assuming instantaneous deceleration) and the actual stopping headways where vehicles join a horizontal queue. The modelled delays are suitable for comparison with other analytical models estimating traffic delays. However, when comparing observed delays obtained from the point sampling method (horizontal queues) with modelled delays using vertical queues it is clear that there is a difference in the time of stopping and subsequently in the delay. The actual headways (H_{wa}) between stopping vehicles are clearly smaller than the theoretical headways (H_{wt}).

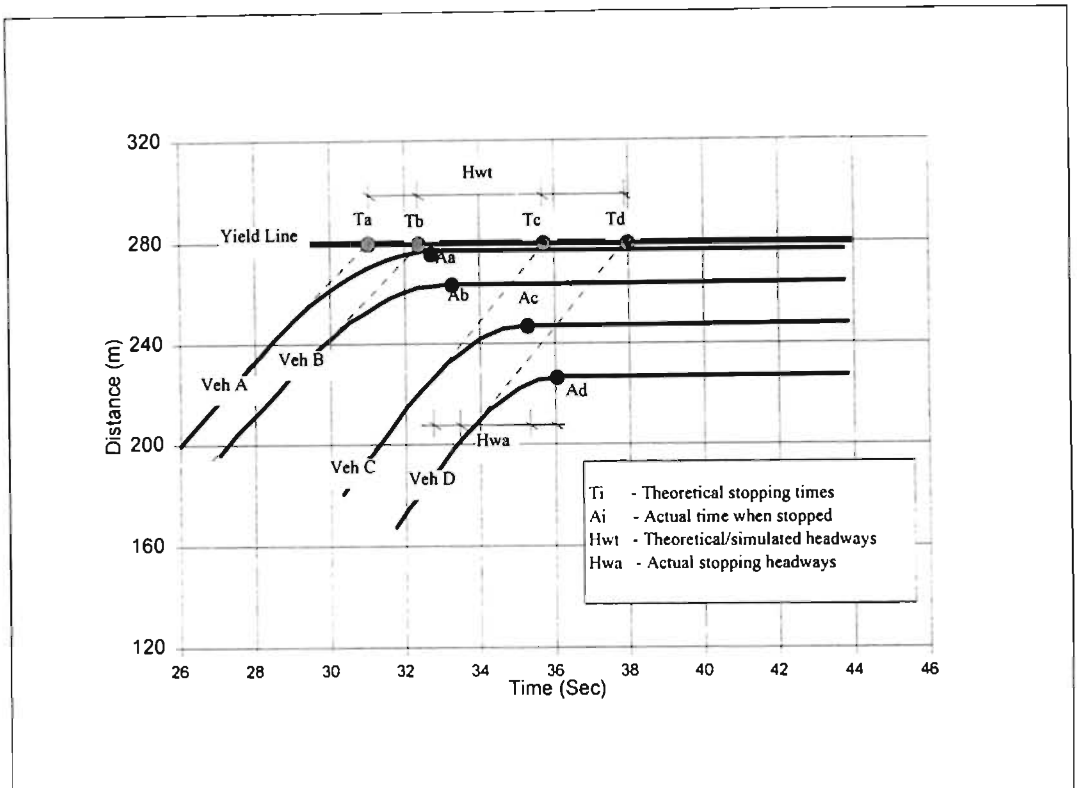


Figure 4.3: Space-time diagram to show difference between observed and simulated delays

The important point in this comparison is the relative size of headways and not the actual point in time when the stop takes place, but how long after the previous stop the next stop will take place. From Figure 4.3 it can be seen that the theoretical and actual stopping headways between vehicles will be similar if there is no queuing. However, the moment a vehicle stops in a queue, the headway timed from the previous stop, will be relatively smaller than if there was no queue, because the vehicle now has to stop sooner. This is a constant difference equal to the stopped spacing (X) divided by the approach speed (V). Thus when comparing simulated delays with observed delays, which have been obtained using the point or queue sampling technique, a more accurate simulated delay will be obtained by reducing the headway between two successive vehicles with X/V, if the second vehicle stops behind the first vehicle. The same reduction needs to be applied to all subsequent vehicles stopping in the queue.

Whenever the modelled delay estimates are to be compared with other delay models which estimate total traffic delay, the vertical queue method can be used without any adjustment for the horizontal queuing effect.

Move-up times

Headways or move-up times between vehicles entering the circle usually follow a normal distribution with the mean of these headways referred to as the move-up time or follow-up headway (Troutbeck, 1989). The slower the second vehicle in the queue moves up to the yield line, following the departure of the first vehicle in the queue, the smaller the size of the remainder of the gap. Troutbeck (1989) found that for circles with more than one approach lane, there is a difference in move-up times between the dominant (lane with most traffic) and sub-dominant lanes and that the move-up time depends on the inscribed circle diameter, the number of entry lanes, the number of circulating lanes, and the circulating flow rate. Troutbeck suggests two different equations for determining the move-up times (see Equations 2-35 and 2-36). In the latest version of SIDRA these equations were adapted to allow for the effect of heavy entry flows against low circulating flows as shown in (2-45) (Akçelik et al, 1995). Although well researched the above-mentioned methods for determining move-up times were developed in Australia and are not necessarily applicable in South Africa.

Gap acceptance

The entering process at any priority intersection where minor vehicles enter a major conflicting stream or circulating stream (in the case of traffic circles) is based on a gap acceptance process. The decision by the driver of a vehicle approaching a circle, whether to enter the circulating stream or not, is based on the availability of gaps in the circulating stream and the size of the smallest gap the driver is prepared to accept. This gap is defined as the driver's *critical gap*. The entering process firstly depends on the distribution of available gaps in the circulating stream and secondly on the distribution of drivers' smallest or critical gaps. This gap varies from driver to driver and can also vary from time to time for the same driver, even at the same intersection.

The most common method of evaluating gaps and lags (remaining part of a gap) and their acceptance at priority intersections is based on a time hypothesis, where both the available gap and the critical gap are measured in terms of time. (Van As & Joubert, 1993). The Australian analytical techniques for evaluating traffic circles are also based on a time hypothesis for gap acceptance (Troutbeck, 1989). Equations 2-37 and 2-44 show the relationships found by Troutbeck (1989) and (Akçelik et al, 1995) between the critical gap, the entry width, the number of circulating lanes, the circulating flow rate and the move-up time. Some other methods which have been used for gap acceptance are (Gibbs, 1968):

i) A time and minimum distance or modified time hypothesis. Mostly used for merging and lane changing models.

ii) Distance hypothesis
iii) Angular velocity hypothesis
iv) Change in dimension hypothesis.

Observations made in the course of this research at traffic circles indicated that relating the gap acceptance process to distance might be more accurate than relating it to time. Relating the process to a specific position in the circle could be even more accurate. This is discussed in more detail in Chapter 7. A driver's first perception of an approaching conflicting vehicle is its position. Only once a gap acceptance decision has been made based on the position of the approaching conflicting vehicle, will drivers reconsider their decision based on the speed of the approaching vehicle. A small distance gap may be rejected but once it is assessed as driving slowly, the gap could be accepted. On the other hand a large distance gap can be accepted "in principle", but once the high speed of the approaching vehicles is perceived, the gap is rejected (Gibbs, 1968).

Once a gap is accepted the vehicle does not immediately enter the circulating lane, but it takes some time to move from the yield line to the circulating pathway. This time is referred to as the ***move-in time*** and must not be confused with the move-up time. A vehicle accepting a gap is first moving-in before it reaches the circulating pathway and then starts to circulate. Similar to move-up times these move-in times which are a function of the vehicles acceleration and cruising speeds follow a normal distribution, described by a mean and standard deviation.

4.3.3 Circulating and exiting

Once vehicles have entered the circle they travel around the central island in a circulating path until they exit the circle when they reach their desired destination. Vehicles circulate at their free speeds until they have to slow down for a slower leading vehicle. This vehicle will be followed until it exits or the following vehicle exits. Upon exit of the slower vehicle the following vehicles will tend to accelerate back to their desired speeds. It is also possible for a fast circulating vehicle to have to slow down for a slow entering vehicle which has accepted too small a gap.

In a simulation model some kind of rotary mechanism is necessary to move the vehicles around the circle. This rotary mechanism can either rotate the vehicles at a uniform constant speed (carousel mechanism) or rotate them at their own free circulating speed. In the latter case where vehicles circulate at free circulating speeds, some kind of vehicle-following model is required to control the movement around the circle (Chung, 1993).

4.4 Model Synthesis

The previous section concentrated on the basic principles involved in the different components or sub-models of the simulation model. In this section the merging or linking of the different models into a working program is discussed. Firstly to generate headways between vehicles, vehicle speeds, turning movements, move-up times and driver characteristics, a random number generator is required to select individual values from an assumed distribution. Secondly, once vehicles and drivers have been defined, the simulation model needs to move the vehicles around the circle using some kind of update mechanism. And lastly to successfully update positions and speeds the relationship between the different sub-modules must be clearly defined.

4.4.1 Random number generators

The simulation of random effects and events such as traffic arrivals and vehicle speeds, requires a set of random numbers which satisfies tests for randomness to ensure that each number occurs with equal frequency without any serial correlation. Although using the most precise and deterministic machine ever made to generate random numbers may seem obtuse, computer algorithms are most efficiently employed for the task. Although these algorithms generate so-called pseudo-random numbers in a deterministic way, these numbers still satisfy the various statistical tests of randomness. One major advantage of such a sequence of computer generated random numbers for simulation programs, is that it can be repeated as often as is necessary.

According to Press et al. (1986) "*A working, though imprecise, definition of randomness in the context of computer generated sequences, is to say that the deterministic program that produces a random sequence should be different from, and - in all measurable respects - statistically uncorrelated with, the computer program that uses its output. In other words, any two different random number generators ought to produce statistically the same results when coupled to your particular applications program.*"

In this section the distinction is made between uniform random deviates and random numbers drawn from a normal distribution, i.e. normal random deviates. Uniform random deviates probably conform to the popular perception of random numbers which are numbers that lie within a specific range (typically 0 to 1), with any number in the range equally likely. The normal random deviates are drawn randomly from a normal distribution and therefore the probability of generating a number close to the mean is much higher than generating a number smaller than three standard deviations from the mean.

A number of computer algorithms for random number generations have been developed over the years (Young et al, 1989; Press et al, 1986; Knuth, 1969) and are not reported on in detail in this section. The following four methods are described in more detail in Appendix B:

i) Mid-square method
ii) Mid-product technique
iii) Linear Congruential generator
iv) Three tiered Multiplicative Congruential generator

Matlab (see Section 4.5.1) has its own built-in uniform random number generator, which is based on the linear congruential method. However, for TRACSIM (see Section 4.5) because of the reasons as discussed in Appendix B, it was opted to use the Wichmann and Hill, Three tiered Multiplicative Congruential method for random number generation instead of the built-in uniform random number generator.

Normal random deviates are invariably based on uniform random numbers. Matlab also has a built-in normal distributed random number generator which uses a second copy of the linear congruential method to generate a set of uniformly distributed random numbers which are then transformed to a set of normal random numbers (Matlab, 1992) using the method as described by Box and Muller (Forsythe et al, 1977). In TRACSIM a separate routine was used to generate normal random deviates. While still employing the Box-Muller method, this routine referenced uniform random numbers produced by the Wichmann and Hill routine and not the built-in generator based on the linear congruential method. A listing of this routine to generate normal random deviates is included in Appendix B.

4.4.2 Simulation Update procedure

As examining all parts of a traffic system simultaneously in any computer simulation is impossible, some method is required to scan or update the traffic system. This method of updating must not only be systematic but also efficient in terms of computing time. Three different update procedures are possible: the first is known as vehicle update, the second, time or periodic update and the third event update (Young et al, 1989). Of the three methods the last two are more commonly used (Chung, 1993; Van As & Joubert, 1993). As these methods are discussed in sufficient detail elsewhere (Young et al, 1989), only a summary of each is given below.

i) Vehicle update. Vehicles are simulated individually and sequentially as they progress through the system and their position in time and space is stored to be used for the simulation of subsequent vehicles. It thus takes each vehicle separately and one vehicle after another. This process can be complicated and is only feasible when a vehicle cannot influence any of the preceding vehicles which have already been simulated. The approach is only useful when a simulated vehicle can only influence the movement of vehicles behind it (Young et al, 1989).

ii) Event update/scanning. With event scanning the system is updated only when predefined events occur. During the update (which happens at unequal time intervals) future events are determined and an event-selector selects the next event from the list of all possible events. This technique can reduce computing time significantly especially where there are long time gaps between events. However, it often requires a number of simplifying assumptions (Van As & Joubert, 1993) and when the number of events increase and occur within short time intervals, computing time can be significantly more than for instance the time update method. According to Chung (1993) vehicle-following cannot be modelled using the event scanning technique, because it requires continued forecasting of vehicle movements. This is however possible, although somewhat complicated, and was used in this research.

iii) Time update or periodic scanning. With the time update technique the simulation process progresses in equal time steps or time slices of a selected length. This method can be simpler to program, but often requires long computing times (Van As & Joubert, 1993). The simulation process involves processing all events at the beginning of a time slice and forecasting the state of the system at the end of the time slice. This forecasted state is then used as the beginning for the next time slice (Chung, 1993). The length of the time slice is important in terms of accuracy and computing time (Van As & Joubert, 1993). A too short time increment could result in many additional and possibly unnecessary computations, increasing the computing time. On the other hand, a long time increment could be too coarse and events might be skipped.

According to Conway, Johnson and Maxwell (1959) the time update method should be used whenever:

$$t < (m - 1)$$

where t is the average duration between events and m is the number of entities that must be examined.

For the purposes of this research it was decided to used the event scanning or event update method. The basic reason for this is that the nature of the operation at a traffic circle lends itself extremely well to be described by specific events. This will increase the accuracy of the model. The possible increase in computing time when compared with the time update method was not seen to be a significant factor due to the increasing availability of fast desktop computers. The different events as identified for this model are summarized in Section 4.4.3.

4.4.3 Identification of events

To enable the simulation model to be updated using an event scheduler, events had to be identified which, when about to occur would cause the system to update and calculate a new system state. Events are identified as those happenings (e.g. vehicle arrivals, exits or changes in speed) which, once they occur will change the state of the system. Sixteen such events were identified, but they can be classified into seven groups which are as follows:

i) Arrival at the yield line. (Events 1 to 4) This can be described as the first event to take place before anything else can happen in the circle. Only once it has arrived at the yield line can a vehicle use the opportunity to accept a gap or not. Vehicles arriving at the back of the queue are not considered as another event for they do not change the system state. The only important aspect of their arrival is their time of arrival, which is recorded for calculating of the delay. For a four-approach circle four such events are possible.

ii) Exiting circle. (Event 5) Once a vehicle exits the circle it provides the opportunity of a gap to be accepted and also allows following vehicles to accelerate to their desired speeds.

iii) Circulating vehicles passing a specific "critical position" in the circle. (Events 6 - 9) These positions are defined in the input to the model. They are defined as a point opposite the entry to the circle, which once passed by a circulating vehicle, ends the conflict this circulating vehicle was to a possible entering vehicle. A circulating vehicle will prevent a vehicle from entering from a specific approach until it passes the critical point for that entry. The vehicle waiting on that approach then has an opportunity to enter if there is a large enough gap available, i.e. if the headway between the circulating vehicles passing the "critical point" and the next conflicting vehicle is large enough. For each entry such a point needs to be identified and so for a four legged circle four such events are possible.

iii) Arrival in the circle. (Events 10 - 13) Vehicles entering the circle from the yield line take time moving from the yield line to the point where they actually start to circulate. The moment they arrive in the circle (starting to circulate) it constitutes another event.

iv) Catching up with a slow circulating vehicle. (Event 14) A fast circulating vehicle catching up with a slow vehicle will have to slow down at a point and follow the slow vehicle at a minimum headway.

v) Catching up with a slow entering vehicle. (Event 15) A fast circulating vehicle might conflict with a slow entering vehicle which has accepted too a small gap, and the fast vehicle will have to slow down and follow the slow vehicle at the minimum headway.

vi) Following vehicle accelerates back to desired speed. (Event 16) Once a slow vehicle exits, the following vehicles can accelerate back to their desired speeds. If there is more than one following vehicle they cannot simultaneously accelerate, but will do so in turn.

4.5 Modules, interaction and program development

The use of different modules in a simulation program allows for structured programming, where a module performs a specific task and in the process employs certain input data to provide output to be used in the other modules. Once a module is programmed, it can be used again in the program without repeating the process. This section discusses the development of the simulation program to model traffic flow around single lane circles. The program is named TRACSIM which is a acronym for TRAfic Circle SIMulation.

Figure 4.3 shows a flow diagram of TRACSIM and how the different modules and subroutines interact. For clarification and ease of comprehension the flow-diagram blocks as shown in Figure 4.4, describe the action during the execution of the subroutine rather than giving the subroutine name. However, these blocks are all numbered and the key to the figure provides the names of the subroutines as used in TRACSIM. The remainder of this section covers first the selection of the programming language and the hard- and software requirements and secondly the different modules used in the program and their interaction as shown in Figure 4.4.

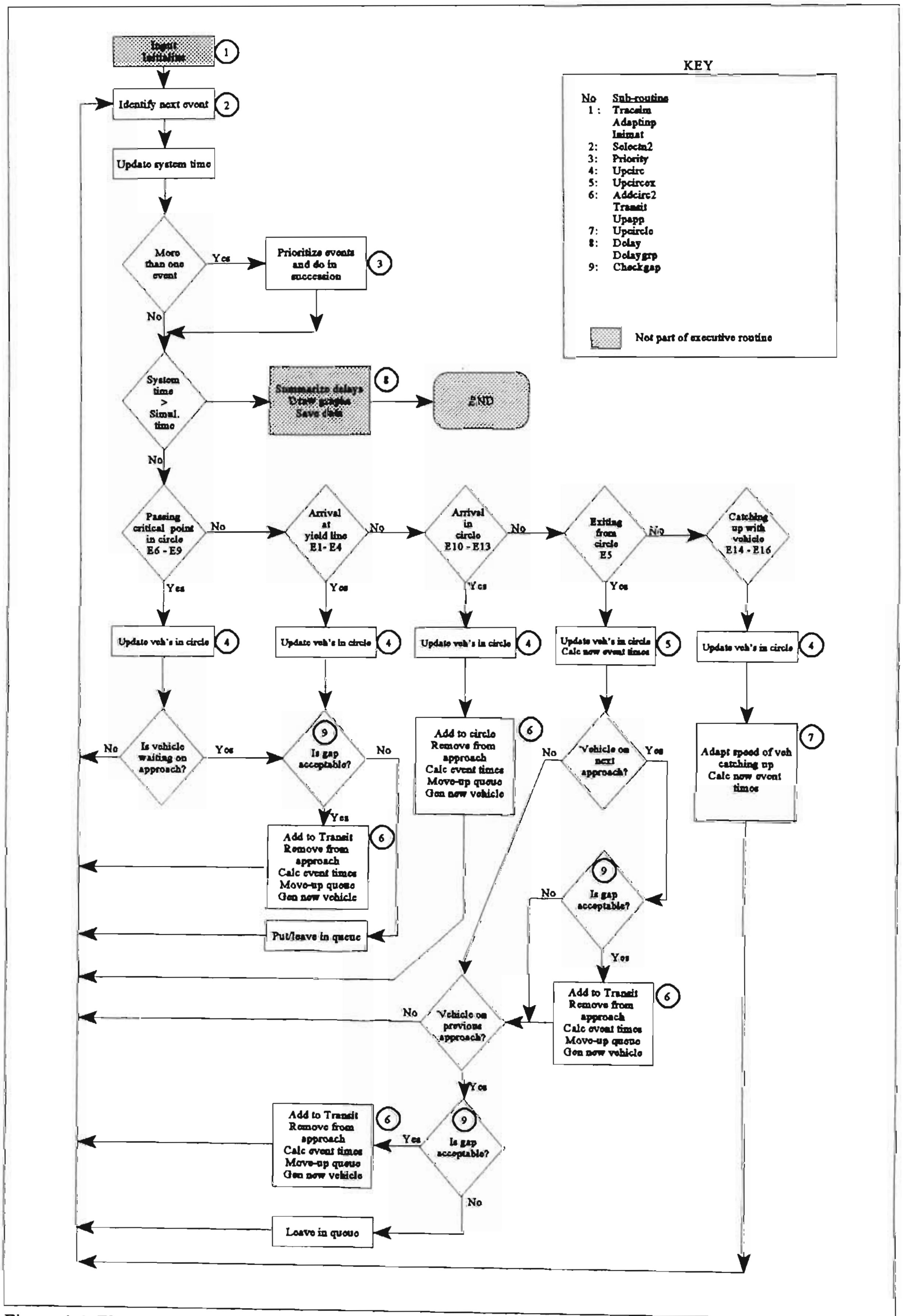


Figure 4.4: Flow diagram for executive routine in TRACSIM (called aaatrac.m appearing on pages B6 to B7 in Appendix B

4.5.1 Programming language, hardware and resources

At present there is a range of programming languages available for use on microcomputers. From the initial third generation languages of BASIC and FORTRAN, programming languages have developed to the sophisticated QuickBASIC, C, turbo Pascal and TurboC which provide a fully integrated programming environment, which includes windows that show system status, program code, execution status, etc. (Young, et al, 1989) Fourth generation languages such as dBaselll and Lotus 123 and the fifth generation languages such as LISP, PROLOG and MATLAB are also finding important applications. These languages are descriptive rather than imperative and provide a general description of the problem, instead of a detailed solution of each step as is required in the imperative languages such as Pascal, C or BASIC. Most of the work is left to the compiler and one result is that the programs are much shorter and the programming of specifically graphic applications are often much easier.

The choice of programming language depends on a number of factors such as availability of software, hardware, operating system and the aims of the model that is to be created. However, the final decision rests with the programmer. This simulation program was written in MATLAB. MATLAB is a fifth generation programming language, suitable for technical computing, high performance numeric computations and easy-to-use visualizations. MATLAB (1992) stands for Matrix Laboratory, and is an interactive system with a matrix (that needs no dimensioning) as a basic data element. This feature enables the solving of many numerical problems in a fraction of the time that it would take to write a program in a language such as Fortran, Basic or C. For example, to find the sine of 1001 numbers between 1 and 10, the MATLAB code will read as follows:

```
t = 0:0.01:10          t is the vector with values from 0 to 10 in 0,01 steps
y = sin(t)
```

Often this negates the use of *for* and *while* loops. This is not only much simpler to program, but is also executed in only a fraction of the time that it would take if the traditional *for* loop was employed (MATLAB, 1992). Matlab code basically consists of a series of ascii-files or m-files, because each file has “m” (*.m) as an extension. These files are then executed from within the Matlab environment, but can be coded or written in any text file editor. Being in the Windows environment the most obvious editor is the notepad available in all the Windows programs.

In MATLAB the modular approach is extremely useful, because a module can be defined as a function and can be called as a function during execution. For instance a module to evaluate the gap acceptance process can be called as a function. Given the matrices with circulating and entering traffic as input to this function, it can return either an acceptance or a rejection. Within the function the matrices can be manipulated without changing the matrices in the program from which it was called. This gap acceptance function can be called from anywhere in the program and as often as desired. This approach was used successfully in the development of TRACSIM. The different sub-modules (approaching, queuing, entering, circulating, and exiting) defined for the model, as listed in Section 4.3, were used as basis for the modular structure of the program, but obviously in the program a few more subroutines were used.

Although this feature of MATLAB to use a matrix as a basic element is not particularly useful in an event or time scanning simulation program where the individual events or time intervals have to be considered sequentially, it is a simple matter to program the code and producing graphic outputs. Moreover, the availability of the MATLAB software within the University of Natal and the option to develop the program in the Windows environment, made it an obvious choice. The program was developed on an IBM compatible microcomputer under the Windows 3.1 environment.

Development started on an IBM compatible microcomputer with a 486DX66 processor, but the final analysis and simulation runs were completed on a 586P166 processor. The 586 processor reduced the simulation time by more than 50%. During vacations a number of machines dedicated for student use, 486DX33 microcomputers, could also be employed to assist with the sensitivity analysis. All the coding and the development of the program were performed by the author.

During the data collection stage, which is discussed in Chapter 5, a number of undergraduates assisted in the collection and analysis of data. Some of this work formed part of the students' final year dissertations (Glass, 1995; Ross, 1995; Bruton, 1996 & Kirkness, 1996). Generally, only the data which they collected were used and all analysis and calculations were repeated and checked before being used in this research. Wherever the undergraduate work has been used, due acknowledgement is made in this thesis.

4.5.2 Input data and Initialize

Prior to the start of the simulation program and for that matter any program, the necessary input data are required. The input data are not necessarily in the format as required by the program and need to be manipulated and changed. In TRACSIM it is also necessary to initialize the program, i.e. prepare and fill some of the data matrices with data before the executive routine can start. In TRACSIM three subroutines are employed for entering (*tracsim.m*), changing the input data to the required format (*adaptinp.m*) and initializing all the matrixes (*inimat.m*). Four matrices are used to store all data:

"A" : Stores all the data for vehicles arriving and queuing on the approaches.

"AA": Stores the data of all the vehicles in transit, moving from the yield line to the circulating carriageway.

"B" : stores all the input data for among others: geometric layout, gap acceptance characteristics, speed distributions and all the miscellaneous data.

"C" : Stores the data for all the circulating traffic.

Before executing the program all input is carried out in *tracsim.m* using a text editor (see section 6.2). Once this is completed, *tracsim.m* is then executed from within Matlab. *Tracsim.m* reads all the input, loads the traffic flows from another text file, initializes all matrices and calls *adaptinp.m* to change the data to the required formats after which the main executive routine is called (*aaatrac.m*). *Adaptinp.m* calls *inimat.m* to generate the first vehicles on each approach. *Inimat.m* in turn uses a number of minor subroutines to generate **random headways, turning movements, approach and circulating speeds, critical gaps, critical lags and move-up times** for each vehicle. *Inimat.m* is called every time another vehicle needs to be generated on an approach, which is whenever a vehicle leaves the yield line to enter the circle. *Tracsim.m* controls the number of times the model is run for the same set of input traffic flows and also how many sets of traffic flows to simulate. The Matlab code for *tracsim.m* (pp B-2 to B-3), *adaptinp.m* (page B-4) and *inimat.m* (page B-5) are included in Appendix B.

In TRACSIM vehicles are generated at the yield line and placed in vertical queues at the yield line if no acceptable gaps are available (see Section 4.3.1). Generating vehicles at the yield line negates the use of approach speeds because vehicle following theory is not used to move vehicles along the approach. However, TRACSIM allows for both methods of estimating delays using either vertical queues or horizontal queues. Whenever, for validation purposes, the estimated delays need to be compared with observed delays, the correction to the stopped headways can be applied (horizontal queues). Whenever the delay estimates are to be compared with other delay models estimating traffic delay, then the default method of headway generation can be used (vertical queues).

For the generation of headways between vehicles TRACSIM allows for the use of three arrival distributions: the negative exponential distribution, the shifted negative exponential distribution and the bunched exponential model (see Section 4.3.1 and equation (4-2)). Given a uniform random variate (z) between zero and one and the average arrival flow rate (q) the headway (h_w) can be calculated as follows by the three different theoretical distributions (Chung , 1993):

Negative Exponential distribution:

$$h_w = \frac{-\log_e(1-z)}{q} \tag{4-3}$$

Shifted Negative Exponential distribution:

$$h_w = \beta_1 - \frac{(1-\beta_1 q)\log_e(1-z)}{q} \tag{4-4}$$

Bunched Exponential distribution:

$$h_w = \beta_1 - \frac{\log_e(\frac{1-z}{\alpha_2})}{\lambda} \tag{4-5}$$

Turning movements observed at intersections are mostly deterministic although there can be a slight daily variation. Part of the required input to TRACSIM is the turning volumes from each approach. In *adaptinp.m* these turning volumes are converted to turning proportions from each approach. Once generated, a vehicle's destination is decided by simply generating another uniform random number between zero and one and comparing that with the turning proportions which are scaled from zero to one. From this comparison the appropriate destination or turning movement is determined.

Individual approach and circulating speeds are generated from user supplied mean speeds and standard deviations. Given a normal random deviate z_i with a mean of zero and standard deviation of one, a mean speed of v_m and a standard deviation of the speed s, an individual speed (v_i) is calculated as follows:

$$v_i = v_m + s z_i \tag{4-6}$$

Critical gaps, lags and move-up times are generated on the same basis as for speeds, except in some instances where the log-normal distribution is employed instead of the normal distribution for the generation of the critical gaps and lags (see Section 5.5).

4.5.3 Executive Routine

This routine was labelled *aaatrac.m* and forms the heart of the logic of the simulation model, as can be seen in Figure 4.4. From the generated headways on the different approaches to the circle, for a given set of data, *aaatrac.m* first finds the next event to happen, identifies what type of event it is and then systematically processes the event. It also updates the system and all other matrices when required while keeping track of all vehicles and their associated attributes (eg. speeds, gaps, and move-up times). To do this it employs a number of subroutines (*addcirc2.m, checkgap.m, delay.m graphc2.m, delaygrp.m, priority.m, upapp.m, upcirc.m, upcircex.m, upcircle.m, selectn2.m and transit.m*). The code for aaatrac.m is attached in Appendix B, pages B6 - B7.

4.5.4 Event Scheduler

The event scheduler in *selectn2.m* searches for the event which is due to take place next in time. It identifies both the type of event and the time of its occurrence. If there is more than one event due to take place at the same time, it records all of them, but it is left for *priority.m* to rank the events in order of importance. The importance of the events is defined in *adaptinp.m* as can be seen from the coding on page B4 in Appendix B.

4.5.5 Evaluating available gaps and lags

Every time an event results in a possible gap being available to a waiting vehicle on an approach, *checkgap.m* is used to evaluate the size of the lag or gap and to decide whether the lag/gap should be accepted. The decision is obviously based on the driver's critical gap or lag. This subroutine can make the decision based on three different gap acceptance criteria, i.e. times, positions or distances (see Section 4.3.2 and Chapter 7). Initially the program was developed around a fixed critical area. For validation, the model was changed to critical time gaps and later another model based on critical positions was added. A copy of the code for the gap acceptance subroutine is attached in Appendix B, pages B9 - B11.

4.5.6 Update vehicles on approach

Upapp.m is used every time a vehicle leaves the yield line to enter the circle. This routine is then used to update the specific approach from which the vehicle is leaving by deleting the vehicle from the approach, moving all other vehicles up in the queue and generating another vehicle to arrive on this approach. *Inimat.m* is used to generate the next vehicle on the approach. The code for this subroutine is summarized on page B12 of Appendix B. The queue lengths and delay incurred by each vehicle while waiting in a queue or at the yield line is recorded in this subroutine.

4.5.7 Update vehicles in circle

As the positions of the vehicles in the circle control the decision of vehicles on the approaches waiting to enter, these positions have to be updated every time a new event occurs. There is one exception to the updating of circulating vehicles and that is when the next event is a circulating vehicle exiting from the circle. Once a vehicle leaves the circle the event times of other circulating vehicles might change. For instance a fast vehicle catching up with a slow vehicle will have an event time for catching up. If the slow vehicle leaves the circle then the event time of catching up must change. Therefore a slightly different routine is required to deal with such events. *Upcirc.m* is the subroutine used to update the positions in the circle if the next event is not a vehicle leaving the circle, while *upcircex.m* updates the positions in the latter case (see page B13 in Appendix B).

In TRACSIM provision is made for both a constant speed rotation and a simplified vehicle-following model where:

i) Vehicles assume their own desired speed upon entry of the circle and will travel at this speed, if unopposed, until they exit.
ii) A fast circulating vehicle catching up with a slower vehicle will decelerate instantaneously and follow the slower vehicle while maintaining a minimum time headway.
iii) Once a leading slow vehicle leaves the circle the following faster vehicle will accelerate back to its initial desired speed.
iv) Fast circulating vehicles will also give way and reduce speed for entering vehicles pushing in and will follow these vehicles while maintaining a minimum time headway.

4.5.8 Move vehicle into transit once a gap is accepted

Once a vehicle accepts a gap it starts to move from the yield line to eventually become part of the circulating traffic. Because these vehicles are accelerating into the circulation lane and are not actually circulating, TRACSIM distinguishes between these "entering" vehicles and circulating vehicles which are already part of the circulating traffic. In the simulation model the vehicle and its attributes are moved from matrix "A" which contains all the vehicles on the approaches, to matrix "AA", which stores the data for all the vehicles in transit. The code for transit.m, which is the subroutine used for moving the vehicle from the approach to "AA", is included on pages B14 and B15 in Appendix B.

4.5.9 Add entering vehicle to circulating vehicles

Once an entering vehicle which has been on its way arrives in the circulating pathway and starts to circulate, a new event takes place. The new event times for this vehicle and all other circulating vehicles have to be calculated and speeds adjusted if necessary. *Addcirc2.m* was used to move the vehicle from matrix "AA" to matrix "C" and to update all the event times where necessary. The code for Addcirc2.m is included in Appendix B, pages B16 and B17.

4.5.10 Adapt speed of faster vehicles catching up with slow vehicles

Events 14, 15(see Section 4.4.3) describe faster circulating vehicles catching up with slow circulating vehicles or slow entering vehicles, while event 16 defines the event when a slowed down following vehicle accelerates back to its desired speed after the slow leading vehicle has left the circle. Upcircle.m deals with all three of these events, by first identifying which of the events 14, 15 or 16 is the next to take place and then the vehicle or vehicles are updated accordingly as can be seen from the code included on page B18. Once the vehicle attributes have been updated then the new event times for all the vehicles in the circle are calculated again.

4.6 Verification

Verification of a simulation program serves to establish the correctness of the program's logical structure and to eliminate errors, which may occur during the execution of the program. If required, the verification process can also suggest the range of the model parameter values for which the model still gives acceptable results.

The process of verifying the logical correctness of the program was obviously an integral part of the development process. The various techniques used to ensure a logically correct and error free program included the following:

i) The program was developed in modules (see Section 4.5) and as far as possible the modules were verified individually.

ii) The program was developed progressively. Initially the program was developed based on a number of simplifying assumptions which reduced the complexity of the model significantly. Two basic assumptions concerned the circulating model and the way vehicles entered the circle. Firstly, it was assumed that vehicles will be circulating at a constant speed, i.e. no vehicle-following models for the circulating traffic. Thus events 14, 15 and 16 (see Section 4.4.3) did not occur and were not catered for in the initial program. Secondly it was assumed that vehicles enter directly into the circle from the yield line which negated the used of matrix "AA" and the *transit.m* subroutine. The program could then be tested for errors and once it worked satisfactorily, the more complicated vehicle-following models and move-in model from the yield line to the circle could be included.

iii) A simple diagrammatic graphical display of the vehicles arriving, entering and circulating assisted in identifying logical errors. This graphic tool which can be run in real time and also step-by-step in a debugging mode, was probably the most useful in identifying and tracking of errors. The actual picture of the process, simplified and facilitated identification of errors and anomalies. Figure 4.5 shows an example of one frame of the graphics as produced by TRACSIM.

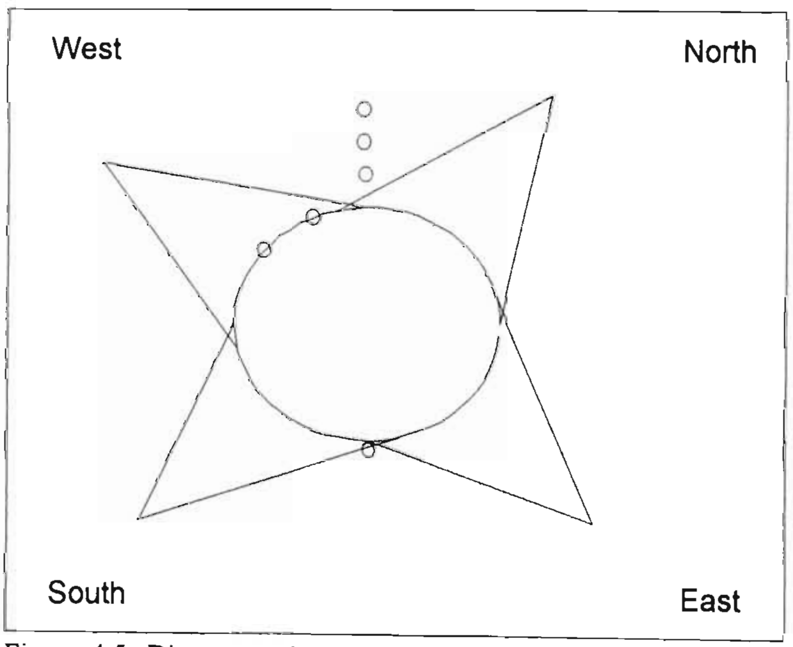


Figure 4.5: Diagrammatic representation of circle as produced by TRACSIM

Figure 4.5 shows the circle with four triangles around it, with two vehicles in the circle, three vehicles queuing on the western approach and one vehicle entering from the eastern approach. The triangles represent the entry and exit roads to the circle from each of the four approaches. As it is a simple radial plot of the data, the easiest way of showing queuing vehicles is on the radial intersecting with the point of entry and not on the actual entry road. The intersections of the entry and the exit roads with the circle are also shown as the actual positions where vehicles enter or leave the circle. The plot was displayed in colour, with the four approaches marked in four different colours and the circulating vehicles marked in a colour which corresponded with the colour of the approach where exiting is due.

iv) MATLAB has an efficient built-in debugging facility, which enables step-by-step checking of the program, and at any stage the value of all variables can be checked or changed. This debugging facility with the graphical display was used often to trace logical errors.

v) MATLAB's inherent character of using matrices as basic elements often revealed logical errors. Being sensitive to the matrix dimensions, matrix multiplication often showed errors when for some reason the matrix dimensions were not as expected.

vi) The program was tested with observed volumes as input values. The simulated output in terms of stopped or queuing delay was then compared with the observed values to ensure that the results were at least approximately similar, although at this stage none of the program parameters had been calibrated.

vii) Error statements in the program are used to warn for inconsistencies such as headways between circulating vehicles being less than the minimum value.

viii) A sensitivity analysis was conducted primarily to establish the sensitivity of the model to the different input parameters. However, the sensitivity analysis also revealed a few aspects of the program which could be more robust. The sensitivity analysis is discussed in detail in Section 4.7.

4.7 Sensitivity Analysis

The estimation of model parameters (see Chapter 5) can be a lengthy and complicated process, especially if equal time is spent on estimating every parameter. It was therefore decided to first assess the sensitivity of the model output to a change in the input parameters. The estimation of the parameters which have a major effect on the model output should then receive more attention than those parameters to which the simulation model is less sensitive. The following input parameters were all included in the sensitivity analysis:

i) Headway distribution on approaches for vehicle arrivals.
ii) Move-up times for vehicles in queues.
iii) Critical gaps for vehicles accepting gaps in circulating flow
iv) Circulating speeds
v) Minimum headways for circulating vehicles

For the sensitivity runs a four-legged circle with Central Island Diameter of 36,0 metres, with single circulating and approach lanes, was used. This is similar to the geometric layout of the Chatsworth circle and was used because during the initial development of the simulation program this circle and its traffic data were used as a reference. Vehicular flows as shown in the Table 4.1 were entered as input to the program. The program was run five times for each of the five sets of vehicular flows for 30 minutes each time of which only the last fifteen minutes was used for capturing data. Five times was used because it gave an adequate spread of data points. Each time a different set of pseudo-random numbers were used to give in effect, twenty five sets of data for each approach or 100 sets of data for the whole circle. The flow from each approach was split into three for the three turning movements - left, through and right. Thus, the flows and turning movements from all approaches were the same. For instance, for the first set, the turning movements from north are 50 left, 50 through, and 50 right.

Table 4.1: Vehicular flows (veh/h) used for sensitivity analysis

Set	North	East	West	South	Total
1	150	150	150	150	600
2	300	300	300	300	1200
3	450	450	450	450	1800
4	600	600	600	600	2400
5	750	750	750	750	3000

The input flows as shown in Table 4.1 were selected to represent as wide a range of flows as possible. The largest total flow of 3000 vehicles per hour is more or less the capacity of the circle. However, although the parameters were carefully selected, none of them had been calibrated at this stage and from the sensitivity model runs it is not obvious what the real capacity of the circle is. The output from the model in terms of delay per vehicle (seconds per vehicle) was used for the comparison.

Due to the randomness of the simulation model there is considerable variation in output for the same set of input values when using a different set of random numbers for the various simulation runs. This variation in the output complicates the comparison of the different input scenarios, consequently an exponential curve was fitted through each data set. This single line representing the output from a simulation run, simplified the comparison with the other sets of output data.

4.7.1 Headway distribution

The estimation of headways between vehicle arrivals is not only essential to the modelling process, but is also the first parameter to estimate for every vehicle in the simulation process. There are various frequency distributions which could be employed to generate theoretical headways (See Sections 4.3.1 and 4.5.2). TRACSIM allows for three different arrival distributions: the negative exponential, shifted negative exponential and bunched negative exponential distributions. Akcelik and Chung (1994a) showed that the bunched exponential distribution is more realistic than the other two distributions and they recommended its general use. They calibrated the bunched exponential distribution and for a single lane traffic stream recommend a bunching factor (b) of 0,6 and a minimum arrival headway of 1,5 seconds (See (4-2)). The sensitivity of the simulation model for a change in arrival headway distribution, together with the effect of different sets of random numbers is shown in Figure 4.6.

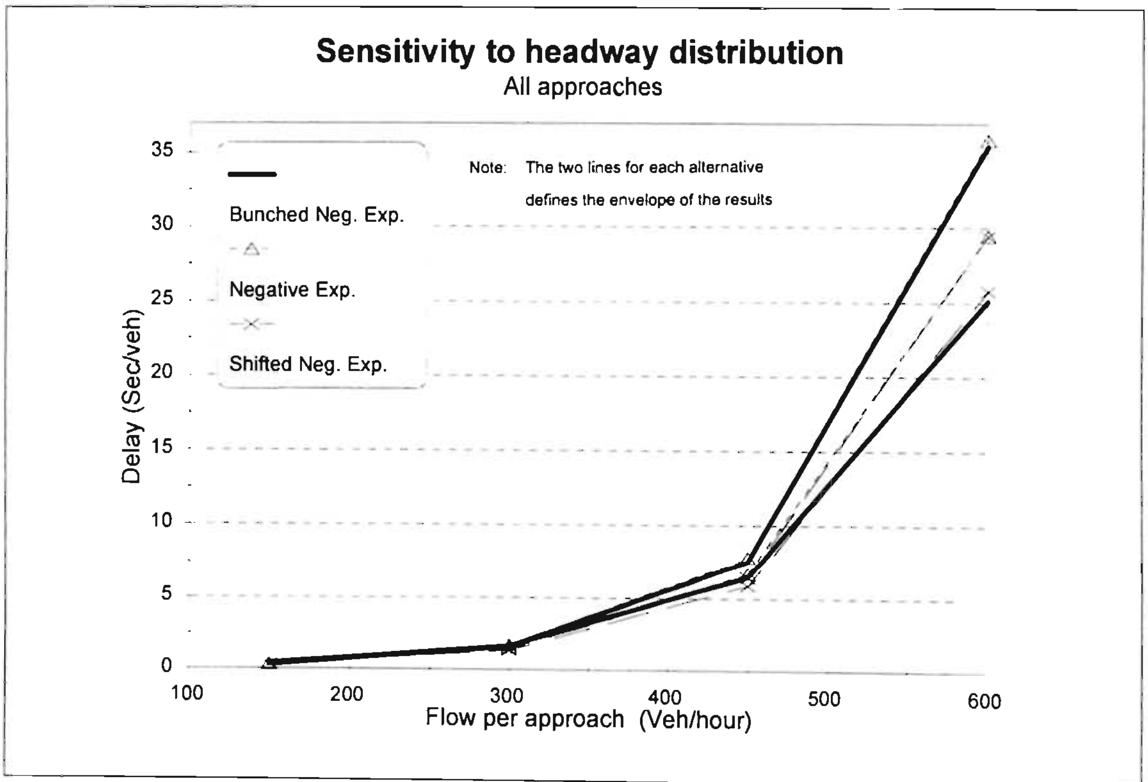


Figure 4.6: Sensitivity of model output to headway distribution

The model was run five times for each arrival distribution, each time with a different set of random numbers. The two lines in Figure 4.6 for each arrival distribution represent the envelope of the results of the five runs. Figure 4.6 only shows values on the x-axis up to 600 vehicles per hour as the 750 input values were close to the approach capacities with associated high delays and when the graph includes the 750 results, the details of the lower flows disappear because of the change in scale of the y-axis. Although there is a wider spread in delays, at 750 vehicles per hour for the different arrival distributions the pattern remains the same.

From Figure 4.6 it can be seen that the simulation model is relatively insensitive to changes in arrival distribution and that it is more sensitive to changes as a result of randomness than it is to changes in the arrival headway distributions. Therefore, pursuing greater accuracy in terms of arrival, headway distributions would not be beneficial. As Akcelik and Chung (1994a) have studied arrival headways in detail and because of the insensitivity of the model to different arrival headway distributions it was decided to use the bunched exponential distribution in the model.

Two important input parameters to the bunched exponential distribution are the bunching factor and the minimum headway of vehicles in platoons (See (4-2)). The sensitivity of the simulation model to both these parameters was subsequently investigated. Once again, a number of simulation runs with different random number seeds have been used to generate an envelope of results for each set of variables. Figure 4.7 shows these envelopes for three different values of the bunching factor; 0,6 which is the value recommended by Açkelik and Chung (1994a) and two further values of 0,4 and 0,8.

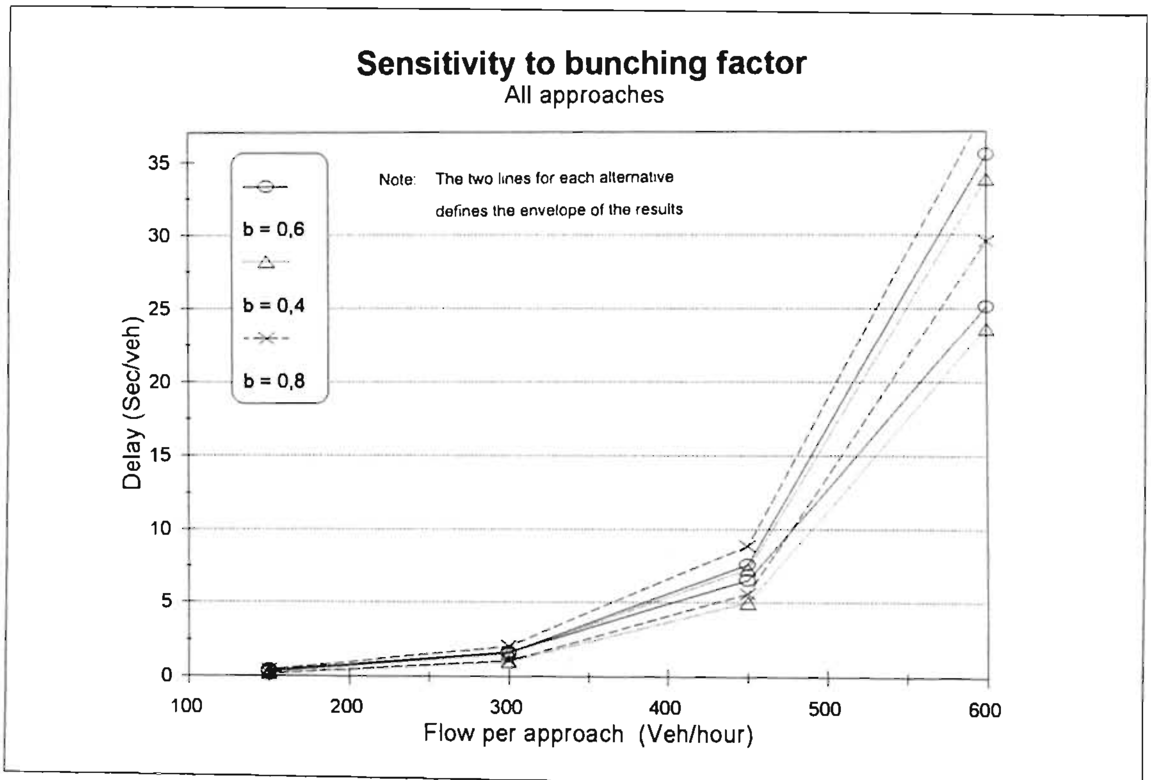


Figure 4.7: Sensitivity of model output to the bunching factor of the bunched negative exponential arrival headway distribution

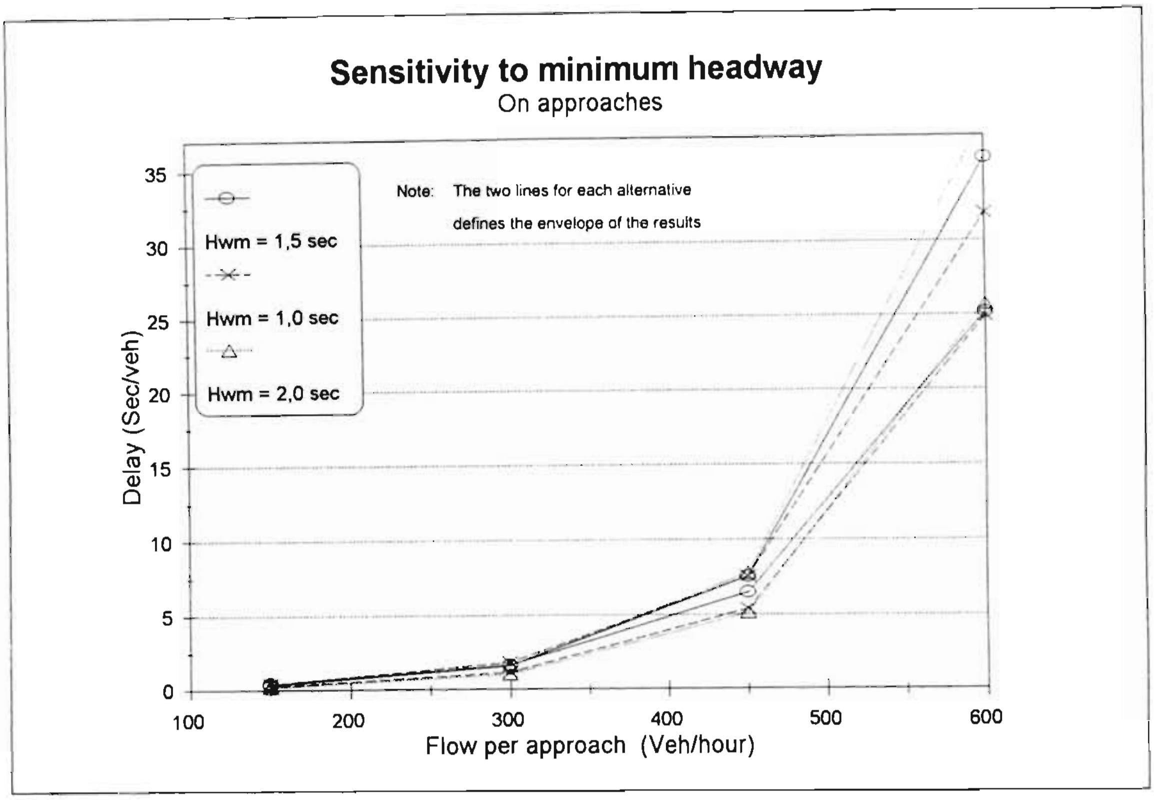


Figure 4.8: Sensitivity of model output to the minimum headway used in the bunched negative exponential arrival headway distribution

From Figure 4.7 it is obvious that the model is relatively insensitive to changes in the bunching factor, and that the random effect probably plays a more important part. From this it is concluded that a bunching factor of 0,6 as recommended by Akcelik and Chung (1994) can be used without a detailed investigation to determine a more accurate value.

On the same basis as in the previous two figures, Figure 4.8 shows the insensitivity of the simulation model to the minimum headway (H_{wm}) which was used in the bunched exponential arrival headway distribution. Based on this it is advisable to use the minimum headway of 1,5 seconds as recommended by Akcelik and Chung (1994a).

4.7.2 Move-up times

The sensitivity of the simulation model to changes in the average move-up times was tested using a number of initial measurements of move-up times. These observations showed an average move-up time of 2,6 seconds with a standard deviation of 0,5 seconds. For the sensitivities the model was run for these observed values and also for averages of 1,5 seconds (- 2 STD's) and 3,5 seconds (+ 2,5 seconds). The model was run four times for each average move-up time, each time with a different set of random numbers. Again, each pair of lines in Figure 4.9 for each move-up time, represents the envelope of the results of the four runs.

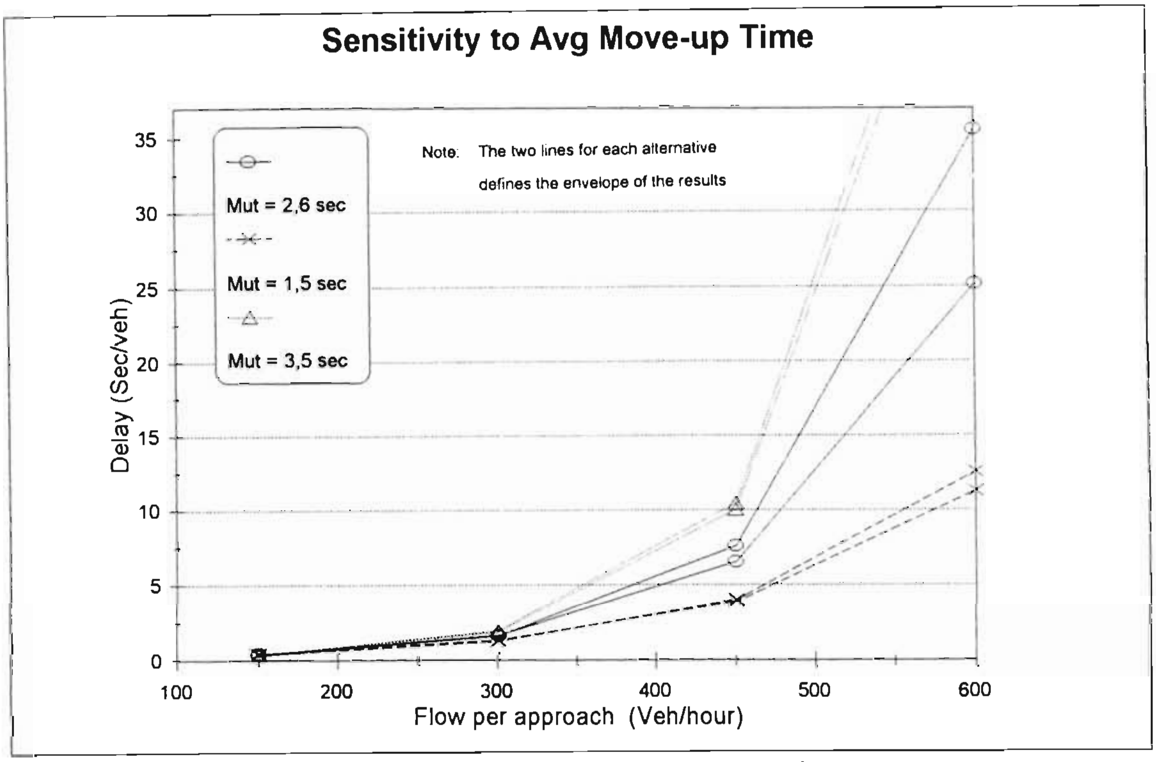


Figure 4.9: Sensitivity of simulation model to changes in average move-up times

From Figure 4.9 the model is clearly quite sensitive to the average move-up times. A short move-up time would allow more vehicles to accept the same gap and vice versa for a long move-up time.

4.7.3 Critical gaps

The capacity (and the delays) of side road vehicles at any priority intersection including traffic circles is based primarily on the critical gap which drivers are prepared to accept. Hence it is expected that any model attempting to predict performance of such an intersection should be sensitive to changes in the critical gap. This expected sensitivity of the simulation model to a change in the critical gap was confirmed, but is not reported in detail in this section. A major part of this study focuses on the aspect of critical gaps and the process of gap acceptance at traffic circles and is discussed in detail in Chapter 7.

4.7.4 Circulating speeds

At the Chatsworth circle (36-metre central island diameter) an average speed of 35 km/h and a standard deviation of 6 km/h was measured. The sensitivity runs were based on a circle with a size similar to the Chatsworth circle. The sensitivity of the model was tested in the same manner as for the other variables by varying the average speed with approximately one standard deviation to 29 km/h and 41 km/h. The result of this is shown in Figure 4.10.

Similar to move-up times, the model also shows a sensitivity to changes in the circulating speed, highlighting the need of careful calibration of this variable.

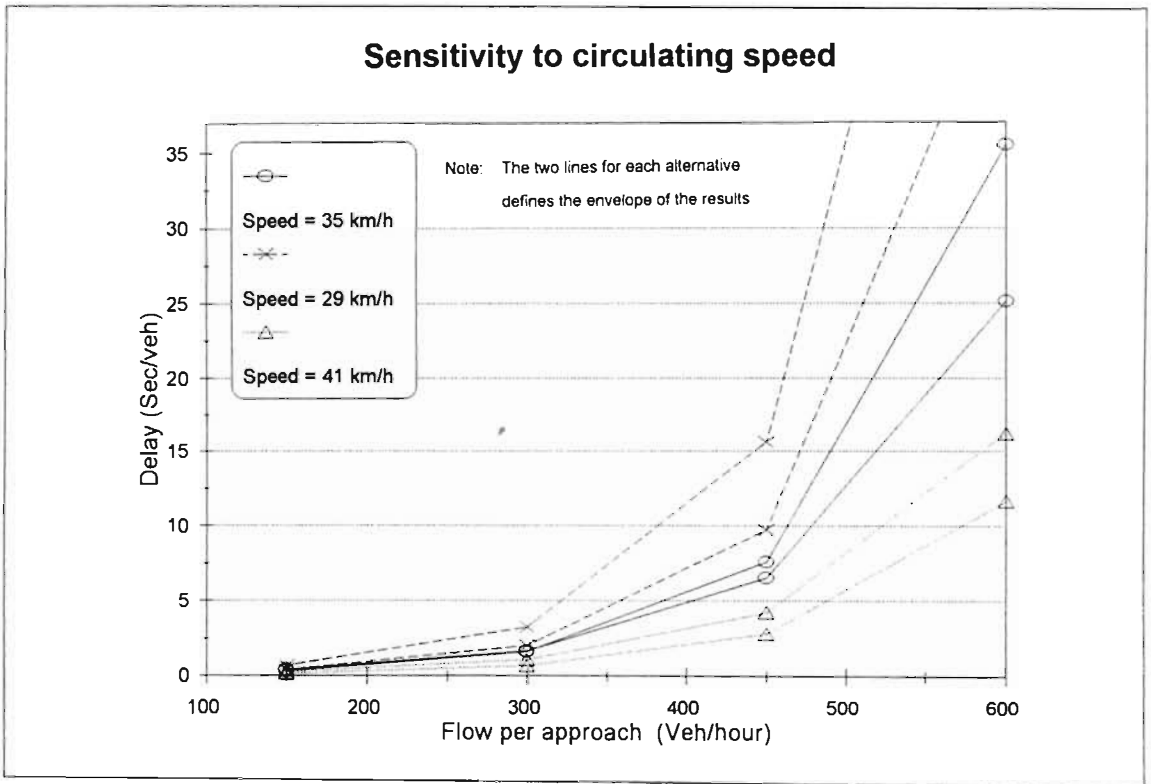


Figure 4.10: Sensitivity of simulation model to changes in circulating speed

4.7.5 Minimum headways in circle

Figure 4.11 shows the sensitivity of the model to changes in the minimum headway for circulating vehicles. The model has been tested with a minimum headway of 1 second, 0,7 seconds and 1,5 seconds. The three envelopes containing the simulated delays for the three different minimum headways are similar, suggesting the insensitivity of the model to the minimum headway for circulating vehicles.

