



UNIVERSITY OF
KWAZULU-NATAL

INYUVESI
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**INFLUENCE OF RAINFALL ON QUALITY OF SERVICE AT MULTILANE
ROUNDBABOUTS AND ITS TIME HEADWAY IMPLICATIONS**

By

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Submitted in fulfilment of the academic requirements for the degree of Doctor
of Philosophy in Civil Engineering

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JULY 2018

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EXAMINER'S COPY

PREFACE

The research contained in this thesis was completed by the candidate while based in the Discipline of Civil Engineering, School of the College of Agriculture, Engineering and Science, University of KwaZulu-Natal, Howard Campus South Africa.

The contents of this work have not been submitted in any form to another university and, except where the work of others is acknowledged in the text, the results reported are due to investigations by the candidate.

Signed: Prof. Johnnie Ben-Edigbe

Date:

DECLARATION 1: PLAGIARISM

I, **Mr Ibijola Stephen Olukayode** declare as follows:

(i) the research reported in this dissertation, except where otherwise indicated or acknowledged, is my original work;

(ii) this dissertation has not been submitted in full or in part for any degree or examination to any other university;

(iii) this dissertation does not contain other persons' data, pictures, graphs or other information, unless specifically acknowledged as being sourced from other persons;

(iv) this dissertation does not contain other persons' writing, unless specifically acknowledged as being sourced from other researchers. Where other written sources have been quoted, then:

a) their words have been re-written, but the general information attributed to them has been referenced;

b) where their exact words have been used, their writing has been placed inside quotation marks, and referenced;

(v) where I have used material for which publications followed, I have indicated in detail my role in the work;

(vi) this dissertation is primarily a collection of material, prepared by myself, published as journal articles or presented as a poster and oral presentations at conferences. In some cases, additional material has been included;

(vii) this dissertation does not contain text, graphics or tables copied and pasted from the Internet, unless specifically acknowledged, and the source being detailed in the dissertation and in the References sections.

Signed: Ibijola Stephen Olukayode

Date: July 2018.

DECLARATION 2: PUBLICATIONS AND TRAINING

Details of publications that form part and/or include research presented in this thesis.

Book Chapter:

1. Ibijola, S. O., & Ben-Edigbe, J. (2016). Effects of Rainfall on Control Delay and Queue at Multilane Roundabout, Proceeding of Canadian Society of Civil Engineer **Conference**, Resilient Infrastructure, ISBN 978-1-5108-4359-2, vol (4), pp 3118-3127.

Publications:

2. Ibijola, S. O. and Ben-Edigbe, J. (2018). Influence of Rainfall on Driver Behaviour and Gap Acceptance Characteristics at Multilane Roundabouts. Open Transportation Journal. (Abstract/Indexed in Scopus, Manuscript DHET Accredited Journal) Vol 12 pp 3-13 doi: 10.2174/187444780181201
3. Ibijola, S. O., & Ben-Edigbe J. (2018). Impact of rainfall on volume/capacity ratio at multilane roundabout, Hong Kong Society for Transportation Studies, Transportation Systems Conference. (Accepted).
4. Ben-Edigbe, J, and Ibijola, S. O. (2018). Anomalous Capacity Shrinkage at Multi-lane Roundabout Caused by Rainfall. ICE-Transport Journal. ISSN 0965-092X, (Abstract/Indexed in SCI and Scopus; impact factor 0.402, DHET Accredited Journal). doi.org/10.1680/jtran.17.00092.
5. Ben-Edigbe, J. and Ibijola, S. O. (2018). Evaluation of Roundabouts Functional Quality of Service Deterioration Caused by Rainfall, Promet-Traffic and Transportation. (Abstract/Indexed in SCI and Scopus; impact factor 0.43, DHET Accredited Journal, Manuscript in press; accepted for publication).

6. Ben-Edigbe J., Amir Pakshir and Ibijola S.O. (2017). Extent of Entry Capacity Loss at Roundabouts Caused by Rainy conditions, *Advances in Civil Engineering*. (Abstract/Indexed in SCI and Scopus; impact factor 0.402, DHET Accredited Journal). doi.org/10.1155/2018/4192323.
7. Ibijola, S. O., & Ben-Edigbe, J. (2016). Effects of Rainfall on Control Delay and Queue at Multilane Roundabout, Paper presentation at Canadian Society for Engineering Conference on 1st to 4th June 2016 at London, Ontario, Canada. (Presented by Ibijola S. O.).