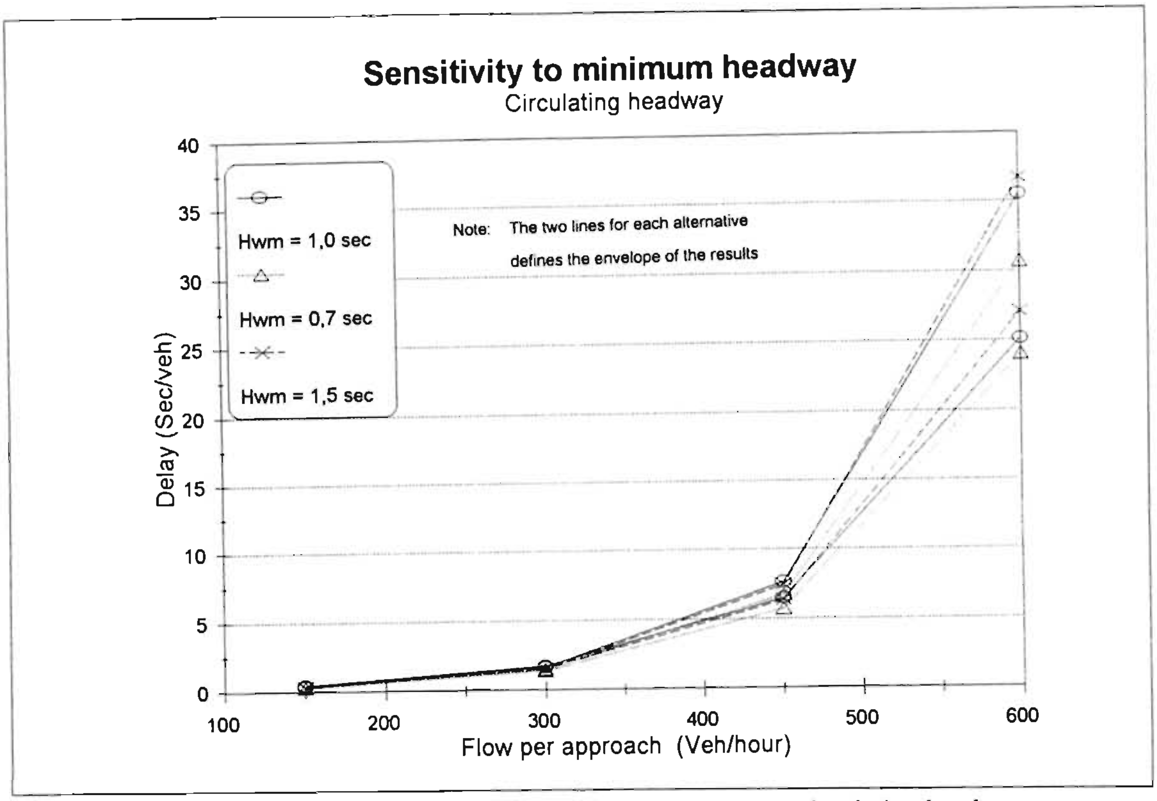


Figure 4.11: Sensitivity of simulation model to changes in minimum circulating headway

4.8 Summary

Having established the need for a simulation program in Chapters 2 and 3, the development process and the details of each stage of the development process of the traffic circle simulation program, TRACSIM are discussed in this chapter. The flow diagram in Figure 4.1 shows the general process which was used to develop the simulation program.

Working from a problem definition, objectives were defined and criteria specified for the simulation model, which has to simulate a system as analysed in terms of the various sub-models (approaching, queuing, entering, circulating and exiting). Employing a fifth generation language (MATLAB), an event scanning updating procedure and Wichmann and Hill's (1982) uniform random number generator, a software program is developed based on a modular structure. Various subroutines are used for the different modules or processes taking place, such as inputting and initializing of the data, finding the next event, updating vehicles on the approaches, updating vehicles in the circle, checking if a gap is acceptable and moving vehicles from the yield line into the circle.

In this chapter a description is also given of how the program was verified to be free of errors, by using among others a progressive approach, a graphical display of the simulation process and a sensitivity analysis. However, the main reason for the sensitivity analysis was to show the sensitivity of the model to changes in the various input variables. This served as an indication of the effort needed to calibrate the different input variables. Those variables to which the model is insensitive do not need as much attention during the calibration process than the variables to which the model is sensitive.

The sensitivity analysis indicates that the simulation model is quite sensitive to changes in move-up times, critical gaps and circulating speeds. On the other hand it was established that it is insensitive to the arrival headway distribution and its associated bunching factor and minimum headway. It is also not sensitive to the minimum circulating headway.

The calibration process and the collection of the data required for the calibration of the various input variables to the simulation program are discussed in Chapter 5.

CHAPTER 5: DATA COLLECTION AND PARAMETER CALIBRATION

Various data were required not only to calibrate the input parameters to TRACSIM, but also to validate the operation of the model and to identify the geometric input values to the program. In this chapter the data needs in terms of input data, calibration data and validation data are discussed. Furthermore, the selection of appropriate circles for the surveys, the different types of surveys (delay -, volume -, move-up time -, speed -, critical gap - and headway surveys) conducted at the different circles, the analysis of the survey data and where necessary, a comparison of the data from the different circles are discussed.

5.1 Data Needs

This research was initiated by an interest in the operation of traffic circles specifically under Southern African conditions. The initial effort was to establish the relevancy of a few of the available foreign delay and capacity models to local conditions (see Chapter 3). Having shown that there is reasonable doubt as to the relevancy of these models to the South African conditions, the next step was to improve on these models and for that purpose local data were needed.

In South Africa there are relatively few traffic circles operating under medium (+700 vph) to high (+1000) approach volumes and therefore it was decided to develop a simulation program, which if calibrated for local conditions, can be used as a research tool. Chung (1993) classified the data requirements for the development of a simulation program into two categories, i.e. geometric and operational data. He then classifies the operational data as input data, calibration data and validation data. For this research the data requirements have been classified as follows:

i) Input data - Traffic and turning volumes, traffic composition and geometric data.
ii) Parameter Estimation - Approach headway distribution, gap acceptance data, move-up times and speeds (approach and circulating).
iii) Validation - Average queuing delay and queue length.

5.1.1 Input data

Basic input to most traffic engineering models and also to TRACSIM, is the geometric layout of the facility being modelled, and the traffic and turning volumes of traffic moving through the system. To convert the arrival volumes (vehicles per hour) to passenger cars (pcu per hour), the traffic composition is required. The geometric data required by the simulation program are the approach and circulating lane widths, the central island diameter, the number of entrances and exits, the positions of the entrances and the exits, and the distance from the yield line to the circle at each approach.

5.1.2 Calibration data

There are a number of parameters and processes which need to be calibrated in a simulation program, and for TRACSIM these are:

i) Approach headways. The distribution of headways on the approaches is necessary to generate vehicles on the approaches.

ii) Gap Acceptance. Critical gap and lag distributions for the gap acceptance process. The gap acceptance process is an integral part of the simulation model and to model it accurately the underlying distribution of gaps which drivers are prepared to accept is essential information. Whether a distribution with a fixed mean should be used or whether to vary the mean critical gap according to the conflicting circulating flow is another issue to address, as well as whether the gap acceptance process should be based on time or distance gaps. These issues are discussed in more detail in Chapter 6.

iii) Move-up times. Drivers of entering vehicles consider available gaps only at the yield line. As it takes time to move-up in the queue from a position second in the queue to a position at the yield line, this move-up time is essential to the simulation process.

iv) Speeds. The distribution of circulating speeds around the circle is required to model the following of vehicles around the circle. If the model is using constant circulating speeds then only the mean circulating speed is required. In the case of TRACSIM, vehicles are generated at the yield line, hence no requirement for approach speeds.

5.1.3 Validation data

Most traffic engineering analysis models (Akcelik & Besley, 1992) or simulation models (Chung, 1993) attempt to estimate the operational performance of the traffic facility being analysed or simulated. One of the operational measures of effectiveness (MoE) which is a good indicator of the operational performance of a traffic facility is the delay traffic experiences at the intersection. This delay is usually expressed as either a mean total delay per vehicle or a mean stopped delay per vehicle (see Section 2.3.3). For validation of TRACSIM, "delay" was used the criterion and specifically stopped delay was used for validating the simulation program against actual observations.

5.2 Selection of appropriate circles

Once the data needs were established, the next step was to identify appropriate circles which could be used for data capturing. The following criteria were used to appraise circles as possible data sources:

i) Geometric: Single lane approaches and single circulating lanes. Preferably a circular central island with four equally spaced and level approaches, although this was not a rigorous requirement.

ii) Traffic: Volumes sufficient to result in queuing on at least one of the approaches. Random arrivals of traffic on the approaches, i.e. no traffic control devices immediately upstream of the approach which would cause significant platooning.

The following four circles all located in the Durban Metropolitan Area were identified for data recording:

i) Chatsworth Circle
ii) Kensington Circle
iii) Pinetown Circle
iv) Queen Mary Circle

Figure 5.1 shows the location of these circles in the Durban Metropolitan Area. The Chatsworth and Pinetown circles have also been used in Chapter 3 to evaluate the performance of Arcady and SIDRA. Level land is not common in the steep terrain of Durban and hence most of the circles have slight slopes on some or all approaches. The geometric layout of the Queen Mary and Kensington circles are shown in Figures 5.2 and 5.3 respectively while Figures 3.1 and 3.4 show the geometric layout of the Chatsworth and Pinetown circles respectively.

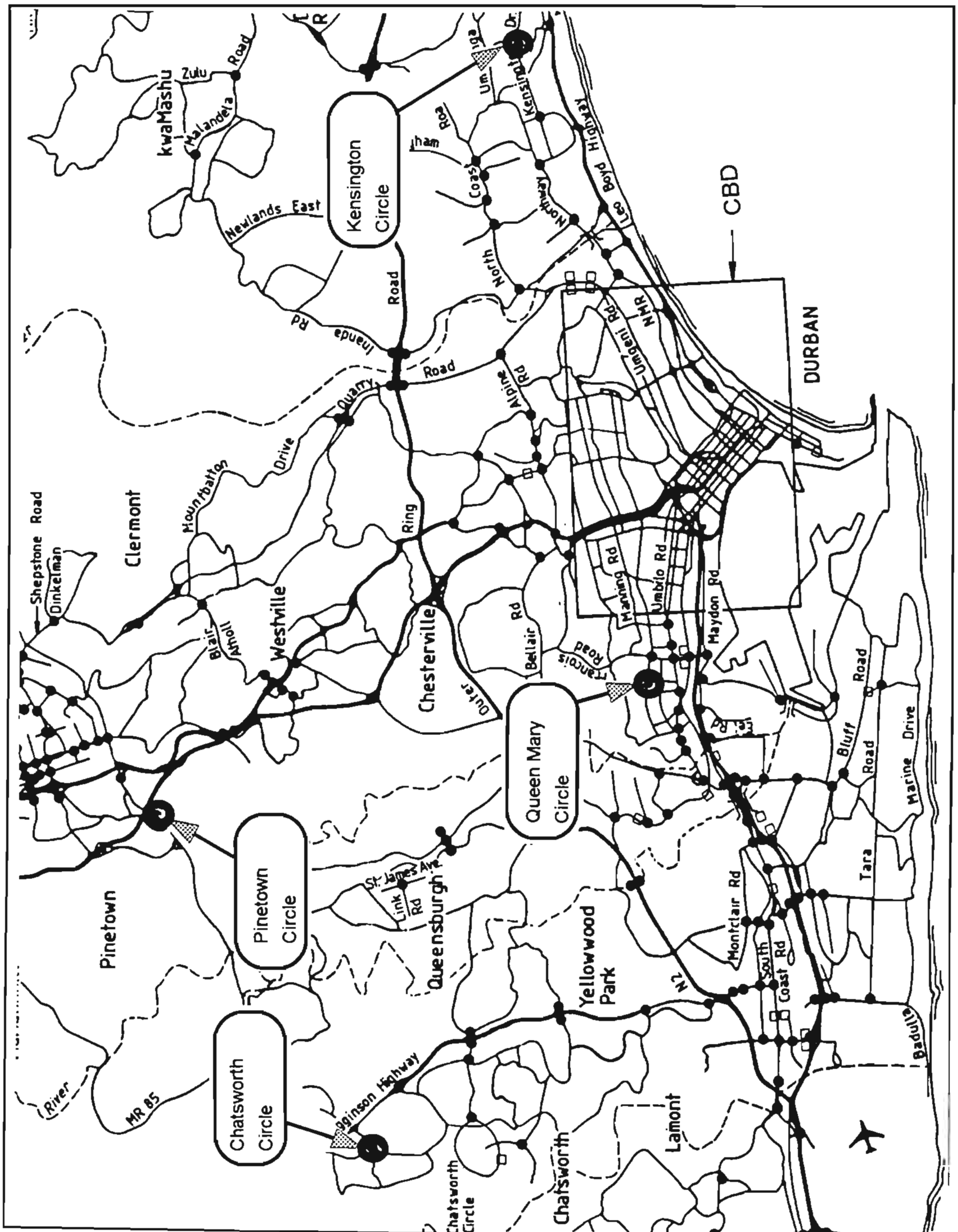


Figure 5.1: Location of the four circles in the Durban Metropolitan Area

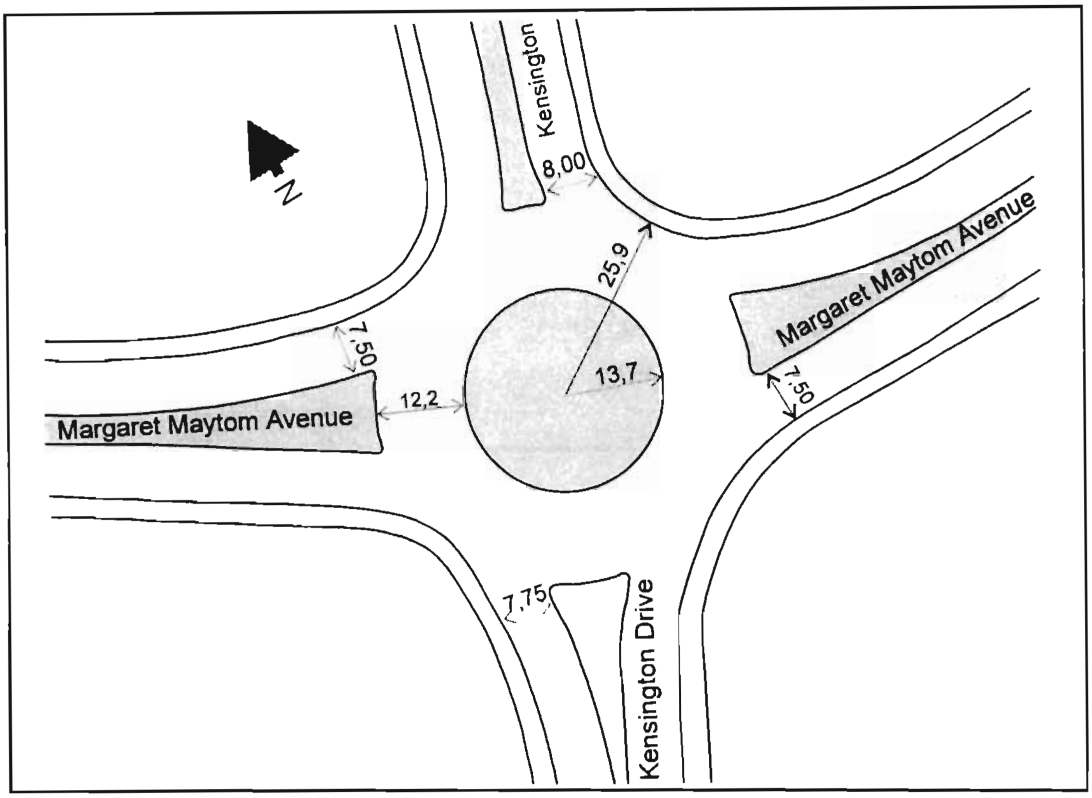


Figure 5.2: Geometric layout of Kensington Circle

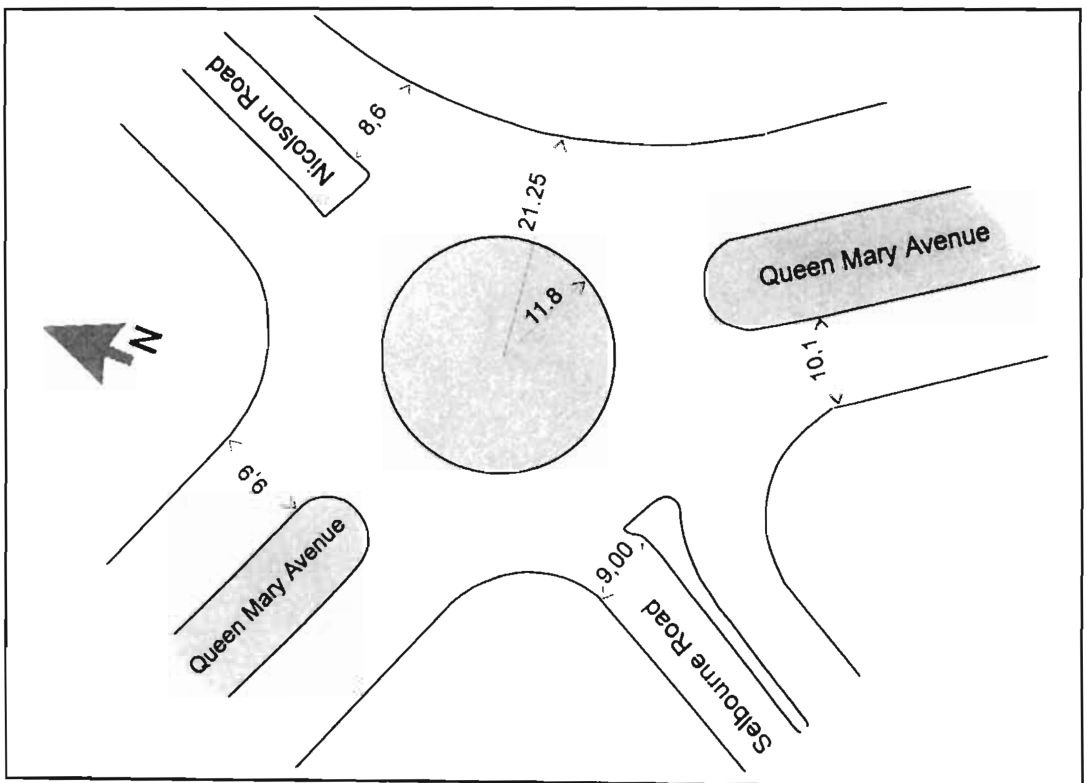


Figure 5.3: Geometric layout of Queen Mary Circle

A summary of the most prominent geometric features of each circle is given in Table 5.1.

Table 5.1: Summary of circle geometrics

Circle	Geometry					
	No of entrances	No of exits	No of effect. circulating lanes	Central Island Diameter	Circulating Lane width	Inscribed Diameter
Chatsworth	4	4	1	36.2	6.9	50.0
Kensington	4	4	1	27.4	12.2	51.8
Pinetown	4	3	1	20.0	6.8	33.6
Queen Mary	4	4	1	23.6	9.5	42.5

5.3 Surveys

To obtain the data needs as listed in Section 5.1 a large number of traffic surveys had to be carried out at the circles listed in Section 5.2. These surveys included among others: delay surveys, gap acceptance surveys, speed surveys and volume surveys. These surveys were carried out by using video recordings. No software was available for the analyses of these video recordings and all data extraction had to be completed manually. However, it simplified the survey procedures significantly and a trained data extractor could be used repeatedly for different approaches or different tasks.

The video recordings were made from vantage points wherever possible and where not, a suitable scaffolding platform was built. One major advantage of the video recordings is that once recorded, the tape can be played repeatedly to obtain different sets of data and errors can be corrected later if detected. However, a major disadvantage was the picture size that could only be increased with a loss in definition. In other words if all queues on all approaches were to be included in the full extent of the picture, it had to be zoomed out to such an extent that the detail of what was happening at the circle itself was lost. To overcome this, either two cameras were used, one for a wide picture including all queues and the other zoomed in on the circle, or queue length recordings were made manually during the time of the video recording.

Table 5.2 summarizes the types of surveys which were conducted, at which circles, and when they were carried out.

Table 5.2: Circles, survey dates/times and types of surveys

Circle	Date	Time	Type of survey				
			Delays	Gaps	Move-ups	Speeds	Volumes
Chatsworth	30/07/93	06:30 - 08:15	X			X	X
	16/08/93	15:30 - 17:30	X	X	X	X	X
	17/08/93	15:45 - 17:30	X	X	X		X
	18/08/93	06:30 - 08:00	X				X
Kensington	30/07/96	07:00 - 08:00		X		X	
	31/07/96	07:00 - 08:00		X			
Pinetown	15/08/95	06:30 - 08:15	X	X	X	X	X
	15/08/95	16:30 - 17:45	X	X			X
Queen Mary	25/07/96	07:15 - 08:00	X	X	X	X	X
	26/07/96	07:00 - 08:00	X				X

5.3.1 Delay surveys

A number of different methods are available for the measurement of delays in the field. Cohen and Reilley (1978) classified these methods into three basic approaches:

i) *Point Sample Method.* Also known as the Queue Sampling Method, it is based on sampling the length of the waiting queue of vehicles at regular intervals of 10 to 15 seconds and is recommended in the HCM (1985) for measuring delay at signalised intersections. Two advantages of this method are that it is self correcting (a sampling error is not transferred to subsequent samples) and that it is independent of the type of traffic facility or traffic signal indications (Van As and Joubert, 1993). Disadvantages are that the accuracy is reduced if the queue lengths become long and that it only measures stopped delay and not total delay.

ii) *Input-Output Method.* This method is based on observing the input to (joining back of the queue) and output (leaving front of the queue) from a system in a time interval. The difference between the input and output is then the delayed number of vehicles. Corrections are necessary for vehicles leaving the system between the two observation points and also for observer's error which might result in a mismatch of input and output data.

iii) *Path-trace Method.* In this method individual vehicles are traced through the system from time of entry to time of exit. Although this method seems simple enough to execute, a large number of observers is required to make an accurate survey.

For this research the queue sampling method was used for measuring field delay for the following reasons:

i) The queues on the approaches to the circles could be counted reasonably accurately because they seldom were very long and because they were in one lane only.

ii) Stopped delay or queuing delay was needed to validate the simulation program. Other models also have stopped delay as output (Chung, 1993).

iii) The method is simple and queue lengths can be extracted from video material or live at the intersection with a small number of trained observers.

The "Manual of Traffic Engineering Studies" (ITE, 1976) and the HCM (1994) describe in detail the application of the queue sampling method. For the field studies of queue lengths, one observer per approach was used, while a single observer was used to do all the approaches whenever the data were extracted from a video tape. An interval of 10 seconds was used for the sampling of the queue lengths and a vehicle was regarded as being part of the queue when it was moving slower than walking speed at the end of the queue. For the studies done in the field a data sheet on a clipboard and a watch, synchronised with those of the other observers, were used to enter the queue lengths at the specified time intervals. The data were then transferred to a computer spreadsheet at a later stage to simplify analysis. For the queue length observations from the video tapes a macro was written for the spreadsheet Quatro Pro. The observer was prompted for the queue length as input and once a number was entered it would enter the spreadsheet in the column next to the next time interval and prompt the user for the next queue length. This proved to work well with a monitor and VCR placed adjacent to a computer and the observer controlling both. Since the actual time during the survey being recorded was displayed on the video screen, all the observer had to do was to watch the time and every ten seconds note the number of vehicles in the queue and enter it on the computer keyboard.

The total stopped delay per approach was found by multiplying the total number of queued vehicles over the entire survey period with the sampling interval. The average stopped delay per vehicle was found by dividing the total stopped delay by the total number of vehicles which have moved through the approach. These queue length observations or delay studies were completed at all the circles as shown in Table 5.2 for the full length of the survey period.

According to Van As and Joubert (1993) "there is a definite tendency to overestimate stopped delay and the percentage of vehicles actually stopped. Observers tend to concentrate more on the upstream end of a queue and thus overestimate queue lengths." Personal experience of the author is that especially under short queue length situations and marginal decisions there is the temptation to rather include than exclude vehicles in the queue length count. According to research by Cohen and Reilly (1978) this error could be as large as 8% and hence they recommend that observed stopped delay be multiplied by a factor of 0,92 to obtain true stopped delay.

Stopped delay is only the time which vehicles spend stationary or in a queue and excludes the delay as a result of deceleration and acceleration. Van As and Joubert (1993) recommends that true stopped delay be increased by 30% to obtain total delay. Although extensive research has resulted in these correction (0,92) and conversion (1,30) factors, there is a reluctancy to take a number and multiply it with carefully observed results. There are always questions such as whether the surveys were not done more accurately with better trained and more experienced observers, and if the absence of long queues for long periods of time did not result in an improvement of the accuracy. To negate the increase of stopped delay to total delay it was decided to use only stopped delay during the calibration and validation of the simulation program. As time and resource constraints prevented the verification of the correction factor for observers' error, it was used. However, during the validation process reference was also made to the original field results.

5.3.2 Gap acceptance surveys

Because the gap-acceptance process is an essential part of the operation at any priority controlled intersection, a number of surveys were conducted at all the circles shown in Table 5.2. These surveys were all made from the video recordings of the traffic operation at the circles and since data extraction was such a time consuming and exacting process, a number of observers were used. All that was required for these surveys was a stopwatch to measure the time lags and gaps and the times, once measured, were entered directly into a spreadsheet program.

During the course of this research a number of theories on gap acceptance at circles were postulated and to test them the data collection process had to allow for the correct data to be abstracted. Traditionally gap acceptance models at priority intersections are based on time gaps but from observation it seemed that a distance gap or a position in the circle could be more appropriate in a model and it is also a more realistic representation of what drivers do in practice. It was also apparent that there could be a difference in the gap acceptance decision depending on whether the next conflicting vehicle is circulating or entering from the entrance just upstream of the entrance under consideration as shown in Figure 5.4. Another distinction was made between lags and gaps where lags are partial gaps or the remainder of a gap from the time a vehicle arrives at the stop line to the arrival of a conflicting vehicle.

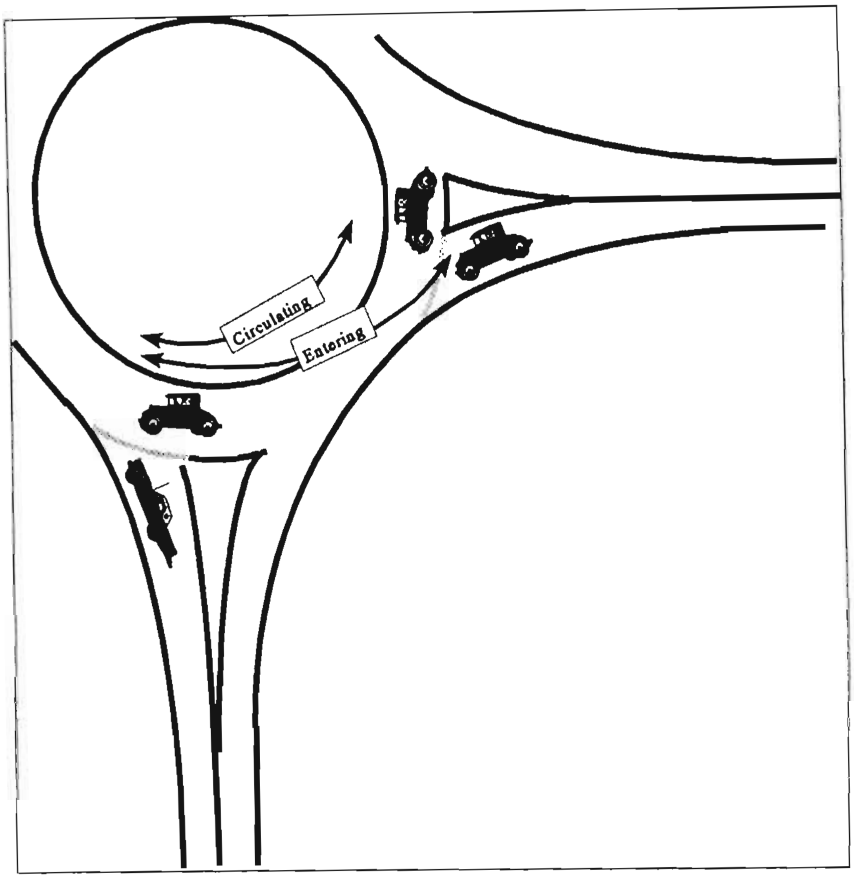


Figure 5.4: Circulating and entering gaps

An observer first recorded the lag: whether it was an entering or circulating lag, whether it was accepted or not, as well as the position of the conflicting vehicle at the time of the arrival of the minor vehicle at the stop line, i.e. at the start of the lag. Once the lag was rejected, the subsequent gaps were recorded in the same manner until a gap was accepted.

5.3.3 Move-up time surveys.

The time for vehicles to move up in the queue was measured at three of the four circles. These surveys were also conducted using the video recordings. Instead of using a stopwatch to measure the move-up times and then transcribing it to paper or directly to a spreadsheet, an event recorder was used. The event recorder is a software program which records the exact times the function keys on the keyboard of a personal computer are pressed and also which key is pressed. (It would define, for instance "F1" as the first vehicles leaving the yield line and "F2" up to "F8" as subsequent vehicles reaching the yield line) The event recorder saves the data in a text file which can be retrieved into a spreadsheet program for analysis and calculation of the move-up times. These are derived from the time difference recorded between the different function keys. "F1" would always indicate the start of a time measurement or a series of measurements if there was a queue of vehicles departing.

5.3.4 Speed surveys

For both the simulation program and for the vehicle following model for circulating traffic the average circulating speed was not only needed but also the standard deviation of the speeds. Again, the video taped recordings were used for speed measurements around the four different circles listed in Table 5.2.

With the help of a scaled drawing of each circle and the available video image, common reference points could be identified on both the drawing and the video image. The video recording could then be used to measure the time it took a vehicle to move between the reference points, while the actual distance covered could be obtained from the scaled drawing. These points were selected so that the possible parallax error was as small as possible. To keep the observers' error to a minimum the distance between the reference points was taken as large as possible and greater than what is recommended in the Manual for Traffic Engineering Studies (ITE, 1976). However, this was not always possible, especially for vehicles making a through manoeuvre at the circle.

The same event recorder as was used for the move-up time measurements was used for the speed recordings. By assigning different events (vehicles passing specific reference points) to specific keys, the exact times could be recorded when vehicles passed these reference points and were recorded directly as electronic data. This data could then be analysed to obtain the speed distributions.

5.3.5 Volume Surveys (Approach and Turning)

Of all turning volume surveys at traffic control facilities the ones at traffic circles are possibly the most difficult to do. At most other control facilities the turning movement of the vehicle is almost immediately clear once it leaves the stop/yield line on the approach and can be recorded immediately, leaving the observer free to observe the next vehicle. At circles however, the turning movement, except possibly for a left turning movement, is not immediately clear and the observer has to "follow" the vehicle until it exits the circle. If this is a right turning vehicle then one or more vehicles following it could have entered the circle in the meantime and turned left or gone straight through without being recorded by the observer. The larger the circle, the more difficult this becomes. At mini circles this is not a such a problem, because of the relatively short time vehicles spend in circle

A number of methods can be used to obtain a turning movement count:

i) An impracticable method would be to have enough observers on an approach to allow all vehicles to be followed and their destinations recorded.
ii) An origin-destination survey could be conducted by recording all, or a sample of, registration numbers at entries and exits.
iii) The turning movements could be sampled randomly and entry and exit counts could then be used to extrapolate the sample to the population (Mountain, et al, 1986).
iv) Video tape recordings could be used to follow all vehicles.

Since the obtaining of an accurate turning movement count was necessary, the use of video recording seemed to be the only practical method. It did however mean that if a right turning vehicle had to be followed and a number of unrecorded vehicles slipped through behind it, the observer had to rewind and record those as well. The VCR was equipped with a sophisticated remote control which allowed various speeds of forwarding and rewinding by turning a control knob either clock wise or anti-clock wise. When there were no vehicles present it could also be forwarded to the next vehicle's arrival. The turning movements were counted and summarised every minute. With the actual time of the recording displayed on the video monitor this was easy to do despite the amount of fast forwarding and rewinding.

To assist in the data capturing process a macro program was once again written for the spreadsheet Quatro Pro. This macro program reacted to designated keys on the keyboard. These keys represented for example: passenger car turning left, passenger car turning right, heavy vehicle turning right, etc. Each time a designated key was pressed particular letters were entered into the next available cell in the column. For instance "cl" for "car left" and "ht" for "heavy vehicle through". At the end of the desired counting interval (usually one minute) another "hot key" summarized the input of the last minute by counting all the vehicles and movements entered in the first column (all the "cl's" and "ht's" etc.), and entered it into another appropriate column in the spreadsheet. The vehicles were classified as passenger cars, buses, heavy vehicles and combi-taxis. The one minute counts were then summarized into five minute and fifteen minute intervals.

These classified turning movement counts were performed at all the circles in Table 5.2 except at the Kensington circle. This circle was only used for the calibration of speeds and gap acceptance and was not used for validation, because it was impossible to do queue length observations.

5.4 Data analysis

This section summarizes the results of the delay and volume surveys, move-up time surveys, speed surveys, critical lag and gap surveys and headway surveys. Because the gap acceptance surveys and results were such an important part of this study they are discussed separately in Section 5.5. Not only are the results of the surveys discussed, but also the historical background to gap/lag acceptance and the different methods which have been used to observe critical gaps and lags.

5.4.1 Delays and Volumes

A detailed summary of the traffic counts providing: 15 minute volumes, hourly volumes, turning volumes, total volumes, peak hour factors, modal splits, stopped delays and average numbers of stopped vehicles for each survey period, are attached in Appendix A. Some of these results are again briefly summarized in Table 5.3.

Table 5.3: Summary of Volumes and delays

Circle	Survey Date	Peak hour Volume (vph)	Peak hour factor	Heavies and buses (%)	Mean delay per vehicle (sec/veh)
Chatsworth	30/07/93 am	1751	0.89	5	5.2
	16/08/93 pm	1922	0.90	5	9.8
	17/08/93 pm	1961	0.94	4	4.6
	18/08/93 am	1537	0.98	5	2.4
Pinetown	15/08/95 am	1508	0.83	1	18.9
	15/08/95 pm	1579	0.93	1	17.7
Queen Mary	25/07/96 am	1677	0.67	2	5.9
	26/07/96 am	2139	0.76	1	5.7

From Table 5.3 it can be seen that all the circles were controlling more than 1500 vehicles per hour, with high peak hour factors which indicate a constant demand throughout the peak hour (except for Queen Mary with the low peak hour factors). Only at the Chatsworth circle a noticeable presence of heavy vehicles and/or buses was evident. Although the mean delays per vehicle are quite low, where data for the 15-minute intervals are considered individually, a considerable variation is found. The traffic at the Pinetown circle experiences the highest mean delays per vehicle.

5.4.2 Move-up times

The move-up time surveys at Chatsworth, Queen Mary and Pinetown Circles are summarized in Table 5.4 with the cumulative distribution of the observed move-up times shown graphically in Figure 5.5.

Table 5.4: Summary results of move-up time surveys

Circle	Central Island Diameter (m)	Move-up times (Seconds)					Sample Size
		Mean	STD	Max	Min	95% confidence interval of mean	
Chatsworth	36.2	2.69	0.63	5.10	1.26	± 0.03	367
Queen Mary	23.6	2.27	0.57	4.12	1.04	± 0.03	382
Pinetown	20.0	2.26	0.48	3.41	1.32	± 0.04	144

STD: Standard deviation, Max: Maximum, Min : Minimum, conf: confidence

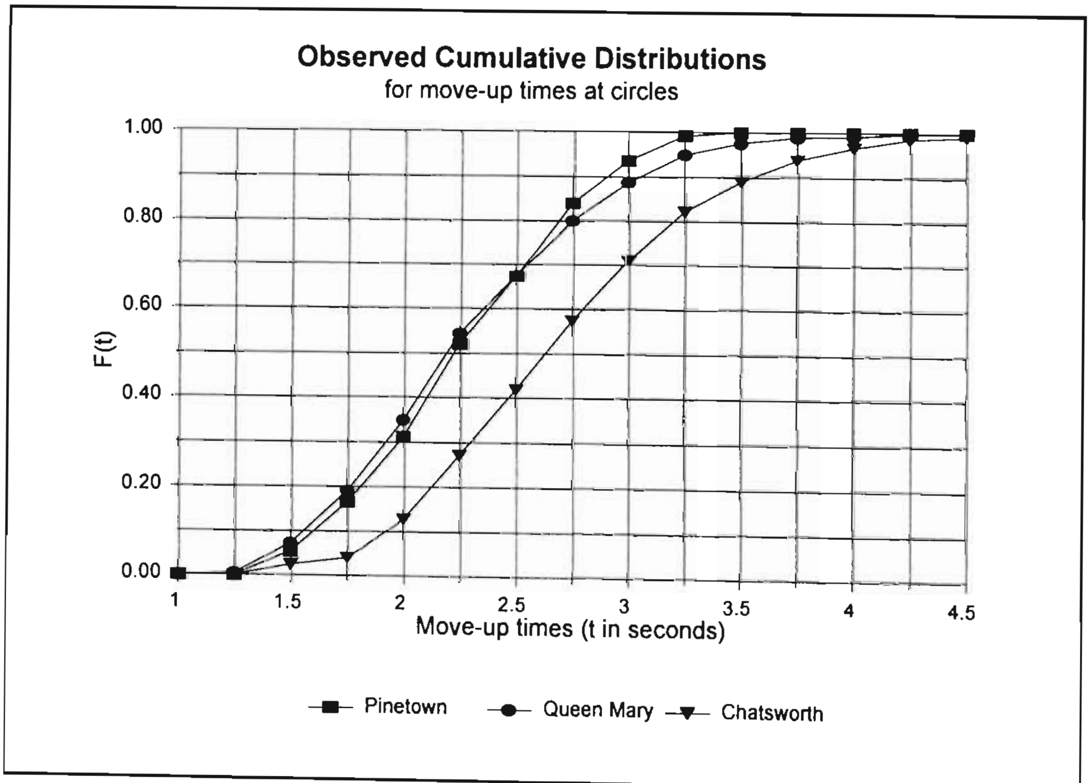


Figure 5.5: Observed cumulative move-up time distributions

The average move-up times vary between 2.06 seconds and 2.69 seconds and increase with an increase in the central island diameter. In the Australian capacity and delay models for traffic circles (Troutbeck, 1989 and Austroads, 1993), the move-up times are calculated as a function of the number of entering lanes, the number of circulating lanes, the conflicting flow and also the Central Island Diameter (see (2-35) to (2-37)). Practical constraints together with the unavailability of a sufficient number of circles operating under reasonable demand volumes, prevented the verification of this Australian approach under South African conditions.

Table 5.5 shows a comparison of the observed move-up times with the calculated move-up times using (2-35). The move-up times were measured over a range of conflicting flows, but for the comparison an average conflicting flow over the observation period was used, as shown in Table 5.5.

Table 5.5: Comparisons of Average Move-up Times

Circle	Central Island Diameter (m)	Circulating Flow (vph)	Avg Move-up times (Seconds)	
			Observed	Calculated (Troutbeck, 1989)
Chatsworth	36.2	450	2.69	2.37
Queen Mary	23.6	800	2.27	2.32
Pinetown	20.0	550	2.26	2.55

For both Queen Mary and Pinetown the estimates according to Troutbeck (1989) are greater than the observed values, while for Chatsworth the estimate is smaller than the observed value. The reasons for this are not apparent, but the data base for the observed values would need to be enlarged considerably, to include more circles with a larger range of flows, before any meaningful conclusions can be reached. The observed move-up times were used in the simulation program for validation purposes.

5.4.3 Speeds

Table 5.6 summarizes the results of the circulating speed surveys as observed at the different circles, showing mean speeds, the standard deviation of the speeds, the minimum and maximum observed speed and also the likely error of the estimate of the mean speed at a 95% degree of confidence.

Table 5.6: Results of circulating speed surveys

Circle	Central island Diameter. (m)	Speed (km/h)					Sample Size
		Mean	STD	Max	Min	95% Confidence interval of mean	
Chatsworth	36.2	37.6	6.6	60.3	22.6	± 0.27	583
Kensington	27.4	32.8	4.0	41.9	23.9	± 0.41	92
Queen Mary	23.6	23.6	2.8	30.6	17.2	± 0.18	228
Pinetown	20.0	23.2	2.5	28.1	15.9	± 0.18	197

The cumulative distributions for the observed circulating speeds at the four circles are compared in Figure 5.6.

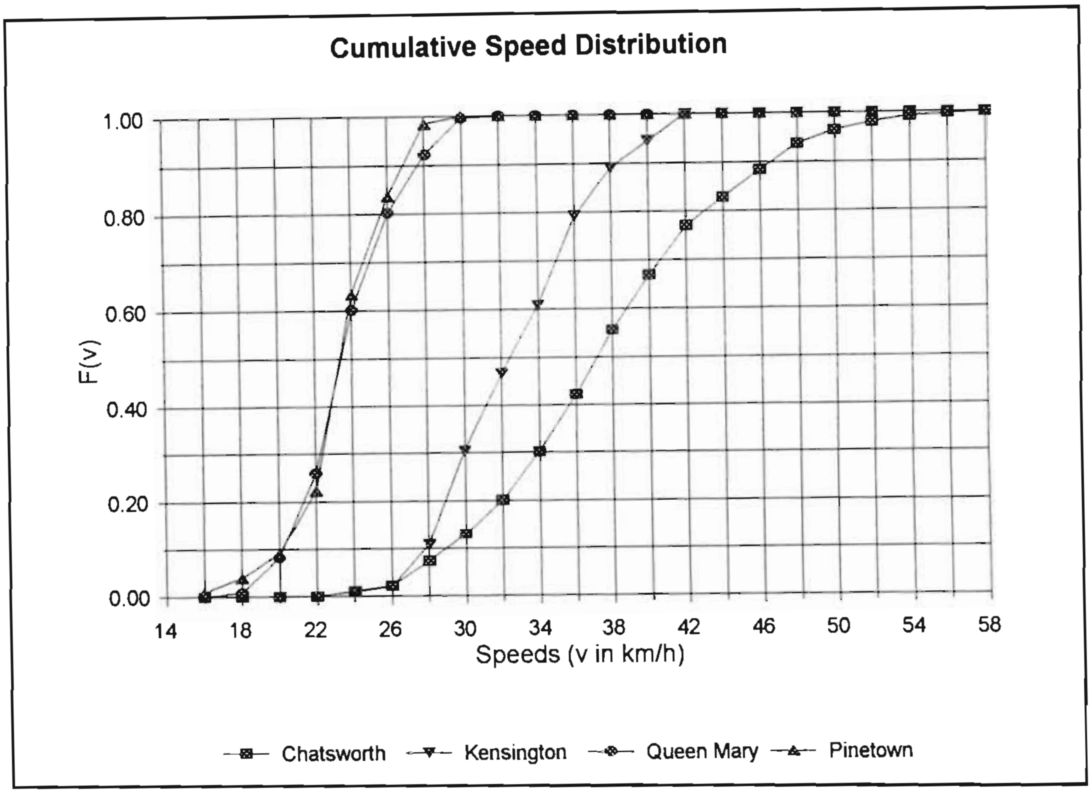


Figure 5.6: Observed cumulative circulating speed distributions

As expected, the circulating speed increases with an increase in diameter. However, this increase is not linear. The large standard deviation of speeds in the Chatsworth circle, compared with the standard deviations observed at the other circles, is partly due to the variation in the age of the vehicles which drive through the circle. Although only passenger cars were included in the sample, some of these cars were rather old which could account for the slower speeds.

These observed circulating speeds were also compared with a value calculated from eq. 5-1, which is usually employed to determine minimum radii for a given design speed (CUTA, 1989). Austroads (1993) recommends the use of this equation for determining design speeds through traffic circles.

$$V^2 = 127R(e+f) \tag{5-1}$$

where R = Radius of curve/circle in metres

e = superelevation rate in metres per metre (negative if sloping away from central island)

f = side friction factor

The superelevation rates (e) were not measured at the different circles and for this comparison the values as shown in Table 5.7 were used. These values are as recommended by CUTA (1993) for design purposes.

Table 5.7: Comparison of observed and design speeds

Circle	Central Island Diameter (m)	Observed Mean speed (km/h)	f	e (m/m)	Design Speed (km/h)
Chatsworth	36.2	37.6	0.4	0.03	31.4
Kensington	27.4	32.8	0.4	0.03	27.4
Queen Mary	23.6	23.6	0.4	0.03	25.4
Pinetown	20.0	23.2	0.4	0.03	23.4

Even with a high value for the friction coefficient (f) the calculated design speeds for the larger circles are still significantly lower (±20%) than the observed speeds. Only at the smaller circles some correlation exists between the observed mean speed and the calculated design speed. This seems to indicate that among others, drivers at the larger circles seem content with a little discomfort when driving around the circle.

The observed mean speeds, standard deviations, minimum, and maximum speeds were used as input to the simulation program for the different circles.

5.4.4 Circulating headways

The distribution of the circulating headways was synthesised from the gap observations . Figure 5.7 shows the cumulative headway distribution as observed at each of the four circles. These headway distributions were not used in the simulation program, but were used to identify a minimum circulating headway and to compare with the simulated headways. The minimum headway was used in the simulation program to prevent the faster vehicles which were catching up with slower vehicles from getting too close to them, and to prevent vehicles from entering too close behind a leading vehicle. It is evident from the observed distributions that a minimum headway of one second is realistic. (This did not mean that no headways smaller than one second would ever occur during the simulation; because of the way the program was written this did happen occasionally)

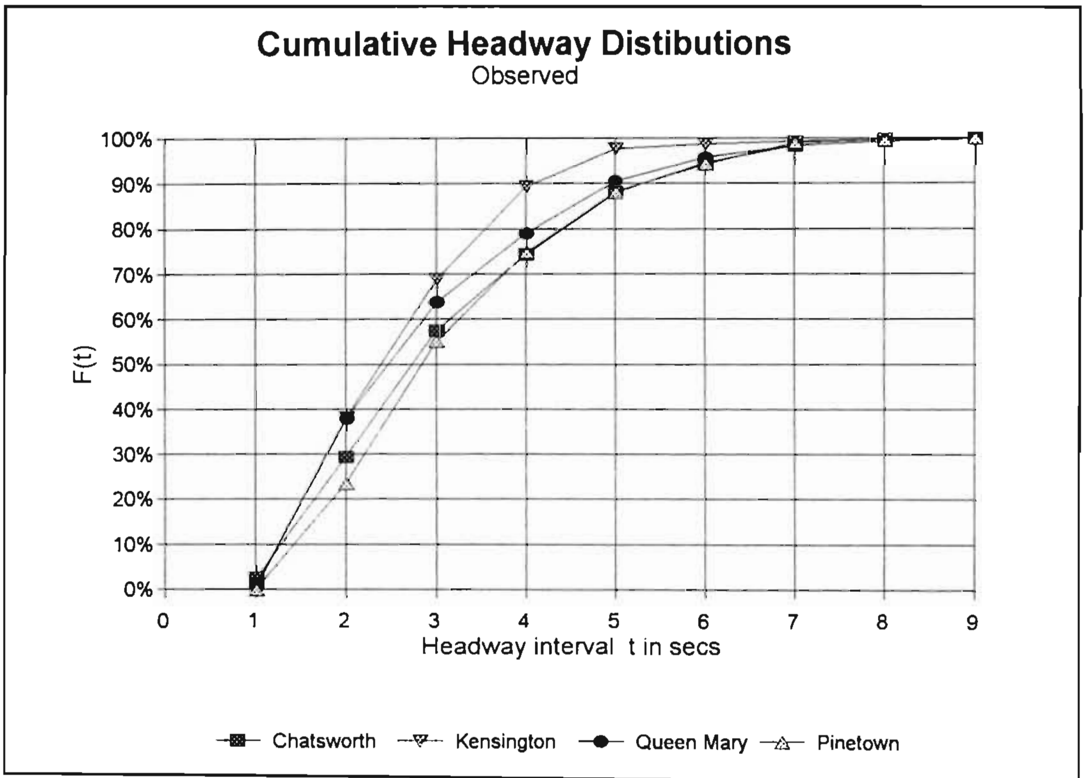


Figure 5.7: Observed cumulative headway distributions

The sensitivity analysis in Chapter 4 indicated that the simulation program is not sensitive to a change in the minimum headway. A range of minimum headways from 0,7 seconds to 1,5 seconds was evaluated. A minimum circulating headway of one second was then used for the simulation of all the circles.

5.4.5 Arrival headways

The only input parameter which has not been validated with local data is the arrival headway distribution. The distribution which was used for generating headways in the simulation model is the bunched exponential distribution as proposed by Cowan (1975). The reason for this is twofold:

i) The sensitivity analysis (see Section 4.7.1) indicated that the model is not sensitive, neither to the arrival headway distribution nor to the parameters of the distribution. This is in contrast to what has been found for other priority and signal controlled intersection models (Akçelik and Chung, 1994a). However, at traffic circles, gaps are not accepted in the original stream of traffic, but in the circulating traffic. The headways of the circulating traffic are no longer directly related to arrival headways, because of the entering process of gap acceptance.

ii) Akçelik and Chung (1994a) recommend the use of the bunched negative exponential distribution for all urban traffic analysis and for all intersection control types. The bunched exponential distribution is a generalization of the negative exponential and shifted negative exponential models (see Section 4.2.1). To calibrate the parameters of the model, Akçelik and Chung (1994a) analysed approximately 29 000 headways on single lane roads. From this they recommend that a minimum headway of 1,5 seconds and a bunching factor of 0,6 be used (see eq. 4-2). As there is no obvious reason why the distribution of arrival headways in this country should be significantly different from that observed in Australia it was decided to use the bunched negative exponential distribution with the calibrated parameters as proposed by Akçelik and Chung (1994a).

5.5 Critical lags and gaps

5.5.1 Background

The process of minor vehicles accepting lags or gaps to cross or merge with a major stream of traffic at any priority intersection has been the subject of many studies in the past, and there is still no conclusive method of how to observe and calculate a mean critical gap or lag. Gaps are defined in terms of the time difference between two successive vehicles in the major traffic stream, but could also be defined as the spacing between two successive vehicles. A lag is defined as a partial gap or the unexpired portion of a gap in the major stream from the time a minor vehicle arrives at the stop or yield line. Any vehicle arriving or waiting to cross or merge with the major traffic stream is presented firstly with a lag, and if that is not accepted, then it is presented with a series of gaps until one of them is accepted. The size of this gap depends on what is defined as the driver's *critical gap*. Any presented gap greater than the driver's critical gap will be accepted and any gap smaller than that will be rejected.

According to Van As & Joubert (1993) differentiating between the first gap presented to the driver (lag) and the subsequent gaps is important, because their acceptance characteristics may be different. Sometimes vehicles are still moving when a lag is considered, which means that a much smaller lag can be accepted compared to when the vehicles are stationary. On the other hand, there could be a restriction in visibility and/or the traffic, and pedestrian movements at the intersection could be quite complex, which would require some time for the driver to assess and become accustomed to (learning process), before deciding. This could result in accepted lags being greater than the accepted gaps.

The most common model for gap and lag acceptance is based on a time hypothesis, i.e. the driver's decision is based on the gap size in terms of time only (Gibbs, 1968). However, other models based on time and minimum distance, distance only, angular velocity and change in dimension have also been considered at some time (Gibbs, 1968). Although these other models might be more sophisticated, they might also be more complex to apply than the time model (Van As & Joubert, 1993).

The earlier gap acceptance models (Tanner, 1962) assumed a fixed critical gap. All drivers rejected a gap smaller than that fixed value and accepted a gap larger than the critical gap. As the critical gap is clearly not fixed and varies not only from one person to the next, but also from time to time for the same driver. Subsequent models tried to incorporate the element of variability and concentrated on a distribution of critical gaps. Two different methods emerged using a variable critical gap. The first method (Weiss and Maradudin, 1962 and Hawkes, 1968) assumed a driver's decision for each gap, based on an independent decision and a different critical gap each time a gap is considered. The second method (Yeo & Weesakul, 1964, and others) applied a fixed critical gap for each driver, but varying from driver to driver. The second method introduced consistency in the decision-making process where a driver will not accept gaps smaller than any other previously rejected by the same driver. In the first method this is possible.

Which of the two methods to use is debatable. Although there is reason to believe that a driver's critical gap does vary, especially under high conflicting flow conditions, it is unlikely that it will vary in the arbitrary manner as suggested in the first method (Ashworth, 1970). In a recent study Polus et al (1996) investigated the change in drivers' gap acceptance over time assuming learning and impatience. They only did a simulation study without validating and confirming the actual change in drivers' critical gaps over time. Their investigation showed that instead of actually estimating the learning rate or the rate at which a driver's critical gaps will change over time, it is possible to estimate delays at priority intersections by using Drew's delay model with a constant critical gap which depends on the learning rate. Drew's model for expected delay or waiting time is defined as follows:

$$E(W) = \frac{1}{\lambda}\left(e^{\lambda\tau} - \lambda\tau - 1\right) \qquad (5\text{-}2)$$

where
E(W) = delay
λ = expected gap value
τ = critical gap

A further problem which received much attention in the past is that of analysing observed gap and lag acceptance data, and to determine an accurate theoretical distribution which describes the process adequately. Apart from being time consuming, labour intensive and hence expensive, the surveys to collect lag and gap acceptance data are straightforward. However, the analysis of the data to derive at an underlying distribution has many pitfalls.

The fundamental problem in the analysis of lag/gap acceptance data arises from the practical situation in which data is collected. An ideal experiment would have been to subject each driver to a series of gaps increasing in size, and record the eventual gap to be accepted (Van As & Joubert, 1993). This not being practical, observations are conducted under normal operating conditions. The problem then, is that a driver might be observed rejecting several small gaps, none of which is close to the driver's critical gap, before accepting a gap much greater than his or her critical gap. Moreover, a driver might accept the first gap without previously rejecting any. Ashworth (1970) ascribes this bias to the fact that the drivers with large critical gaps will reject several shorter gaps before finding an acceptable gap, while the drivers with smaller critical gaps are more likely to accept one of the first gaps presented. This means that the observed proportion of drivers who are prepared to accept a given gap size is somewhat less than the actual proportion of drivers who will accept such a gap (Ashworth, 1970; Miller, 1971; Drew, 1968). When all the gaps are considered, the careful driver with the larger critical gap will be over-represented compared to the "faster" driver.

Due to this problem of bias many researchers have concentrated on lags which are not affected by this problem, because every driver is presented only with one lag and it is either rejected or accepted. Two methods which have found common favour are *Raff's method* and *probit analysis* (Van As & Joubert, 1993). Raff's (1950) method of analysis of lags is probably still the most commonly used (Miller, 1971), but this method only estimates a quantity called a critical lag. This critical lag is defined by Raff (1950) such that the number of accepted lags shorter than the critical lag is the same as the number of rejected lags greater than the critical lag. However, this critical lag is not clearly defined and is different from the mean critical lag. Ashworth (1970) and Miller (1971) shows how Raff's critical lag relates to the mean critical gap.

The second method for analysing lags (Miller, 1971) involves the use of Probit analysis, which is a standard statistical technique for fitting normal or log-normal distributions to observed statistical data (Finney, 1962). The proportions of, for example, acceptances are transformed to probits, where a probit is the number of standard deviations from the mean which equals the same proportion for the cumulative normal distribution. For any cumulative normal or log-normal distribution this transformation changes the proportions on the y-axis to the number of standard deviations, with the resulting change of the well-known S-curve to a straight line. Doing this transformation on observed data or the logs of the data, and fitting a straight line through these, a best fit normal or log-normal distribution of the data is obtained. Miller (1971) confirms that provided the class interval is fairly small and the sample size fairly large, probit analysis yields an unbiased estimate of the lag acceptance parameters.

Apart from the above two methods for lag acceptance analysis, Miller (1971) used computer simulation to evaluate seven other suggested methods for gap acceptance analysis. Ashworth (1970) did a similar exercise. Among the different methods evaluated were methods proposed by Ashworth (1968), Blunden, Clissold and Fisher (1962), Drew (1968),McNeil and Morgan (1968) and another standard statistical technique known as Maximum Likelihood as proposed by Miller (1971). These methods are discussed in detail in these papers and also to some extend by Van As and Joubert (1993) and will not be repeated here, except for some detail of the recommended methods which were also used in this study.

According to Miller (1971) the two methods which give satisfactory results are Ashworth's method (1968) and the Maximum likelihood method as applied by Miller (1971). He states that although Ashworth's method is slightly less accurate, it is much easier to apply than the more complicated Maximum Likelihood method. However, with the general availability of high power desktop computers, the Maximum Likelihood method is just as easy to apply. Miller (1971) also argues that methods using only lag information when trying to predict gap acceptance behaviour are inferior to those methods which use all gap information. This is so because firstly they are based on less information and hence are less accurate, and secondly, lags are more difficult to measure accurately than gaps.

Similar to probit analysis for lags, Ashworth's method is based on fitting a normal distribution through all observed gaps also employing probit analysis. To remove the bias introduced by the slower, more careful drivers, this distribution is then moved to the left (reducing the mean) with an amount of qS^2 as shown in (5-3).

$$\tau_A = \tau - qS^2 \tag{5-3}$$

where τ_A = Adjusted mean critical gap (Sec)

τ = Observed mean critical gap - obtained from probit analysis (sec)

q = Average conflicting flow (veh/second)

S = Standard deviation of observed critical gap - from probit (sec).

While Ashworth (1968) makes use of all accepted and rejected gaps, Miller (1971) advocates the use of the Maximum Likelihood method for estimating critical gap distribution parameters, using only the *largest rejected gap* and the *accepted gap*. Maximum Likelihood is a standard statistical technique used to estimate distribution parameters. With probability theory Miller builds up an argument to an eventual definition of the log-likelihood function (**L**):

$$L = constant + \sum \log_e[F(a_i) - F(r_i)] \tag{5-4}$$

where: $F(t)$ = Cumulative probability distribution of the critical gaps, i.e. $F(t)=P(t_i \leq t)$

a_i = Size of the accepted gap

r_i = Size of the largest rejected gap

The objective of the method is then to maximize L, but to do that a distribution describing the critical gaps has to be assumed. Miller (1971) assumes a log-normal distribution for the critical gaps. With the assumed distribution with unknown parameters (μ and σ in the case of the normal and log-normal distributions) replacing F(a) and F(r) in (5-4), L can be maximized by either differentiating it in terms of μ and σ, equating the two subsequent equations to zero and then solving for μ and σ, or by using a numerical optimization technique.