The research reported on is based on the data collected from at roundabouts in Durban, Kwazulu-Natal, South Africa. This work is an analysis of data collected at four roundabouts in Durban, Kwazulu-Natal, South Africa. The empirical capacity model was fused into the highway capacity manual (HCM) delay model which introduced the roundabout geometry into the delay model, the effect of rainfall on delay and queue length was investigated in conference paper in book chapter. An empirical method of estimating the follow-up time and critical gap was developed and the rainfall effect on follow-up time and critical gap under varying volume capacity ratio is investigated in paper 2. The effect of rainfall on volume capacity ratio was investigated in conference paper 3. A linear empirical capacity model for capacity estimation under two weather conditions (Dry and rainy conditions) was developed and the effect of rainfall on entry and circulating capacity, and reserved capacity was investigated in paper 4. A functional quality of service criteria for assessment of roundabout service delivery was developed, using the user and provider's perspective and the table is used in assessing the rainfall effect on the roundabout service delivery paper 5. The effect of rainfall on entry capacity was investigated in paper 6 and the effect of rainfall on delay and queue length was investigated in conference paper 7.

Training:

- (i) Transportation planning and safety course (2016) at Indian Institute of Technology, Delhi, India, 1st – 8th December 2016.

This research would not have been possible without assistance from my supervisor Prof. Johnnie Ben-Edigbe who mentored me all through the research work, the Metro Police who provided support during the traffic data collection, eThekwini Transport Authority who gave an approval for traffic data collection, eThekwini engineering and record service department who gave me a password to access the live rain data on their website.

Signed: Ibijola Stephen Olukayode

Date: July 2018.

DEDICATION

This thesis is dedicated to my mother (Mrs. Florence Beatrice Ibijola).

ACKNOWLEDGEMENTS

I thank God, my creator who blessed my life with good health, made provision for my finances, and showed me favour in all ways during the research work.

I appreciate my supervisor, Prof. Johnnie Ben-Edigbe, who mentored me in this study. I learnt much from his wealth of experience, both academically and morally. I appreciate his wisdom, words of fatherly advice and encouragement during the period of the research work.

The **Academic Leader** (Prof. Derek), Prof. Smith, Ooma Chetty, Mr Logan, Mr. Ishan and all staff of the Civil Engineering Department of the University of **KwaZulu-Natal** are appreciated for making the environment friendly.

I acknowledge my beautiful wife (Kemi), wonderful children (Seyi, Samuel and Nifemi) for their prayers, endurance and support during the period of my study. My dad, mother, brothers and sisters are well appreciated for the moral support throughout the course of my study.

I thank the Ekiti State government, the Head of Service (Dr. Faseluka), Mr Babatunde Akilo, Mr Deji Ajayi, Engr. Adeola Johnson and my Director (Engr. Julius Olofin) for their moral and financial support. I also appreciate my professional colleagues Engr. Adewumi, Engr. Ayegbusi Ayo, Engr. Richard Balogun, Engr. Adelodun Tope, Engr. Funke Adejuwon and others who gave both moral and financial support during the study.

The University of **KwaZulu-Natal** is well appreciated for the fee remission, travel grant, and other benefits enjoyed during the study. I thank J.W. Nelson for the scholarship given to me during the research period. The Indian Institute of Technology is appreciated for the full scholarship to attend a training course in India in 2016.

The Kopek Construction Company, Mr. Maroon Mechleb, and Abosede Nwachuckw are appreciated for paying for my flight tickets the three times I went on holiday during the course of my study.

I appreciate the eThekwini Transport Authority, most especially Mr. Ashok Harridaw, the Engineering and Record Service Department most especially Mr. Angus Gwara, and the South Africa Metro Police most especially Mr Parbool in the eThekwini Municipality for the support given during data collection for the study. I express my gratitude to St. Augustine Hospital most especially Mrs Miriam Peter for giving me good care during my visit to the hospital. I appreciate the language editor (Barbara Dupont) for her support by removing the figures, tables and blank spaces from my thesis editing charges.