In another attempt to negate the effect of bias in the analysis of all time gaps, it was decided to look at first gaps only. This is not the lag, but the first gap a driver evaluates once the lag has been rejected. Based on the same arguments as before there would be some bias towards the slower drivers as faster drivers might accept the lag and hence not be considered in the sample where only first gaps are evaluated. This bias however, will be much smaller than when all rejected gaps are included in the sample. Distinguishing between first gaps and all subsequent gaps, also brings in the effect of drivers' learning and impatience. If a model differentiates between first gaps and subsequent gaps, the opportunity exists for larger first critical gaps than any subsequent critical gaps.

5.5.2 Analysis

As discussed in Section 5.3.2 the data collection process included the recording of lags and gaps separately and it also distinguished between entering and circulating lags or gaps. The data sets for gaps also labelled all first gaps and largest rejected gaps.

In any analysis the first step is to group the data observations into class intervals, for example number of acceptances and rejections observed at gaps less than one second, between one and two seconds, etc. The size and/or number of these class intervals are often important because it can alter the impression of the data's behaviour a great deal. Benjamin and Cornell (1970) suggests the following equation to estimate the number of intervals k.

$$k = 1 + 3{,}3 \log n \qquad (5\text{-}5)$$

where n is the number of data values, and logarithms to the base 10 are used. Having established the number of intervals, the next step was to establish the sizes of the class intervals The following three methods were evaluated:

i) Equal size intervals,
ii) Equal number of observation in each interval,
iii) Equal probabilities for each interval.

Fitting a log-normal distribution to a number of sets of observations the most consistent best fit was obtained by using equal class intervals of 1 second. This class interval size was then used for all subsequent analysis.

Jordaan and Joubert (1983) investigated gap acceptance in South African cities and found that critical gaps are described well by the log-normal distribution. In this analysis the log-normal distribution clearly fitted the data better than the normal distribution. This applied for both critical lags and critical gaps. Only in Ashworth's method, the log-normal distribution could not be used, because Ashworth's method assumes a normal distribution. The analysis of the lag and gap results are discussed in the next two sections, 5.5.3 and 5.5.4 respectively.

5.5.3 Lags

As there is no bias in the lag observations, probit analysis was used to fit a log-normal distribution to the lag observations. From the regression of finding a best fit line through the data, the mean and standard deviation of the log values are obtained. The natural mean (μ) and standard deviation (σ) of the critical lags are related to the mean (μ_{ln}) and standard deviation (σ_{ln}) of the log-normal distribution as follows (Benjamin and Cornell, 1970):

$$\mu = e^{(\mu_{ln} + \sigma_{ln}^2/2)}$$
$$\sigma^2 = \mu^2 (e^{\sigma_{ln}^2} - 1) \qquad (5\text{-}6)$$

Initially a distinction was made between entering lags and circulating lags. The validity of this distinction was checked for the Chatsworth circle. The results of this comparison are shown in Table 5.8 and in Figure 5.8. In all the Chi-square goodness-of-fit tests a level of significance is quoted which if greater than 5%, means that the theoretical distribution cannot be rejected. Traditionally the actual and theoretical (at 5% level of significance) Chi-square values are compared, and if the actual is less than theoretical then the fit is accepted.

Table 5.8: Comparing entering and circulating lags

	Mean		STD	Goodness of fit Chi-Square test Level of Significance	Number of Observations
	Estimated	Error 95% conf			
Entering lags	3.86	0.10	1.26	49.63%	352
Circulating lags	3.84	0.14	1.18	7.00%	216

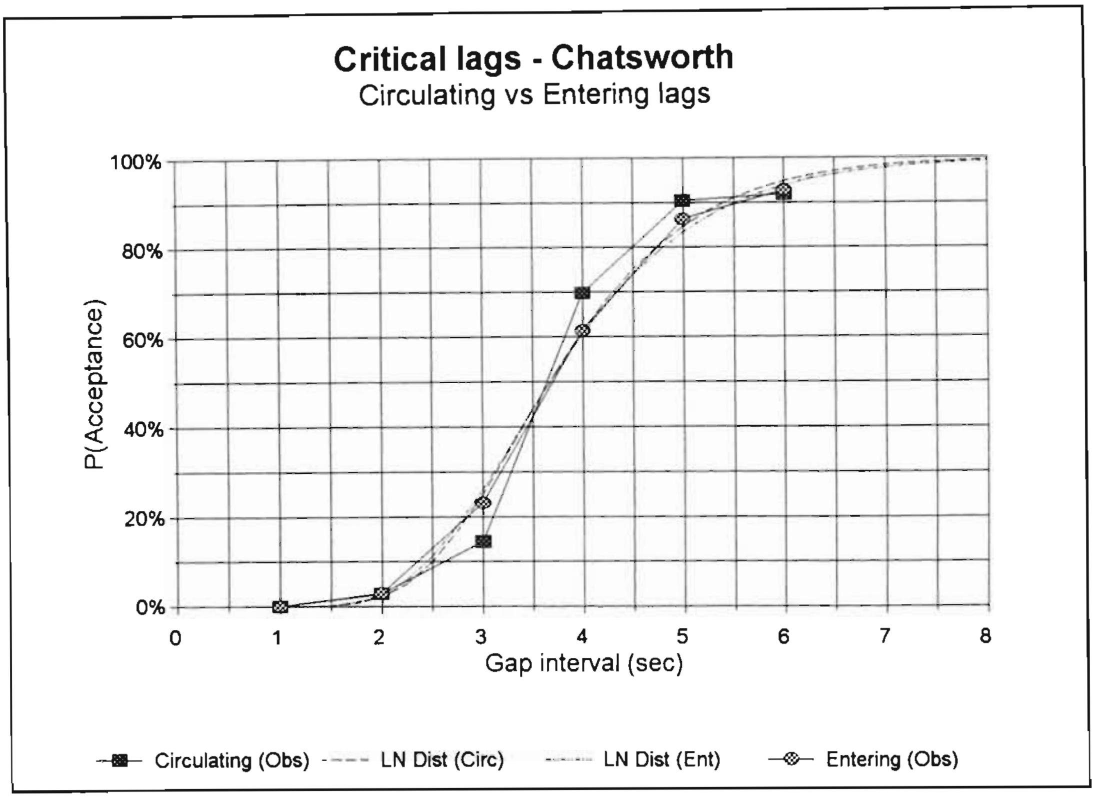


Figure 5.8: Critical lags - Entering lags vs Circulating lags

From the above the log-normal distribution clearly seems a reasonable fit for both the entering and circulating lags. The estimated mean values are also very close and there is a suspicion that no difference exists between them, given the size of the error at the 95% confidence level. To check this suspicion, the z-statistic for large sample tests concerning the difference between two means was calculated. This z-statistic for the above two samples is equal to 0,21 which is less than 1,96 (95% confidence level) and thus supports the null hypothesis that there is no difference between the two sample means. Therefore, all the lag data were combined into one sample for analysis.

The above comparison between entering and circulating lags was only made for the Chatsworth circle and due to insufficient data could not be repeated at the other circles. Consequently, all lag data per circle were combined for analysis of critical lags. The log-normal distribution gave the best fit of the observed probability of acceptance, and the mean critical gap and standard deviations for each circle are listed in Table 5.9. Also shown in the table is the error which would include the 95% confidence interval and the results of the Chi-square goodness-of-fit test. Figure 5.9 gives a graphic comparison of the variation of the means and standard deviations from circle to circle, and also shows the 95% confidence interval for both the mean and standard deviation.

Table 5.9: Critical lags - Parameter estimates from log-normal distribution fit

Circle	Mean		STD		Sample Size	Goodness-of-fit Chi-Square Level of Significance
	Estimate	Error (95% Conf)	Estimate	Error (95% Conf)		
Chatsworth	3.87	0.08	1.25	0.06	872	12.3%
Kensington	3.93	0.11	1.03	0.09	320	38.8%
Queen Mary	3.56	0.16	0.98	0.13	145	45.6%
Pinetown	4.06	0.10	0.77	0.08	234	12.1%

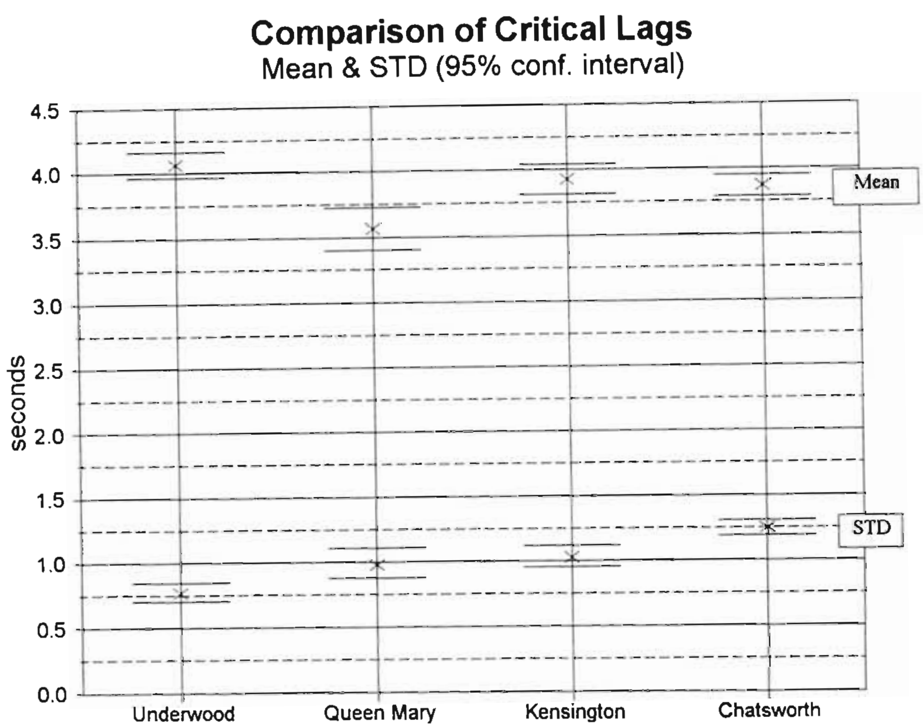


Figure 5.9: Comparison of critical lags and standard deviations of lags

The mean critical lags range from about 3,5 seconds to just more than 4,0 seconds, and except for Queen Mary circle, the mean critical lags increase with a decrease in circle size. Queen Mary circle however, has higher circulating flows and mean delays, and hence the possible discrepancy. The Chi-square goodness-of-fit tests indicate that at the 5% level of significance, the log-normal distribution fits the observed data. The cumulative log-normal distributions with the above estimated parameters for the different circles are shown in Figure 5.10.

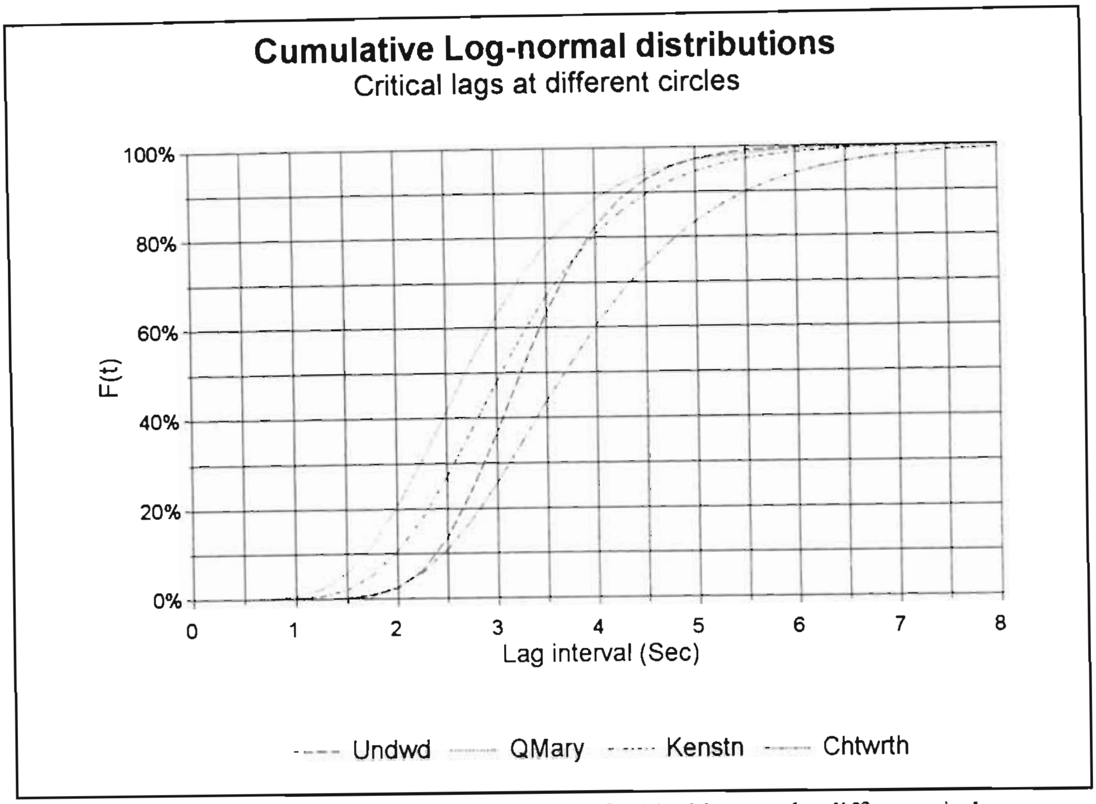


Figure 5.10: Cumulative log-normal distributions of critical lags at the different circles

5.5.4 Gaps

Similar to the analysis of the critical lags, an attempt was made to distinguish between critical entering and critical circulating gaps. However, even more so than in the case of lags, insufficient sample sizes compelled the combination of all data.

For the reasons discussed in Sections 5.5.1 and 5.5.2, gaps were analysed in three different ways. In the first method only first gaps were included in the sample and because there is only a small bias, no correction for the bias was made. With probit analysis a log-normal distribution was fitted to the observed percentages of acceptance. Secondly, Ashworth's method was used to shift the normal distribution as fitted through the observed percentages of acceptance. The third method used was that of Maximum Likelihood, where only the largest rejected gap and the accepted gap were included in the analysis. A summary of the results of the analysis is given in Table 5.10 with a graphical comparison in Figure 5.11.

Table 5.10: Critical gaps - Parameter estimates using different methods

Circle	Method of Analysis	Mean		STD		Sample Size	Goodness-of-fit Chi-Square LoS
		Estimate	Error (95% Conf)	Estimate	Error (95% Conf)		
Chatsworth	First Gaps	4.92	0.17	1.63	0.13	358	5.3%
	Ashworth	4.51	0.08	1.08	0.06	625	
	Max Likelihood	4.57	0.09	0.92	0.06	446	
Kensington	First Gaps	4.85	0.26	1.70	0.20	169	14.8%
	Ashworth	4.36	0.10	1.07	0.07	454	
	Max Likelihood	4.30	0.12	0.98	0.09	276	
Queen Mary	First Gaps	4.05	0.16	0.74	0.14	79	59.5%
	Ashworth	4.29	0.09	1.03	0.07	498	
	Max Likelihood	3.96	0.11	0.78	0.09	180	
Pinetown	First Gaps	4.79	0.17	1.13	0.14	167	18.9%
	Ashworth	4.27	0.11	1.04	0.08	375	
	Max Likelihood	4.21	0.11	0.89	0.09	242	

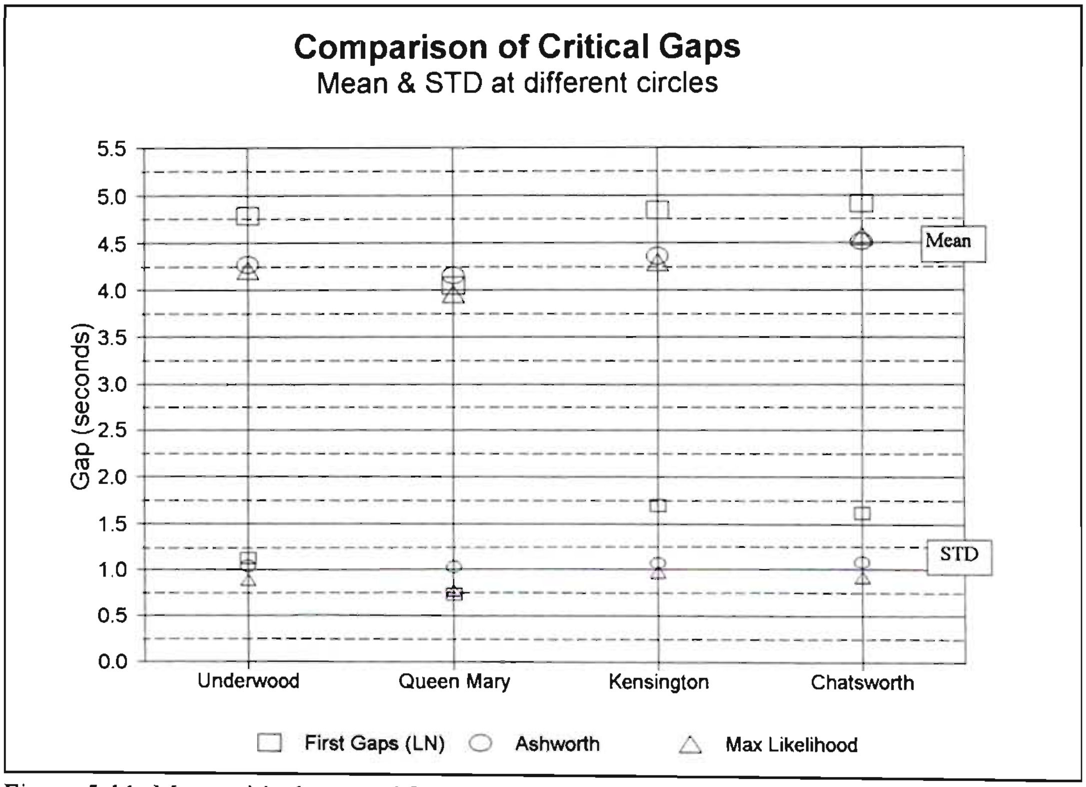


Figure 5.11: Mean critical gap and STD comparisons

The results from Ashworth's method and the Maximum Likelihood method are very similar except for the Queen Mary circle. The first gap method gives higher average critical gaps and standard deviations, again excluding Queen Mary circle. Considering the learning process, it may be expected that once the lag is rejected, the driver could still be somewhat diffident initially whilst becoming familiar with the intersection and the environment. It is only at Queen Mary circle where this first gap analysis gives results similar to the other two methods which include all other gaps. The reason for this is not obvious. The circle is in a residential area, with high entering and circulating volumes. The best explanation is probably that the wide circulating carriageway with wide approaches doubles up for left turning vehicles, and sometimes vehicles also double-up when going around the circle. The favourable geometry thus allows flexibility to drivers and room for chances to be taken, especially for regular users. At Queen Mary with the higher circulating volumes there is definitely a trend to smaller gaps being accepted.

Again ignoring the mean critical gap at Queen Mary, there seems to be an increasing trend in the critical gaps with an increase in circle size. In Figure 5.11 the results are shown per circle and the circles are listed increasing in central island diameter, although not to any x-axis scale. The critical gaps seem to be around 4,5 seconds with a standard deviation of about 1,0 seconds. For the simulation program the mean critical gaps as estimated with the Maximum Likelihood method were used for all gaps after the lag and first gaps. For the first gap decisions the mean critical gaps as observed were used. This, to some extent tried to simulate driver behaviour and specifically the learning process, more accurately.

Most gap acceptance models (Joubert & Van As, 1993; Chung, 1993) use a single critical gap model, where all the lags and gaps are combined to give one mean critical gap with a standard deviation. Amongst others, Jordaan and Joubert (1983), concluded that the differences between lags and gaps are small. According to observations completed during this research there is a significant difference between lags and gaps at traffic circles. Entering vehicles are only required to yield, and often vehicles are moving when drivers make their first decision. Either the entering vehicles are arriving at the circle or are moving up in the queue while approaching the yield line. The mean critical lag, as observed in this research, is significantly less than the mean critical gap for the first gap decision. It is also smaller than the mean critical gap of all subsequent gap decisions.

The cumulative log-normal distributions from the Maximum Likelihood method for the critical gaps for the four different circles are shown in Figure 5.12, with the cumulative normal distributions from Ashworth's method shown in Figure 5.13. From these graphs it can be seen that Ashworth's method gives a closer estimate of the critical gap distribution than the Maximum Likelihood method.

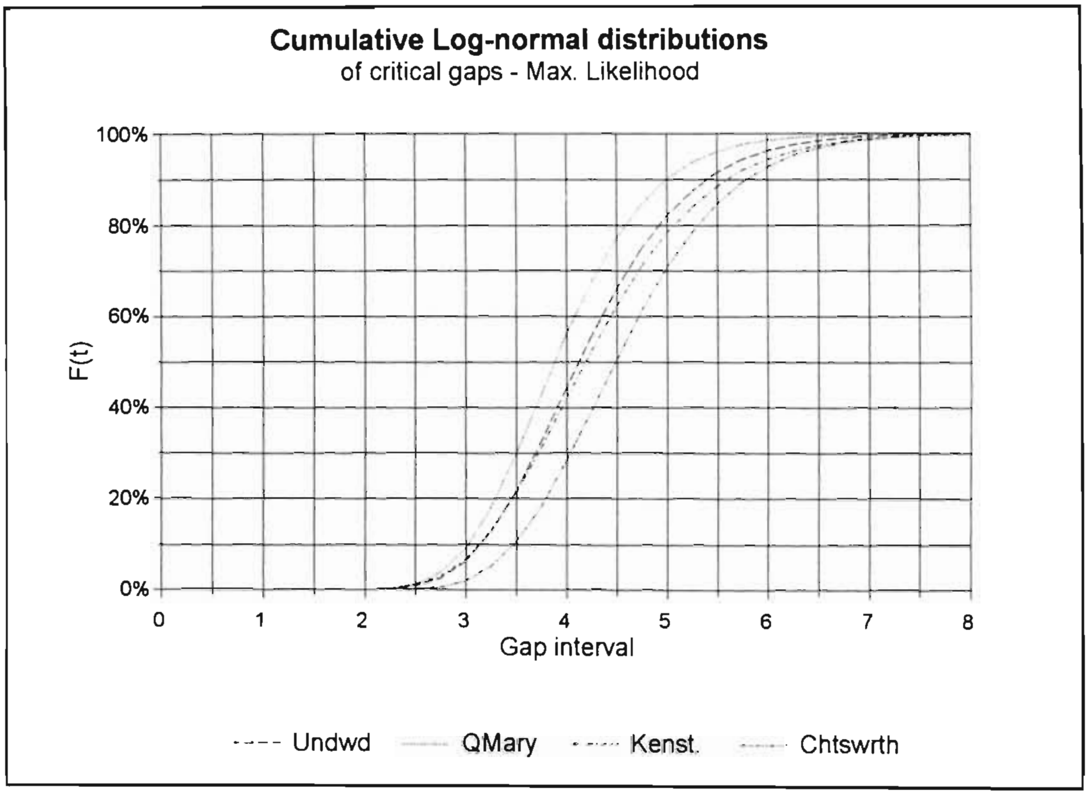


Figure 5.12: Cumulative log-normal distributions of critical gaps (Maximum Likelihood)

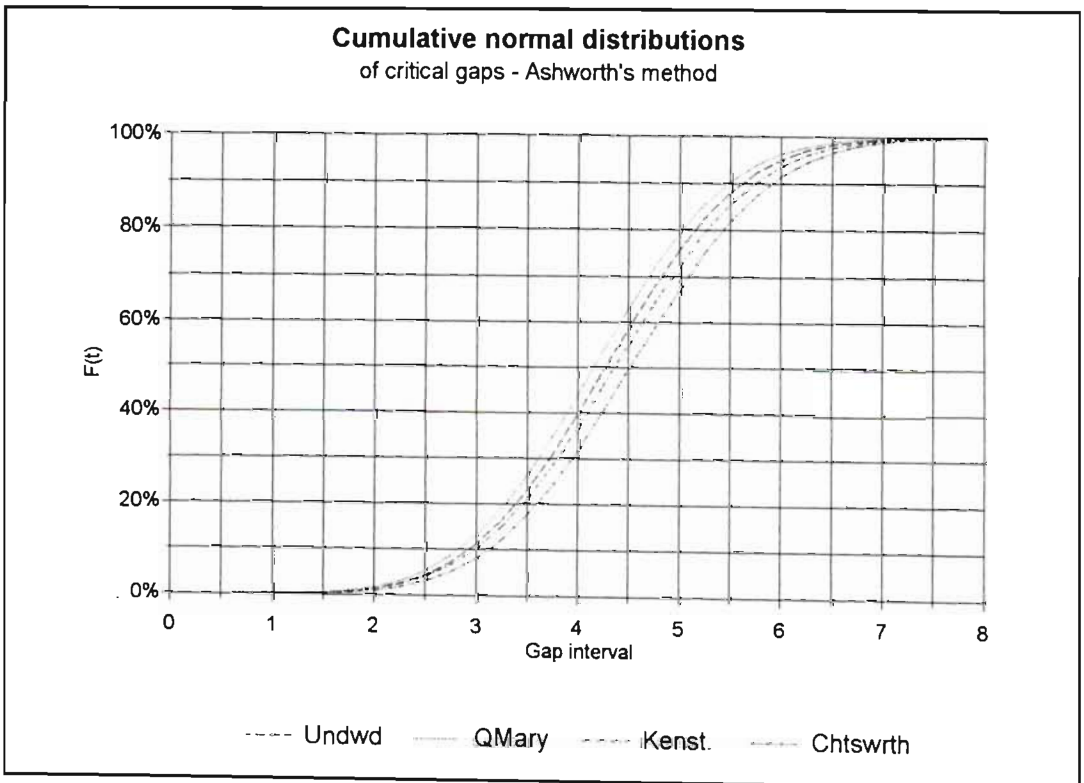


Figure 5.13: Cumulative normal distributions of critical gaps (Ashworth's method)

To put the gap acceptance behaviour observed during this study into context, it was compared with other work on this subject. In Table 5.11 the estimated mean critical gaps are compared with a calculated value (see (2-37), (2-35) and (2-44)) using the Australian method (Chung, 1993; Troutbeck, 1989). Note that the observed mean critical gap, was observed over a range of conflicting flows. The Australian method of calculating critical gaps uses the mean conflicting flow as an input parameter. Quoted in the table is the mean conflicting flow over the period during which the observations were made. This mean conflicting flow was used to estimate the move-up times and the critical gaps. The table also combines all the approaches, while (2-35) and (2-37) are developed for estimating move-up times and critical gaps for a specific approach under variable conflicting flows. The average entry width was measured as shown in Figure 2.6

Table 5.11: Comparison of observed and calculated critical gaps.

Circle	Inscribed Diameter (m)	Avg. entry lane width (m)	Avg. conflicting flow (veh/h)	Move-up Time (Eq 2-35)	Mean Critical Gap (sec)	
					Calculated (Eq 2-37)	Estimate (observations)
Chatsworth	50.0	3.8	450	2.37	4.51	4.57
Kensington	51.8	7.5	450	2.35	1.53	4.30
Queen Mary	42.5	5.0	800	2.32	3.23	3.96
Pinetown	33.6	4.7	550	2.55	4.00	4.21

From the comparison in Table 5.11 it is obvious that except for the Kensington circle the differences between the values estimated using the Australian method (2-37) and the estimates from the local observations are small. What is also evident is that although the differences are small, the Australian estimates are all lower than the locally observed gaps. The large difference for the Kensington circle is because even though the approaches to the circle are very wide, they are used as single entering lanes. If the entry lane width is halved, then the Australian estimate would increase to 4,5 seconds.

Equation (2-37) is sensitive to a change in the average entry lane width and it is not always clear how it should be applied, especially when the entry lane is only occasionally used as a double lane entry for the odd left turning vehicle. As in the case of the Kensington circle where the approaches are wide enough to accommodate two vehicles, but are only used as single lane approaches because of the rest of the circle geometry.

What can be concluded from this comparison is that the locally observed critical gaps are similar to Australian estimates, although slightly larger. This can be explained by taking into account that South

African drivers are not as accustomed to negotiating traffic circles than their Australian counterparts and are hence slightly more reluctant to accept smaller gaps. The nett affect of this is that traffic delays at local circles should be more than at similar circles in Australia with similar traffic volumes.

The results of various other gap acceptance studies conducted mainly in the United Kingdom and Australia are summarized in Table 2.2. The mean critical gaps observed during these studies vary between 3,2 and 4,0 seconds, while the gaps observed during this research vary between 3,96 and 4,57. Again this illustrates that when comparing South African drivers and their behaviour at traffic circles with their counterparts in Australia and the United Kingdom, the local drivers are more conservative in accepting gaps in the circulating traffic stream.

5.6 Summary

In this chapter a discussion is given on the various data which were required to calibrate the input parameters to TRACSIM, to validate the operation of the model and to identify the geometric input values to the program. The data need in terms of input data, calibration data and validation data are evaluated and discussed.

Four circles were identified in and around the Durban Metropolitan area as being appropriate to use for data collection. At each of these circles, different types of surveys (delay -, volume -, move-up time -, speed -, critical gap - and headway surveys) were conducted. The analyses of the survey data are presented and where necessary, comparisons of the results are made with those obtained at the other circles, as well as with theoretical estimates.

The observed move-up times and circulating speeds seem to increase with central island diameter, with the circulating speeds in km/h comparable with the central island diameter in metres. The observed move-up times vary between 2,3 and 2,7 seconds, and the circulating speeds between 23,2 and 37,6 km/h. The comparison of the observed move-up times and speeds with theoretical estimates are not favourable, but a larger sample across more circles would be required to make any significant conclusions about the differences.

A distinction is made between lags and gaps. The observations of the gap and lag results from the circles compare favourably, and vary between 3,5 to 4,1 seconds for lags, and 4,3 to 4,5 seconds for gaps (using Ashworth's method). The results for the Queen Mary circle seem to differ slightly from those at the other circles.

CHAPTER 6: VALIDATION OF SIMULATION PROGRAM

Validation of the simulation model is the last interactive step in the model development process before the model can be applied. During this process the performance of the model is compared with data collected in the field. According to Young et al (1989) this is probably the most difficult step in the development process due to the following possible problems which can arise:

i) Incorrect methodology. These errors are usually a result of poor numerical analysis and/or inappropriate random number generators.

ii) Poor experimental design. Problems associated with data collection are: often the data available are not relevant, the quantity of data is not adequate, or the data might contain errors.

iii) Incorrect interpretation. This kind of error is difficult to detect, especially if the model is used to model behaviour of a system outside the range for which it was calibrated or outside the range for which the parameters have been verified.

iv) Model instability. This requires that small changes to the input data result in similar changes to the model predictions.

In this chapter the validation process is discussed, by firstly investigating the methodology which was followed and then comparing the TRACSIM delay estimates with observations at three different circles. The comparisons are made by approach, and combined where all the data for the different approaches are combined and categorized.

6.1 Methodology

The delay observations at three of the circles, Chatsworth, Pinetown and Queen Mary(see Table 5.3), were used to validate the simulation model. These delay observations and volume counts at the different circles were summarized in 15 minute intervals. The 15 minute flows were then used as input to the simulation model. The simulation model estimated a mean delay per vehicle which could then be compared with the observed mean delay per vehicle over the same 15 minute period. The number of 15 minute periods observed and simulated per circle is summarized in Table 6.1. Chatsworth was observed for four peak periods while Pinetown was only observed for two peak periods and Queen Mary circle only for one peak period.

Table 6.1: No of periods simulated per circle

Circle	No of 15 min periods
Chatsworth	26
Pinetown	12
Queen Mary	7

Although the same circles and the data from the same observation periods have been used during the estimation of the input parameters, the parameters have all been estimated directly. That is, none of the parameters have been calibrated or estimated indirectly by using the estimation results of the simulation program and comparing these with the observed values for a best fit. Using the parameters estimated from direct observations and the delays observed during the same period, to compare with the delay outputs of the simulation model, is thus a true method of validating.

The Chatsworth circle (see Figure 3.1) was the only circle where all the approaches were used for validation. At the Pinetown circle only the southern approach was used (see Figure 3.4). The reasons for this are that the other approaches either had low traffic flows and/or low delays, or the gap acceptance behaviour was different to what was expected for traffic circles due to the unconventional geometric layout. From Figure 3.4 it can be seen that the western, northern and eastern approaches are all contained within only 110°, with the eastern exit and entrance being close to the northern entrance, and the northern exit not existing. Moreover, traffic from the western entrance is not adequately deflected when approaching and entering the circle, and hence vehicles enter at high speeds. Traffic entering from north then gives way to these vehicles, and the operation can be more comparable with minor road traffic entering a major traffic stream at a T-intersection, than with a gap acceptance process at a circle.

Queen Mary circle (see Figure 5.3) has wide approaches with a wide circulating carriageway and a separate queue for left turning traffic on all approaches. The turning movements around the circle are such that, although the approach volumes (except for the western approach) are fairly similar, the eastern and southern approaches experience short delays (less than 5 seconds per vehicle). Therefore only the western and northern approaches where the maximum mean delays exceed 15 seconds per vehicle have been used for validation. The simulation program can only handle single approach lanes and a single circulating carriageway. Therefore, because the left turning vehicles queue in a separate queue, not entering the circle and conflicting with entering traffic from the next approach, the left turning traffic was excluded from the simulation and also from the queue length observations. During the validation Queen Mary circle was thus simulated using only right and through vehicles while the queue observations excluded the queue lengths of the left turning traffic.

There are two possible approaches to using the observed and estimated delays for validation. The first approach would be to run the simulation program for a long period so that all the random fluctuations are averaged out and the program eventually predicts a mean delay per vehicle for each approach for the given input flows. However, this would mean that the observed delays would also need to be observed over a long period of time to find a comparable average delay. This is obviously impractical from a cost and manpower point of view.

The second approach is to view the observed delays as one sample from a population and then to use the simulation program to predict another sample. Statistical analysis can then be used to verify if there is a possibility that the two samples are from the same population. To obtain a comparable sample from the simulation program it was run for a similar time period of 17 minutes, two minutes to "warm up" and not start with an empty system, and then 15 minutes of simulation. The initial time of two minutes was considered adequate to allow the first vehicles to enter the circle so that when the delay measurements start, the entering vehicles are not entering an empty system. The simulation program can also be used to describe the possible statistical variation by starting the program with different seed numbers for the random number generators. Running enough of these 15 minute simulations, starting each series with a different seed number, should result in an envelope which should contain all the possible random fluctuations. For this study the program was run at least 15 times for each 15 minute simulation period, starting each time with a different seed number.

6.2 Comparison of observed and simulated results

This section compares the simulation results with the observed average delay per vehicle at the three circles. The comparison will first be made circle by circle, after which all the data are combined. As there is often a large variation in estimates, which makes comparisons difficult, a straight line was fitted through the data where possible. In all cases, the observed data points were taken as being correct except for observer's error. However, most comparisons were made with both the actual observed data and the observed data corrected for observer's error.

6.2.1 Chatsworth circle

All four approaches at the Chatsworth circle were used for the validation. A total of twenty six 15-minute intervals (See Table 6.1) were used for the comparison. Each 15-minute interval resulted in an observed delay per approach and an estimated delay per approach. So from the four approaches and the 26 data intervals, 104 data points resulted. For the individual approaches 26 data points were available. With the input volumes of each 15-minute interval the simulation program was run fifteen times, each time with a different random number seed. The following graphs show a line for the observed values, a mean estimated line and a minimum and maximum estimated line. The mean, maximum and minimum estimates are the results of a regression analysis on each of the fifteen simulation runs.

The following four graphs (Figures 6.1 to 6.4) show the comparison between the observed and estimated results for the northern, eastern, southern and western approaches respectively. As was discussed in Section 5.3.1 it was not clear whether to adjust the observed delays for observer's error or not and therefore both curves are shown in the graphs.

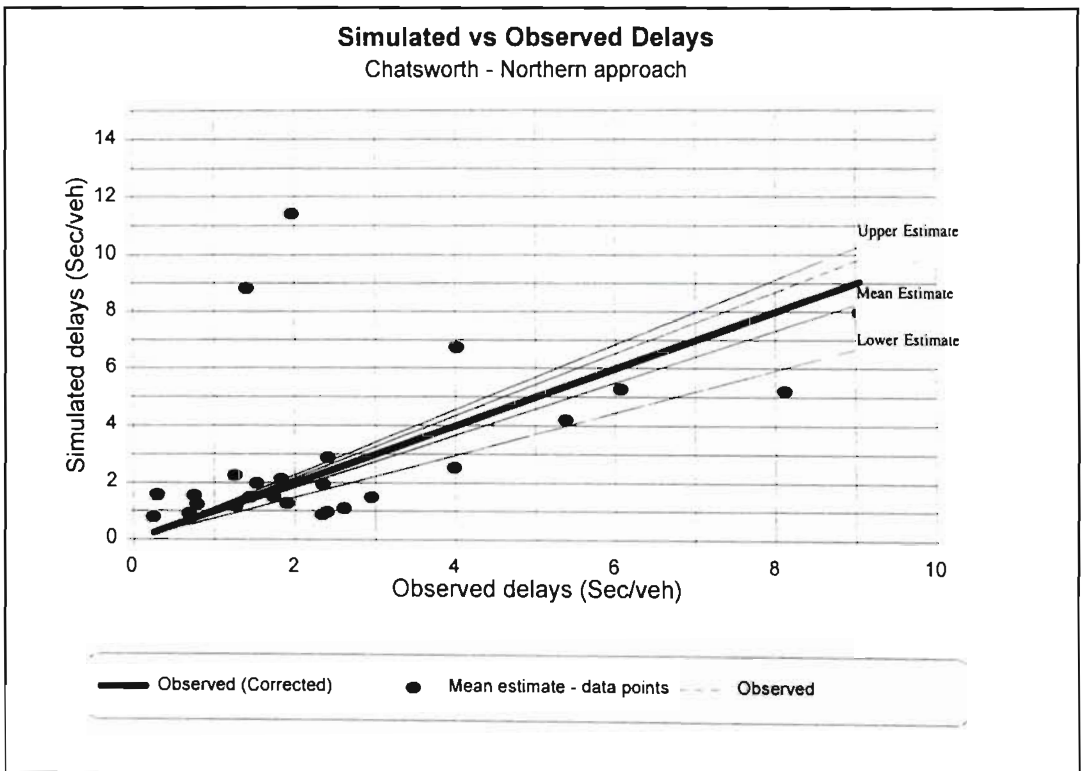


Figure 6.1: Simulated and observed delays at Chatsworth circle - northern approach

In Figure 6.1 the corrected observed regression line lies, close to the mean estimated regression, while both the observed and corrected observed lines lie well inside the minimum and maximum estimated lines. If the two outliers are ignored then the estimated mean data points indicate an upward trend.

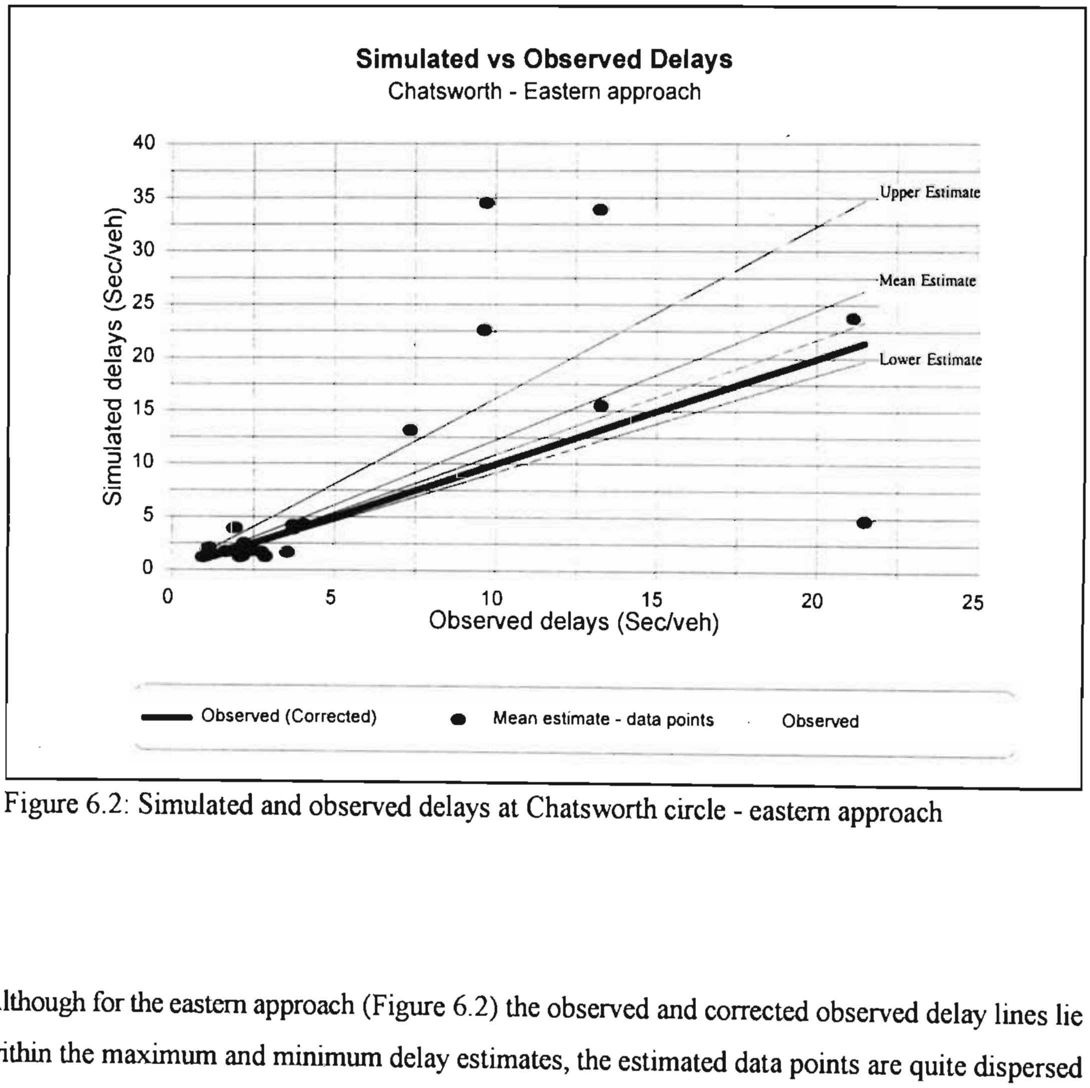


Figure 6.2: Simulated and observed delays at Chatsworth circle - eastern approach

Although for the eastern approach (Figure 6.2) the observed and corrected observed delay lines lie within the maximum and minimum delay estimates, the estimated data points are quite dispersed and show a low correlation to the best fit straight line. However, the delays are relatively high and it is expected that the higher the delays, the greater the variability in delay estimates.

For the southern approach (Figure 6.3), both the observed and corrected observed delay lines fall outside the estimated envelope with the estimated delays generally lower than the observed delays. However, if the two low outliers are ignored there would be a much better fit - for delay less than 10 seconds per vehicle.

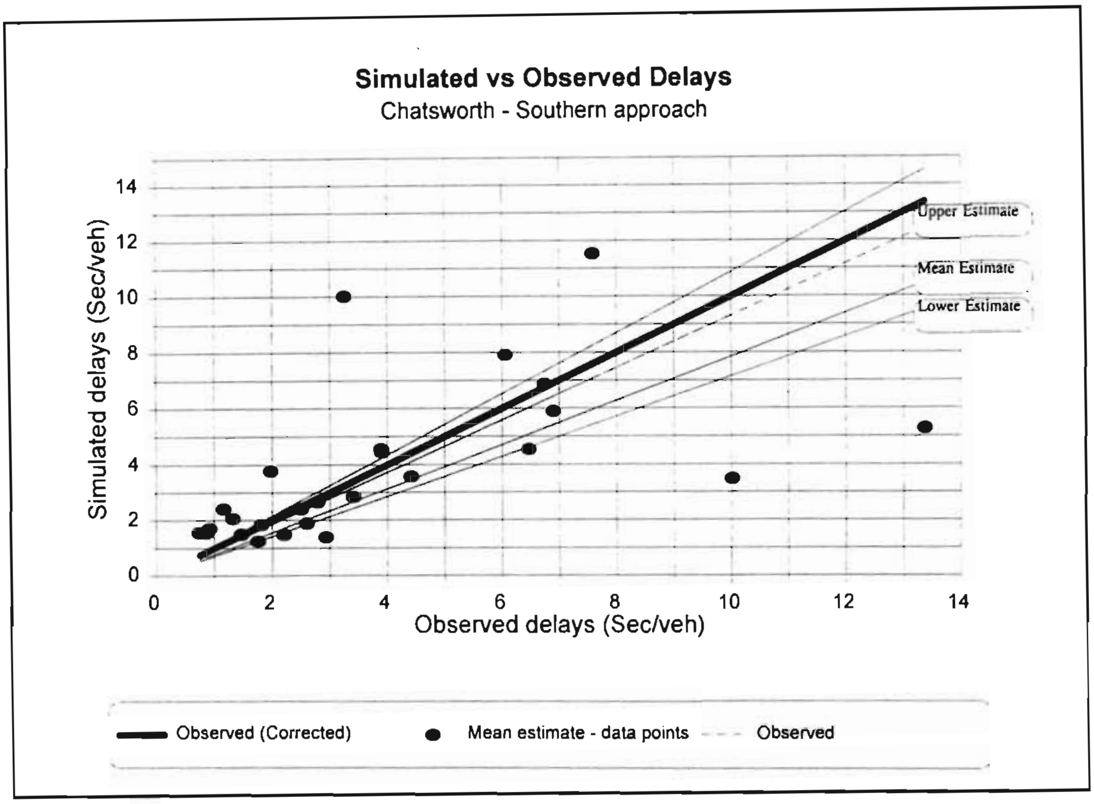


Figure 6.3: Simulated and observed delays at Chatsworth circle - southern approach

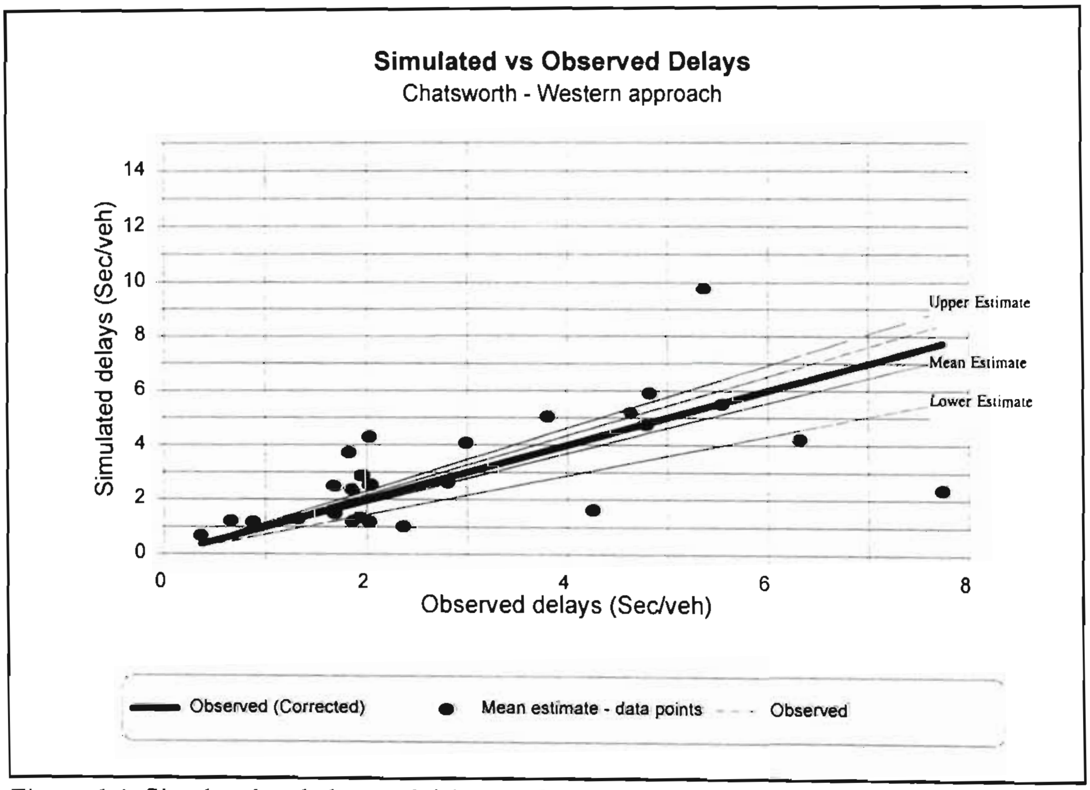


Figure 6.4: Simulated and observed delays at Chatsworth circle - western approach

For the western approach (see Figure 6.4) the observed data fall within the estimated envelope of delays, with the estimated mean delay being close to the corrected observed delay.

From the above it seems apparent that there is reason to conclude that the simulation program does model reality accurately. What is also evident is the effect of outliers on the regression lines, especially for the higher flows and for small data sets. To reduce some of the effects of the outliers all the data for Chatsworth were combined and a similar regression analysis as for the individual approaches was completed. The result of the analysis is shown in Figure 6.5.

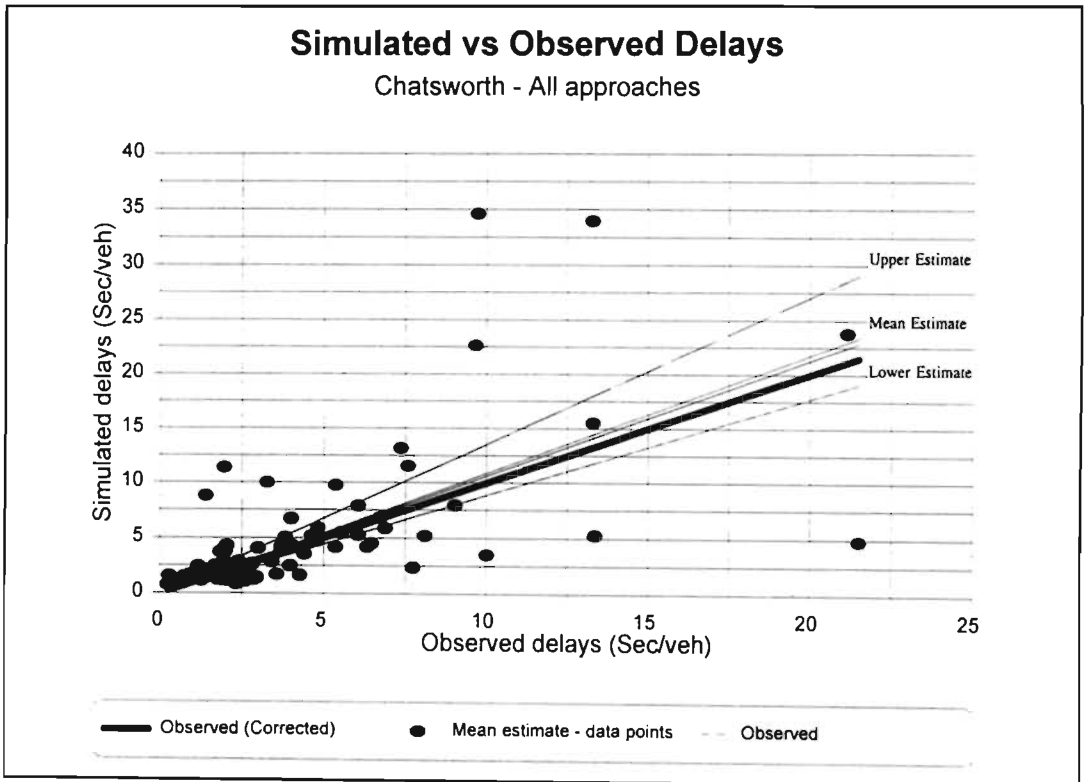


Figure 6.5: Simulated and observed delays at Chatsworth circle - all approaches

The uncorrected observed delay line and the mean estimated regression line are basically on the same line. The reason for the outliers can only be sought in the observations as the simulated values are the mean values of at least 15 simulation runs. It is possible that for the two or three upper outliers, the delay observations for those specific 15-minute intervals were low and vice versa for the lower outlier. Dismissing the validity of the simulation program given the above comparison of observed and simulated average delays per vehicle is not possible.

6.2.2 Queen Mary circle

For the reasons discussed in Section 6.1, only two of the four approaches at the Queen Mary circle were used for validation. A total of seven, 15-minute intervals (See Table 6.1) were used for the comparison. Each 15 minute interval resulted in an observed delay per approach and an estimated delay per approach. So from the two approaches and the seven data intervals, 14 data points resulted. For the individual approaches seven data points were available. With the input volumes of each 15 minute interval the simulation program was run at least 30 times, each time with a different random number seed. As for the Chatsworth circle, the following graphs show a line for the observed values, a mean estimated line and a minimum and maximum estimated line. The mean, maximum and minimum estimates are the results of a regression analysis on each of the 30 simulation runs.

Figures 6.6 and 6.7 show the results of the analysis for the western and southern approaches respectively.

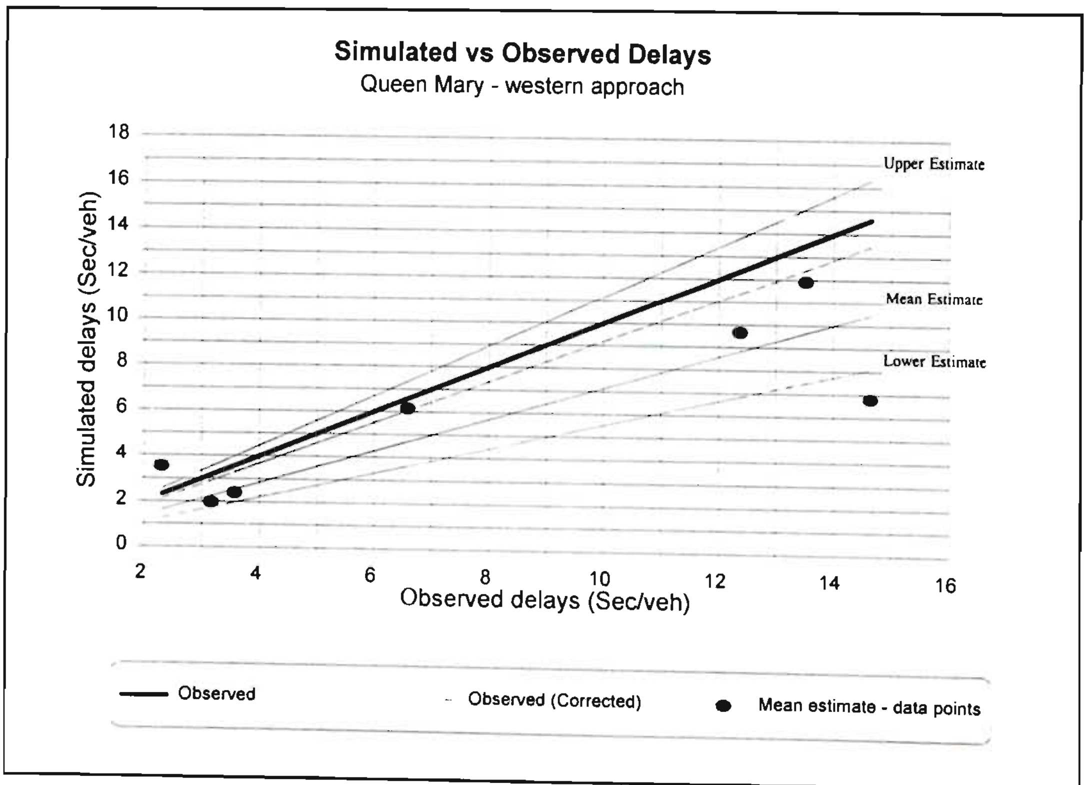


Figure 6.6: Simulated and observed delays at Queen Mary circle - western approach

For the western approach the estimated values are lower than the observed values although the corrected observed values are still within the upper boundary of the envelope (see Figure 6.6). The uncorrected observed regression line falls marginally outside the upper boundary of the envelope. From the data points it is clear that there is a reasonable positive correlation between the observed and mean estimated delay values.

The regression results of the observed versus estimated delay values for the southern approach as shown in Figure 6.7, shows a favourable comparison between the corrected observed delays and the mean estimated delay values. However, there is one outlier in the estimated results which pulls the regression lines up and towards the observed regression lines. If this outlier is ignored, the results would be similar to that of the northern approach, with the observed regression lines around the upper envelope boundary. Again, as for the northern approach, there is a reasonable positive correlation between the observed and estimated delay values, if the outlier is ignored.

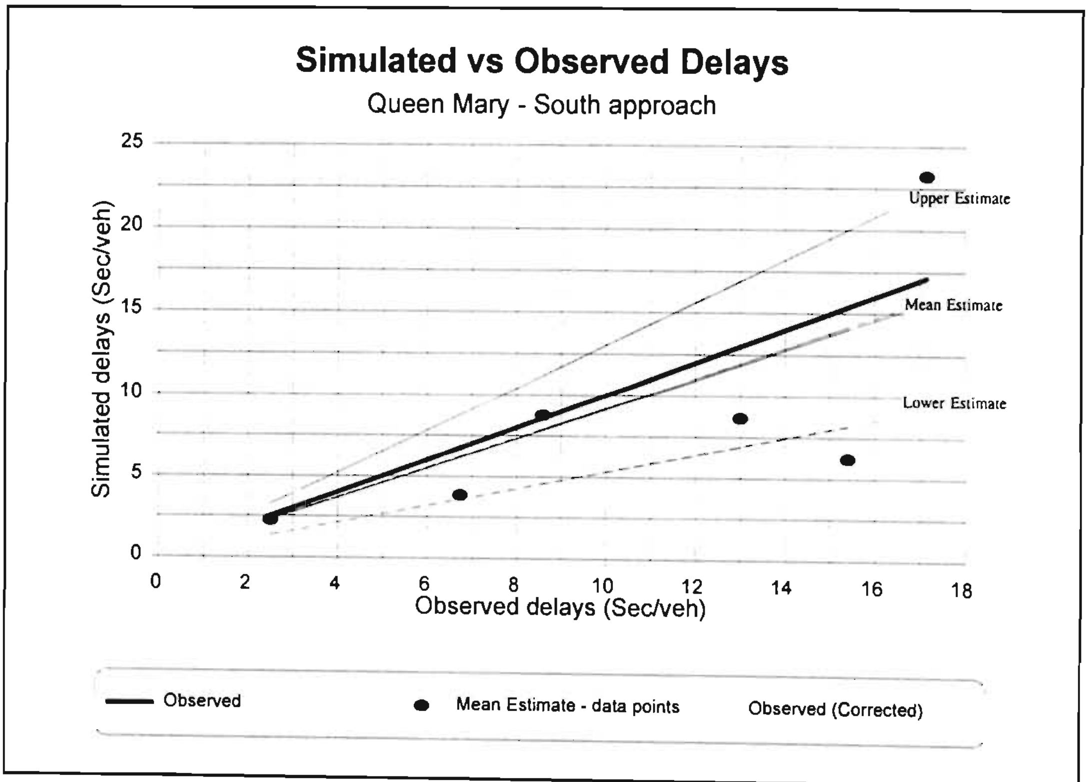


Figure 6.7: Simulated and observed delays at Queen Mary circle - southern approach

6.2.3 Pinetown circle

For the Pinetown circle only one approach was suitable for use in the validation process. Altogether twelve 15-minute intervals (See Table 6.1) were used for the comparison. Each 15-minute interval resulted in an observed delay per approach and an estimated delay per approach. For this single approach being used in the validation, twelve data points were available. With the input volumes of each 15-minute interval the simulation program was run at least 30 times, each time with a different seed number for the random number generator.

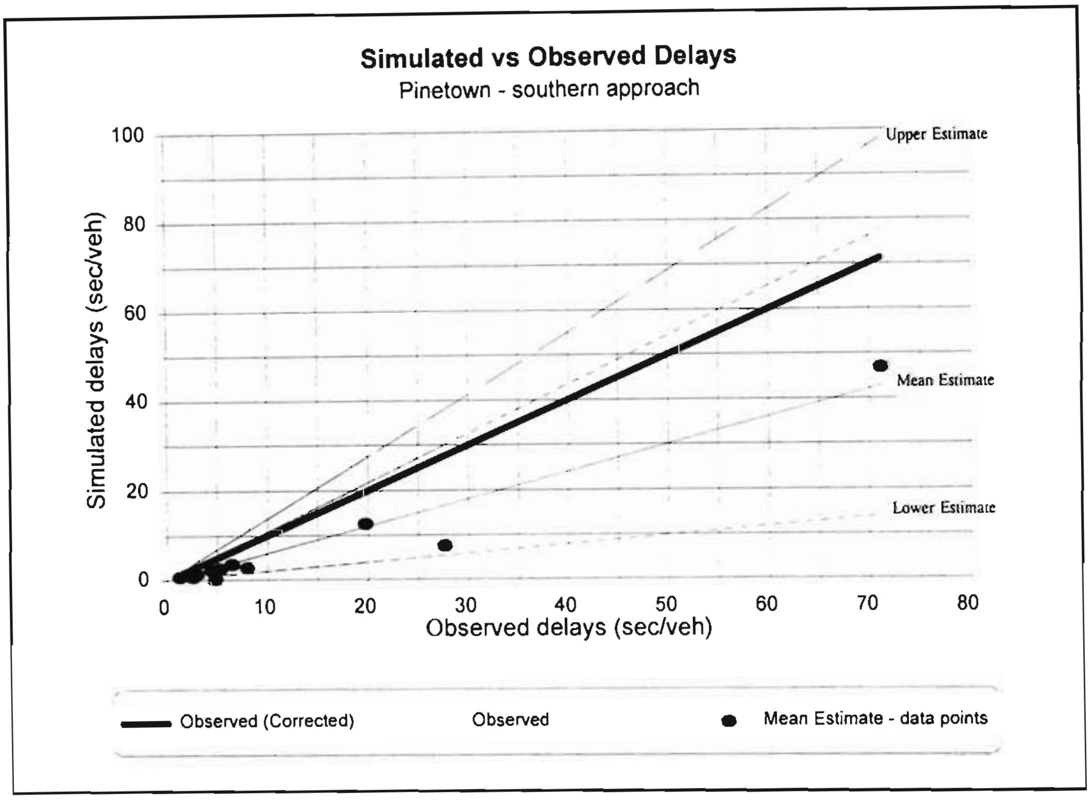


Figure 6.8: Simulated and observed delays at Pinetown circle - southern approach

As for the previous two circles Figure 6.8 shows a regression line for the observed values, a mean estimated line and a minimum and maximum estimated line. The mean, maximum and minimum estimates are the results of a regression analysis on each of the 30 simulation runs.

From Figure 6.8 it is clear that again there is a reasonable comparison between observed and estimated vehicular delays. The one data point (± 70 seconds per vehicle) where high delays were observed is the reason for the wide spread of the estimated envelope. Under these flows, because of random variations, a wide range of delays is possible. It only takes one slow driver waiting for a long time for a gap, to cause excessive delays. The next slow driver might on arrival find an acceptable gap much sooner. Although the observed regression lines are higher than the mean estimated line, they are well within the estimated envelope, and again there is sufficient reason not to reject the simulation results.

6.2.4 Combining all data

The next step in the validation process was to combine the data from all the circles for comparison of observed and estimated delays. Because of the lack of data at some circles and the variability of the observed and estimated delays, the data were classified into different groups. The classification was conducted according to the input and circulating traffic volumes. First the data were grouped according to similar entry volumes and then the data in these groups were in turn classified according to equivalent circulating volumes. The classification for both the entry and circulating volumes was based on 100 vehicles per hour. As before, a straight line was fitted through the estimated data points to compare with the observed data points. As it is expected that the regression lines should intersect the y-axis at zero, all the lines were fitted forcing the y-intercept to zero. A statistical analysis was conducted on the x-coefficient or slope of the best fit regression line for the estimated delay values, to determine the 95% confidence interval for this parameter. The results of the regression analysis are given in Table 6.2.

Table 6.2: Regression results on grouped volume data for delay observations

			95% Confidence Interval	
	Value	t-Statistic	Lower	Upper
Correlation coefficient	0.60			
Standard error	0.22			
Observations	32			
Intercept	0.00		0.00	0.00
X-coefficient	1.00	4.47	0.55	1.46

In Figure 6.9 the regression lines for the observed and estimated data points are shown together with the 95% confidence interval (lower and upper estimates) for the estimated regression line.

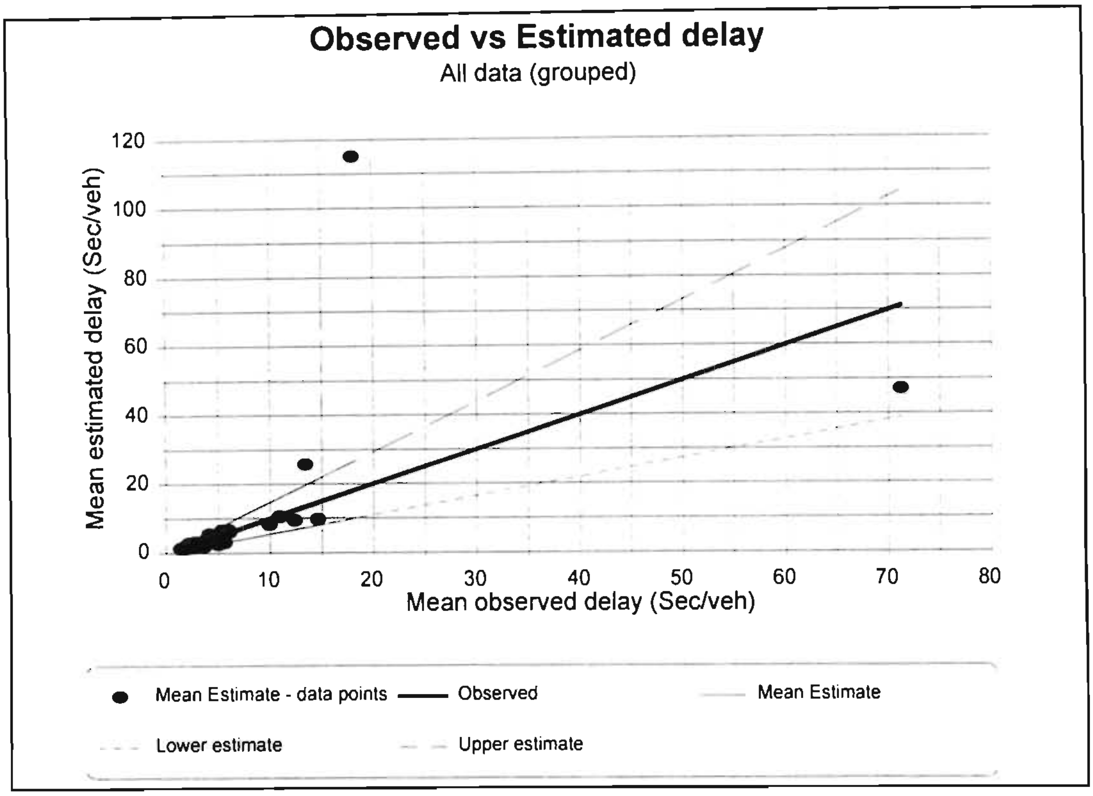


Figure 6.9: Observed vs estimated delays at all circles - grouped data

The observed data apparently falls within the confidence interval of the estimated regression lines, with the observed line falling virtually on the mean estimated line. However, what seemed suspicious, because of the two outliers, is the fact that there is no close correspondence between the mean values even after the data were grouped and the averages for the groups were used for the comparison. It was noticed that the one outlier, the 71 seconds per vehicle delay which was observed at the Pinetown circle, has a significant effect on the regression results. Hence, the two outliers were excluded and the analysis repeated. The regression results are summarized in Table 6.3 with the regression lines shown in Figure 6.10.

Table 6.3: Regression results on grouped data - excluding outliers.

			95% Confidence Interval	
	Value	t-Statistic	Lower	Upper
Correlation coefficient	0.77			
Standard error	2.68			
Observations	30			
Intercept	0.00		0.00	0.00
X-coefficient	1.00	12.57	0.84	1.16

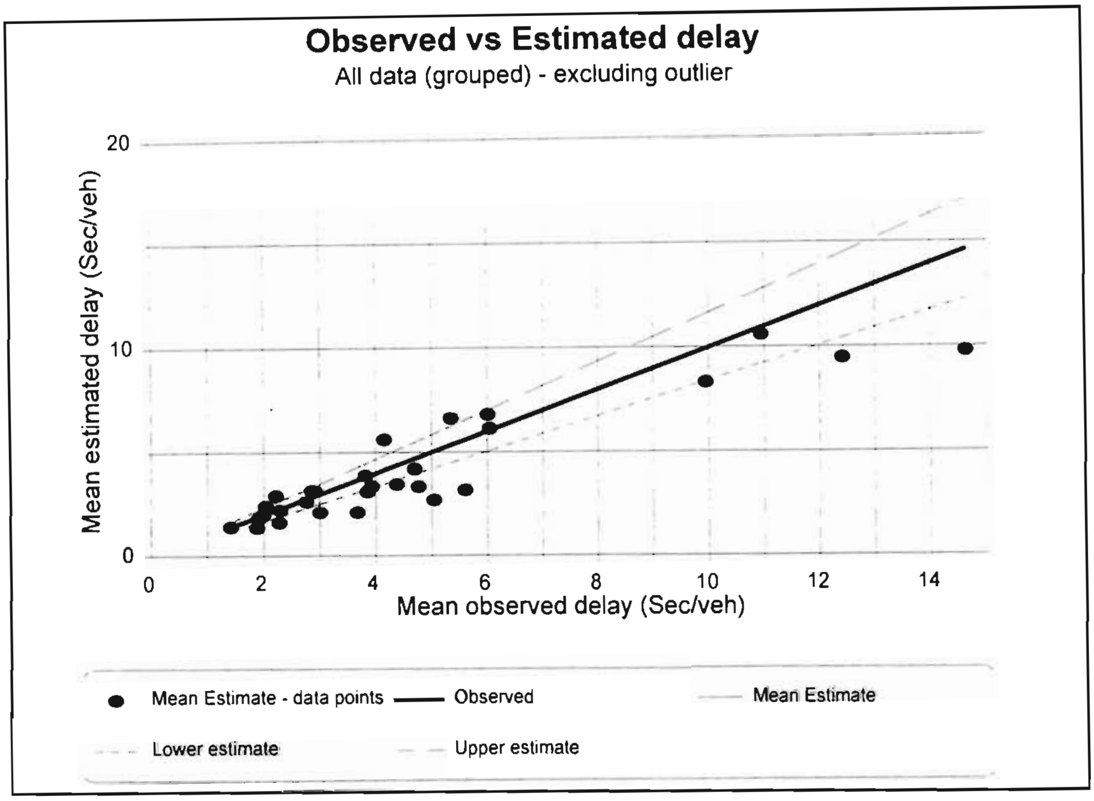


Figure 6.10: Observed vs estimated delays at all circles - grouped data, excluding outliers

With the two outliers excluded, the correlation between the observed and estimated results are obviously much closer, but the mean estimated regression line is virtually, once again, on top of the observed line. This confirms that the effect of the outliers does not improve the results and that the good comparisons in Figure 6.9 are not because of a statistical quirk caused by the outliers. This further strengthens the notion of not rejecting the simulation model as a true model of reality.

A final comparison to check the validity of the simulation model, was to compare the observed and simulated data, not against the observed delay data points, but to compare them using the degree of saturation on each approach as the independent variable. However, to establish a degree of saturation (approach flow divided by capacity: X = Q/C) an estimate of the capacity of the approach is required. Not having reliable models which can predict capacities of traffic circle approach roads under local conditions, it was decided to use an estimate of capacity based on the British Method (see (2-11)). This is a straight line relationship between entering capacity and circulating volume, and for this exercise the following equation was used:

$$Q_e = 1300 - 0{,}8\,Q_c \tag{6-1}$$

where Q_e is the capacity of the approach and Q_c the circulating volume in vehicles per hour. With zero circulating flow the entry capacity is thus 1300 vehicles per hour, while the capacity reduces to zero with 1625 vehicles circulating per hour. The exact values of 1300 and 0,8 are not critical for this comparison as it is a relative comparison and both the observed and estimated delays per vehicle are plotted against the same degree of saturation. However, the degree of saturation would probably not relate to the actual saturation. The analysis was conducted on the classified data, where the classification was according to the entering and circulating volumes. As it is expected that the delays increase exponentially with degree of saturation, an exponential curve was fitted through the observed and estimated data points. The results of this comparison are shown in Figure 6.11

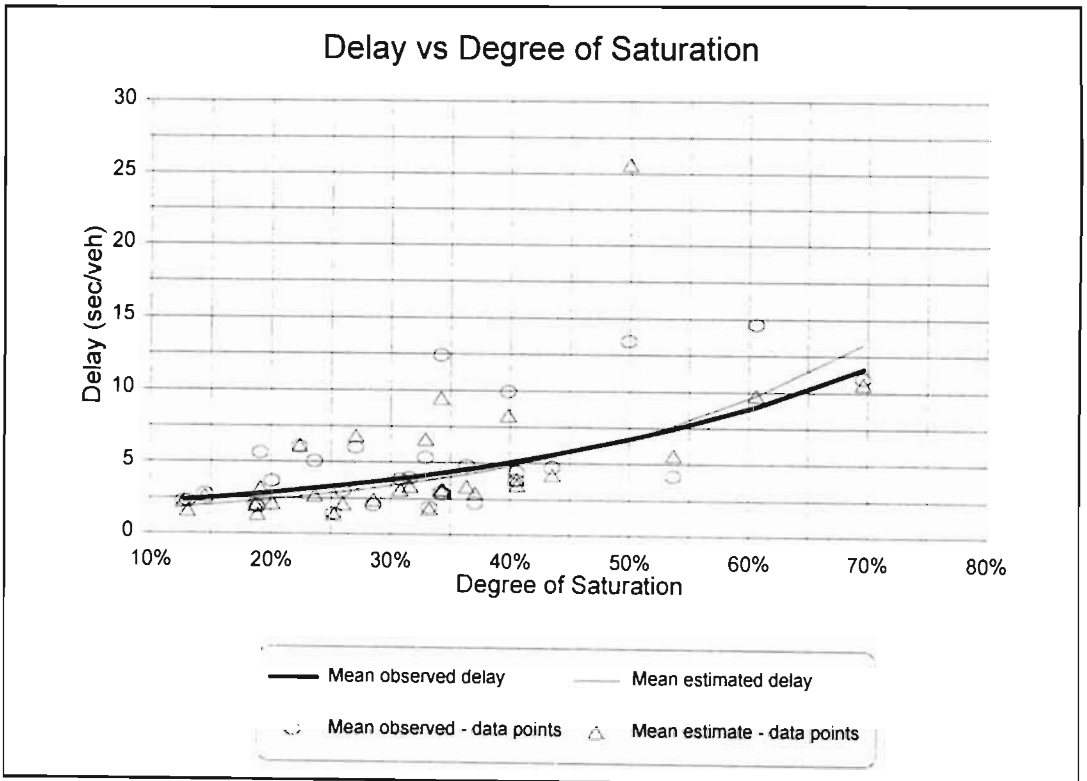


Figure 6.11: Delay versus Degree of Saturation (Grouped data, excluding outliers)

Although the degree of saturation is not calibrated against any actual data, the values for calculating capacities have been selected carefully at least to give realistic results. The results in Figure 6.11 show that for low degrees of saturation (less than ± 50%) the estimated values are slightly lower than the observed values and for higher degrees of saturation (greater than ± 50%) the estimated delays are slightly greater than the observed delays. This is an interesting tentative finding and supports the variable critical gap theory on which the Australian method (Troutbeck, 1989) is based. The critical gap observations were conducted for degrees of saturation in the range 30% to

60% and hence the relatively favourable comparison of observed and estimated delays over that range. For lower degrees of saturation it is possible that drivers have larger critical gaps and hence a larger mean delay per vehicle, while for higher degrees of saturation the critical gap reduces and hence reduces the mean delay per vehicle.

This last comparison again supports the acceptance of the simulation model, but underlines the fact that it is only accurate over the range for which it was calibrated. To increase this range, more work is required to determine the variation of the critical gap with a change in degree of saturation. This could be completed either directly by more field surveys, or indirectly by using the simulation model to calibrate the variation of the critical gap, while measuring it against observed delays. However, the direct method would be the preferred method.

6.3 Summary

In this chapter the validation process and a comparison of the TRACSIM delay estimates with field observations at three different circles are discussed. The comparisons are made by individual approach and by circle in the case of Chatsworth. For a final comparison all the data are combined and categorized to remove some of the statistical variation. Because of statistical variation the simulation runs were made with different seed numbers for the random number generators to produce an envelope of delay estimates with which the observed delays could be compared.

In all cases except for the southern approach to the Chatsworth circle, the observed delays fall within the estimated envelope and often close to the mean estimated line. When all the data are combined and categorized the comparison between the observed and mean estimated line is virtually perfect, whether or not the outliers are considered. There is thus no apparent reason for rejecting the simulation model and it is concluded that the model accurately estimates delays at traffic circles.

From the combined data and doing a comparison between delays and degree of saturation of the approach, an interesting finding arose. Although using an estimate of capacity with which the degree of saturation of the approach is calculated, the data indicate that the estimated delays are lower than the observed delays for low degrees of saturation and vice versa for high degrees of saturation. The simulation model used a fixed critical gap for each driver which was observed at average degrees of saturation and this is where the model is most accurate. This then indicates that there is reason to believe that the critical gaps should vary with degree of saturation or with conflicting vehicle volumes, and confirms the method proposed by the Australians (Troutbeck, 1989). However, a lack of data and the unavailability of circles to study this in South Africa prevents the verification of the exact nature of the relationship between mean critical gaps and conflicting flows.

CHAPTER 7: GAP ACCEPTANCE BEHAVIOUR AT TRAFFIC CIRCLES

Traditionally gap/lag acceptance models are based on time gaps/lags. However, from the observations made during the course of the development of the simulation model it was postulated that using another gap/lag acceptance model, such as distances, might be more appropriate. The initial simulation model, before being calibrated using critical time gaps, was based on a fixed critical area. During the validation process it was postulated that gap acceptance behaviour can be modelled by employing a probability of acceptance based on the position of the conflicting vehicle in the circle. Every gap was evaluated in isolation, even successive gaps available to one driver were evaluated randomly. This approach did not seem to be successful as the probabilities of acceptance had to be increased significantly more than the observed values before any correlation between observed and simulated values occurred.

In this chapter the gap acceptance approach based on a critical distance is discussed. Firstly, the observations which were made to investigate the critical distances, the analysis to obtain the critical distance distributions and also the comparison between estimated delays using the critical distance approach and the observed delays are discussed. Secondly this chapter reports on an approach to derive the critical distances from the geometric features of the circle. This negates the use of expensive surveys to find these parameters. To justify the use of the proposed method the observed delay values are again compared to the simulated delays using the critical distance gaps obtained from the geometric layout of the circles.

7.1 Gap acceptance based on distance

This section discusses the observations at the different circles and the methods used to observe gap acceptance based on distance. It also reports on the methods of analysis to estimate the mean critical gaps and the standard deviations of the gap distributions and finally TRACSIM is used to test the proposed gap acceptance model based on critical distances.

7.1.1 Distance gap/lag observations

Similar to the observations of gap acceptance behaviour based on **time gaps/lags** (see section 5.3.2), gap acceptance behaviour based on **distance gaps/lags** can be observed through recording each distance gap/lag as it is accepted or rejected. However, the observation and recording of distance gaps are much more demanding than the observation of time gaps. This is simply because it is easier and faster to record gap times with a stop watch than it is to record distances. Possible approaches to observed distances gaps/lags are (i) to physically mark the roadway with distance markers or (ii) to record the time gap/lag for each acceptance or rejection and also the speed of the conflicting vehicle. With the speed of a vehicle and the gap/lag time, a distance can be calculated. These methods are complicated, manpower intensive and marking the roadway can cause disturbances to drivers and possible behavioural changes.

Usually, during the analysis of statistical data such as time or distance gaps, the data are grouped into intervals (Benjamin and Cornell, 1970). Therefore, to simplify the data capturing, the gap/lag distances were initially recorded in distance intervals and not as exact distances for each vehicle. The interval number (see Figure 7.1) in which the conflicting vehicle was when the gap/lag was accepted or rejected, was recorded. The distance intervals were chosen to coincide with specific geometric features in the circle, because it was suspected that the geometric layout of the circle determines the gap/lag acceptance behaviour.

From the gap/lag analysis based on times (see Sections 5.5) there is evidently reason to expect that at traffic circles there is a difference between gap/lag acceptance behaviour in circulating and entering traffic streams. Hence, a distinction was made firstly between gaps and lags and secondly whether these gaps/lags were presented in the entering or the circulating traffic.

In Figure 7.1 the different distance intervals or positions of recording of the distance gaps are defined for an entering vehicle on the approach as shown. The positions in which a possible conflicting vehicle may be while circulating or entering the circle are defined by the radial lines and labelled according to the numbers as defined in the figure. After careful observations at a number of traffic circles it was decided to define the positions of the radial lines as shown in Figure 7.1. The first line (OA) upstream of the entering vehicle intersects the corner of the island ("a") of the approach road while the third line (OB) goes to the corner ("b") of the first upstream approach road. The second line (OC) bisects the angle between lines one and three.

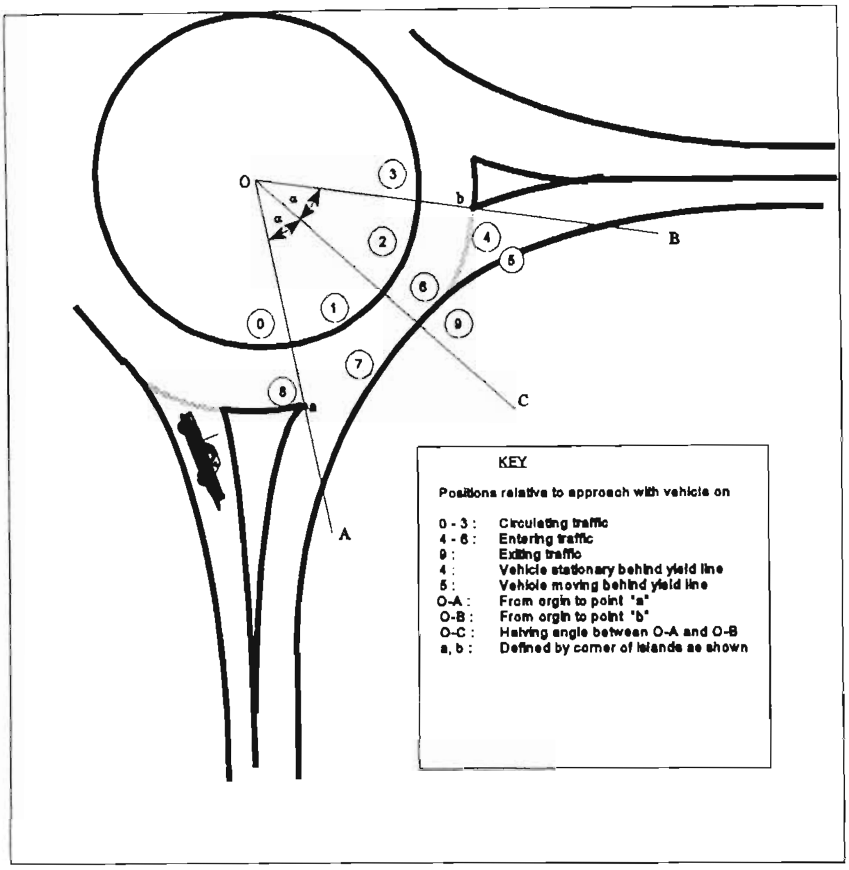


Figure 7.1: Positions in circle for gap/lag observations

The frequency analyses of the observed gaps according to the different positions of the next conflicting vehicle are summarized in Table 7.1 and the summary for the observed lags are given in Table 7.2.

Table 7.1: Frequency analysis of gaps according to positions

Posi-Tions	Chatsworth			Kensington			Queen Mary			Pinetown		
	Frequency		Prob. of Accept	Frequency		Prob. of Accept	Frequency		Prob. of Accept	Frequency		Prob. of Accept
	Reject	Accept		Reject	Accept		Reject	Accept		Reject	Accept	
	Circulating											
0	151	0	0.0%	1	0	0.0%	123	0	0.0%	24	0	0.0%
1	15	1	6.3%	14	0	0.0%	112	1	0.9%	70	0	0.0%
2	7	7	50.0%	4	4	50.0%	69	29	29.6%	70	34	32.7%
3	2	47	95.9%	2	17	89.5%	14	85	85.9%	4	45	91.8%
	Entering											
4	2	4	66.7%	4	4	50.0%	1	12	92.3%	0	1	100.0%
5	31	43	58.1%	117	25	17.6%	14	19	57.6%	9	8	47.1%
6	99	31	23.8%	126	4	3.1%	52	1	1.9%	39	3	7.1%
7	195	22	10.1%	73	1	1.4%	61	0	0.0%	32	0	0.0%
8	6	0	0.0%	0	0	0.0%	0	0	0.0%	0	0	0.0%
	Exiting											
9	0	2	100.0%	2	17	89.5%	2	18	90.0%	0	0	100.0%

Table 7.2: Frequency analysis of lags according to positions

Posi-Tions	Chatsworth			Kensington			Queen Mary			Pinetown		
	Frequency		Prob.	Frequency		Prob.	Frequency		Prob.	Frequency		Prob.
	Reject	Accept	of Accept	Reject	Accept	of Accept	Reject	Accept	of Accept	Reject	Accept	of Accept
	Circulating											
0	97	2	2.0%	9	2	18.2%	43	0	0.0%	12	0	0.0%
1	41	13	24.1%	16	13	44.8%	41	6	12.8%	52	0	0.0%
2	12	28	70.0%	8	28	77.8%	19	54	74.0%	36	41	53.2%
3	9	145	94.2%	0	145	100.0%	0	35	100.0%	3	51	94.4%
	Entering											
4	0	12	100.0%	1	12	92.3%	0	1	100.0%	0	0	100.0%
5	6	82	93.2%	82	52	38.8%	7	15	68.2%	3	11	78.6%
6	77	96	55.5%	96	13	11.9%	20	8	28.6%	8	1	11.1%
7	158	58	26.9%	58	0	0.0%	23	1	4.2%	19	0	0.0%
8	108	0	0.0%	24	0	0.0%	0	0	0.0%	0	0	0.0%
	Exiting											
9	0	256	100.0%	2	50	96.2%	1	75	98.7%	0	0	100.0%

In these tables a probability of acceptance was inferred from the previous observations wherever no observations were made for that specific position. For example, very few observations at position 8 were made, and in most cases inferring a zero probability of acceptance seemed reasonable. The data for gaps and lags are also summarized graphically in Figures 7.2 and 7.3 respectively.

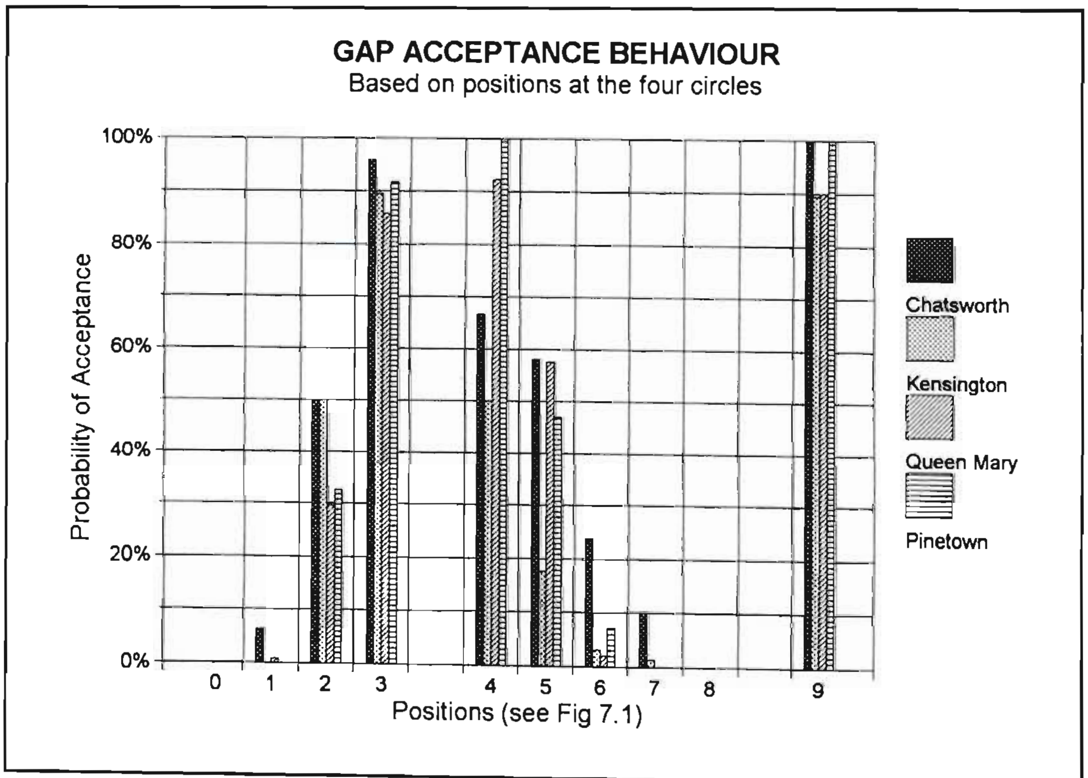


Figure 7.2: Gap acceptance according to positions

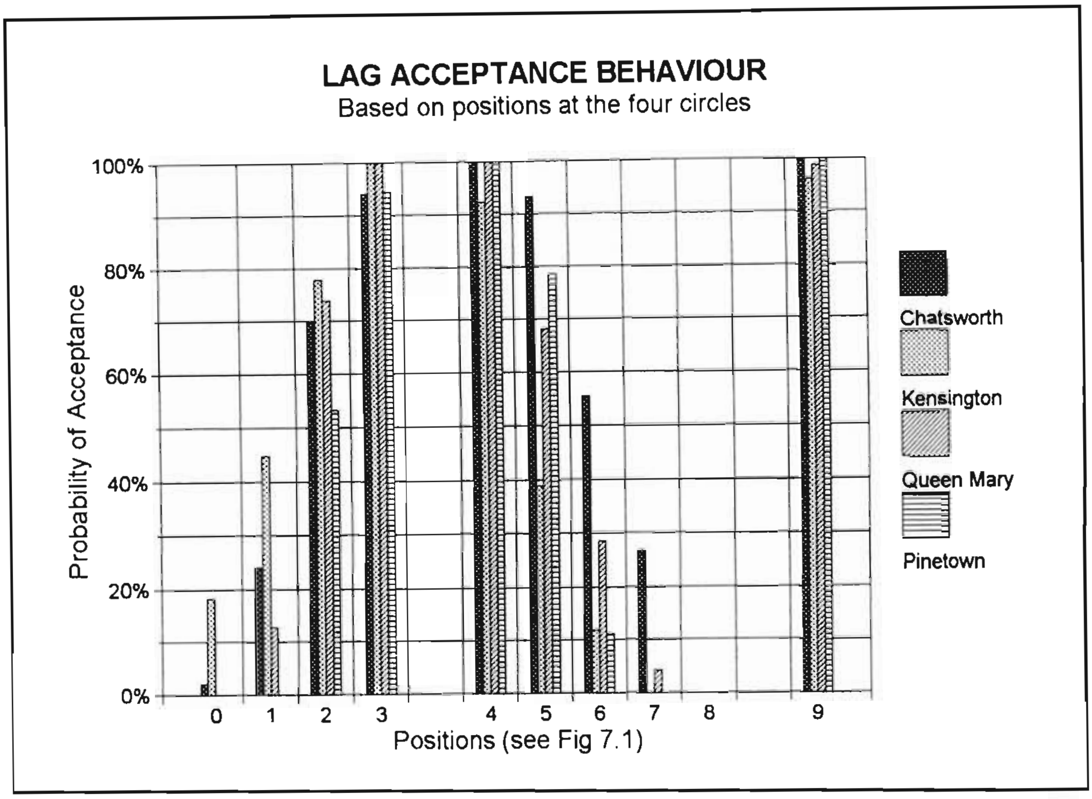


Figure 7.3: Lag acceptance according to positions

Although the trends at all the circles are similar, the gap/lag acceptance behaviour at the Kensington circle is significantly different, specifically for entering traffic and more so for lags in the entering traffic. The reason for this is that the observations at Kensington were made from the northern and western approaches which gave way to predominantly entering traffic from the previous approaches which in turn had little conflicting traffic to give way to. The traffic from the previous approaches usually then entered at reasonable speeds causing the traffic under observation to give way even if the conflicting vehicles were still behind the previous yield line. This indicates the effect of origin-destination patterns on gap/lag acceptance behaviour and the possible effect it can have on the capacity of an approach.

7.1.2 Analysis of gap acceptance data

Similar to the time based gap/lag analysis (see section 5.5) a normal distribution was fitted to the observed frequencies of gap and lag acceptance behaviour in an effort to determine a critical distance. This was done only for the circulating traffic as the entering traffic do not arrive at constant speeds and sometimes a stationary vehicle on the previous approach can cause a conflict. The normal distributions were fitted using probit analysis (Finney, 1971) as explained in Chapter 5. The estimates for the mean (probit of zero) and standard deviations (inverse of the slope of probit line) of the normal distributions were obtained from the probit analysis and are summarized in Table 7.3. No analysis was conducted for the Kensington circle because Kensington was not used for validation and therefore cannot be tested with the simulation program.

Table 7.3: Results of critical distance gap and lag analysis

Circle	Mean (metre)		STD (metre)		Sample Size
	Estimate	Error (95% Conf)	Estimate	Error (95% Conf)	
	GAPS				
Chatsworth	39.30	0.92	7.13	0.09	230
Queen Mary	24.49	0.30	3.18	0.07	433
Pinetown	29.32	0.19	1.49	0.09	248
	LAGS				
Chatsworth	35.95	0.98	9.29	0.07	347
Queen Mary	20.59	0.45	3.25	0.10	198
Pinetown	28.55	0.25	1.75	0.10	195

As expected, the mean critical distance lag is less than the mean critical distance gap for all the circles. There is a significant difference in the standard deviation of the three circles. This is due to the number of data points which were included in the probit analysis. At the Pinetown circle for instance, the probability of acceptance at both positions 0 and 1 are zero percent. These data points cannot be included in the probit analysis because the probit value of zero is infinity. There are thus fewer points to fit a straight line through and hence a better fit with a better standard deviation.

Another point to note is that the estimated critical distance gap/lag does not seem to be related to the diameter. The circles are listed in the table according to a decreasing diameter. The Pinetown circle for instance has a smaller diameter than the Queen Mary circle, but both the critical distance gap and lag are significantly greater.

7.1.3 Simulation results

Subsequent to the data collection, the next step was to change the gap/lag acceptance model in the simulation program to be based on critical distances, rather than critical times. The simulation model could then be employed to evaluate the proposed gap acceptance model based on critical distances. However, prior to changing the model to evaluate gaps/lags on a distance basis, it was used to evaluate gap acceptance based on probabilities. For the areas/positions as defined in Figure 7.1 specific probabilities of acceptance/rejection were observed as summarized in Tables 7.1

and 7.2 for gaps and lags respectively. A vehicle/driver arriving on an approach was not assigned any critical gap/lag, but the probability of acceptance of the specific driver accepting a gap, given a conflicting vehicle in a specific area/position, was based on the observed probabilities of acceptance. This was engineered in the simulation model by generating a uniformly distributed random number and comparing that with the probability of acceptance for the specific position/area in which the conflicting vehicle was. If the random number was smaller, then the gap was accepted and vice versa. A driver/vehicle had no consistent behaviour and had no memory. Once a gap was rejected the next gap was considered completely independent of the previous one and it was possible that a smaller gap was accepted than what was previously rejected.

Comparing the estimated delays with the observed delays showed that this method of gap/lag acceptance completely underestimated delays. To obtain a positive correlation between the observed and estimated delays the probabilities of acceptance had to be reduced by at least 60%. It was concluded that this method of gap/lag acceptance behaviour based on observed probabilities does not result in favourable delay estimates, and hence was not pursued any further. The reasons for the low estimates are not obvious and should be investigated, with a larger data base. This was however not pursued as part of this thesis.

However, what was pursued was the method of gap/lag acceptance based on critical distances. The simulation model was changed to incorporate a gap/lag acceptance model based on critical distances. As previously where gaps/lags were based on critical times, each vehicle was assigned a critical distance which was generated from normal distribution with a known mean critical gap/lag and a standard deviation. A vehicle arriving at the yield line would then consider gaps/lags based on its critical distance. Any gap/lag greater than its critical gap/lag would be accepted and any smaller gap/lag would be rejected. A difference was made between entering and circulating conflicting traffic. For circulating traffic to find a distance to the point of conflict is straight-forward, but for entering traffic it is slightly more complex.

From the observations it was noted that at some circles conflicting entering traffic upstream of the previous entry's yield line did sometimes cause the driver considering the gap/lag at the next entry, to reject it. This decision is not only based on the distance, but also on whether the conflicting vehicles are stationary at the previous yield line, starting to move or moving at a significant speed into the circle from the previous entry. To allow for this effect in the simulation model, the observed probabilities were used. It was mentioned in Section 7.1.2 that the critical distances were calculated using only circulating traffic. In the simulation model this critical distance was used for both entering and circulating conflicting traffic, but a further correction was made for entering gaps/lags.

Whenever the critical distance gap/lag of a vehicle waiting for a gap/lag was smaller than the gap/lag in the entering traffic, the gap was accepted. However, when the critical gap/lag was greater than the gap/lag presented in the entering traffic, it was not summarily rejected. The position of the conflicting vehicle was first evaluated. If this vehicle had already started to enter the circle, the decision was upheld and the gap rejected. However, if it was stationary at the yield line or moving upstream of the yield line a probability of acceptance was used to decide whether the gap should really be discarded or if sufficient reason exist to accept the gap. These probabilities were obtained from the observation as summarized in Table 7.1 and 7.2.

The gap acceptance model was thus not only based on any critical distance, but if this critical distance gap/lag exceeded the distance to the previous yield line and beyond, a second decision was employed to decide whether to accept or reject. This second decision which was based on the observed probabilities of acceptance, allowed for more gaps/lags to be accepted than would have been the case if only a critical distance had been used. This effectively reduced the estimated delays. A similar comparison between estimated delays and observed delays was made as is reported in Section 6.2 for the different approaches to the different circles. Figures 7.4 to 7.8 compare the observed and estimated values for the northern, eastern, southern, western and all approaches of the Chatsworth circle and Figures 7.9 and 7.10 for the Queen Mary and Pinetown circles.

As before, the model was run at least ten times for each set of approach flows at the three circles. Each of the ten simulation runs was started with a different random number seed and it was run for two minutes to "warm up" and then for 15 minutes from which the delay data were recorded. The mean value of the ten simulation runs was calculated and used as the mean estimated delay through which a straight line was fitted. The ten different runs were analysed to find the minimum and maximum estimates and a maximum and minimum regression line was fitted through these estimates. These minimum and maximum lines constituted the boundaries of the envelope in which the estimated delays lie. These mean, minimum and maximum regression lines are shown on Figures 7.4 to 7.8 to compare with the 45° observed delay line. Also shown in these figures are the data points for the mean delay estimates, through which the best fit mean line was regressed.

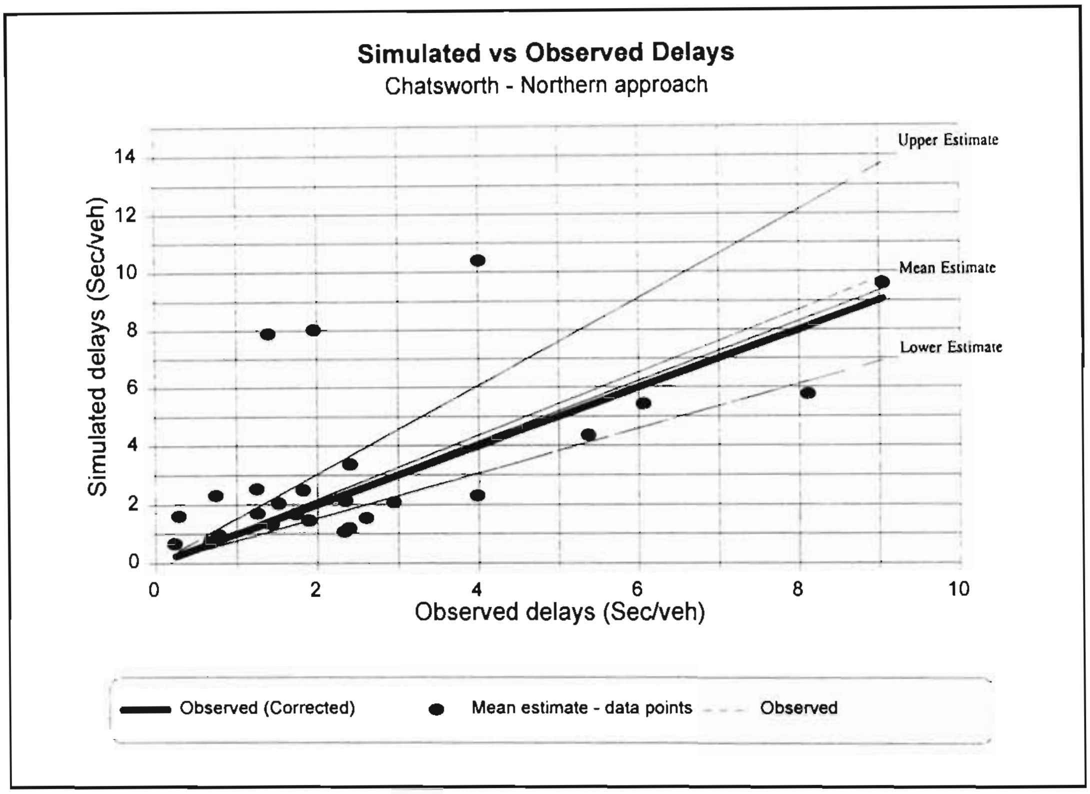


Figure 7.4: Observed and estimated delays at Chatsworth circle based on critical distances : northern approach

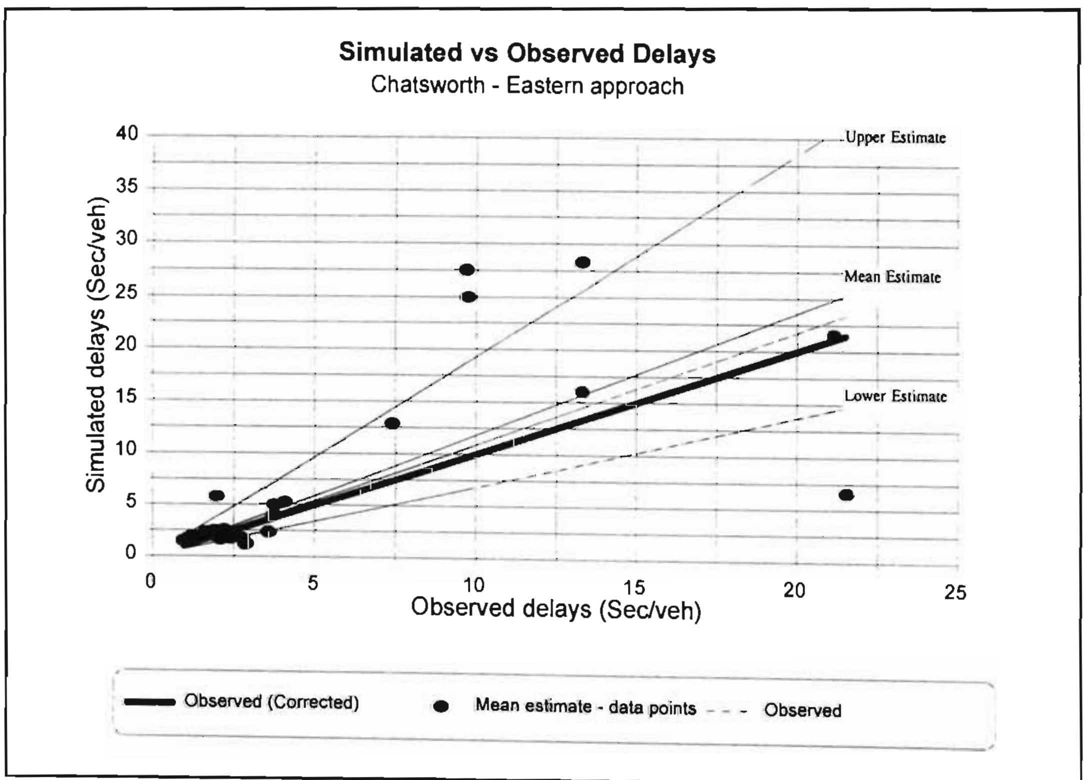


Figure 7.5: Observed and estimated delays at Chatsworth circle based on critical distances : eastern approach

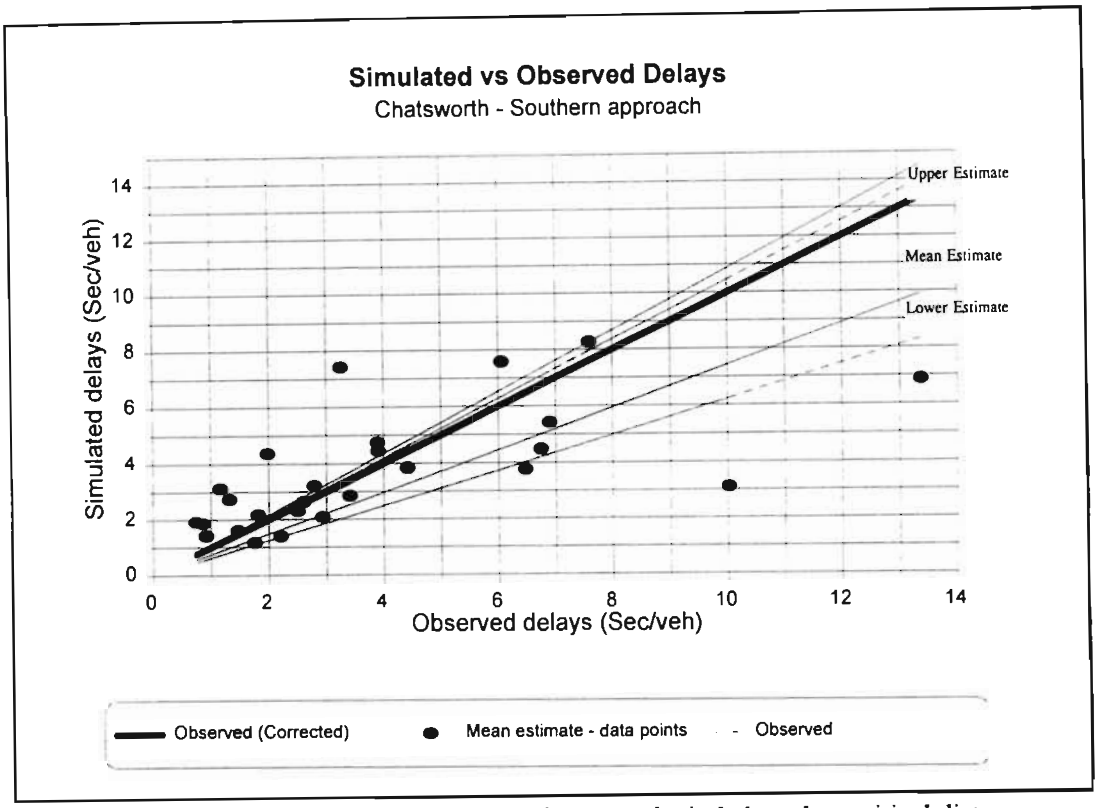


Figure 7.6: Observed and estimated delays at Chatsworth circle based on critical distances : southern approach

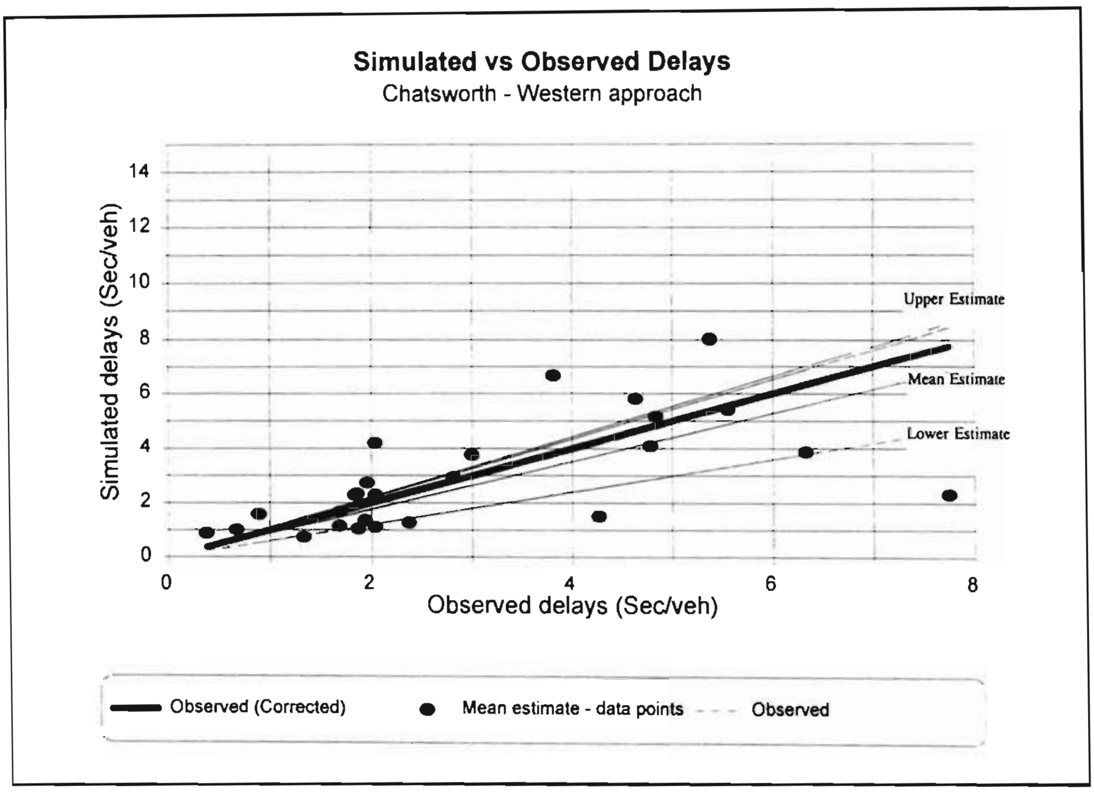


Figure 7.7: Observed and estimated delays at Chatsworth circle based on critical distances : western approach

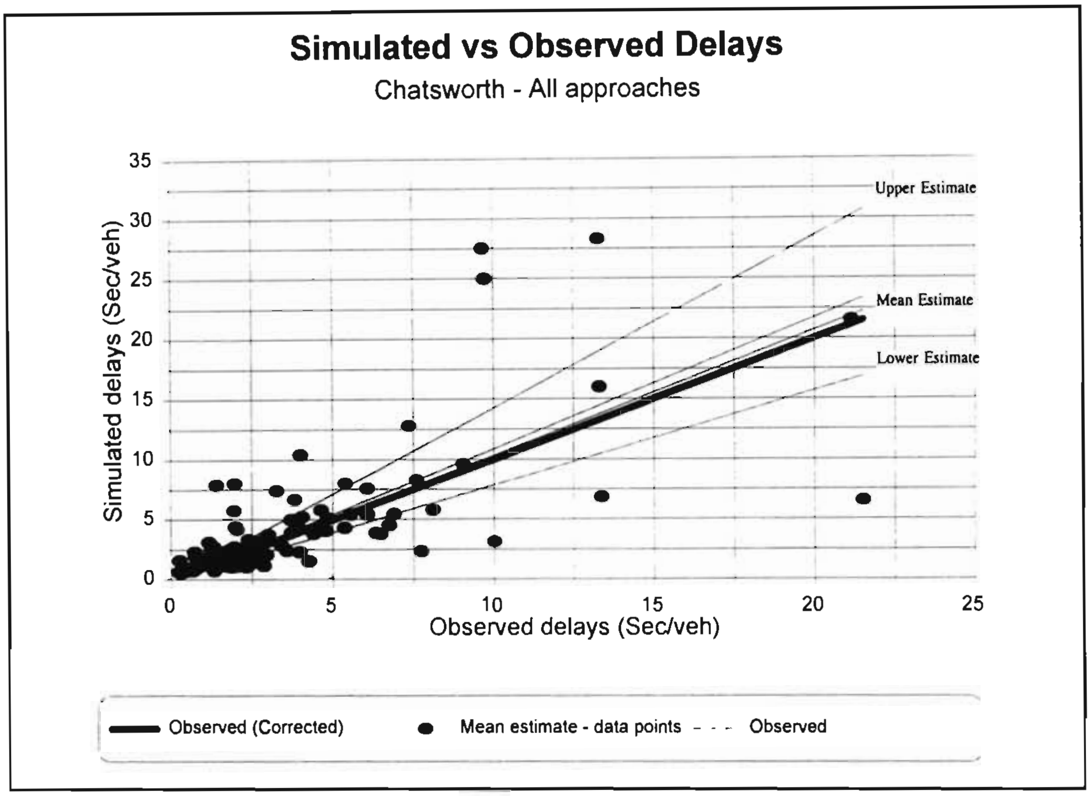


Figure 7.8: Observed and estimated delays at Chatsworth circle based on critical distances : All approaches

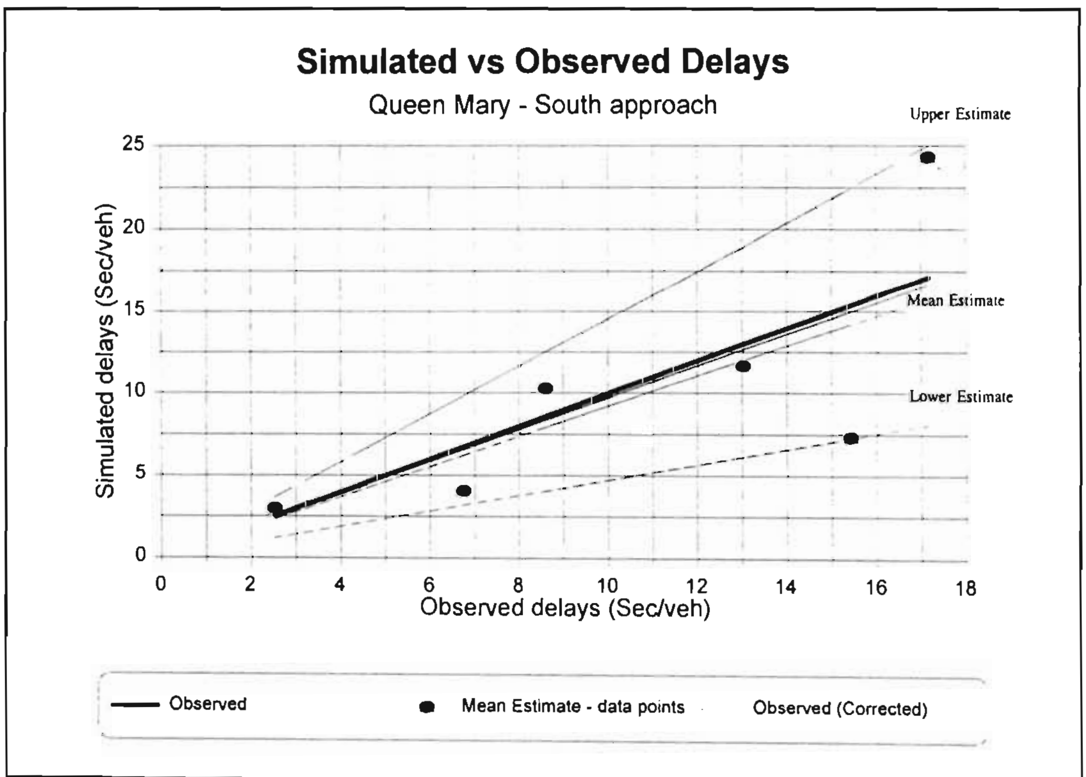


Figure 7.9: Observed and estimated delays at Queen Mary circle based on critical distances : Southern approach

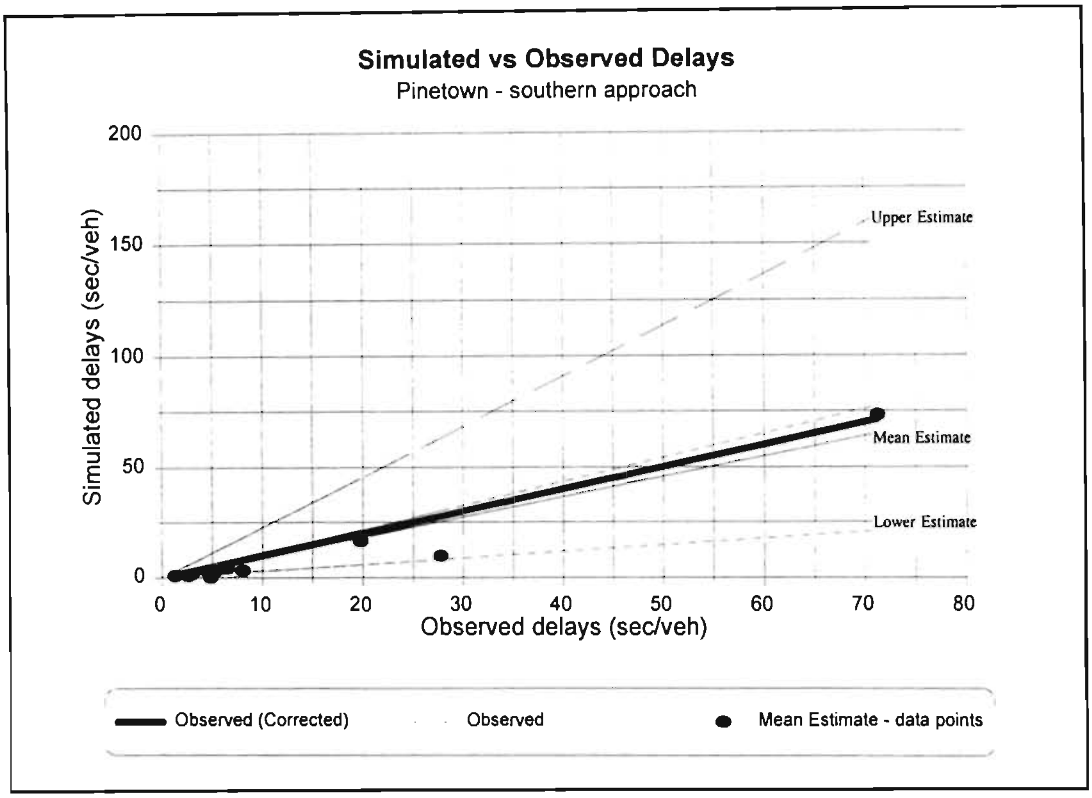


Figure 7.10: Observed and estimated delays at Pinetown circle based on critical distances : Southern approach

From these figures it is clear that the simulation model using critical distance for the gap acceptance process is as reliable if not better, as the same model with gap acceptance based on a critical time. For instance, the distance-based model provides a more accurate estimate of the delays on the southern approach to the Chatsworth circle than the time-based model (see Figures 6.3 and 7.6). Table 7.4 summarizes the slopes of the regression lines for the average estimates of the model for the above three circles. A slope of one (45° line) represents a perfect fit to the observed delays.