My friend and flatmate (Kayode Olowe, Eric Ayom), my friends and course mates (Fatukasi Tope, Mrs Yemisi Makinde, Janet Oyaro) who are like siblings, are well appreciated for the jokes and words of encouragement when facing tough academic challenges. Mr Lawrence Adepoju, Kola Fabiti, Ayo Ajayi, Bar. Lawal, Toyin Sadiku, Wahab, Sola Olajiga, Dr. Tope Aladetuyi, who gave moral and financial support to my family in my absence, they are all appreciated. Dr. Adeyeye is appreciated as he stood by me when I needed him and throughout the programme he was full of encouragement. Ayobamidele Lawal is like a brother, and he encouraged me to undertake this PhD programme, Samson Akpotu, and a host of friends are also well appreciated. I appreciate all those I might not be able to mention but whose contributions made my research a success.

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LIST OF ABBREVIATIONS

Abbreviation		Full meaning
ART	-	Rigid Vehicle
ANOVA	-	Analysis of Variance
ATC	-	Automatic Traffic Counter
AOD	-	Above Ordinance Datum
BD	-	Heavy Truck and Trailer
DRT	-	Heavy Truck and Two Trailer
ETA	-	eThekwini Transportation Authority
FQS	-	Functional Quality of Service
GSM	-	Global System for Mobile
GPS	-	Global Positioning System
HCM	-	Highway Capacity Manual
HR	-	Heavy Rainfall
HV	-	Heavy Vehicles
ICU	-	Intersection Capacity Utilization
ID	-	Identification Number
IT	-	Information Technology
LOS	-	Level of Service
LR	-	Light Rainfall
M	-	Municipality road
MR	-	Moderate Rainfall
MV	-	Medium Vehicle
MC	-	Motorcycle
N	-	National Roads

NCHRP	-	National Cooperative Highway Research Program
R	-	Provincial Roads
PC	-	passenger Car
PCE	-	Passenger Car Equivalent
PCU	-	Passenger Car Unit
PDF	-	Probability Density Function
QOS	-	Quality of Service
SANRAL	-	South Africa National Roads Agency limited
SIDRA	-	Signalised and Unsignalised intersection Design and Research Aid.
SQS	-	Structural Quality of Service
SV	-	Short Wagon
SVT	-	Short Wagon with Trailer
TB	-	Axle Truck
TRB	-	Transportation Research Board
UK	-	United Kingdom
US	-	United States
USB	-	Universal Serial Bus
WMO	-	World Meteorological Organisation

LIST OF SYMBOLS

Symbol	-	Meaning
b	-	Bunch factor
D	-	Inscribed circle diameter
d	-	Delay
d_2	-	Queueing delay
d_3	-	Geometric delay
d_f	-	Yield line delay
d_t	-	Average delay on approach
d_m	-	Average delay for isolated minor road vehicles
e	-	Entry width
E_T	-	Passenger car equivalent for heavy vehicles
f_c	-	Gradient slope, Geometric parameter
F	-	Y-intercept, Geometric parameter
F_a	-	Accepted gap
f_{HV}	-	Heavy vehicle adjustment factor
F_r	-	Rejected gap
H	-	Headway
i	-	Rain intensity
k	-	Correction factor
L	-	Queue length
L_o	-	Initial queue length
L_w	-	Weaving length
l'	-	Flare length
O_F	-	Front overhanging

O_R	-	Rear overhanging
p_{in}^{De}	-	Probability of through vehicle entering inner circulating lane.
ρ	-	Traffic intensity
P_T	-	Proportion of demand volume that consist of heavy vehicles
p_{out}^{De}	-	Probability of through vehicle entering outer circulating lane.
q_{ae}	-	Arrival rate at entry approach
q_A	-	Traffic flow rate of flow rate ahead
q^{DC}	-	Flow rate of conflict stream of approach D
q_c, v_c	-	Circulating flow rate
q_e	-	Entry flow rate
q^e	-	Total flow rate at all approaches at the roundabout
Q_{Ese}, q_m, C	-	Entry capacity
Q_c	-	Circulating capacity
q_L	-	Traffic flow rate of left turning vehicle
q_k^e	-	Total flow rate of turning vehicles on all approaches at the roundabout
Q_p	-	Practical capacity
q_R	-	Traffic flow rate of right turning vehicle
Q_R	-	Reserved capacity
Q_T	-	Theoretical capacity
Q_{95}	-	95 percentile queueing length
φ	-	Entry angle
R, r	-	Entry radius
\bar{t}_a	-	Average accepted gap

\bar{t}_c	-	Average critical gap
G	-	Standard deviation
S	-	Spacing
t_c	-	Critical gap
t_f	-	Follow-up time
T	-	Flow rate duration
τ, t_c	-	Critical gap
w	-	Weaving width
w_B	-	Wheel base
$x, v/c$	-	Volume to capacity ratio, Degree of saturation
X^2	-	Chi-square
μ	-	Mean
σ	-	Variance

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ABSTRACT

Roundabouts, or traffic circles as **they are** often called in South Africa, are priority intersections with a unique yield rule. Drivers approaching the roundabout must give way to those that are already circulating the central island. The fixed features and yield rule do not change relative to rainfall; however, vehicular flow rate and driver behaviour are often affected by ambient conditions like rainfall among others. Consequently, **in this** the study the influence of rainfall on the quality of service delivery at multilane roundabouts and their implications for time headways **have been investigated**. Based on the hypothesis that rainfall, irrespective of intensity, has adverse effects on the quality of service delivery and time headway at roundabouts, an impact study was carried out in Durban, South Africa. Entry, circulating traffic flow rate and rainfall data were collected at four selected sites in Durban, South Africa. Over one million traffic volume data was collected during the **August 2016 to February 2017** rainy season. The key selection criterion is proximity to an active rain gauge. Empirical data **were** collected continuously for six weeks on each selected roundabout. **Rainfall** data **were** collected from surface rain gauge stations with a distance range of 0.75km – 1.18km from the selected sites. Three classes of rain precipitation intensity (i) (light rain, $i < 2.5\text{mm}$; moderate rain, $2.5\text{mm} < i \leq 10\text{mm}$; and heavy rain $10\text{mm} < i \leq 50\text{mm}$) were considered. Very heavy rain, with an intensity greater than 50mm/h, was not considered because of associated drag force and **aquaplaning** which might be difficult to separate from the rainfall effect. Daylight data **were** separated into peak and off-peak traffic periods. Peak period data **were** used to develop a quality of service criteria table and the off-peak data **were** used to determine traffic flow rate performance. Passenger car equivalent (PCE) values used to convert vehicles per hour to pce per hour was investigated for analytical suitability given rainy conditions. Entry flow rate was used as a function of circulating flow rate to model entry capacity and, hence, determine the reserve capacity. Initially, both linear and exponential **models** were used, in turn, to test for analytical suitability. Linear model was the preferred after exponential function failed empirical tests. Linear function was used to model the relationships between entry and circulating traffic flow rates. The ensuing entry capacity was also used in conjunction with headway and degree of saturation to estimate entry delay under dry, light, moderate and heavy rainy conditions. The impact study reasons that quality of service is not the same as level of service, hence, the criteria table cannot be the same. This is a clear departure from Highway Capacity Manual (HCM) prescription for roundabout level of service criteria table. The novel quality of service criteria table prescribed in this thesis, has delay and reserve capacity as the

key determinants of service grade. It is also referred to as **Functional Quality of service (FQS)** in the thesis. FQS criteria table was developed for each study site and used to assess their service delivery. The criteria table was divided into six classes (A to F), where A is the best grade and F is the worst. In any case, traffic performances were analysed and results show that; i) there is no significant difference between South Africa passenger car equivalent values and those estimated in the study; ii) the novel criteria table developed in the study is an effective determinant of FQS delivery at roundabouts; iii) entry traffic flow rate rates decreased because of rainfall and by extension induced a reduction in quality of service delivery at all surveyed sites; iv) entry delay and attendant queue increased during rainfall; v) time headway increased and entry reserve capacity decreased because of rainfall. **It has been** concluded that rainfall has an adverse effect on the FQS and also, that heavy rainfall has the most significant impact on FQS at roundabouts. **It is** proposed that in future research, on roundabout entry capacity estimation based on polynomial quadratic function where the single-variable quadratic polynomial would have density as the independent variable and flow rate as the dependent **be considered**.