Table 7.4: Comparison of regression line slopes

Circle	Approach	Gap Acceptance Model	
		Time	Distance
Chatsworth	North	0.92	1.04
Chatsworth	East	1.23	1.18
Chatsworth	South	0.78	0.74
Chatsworth	West	0.92	0.88
Chatsworth	All	1.06	1.03
Queen Mary	South	0.91	0.97
Pinetown	South	0.60	0.92
	AVG	0.92	0.97

From Figure 7.4 to 7.10 and from Table 7.4 it is clear that the simulation model employing a gap acceptance model based on critical distances gives better estimates of the observed delays when compared to the same simulation model using a gap acceptance model based on critical times. From Table 7.4 it is obvious that in most cases, the slope of the regression line fitted through the mean estimates of delay is closer to unity. The exceptions are the southern and western approaches to the Chatsworth circle. However, the mean values of all the estimated slopes indicate a slight improvement when using the gap acceptance model based on distance.

If it is then assumed that the observed delays represent the actual delays which would be experienced under the prevailing traffic and roadway conditions, it can be concluded that a simulation model based on critical distances could result in more accurate estimates of traffic performance. Although the collection of gap-acceptance data based on distances can be a ponderous task, the gap-acceptance data as collected during this research and based on rather coarse positions in the circle, required significantly less effort. Even less effort is required than gap acceptance data based on times, because there is no need to time every gap/lag. Moreover, the critical distance gaps obtained from the analysis of the observed positions proved to be sufficiently accurate to improve the simulation model estimates. Therefore, it is not only possible to obtain more accurate simulation results with less effort during the data collection process, but it is possible that the gap acceptance process based on distances reflects the actual behaviour of drivers more accurately. Drivers have a better notion of distance and position of a conflicting vehicle than they have for the time gap available. Based on this it was decided to pursue the idea of a critical distance and to try and relate the critical distance to one or more of the geometric features of the circle. This is discussed in the next section.

7.2 Critical distance versus geometric layout of circle

From the positive results obtained from using distances as the criteria for the gap acceptance model in the simulation model, it was decided to pursue this method and to endeavour to find a way to easily identify a critical distance.

7.2.1 Distance versus angles in circle

This distance would obviously be related to a number of factors, such as amongst others, circulating speed, diameter and position of approach roads. Table 7.5 shows a comparison of the four circles, the circulating and angular speeds in each and the observed time and distance gaps converted to an angle. The definitions of the different terms are explained in Figure 7.11

Table 7.5: Comparison of geometric - and gap characteristics at the different circles

	Chatsworth	Queen Mary	Pinetown	Kensington	Units
Central Island Diameter	36.20	23.60	20.00	27.4	m
Central Island Radius	18.10	11.80	10.00	13.70	m
Circulating Radius	21.10	14.80	13.00	16.70	m
Speed	37.60	23.60	23.20	27.40	km/h
Speed	10.44	6.56	6.44	7.61	m/s
Angular speed	0.49	0.44	0.50	0.46	rad/sec
Angular speed	28.36	25.38	28.40	26.11	deg/sec
Mean observed time gap	4.57	3.96	4.21	4.30	sec
Distance based on gap	47.7	25.9	27.1	32.7	metres
Angle based on gap	2.26	1.75	2.09	1.96	radians
Angle based on gap	129.6	100.5	119.6	112.3	degrees
Mean observed distance	39.3	24.5	29.3	Not observed	m
Angle based on distance	1.86	1.66	2.25		radians
Angle based on distance	106.7	94.9	129.1		degrees

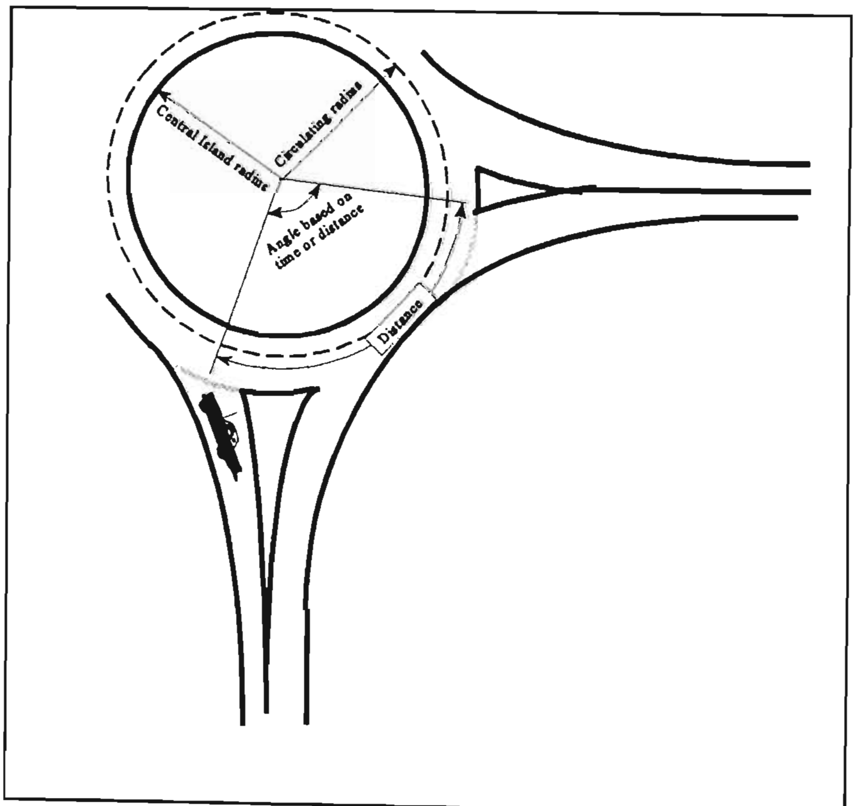


Figure 7.11: Definitions of terminology

An interesting point to note from Table 7.5 is the similarity of the mean angular speeds. These speeds vary between 0,44 and 0,50 radians per second or 25,4 to 28,4 degrees per second. The mean angular speed measured at Chatsworth - the largest circle - is virtually the same as the angular speed observed at the smallest circle, the Pinetown circle. Due to their geometric layouts, circles have the ability to regulate the speeds of vehicles moving through them.

The critical angles based on the observed mean critical time gaps vary between 100 and 130 degrees while the critical angles based on the observed critical distance gaps vary between 95 and 130 degrees. Although the ranges for these angles are similar, a significant difference is observed when angles for the individual circles are compared. For both the Chatsworth and Queen Mary circles, the critical angle based on the observed critical distance gaps is smaller than the critical angle based on the observed critical time gap. In the case of the Pinetown circle the opposite is true. This is due to the inaccuracy of the critical distance values as observed in the Pinetown circle. As explained in Section 7.1.2 the probit analysis for this circle is based on a reduced number of data points, which reduces the reliability of the results.

7.2.2 Proposed method of determining critical distance

These observed critical angles based on both times and distances were then compared with other relevant angles in the circle, which could have an influence on the gap acceptance decision. The only clearly defined angle which compared well with the observed angles is the angle between the nearside of the entry island curb where a vehicle is waiting to enter the circle and the centre line of the approach road of the previous entry to the circle (see Figure 7.12). The circulating distance obtained from this angle will take into account the speeds of circulating vehicles, because the circulating speeds at circles are a function of the circulating diameter. It will also take into account the position of entry and exit of conflicting vehicles at the previous approach.

This proposed method ("measured" critical distances) is based on observations at three circles, and takes into account data obtained from observed time gaps and observed distance gaps. To verify the "measured" critical distance method, the simulation model was again used with a gap acceptance model based on distances and the critical distances obtaining from actual geometric measurements of the critical angles (see Fig 7.12). The delays as estimated by the simulation model could then be compared with the observed delays obtained at the various circles.

According to the analysis of the observed distances (see Table 7.3) the critical distance lag is some ten percent less than the critical distance gap while the standard deviations for both the gaps and lags are on average between 12% to 15% of the mean distance gap or lag. These values were therefore used as a guide for determining critical distance lags and standard deviations for the gaps and the lags, based on the "measured" critical distance gap. No special consideration was given to the first gap and every vehicle was assigned only a critical distance lag and a critical distance gap.

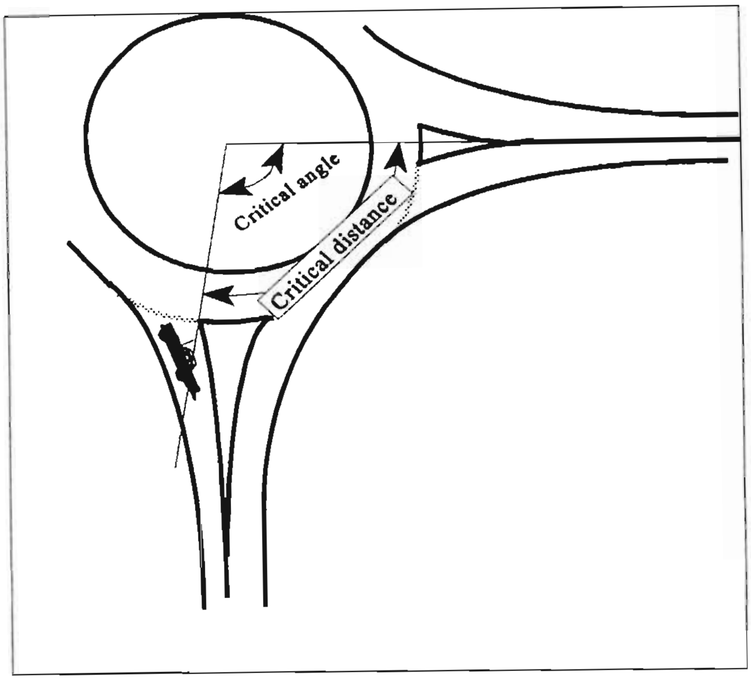


Figure 7.12: Measurement of critical distance

7.2.3 Simulation results

The simulation model was employed to estimate delays at the Chatsworth, Pinetown and Queen Mary circles. The input data and the model were the same as before (see Section 6.1) except for the critical distances which were based on the angle and distance as described in Section 7.2.2. The gap acceptance model was based on the measured critical distances. As before, the model was run at least ten times for each set of approach flows at the three circles. Each of the ten simulation runs was started with a different seed number for the random number generators and was run for two minutes to "warm up" and then for 15 minutes to simulate. The mean value of the ten simulation runs was calculated and used as the mean estimated delay through which a straight line was fitted. The ten different runs were analysed to find the minimum and maximum estimates and a maximum and minimum regression line was fitted through these estimates. These minimum and maximum lines constituted the boundaries of the envelope in which the estimated delays lay. These mean, minimum and maximum regression lines were then compared with the 45° observed delay line. Also shown in these figures are the data points for the mean delay estimates, through which the best fit mean line was regressed, and the corrected observed delay line (see Section 6.2). The results of the simulation runs using the measured critical distances are shown in Figures 7.13 to 7.19.

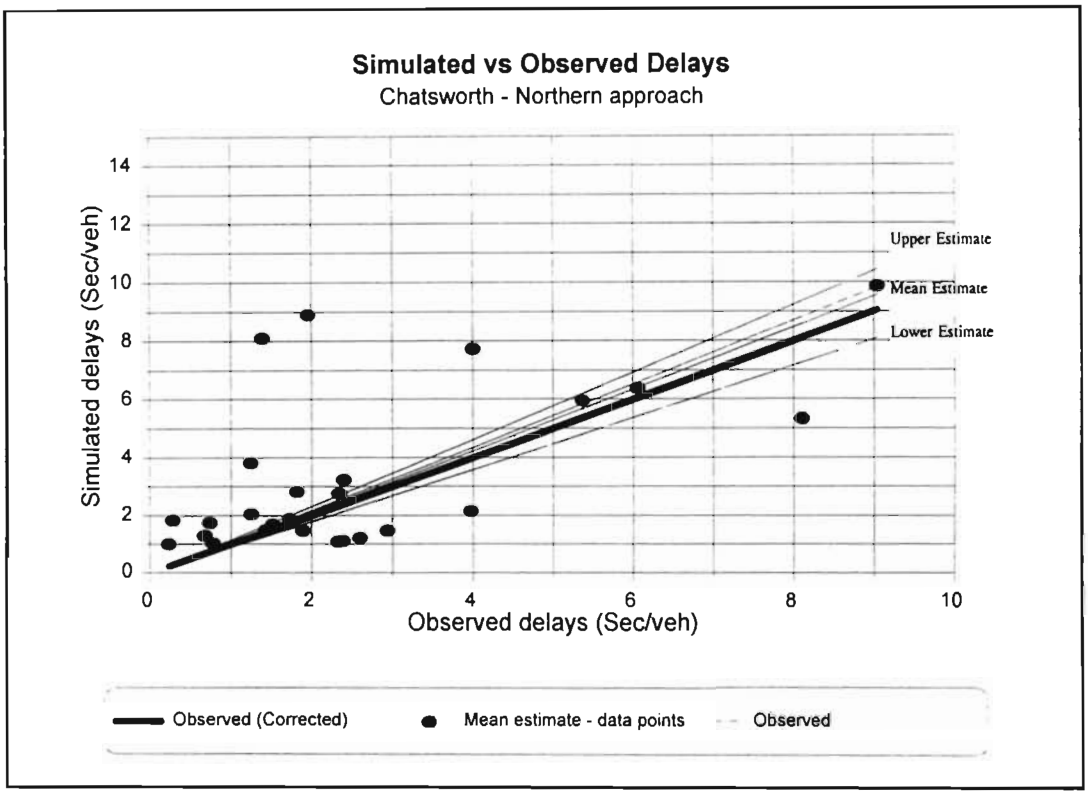


Figure 7.13: Delay comparison at Chatsworth circle based on measured critical distances Northern approach

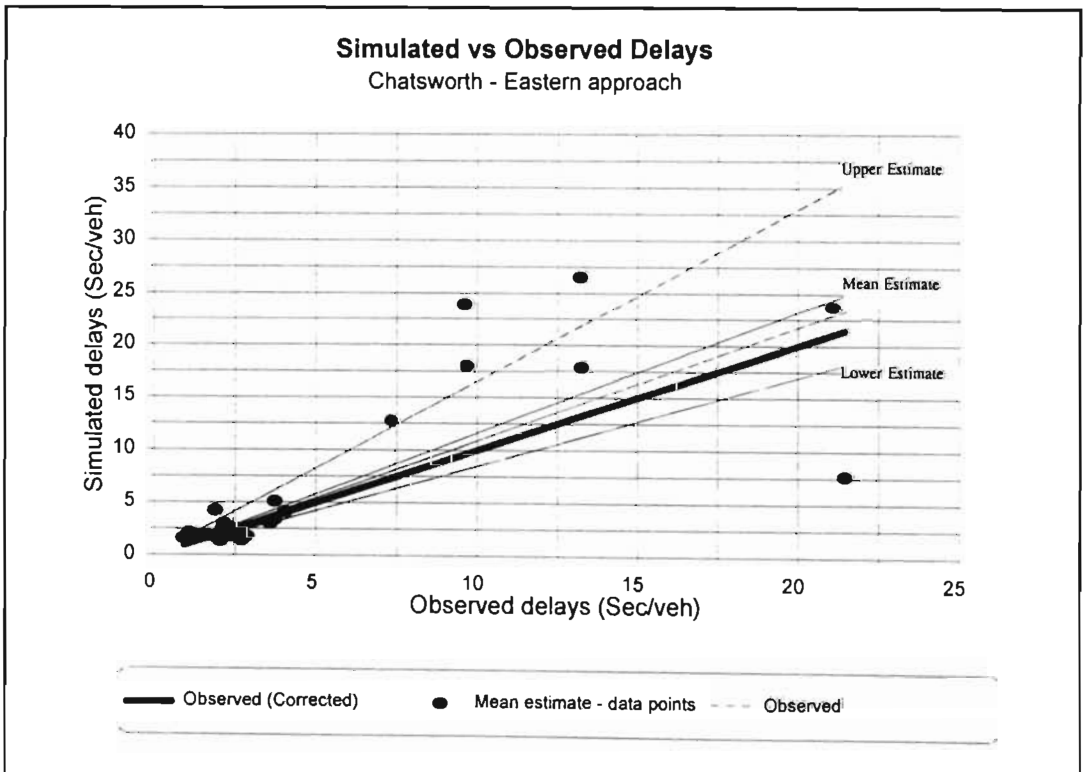


Figure 7.14: Delay comparison at Chatsworth circle based on measured critical distances Eastern approach

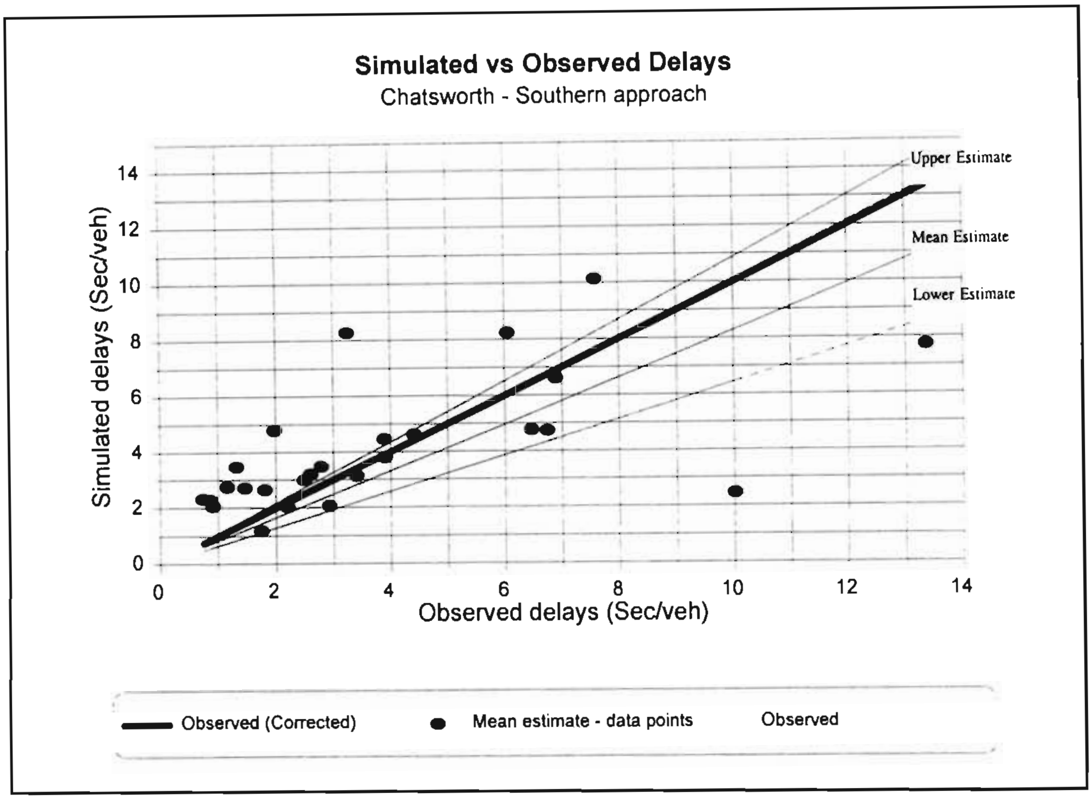


Figure 7.15: Delay comparison at Chatsworth circle based on measured critical distances
Southern approach

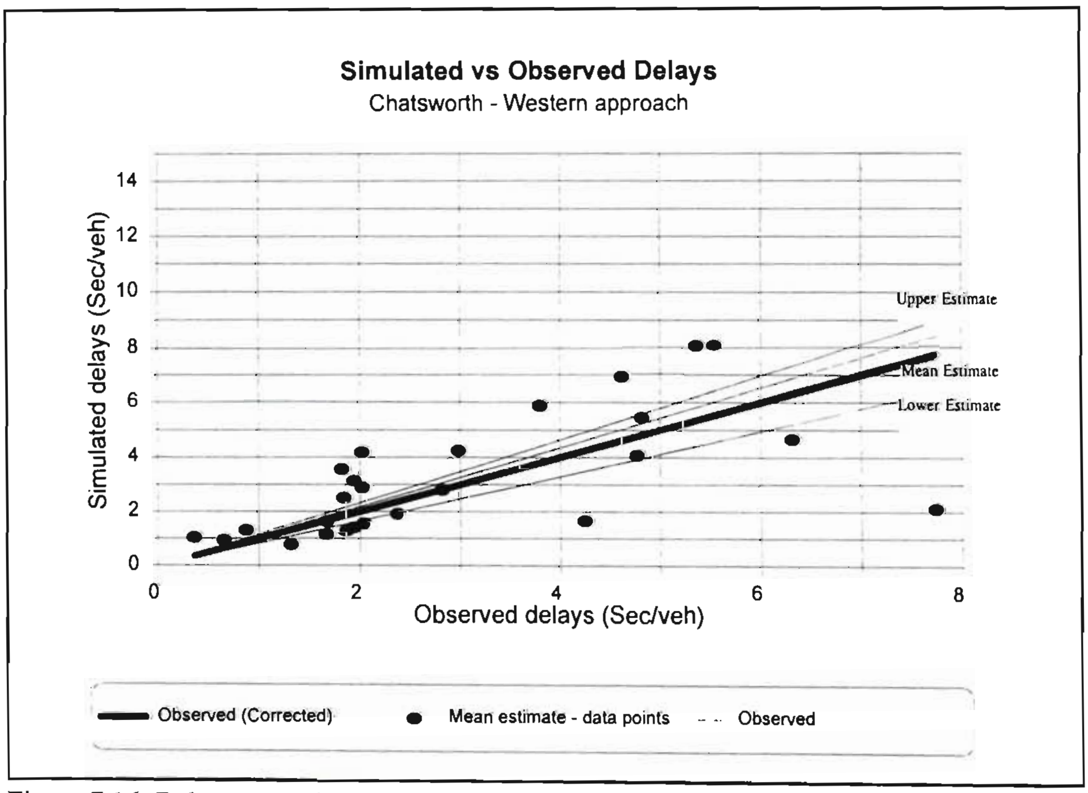


Figure 7.16: Delay comparison at Chatsworth circle based on measured critical distances
Western approach

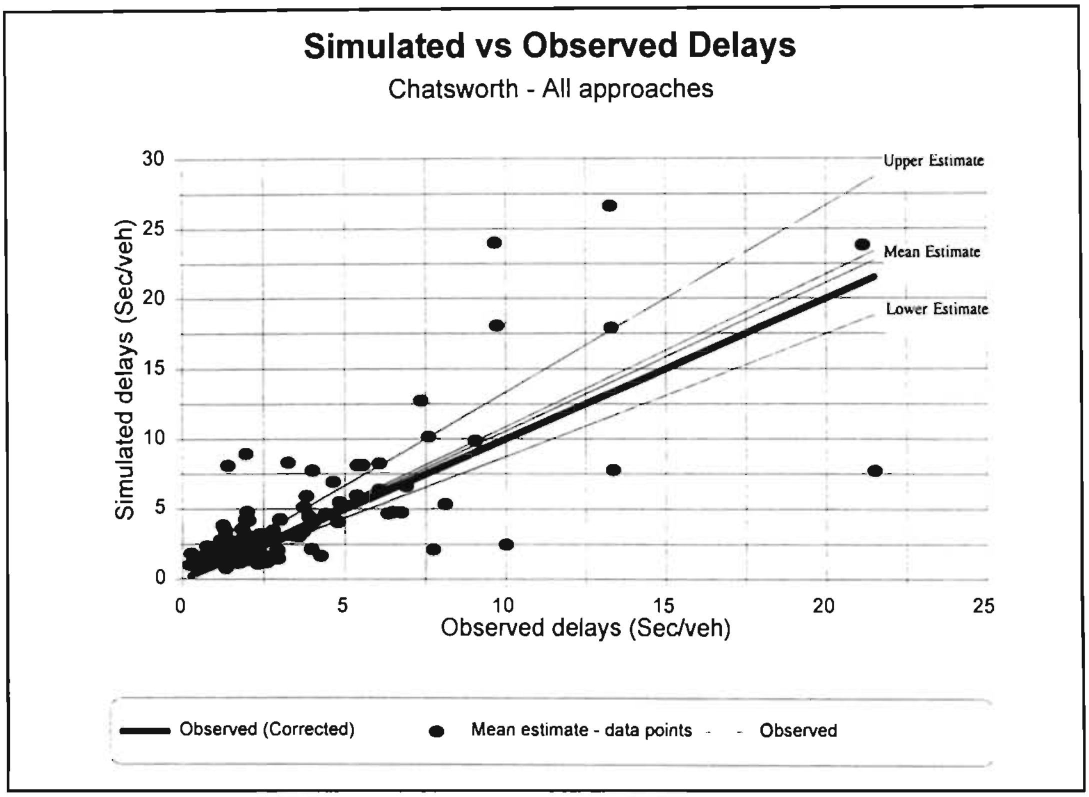


Figure 7.17: Delay comparison at Chatsworth circle based on measured critical distances
All approaches

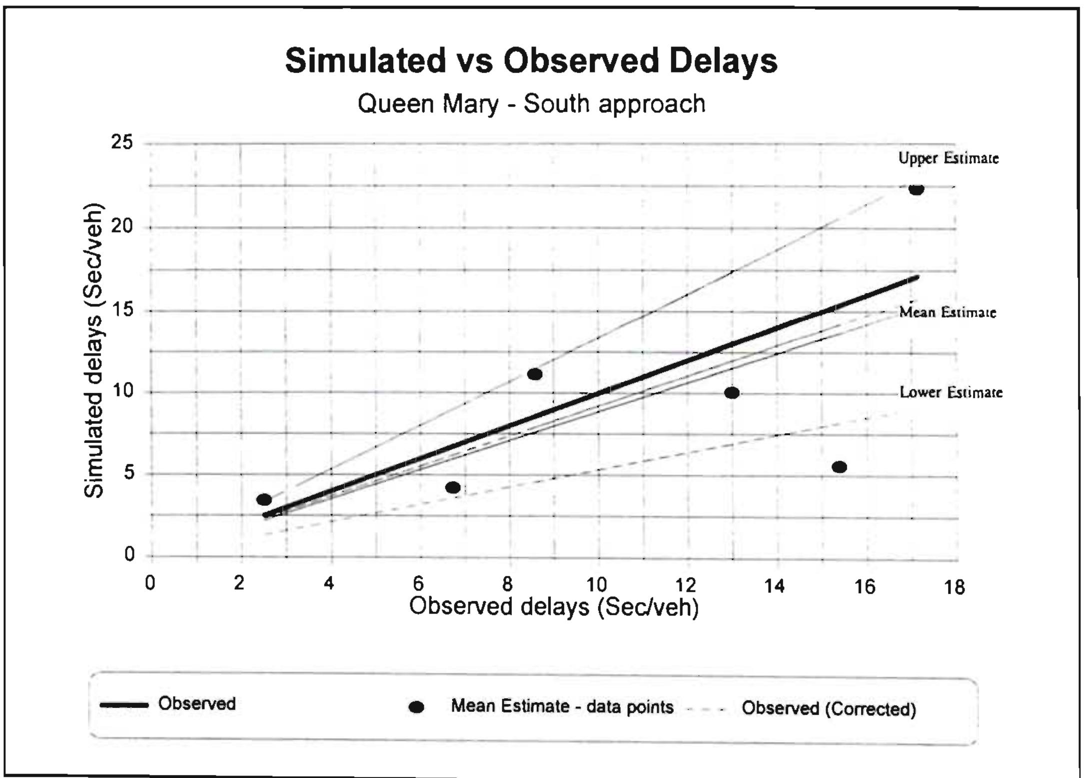


Figure 7.18: Delay comparison at Queen Mary circle based on measured critical distances
Southern approach

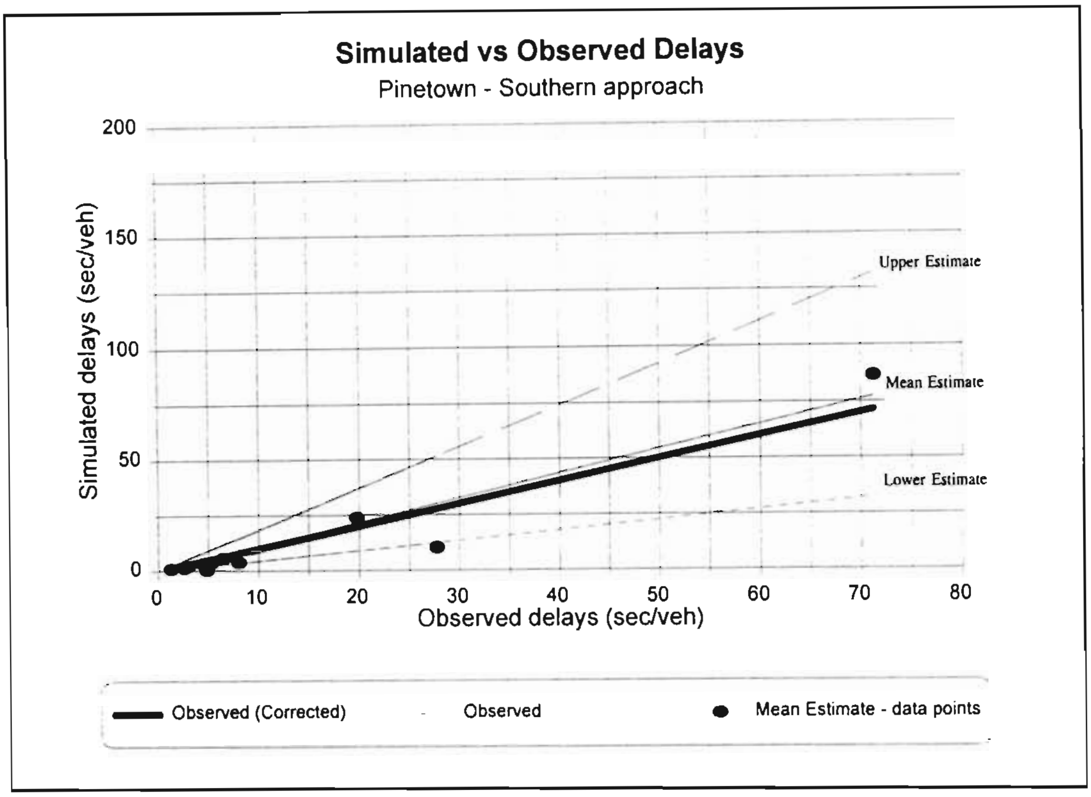


Figure 7.19: Delay comparison at Pinetown circle based on measured critical distances Southern approach

Similar to Table 7.4, Table 7.6 summarizes the slopes of the regression lines for the average estimates of the model for the above three circles, using gap acceptance based on critical times, gap acceptance model based on the observed critical distances and gap acceptance model based on the critical distance measured as described in Section 7.2.2. A slope of one (45° line) represents a perfect fit to the observed delays.

Table 7.6: Slopes of regression lines through mean delay estimates

Circle	Approach	Gap Acceptance Model		
		Time	Distance	
			Observed	Measured
Chatsworth	North	0.92	1.04	1.06
Chatsworth	East	1.23	1.18	1.16
Chatsworth	South	0.78	0.74	0.83
Chatsworth	West	0.92	0.88	0.98
Chatsworth	All	1.06	1.03	1.06
Queen Mary	South	0.91	0.97	0.89
Pinetown	South	0.60	0.92	1.09
	AVG	0.92	0.97	1.01

From Figures 7.13 to 7.19 it is clear that the delay estimates from the simulation model, based on distance gaps where the critical gaps were determined from a critical angle as described in Section 7.2.2 ("measured" critical distances), are reasonably close to the observed delays. From the results there is obviously no apparent reason for rejecting the gap acceptance model as wrong and hence it has to be accepted.

From comparisons of Figures 7.4 to 7.10 and Figures 7.13 to 7.19 and from Table 7.6 it is clear that the "measured" critical distance model provides delay estimates close to the "observed" (critical distances obtained from field observations) distance model. From the comparison of the slopes in Table 7.6 the average values of most slopes are evidently close to unity. Moreover, it is closer to unity than both gap acceptance models based on critical times and "observed" distances.

From this it is concluded that the gap acceptance model based on"measured" (from the geometric layout) critical distances is just as accurate, if not more so, than either of the models based on critical times or "observed" (obtained from actual field observations of gap acceptance) critical distances.

7.2.4 Combining all data

Similar to Section 6.2.4 where all data were combined to verify the simulation model based on a critical time gap acceptance model, the delay estimates from the model based on measured critical distances were also combined for comparison. The combined data were classified and the classification was according to the input and circulating traffic volumes. Initially the data were grouped according to similar entry volumes and then the data in these groups were in turn classified according to equivalent circulating volumes. The classification for both the entry and circulating volumes was again based on 100 vehicles per hour. As before, a straight line was fitted through the estimated data points to compare with the observed data points. Since it may be expected that the regression lines should intersect the y-axis at zero all the lines were fitted, forcing the y-intercept to zero. A statistical analysis was conducted on the x-coefficient or slope of the best fit regression line for the estimated delay values, to determine the 95% confidence interval for this parameter. The results of the regression analysis, excluding two outliers, are given in Table 7.7.

Table 7.7: Regression results on grouped volume data for delay observations

	Value	t-Statistic	95% Confidence Interval Lower	Upper
Correlation coefficient	0.77			
Standard error	2.89			
Observations	30			
Intercept	0.00		0.00	0.00
X-coefficient	0.95	11.1	0.76	1.13

The results of the regression analysis given in Table 7.7 are shown graphically in Figure 7.20. In this figure the mean regression line is shown, together with the envelope of variation of this line given a 95% confidence interval. Also shown are the corrected observed delays.

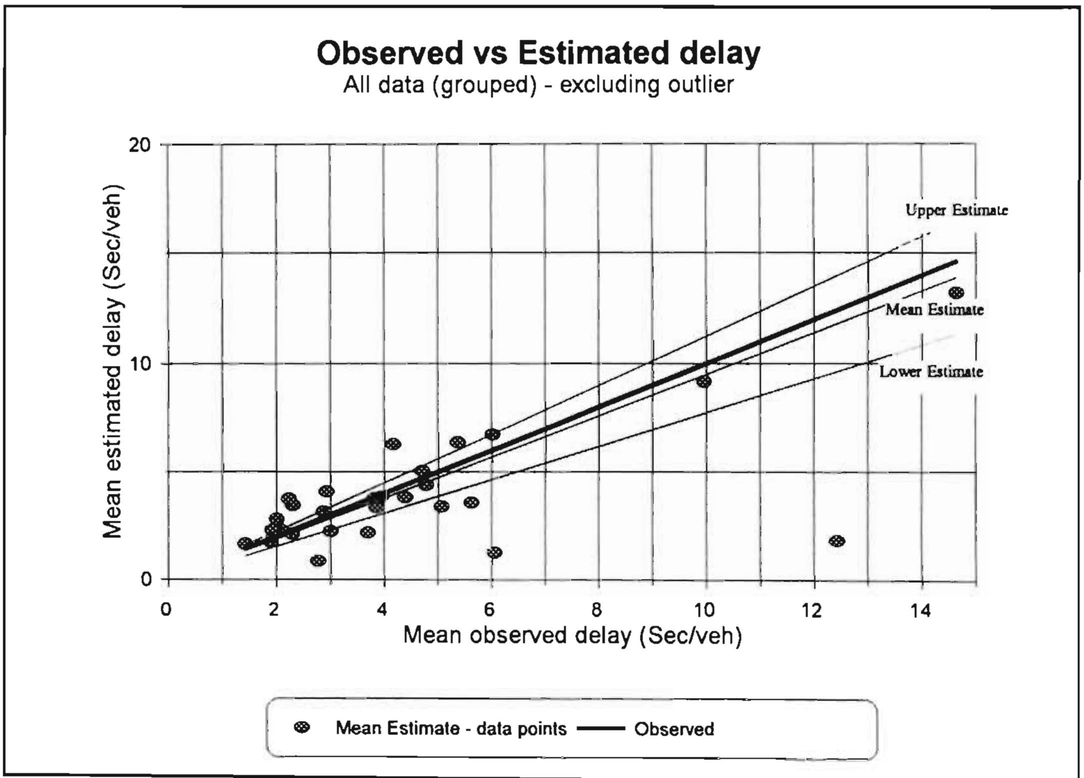


Figure 7.20: Observed vs estimated delays at all circles - grouped data

It is clear that the observed data falls within the confidence interval of the estimated regression lines, and in fact the observed line is reasonably close to the mean estimated line. Hence, the rejection of the validity of the simulation model is not possible.

A final comparison, to check the validity of the simulation model based on "measured critical distances" was to compare the observed and simulated data, not against the observed delay data points, but to compare them using the degree of saturation on each approach as the independent variable. This is similar to what was done in Section 6.2.4. Again, to establish a degree of saturation (approach flow divided by capacity: X = Q/C) an estimate of the capacity of the approach was required and equation (6-1) was used to give an estimate of capacity. The analysis was conducted on the grouped data, where the classification was according to the entering and circulating volumes. Since it was expected that the delays would increase exponentially with degree of saturation, an exponential curve was fitted through the observed and estimated data points. The results of this comparison are shown in Figure 7.21.

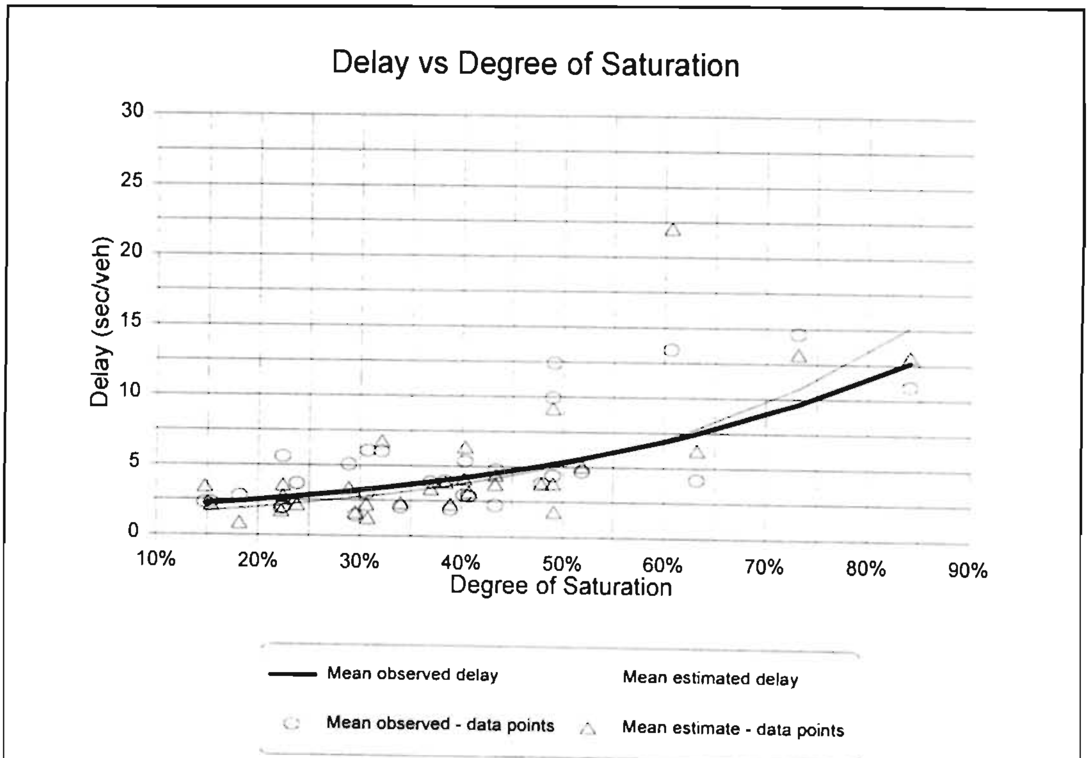


Figure 7.21: Delay versus Degree of Saturation (Grouped data, excluding outliers)

Although the degree of saturation is not calibrated against any actual data, the values for calculating capacities were selected to give realistic results. The results as displayed in Figure 7.21 show that for low degrees of saturation (less than 50%) the estimated values are lower than the observed values and for higher degrees of saturation (greater than 50%) the estimated delays are greater than the observed delays. This is an interesting result, once again supporting the variable critical gap theory on which the Australian method (Troutbeck, 1989) is based. The critical gap observations were made for degrees of saturation in the range 30% to 60% and hence the relatively good comparison of observed and estimated delays over that range. For lower degrees of saturation it is possible that drivers have larger critical gaps and hence a larger mean delay per vehicle, while for higher degrees of saturation the critical gap reduces and hence reduces the mean delay per vehicle.

This last comparison again supports the acceptance of the simulation model based on measured critical distances, but underlines the fact that it is only applicable over the range of geometric features and traffic flows for which it was calibrated.

7.2.5 Discussion

In this section the application of the simulation model, based on a critical distance obtained from a angle measured to the entry angle of the previous approach, has been shown to give favourable estimates of delay. Therefore, it is concluded that the delays on the approaches to a traffic circle can be estimated using a simulation program with a critical distance based on the method as described in Section 7.2.2. This provides the opportunity of studying traffic circles without having to carry out extensive and expensive traffic surveys to estimate critical time or distance gaps. However, from the simulation results it was clear that this method only works for circles where the approaches are spaced at approximately 90 degrees. The same method of determining the critical distances was followed for the other approaches at the Pinetown circle where the approaches are spaced at angles considerably less than 90 degrees. For these approaches the estimated delays were much lower than the observed delays. It must also be stressed that a contributing factor to the inaccurate estimates on the other approaches was either low volumes, or traffic flows and entry patterns atypical of a traffic circle. Although the angle between the southern approach at the Pinetown circle and the previous approach (112 degrees) was larger than 90 degrees, the method still provided accurate results.

It would seem that this method should only be used where the approaches to the traffic circle are spaced at more or less 90 degrees and that it would be more conservative to use a larger critical angle when the previous approach is spaced at an angle of less than 90 degrees. From the southern approach to the Pinetown circle it is clear that the opposite is not necessarily true and if the previous approach is spaced at an angle greater than 90 degrees, then the measured critical angle should be used. Obviously an upper limit to this greater angle exists, but from the circles and data analysed in this research, this could not be determined.

7.3 Summary

In this chapter gap acceptance based on a critical distance approach is investigated. Firstly, the observations which were made to investigate critical distances are discussed as well as the analysis to obtain the critical distance distributions ("observed" critical distances from actual field observations of gap acceptance). Secondly, a comparison is made between estimated delays, using the "observed"critical distance approach and the observed delays. Thirdly, this chapter reports on an approach to derive the critical distances from the geometric features of the circle ("measured" critical distances). This negates the use of expensive surveys to find these parameters. To justify the use of the proposed method, the observed delay values are compared with the simulated delays using the critical distance gaps obtained from the geometric layout of the circles.

The mean "observed" critical distance lag varied from 20,6 to 39,3 metres while the mean critical distance gap varied between 24,5 and 39,3 metres. Using a gap acceptance model in TRACSIM based on critical distances, a satisfactory comparison was obtained with the observed delays at all the circles. In fact, a slightly better result was obtained than with the gap acceptance model based on critical times. From the positive results obtained from the critical distance approach for gap acceptance models, a method is proposed and tested on how to determine critical distances from the geometric characteristics of a circle ("measured" critical distances). This method proved to be successful when the simulated results using TRACSIM and the critical distance gaps based on the proposed method were compared with the observed delays. From Table 7.6 it can be seen that this proposed method based on "measured" critical distances produces at least the same if not slightly better estimates than any of the two methods based on observed time or observed distance gaps.

CHAPTER 8: SIMULATION MODEL VERSUS SIDRA

Having developed a simulation model validated for South African conditions, the work completed during this research is put in context of other research by comparing the delay estimates from the simulation model with delay estimates from another analytical model, SIDRA. SIDRA is an established and widely used analysis package which by June 1995 was being used by more than 500 organisations in more than 40 countries (Akçelik, 1995) and is increasingly used in South Africa (Green, 1996). Firstly the basis for this comparison is discussed after which two sample circles with traffic flows are defined for the comparison. The last section of the chapter compares the estimates from the simulation program with the SIDRA estimates.

8.1 Basis of comparison

After the development of the simulation program to estimate traffic performance at traffic circles and having validated it for local conditions, the next step was to compare the simulation estimates with other available models. Not only is the simulation model based on event scanning, but it also uses a gap acceptance model based on critical distances, which is different from the more general use of critical times for gap acceptance. Although it was shown in Chapter 7 that the proposed methods for estimating critical distances seem to work for the delays observed at three local circles, the model should be compared with other established methods.

Of the available analysis models, SIDRA is the only analytical model which is based on gap acceptance. Moreover, the SIDRA model allows changes to the input variables, such as the critical gap and move-up times. This feature was considered important in the selection of appropriate models for comparison with TRACSIM so that the same input variables are available for both TRACSIM and the model to which it is compared. ARCADY, which was evaluated in Chapter 3, is an empirical model with few of the input parameters comparable to those used in TRACSIM as input for the gap acceptance process. Therefore, it was decided to use only SIDRA for the comparison.

An important factor in the comparison between observed and estimated delays is that a similar definition is used for delay. SIDRA provides four different types of delay estimates (see Section 2.3.3), while TRACSIM was developed to estimate stopped delay only. The TRACSIM stopped delay estimates were also calibrated from observed stopped delays using the queue sampling technique. The option in SIDRA which estimates stopped delays using the HCM method was used.

This method takes the total traffic delay and reduces it by some 23% to allow for the accelerations and decelerations.

Being an analytical model, SIDRA's estimates are average values representing a range of possible delays. To obtain similar average delay estimates from TRACSIM's simulations, the program was used to simulate at least two hours for each set of input flows. This allowed for random variations to be averaged out.

The TRACSIM estimates used in this comparison were based on the critical distances obtained through the method proposed in Chapter 7.

8.2 Sample problems for comparison

For the comparison it was decided to use two standard circles with four approach roads with single entering, circulating and exiting lanes; one with a large diameter and one with a smaller diameter. The first circle (Circle One) is the large circle with a central island diameter of 36 metres, and a layout similar to the Chatsworth circle. The second circle (Circle Two) is the smaller circle with a central island diameter of 23,0 metres, similar to the Queen Mary circle.

A summary of the most important geometric parameters is given in Table 8.1 with a diagrammatic layout (not to scale) of Circles One and Two in Figures 8.1 and 8.2 respectively.

Table 8.1: Geometric characteristics of Circles One and Two.

Circle	Central Island Diameter (m)	Circulating lane width (m)	Inscribed diameter (m)	No of circulating lanes	No of entering lanes	Avg entry lane width (m)
Circle One	36,0	7,0	50,0	1	1	4,0
Circle Two	23,0	9,0	41,0	1	1	5,0

The average entry lane width was determined as the entry width measured along the normal to the outer kerb line of the circle as is shown in Figure 2.6. These two circles are representative of a large number of circles, excluding mini-circles.

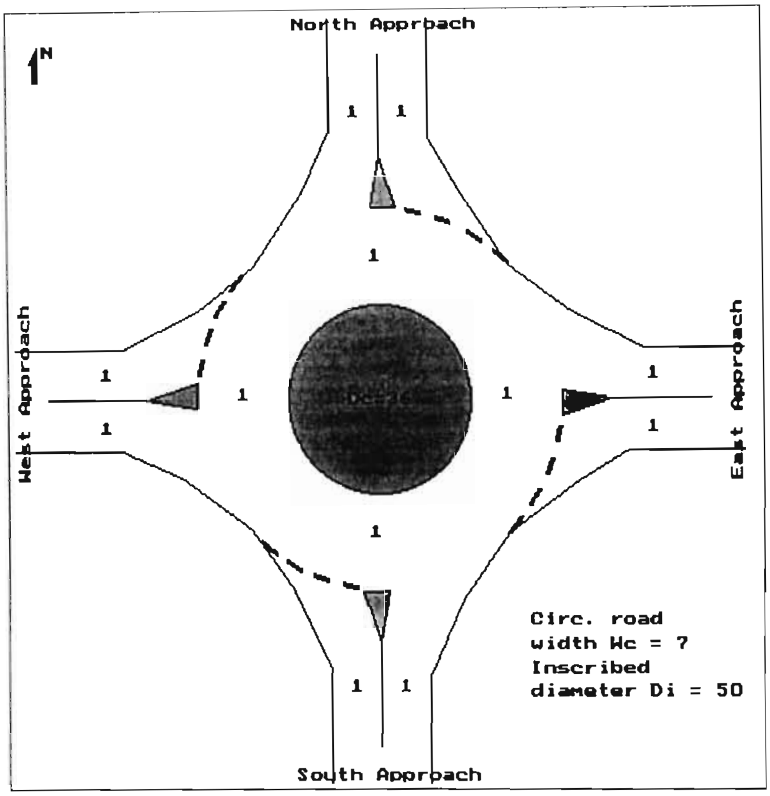


Figure 8.1: Diagrammatic layout of Circle One

Source: Sidra 4.1

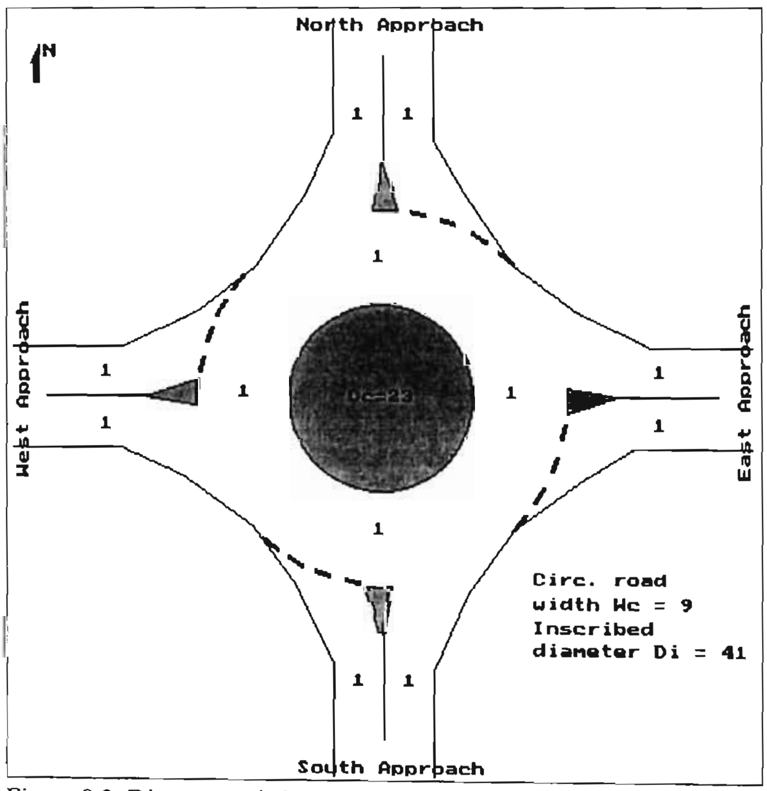


Figure 8.2: Diagrammatic layout of Circle Two

Source: Sidra 4.1

For the comparison between SIDRA and TRACSIM a range of eight sets of traffic flows was used. The range of flows was similar for all approaches to the circle and was selected so that it would cover a wide range of conditions, from low degrees of saturation to flows close to capacity. Table 8.2 summarizes these flows for one approach and Figure 8.3 shows the lowest of these flows, ie. flow set no. 1 with 150 pcuph per approach.

Table 8.2: Traffic Flows (pcuph) for the analysis

No	Left	Through	Right	Total per approach	Total for Intersection
1	50	50	50	150	600
2	100	100	100	300	1200
3	150	150	150	450	1800
4	200	200	200	600	2400
5	215	215	215	645	2580
6	230	230	230	690	2760
7	250	250	250	750	3000
8	260	260	260	780	3120

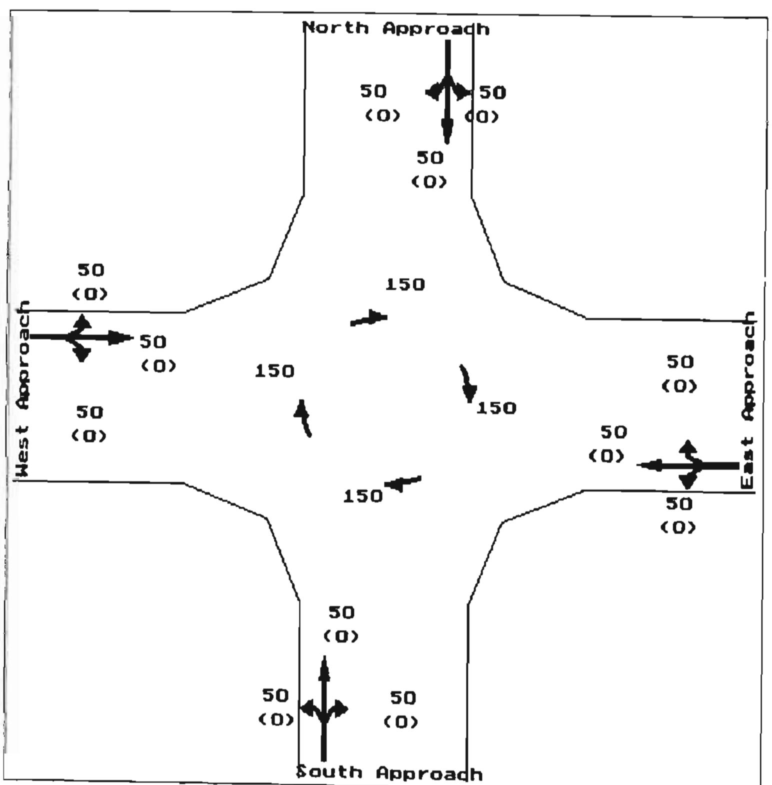


Figure 8.3: Typical traffic flows for comparison

Source: Sidra 4.1

8.3 Analysis and comparison of results

For the comparison, TRACSIM and SIDRA were each used to estimate the average delay per vehicle at each of the two circles for each of the traffic flows as given in Table 8.2. Because of the option of different sets of input variables for SIDRA and its sensitivity to some of these variables, the SIDRA estimates were made for more than one set of input variables to account for the variation due to different input variables. Sidra's analytical capacity and delay models are based on a variable critical gap (Akçelik, 1995) as given by (2-44) and repeated here as (8-1):

$$\tau = (3{,}6135 - 0{,}0003137Q_c - 0{,}339\bar{e} - 0{,}2775n_c)\,\beta_2 \quad \textit{for } Q_c \leq 1200$$
$$= (3{,}2371 - 0{,}339\bar{e} - 0{,}2775n_c) \quad \textit{for } Q_c \geq 1200 \qquad (8\text{-}1)$$
$$\textit{subject to } 3{,}0 \geq \frac{\tau}{\beta_2} \geq 1{,}1 \quad \textit{and} \quad \tau \geq \tau_{min}$$

where τ - Critical gap (sec)

β_2 - Move-up time (sec)

$\bar{e}$ - Avg entry lane width (m)

Q_c - Circulating flow (pcuph)

From (8-1) it can be seen that the critical gap is relatively sensitive (see also the discussion in Section 5.5.4) to the average entry lane width, and therefore the comparisons were made using at least two entry lane widths. Furthermore, SIDRA allows for either a variable critical gap and move-up time (calculated using the above (8-1)), or a fixed user-supplied critical gap and move-up time. Therefore, the comparisons were carried out with variable critical gaps and move-up times as calculated by SIDRA and also with fixed critical gaps and move-up times. These fixed gaps and move-up times were set cqual to the observed gaps at the Chatsworth circle for Circle One (see Section 5.5) and equal to the observed gaps at the Queen Mary Circle for Circle Two.

Table 8.3 shows the results of the comparison for Circle One with a variable gap using an entry width of 3,6 and 4,0 metres and also with a fixed critical gap of 4,3 and 4,5 seconds. The Move-up times were fixed equal to the observed move-up times (see Section 5.4.2) whenever fixed critical gaps were used. Table 8.3 also summarizes the critical gaps and move-up times for the variable critical gap analysis as they are calculated by the SIDRA model.

Table 8.3: SIDRA vs TRACSIM: Comparison of delay estimates at Circle One

Total intersection flow (pcu/h)	Estimated delays								
	Tracsim Estimates	Sidra Estimates							
		Variable Cgap and MuT						Fixed Cgap & MuT	
		Avg entry width = 3.6 m			Avg entry width = 4.0 m				
		Delay (sec)	Cgap (sec)	MuT (sec)	Delay (sec)	Cgap (sec)	MuT (sec)	Cgap = 4.5 MuT = 2.7	Cgap = 4.3 MuT = 2.7
600.00	0.72	0.5	4.71	2.28	0.5	4.41	2.28	0.5	0.5
1200.00	1.75	1.3	4.57	2.26	1.2	4.26	2.26	1.3	1.2
1800.00	4.24	2.5	4.43	2.23	2.2	4.13	2.24	3.20	2.9
2400.00	13.44	7.30	4.29	2.23	6.10	3.99	2.23	13.40	11.5
2580.00	24.42	11.60	4.25	2.22	9.30	3.95	2.22	40.80	29.6
2760.00	70.43	23.70	4.21	2.22	16.9	3.91	2.22	90	80

MuT - Move-up time, Cgap - Critical gaps.

The above estimated delays by TRACSIM and SIDRA for four different input scenarios are also summarized graphically in Figure 8.4.