CHAPTER 1

INTRODUCTION

1.1 Overview

A roundabout is a priority intersection where traffic flow rates in one direction around a central island. It operates on a unique yield, where the entry vehicles yield to the circulating vehicles. Flouting the roundabout yield rule may cause a traffic gridlock, trigger road rage, or even result in **traffic** accidents. Complying with yield rule will often lead to a delay for the drivers entering the roundabout. In circumstances where disturbances external to the road system occur, it is pertinent to expect an additional entry delay and time headways. These external disturbances include rainfall.

This thesis presents **results of studies on** the influence of rainfall at multilane roundabouts on the quality of service delivery and its implications for time headway. The studies are premised on the hypothesis that rainfall, irrespective of intensity, would affect the quality of service delivery at roundabouts and by extension the time headways. Using the empirical relationship between vehicles entering and circulating **the roundabout** under dry and rainy conditions, delay models will be formulated for different dry and rainy scenarios and compared. **Furthermore**, time headway that will depict drivers' behaviour at multilane roundabouts will also be investigated.

From the foregoing, initiatives and measures that include **investigation** into the influence of rainy condition on the quality of service at roundabouts must be taken into account to tackle avoidable delays associated with rainfall in South Africa. Therefore, this introductory chapter has been divided into six sections; in the immediate section, background information to the research problem is presented. It **is** followed by the research aim and objectives in section 1.3. The method of study is discussed in section 1.4. The scope and limitations of the **study** are presented in section

1.5. The significance of this **study** is discussed in section 1.6. The organisation of the thesis is presented in section 1.7

1.2 Background to the Research Problem

South Africa's road system can be divided into three categories; National, Provincial, and municipal roads. National routes connect major cities and are the highest category. South Africans often refer to roundabouts as traffic circles and most of them are installed on regional and municipal roadways. Bearing in mind that South Africa is **left-hand-travel**, vehicles approaching the roundabouts must give way to vehicles circulating on the right-hand side.

South Africa is a subtropical country with the coldest days **between** June to August. The average annual rainfall is 450mm (Otieno and Ochieng, 2004), but large and unpredictable variations are common. Overall, rainfall is greatest in the east and gradually decreases westward. For most part of the country, rain falls mainly in the summer months with brief afternoon thunderstorms. Notwithstanding the amount of rainfall on South African roundabouts, studies into their influence on the quality of service delivery have been very limited, if at all existing.

Roundabout quality of service and delay are intertwined. Good quality of service means minimised delay **while** poor quality of service means increased delay. Delay and degree of saturation also called volume capacity ratio (v/c) are sometimes used, albeit separately, to assess roundabout level of service. Delay is dependent on two key parameters, capacity and time headway, among others. It formed the basis for the HCM roundabout level of service criteria table. **In as much as reserve capacity is a parameter that could be used in traffic management, then why are road service providers keen on using delay as a parameter for effectiveness instead of reserve capacity?** After all, roundabouts are designed for traffic capacity not delay, it can be argued.

The overarching research problem is the evaluation of the extent of rainfall at **the** roundabout and its effect on the quality of service. Quality of service has often been used interchangeably with level of service, even though each has a unique definition. According to the Highway Capacity Manual (HCM), level of service is a measure of effectiveness, whereas, quality of service is

defined in this thesis as a measure of performance based on the perceptions of service provider and service user. According to the Florida Department of Transport, quality of service is defined as, “*how well the transportation facility has performed based on the road user’s perception*” (Florida-DOT, 2013). Kotler and Armstrong assert that quality of service is the perception of the performance of actual service and the user's expectation (Kotler and Armstrong, 2010). In Vuillemin’s opinion, quality cannot stand alone without considering the product or service from the provider (Vuillemin, 1999). Sakai **opined** that quality of service assessment should take into account the service provider and the customer’s perceptions of quality (Sakai *et al.*, 2011).

There are many hypothetical issues raised in this thesis. For example, could the roundabout capacity estimation method be stochastic, empirical, and linear or exponential? What about reserved capacity. Why has it not been used as an assessment criterion in previous studies? If it is assumed that the yield rule holds at roundabouts; to what extent would service delivery be affected by the rainy conditions? Given that when it rains, vehicles entering, and circulating are affected by the same weather conditions under the same yield rule. Is it possible that rainfall could induce bunching of circulating vehicles, thus, making acceptable gaps difficult for vehicles wanting to join the circulating stream? Road users and service providers have different perceptions of service delivery. Users are probably more concerned about the time spent and the consequential junction delay, whereas service providers’ concerns should be the performance of design parameters. This calls for the development of an assessment criteria that will take into account road users’ and service providers’ perception of service quality.