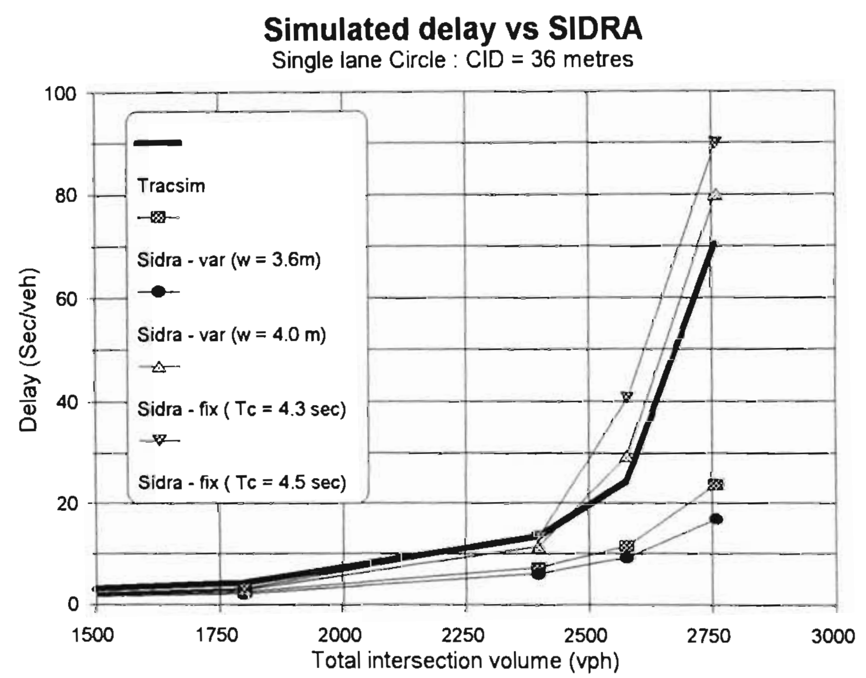


Figure 8.4: Comparison between TRACSIM and SIDRA for Circle One.

It is obvious from the comparison that SIDRA's delay estimates using the default variable critical gap model with an entry lane width of 4,0 metres are much lower than the TRACSIM delay estimates. If it is assumed that the TRACSIM estimates are the more correct ones (see Chapters 3, 6 and 7) then this comparison agrees with the conclusions of Chapter 3, that the SIDRA delay estimates are lower than the delays observed at local traffic circles. Even a reduction of the entry lane width to 3,6 metres does not result in a significant improvement in the SIDRA delay estimates when compared with the TRACSIM estimates..

Using a fixed critical gap of 4,5 seconds (approximately equal to the observed critical gap), the SIDRA delay estimates are reasonably close to the TRACSIM estimates for total intersection traffic volumes up to 2400 vehicles per hour. For traffic flows of above 2400 vehicles per hour the SIDRA estimates are significantly higher than the TRACSIM estimates. However, a slightly lower fixed critical gap of 4,3 seconds results in a better comparison. Thus it can be concluded from these results that the TRACSIM delay estimates are similar to SIDRA's estimates when using a fixed critical gap for the SIDRA analysis. However, when using a variable critical gap, based on (8-1), the comparison is not favourable as the SIDRA estimates are significantly lower than the delays estimated by TRACSIM.

The results of the comparison for Circle Two are summarized in Table 8.4 and in Figure 8.5. Similar to Circle One, the SIDRA analysis was first conducted for a variable critical gap using two different entry lane widths. For the second circle, the two entry widths (3,6 m and 5,0 m) used in the analysis were quite different and not as similar as for Circle One (3,6 m and 4,0 m). This was followed by a fixed critical gap analysis using critical gaps and move-up times equal to the observed times and gaps at the Queen Mary circle (see Section 5.4.2 and 5.5).

Table 8.4: SIDRA vs TRACSIM: Comparison of delay estimates at Circle Two

Total intersection flow	Estimated delays									
	Tracsim Estimates	Sidra Estimates								
		Variable Cgap and MuT						Fixed Cgap & MuT	
		Avg entry width = 3.6 m			Avg entry width = 5.0 m				
(pcu/h)		Delay (sec)	Cgap (sec)	MuT (sec)	Delay (sec)	Cgap (sec)	MuT (sec)	Cgap =3.9 MuT = 2.3	Cgap =3.6 MuT = 2.3
600.00	0.52	0.5	4.77	2.31	0.4	3.76	2.36	0.4	0.3
1200.00	1.65	1.3	4.63	2.29	0.9	3.64	2.35	1	0.9
1800.00	2.84	2.5	4.49	2.27	1.8	3.51	2.34	2.10	1.8
2400.00	7.62	7.80	4.35	2.26	4.70	3.38	2.33	6.20	5.2
2580.00	9.42	12.60	4.31	2.25	6.90	3.34	2.32	9.80	8
2760.00	14.70	28.20	4.27	2.25	11.3	3.31	2.32	20.1	14.5
3000.00	84.79	110.00	4.27	2.24	48.5	3.26	2.32	100	90

MuT - Move-up time, Cgap - Critical gaps.

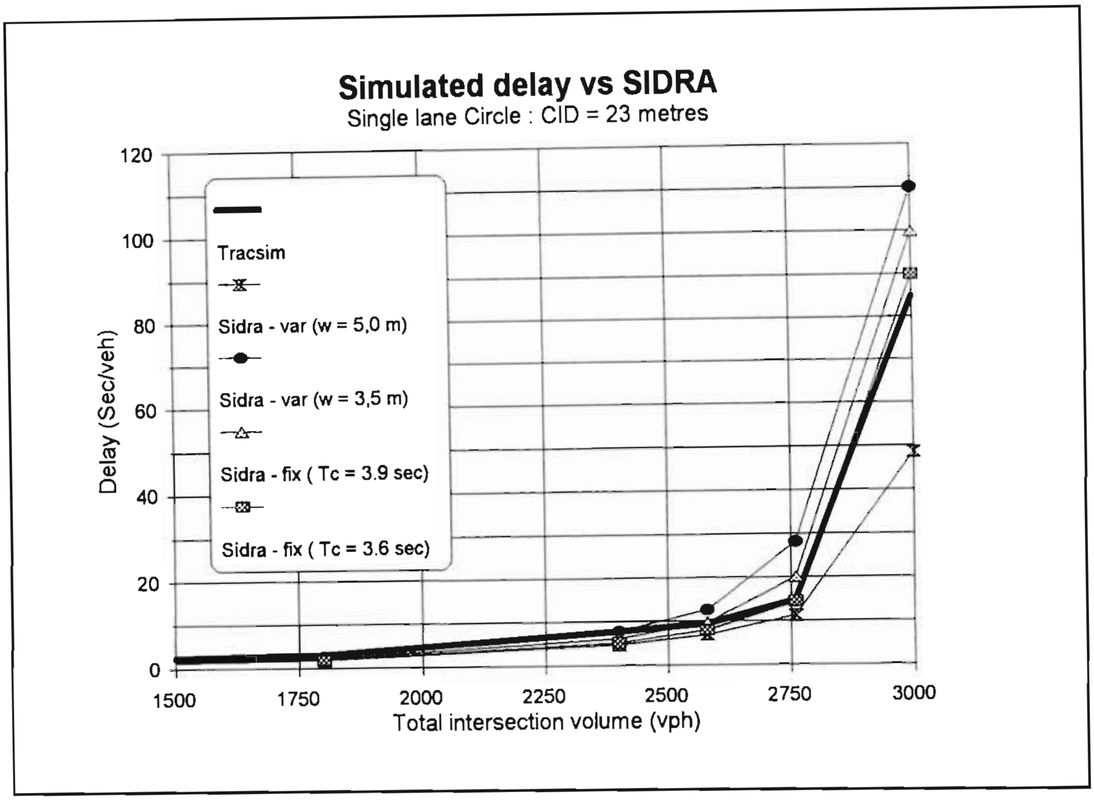


Figure 8.5: Comparison between TRACSIM and SIDRA for Circle Two

From the variable critical gap analysis it is clear that the SIDRA model for critical gaps is sensitive to the lane width. For a lane width of 3,6 metres the critical gap for a flow of 2400 vehicles per hour is equal to 4,35 seconds, compared with a gap of 3,38 seconds for an entry width of 5,0 metres. With the observed critical gap equal to approximately 3,9 seconds it may be expected that the SIDRA model will overestimate the delay for a lane width of 3,6 metres and underestimate the delays for a lane width of 5,0 metres. This is also clear from the analysis and it can be seen in Figure 8.5 that for a lane width of 5,0 metres the SIDRA delay estimates are lower than the TRACSIM estimates, while for a lane width of 3,6 metres the opposite is true.

When using a fixed critical gap of 3,9 seconds and a move-up time of 2,3 seconds the SIDRA and TRACSIM estimates are close together except for the higher intersection volumes. If the critical gap is reduced to 3,6 seconds then there is closer agreement at the higher flows, but the SIDRA estimates are lower than the TRACSIM estimates for the lower intersection flows.

However, as for Circle One, it is clear that the delay estimates for TRACSIM and SIDRA are similar when a fixed critical gap is used for the SIDRA estimates. It can thus be concluded that the TRACSIM delay estimates are similar to the SIDRA estimates if appropriate fixed critical gap and move-up times are used in the SIDRA analysis. Appropriate gaps and move-up times refer to the gaps and move-up times observed at the circle under consideration or which reflects the local conditions accurately.

8.4 Summary

In this chapter the research and the work conducted on TRACSIM are put into context by comparing them with another established analytical model. Firstly, the basis for this comparison is investigated and then secondly two sample circles are defined with the associated traffic flows which were used for the comparisons. Lastly, the delay estimates from TRACSIM are compared with the SIDRA estimates for the traffic operations at the two sample circles. The TRACSIM estimates were based on the critical distance gap acceptance model as proposed in Chapter 7.

From the comparison between TRACSIM and SIDRA it is obvious that the delay estimates follow at least the same trend. However, employing SIDRA's default variable critical gap method leads to low delay estimates when compared with that of TRACSIM. Reducing the entry lane width significantly can improve the SIDRA delay estimates. However, even better results (improved correlation between Sidra and Tracsim delay estimates) were obtained while using fixed critical gaps and fixed move-up times for the SIDRA analysis. The critical gaps and times used were similar to the observed values, which were also used in TRACSIM analysis. For the higher traffic flows a better comparison was possible if the fixed critical gaps were reduced slightly.

It can be concluded that the TRACSIM delay estimates are similar to the SIDRA estimates if applicable (similar to what was used in TRACSIM) fixed critical gaps and move-up times are used for the SIDRA analysis.

CHAPTER 9: UNBALANCED FLOWS

9.1 Background

In this chapter the effects of the origin-destination patterns on approach delays at traffic circles are investigated. Because the simulation program TRACSIM was not calibrated for capacity conditions, the effect on approach capacity cannot be investigated. However, because approach delays are a function of entry capacities, the effect on capacity would be similar to the effects on delay.

Except for the latest version of SIDRA, most traffic circle analysis methods treat a traffic circle as a series of T-intersections with no interaction between the approaches (Akçelik et al, 1995). Therefore the origin of the circulating flows passing an approach does not affect the capacity and delay estimates for that approach. According to Akçelik et al (1995) this is not true in reality as many real life traffic circles indicated that capacities can be over-predicted with an imbalance of flows, especially at multi-lane circles. Chung et al (1992) and Chung (1993) showed with the aid of preliminary simulations that the arrival approach patterns affect entry capacities and that capacity decreased with an increase in unbalanced flows. The highest capacities were obtained with well-balanced flows. They also showed that the amount of queuing on the approaches to a traffic circle affected the capacity of the approach. The higher the proportion of queuing vehicles on an approach, the lower the capacity.

Akçelik et al (1995) expanded on the issue of unbalanced flows and subsequently developed a factor f_{od} which is used to reduce the capacity estimate obtained from gap acceptance techniques. This reduction then takes into account the origin-destination pattern and the approach queuing characteristics. The adjustment factor f_{od} is obtained as follows:

$$f_{od} = 1 - f_{qc}(p_{qd}p_{cd}) \qquad (9\text{-}1)$$

with f_{qc} a calibration parameter which is defined as follows for single lane circulating streams:

$$f_{qc} \begin{cases} = 0.04 + 0.00015q_c & for\ q_c < 600 \\ = 0.0007q_c + 0.29 & for\ 600 \leq q_c \leq 1200 \\ = 0.55 & for\ q_c > 1200 \end{cases} \qquad (9\text{-}2)$$

and with

p_{qd} the proportion of queued vehicles on the dominant approach

p_{cd} the proportion of total circulating flow which originated from the dominant approach equal to q_{cd}/q_c

q_c the total circulating flow rate

q_{cd} the part of the total circulating flow stream originating from the dominant approach.

Akçelik et al (1995) define the dominant approach as the approach which contributed the highest proportion of the queued traffic in the circulating flow. This refinement of capacity predictions has been included in the latest version of SIDRA.

The work as reported in this chapter takes another look at specifically the effect of origin-destination patterns. The work by Akçelik et al (1995) is based on simulations obtained from MODELC (Chung, 1993) which simulates traffic entering a traffic circle based on a variable critical gap. However, the gap acceptance behaviour is based on a single model and no distinction is made between gaps accepted in circulating or entering traffic streams. The work completed during the course of this research (see Section 5.5 and Chapter 7) indicated that there is evidence to suggest that gap acceptance behaviour depends on whether it is a gap or a lag and on whether the gap/lag exists in the circulating stream or in the entering stream from the previous approach.

9.2 Definition of unbalanced flows

To evaluate the effect of unbalanced flows ρ_i was defined as the proportion of circulating traffic passing an approach originating from the first upstream approach. For the northern approach the proportion of circulating traffic (ρ_n) is defined as follows (see Figure 9.1 for definitions of flows):

$$\rho_n = \frac{Q_{wn}}{Q_{nc}} = \frac{Q_{wn}}{Q_{wn} + Q_{sn}} \qquad (9\text{-}3)$$

This proportion of circulating flow (ρ_n) which relates to the circulating flow from the previous approach is different to the proportion p_{cd} used by Akçelik et al (1995) where the proportion relates to the circulating flow originating from the dominant approach. The reason for the difference in definition is related to the difference in gap acceptance for circulating and entering traffic streams. Because of this difference it is possible that a dominant flow from the first upstream entry might have a different effect on the approach delays to that of a dominant flow from the second upstream approach. The different effects are discussed in this chapter.

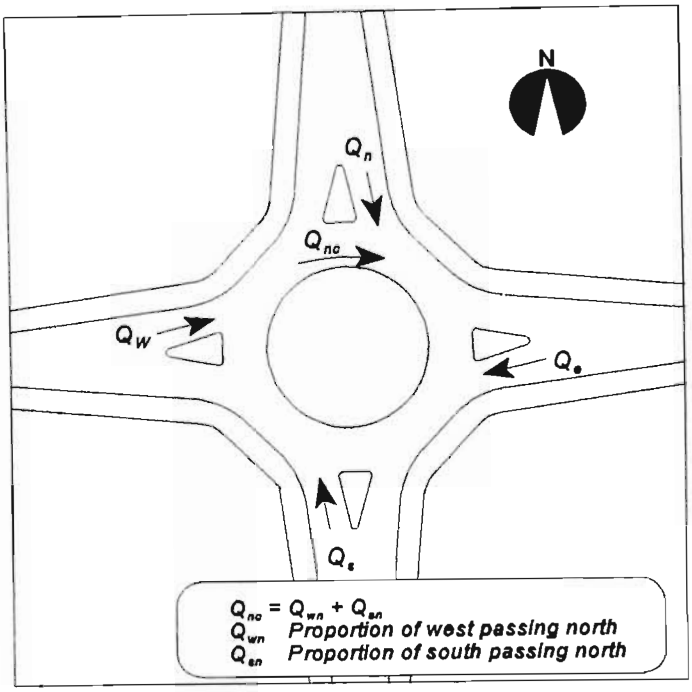


Figure 9.1: Flow definitions

A similar definition applies to all the other approaches. This obviously assumes that there is no circulating traffic past the northern approach, which originates from the eastern approach, i.e. no U-turns from each. Although U-turns do occur at traffic circles they occur seldom and are therefore not included in this definition. During the simulations no U-turns were simulated. If U-turns form most of the conflicting traffic past an approach, then they would necessarily have to be included in the definition.

An ideally balanced circle would have $\rho_i = 0,5$ for all the approaches (i = n,e,s,w), which means that the circulating traffic at each approach would consist of 50% from the previous entry and 50% from the second upstream entry. A totally unbalanced circle would have $\rho = 0$ or $\rho = 1$ for each approach. With $\rho = 0$ all the circulating traffic is from the second upstream approach and none from the first upstream approach while for $\rho = 1$, all the circulating traffic is from the first upstream approach.

Another factor which needs to be taken into account when defining imbalances, is the influence of imbalances at the previous upstream approach on traffic entries at the next approach, where the conflicting traffic might be balanced. For instance, if $\rho_n = 0,5$ (50% of circulating traffic comes from west and 50% from south), then the question is: Would an imbalance of conflicting flow passing west have an effect on the operation at north? With $\rho_n = 0,5$, would it make a difference to the entering capacity of north if the circulating flow past west is totally from south ($\rho_w = 1$) or totally from east

(ρ_w = 0)? If the answer to this is positive, then an approach cannot be analysed in isolation. Before any conclusion can therefore be reached on whether the conflicting flow past an approach is balanced or not, the balance of circulating flows at the previous approach should be investigated. However, if the balance of circulating flows at the previous approach is not significant, then the proportion (ρ_i) of circulating traffic past an approach can be considered in isolation to decide whether the circulating flows are balanced or not.

9.3 Research methodology

To evaluate the sensitivity of approach delays to the balance of circulating flows, two test circles were used. The sizes and geometric layouts of these circles were selected to correspond with the Chatsworth and Queen Mary circles (see Chapters 5 and 6). Similar to the analysis in Chapter 8, these circles are also labelled Circle One (36 metre CID) and Circle Two (23 metre CID). The geometric layouts of these circles are shown in Figures 3.1 and 5.3 for Chatsworth and Queen Mary respectively and are also summarized in Table 8.1.

The data sets defined for this analysis were designed according to the following criteria:

i) The entry flows and circulating flows must remain constant while the proportion of circulating traffic varies.

ii) A range of entry and circulating flows must be investigated to cover operating conditions from low degrees of saturation through to high degrees of saturation.

iii) While the entry volume, circulating volume and proportion of circulating traffic at one approach remain constant, the proportion of circulating traffic at the previous upstream approach must be varied.

It was decided to only evaluate the operating conditions at one approach and specifically the northern approach. The turning movements which were used to evaluate the effect of imbalances in the circulating traffic stream conflicting with the northern approach are shown in Figure 9.2.

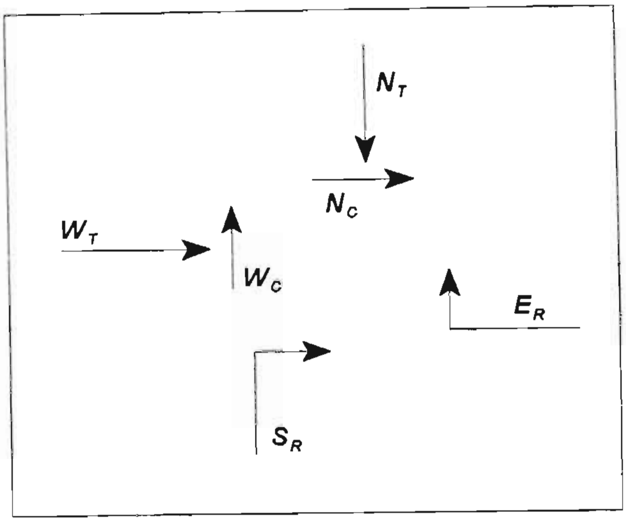


Figure 9.2: Turning movements

$$\rho_n = \frac{W_T}{N_c} = \frac{W_T}{W_T + S_R} \qquad (9\text{-}4)$$

For the above flows the proportions of circulating flows past north (ρ_n)and west (ρ_w) are defined as follows:

$$\rho_n = \frac{W_T}{N_c} = \frac{W_T}{W_T + S_R} \qquad (9\text{-}5)$$

$$\rho_w = \frac{S_R}{W_c} = \frac{S_R}{S_R + E_R} \qquad (9\text{-}6)$$

The range of entry and circulating flows for the northern approach which were used for the simulation runs are shown in Table 9.1. Initially a low (400 vph), medium (600 vph) and high (800 vph) scenario was investigated. However, the 800 vph often resulted in over-saturated conditions and therefore the 700 vph scenario was also included.

For each of these flow scenarios the proportions of circulating flows, ρ_n and ρ_w were varied. This was conducted in such a way that for each proportion of circulating flows past north (ρ_n) the proportion past west (ρ_w) was varied. This allowed an investigation into the effect of an imbalance in flows because of the second upstream entry. The actual entry flows used as input to the simulations for the medium (600 vph) and high (800 vph) flow scenarios are included in Tables 9.2 and 9.3. Refer to Figure 9.2 for definition of turning movements.

Table 9.1: Range of entry and circulating flows investigated

Entry Flow N_T (vph)	Circulating flow N_c (vph)	Proportion of circulating traffic at north (ρ_n)	Proportion of circulating traffic at west (ρ_w)
400	400	0 to 1	0,25 to 1
600	600	0 to 1	0,25 to 1
700	700	0 to 1	0,25 to 1
800	800	0 to 1	0,25 to 1

Table 9.2: Turning volumes and proportions of circulating flows for medium flow scenario

North Straight	West Straight	South Right	East Right	Q_{nc}	Q_{wc}	ρ_n	ρ_w
600	600	0	0	600	0	1	∞
600	450	150	1050	600	1200	0.75	0.13
600	450	150	450	600	600	0.75	0.25
600	450	150	150	600	300	0.75	0.50
600	450	150	49.5	600	199.5	0.75	0.75
600	450	150	0	600	150	0.75	1.00
600	300	300	900	600	1200	0.5	0.25
600	300	300	300	600	600	0.5	0.50
600	300	300	300	600	600	0.5	0.50
600	300	300	99	600	399	0.5	0.75
600	300	300	0	600	300	0.5	1.00
600	150	450	1350	600	1800	0.25	0.25
600	150	450	450	600	900	0.25	0.50
600	150	450	148.5	600	598.5	0.25	0.75
600	150	450	0	600	450	0.25	1.00
600	0	600	1800	600	2400	0	0.25
600	0	600	600	600	1200	0	0.50
600	0	600	198	600	798	0	0.75
600	0	600	0	600	600	0	1.00

Table 9.3: Turning volumes and proportions of circulating flows for high flow scenario

North Straight	West Straight	South Right	East Right	Q_{nc}	Q_{wc}	ρ_n	ρ_w
800	800	0	0	800	0	1	∞
800	600	200	1400	800	1600	0.75	0.13
800	600	200	600	800	800	0.75	0.25
800	600	200	200	800	400	0.75	0.50
800	600	200	66	800	266	0.75	0.75
800	600	200	0	800	200	0.75	1.00
800	400	400	1200	800	1600	0.5	0.25
800	400	400	400	800	800	0.5	0.50
800	400	400	300	800	700	0.5	0.57
800	400	400	132	800	532	0.5	0.75
800	400	400	0	800	400	0.5	1.00
800	200	600	1800	800	2400	0.25	0.25
800	200	600	600	800	1200	0.25	0.50
800	200	600	198	800	798	0.25	0.75
800	200	600	0	800	600	0.25	1.00
800	0	800	2400	800	3200	0	0.25
800	0	800	800	800	1600	0	0.50
800	0	800	264	800	1064	0	0.75
800	0	800	0	800	800	0	1.00

Similar flows were used for the 400 vph and 700 vph flow scenarios. Each of the flow scenarios consists of 19 different sets of flows and every set of flows was simulated for at least one and a half hours after an initial "warm-up" period of two minutes. This long simulation period was selected to even out random variations to find an average delay. Under over-saturated conditions this is not possible because the delay keeps increasing all the time without levelling out at an averaging delay. This trend can be observed either in real time or by checking the historical results. When the average delay increases continuously, then the approach is over-saturated.

The 76 (4x19) sets of flows were simulated for both circles while using the observed geometric layouts, gap acceptance behaviour, speeds, and move-up times at each circle. It must be stressed that these observations were conducted for a range of operating conditions and excluded for instance over-saturated conditions. Therefore, the simulation program was only calibrated for this range of operating conditions, and for any conditions outside the range for which it was calibrated, the estimates are not necessarily accurate. For the simulations required for this research; specifically for the high flows scenarios - over-saturation occurred often and therefore the associated estimated delays must be treated with the necessary caution.

9.4 Simulation results

The detailed simulation results of the runs for the two circles are included in Appendix D. In Figure 9.3 simulation results for Circle One are summarized by plotting the mean delay per vehicle for each of the simulation runs against the proportion of circulating traffic (ρ_n) from the previous entry - the first upstream entry. The 800 vph flow scenario resulted in totally over-saturated conditions and was therefore excluded from the results. Even the 700 vph flow scenario became over-saturated under unbalanced conditions.

For each proportion of circulating traffic (ρ_n) at north, four proportions of circulating traffic (ρ_w) at west were simulated, hence the range of delay values for each proportion ρ_n. The spread of the results for each proportion (ρ_n) represents the sensitivity of the entry delays at north to the imbalance of the flows passing west, because the proportions of circulating flows past north (ρ_n) were kept constant while the proportions past west (ρ_w) were varied.

Similarly, Figure 9.4 summarizes the simulation results for Circle Two, but here the high flow scenario of 800 vph is included. However, for the unfavourable unbalanced flow conditions the entry becomes completely over-saturated. A best fit exponential curve is also shown for each of the flow scenarios.

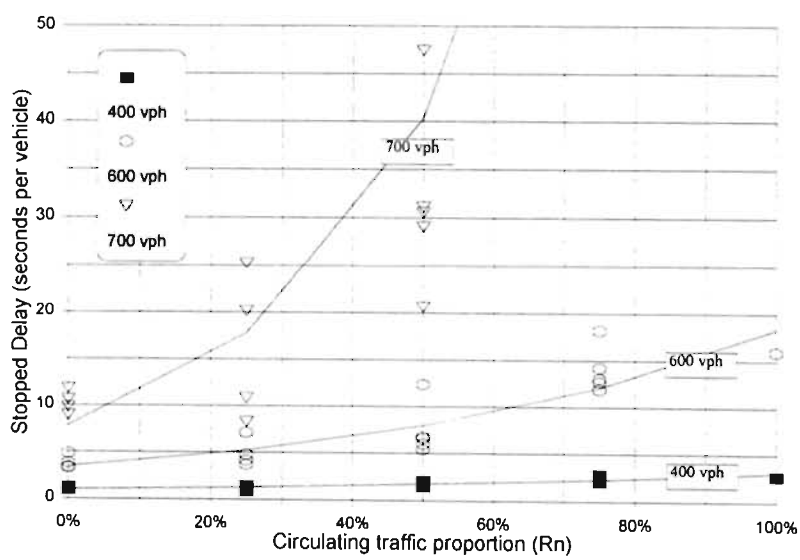


Figure 9.3: Vehicular delay versus proportion of circulating flows past north for Circle One

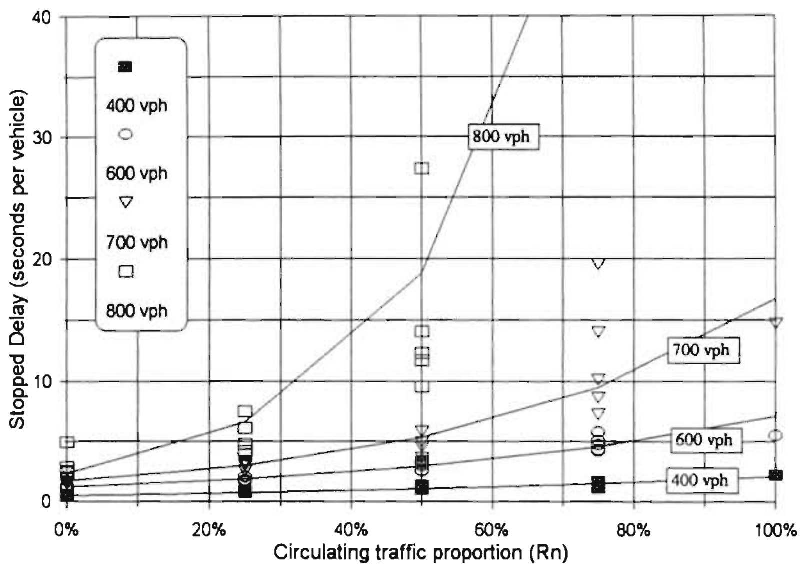


Figure 9.4: Vehicular delay versus proportion of circulating flows past north for Circle Two

The higher proportions of circulating traffic (ρ_n) mean that more vehicles are entering from the previous approach than are circulating from the second upstream approach (turning right from south) and vice versa. The lower the proportion of circulating traffic the fewer vehicles are entering from the previous approach. A 50% proportion of circulating flow represents a totally balanced situation for the conflicting stream circulating past north.

From Figures 9.3 and 9.4 the following can be stated:

i) The entry delays are sensitive to an imbalance in circulating flows. The change in delays purely because of a change in the proportion of circulation flows are significant.
ii) This sensitivity is amplified with an increase in traffic volumes (degree of saturation).
iii) The delay changes exponentially with an increase in ρ_n.
iv) The variation because of changes in ρ_w is significant for higher degrees of saturation and higher values of (ρ_n).
v) At low flows there are hardly any variations as a result of changes in ρ_w.
vi) These trends are similar for both circles.

vii) For the higher proportions of circulating traffic ($\rho_n > 50\%$), the gap acceptance behaviour is more conservative. Drivers waiting for a gap at north are less likely to accept a similar size gap in the entering stream than in the circulating stream. Entering stream refers to the next conflicting vehicle coming from the previous entry, while circulating stream refers to the next conflicting vehicle circulating around the circle.

viii) For the lower proportions ($\rho_n < 50\%$) the gap acceptance behaviour is less conservative. Drivers are more prepared to accept a similar size gap in the circulating stream than in the entering stream.

To evaluate the effect of the proportion of circulating traffic at west (ρ_w - the first upstream approach) on the delays at north, the results of the simulation analysis are summarized in Figures 9.5 and 9.6 for Circles One and Two respectively. These graphs show the variance in delays at north for a constant proportion of circulating traffic (ρ_n) past north and a variable proportion of circulating traffic past west (ρ_w). As the variability at low traffic flows and low proportions of circulating traffic are not significant only two of the higher flow scenarios (600 vph and 700 or 800 vph) are shown for only two of the higher proportions of circulating traffic past north (ρ_n = 0,5 and 0,75). A best fit straight line is also shown for each of the data sets.

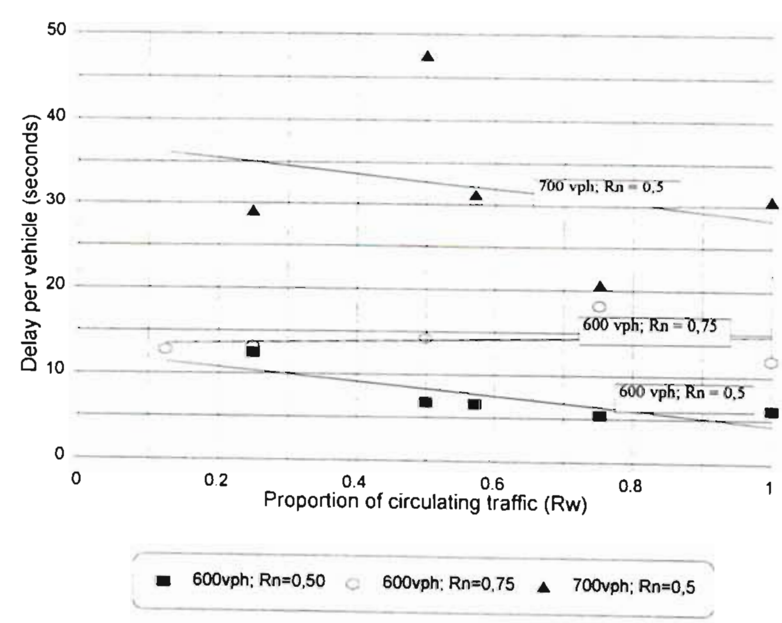


Figure 9.5: Vehicular delay versus proportion of circulating flows past west for Circle One

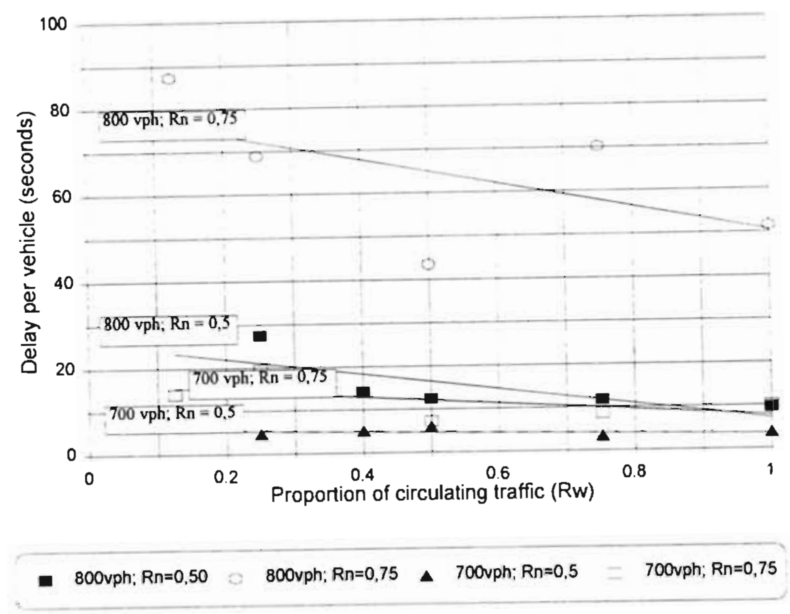


Figure 9.6: Vehicular delay versus proportion of circulating flows past west for Circle Two

From Figures 9.5 and 9.6 it follows that:

i) For the higher flow scenarios the results are somewhat scattered.

ii) Despite the scatter at the higher flows there is a definite trend of reduced delays with an increase in ρ_w, although this trend is not always significant.

iii) An increase in the proportion of circulating flows (ρ_w) at west means that more of the circulating vehicles past west are coming from south than from east. If the same gap acceptance behaviour applies to east as what has been shown above to apply to north, then an increase in circulating flow from south would reduce the entry flows from west and so reduce the delays at north.

9.5 Discussion of results

Unbalanced flows affect the average delays experienced by entering vehicles and therefore also the capacity of the approach. The effect on the delays can be positive (reduce delay) or negative (increase delay), depending on the imbalance. If the imbalance is caused by strong entering flows from the previous adjacent entry then the effect on the delay is negative, i.e. causing greater delay than if the flow is balanced. If the imbalance is caused by strong flows from the second upstream entry then the effect on delays is positive, i.e. the delays are lower than if the conflicting flows were balanced.

Table 9.4 shows the change in mean delay for two flow scenarios and for the two circles. The balanced situation with $\rho_n = 50\%$ is taken as the base against which the change is measured. These mean delays were obtained from the best fit exponential line, fitted through the data points as shown in Figure 9.3 and 9.4 for circles One and Two respectively.

Table 9.4 Change in mean delay versus ρ_n

ρ_n	Circle One				Circle Two			
	700 vph		400 vph		600 vph		400 vph	
	Delay per vehicle (sec)	% Change	Delay per vehicle (sec)	% Change	Delay per vehicle (sec)	% Change	Delay per vehicle (sec)	% Change
0%	2	33%	0.51	46%	3	40%	1.1	63%
25%	3	50%	0.75	68%	5	67%	1.3	74%
50%	6	100%	1.1	100%	7.5	100%	1.75	100%
75%	9	150%	1.5	136%	12.5	167%	2.3	131%
100%	17	283%	2.2	200%	18	240%	3.1	177%

From the summary in Table 9.4 it can be seen that the change in delay as a result of a change in the proportion of circulating traffic is not uniform across different flow scenarios neither across the two circles. Increasing the proportion of circulating flow from 50% to 75% could lead to an increase in delay of between 36% and 50% for Circle One and between 31% and 67% for Circle Two. An increase in the proportion of circulating traffic originating from the previous entry (west) would result in an increase in delays from north. A similar reduction in the proportion of circulating flows from west would result in a reduction in delays of between 26% and 50% depending on the circle and the flows.

The significant changes in delays as a result of changes in the proportions of circulating traffic might seem excessive, but what has to be kept in mind is that the simulations were executed with a constant critical gap. As discussed in Section 5.5 the lack of data for a wide range of entering and circulating flows prevented the development of a variable critical gap model. When employing a variable critical gap - which is more representative of what is happening in practice (Chung, 1993) - the sensitivity of the delays to a change in the proportion of circulating traffic would not be so great. Under higher flows or higher approach delays the critical gap would reduce, thereby reducing the delays. Similarly, under low flows the critical gaps might be larger (drivers are more relaxed) and thus an increase in the average approach delays.

The variable critical gap model included in SIDRA (Akçelik et al, 1995) is based on the geometric parameters and the circulating conflicting flow (see (2-44)) past the approach. Because of the effect of unbalanced flows it will be more accurate to base a variable critical gap on the approach delays rather than on the circulating conflicting flows, i.e. the critical gap will change as a function of the average approach delay and not the conflicting flow. The results of this section suggest that unbalanced circulating flows could mean significant increases or decreases in the approach delays. Therefore, it can be expected that similar to an increase in circulating flows, the increase or decrease in delays would have a similar effect on the drivers' critical gaps.

Notwithstanding the constant gap acceptance model employed in the program, the research results suggest a definite increase in approach delay for a significant proportion of circulating traffic from the previous entry and more favourable delays if the proportion of circulating traffic has less traffic from the previous entry. Although not significant, the imbalance of circulating flows past the previous entry also affects the approach delays.

9.6 Summary

In this chapter the effects of unbalanced flows on approach delays are investigated. The simulation program TRACSIM was employed to simulate four different flow scenarios with different proportions of circulating flow past north (the approach under investigation) and west (the first upstream approach). A definition of the proportion of circulating flows is proposed as being the proportion of flow from the first upstream entry divided by the total circulating flow.

The simulation results show that the entry delays are sensitive to a change in the balance of the circulating flows. A large proportion of entering flows from the previous entry in the circulating flows can result in a significant increase in the delays for similar circulating volumes. On the other hand, a low proportion of circulating traffic from the previous entry could lead to a decrease in the approach delays. Although the simulation program is based on a constant gap acceptance model which may result in unrealistically high or low delays, the results obtained here show the effect of an imbalance in circulating flows. The increase in delays can be as much as 200% or even more for high degrees of saturation.

This significant change in the simulated delays under imbalanced flows indicates that the constant critical gap model as used in the simulation program are not accurate at the extremes of the flow ranges. A variable critical gap model would probably give a more accurate result. However, this variable critical gap would have to be a function of entry delay and not of the circulating traffic. Entry delay is not only affected by circulating flow, but also by the balance in the circulating flow. Moreover, an imbalance in circulating flow increase the entry delay as does an increase in circulating flows. Therefore, a change in the balance of the circulating flows will result in a change in the drivers' critical gaps.

The entry delays are not only sensitive to the imbalance of flows past the approach, but are also influenced - although to a much lesser extent - by the imbalance of flows past the previous entry.

CHAPTER 10: CONCLUSIONS AND RECOMMENDATIONS

The goal for this research as identified in Chapter One is as follows:

To study traffic operations at traffic circles under South African conditions and to verify and improve, where possible, the existing models used for traffic circle analysis. Two important aspects to be investigated are the gap acceptance process and the effect of unbalanced flows at circles.

To reach the above goal a research outline was defined. The first step was to conduct a comprehensive literature survey to identify the important aspects regarding traffic circles and the various analysis models that are being used internationally. Secondly the accuracy of the available traffic circle analysis models had to be verified for South African conditions. With the sparse distribution of traffic circles, and hence the unavailability of a large data source in mind, the third step would be to develop a simulation program to assist in the research process. Fourthly, the simulation program had to be calibrated and validated from data obtained from detailed field studies at suitable local traffic circles. Because the gap acceptance process is such a vital part of the operation of a traffic circle, the fifth step was to investigate the gap acceptance process and the use of gap acceptance models based on variables other than time, such as distance and position in the circle. To place the research in context, the sixth step was to compare any new methods or models developed during this research with other widely used models. Finally the simulation model was used to investigate the effect of unbalanced flows on the entry delays.

The estimation of input parameters and validation of the simulation model was completed for a range of flows, which were lower than capacity flows. The validation was therefore conducted based on approach delay rather than capacity. The reason for concentrating on delay rather than on capacity is the scarcity of circles operating under capacity conditions. The model can be used to estimate capacity of an approach or a circle, but will then be used outside the range in which it was validated.

10.1 Summary of findings

The following list of findings resulted either from the literature survey or from the work completed during this research. The findings are discussed in the same order as the structure of this thesis. The italicised text serves to highlight the significant findings which followed from this research.

i) There are basically three schools of thought concerning the analysis of traffic circles. The empirical approach has found the greatest application in Europe, while the analytical approach has found favour in Australia. The simulation approach has also been used by a number of researchers in both Europe and Australia. Most of the research on circles was conducted in Australia and Europe with little originating in North or South America.

ii) In South Africa , two software programs have found wide application in the analysis of traffic circles. These programs are ARCADY (developed in the UK and based on an empirical approach) and SIDRA (developed in Australia and based on an analytical approach). SIDRA, finding an increase in application world wide, is a user-friendly program with attractive graphics to display output and results.

iii) *Both ARCADY's and SIDRA's delay estimates are significantly lower than what has been observed at local traffic circles and their application in the analysis of local traffic circles should be viewed with circumspection.*

iv) The event-scanning approach can be employed to simulate traffic operations at a traffic circle and was used to develop TRACSIM, the simulation program used during this research. Most of the reviewed simulation programs employed a time-update approach.

v) Based on a sensitivity analysis it followed that approach delays are not sensitive to the approach headway distribution and its calibration parameters or the minimum headway between circulating vehicles. On the other hand, the approach delays are sensitive to the size of the critical gaps, the circulating speeds and the move-up times for vehicles moving up in the queue.

vi) *From the gap and lag acceptance observations it follows that:*

a) there is a difference between accepting gaps/lags in the entering or in the circulating stream of conflicting traffic,

b) *there is a significant difference between the critical gaps and the critical lags, and*

c) *the mean critical gaps/lags observed at local traffic circles are larger than gaps/lags observed in other countries.*

vii) The larger critical gaps and lags indicate that approach delays at local circles should be greater than delays observed elsewhere, for similar traffic flows and geometric layouts. This explains why analysis models developed abroad underestimate delays at local circles (see iii above).

viii) The variation in circulating speeds at a circle is small, and the circulating speeds are a function of the circulating diameter, decreasing with a decrease in diameter.

ix) Gap and lag acceptance models based on critical distances, as used in the simulation program TRACSIM, provide delay estimates similar if not better than similar models based on critical times.

x) *A proposed method to estimate critical distances from the geometric layout of the circle has proved to be at least as accurate as the other methods where critical gaps and distances are obtained from field observations.*

xi) TRACSIM produces delays similar to the Australian analytical method contained in SIDRA if fixed critical gaps and move-up times are used and where these gaps and move-up times are obtained from local observations.

xii) *There is evidence to suggest that the critical gap (time or distance based) is not fixed, but should vary with at least the conflicting flows. At low rates of conflicting flows the critical gaps tend to be larger than at high rates of conflicting flows.*

xiii) *The entry delay is sensitive to the balance in circulating flows past the entering approach. The entry delay increases with an increase in the proportion of circulating traffic originating from the previous entry and vice versa.*

xiv) Although to a much lesser extent, the entry delay is also affected by an imbalance of circulating flow past the previous upstream entry.

xv) Entry delay is a more reliable predictor of the variance in drivers' critical gaps than conflicting flow. If a variable critical gap model is based on conflicting flows only, then the critical gap will not change if the conflicting flow remain constant. However, an imbalance in the conflicting flow, even if the flow rate remains constant, will have an effect on the entry delay (see xiii above).

xvi) The simulation model, TRACSIM, developed as part of this research and calibrated for local conditions under a range of flows at single entry and circulating lane circles can be applied to:

a) Do an operational analysis of existing circles.
b) Test the effect of an increase in traffic flows at an existing circle.
c) Plan/design new single lane circles.
d) Evaluate other analysis models.

10.2 Conclusions

From the above findings and the research completed during this study, the following can be concluded:

i) Event scanning can be used to simulate traffic operations at traffic circles and TRACSIM can be used as a tool to study traffic circles.

ii) The existing analysis methods for traffic circles (ARCADY and SIDRA) should be applied with great circumspection under local conditions.

iii) There is a difference between gaps and lags, and also between gaps and lags accepted in the entering or circulating conflicting traffic streams.

iv) Critical distances instead of critical times provide a similar if not better estimate of traffic delays when used in the gap acceptance model of a simulation program, i.e. gap acceptance is not necessarily a function of time.

v) The critical distance is a function of the circle geometry, because the geometry does not only regulate the circulating speeds, but it also defines the positions upstream where conflicting vehicles exit and enter the circle.

vi) The proposed method of determining the critical distances from the circle geometry results in similar, if not better, delay estimates than when compared to the critical distances determined from field observations.

vii) Unbalanced circulating flows have a significantly different effect on entry delays than balanced flows. The imbalance could increase entry delays if the imbalance is due to a high proportion of conflicting flow entering from the adjacent upstream approach or could decrease delays if the imbalance is due to a low proportion of conflicting flow entering from the adjacent upstream approach.

viii) Because of the effect of unbalanced flows, a variable critical gap model should not be based on conflicting flows only, but also on the average entry delays.

ix) Computer simulation lends itself perfectly to deal with effects such as an imbalance in conflicting flows, difference between gaps/lags accepted in circulating or entering traffic streams, and the difference between gaps and lags. These effects will be demanding to incorporate in any analytical or empirical model.

10.3 Recommendations

Following from the results of this research it is recommended that:

i) The data base on which this research is based be expanded to include more circles and a greater range of traffic flows so that:

* the variation of critical gaps with conflicting traffic and the effect of an imbalance in conflicting flows can be defined more accurately.
* the proposed method for determining critical distance gaps can be extended to a greater range of circles, and
* the difference between gaps in the entering and circulating streams can be quantified.

ii) Local traffic engineers take into account the results of this study when analysing and designing traffic circles, especially when the software programs ARCADY or SIDRA are used.

iii) TRACSIM be extended to multi-lane circles and that it be made more user-friendly so that it can be made available for general application.

iv) TRACSIM be used :

a) for the operational analysis of existing single lane circles,

b) to test the effect of an increase in traffic flows at an existing circle,

c) to plan/design new single lane circles , and

d) to evaluate other analysis models.

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APPENDIX A: SUMMARY OF TRAFFIC COUNTS

TABLE A1 : SUMMARY OF TRAFFIC SURVEY

CHATSWORTH CIRCLE - 30 / 7 / 1993 AM

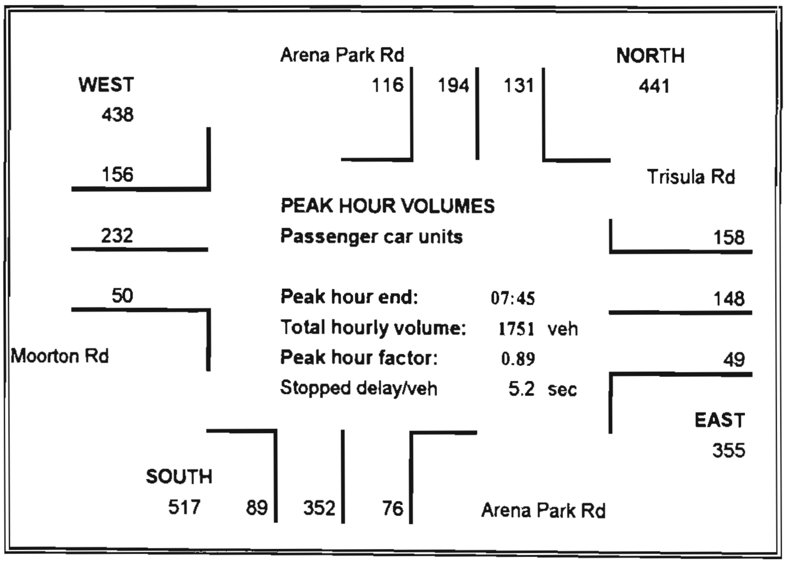
15 minute Volumes (PCU's)

15 min's Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
06:45	25	17	22	64	3	13	21	37	9	58	12	79	37	28	13	78	258
07:00	24	42	19	85	10	28	42	80	16	86	25	127	36	52	9	97	389
07:15	25	54	25	104	11	33	28	72	13	97	22	132	35	51	15	101	409
07:30	35	46	34	115	11	33	43	87	23	100	16	139	45	61	12	118	459
07:45	47	52	38	137	17	54	45	116	37	69	13	119	40	68	14	122	494
08:00	20	41	18	79	15	23	19	57	17	44	12	73	20	30	8	58	267
08:15	9	11	13	33	3	12	18	33	3	10	7	20	10	16	2	28	114

Hourly Volumes (PCU's)

Hour Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
07:30	109	159	100	368	35	107	134	276	61	341	75	477	153	192	49	394	1515
07:45	131	194	116	441	49	148	158	355	89	352	76	517	156	232	50	438	1751
08:00	127	193	115	435	54	143	135	332	90	310	63	463	140	210	49	399	1629
08:15	111	150	103	364	46	122	125	293	80	223	48	351	115	175	36	326	1334
08:30	76	104	69	249	35	89	82	206	57	123	32	212	70	114	24	208	875
08:45	29	52	31	112	18	35	37	90	20	54	19	93	30	46	10	86	381

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	80%	77%	90%	93%	89%
MODAL SPLIT					
Cars	82%	83%	88%	88%	84%
Taxis	11%	14%	6%	7%	10%
Buses	5%	1%	4%	3%	3%
Trucks	2%	3%	2%	2%	2%
STOPPED DELAY PER APPROACH (Secs per vehicle)					
06:45	2.2	1.6	6.8	10.5	6.0
07:00	1.9	1.3	4.8	5.3	3.5
07:15	4.3	2.1	3.7	6.0	4.2
07:30	2.6	2.4	7.3	5.8	4.8
07:45	5.8	4.1	14.5	6.9	7.8
08:00	2.5	2.3	1.9	2.6	2.3
08:15	0.0	0.0	0.0	0.0	0.0
08:30	0.0	0.0	0.0	0.0	0.0
08:45	0.0	0.0	0.0	0.0	0.0
AVG NO OF STOPS PER 100 VEHICLES					
06:45	16.4	16.2	74.1	30.8	38.4
07:00	15.9	12.5	33.1	34.0	25.3
07:15	35.6	34.7	38.3	34.7	36.1
07:30	22.6	20.1	35.3	37.3	29.7
07:45	34.3	30.2	39.5	61.1	41.2
08:00	43.7	38.6	36.3	60.3	44.2
08:15	6.1	9.1	15.0	3.6	7.9
08:30					
08:45					



15 MINUTE VOLUMES

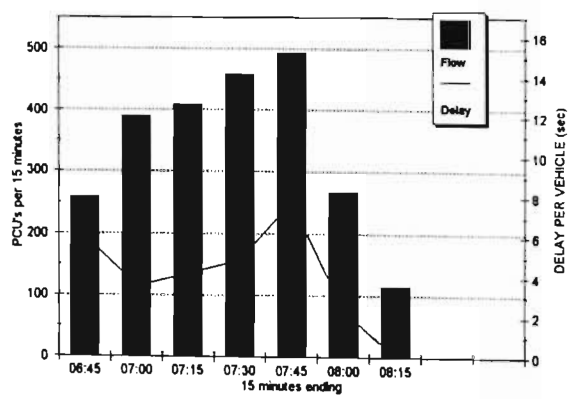


TABLE A2 : SUMMARY OF TRAFFIC SURVEY

CHATSWORTH CIRCLE - 16 / 8 / 1993 PM

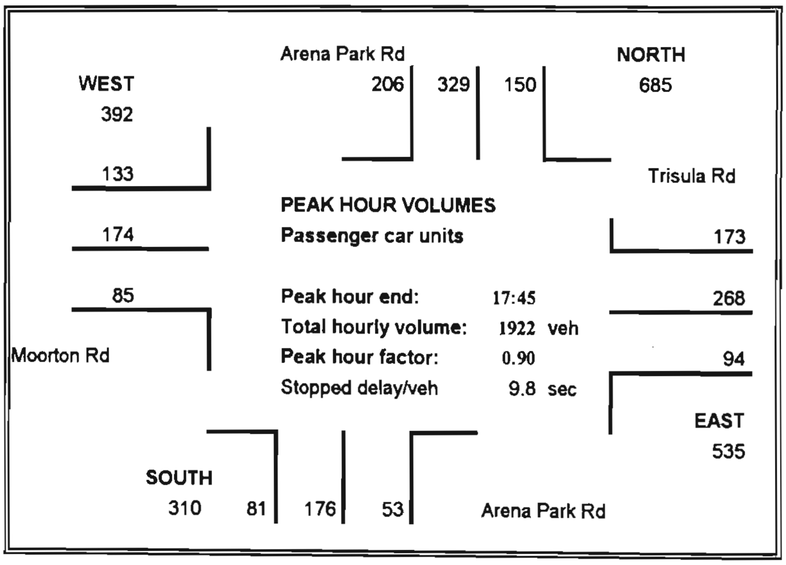
15 minute Volumes (PCU's)

15 min's Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
15:45	8	15	6	29	6	9	11	26	3	8	2	13	8	9	1	18	86
16:00	30	37	25	92	10	30	32	72	10	31	12	53	21	26	15	62	279
16:15	32	33	18	83	19	36	31	86	9	34	7	50	27	41	4	72	291
16:30	31	45	28	104	11	35	30	76	12	24	4	40	21	31	10	62	282
16:45	28	61	35	124	24	46	34	104	11	36	8	55	24	36	11	71	354
17:00	45	80	48	173	18	81	44	143	23	49	13	85	25	37	27	89	490
17:15	39	100	39	178	30	75	42	147	17	40	9	66	29	50	19	98	489
17:30	34	86	60	180	17	71	54	142	21	48	23	92	48	47	26	121	535
17:45	32	63	59	154	29	41	33	103	20	39	8	67	31	40	13	84	408

Hourly Volumes (PCU's)

Hour Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
16:30	101	130	77	308	46	110	104	260	34	97	25	156	77	107	30	214	938
16:45	121	176	106	403	64	147	127	338	42	125	31	198	93	134	40	267	1206
17:00	136	219	129	484	72	198	139	409	55	143	32	230	97	145	52	294	1417
17:15	143	286	150	579	83	237	150	470	63	149	34	246	99	154	67	320	1615
17:30	146	327	182	655	89	273	174	536	72	173	53	298	126	170	83	379	1868
17:45	150	329	206	685	94	268	173	535	81	176	53	310	133	174	85	392	1922

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	95%	91%	81%	84%	90%
MODAL SPLIT					
Cars	84%	94%	90%	83%	89%
Taxis	9%	3%	6%	10%	6%
Buses	5%	0%	1%	6%	2%
Trucks	2%	3%	3%	2%	3%
STOPPED DELAY PER APPROACH [Secs per vehicle]					
15:45	1.7	0.8	0.0	0.6	0.9
16:00	2.8	2.4	0.9	1.0	1.9
16:15	0.8	1.0	1.6	2.2	1.4
16:30	2.6	2.6	1.0	1.5	2.1
16:45	0.8	4.0	2.7	2.1	2.3
17:00	6.6	14.5	6.6	8.4	9.2
17:15	8.8	23.0	4.2	1.8	11.1
17:30	9.8	14.4	8.3	5.2	9.7
17:45	1.4	23.4	10.9	4.6	9.2
AVG NO OF STOPS PER 100 VEHICLES					
15:45	12.1	30.8	26.9	5.6	18.6
16:00	28.8	19.4	14.2	10.5	19.5
16:15	15.1	10.5	25.0	11.1	14.4
16:30	12.5	30.3	31.3	8.1	19.0
16:45	11.7	45.2	20.0	21.1	24.7
17:00	9.2	53.1	28.2	34.8	30.0
17:15	29.2	59.2	26.5	26.5	37.3
17:30					
17:45					



15 MINUTE VOLUMES

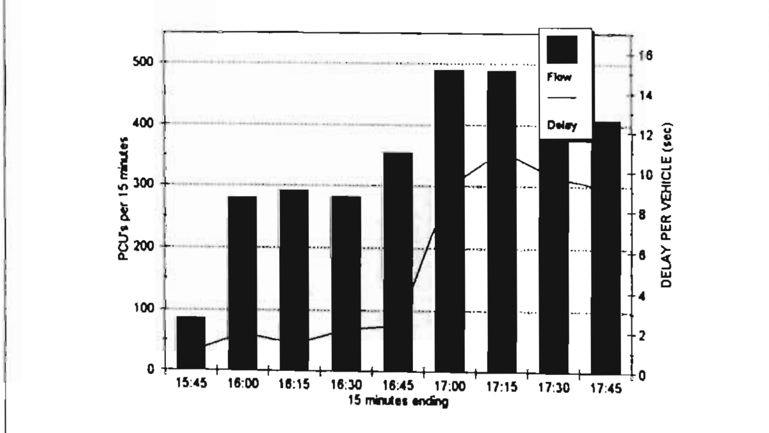


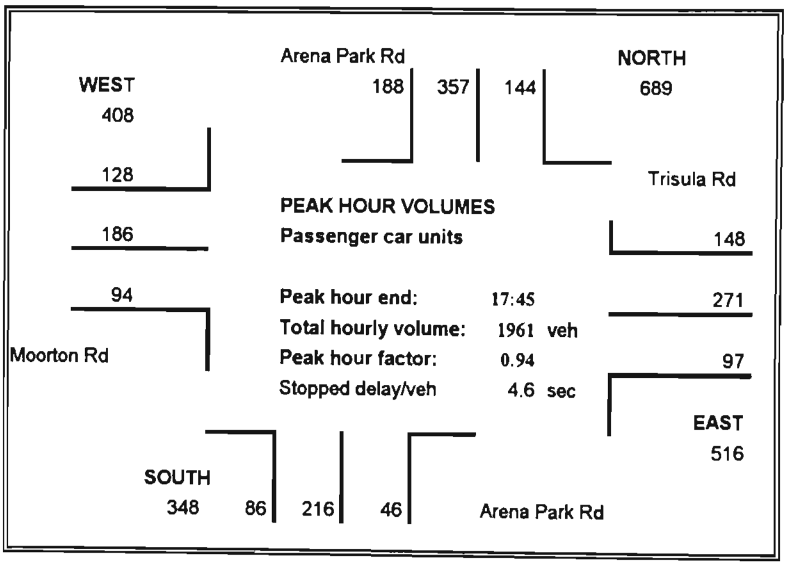
TABLE A3 : SUMMARY OF TRAFFIC SURVEY

CHATSWORTH CIRCLE - 17 / 8 / 1993 PM

15 min's Ending	15 minute Volumes (PCU's)																
	North				East				South				West				Total
	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	
15:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
16:00	23	30	24	77	13	29	31	73	8	34	7	49	19	31	5	55	254
16:15	29	42	24	95	15	37	37	89	9	28	10	47	17	39	15	71	302
16:30	25	47	36	108	13	42	17	72	10	31	9	50	16	30	3	49	279
16:45	34	59	36	129	18	59	36	113	18	34	4	56	17	22	15	54	352
17:00	43	119	49	211	20	54	32	106	25	53	13	91	32	39	19	90	498
17:15	41	90	48	179	26	72	41	139	21	55	8	84	29	45	37	111	513
17:30	34	87	62	183	24	80	33	137	22	57	14	93	35	53	20	108	521
17:45	26	61	29	116	27	65	42	134	18	51	11	80	32	49	18	99	429

Hour Ending	Hourly Volumes (PCU's)																
	North				East				South				West				Total
	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	
16:30	77	119	84	280	41	108	85	234	27	93	26	146	52	100	23	175	835
16:45	111	178	120	409	59	167	121	347	45	127	30	202	69	122	38	229	1187
17:00	131	267	145	543	66	192	122	380	62	146	36	244	82	130	52	264	1431
17:15	143	315	169	627	77	227	126	430	74	173	34	281	94	136	74	304	1642
17:30	152	355	195	702	88	265	142	495	86	199	39	324	113	159	91	363	1884
17:45	144	357	188	689	97	271	148	516	86	216	46	348	128	186	94	408	1961

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	82%	93%	92%	94%	94%
MODAL SPLIT					
Cars	82%	95%	88%	81%	89%
Taxis	11%	4%	8%	12%	7%
Buses	6%	0%	1%	6%	2%
Trucks	1%	1%	3%	2%	2%
STOPPED DELAY PER APPROACH [Secs per vehicle]					
15:45					
16:00	0.3	1.2	0.8	0.7	0.7
16:15	1.4	1.8	3.2	1.8	1.9
16:30	0.7	3.9	2.4	0.4	1.8
16:45	0.3	2.1	3.0	2.0	1.6
17:00	1.5	8.0	7.0	2.0	4.0
17:15	4.4	10.6	7.5	3.1	6.3
17:30	2.1	10.5	3.5	2.1	4.6
17:45	2.0	4.4	4.3	2.2	3.2
AVG NO OF STOPS PER 100 VEHICLES					
15:45					
16:00	20.1	10.3	16.3	13.6	15.2
16:15	20.5	19.1	27.7	16.9	20.4
16:30	11.1	41.0	11.0	26.5	21.5
16:45	3.1	27.4	19.6	27.8	17.3
17:00	10.7	64.2	18.1	32.2	27.3
17:15	30.2	71.2	32.1	43.7	44.5
17:30					
17:45					



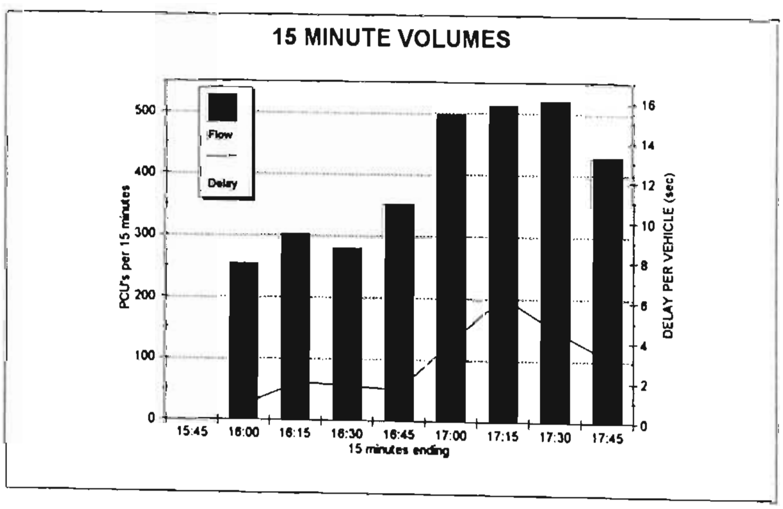


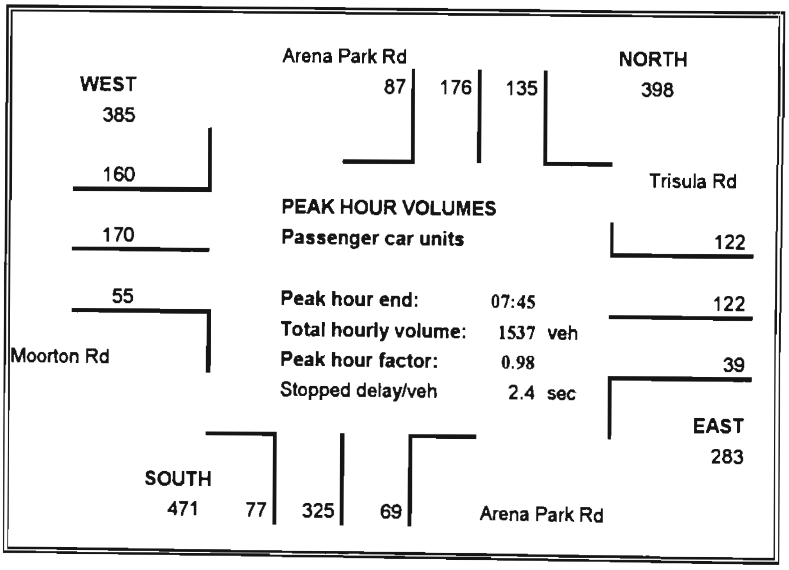
TABLE A4 : SUMMARY OF TRAFFIC SURVEY

CHATSWORTH CIRCLE - 18 / 8 / 1993 AM

15 min's Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
	15 minute Volumes (PCU's)																
06:30	14	27	18	59	6	18	9	33	12	49	7	68	40	34	8	82	242
06:45	23	32	20	75	7	34	20	61	5	92	21	118	50	57	11	118	372
07:00	33	39	18	90	9	30	28	67	23	91	11	125	42	38	18	98	380
07:15	25	52	20	97	9	23	33	65	17	83	20	120	37	42	11	90	372
07:30	32	34	31	97	5	29	33	67	11	94	20	125	47	48	8	103	392
07:45	45	51	18	114	16	40	28	84	26	57	18	101	34	42	18	94	393
08:00	19	13	7	39	6	19	16	41	9	35	8	52	9	35	6	50	182
08:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
08:30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Hour Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
	Hourly Volumes (PCU's)																
07:15	95	150	76	321	31	105	90	226	57	315	59	431	169	171	48	388	1366
07:30	113	157	89	359	30	116	114	260	56	360	72	488	176	185	48	409	1516
07:45	135	176	87	398	39	122	122	283	77	325	69	471	160	170	55	385	1537
08:00	121	150	76	347	36	111	110	257	63	269	66	398	127	167	43	337	1339
08:15	96	98	56	250	27	88	77	192	46	186	46	278	90	125	32	247	967
08:30	64	64	25	153	22	59	44	125	35	92	26	153	43	77	24	144	575

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	87%	84%	82%	94%	98%
MODAL SPLIT					
Cars	80%	83%	87%	89%	83%
Taxis	12%	15%	7%	7%	11%
Buses	5%	1%	4%	2%	3%
Trucks	2%	2%	2%	1%	2%
STOPPED DELAY PER APPROACH [Secs per vehicle]					
06:30	0.7	1.5	0.7	1.0	0.9
06:45	3.2	3.1	1.3	4.2	2.9
07:00	1.6	3.0	1.4	3.3	2.2
07:15	1.6	2.6	2.8	2.2	2.3
07:30	2.1	1.3	2.2	5.0	2.8
07:45	2.5	2.5	2.0	2.0	2.3
08:00	1.8	1.7	1.7	0.6	1.4
08:15	0.0	0.0	0.0	0.0	0.0
08:30	0.0	0.0	0.0	0.0	0.0
AVG NO OF STOPS PER 100 VEHICLES					
06:30	13.6	19.7	21.3	16.5	17.6
06:45	17.3	21.3	36.0	14.8	23.1
07:00	14.4	35.8	32.4	24.5	26.7
07:15	12.9	20.0	20.8	21.1	18.7
07:30	18.0	20.9	22.8	21.4	20.9
07:45	16.7	22.0	23.8	16.0	19.5
08:00	12.8	12.2	15.4	6.0	11.5
08:15					
08:30					



15 MINUTE VOLUMES

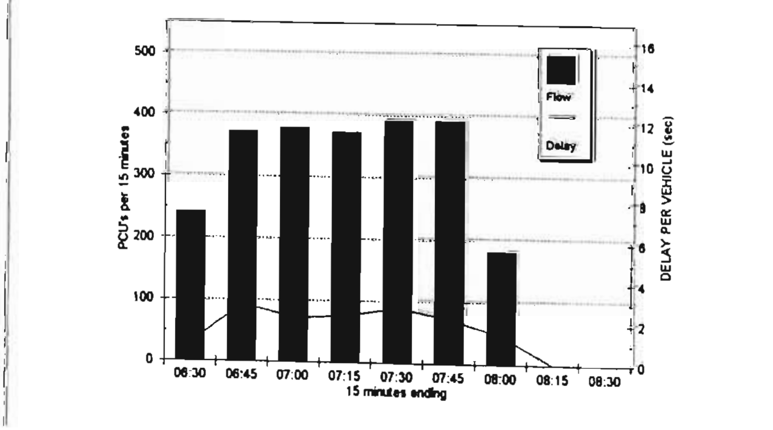


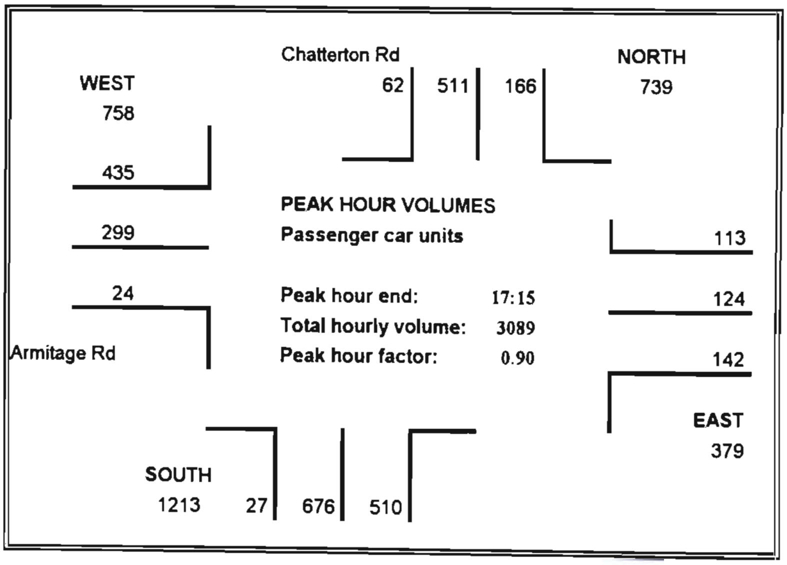
TABLE A5 : SUMMARY OF TRAFFIC SURVEY

CHATTERTON CIRCLE - 31 /01/ 1994 PM

15 min's Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
	15 minute Volumes (PCU's)																
16:00	42	131	22	195	35	28	32	95	8	114	39	161	80	52	6	138	589
16:15	36	121	19	176	32	37	22	91	6	140	52	198	115	63	10	188	653
16:30	42	119	22	183	40	34	19	93	6	138	66	210	92	54	13	159	645
16:45	61	144	16	221	51	34	29	114	11	166	130	307	119	95	6	220	862
17:00	39	139	13	191	29	26	38	93	6	169	111	286	116	70	3	189	759
17:15	24	109	11	144	22	30	27	79	4	203	203	410	108	80	2	190	823
17:30	16	71	15	102	13	29	43	85	5	129	104	238	115	73	3	191	616
17:45	14	51	22	87	8	36	21	65	71	32	0	103	0	79	52	131	386
18:00																	

Hour Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
	Hourly Volumes (PCU's)																
16:45	181	515	79	775	158	133	102	393	31	558	287	876	406	264	35	705	2749
17:00	178	523	70	771	152	131	108	391	29	613	359	1001	442	282	32	756	2919
17:15	166	511	62	739	142	124	113	379	27	676	510	1213	435	299	24	758	3089
17:30	140	463	55	658	115	119	137	371	26	667	548	1241	458	318	14	790	3060
17:45	93	370	61	524	72	121	129	322	86	533	418	1037	339	302	60	701	2584
18:00	54	231	48	333	43	95	91	229	80	364	307	751	223	232	57	512	1825

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	84%	83%	86%	74%	90%
MODAL SPLIT					
Cars	91%	95%	98%	96%	94%
Taxis	6%	1%	1%	2%	3%
Buses	0%	0%	0%	0%	0%
Trucks	3%	3%	1%	1%	3%
TOTAL DELAY PER APPROACH [Secs per vehicle]					
16:00	8.1	12.8	11.8	10.8	10.5
16:15	11.7	13.6	9.7	12.3	11.5
16:30	14.5	11.9	10.1	10.8	11.8
16:45	30.4	17.2	10.4	21.0	19.2
17:00	14.2	13.8	10.2	21.2	14.4
17:15	19.5	10.5	11.0	56.2	22.9
17:30	11.1	11.6	10.5	14.7	12.0
17:45	8.7	9.3	5.9	10.6	8.7
18:00					
AVG NO OF STOPS PER 100 VEHICLES					
16:00					
16:15					
16:30					
16:45					
17:00					
17:15					
17:30					
17:45					
18:00					



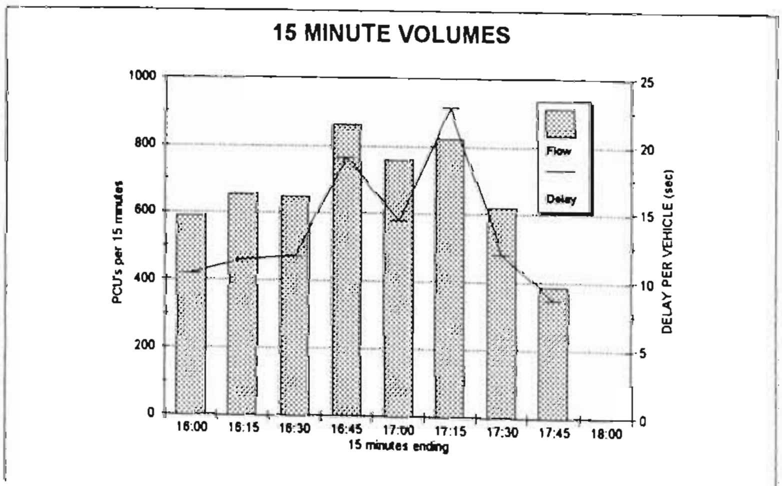


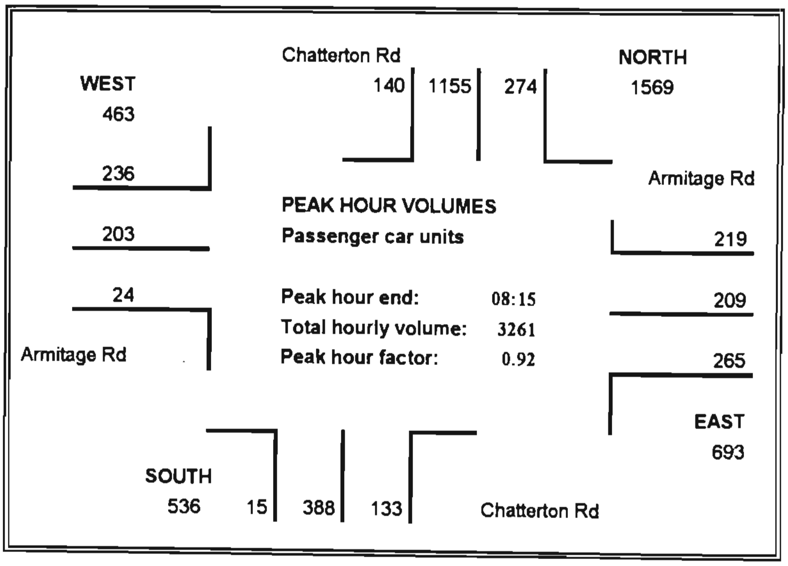
TABLE A6 : SUMMARY OF TRAFFIC SURVEY

CHATTERTON CIRCLE - 02 /02/ 1994 AM

15 min's Ending	15 minute Volumes (PCU's) North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
06:45	30	87	18	135	30	26	40	96	3	41	7	51	46	20	1	67	349
07:00	27	133	24	184	42	69	35	146	9	46	10	65	32	36	5	73	468
07:15	43	204	39	286	49	67	44	160	7	64	18	89	34	48	4	86	621
07:30	75	303	48	426	68	59	61	188	5	103	20	128	55	63	4	122	864
07:45	82	330	28	440	60	53	73	186	1	111	34	146	53	50	7	110	882
08:00	72	324	39	435	82	54	56	192	6	96	32	134	65	50	8	123	884
08:15	45	198	25	268	55	43	29	127	3	78	47	128	63	40	5	108	631
08:30	28	147	24	199	42	25	24	91	38	34	3	75	2	63	29	94	459
08:45																	

Hour Ending	Hourly Volumes (PCU's) North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
07:30	175	727	129	1031	189	221	180	590	24	254	55	333	167	167	14	348	2302
07:45	227	970	139	1336	219	248	213	680	22	324	82	428	174	197	20	391	2835
08:00	272	1161	154	1587	259	233	234	726	19	374	104	497	207	211	23	441	3251
08:15	274	1155	140	1569	265	209	219	693	15	388	133	536	236	203	24	463	3261
08:30	227	999	116	1342	239	175	182	596	48	319	116	483	183	203	49	435	2856
08:45	145	669	88	902	179	122	109	410	47	208	82	337	130	153	42	325	1974

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	89%	90%	92%	94%	92%
MODAL SPLIT					
Cars	91%	95%	84%	94%	90%
Taxis	6%	2%	10%	3%	6%
Buses	0%	0%	0%	0%	0%
Trucks	2%	2%	5%	2%	3%
TOTAL DELAY PER APPROACH [Secs per vehicle]					
06:45	5.8	12.4	8.9	6.5	8.2
07:00	6.4	14.1	8.0	7.4	9.2
07:15	9.7	19.3	10.3	8.4	12.1
07:30	10.6	27.6	11.5	10.7	14.4
07:45	13.4	33.6	10.9	8.6	16.6
08:00	16.3	35.8	9.3	8.4	18.4
08:15	8.3	16.2	9.0	8.0	10.0
08:30	6.8	11.3	5.5	10.3	8.2
08:45					
AVG NO OF STOPS PER 100 VEHICLES					
06:45					
07:00					
07:15					
07:30					
07:45					
08:00					
08:15					
08:30					
08:45					



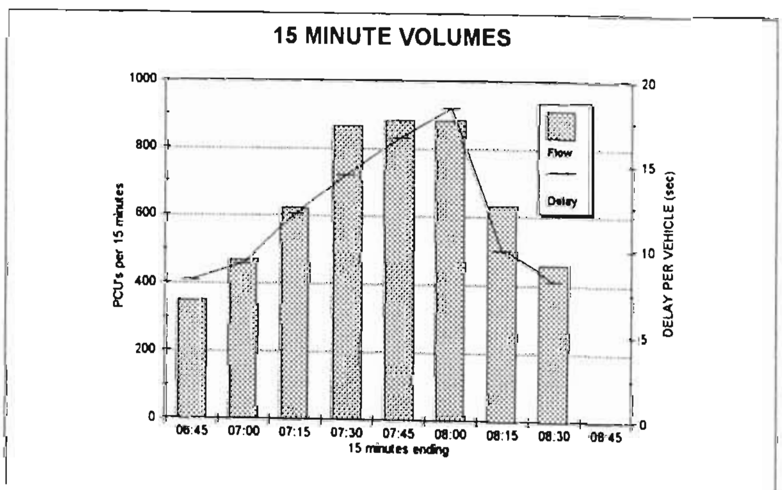


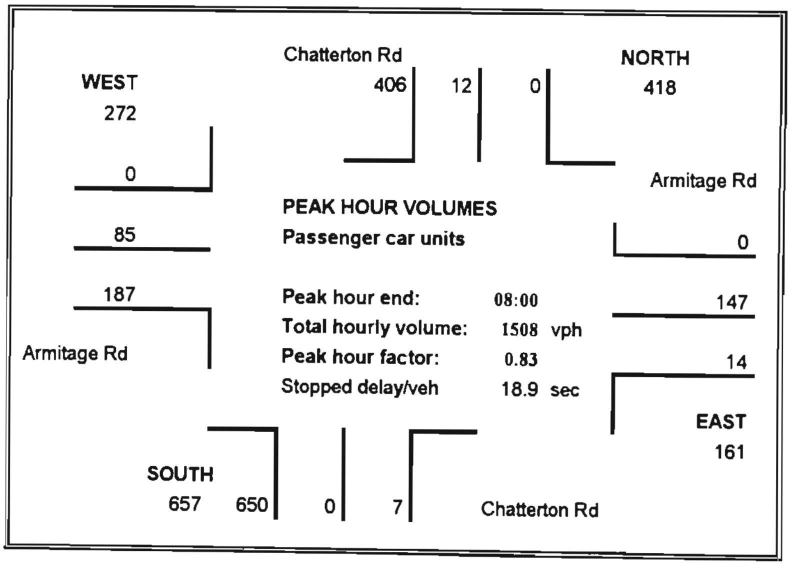
TABLE A7 : SUMMARY OF TRAFFIC SURVEY

PINETOWN CIRCLE - 15 /08/ 1995 AM

15 min's Ending	15 minute Volumes (PCU's)																
	North				East				South				West				Total
	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	
06:45	0	4	25	29	1	12	0	13	56	0	1	57	0	6	17	23	122
07:00	0	0	54	54	0	18	0	18	85	0	3	88	0	9	43	52	212
07:15	0	2	66	68	1	35	0	36	107	0	1	108	0	14	32	46	258
07:30	0	3	99	102	5	37	0	42	166	0	1	167	0	17	57	74	385
07:45	0	6	108	114	3	36	0	39	174	0	3	177	0	30	50	80	410
08:00	0	1	133	134	5	39	0	44	203	0	2	205	0	24	48	72	455
08:15	0	3	83	86	2	22	0	24	96	0	2	98	0	14	33	47	255
08:30																	
08:45																	

Hour Ending	Hourly Volumes (PCU's)																
	North				East				South				West				Total
	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	
07:30	0	9	244	253	7	102	0	109	414	0	6	420	0	46	149	195	977
07:45	0	11	327	338	9	126	0	135	532	0	8	540	0	70	182	252	1265
08:00	0	12	406	418	14	147	0	161	650	0	7	657	0	85	187	272	1508
08:15	0	13	423	436	15	134	0	149	639	0	8	647	0	85	188	273	1505
08:30																	
08:45																	

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	78%	91%	80%	85%	83%
MODAL SPLIT					
Cars	95%	95%	92%	84%	94%
Taxis	2%	4%	7%	14%	4%
Buses	0%	0%	0%	0%	0%
Trucks	2%	0%	1%	1%	1%
STOPPED DELAY PER APPROACH [Secs per vehicle]					
06:45	5.2	3.8	1.4	0.0	2.3
07:00	4.1	5.0	3.2	0.2	2.8
07:15	2.9	5.0	5.7	0.9	4.0
07:30	5.9	11.4	30.2	0.4	16.0
07:45	4.7	6.4	21.5	0.4	11.3
08:00	3.9	7.5	77.5	0.1	36.8
08:15	3.7	3.3	5.7	0.6	3.9
08:30	0.0	0.0	0.0	0.0	0.0
08:45	0.0	0.0	0.0	0.0	0.0
AVG NO OF STOPS PER 100 VEHICLES					
06:45					
07:00					
07:15					
07:30					
07:45					
08:00					
08:15					
08:30					
08:45					



15 MINUTE VOLUMES

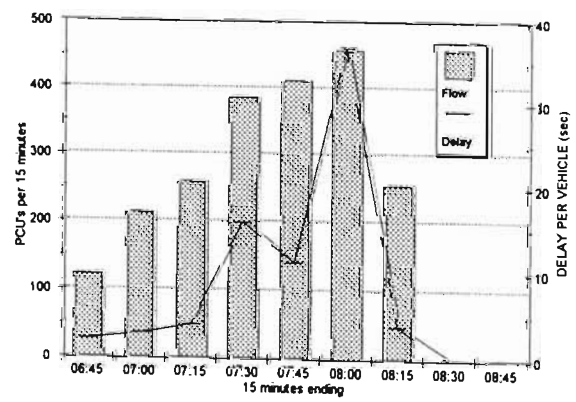


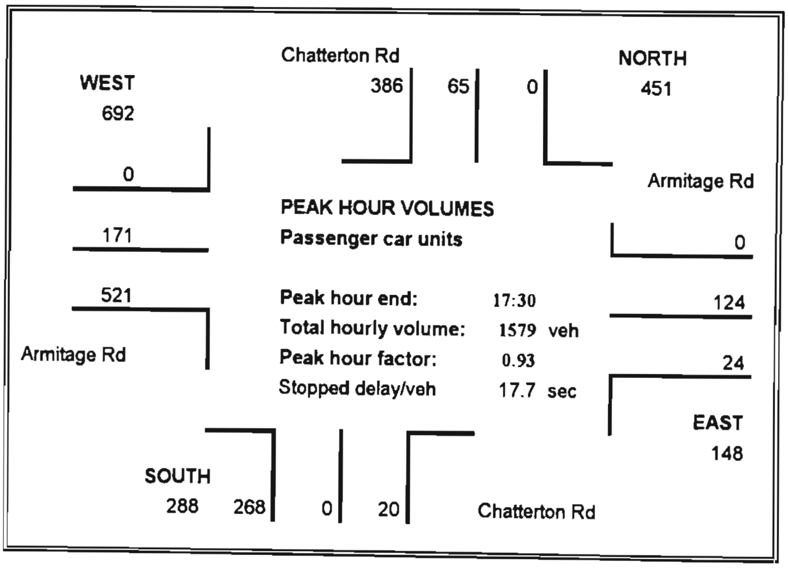
TABLE A8 : SUMMARY OF TRAFFIC SURVEY

PINETOWN CIRCLE - 15 /08/ 1995 PM

15 min's Ending	15 minute Volumes (PCU's) North L	T	R	Tot	East L	T	R	Tot	South L	T	R	Tot	West L	T	R	Tot	Total
16:45	0	13	98	111	3	29	0	32	73	0	4	77	0	40	118	158	378
17:00	0	15	100	115	3	16	0	19	76	0	7	83	0	52	154	206	423
17:15	0	20	90	110	12	61	0	73	57	0	4	61	0	56	117	173	417
17:30	0	17	98	115	6	18	0	24	62	0	5	67	0	23	132	155	361
17:45	0	4	48	52	0	10	0	10	30	0	2	32	0	16	47	63	157
18:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18:30																	
18:45																	

Hour Ending	Hourly Volumes (PCU's) North L	T	R	Tot	East L	T	R	Tot	South L	T	R	Tot	West L	T	R	Tot	Total
17:30	0	65	386	451	24	124	0	148	268	0	20	288	0	171	521	692	1579
17:45	0	56	336	392	21	105	0	126	225	0	18	243	0	147	450	597	1358
18:00	0	41	236	277	18	89	0	107	149	0	11	160	0	95	296	391	935
18:15	0	21	146	167	6	28	0	34	92	0	7	99	0	39	179	218	518
18:30																	
18:45																	

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	98%	51%	87%	84%	93%
MODAL SPLIT					
Cars	90%	100%	90%	98%	93%
Taxis	9%	0%	9%	2%	6%
Buses	0%	0%	0%	0%	0%
Trucks	1%	0%	1%	1%	1%
STOPPED DELAY PER APPROACH [Secs per vehicle]					
16:45	33.0	10.0	8.7	0.8	12.6
17:00	61.8	8.4	4.8	0.3	18.3
17:15	82.8	24.4	7.0	0.3	27.3
17:30	29.0	12.5	5.2	0.5	11.3
17:45	6.2	8.0	2.8	0.3	3.2
18:00	0.0	0.0	0.0	0.0	0.0
18:15	0.0	0.0	0.0	0.0	0.0
18:30	0.0	0.0	0.0	0.0	0.0
18:45	0.0	0.0	0.0	0.0	0.0
AVG NO OF STOPS PER 100 VEHICLES					
16:45					
17:00					
17:15					
17:30					
17:45					
18:00					
18:15					
18:30					
18:45					



PEAK HOUR VOLUMES
Passenger car units

Peak hour end: 17:30
Total hourly volume: 1579 veh
Peak hour factor: 0.93
Stopped delay/veh 17.7 sec

15 MINUTE VOLUMES

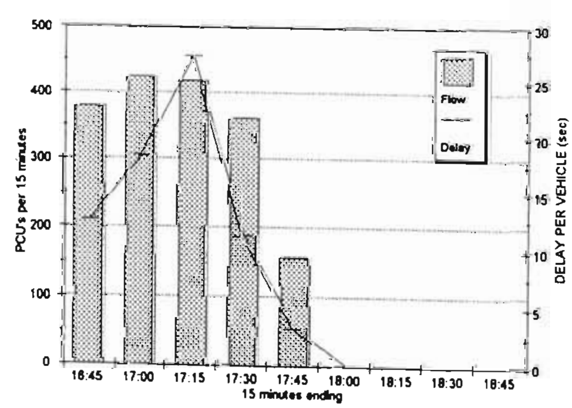


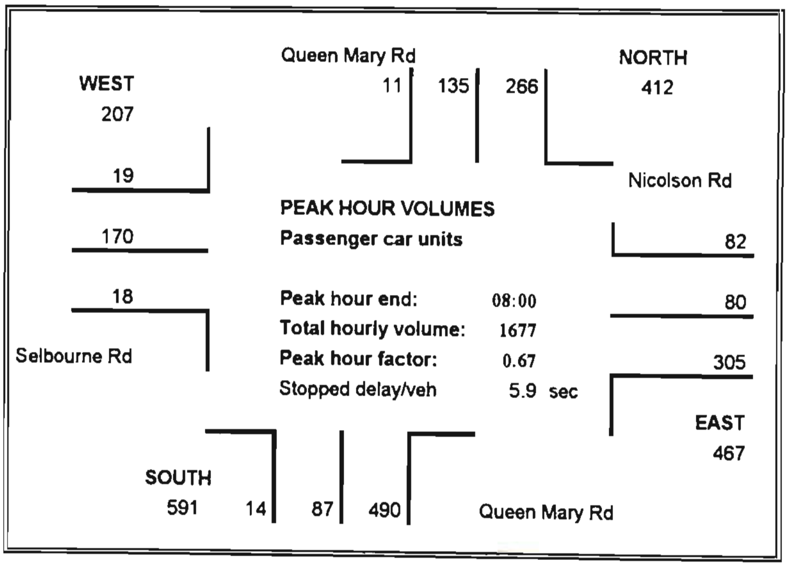
TABLE A9 : SUMMARY OF TRAFFIC SURVEY

QUEEN MARY CIRCLE - 25 /07/ 1996 AM

15 min's Ending	15 minute Volumes (PCU's)																
	North				East				South				West				Total
	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	
07:30	110	56	4	170	80	28	22	130	7	23	176	206	6	60	7	73	579
07:45	76	53	5	134	103	36	34	173	4	36	195	235	9	69	10	88	630
08:00	79	25	2	106	122	16	26	164	3	28	119	150	4	41	1	46	466
08:15	1	1	0	2	0	0	0	0	0	0	0	0	0	0	0	0	2
08:30																	
08:45																	
09:00																	
09:15																	
09:30																	

Hour Ending	Hourly Volumes (PCU's)																
	North				East				South				West				Total
	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	L	T	R	Tot	
08:00	266	135	11	412	305	80	82	467	14	87	490	591	19	170	18	207	1677
08:15																	
08:30																	
08:45																	
09:15																	
09:30																	

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	61%	67%	63%	59%	67%
MODAL SPLIT					
Cars	97%	95%	93%	96%	95%
Taxis	1%	3%	7%	4%	4%
Buses	1%	1%	0%	0%	1%
Trucks	1%	0%	0%	0%	1%
STOPPED DELAY PER APPROACH [Secs per vehicle]					
07:30	17.3	2.0	1.3	13.0	7.6
07:45	13.5	3.8	2.3	17.2	7.2
08:00	4.8	0.8	0.0	6.7	2.0
08:15					
08:30					
08:45					
09:00					
09:15					
09:30					
AVG NO OF STOPS PER 100 VEHICLES					
07:30					
07:45					
08:00					
08:15					
08:30					
08:45					
09:00					
09:15					
09:30					



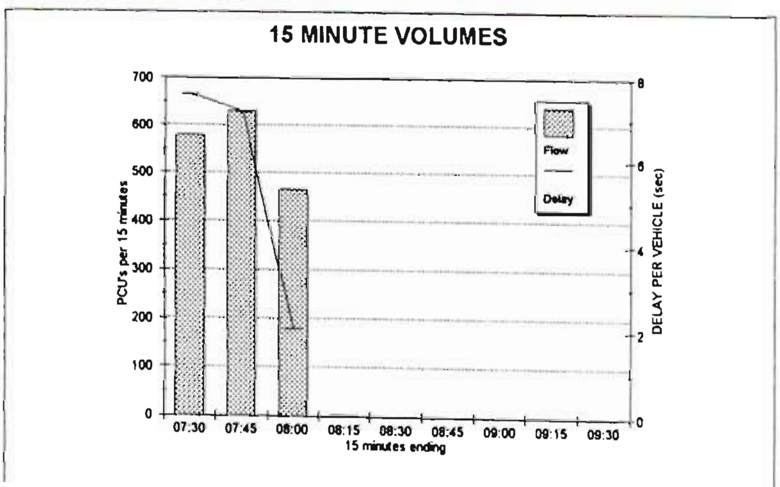


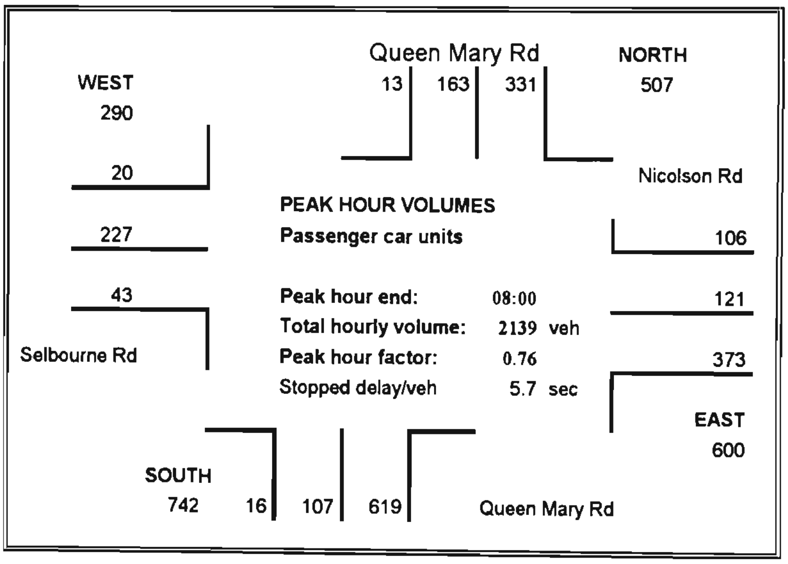
TABLE A10 : SUMMARY OF TRAFFIC SURVEY

QUEEN MARY CIRCLE - 26 /07/ 1996 AM

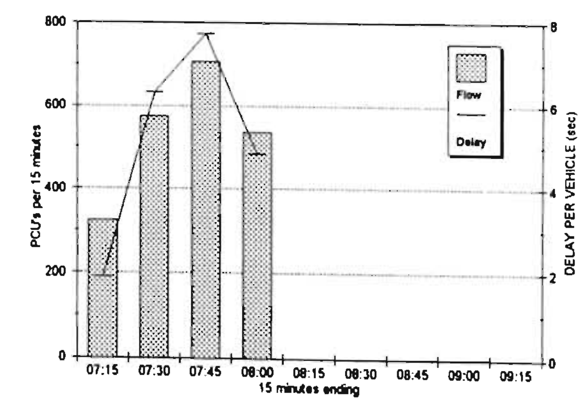
15 min's Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
	15 minute Volumes (PCU's)																
07:15	62	39	2	103	45	11	6	62	5	12	94	111	2	38	8	48	324
07:30	102	38	5	145	95	37	20	152	6	14	179	199	6	61	11	78	574
07:45	93	49	4	146	105	47	43	195	3	50	209	262	8	80	15	103	706
08:00	74	37	2	113	128	26	37	191	2	31	137	170	4	48	9	61	535
08:15																	
08:30																	
08:45																	
09:00																	
09:15																	

Hour Ending	North L	North T	North R	North Tot	East L	East T	East R	East Tot	South L	South T	South R	South Tot	West L	West T	West R	West Tot	Total
	Hourly Volumes (PCU's)																
08:00	331	163	13	507	373	121	106	600	16	107	619	742	20	227	43	290	2139
08:15	282	127	11	420	328	110	100	538	11	95	525	631	18	189	35	242	1831
08:30																	
08:45																	
09:00																	
09:15																	

	North	East	South	West	Avg
PEAK HOUR FACTOR					
	87%	77%	71%	70%	76%
MODAL SPLIT					
Cars	96%	94%	96%	94%	95%
Taxis	3%	5%	2%	6%	3%
Buses	1%	1%	1%	0%	1%
Trucks	0%	0%	1%	0%	0%
STOPPED DELAY PER APPROACH [Secs per vehicle]					
07:15	4.0	0.5	0.5	2.5	1.9
07:30	11.5	1.8	5.1	8.6	6.3
07:45	14.8	3.0	2.6	19.7	7.7
08:00	7.4	2.5	1.9	15.4	4.8
08:15					
08:30					
08:45					
09:00					
09:15					
AVG NO OF STOPS PER 100 VEHICLES					
07:15					
07:30					
07:45					
08:00					
08:15					
08:30					
08:45					
09:00					
09:15					



15 MINUTE VOLUMES



APPENDIX B: MATLAB CODE FOR TRACSIM

```
%-------------------------------------------- tracsim.m ---------------------------------------------
% MASTER PROGRAM FOR ENTERING INPUT AND CALLING OF SUB-ROUTINES
%------------------------------------------------------------------------------------------------------
clear all
format short
format compact
simt1 = clock;
DDD2=[0 0];                  % Matrix to record all DDD1's
B  = zeros (42,6);           % Matrix for different variables
for zzz = 1 : 10             % Loop for n different simulations to record zzz random 15 minute
grph   = 0;                  % Do you want to see simulation y-(1) n-(0)
grph2  = 0;                  % Do you want a graph at end to draw delay
nsim   = 26;                 % No of Flows to simulate
SIMTIME = 1020;              % Simulation time (Seconds)
RD     = 0;                  % Rnd number gen's re-seeded to gen same sequence of no's?  y:(1)   n:(0)
B(15,5) = 0;                 % Delay estimates
                             %  0 - Horizontal queues for validation with delays observed using queue lengths
                             %  1 - Vertical queues for stopped delay comparable with other models.
% ----------------- Generating vehicles on approaches----------------
B(1,5)   = 2;                        % Which model for generating veh's on approaches?
                                     %     Negative Expn distribution:          (0)
                                     %     Shifted Neg Expn distribution:       (1)
                                     %     Bunched Neg Expn distribution:       (2)
B(22,1:4) = [1.5 1.5 1.5 1.5];       % Minimum Hw on approaches 1 to 4
B(2,5)   = 0.6;                      % Bunching factor for Bunched Exp Distribution
B(22,5)  = 50/3.6;                   % Approach speeds (km/h)
B(23,5)  = 8;                        % Stopped spacing (metre)
%----------------- Move-in and Move-up times--------------------
B(18,1:4) = [2.7 2.7 2.7 2.7];       % Avg move-up times on approach
B(19,1:4) = [0.6 0.6 0.6 0.6];       % STD of move-up times
B(18:19,6)= [3.8; 0.5];              % Maximum and Minimum MuT's (sec).
B(25,1:4) = [11 11 11 12];           % Distance between yield line and circle
B(37,5)  = 0.45;                     % Prop of Vcirc used to calc move-in times.
%------------------- Vehicles in circle------------------------------
B(3,5)   = 2;                        % Constant speeds (1) or vehicle following (2)?
B(4,5)   = 37/3.6;                   % Avg speed in circle (km/h)
B(5,5)   = 6.5/3.6;                  % Std for speeds in circle (km/h)
B(4:5,6) = [3.5; -2.2];              % Max & Min speeds (# of STD's)
B(10,5)  = 1.0;                      % Minimum Hw for circulating vehicles

%----------------- Geometrics and Gap acceptance -------------------
B(24,5)  = 4;                % On what is gap acceptance based:
                                     %      1 - Critical gaps
                                     %      2 - Positions (Probabilities)
                                     %      3 - Area (original program)
                                     %      4 - Critical distance
B(6,5)   =36.2/2 + 3;                % Dia/2 + distance from curb
RPe      =[0 89 181 269];            % Angle of entry to circle from approach 1 or A
e2       =[-7.5 -7.5 -7.5 -8.5];     % Distance downstream - end of critical area (m)
e4       =[38 38 39 37];             % Distance upstream - exit to approach (m)
e3       =[35 40 40 38];             % AVG Distance upstream - start of critical area (m) Critical Gap
B(21,1:4) =[305 45 135 215];         % Angle of approach road
B(26,1:4) =[5 5 5 5];                % STD for e3, i.e. critical area (m)
if B(24,5) == 1        % ---------------------------------------------------------
  e5   = [3.87 3.87 3.87 3.87];      % Mean time (sec) for LN Dist for Lags  [N E S W] approaches
  e6   = [1.25 1.25 1.25 1.25];      % STD  for LN Dist for Lags                "
  B(6:7,6)= [7; 0.9];                % Max & Min Lags (sec)
  e7   = [4.92 4.92 4.92 4.92];      % Mean time (sec) for LN Dist for First Gaps   "
  e8   = [1.63 1.63 1.63 1.63];      % STD  for LN Dist for First Gaps              "
  B(8:9,6)= [10.1; 1.2];             % Max & Min first Gaps (sec)
  e9   = [4.57 4.57 4.57 4.57];      % Mean time (sec)for LN Dist for Gaps          "
  e10  = [0.92 0.92 0.92 0.92];      % STD  for LN Dist for Gaps                    "
  B(10:11,6) = [9.1; 1.2];           % Max & Min Gaps (sec)
  e11  = [4.51 4.51 4.51 4.51];      % Mean time (sec)for Normal Dist for Gaps      "
  e12  = [1.08 1.08 1.08 1.08];      % STD  for Normal Dist for Gaps                "
  B(20,5)  = 350;                    % Avg Circ volume at which above observations were made
  B(21,5)  = 0.0;                    % Gradient of Crit gap curve
elseif B(24,5) == 2    % ---------------------------------------------------------
  B(25:28,5)=[ 2; 24; 70; 94];       % P(Acceptance lags- %) - circulating veh in positions 0,1,2,3.
  B(29:33,5)=[100; 94; 56; 27; 0];   % P(Acceptance lags- %) - entering veh in positions 4,5,6,7,8.
  B(34,5)  = 90;                     % P(Acceptance lags- %) - circulating veh exiting position 9.
  B(25:28,6)=[ 0;  6; 50; 96];       % P(Acceptance gaps- %) - circulating veh in positions 0,1,2,3.
  B(29:33,6)=[80; 58; 24;  10; 0];   % P(Acceptance gaps- %) - entering veh in positions 4,5,6,7,8.
```

```
 B(34,6)  = 90;                           % P(Acceptance gaps- %) - circulating veh exiting position 9.
 B(39,1:4) =[284.6 262.2 239.8 191.3]; % Critical angles for entry 1
 B(42,1:4) =[191.3 171.9 152.4 104.5]; % Critical angles for entry 4
 B(41,1:4) =[104.5  82.3  60.1  17.8];  % Critical angles for entry 3
 B(40,1:4) =[ 17.8 352.9 328.8 284.6]; % Critical angles for entry 2
 B(35,5)  = 0.5;                          % Proportion of AA, part of area 6 - rest is 7.
 B(36,5)  = 1.5;                          % Time to yield line - veh considered in gap acceptance as position 5.
elseif B(24,5) == 4    % ----------------------------------------------------
 e5  = [36.0 36.0 36.0 36.0];            % Mean dist (m) for Normal Dist for Lags  [N E S W] approaches
 e6  = [3.6   3.6  3.6  3.6];            % STD  (m) for Normal Dist for Lags              "
 e7  = [40.5 40.5 40.5 40.5];            % Mean dist (m) for LN Dist for First Gaps     "
 e8  = [ 4.1  4.1  4.1  4.1];            % STD  (m) for LN Dist for First Gaps          "
 e9  = [40.5 40.5 40.5 40.5];            % Mean dist (m) for LN Dist for Gaps           "
 e10 = [ 4.1  4.1  4.1  4.1];            % STD  (m) for LN Dist for Gaps                "
 e11 = [40.5 40.5 40.5 40.5];            % Mean dist (m) for Normal Dist for Gaps       "
 e12 = [ 4.1  4.1  4.1  4.1];            % STD  for Normal Dist for Gaps                "
 B(12:13,6)  = [52; 17];                  % Max & Min lags (m)
 B(14:15,6)  = [54; 25];                  % Max & Min gaps (m)
 B(20,5)  = 350;                          % Avg Circ volume at which above observations were made
 B(21,5)  = 0.0;                          % Gradient of Crit gap curve
 B(29:30,5)=[100; 70];                    % if Crit Dist > Dist to yield line what is P(Acceptance lags- %) - entering
                                          veh in positions 4,5
 B(29:30,6)=[70; 50];                     % if Crit Dist > Dist to yield line what is P(Acceptance gaps- %) - entering
                                          veh in positions 4,5
end
%------------------- General ------------------------------------
B(7,5)   = 40;                            % number of vehicles in initial matrices. If n is
                                          % changed the results won't be exactly similar due to
                                          % random numbers generated in a different sequence.
B(8,5)   = 10;                            % number of columns needed for approach matrices.
B(9,5)   = 1.0;                           % The effect of exiting traffic on gap acceptance. 0.8 means that
                                          % 20% of exiting conflicting veh's will prevent the gap being accepted.
%-----------------------------------------------------------------
% START SIMULATION
%-----------------------------------------------------------------
load flowchat.dat;                        % Load flow data - File must contain "nsim" rows
FL   = flowchat;                          % Store flow data in FL
DDD1 = [0 0];                             % Matrix for plotting delay
% -------------- Run simulation n times for all flows ---------------
for ii=1:nsim                             % number of flow simulation runs to make
 xyz = xyz + 1                            % To display on screen the number of the run
 adaptinp                                 % Run subroutine to update input and initialize matrices
 aaatrac                                  % Run simulation for one set of flows
 DDD1 = [DDD1; Q' DD1(:,2)];              % Add delay data for this run to previous runs
 B(23:24,1:4)= zeros(2,4);                % Reset arrival time and speed of last vehicle for new sim run.
 B(18:19,5)  = zeros(2,1);                % Reset # veh's in AA and C.
end  % ------------- end of loop ---------------------------------
simt2=clock;
simt2-simt1              % Display total simulation time
%---------------- DRAW FIGURE "AVG DELAY PER APPROACH" --------------
%figure(2)
%plot(DDD1(:,1),DDD1(:,2),'g*');
%axis ([0 1300 0 100]);
%xlabel('approach volume (veh/h)')
%ylabel('avg delay (sec/veh)')
DDD2=[DDD2; DDD1]           % Add different DDD1's together for summary
save chatdis2.txt DDD2 -ascii  % Save results in an ascii file
end % ----------------- END ---------------------------------
```

```
function [A,B]=inimat(i,j,B,T)
%-----------------------------INIMAT.M-----------------------------
% Subroutine to generate vehicles on approaches arriving in platoons
% --------------- Generate Headway --------------------------------
A = zeros(j,B(8,5));
[dT1,B] = randbe(i,j,B);          % Generate j headways - Bunched Negative exp distribution
% --------------- Generate Turning Movement -----------------------
T1 = cumsum(dT1);                  % Ensure headways are cumulative
A(:,1) = T1 + T;                  % Add to greatest headway previously generated.
A(:,3) = A(:,1);
[T2,B] = turn(j,B,i);             % Turning movement (1,2,3 = L,T,R)
A(:,2) = (T2 + i) - ((T2+i)>4)*4;    % Exit/approach number
%-----------------------------------------------------------------
if B(24,5) == 1                             % Only do if using critical gaps for gap acceptance
  % --------------- Generate Critical Lag ------------------------
  %[z,B] = randn5(j,B);               % Normal rand deviate - for critical lag and gap
  while  any(A(:,6) > B(6,6) | A(:,6) < B(7,6)) > 0            % Check that 1.0 < lag > 7 seconds
   x  = find(A(:,6) > B(6,6) | A(:,6) < B(7,6));
   z = randn(size(x,1),1);
   A(x,6) = exp(ones(size(x,1),1).*B(28,i)+z.*B(29,i)); % Generate critical lags from AVG and STD.
  end
  % --------------- Generate Critical Gap ------------------------
  %[z,B] = randn5(j,B);               % Normal rand deviate - for critical lag and gap
  while  any(A(:,8) > B(10,6) | A(:,8) < B(11,6) | A(:,8) < A(:,6)) > 0   % Check that 6.1 < gap > 1.2 sec and gap >
     lag
   x =  find(A(:,8) > B(10,6) | A(:,8) < B(11,6) | A(:,8) < A(:,6));
   z = randn(size(x,1),1);
   A(x,8) = exp(ones(size(x,1),1).*B(32,i)+z.*B(33,i)); % Generate critical gaps from AVG and STD.
  end
  % --------------- Generate Critical first Gap ------------------
  %[z,B] = randn5(j,B);               % Normal rand deviate - for critical lag and gap
  while  any(A(:,7) > B(8,6) | A(:,7) < B(9,6) | A(:,7) < A(:,8)) > 0        % Check that 6.1 < 1st gap > 1.2 seconds
     and that 1st Gap > gap
   x =  find(A(:,7) > B(8,6) | A(:,7) < B(9,6) | A(:,7) < A(:,8));
   z = randn(size(x,1),1);
   A(x,7) = exp(ones(size(x,1),1).*B(30,i)+z.*B(31,i)); % Generate critical 1st gaps from AVG and STD.
  end
  %---------------------------------------------------------------
 elseif B(24,5) == 3                         % Do if critical area is used for gap acceptance
  % --------------- Generate Critical distance -------------------
  %[z,B] = randn6(j,B);
  z = randn(j,1);                    % Normal rand deviate - for crit gap generation
  while any(z > 2 | z < -2) > 0         % Check that deviation from avg is not > abs|2*STD|
   z = randn(j,1);                   % If greater than, then generate another set of rand no's
  end
  A(:,5) = ones(j,1).*B(3,i)+z.*B(26,i);  % Generate critical position in circle from AVG and STD.
  A(:,5) = A(:,5)+(A(:,5)<0).*2*pi*B(6,5);% Remove negative values (A<0) contains 1 where cond is true
  %---------------------------------------------------------------
 elseif B(24,5) == 4                         % Only do if using critical distances for gap acceptance
  % --------------- Generate Critical Lag ------------------------
  %[z,B] = randn5(j,B);                 % Normal rand deviate - for critical lag and gap
  while any(A(:,6) > B(12,6) | A(:,6) < B(13,6)) > 0                % Check that 1.0 < lag > 7 seconds
   x = find(A(:,6) > B(12,6) | A(:,6) < B(13,6));
   z = randn(size(x,1),1);
   A(x,6) = ones(size(x,1),1).*B(28,i)+z.*B(29,i);      % Generate critical lags from AVG and STD.
  end
  % --------------- Generate Critical Gap ------------------------
  %[z,B] = randn5(j,B);                 % Normal rand deviate - for critical lag and gap
  while any(A(:,8) > B(14,6) | A(:,8) < B(15,6) | A(:,8) < A(:,6)) > 0   % Check that 6.1 < gap > 1.2 sec and gap >
     lag
   x = find(A(:,8) > B(14,6) | A(:,8) < B(15,6) | A(:,8) < A(:,6));
   z = randn(size(x,1),1);
   A(x,8) = ones(size(x,1),1).*B(32,i)+z.*B(33,i);      % Generate critical gaps from AVG and STD.
  end
  % --------------- Set Critical First Gap = Critical Gap --------
  A(:,7) = A(:,8);
 end
end
```

```
%-------------------------------aaa3.m------------------------------------------
% SUBROUTINE TO SIMULATE TRAFFIC ON CIRCLE FOR ONE SET OF TRAFFIC FLOWS
%-----------------------------------------------------------------------------
tic                         % Start of event scan - start clock
while B(12,5) < SIMTIME             % Stop condition for system time
B(11,5) = B(12,5);              % To remember previous system time.
selectn2                      % Find next event to occur in time (min Ts)
if M > 1                    % If there are more than one event with similar event times (Ts).
  priority                  % Rank events according to priority
end
for N = 1:M;                  % Repeat M times for all M events with similar Ts
 if N > 1
  B(13,5) = 0;                % Set dT = 0 for further updates of vehicles around circle
 end
 I = J((N),1);               % Select next event from J containing list of all events with similar Ts.
%-----------------------------------------------------------------------------
if I <= 4                   % Event(1): Arrival at yield line
 if B(18,5) > 0               % Are there any vehicles in circle
  C = upcirc(C,B);               % If yes then update vehicles in circle
 end
 if B(19,5) > 0               % Are there any vehicles in transit
 AA(:,10) = AA(:,10) - B(13,5);          % Update veh's in transit by subtracting dT from all event times.
 end
 G = checkgap(B,I,C,AA,A);              % Check if gap is acceptable
 if G == 0                  % If available gap is acceptable then
  [AA,B,C]  = transit(A,AA,B,C,I);          % Move vehicle from yield line on way to circle
  [A,D,B]   = upapp(A,D,B,I);            % Remove from approach
 else
  A(1,4+B(8,5)*(I-1)) = 1;               % 1 indicates waiting in queue at yield line
  A(1,3+B(8,5)*(I-1)) = A(1,3+B(8,5)*(I-1)) + 80;    % Increase time of arrival at yield line to prevent event(1) again
  A(1,9+B(8,5)*(I-1)) = 1;               % 1 indicates veh waiting for first gap
 end
%-----------------------------------------------------------------------------
elseif I==5                  % Event(3): exit circle: I=5
 [k,l] = find (C(:,6)==B(12,5));          % Locate vehicle in circle
 k = k(1,1);                 % Select one if there are two occuring at same time
 i=C(k,2);                  % Identify the approach
 C(k,:)=[];                 % Delete the exiting vehicle
 B(18,5) = B(18,5) -1 ;
 if B(18,5) > 0               % Are there any vehicles in circle
  C = upcircex(C,B);             % Update vehicles in circle
 end
 if B(19,5) > 0               % Are there any vehicles in transit
 AA(:,10) = AA(:,10) - B(13,5)*(AA(:,2) > 0);  % Update veh's in transit by subtracting dT from all event times.
 end
 if A(1,4+B(8,5)*(i-1)) == 1          % Is veh waiting at yield line on next approach i.e app of exit
  G = checkgap(B,i,C,AA,A);            % Check if gap is acceptable
  if G == 0                 % If available gap is acceptable then
   [AA,B,C]  = transit(A,AA,B,C,i);        % Move vehicle from yield line on way to circle
   [A,D,B]   = upapp(A,D,B,i);          % Remove from approach
  else
   A(1,9+B(8,5)*(i-1)) == 2;          % 2 indicates veh waiting for subsequent gaps having rejected a lag and 1st
    gap
  end
 end
 j=(i-2)+(((i-2)<0)*4);             % j relates to the position in [A]
 if A(1,4+B(8,5)*j) == 1              % Is veh waiting at yield line on approach being passed
  i = (i-1)+(i==1)*4;              % Change i to relate to app upstream of exit
  G = checkgap(B,i,C,AA,A);            % Check if gap is acceptable
  if G == 0                 % If available gap is acceptable then
   [AA,B,C] = transit(A,AA,B,C,i);         % Move vehicle from yield line on way to circle
   [A,D,B] = upapp(A,D,B,i);           % Remove from approach
  else
   A(1,9+B(8,5)*(i-1)) == 2;          % 2 indicates veh waiting for subsequent gaps having rejected a lag and 1st
    gap
  end
 end
%-----------------------------------------------------------------------------
```