Then there is the issue of rainfall classifications. According to the American Meteorological Society, the intensity of rainfall at any given time and place may be classified as: ‘light’, the rate of fall varying between a trace and 0.25 cm per hour, the maximum rate of fall being no more than 0.025 cm in six minutes; ‘moderate’, from 0.26 to 0.76 cm per hour, the maximum rate of fall being no more than 0.076 cm in six minutes; ‘heavy’, over 0.76 cm per hour or more than 0.076 cm in six minutes (AMS, 2018). In previous studies rainfall was classified as light rain with intensity ($i \leq 2.5$ mm/h, moderate rain ($2.5 < i \leq 10$ mm/h) and heavy rain ($10 < i \leq 50$ mm/h) (Jarraud, 2008). The Spanish National Meteorological Institute defined rainfall intensity based on the following thresholds; light rainfall ($i \leq 2$ mm/h); moderate rainfall ($2 < i \leq 15$ mm/h); heavy

rainfall ($15 < i \leq 30$ mm/h); very heavy rainfall ($30 < i \leq 60$ mm/h) and torrential rainfall ($i > 60$ mm/h) (Llasat, 2001), the rain classification for these countries is presented in Table 1. 1.. Rainfall intensity thresholds vary considerably from one country to another, thus, affirming that it would be difficult to have a universal classification.

Table 1.1: Different Rain Classification System in the World

Type of rain	Intensity (mm/h)		
	AMS	WMO	SNMI
Light rain	< 2.5	< 2.5	< 2.0
Moderate rain	2.6 – 7.6	2.5 - 10	2-15
Heavy rain	>7.6	Oct-50	15-30
Very heavy	-	>50	30-60
Torrential rain	-	-	>60

Note: AMS is America meteorological Society, WMO is World Metrological Society, SNMI Spanish National Meteorological Institute

In quantitative and qualitative traffic analyses, the passenger car equivalent value is an essential parameter. It is used to state the traffic flow rate rates with heterogeneous composition. US Highway Capacity Manual (TRB, 2010) has defined the passenger car equivalent value as, “*the number of passenger cars that are displaced by a single heavy vehicle of a particular type under the prevailing roadway, traffic and control conditions*”. Based on the definition, it is apparent that the passenger car equivalent value is constrained by prevailing ambient conditions like rainfall. Thus, it could be argued that the passenger car equivalent value would also vary, giving prevailing conditions. In previous studies the application of the passenger car equivalent values has often been used with little or no explanation of their implication under constrained conditions (Ibrahim, A. T. & Hall, F. L. 1994, SHIn et. al 2011). It can even be suggested that the passenger car equivalent values are sometimes taken for granted on the premise that their effect on study outcomes is negligible. Researchers tend to apply the values broadly. That cannot be. In this thesis, passenger car equivalent values are investigated, appraised, and modified as required.

1.3 Research Aim and Objectives

The aim of this study is to investigate the impact of rainfall on the quality of service delivery at multilane roundabouts and their time headway implications. The research objectives are to:

- i. develop a quality of service criteria table for multi-lane roundabouts that would be used to assess roundabout performance under dry daylight and rainy conditions,
- ii. estimate the entry delay for multilane roundabouts under daylight dry and rainy conditions,
- iii. determine the quality of service for dry and rainy conditions from the criteria table developed in subsection (i) and compare the outcomes,
- iv. evaluate time headways under dry daylight and rainy weather conditions and compare the results.

1.4 Method of the Study

The method of study is both empirical and analytical. It is empirical because sample surveys were taken at selected sites and analytical because entering and circulating flow rate relationships were used to develop models. Models were developed for two scenarios (dry and rainfall) under daylight conditions.

Empirical data collected at selected sites reflected the study objectives as stated in section 1.3. Automatic traffic counters were installed at multi-lane roundabout approach entry and circulating entry points per arm. Collected data was collated and fed into the developed models for evaluation in regard to relevant traffic parameters. Once the service delivery objective was achieved, associated time headway as well as the acceptability of passenger car equivalent values were investigated. Passenger car equivalent values were adjusted where necessary.