(Continued on next page)

```
elseif l >= 6 & l <= 9                % Event(2): Circulating veh passing point (end of critical area)
 if B(18,5) > 0                      % Are there any vehicles in circle
  C = upcirc(C,B);                     % If yes then update vehicles in circle
 end
 if B(19,5) > 0                      % Are there any vehicles in transit
 AA(:,10) = AA(:,10) - B(13,5)*(AA(:,2) > 0);  % Update veh's in transit by subtracting dT from all event times.
 end
 i=l-5;                            % i identifies relevant approach
 [k,l] = find(C(:,l+1)==B(12,5));          % Locate vehicle in circle
 B(23,i) = B(12,5);                    % Store time last vehicle passed critical point in B to prevent veh's entering to
    fast
 B(24,i) = C(k(1,1),11);                 % Store speed of last vehicle past critical point in B
 if A(1,4+B(8,5)*(i-1)) == 1;              % If veh is waiting at yield line on relevant approach
  G = checkgap(B,i,C,AA,A);                 % Check if gap is acceptable
  if G == 0                        % If available gap is acceptable then
   [AA,B,C] = transit(A,AA,B,C,i);          % Move vehicle from yield line on way to circle
   [A,D,B]   = upapp(A,D,B,i);            % Remove from approach
  else
   A(1,9+B(8,5)*(i-1)) == 2;              % 2 indicates veh waiting for subsequent gaps having rejected a lag and 1st
    gap
  end
 end
%---------------------------------------------------------------------------------
elseif l >= 10 & l <= 13               % Event(4) vehicle arrive in circulating lane
 if B(18,5) > 0                      % Are there any vehicles in circle
  C = upcirc(C,B);                     % If yes then update vehicles in circle
 end
 if B(19,5) > 0                      % Are there any vehicles in transit
 AA(:,10) = AA(:,10) - B(13,5)*(AA(:,2) > 0);  % Update veh's in transit by subtracting dT from all event times.
 end
 i =l-9;                           % i identifies from which approach
 in=((i+1)-4*(i==4));                          % in Number of next entry
 [k,l] = find(AA(:,4)==B(12,5) & (AA(:,5)==i)); % Identifies vehicle arriving in circle
 [C,B] = addcirc2(AA,C,B,i,k);             % Move veh onto circle
 b = AA(k,2);                                  % Remember exit number
 AA(k,:)=[];                        % Delete the exiting vehicle from AA
 if b == in & A(1,B(8,5)*(in-1)+4) == 4         % Is this a left turning vehicle and is there a vehicle waiting on the
    next approach
  G = checkgap(B,in,C,AA,A)               % Check if gap is acceptable
  if G == 0                        % If available gap is acceptable then
   [AA,B,C] = transit(A,AA,B,C,in);         % Move vehicle from yield line on way to circle
   [A,D,B]   = upapp(A,D,B,in);           % Remove from approach
  else
   A(1,9+B(8,5)*(i-1)) == 2;              % 2 indicates veh waiting for subsequent gaps having rejected a lag and 1st
    gap
  end
 end
 B(19,5) = B(19,5) - 1;
%---------------------------------------------------------------------------------
else        %(Events 14,15,16)   Veh catching up with slow veh in circle (Event 14 and 15) or
            accelerating behind leading veh (Event 16)
 C = upcirc(C,B);                     % Update vehicles in circle
 if B(19,5) > 0                      % Are there any vehicles in transit
 AA(:,10) = AA(:,10) - B(13,5)*(AA(:,2) > 0);  % Update veh's in transit by subtracting dT from all event times.
 end
 C = upcircle(C,B,AA,l);                 % Adjust veh speed and event times
%---------------------------------------------------------------------------------
 end                               % end of if statement to select l.
 end                               % end of for N=1:M loop for similar Ts times.
%---------------------------------------------------------------------------------
 if grph==1
  graphc2               % Sub-routine to draw veh's around circle
 end
end                     % End of while statement-------------------
delay                   % Sub-routine to calc avg cum delay for simulated
                      % flow and turning movements
if grph2==1
 delaygrp               % Sub-routine to graph delays
end
t=toc
%---------------------------- end of aaatrac.m ------------------------------------------
```

```
%---------------------------selectn2.m----------------------------------------
% Subroutine selectn2.m to search for next event
%----------------------------------------------------------------------------
% --- AA1 extracts events from AA into 4 columns for ID of approach ------
% --- A1 contains info from A but with increased Ts > Ts.
%--- M1 contains the min times for the 16 possible events ----------------------------
%--- (1-4:  Arrival on app    5:  Exit      6-9: Passing a point) --------------------
%--- (10-13: Arrive in circle   14: Catch-up    15:  Conflict with entering vehicle) ------------------
%--- (16: Following leading veh when accelerating) ----------------------------------
M1 = ones(B(7,5),16).*B(12,5)+2000;    % Initialize matrix M1 for selection of minimum Ts
A1  = A+((A<=B(12,5))*(B(12,5)+2000));  % Remove zeros in A by increasing them
M1(1,1:4)     = [(A1(1,3)) (A1(1,3+B(8,5))) (A1(1,3+2*B(8,5))) (A1(1,3+3*B(8,5)))];
if B(19,5) > 0                          % Do same for AA if AA is not empty
 AA1 = [AA(:,4).*(AA(:,5)==1) AA(:,4).*(AA(:,5)==2) AA(:,4).*(AA(:,5)==3) AA(:,4).*(AA(:,5)==4)];
 AA1 = AA1+((AA1<B(12,5))*(B(12,5)+2000));
 M1(1:B(19,5),10:13) = [(AA1(:,1)) (AA1(:,2)) (AA1(:,3)) (AA1(:,4))];  % Extract important events
end
if B(18,5) > 0                          % Do same for C if C is not empty
 C1  = C+((C<=B(12,5))*(B(12,5)+2000));
 M1(1:B(18,5),5:9)  = [(C1(:,6))  (C1(:,7))  (C1(:,8))  (C1(:,9)) (C1(:,10))];
 M1(1:B(18,5),14:16) = [(C1(:,13)) (C1(:,14)) (C1(:,16))];              % Extract important events
end
B(12,5) = min(min(M1));                 % Ts minimum event time selected from all events
J=[];
[J(:,2),J(:,1)]  = find(M1==B(12,5));   % Find all events with same minimum Ts
                                        % J(:,1) returns the column no =  event no
                                        % J(:,2) returns the row no = row in data matrix (C,A or AA)
M  = size(J,1);                  % Count number of events with same Ts
B(12,5)=B(12,5);
B(13,5) = B(12,5)-B(11,5);       % Time increment from previous to new system time
%----------------------------------------------------------------------------
```

```
%------------------------------- priority.m ------------------------------------
% Subroutine priority.m to rank events according to importance in time
%----------------------------------------------------------------------------
% PRIOR matrix is identified in adaptinp.m

J(:,3) = PRIOR(2,(J(:,1)'))';           % Set priorities of events in column 3
[Y1,X1] = sort(J(:,3));                 % Sort column 3 remembering indices.
J = [J(X1,1) J(X1,2)];                  % Sort rest of matrix according to column 3.

%------------------------------------------------------------------------
```

```
function G=checkgap(B,I,C,AA,A)
%------------------------------------ checkgap.m ------------------------------------
x = A(1,(B(8,5)*(I-1))+9);                    % Identify whether veh is waiting for lag, first gap of sub gaps.
%-----------------------------------------------------------------------------------
if B(24,5) == 1                                          % Gap acceptance based on critical gaps
 % -------------- Check the critical gap against available circ gaps --------------------
 if B(18,5) == 0                                  % Are there any vehicles in the circle
  G = 0;                                          % If not then gap is acceptable
 else
  C(:,7:10) = C(:,7:10)+100*(C(:,7:10)<=B(12,5));    % Increase avail gaps = to & < Ts
  [Y,Z]=min(C(:,6+I));                          % Find the minimum gap and its row number in C.
  G = (A(1,B(8,5)*(I-1)+(6+x)))>min(C(:,6+I)-B(12,5)); % Look for smallest time to critical point on approach.
  if G == 1 & C(Z,2) == I                         % If there is a vehicle conflicting and if this is an exiting
    vehicle.
   G = G * rand > B(9,5);                         % When rand > B then gap will be rejected, i.e. G = 0.
  end
 end
 % -------------- Check the critical gap against available entering gaps -----------------
 if G == 0 & B(19,5) > 0                          % If gap is acceptable and there are veh's in transit
  i = (I-1) + 4*(I==1);                         % Identify previous entering approach
  k = min((AA(:,10)+100.*(AA(:,5)~=i))-B(12,5));  % Find the min gap - increase all which are not from previous
    approach
  G = (A(1,B(8,5)*(I-1)+(6+x))) > k ;             % Check critical gap/lag agains minimum available,
  z = find(AA(:,10)==(k+B(12,5)));                % Identify the vehicle which could be conflicting
  if G == 1 & AA(z,2) == I                        % If there is a vehicle conflicting and if this is an exiting
    vehicle.
   G = 0;                                         % If this is a left turning vehicle then gap is acceptable.
  end
 end
elseif B(24,5) == 2                               % Gap acceptance based on positions
 if B(18,5) == 0                                  % Are there any vehicles in the circle
  G = 0;                                          % If not then gap is acceptable
 else
  x = x - 1*(x == 2);                             % Change x to 1 if = 2.
  D = 2*pi*B(6,5);                                % Calc Circumference
  i = (I-1)+4*(I==1);                             % Number of previous approach
  C(:,1) = (C(:,1)-B(1,I));                       % Change Positions relative to approach I.
  C(:,1) = C(:,1)+D.*(C(:,1)<0);                  % Subtract Circumf where > Circumf.
  B(39:42,1:4)=B(39:42,1:4)-B(1,I);               % Change Critical posit relative to approach I.
  B(39:42,1:4)=B(39:42,1:4)+D.*(B(39:42,1:4)<0);  % Subtract Circumf where > Circumf.
  C(:,7:10) = C(:,7:10)+1000*(C(:,7:10)<=B(12,5));  % Increase exist gaps = & < Ts with 1000
  % -------------- Check available circ gaps -----------------------
  [Y,Z]=min(C(:,6+I));        % Find the minimum gap and its row number in C.
  if Y < 1000                       % Is there a min gap which was not increased with 1000
   if C(Z,1) >= B(38+I,1)           % Is 1st conflicting veh closer than crit angle 1?
    r=(C(Z,1)-D)/(B(38+I,1)-D);  % Ratio of pos of veh to distance in area.
   if C(Z,17) == 1                  %
    G = 1;                          % If the conflicting veh is following then reject and G=1
   elseif C(Z,5) == i           % Did this vehicle come from previous entrance?
    G = (rand >= B(33,5+x)*r);      % If rand > P(Acceptance 0 - ent gap) then reject and G = 1
   else
    G = (rand >= B(25,5+x)*r);      % If rand > P(Acceptance 8 - circ gap) then reject and G = 1
   end
  elseif C(Z,1) >= B(38+I,2)        % Is 1st conflicting veh closer than crit angle 2?
   if C(Z,5) == i                   % Did this vehicle come from previous entrance?
    G = (rand >= B(32,5+x));        % If rand > P(Acceptance 1 - ent gap) then reject and G = 1
   else
    G = (rand >= B(26,5+x));        % If rand > P(Acceptance 7 - circ gap) then reject and G = 1
   end
  elseif C(Z,1) >= B(38+I,3)        % Is 1st conflicting veh closer than crit angle 3?
   G = (rand >= B(27,5+x));         % If rand > P(Acceptance 2 - circ gap) then reject and G = 1
  elseif C(Z,1) >= B(38+I,4)        % Is 1st conflicting veh closer than crit angle 4?
   G = (rand >= B(28,5+x));         % If rand > P(Acceptance 3 - circ gap) then reject and G = 1
   if G == 1 & C(Z,2) == i          % If gap is rejected and conflicting veh is exiting
    G = (rand >= B(34,5+x));        % change to acceptance if rand > P(Acceptance)
   end
  else
   G = 0;                           % Vehicle upstream of last crit angle and gap accepted G = 0
  end
 else
  G = 0;                            % If there is no min gap < 1000 then gap is acceptable
 end
end
```

(checkgap.m continued on next page)

```
 % ---------If circ gap available check for conflict's with ent traffic ---------------------------
 if G == 0 & B(19,5) > 0                    % If no circ veh's, but entering vehicle from previous approach
  if any(AA(:,5)==i) > 0
   [u,o]=find(AA(:,5)==i);
   [k,z] = min(AA(u,4)-B(12,5));           % Find the min gap - increase all which are not from previous approach
   D = k*AA(u(z,1),6)*B(37,5);  % Distance left to travel to enter circle
   if D < (1-B(35,5))*B(25,i)              % Check if vehicle is in Section 7 of critical positions.
    G = (rand >= B(32,5+x));               % If rand > P(Acceptance 7 - ent gap) then reject and G = 1
   else
    G = (rand >= B(31,5+x));               % If rand > P(Acceptance 6 - ent gap) then reject and G = 1
   end
  end
 end
 if G==0 & A(1,B(8,5)*(l-1)+4)==1          % If gap still acceptable and vehicle stationary on i - the previous approach
  G = (rand >= B(29,5+x));                 % If rand > P(Acceptance 4 - ent gap) then reject and G = 1
 end
 if G==0 & A(1,B(8,5)*(l-1)+3)<=(B(36,5)+B(12,5))         % Gap still accept. and veh moving at previous yield line
    - posit. 5.
  G = (rand >= B(30,5+x));                 % If rand > P(Acceptance 5 - ent gap) then reject and G = 1
 end

 %-----------------------------------------------------------------------------------
elseif B(24,5) == 3                        % Gap acceptance based on critical area.
 %-----------------------------------------------------------------------------------

 rpe=B(1:4,1:4);
 m = B(8,5);
 rpe(3,:) = [A(1,5) A(1,m+5) A(1,2*m+5) A(1,3*m+5)];      % Get start of critical area from A
 if (rpe(3,l) > rpe(2,l))                             % Check if area includes reference point.
  z = (C(:,1) < (rpe(2,l)-.002))|(C(:,1) > rpe(3,l))&(C(:,2)>0);          % < end or  > start of area
 else
  z = (C(:,1) < (rpe(2,l)-.002))&(C(:,1) > rpe(3,l));   % < end and > start of area
 end
 G = any(z);                               % G=1 if any nonzeros and = 0 when all zeros
                                           % if there are any nonzeros then gap is not acceptable
                                           % and G=1
 % --------------- Check vehicles in 'transit' from previous app -------------------------
 %if G == 0                                % Check only if there are no vehicles in critical area
 % i =  (l-1)+(4*(l==1));                  % Find number of previous approach
 % G = any (AA(:,5)==i);                   % Any veh's in transit from previous approach
 %end

 %-----------------------------------------------------------------------------------
elseif B(24,5) == 4                               % Gap acceptance based on critical distances
 %-----------------------------------------------------------------------------------

 % ---------------- Check gap against available circulating gaps -------------------------
 if B(18,5) == 0                                  % Are there any vehicles in the circle
   G = 0;                                         % If not then gap is acceptable
 else
  i = (l-1)+4*(l==1);                             % Number of previous approach
  C(:,1)  = (C(:,1)-B(1,l));                      % Change Positions relative to approach l.
  C(:,1)  = C(:,1)+B(14,5).*(C(:,1)<0);           % Subtract Circumf where > Circumf.
  z = find(C(:,6+l)<= B(12,5));                   % Find vehicles past and at end of critical area
  C(z,:) = [];                                    % Remove these vehicle from C
  if size(C) > 0
   [Y,Z]=min(B(14,5)-C(:,1));                     % Find the minimum gap and its row number in C.
   G = (A(1,B(8,5)*(l-1)+(6+x)) > Y);             % Check if crit dist > min dist available if yes then Gap unaccpt
    and G = 1
   if G == 1 & C(Z,2) == l                        % If there is a vehicle conflicting and if this is an exiting vehicle.
    G = G * rand > B(9,5);                        % When rand > B then gap will be rejected, i.e. G = 0.
   end
  else
   G = 0;                                         % Gap is acceptable.
  end
 end
```

(checkgap.m continued on next page)

```
% ------------- Check the critical gap against transit vehicles -------------------------
 if G == 0 & B(19,5) > 0                          % If gap is acceptable and there are veh's in transit
  i = (l-1) + 4*(l==1);                    % Identify previous entering approach
  if any(AA(:,5)==i) > 0
   [u,o]=find(AA(:,5)==i);
   [k,z] = min(AA(u,4)-B(12,5));                  % Find the min gap
   D = k*AA(u(z,1),6)*B(37,5);                    % Distance left to travel to enter circle
   D1= (B(2,l)-B(1,i))+B(14,5)*(B(2,l)<B(1,i));% Distance from entry to end of crit area - add circ. if negative.
   D = D + D1;                                           % Total distance to end of crit area.
   G = (A(1,B(8,5)*(l-1)+(6+x)) > D);            % Check dist against crit dist. G=1 if Dist < Crit Dist.
   if G == 1 & AA(u(z,1),2) == l;                 % If the conflict is caused by a left turning vehicle
    G = 0;                                        % If this is a left turning vehicle then gap is acceptable.
   end
  end
 end

 % ------------ Check the gap against vehicles moving/stationary on previous approach -----------
 D1= (B(2,l)-B(1,i))+B(14,5)*(B(2,l)<B(1,i));     % Distance from entry to end of crit area - add circ. if negative.
 D = D1 + B(25,l);                                % Total distance to previous yield line
 if G==0 & A(1,B(8,5)*(l-1)+5+x) > D              % If gap is still acceptable and crit dist > total dist to yield line
  Td = (A(1,B(8,5)*(l-1)+3)-B(12,5))*B(22,5);     % Distance approaching vehicle is from yield line.
  if A(1,B(8,5)*(l-1)+4) == 1                     % Is there a vehicle waiting on the approach?
   x = x - 1*(x == 2);                            % Change x to 1 if = 2. Lag (0) or Gap (1)
   G = (rand >= B(29,5+x));                       % If rand > P(Acceptance 4 - ent gap) then reject and G = 1
  elseif A(1,B(8,5)*(l-1)+5+x) > D + Td    % If crit dist > total dist + dist left for vehicle to reach yield line
   x = x - 1*(x == 2);                            % Change x to 1 if = 2. Lag (0) or Gap (1)
   G = (rand >= B(30,5+x));                       % If rand > P(Acceptance 5 - ent gap) then reject and G = 1
  end
 end

 %-----------------------------------------------------------------------------------
end                                      % end - circle empty or not?
%-----------------------------------------------------------------------------------
```

```
function [A,D,B]=upapp(A,D,B,i)
%------------------------------- upapp.m------------------------------------
% Subroutine to update approach
%---------------------------------------------------------------------------
Mut = 0;
while any(Mut > B(18,6) | Mut < B(19,6)) > 0   % z has to be < 3.8 and > -2.3 : STD's (as observed)
z  = randn;                     % If outside range, then generate another set of rand no's
Mut = B(18,i) + z*B(19,i);              % Generate a move-up time from Avg and STD for next vehicle
end

if B(13,i) > 1/B(18,i)
 nn = 'Warning: Flow is greater than move-up times'
 nn = 'Errors will occur in program!'
end

AS=A(:,(1+B(8,5)*(i-1)):(B(8,5)+B(8,5)*(i-1))); % submatrix from A for approach i
if B(12,5) > 120                   % Warm-up period: Only start measuring delay when elapsed.
 D=[D; i (B(12,5)-AS(1,1))];             % Approach delay
end

% ------- Mark vehicles in queues and move stopping time back --------------------------
if B(15,5) == 0                                   % Do if horizontal queues are simulated to compare with
    observed queeing delay.
 z1 = find(AS(:,10) == 1);            % Check for veh's whose stopping time have been adjusted.
 z2 = find(AS(:,1)<B(12,5) & AS(:,10)~=1);     % Check for vehicles waiting in queue whose stopping time have not
    been adjusted.
 if z2~=[];
  AS(z2,1) = AS(z2,1)-(B(23,5)/B(22,5).*(z2-1)); % Move back stopping time
  AS(z2,10)= ones(size(z2));            % Mark vehicles as been adjusted
 end
end

AS(1:(B(7,5)-1),:)=AS(2:B(7,5),:);          % --------- Remove leaving vehicle from AS

z2 = sum(AS(:,1) < B(12,5));             % Number of vehicles in queue - excluding entering vehicle

if size(z2,1)==(B(7,5)-1)
 nn = 'Warning: Queue exceeding matrix size - Increase # rows in A'
 nn = 'Output will be inaccurate - especially q-length estimates'
end

% ------- If there is a queue store the stats -------------------------------------
if z2 > 0
 B(36,i) = B(36,i) + z2;          % Store total number of queuing vehicles
 B(37,i) = B(37,i) + 1;           % Store the number of queues for average
 z1 = [B(38,i) z2];
 B(38,i) = max(z1);               % Store the maximum queue length to date
end

% ------ Generate new n-th vehicle ----------------------------------------------
[AS(B(7,5),:),B] = inimat(i,1,B,max(AS(:,1)));

% ------- Move up vehicles in queue -----------------------------------------------
z = find(AS(:,1) <= B(12,5));              % check for vehicles waiting in queue
if z~=[];
 AS(z,3) = (B(12,5)+Mut)*cumsum(ones(size(z)));      % move only those waiting in queue and only
end                              % arrival of first one is important

% ---------- Complete A again by returning submatrix AS to A -------------------
A(:,(1+B(8,5)*(i-1)):(B(8,5)+B(8,5)*(i-1))) = AS;
%---------------------------------------------------------------------------
```

```
function C=upcirc(C,B)
%-------------------------- upcirc.m ---------------------------------------------
% Update of vehicles in circle
%----------------------------------------------------------------------------------

% ---------Update positions of all vehicles-------------------------
C(:,1)=C(:,1)+C(:,11)*B(13,5);        % Update positions on circle at Ts using individual speeds
z = C(:,1) > 2*pi*B(6,5);             % Check for positions > Circumference
C(:,1) = C(:,1) - z.*2*pi*B(6,5);     % Change positions > Circumference
[Y,X] = sort(C(:,1));                 % Sort positions around circle in ascending order for
C = [Y C(X,2:17)];                    % Sort rest of matrix accord. to first column
% ------------- T0 highlight all Hw < Hwmcirc : AS A CHECK FOR POSSIBLE ERRORS --------------
y = B(18,5);                          % Count number of non-zero entries in C
if y > 1
 C1(:,1) = C(:,1);                    % Col 1: Positions around circle in ascending order
 C1(1:(y-1),2) = C1(2:y,1);           % Col 2: Positions of veh's immedtly ahead
 C1(y,2) = 2*22/7*B(6,5)+C1(1,1);     %     ditto
 C1(:,3) = (C1(:,2)-C1(:,1));         % Col 3: Distances between vehicles
 C1(:,4) = C(:,11);                   % Col 4: Speeds of vehicles
 C1(1:(y-1),5) = C1(2:y,4);           % Col 5: Speeds of veh's immediatly ahead
 C1(y,5) = C1(1,4);                   %     ditto
 H = C1(:,3)./C1(:,5);                % Headways between vehicles
X = H < B(10,5)-.2;                   % Print all headways < Hwm to screen
 if any(X)==1
  H
  B(12,5)
 end
end
%------------------- end of upcirc.m ------------------------------------------


function C=upcircex(C,B)
%------------------------------ upcircex.m --------------------------
% Update of vehicles in circle when vehicle exits circle
%------------------------------------------------------------
% ---------Update positions of all vehicles----------------------
C(:,1)=C(:,1)+C(:,11)*B(13,5);  % Update positions on circle at Ts using individual speeds
z = C(:,1) > 2*pi*B(6,5);       % Check for positions > Circumference
C(:,1) = C(:,1) - z.*2*pi*B(6,5); % Change positions > Circumference
if B(18,5) > 1                  % Skip headway check if only one vehicle in circle
 [Y,X] = sort(C(:,1));          % Sort positions around circle in ascending order for
 C = [Y C(X,2:17)];             % Sort rest of matrix accord. to first column

% -------- Find new event times for remaining circ vehicles ------------------------------
 y = B(18,5);
 C1 = zeros(y,8);                      % Define C1
 C1(:,1) = C(:,1);                     % Col 1: Positions around circle in ascending order
 C1(1:(y-1),2) = C1(2:y,1);            % Col 2: Positions of veh's immedtly ahead
 C1(y,2) = 2*22/7*B(6,5)+C1(1,1);      %     ditto
 C1(:,3) = (C1(:,2)-C1(:,1));          % Col 3: Distances between vehicles
 C1(:,4) = C(:,11);                    % Col 4: Speeds of vehicles
 C1(1:(y-1),5) = C1(2:y,4);            % Col 5: Speeds of veh's immediatly ahead
 C1(y,5) = C1(1,4);                    %     ditto
 C1(:,6) = C1(:,4)-C1(:,5);            % Speed differential: Following - leading
 z = C1(:,6) > 0.1;                    % Check where speed differential is +
 C1(:,6) = C1(:,6).*z+(1-z);           % Changing all Vc's <= 0.1  to 1
 C1(:,7) = C1(:,3)-C1(:,5).*B(10,5);   % Distance to cover before encrouching Hwm
 C1(:,8) = (C1(:,7)./C1(:,6));         % Time for vehicle to catch up front veh
 z1=any(C1(:,7)<=0);                   % Print when any Hw <= 0 - Warning
 C(:,13) = (C1(:,8)+B(12,5)).*z;       % Add times to Ts for event timings

end                                    % end of: if y > 1

%-------------------------------------------------------------
```

```
function [AA,B,C]=transit(A,AA,B,C,I)

%------------------------------ transit.m -----------------------------------------
% Function to move vehicle from approach A to an intermediate stage
%--------------------------------------------------------------------------------------

%[z,B]   = randn4(1,B);
z = randn;                                % Normal rand no for circulating speed generation
while any(z > 3.6 | z < -2.2) > 0                % z has to be > -2.2 & < 3.6 std's (as observed)
 z = randn;                               % If greater than, then generate another set of rand no's
end
Vinc = B(4,5) + B(5,5)*z;                        % Speed in circle generated from normal random deviate
 MiT  = B(25,I)/(Vinc*B(37,5));                  % Calc move-in time using speed of vehicle and distance
AS   = A(:,(1+B(8,5)*(I-1)):(B(8,5)+B(8,5)*(I-1)));   % Submatrix from A for approach I
AA1  = [AS(1,1) AS(1,2) AS(1,3) (B(12,5)+MiT) I Vinc 0 0 0 0];  % Move vehicle to AA1 from AS

% ----- Check headway between this veh and previous veh which has entered circle --------------
if (AA1(1,4)-B(23,I))<B(10,5) & AA1(1,4)>B(10,5)          % Check if Hw is < Hwm and if Ts > Hwm (for first veh
     generated at Hw < Hwm)
 AA1(1,9)  = 1;                           % Mark vehicle as following
 AA1(1,4)  = B(23,I)+B(10,5);                             % Arrival in circle delayed to ensure Hwm
 if AA1(1,6) > B(24,I)                     % If speed of next vehicle > leading vehicle
  AA1(1,8) = AA1(1,6);                       % Save speed of vehicle
  AA1(1,6) = B(24,I);                      % Set speed of following veh equal to leading vehicle
 end
end

% ----- Check headway between this veh and other entering vehs on this approach ----------------
if B(19,5)>0                                             % Are there any vehicles in Transit
 if any(AA(:,5)==I) > 0 ;                                % Are there any other Transit veh's on this approach
  if AA1(1,4)-max(AA((find(AA(:,5)==I)),4)) > B(10,5);   % Check the Hw between this vehicle and the one in
     front
   AA1(1,9)  = 1;                         % Mark vehicle as following
   AA1(1,4)  = max(AA((find(AA(:,5)==I)),4))+B(10,5);    % Arrival in circle delayed to ensure Hwm
   if AA1(1,6) > AA((find(AA(:,5)==I)),6)        % If speed of next vehicle > leading vehicle
    z = find(AA(:,4)==max(AA((find(AA(:,5)==I)),4)));    % Identity the position in AA of leading vehicle in transit
    AA1(1,8) = AA1(1,6);                         % Save speed of vehicle
    AA1(1,6) = AA(z,6);                                  % Set speed of following veh equal to leading vehicle
   end
  end
 end
end

% ---- Check Hw to conflicting veh passing approach, if veh is present ------------------------
if B(18,5) > 0
 z=find(C(:,6+I)==B(12,5));                             % Is there a confl. veh passing approach or is this event
    caused by that vehicle?
 if z > 0                                               % If so then
  z=z(1,1);                                             % Take the first one - to prevent errors
  Dd = B(1,I)-B(2,I)+2*pi*B(6,5).*((B(1,I)-B(2,I))<0);  % Distance between end of crit area and point of entry
  Ta = B(12,5) + Dd/C(z,11);              % Time conflicting veh will pass point of entry.
  if (Ta-AA1(1,4)) < B(10,5)              % Check if ent veh arrives within Hwm of circ veh at point of entry
   AA1(1,4) = Ta + B(10,5);               % If so delay arrival of entering vehicle
   if AA1(1,6) > C(z,11)                  % if V(ent vehicle) < V(circ veh)
    AA1(1,8) = AA1(1,6);                  % Save speed of entering vehicle
    AA1(1,6) = C(z,11);                          % Set V(ent veh) = V(circ veh)
   end
  end
 end
end

AA1(1,10)= AA1(1,4)+B(8+I,I+1-4*(I==4))/AA1(1,6);    % Time veh will arive at next critical point immediately after
    entry.
AA((B(19,5)+1),1:10) = AA1;                    % Place new vehicle at bottom of AA matrix
B(19,5) = B(19,5) + 1;
Ai = B(19,5);
B(23,I) = AA1(1,4);                     % Store time of arrival in circle
B(24,I) = AA1(1,6);                     % Store speed of veh in B
```

(addcirc2.m continued on next page)

```
% ---------------- Identify event 15 for circulating vehicles -----------------------------
X = B(10,5)*AA1(1,6);              % Distance covered in Hwm @ speed of entering vehicle
X = B(1,I)-X;                 % Critical point on circle for vehicles having to slow down not to collide with entering
    vehicle
X = X+(2*pi*B(6,5)).*(X<0);        % Increase negative positions by adding circumference
if B(18,5) > 0
  Da = X-C(:,1);                % Distance to critical point
  Da = Da+2*pi*B(6,5)*(Da<0);       % Increase negative values
  Da = (Da./C(:,11)+B(12,5));      % Time when will be at critical point (Event 15)
  Da = Da-(AA1(1,4)-Da);%.*(Da~=0);   % Move event 15 back in time (How long does ent veh require to enter)
  [x,y] = find(Da<AA1(1,4));          % Which veh's will reach crit point before Ent veh hits the circle
  if x > 0
   C(x,14)=Da(x,1);                   % Set event 14 times for the above vehicles
   if AA1(1,6) < C(x,11)              % Check if speed of entering veh < circ vehicle
    C(x,15)= AA1(1,6).*ones(size(x));  % If so store speed of entering vehicle in C to later adapt speed in upcircle
     to that of entering veh
   else
    C(x,15)= C(x,11);                 % If not store same vehicle speed
   end
  end
end

% -------------- Slow down transit veh if slow vehicles on next approach --------------------
Iu = I+1-(4*(I==4));            % Number of next approach
if any(AA(:,5)==Iu)             % Check if there are any vehicles on next approach in Transit
 x=find(AA(:,5)==Iu);            % Indices of vehicles on next approach
 [y,z]=max(AA(x,4));             % Time of arrival of last veh in circle from next approach and row number in x
 z=x(z,1);                   % Change row number z in x to z in AA.
 X = B(10,5)*AA(z,6);              % Distance covered in Hwm @ speed of last entering vehicle
 X = B(1,Iu)-X;                % Critical point on circle for vehicles having to slow down not to collide with entering
    vehicle
 X = X+(2*pi*B(6,5)).*(X<0);        % Increase negative positions by adding circumference
 Dc = X-B(1,I);                % Distance between upstream critical point and point of entry
 Dc = Dc+(2*pi*B(6,5)*(Dc<0));       % Make + when -
 K  = Dc./AA1(1,6)+AA1(1,4);        % Event_15 time for transit vehicle
 if K < y                   % If transit veh arrive at Crit point before last vehicle on next approach
  AA(Ai,8) = AA(Ai,6);          % Store speed of entering vehicle
  AA(Ai,6) = Dc./(y-AA1(1,4));           % Slow down transit vehicle
  AA(Ai,7) = 1;                 % Mark vehicle as been slowed down - change again when it reaches critical point
 end
end

% -------------- Slow down fast entering veh from previous approach --------------------
Iu = I-1+(4*(I==1));            % Number of previous approach
if any(AA(:,5)==Iu)             % Check if there are any vehicles on previous approach in Transit
 x=find(AA(:,5)==Iu);            % Indices of vehicles on previous approach
 X = B(10,5)*AA1(1,6);              % Distance covered in Hwm @ speed of entering vehicle
 X = B(1,I)-X;                % Critical point on circle for vehicles having to slow down not to collide with entering
    vehicle
 X = X+(2*pi*B(6,5)).*(X<0);        % Increase negative positions by adding circumference
 Dc = X-B(1,Iu);                % Distance between critical point and upstream entry
 Dc = Dc+(2*pi*B(6,5)*(Dc<0));       % Make + when -
 K  = Dc./AA(x(1,1),6)+AA(x(1,1),4);   % Event_15 time for first veh in transit vehicle from previous approach
 if K < AA1(1,4);               % Does upstream veh arrive at critical point before next veh enters
  y = x(1,1);                 % AA-row index of first veh on upstream app as indentified in x
  AA(y,8) = AA(y,6);            % Store speed of upstream veh
  AA(y,6) = (Dc./(AA1(1,4)-AA(y,4)));  % Reduce speed of upstream vehicle
  B(24,Iu)= AA(y,6);            % Store new speed in B
  AA(y,7) = 1;               % Mark vehicle as been slowed down - change again when it reaches critical point
  z = x(find(AA(x,9)==1),1);       % Are there other veh's following one of which speed has been changed
  if any(z>0)
   AA(z,8) = AA(z,6);              % Store speed of other following vehicles
   AA(z,6) = AA(y,6)*(ones(size(AA(z,6))));   % Change speed of those vehicles
   AA(z,7) = ones(size(AA(z,7)));          % Mark vehicle as been slowed down - change again when it reaches
    critical point
  end
 end
end
%---------------------------------------------------------------------------------
```

```
function [C,B] = addcirc2(AA,C,B,i,k)

%----------------------------------ADDCIRC2.M-----------------------------------------
% Function to move veh from AA to C and update
%-------------------------------------------------------------------------------------

rpe  = B(1:4,1:4);              % Position of entries, exits, etc.
rpex = B(5:8,1:4);              % Distance from entry to exits
rpep = B(9:12,1:4);             % Distance form entry to critical points

C2(1,1:5)=[rpe(1,i) AA(k,2) AA(k,1) AA(k,3) i]; % Move vehicle onto circ
C2(1,7:10) = rpep(i,:)./AA(k,6) + B(12,5);          % Time to critical points (event 2)
C2(1,6) = rpex(i,AA(k,2))./AA(k,6) + B(12,5);       % Time to exit (Event 3)
C2(1,11) = AA(k,6);                                 % Circulating speed of vehicle
C2(1,12) = B(12,5);                                 % System time
C2(1,14) = AA(k,7);                                 % Event 15 time
C2(1,15) = AA(k,8);                                 % Speed of slower vehicle on following approach
C2(1,17) = AA(k,9);                                 % Is vehicle following or not
if AA(k,2) == i+1 + (i==4)*(-4)   % left turn: leave out
 C = C;
else                                  % Combine new vehicle with existing C matrix
 C((B(18,5)+1),:)=C2;                 % Place new vehicle at bottom of C matrix
 B(18,5) = B(18,5) + 1;               % Add to counter of circ vehicles in B
end

% ----- Change speed of first following veh which slowed down to allow this veh to enter -----------
if B(18,5) > 1
 [Y,X] = sort(C(:,1));                % Sort positions around circle in ascending order for
 C = [Y C(X,2:17)];                   % Sort rest of matrix accord. to first column
 lu = i-1+(4*(i==1));                 % Number of previous approach
 x = find(C(:,14)==1 & C(:,5)==lu);   % Identify first veh's from previous app with reduced speed
 if any(x>0)
  z = find(C(:,14)==1 & C(:,11)==C(x(1,1),11)); % Find all vehicles following first vehicle including the first vehicle,
  [J,K]=size(z);                              % J would be the number of vehicles to change
  zz=z(J,1);
  if C(zz,15) > C2(1,11);             % IF desired speed of first following vehicle > leading veh
   V = C2(1,11);                      % Then speed is that of entering vehicle
  else                                        % if not then
   V = C(zz,15);                      % speed is that of initial leading vehicle
  end
  C(zz,11) = V;                               % Set speed of leading veh to the appropriate V

   % ----------- Calc new event times for vehicle with increased speed ---------------
   Dc = rpe(2,:)-C(zz,1);             % Distance to crit points: ends of crit area
   Dc = Dc+(Dc<0).*(2*pi*B(6,5));     % Remove negatives
   Dx = rpe(4,C(zz,2))-C(zz,1);       % Distance to exit
   Dx = Dx+(Dx<0).*(2*pi*B(6,5));     % Remove negatives
   C(zz,7:10) =(Dc./C(zz,11))+B(12,5); % New event times to reach critical points
   C(zz,6) = Dx./C(zz,11)+B(12,5);    % New event time for exiting
  if J > 1                            % if there are more than one vehicle following calc times when other
     vehicles will increase speeds
   dV = C(z(J,1),11)-C(z(J-1,1),11);  % Difference in speeds
   dX = C(z(J,1),1)-C(z(J-1,1),1);    % Existing distance headway
   dX = dX + 2*pi*B(6,5)*(dX < 0);    % Make + if -
   X = C(zz,11)*B(10,5) - dX;         % Additional distance headway required to travel at Hwm
   L = ones(J-1,1);
   if dV ~= 0                                  % If the speed diff is not equal to zero
    C(z(1:J-1),16)= X/dV.*cumsum(L) + B(12,5).* L;    % Event 16: Ts for other vehicles following in platoon to
     change speeds.
   else
    C(z(1:J-1),16)= B(12,5).* L;              % Event 16: Ts for other vehicles following in platoon to change
     speeds.
   end
   C(z(1:J-1),15)= V.*L;                      % Store speed of leading vehicle for later use when
     vehicle increase speed - event 16
  end
 end
```

(addcirc2.m continued on next page)

```
% ----------- Calc new event 14 : Catching up with slow vehicle ---------------------------
y = B(18,5);
C1 = zeros(y,8);                 % Define C1 for headway calcs
C1(:,1) = (C(:,1));              % Col 1: Positions around circle in ascending order
C1(1:(y-1),2) = C(2:y,1);        % Col 2: Positions of veh's immedtly ahead
C1(y,2) = 2*22/7*B(6,5)+C(1,1);%    ditto
C1(:,3) = (C1(:,2)-C1(:,1));     % Col 3: Distances between vehicles
C1(:,4) = C(:,11);               % Col 4: Speeds of vehicles
C1(1:(y-1),5) = C1(2:y,4);       % Col 5: Speeds of veh's immediatly ahead
C1(y,5) = C1(1,4);               %    ditto
C1(:,6) = C1(:,4)-C1(:,5);       % Speed differential: Following - leading
z = find(C1(:,6) > 0.1);         % Check where speed differential is > 0.1
if any(z>0)
 C1(z,7) = C1(z,3)-C1(z,5).*B(10,5);    % Distance to cover before encrouching Hwm
 C1(z,8) = (C1(z,7)./C1(z,6));          % Time for vehicle to catch up front veh
 C(z,13) = (C1(z,8)+B(12,5));           % Add times to Ts for event timings
 if any(C1(z,7)<0)==1;                         % If the vehicle is already closer than Hwm
  [x,y]=find(C1(z,7)<0);                % Identify which vehicle is closer
  C(z(x,1),11) = C1(z(x,1),5);          % Set speed equal to that of leading veh
   % ---- Update event times of the vehicle which speed was changed ------------------
  for i = 1:size(x)                            % If x > 1 do one at a time
   if size(x,1)>1
    nn = 'WARNING: THERE ARE MORE THAN TWO VEHICLES CLOSER THAN Hwm.'
   end
   x1 = x(i,1);                         % Identify which vehicle is closer
   Dc = rpe(2,:)-C(z(x1,1),1);                 % Distance to crit points: ends of crit area
   Dc = Dc+(Dc<0).*(2*pi*B(6,5));              % Remove negatives
   Dx = rpe(4,C(z(x1,1),2))-C(z(x1,1),1);      % Distance to exit
   Dx = Dx+(Dx<0).*(2*pi*B(6,5));                     % Remove negatives
   C(z(x1,1),7:10) =(Dc./C(z(x1,1),11))+B(12,5);  % New event times to reach critical points
   C(z(x1,1),6) = Dx./C(z(x1,1),11)+B(12,5);          % New event time for exiting
  end
 end
end
end                                     % end of: if y > 1

%------------------------------------------------------------------------------------------
```

```
function [C] = upcircle(C,B,AA,I)

%-------------------------------UPCIRCLE.M------------------------------------
% Function to update speed and events for veh catching up with faster vehicle
%------------------------------------------------------------------------------
rpe = B(1:4,1:4);                        % Positions in circle of entry, exits, etc.
if I == 14                               % Event 14 - catching up with vehicle
 [k,l]=find(C(:,13)==B(12,5));           % Which vehicle in C to be updated
 if size(k) > 0                          % To prevent errors
  k=k(1,1);                              % If k>1 then only take first one - only to prevent errors from occuring
    during execution
  C(k,17) = 1;                           % Mark as following vehicle
  if k==B(18,5)                                   % If vehicle to change is last vehicle in matrix
   C(k,11) = C(1,11);                    % then speed = speed of first vehicle
  else                                            % else adjust to speed of vehicle
   C(k,11) = C(k+1,11)*ones(size(k));    % immediatly ahead : k+1
  end
 end
elseif I == 15                           % Event 15 - circ veh conflict with entering vehicle
 [k,l]=find(C(:,14)==B(12,5));   % Which vehicle in C to be updated
 if size(k) > 0
  k=k(1,1);                      % If k>1 then only take first one - only to prevent errors from occuring during
    execution
  C(k,17) = 1;                   % Mark as following vehicle
  C(k,11) = C(k,15);             % Change speed to that of entering vehicle
 end
else                                     % Event 16 - following veh to accelerate back to desired speed
 [k,l]=find(C(:,16)==B(12,5));   % Which vehicle in C to be updated
 if size(k) > 0                  % Did this to prevent errors that dit occur.
  k=k(1,1);                      % If k>1 then only take first one - only to prevent errors from occuring during
    execution
  C(k,11) = C(k,15);             % Change speed to that of leading vehicle as stored in column 15.
 end
end
Dc = rpe(2,:)-C(k,1);            % Distance to crit points: ends of crit area
Dc = Dc+(Dc<0).*(2*pi*B(6,5));           % Remove negatives
Dx = rpe(4,C(k,2))-C(k,1);       % Distance to exit
Dx = Dx+(Dx<0).*(2*pi*B(6,5));   % Remove negatives
if size(k) > 0                   % Did this to prevent errors that dit occur.
 k=k(1,1);                       % If k>1 then only take first one - only to prevent errors from occuring during
    execution
 C(k,7:10) =(Dc./C(k,11))+B(12,5);% New event times to reach critical points
 C(k,6) = Dx./C(k,11)+B(12,5);   % New event time for exiting
end
%------------------------------------------------------------------------------
```

APPENDIX C: RANDOM NUMBER GENERATORS

A number of uniform pseudo-random number generators for computers, where each number is determined by its predecessor, are available. Four of these are discussed below:

i) Mid-square Method.

This method consists of (Hammer, 1951) squaring a z digit number y_0. Then the middle z digits of the result are taken to give the next random number y_1, which in turn is squared for the next number. This method suffers a number of disadvantages of which among others is a short cycle, repetition of random numbers if the wrong seed is selected and the possibility of a string of zero numbers if the middle z digits go to zero.

ii) Mid-product Technique.

This method is similar to the Mid-square method (Young et al, 1989) in that it involves taking the z digits of the product of the two previous random numbers $y_{(I-1)}$ and $y_{(I-2)}$. This method requires two seed numbers and although it offers the possibility of a longer cycle this technique has fallen into disfavour.

iii) Linear Congruential Generator.

This method can be expressed as follows (Press et al, 1986):

$$Z_i = (aZ_i + c) \mod m \tag{1-1}$$

where:	a -	multiplier (integer)
	c -	constant (integer)
	m -	modulus - ± word size of the machine 2^{32} (integer)
	Z_i -	ith pseudo-random number

If the constant c is equal to zero, this method is referred to as the multiplicative congruential (power residue) technique, else it is also known as the mixed congruential method (Young et al, 1989). One advantage of the mixed congruential method is that it can generate a full cycle of m random numbers before repeating itself, while the cycle of the multiplicative congruential technique is less than m. However, the mixed congruential method will eventually generate a number equal to zero which can create problems for some mathematical functions. Most systems and/or software are supplied with built-in random number generators, which are almost always linear congruential generators. Although these generators have the advantage of speed, they have one major disadvantage in that they are not free of sequential correlation on successive calls (Press et al, 1986).

iv) Three tiered Multiplicative Congruential Generator

Wichmann and Hill (1982) combined the results of three simple multiplicative congruential generators by adding them and taking the fractional part. Each generator uses a prime number as a modulus and a primitive root for a multiplier which guarantees a complete cycle. Wichmann and Hill (1982) claim that their algorithm is reasonably short, reasonably fast, machine independent, easily programable in any language and statistically sound. According to them it has a cycle length exceeding 6.95×10^{12} which would mean that even if some 1000 random numbers per second are generated continuously, the sequence will not repeat itself for more than 220 years. A Matlab listing of the program is given in Listing 4.1.

Listing 4.1: A Matlab routine for the Wichmann and Hill random number generator

```
B1 , B2 and B3 set as integers between 1 and 30000 as seed numbers before the first entry.

B1 = 171 .* mod(B1,177) - 2 .* B1/177;
B2 = 172 .* mod(B2,176) - 35 .* B2/176;
B3 = 170 .* mod(B3,178) - 63 .* B3/178;

B1 = B1 + 30269   if (B1 < 0);
B2 = B2 + 30307   if (B2 < 0);
B3 = B3 + 30323   if (B3 < 0);

R = B1./30269 + B2./30307 + B3./30323;
R = R - round (R - .5);
end
```

Normal random deviates are invariably based on normal random deviates. Matlab also has a built-in normal distributed random number generator which uses a second copy of the linear congruential method to generate a set of uniformly distributed random numbers which are then transformed to a set of normal random numbers (Matlab, 1992) using the method as described by Box and Muller (Forsythe et al, 1977). In TRACSIM a separate routine was used to generate normal random deviates. While still employing the Box-Muller method, this routine referenced uniform random numbers produced by the Wichmann and Hill routine and not the built generator based on the linear congruential method. A listing of this routine to generate normal random deviates is shown in Listing 4.2.

Listing 4.2: A Matlab routine for normal random deviates using the Box- Muller transformation

```
S = 1.5;
while max(S) > 1
[U,B] = rand2(2*j,B);
V = 2.*U - 1;
S = V(1:j,1).^2 + V(j+1:2*j,1).^2;
end
R = V(1:j,1).*(-2.*log(S)./S).^.5;
```

APPENDIX D: SIMULATION RESULTS OF OD-STUDY

Table D1: Detailed simulation results for Circle 1.

Effect of Origin-Destination pattern on delays

North					West					East				South			
Flows (veh/hour)			Delay	Rho	Flows (veh/hour)			Delay	Rho	Flows (veh/hour)			Delay	Flows (veh/hour)			Delay
Input	Simulated	Circle	(Sec)		Input	Simulated	Circle	(Sec)		Input	Simulated	Circle	(Sec)	Input	Simulated	Circle	(Sec)
602	601	558	3.40	0.00	2	3	1257	17.20	0.25	1802	698	603	1759.65	602	559	699	198.71
602	627	608	4.88	0.00	2	3	1212	1.72	0.50	602	605	627	26.70	602	610	605	35.93
602	605	589	3.32	0.00	2	4	785	4.19	0.75	200	197	605	4.89	602	589	197	1.81
602	607	607	3.85	0.00	2	2	607	1.48	1.00	2	3	609	5.40	602	607	2	0.02
602	611	588	7.16	0.25	152	139	1123	9.30	0.25	1352	673	611	1433.70	452	449	673	25.12
602	601	615	4.18	0.25	152	150	910	4.42	0.50	452	447	599	7.28	452	466	446	4.14
602	604	599	4.82	0.25	152	147	613	4.88	0.75	151	158	604	5.25	452	458	157	1.35
602	636	591	3.72	0.25	152	137	456	3.85	1.00	2	1	636	0.90	452	457	2	0.00
602	593	611	12.40	0.50	302	313	1014	18.44	0.25	902	717	590	682.55	302	300	717	11.32
602	599	649	6.77	0.50	302	313	640	5.76	0.50	302	303	601	4.31	302	339	305	1.59
602	604	613	6.61	0.50	302	289	629	4.50	0.57	302	306	603	5.77	302	325	307	1.93
602	625	607	5.40	0.50	302	309	392	3.71	0.75	101	95	626	4.40	302	301	97	0.55
602	610	606	6.05	0.50	302	297	311	4.79	1.00	2	3	611	1.51	302	313	3	0.01
602	607	599	12.73	0.75	452	451	839	10.51	0.13	1052	691	605	1073.35	152	149	693	6.17
602	589	581	13.08	0.75	452	440	607	5.31	0.25	452	465	591	9.41	152	144	467	2.52
602	607	631	14.24	0.75	452	473	313	4.35	0.50	152	154	607	3.55	152	164	153	0.64
602	629	612	18.20	0.75	452	461	203	6.01	0.75	52	52	631	4.37	152	154	53	0.11
602	613	598	11.90	0.75	452	445	153	7.75	1.00	2	2	615	2.77	152	155	3	0.02
602	603	597	15.95	1.00	602	599	0	0.00	0.00	2	1	604	3.97	2	0	1	0.00
702	709	680	11.81	0.00	2	3	1275	12.96	0.25	2102	599	709	2021.61	702	681	598	147.54
702	699	686	8.97	0.00	2	0	1284	0.00	0.50	702	601	696	527.76	702	685	601	225.62
702	712	717	10.64	0.00	2	3	950	7.45	0.75	233	236	713	5.27	702	717	234	2.76
702	711	708	9.84	0.00	2	1	709	11.76	1.00	2	3	711	1.42	702	709	2	0.02
702	700	688	20.17	0.25	177	171	1123	10.86	0.25	1577	603	701	1791.09	527	521	604	19.38
702	709	761	25.32	0.25	177	211	1069	10.03	0.50	527	519	709	71.71	527	553	519	10.25
702	745	683	10.89	0.25	177	164	699	6.37	0.75	175	182	744	7.01	527	519	184	0.91
702	675	699	8.31	0.25	177	185	516	6.23	1.00	2	3	674	3.96	527	517	3	0.00
702	735	677	29.11	0.50	352	339	904	10.37	0.25	1052	563	737	1305.37	352	344	564	6.78
702	701	727	47.52	0.50	352	379	704	10.13	0.50	352	355	699	10.42	352	350	355	2.92
702	703	705	31.16	0.50	352	355	659	7.83	0.57	302	307	701	10.81	352	354	309	2.08
702	717	689	20.63	0.50	352	334	463	5.91	0.75	118	106	717	5.05	352	357	105	0.53
702	730	711	30.59	0.50	352	339	373	6.30	1.00	2	1	729	1.34	352	372	2	0.00
702	681	707	66.92	0.75	527	533	794	23.27	0.13	1227	623	682	1501.99	177	173	624	5.53
702	660	729	350.90	0.75	527	548	687	15.09	0.25	527	504	659	28.40	177	184	503	3.45
702	686	695	87.23	0.75	527	513	377	17.03	0.50	177	193	685	9.70	177	186	193	1.19
702	699	674	71.22	0.75	527	522	207	9.46	0.75	60	55	701	6.81	177	153	57	0.57
702	683	733	97.90	0.75	527	559	174	8.35	1.00	2	3	681	8.85	177	174	1	0.01
702	633	714	272.21	1.00	702	715	1	0.00	0.00	2	2	635	3.55	2	3	1	0.00
802	793	799	141.66	0.00	2	1	1271	8.40	0.25	2402	472	791	2230.25	802	801	472	100.37
802	787	774	93.11	0.00	2	3	1264	3.78	0.50	802	492	788	1082.18	802	771	493	41.31
802	809	809	116.75	0.00	2	0	1055	0.00	0.75	266	247	808	18.46	802	812	245	6.32
802	817	792	36.60	0.00	2	3	791	9.98	1.00	2	1	817	0.00	802	793	2	0.01
802	752	802	293.17	0.25	202	187	1151	18.92	0.25	1802	536	751	1954.43	602	618	537	21.12
802	756	820	311.31	0.25	202	209	1125	13.34	0.50	602	512	755	330.80	602	616	513	18.25
802	757	805	245.80	0.25	202	198	791	7.84	0.75	200	185	757	10.34	602	608	186	2.02
802	785	787	46.70	0.25	202	196	590	7.44	1.00	2	3	784	4.50	602	593	2	0.01
802	733	756	214.92	0.50	402	377	929	22.36	0.25	1202	548	734	1540.37	402	383	549	8.30
802	713	797	416.41	0.50	402	401	805	11.91	0.50	402	408	710	15.38	402	398	408	4.26
802	678	829	407.42	0.50	402	428	708	7.74	0.57	302	304	680	7.71	402	407	303	1.76
802	712	803	482.46	0.50	402	401	525	12.66	0.75	134	123	711	7.82	402	407	123	0.69
802	704	787	261.62	0.50	402	393	397	10.59	1.00	2	1	707	0.00	402	398	0	0.00
802	641	762	511.89	0.75	602	569	859	46.57	0.13	1402	664	642	1475.25	202	195	665	5.23
802	595	821	808.63	0.75	602	598	829	54.99	0.25	602	602	596	25.32	202	228	601	5.10
802	615	817	611.24	0.75	602	594	426	7.52	0.50	202	201	613	4.54	202	225	202	1.07
802	663	778	518.04	0.75	602	590	249	17.17	0.75	68	62	663	6.59	202	190	62	0.40
802	636	800	571.57	0.75	602	615	185	13.91	1.00	2	1	635	9.05	202	186	1	0.02
802	557	803	808.70	1.00	802	805	0	0.00	0.00	2	1	557	3.24	2	1	1	0.00

Table D2: Detailed simulation results for Circle 2.
Effect of Origin-Destination pattern on delays

North					West					East				South			
Flows (veh/hour)			Delay	Rho	Flows (veh/hour)			Delay	Rho	Flows (veh/hour)			Delay	Flows (veh/hour)			Delay
Input	Simulated	Circle	(Sec)		Input	Simulated	Circle	(Sec)		Input	Simulated	Circle	(Sec)	Input	Simulated	Circle	(Sec)
600	608	561	1.21	0.00	1	1	1450	0.10	0.25	1800	892	608	1540.00	600	559	891	294.00
600	596	640	1.35	0.00	1	1	1252	10.48	0.50	600	616	596	7.82	600	636	615	11.89
600	600	610	1.19	0.00	1	1	808	3.06	0.75	198	199	600	3.50	600	609	199	1.55
600	629	610	1.08	0.00	1	1	610	6.16	1.00	1	1	630	0.00	600	610	0	0.01
600	601	593	2.05	0.25	150	137	1348	6.23	0.25	1350	889	602	981.24	450	458	889	48.60
600	609	627	1.82	0.25	150	163	890	4.13	0.50	450	424	608	4.37	450	465	425	2.87
600	588	607	1.68	0.25	150	145	601	4.71	0.75	149	140	587	2.73	450	461	141	0.67
600	601	603	1.73	0.25	150	148	454	4.72	1.00	1	1	601	0.00	450	454	0	0.00
600	611	600	3.47	0.50	300	290	1182	6.10	0.25	900	870	611	192.22	300	313	870	11.93
600	595	611	3.02	0.50	300	315	585	4.37	0.40	300	288	595	3.37	300	295	289	1.41
600	607	608	3.19	0.50	300	311	589	3.17	0.50	300	293	607	3.66	300	296	293	1.58
600	611	593	3.33	0.50	300	282	405	3.24	0.75	99	95	612	2.27	300	311	94	0.36
600	603	608	2.54	0.50	300	310	298	3.72	1.00	1	1	603	4.04	300	298	0	0.00
600	621	602	5.80	0.75	450	442	1033	5.58	0.13	1050	871	620	620.95	150	161	871	7.34
600	613	604	4.76	0.75	450	445	635	3.63	0.25	450	477	612	5.73	150	159	476	2.33
600	625	608	5.10	0.75	450	468	289	2.99	0.50	150	150	625	2.90	150	140	149	0.40
600	618	594	4.20	0.75	450	436	204	2.19	0.75	50	47	618	1.83	150	159	47	0.19
600	575	649	4.37	0.75	450	482	169	2.98	1.00	1	1	576	5.30	150	168	0	0.00
600	618	622	5.52	1.00	600	620	1	0.00	0.00	1	1	616	925.00	1	1	1	0.00
700	691	632	1.49	0.00	1	1	1437	13.43	0.25	2100	805	691	1711.96	700	631	805	302.29
700	743	709	3.14	0.00	1	2	1406	7.68	0.50	700	701	742	66.31	700	707	700	50.94
700	701	716	1.68	0.00	1	0	937	4.33	0.75	231	221	702	4.83	700	715	221	2.37
700	680	733	1.97	0.00	1	0	737	0.50	1.00	1	3	679	1.11	700	733	3	0.00
700	721	716	3.48	0.25	175	171	1290	9.89	0.25	1575	745	720	1507.98	525	544	745	25.76
700	739	691	2.77	0.25	175	163	1031	6.72	0.50	525	503	739	8.64	525	527	503	4.88
700	705	729	3.72	0.25	175	181	729	7.19	0.75	173	181	707	6.48	525	549	181	1.52
700	715	701	2.33	0.25	175	159	541	6.87	1.00	1	0	715	4.06	525	541	0	0.00
700	703	675	5.39	0.50	350	327	1129	8.35	0.25	1050	781	704	814.78	350	349	781	13.70
700	730	721	8.95	0.50	350	373	713	5.52	0.50	350	365	730	7.34	350	350	365	2.26
700	718	666	5.09	0.50	350	337	629	4.50	0.54	300	301	717	5.72	350	329	301	1.86
700	715	683	4.95	0.50	350	333	481	4.85	0.75	116	134	715	4.67	350	348	133	0.43
700	711	709	5.54	0.50	350	359	351	5.23	1.00	1	1	711	0.00	350	350	1	0.00
700	715	715	10.79	0.75	525	543	940	6.26	0.13	1225	766	714	1089.10	175	173	767	6.04
700	733	731	25.90	0.75	525	550	708	7.20	0.25	525	528	733	14.75	175	181	528	2.65
700	716	726	13.04	0.75	525	550	339	3.18	0.50	175	163	716	6.30	175	177	163	0.57
700	724	703	7.39	0.75	525	517	245	4.76	0.75	58	59	725	3.26	175	185	59	0.15
700	689	700	11.71	0.75	525	545	156	8.23	1.00	1	1	689	0.00	175	155	1	0.00
700	684	719	15.40	1.00	700	719	3	0.00	0.00	1	3	684	1.11	1	0	3	0.00
800	828	760	4.96	0.00	1	1	1422	10.29	0.25	2400	659	828	2010.00	800	762	660	91.89
800	787	742	2.91	0.00	1	1	1438	1.42	0.50	800	696	787	319.96	800	742	696	448.00
800	769	834	2.60	0.00	1	1	1088	7.79	0.75	264	256	769	7.92	800	831	256	2.42
800	816	796	2.46	0.00	1	1	796	3.94	1.00	1	1	816	0.00	800	795	0	0.00
800	832	816	6.14	0.25	200	211	1219	11.79	0.25	1800	614	832	1882.83	600	605	615	11.68
800	824	829	7.53	0.25	200	215	1201	9.77	0.50	600	589	824	84.72	600	613	589	10.20
800	809	803	4.24	0.25	200	187	815	8.14	0.75	198	199	809	6.32	600	616	199	1.44
800	801	774	4.81	0.25	200	184	590	8.58	1.00	1	1	802	5.57	600	590	0	0.02
800	840	791	27.39	0.50	400	389	1035	10.53	0.25	1200	631	840	1346.03	400	402	631	6.87
800	802	791	14.05	0.50	400	379	697	6.82	0.40	300	285	803	10.23	400	411	285	1.72
800	818	799	12.34	0.50	400	395	806	9.74	0.50	400	403	819	26.70	400	404	403	2.42
800	780	791	11.67	0.50	400	415	499	5.51	0.75	132	123	781	5.00	400	375	123	0.55
800	792	828	9.58	0.50	400	394	435	6.67	1.00	1	1	794	2.13	400	434	0	0.01
800	795	803	87.32	0.75	600	605	835	11.14	0.13	1400	636	795	1471.72	200	199	635	4.78
800	793	819	68.96	0.75	600	623	805	10.32	0.25	600	608	793	83.15	200	195	611	3.35
800	803	795	43.44	0.75	600	603	383	5.53	0.50	200	189	804	6.24	200	194	189	0.65
800	820	801	70.25	0.75	600	591	279	5.60	0.75	66	70	819	4.85	200	209	70	0.24
800	820	791	51.53	0.75	600	599	192	4.51	1.00	1	1	821	4.91	200	192	0	0.00
800	783	790	217.72	1.00	800	788	1	0.01	0.00	1	1	782	1174.00	1	1	1	0.00