Rainfall and traffic data was collected concurrently. Rain gauges were used to collect rainfall **data**. The setup of the rainfall impact study adopted and modified a method used previously by Mashros and Ben-Edigbe (2014) bearing in mind that the study differs. A ‘with and without’ rainfall study was initially considered but the approach suggests that without rainfall could imply wet road surfaces after rainfall. Because of this ambiguity, the method was renamed ‘dry and rainfall’ impact study to affirm that only results during rainfall and dry weather were considered. Rainfall classified as very heavy was not analysed in the thesis **because of aquaplaning**.

1.5 Research Scope and Limitations

The scope of this research is restricted to multi-lane roundabouts or traffic circles in Durban, South Africa because single-lane and mini roundabouts are not common. Most roundabouts are installed on regional roads. Regional roads are the third category of road in South Africa. They are feeder roads connecting towns to national and provincial roads. **Data on traffic** parameters were collected with automatic traffic counters continuously for six weeks at each surveyed site under dry and rainy conditions. Selected sites are located within a rain gauge station capture range of about 1.5km. Fixed rain gauge readings are checked intermittently against manually operated rain gauges serving to check and minimise errors.

Only dry and rainy conditions during daylight were processed and analysed. All selected sites have the same geometric design, good road surface and layout to minimise errors associated with traffic volume, and speed data collection. Manual measurements of key geometric parameters were carried out before equipment was installed. They serve **as** check on geometric design parameters provided by the municipal authorities. Each directional **flow rate** per arm is treated exclusively.

Different empirical road capacity estimation methods were considered and tested for suitability. Since passenger car equivalent values are instruments of traffic **flow rate** estimation, they cannot be ignored or treated casually. Consequently, South African passenger car equivalent values in use would be modified and used to convert traffic volume to flow.

The research limitations included, among others; rainfall periods in Durban **which** is between August and March every year. Peak rainfall is in January with an average of 134mm. This means that traffic data collections are restricted to the rainfall period. Only motorised vehicles were considered. Non-motorised transport is beyond the scope of this research. The total number of survey sites were constrained by funding, equipment, and manpower; nonetheless, four sites were surveyed. Automatic traffic counters were often chained to the nearest pole to minimise theft and vandalism. Survey sites were visited daily during data collection periods, partly to check the state of the equipment and to download captured data from the equipment to a laptop.

1.6 Significance of the Study

This thesis contributes to the state-of-art in modelling traffic flow rate at roundabouts during rainfall. It uses a unique criteria table developed for the surveyed sites and, hence, enriches literature with this method. The criteria table uses delay and reserve capacity as key indicators of traffic flow rate performance at roundabouts. **Reserve capacity is a measure of the overall physical design features of the intersection and a measure of additional veh/h or pc/h that can be accommodated by the traffic control device, and the concept that is used in several instances in this thesis which is really the net degree of saturation available.** This is a departure from the singular approach where only delay is relied upon for level of service determination. The criteria table can be used elsewhere with **appropriate** modification to the local environment. The incorporation of road user and provider's perspectives in the assessment of roundabout service delivery also enriches the existing literature. The influence of rain on the time headway at both the entry and circulating traffic using the empirical method is also a novel approach.

There have been studies on rainfall's influence on vehicular traffic and how these parameters are affected by rainfall in many countries, but very little has been done in South Africa if any. Moreover, the influence of rainfall intensity at roundabouts on quality of service delivery is yet to be studied and fully understood. Its significance is in its attempt to show that by mapping out specific areas where action is needed, delay at roundabout induced by rainfall can be minimised. In previous studies passenger car equivalent values were broadly applied to all conditions; an

approach that is questionable. Modified passenger car equivalent values can point to overestimation or underestimation of entry capacity values on specific sites and conditions.

This research has relevance to traffic management policy and decision-making processes. The findings in this thesis can be incorporated into a wider traffic management strategy. Predicted delay can be used for scenario building for traffic management under dry and rainy conditions. In **summary**, the study gives an understanding of how roundabouts perform under rainfall, which could be useful in roundabout traffic management and planning under rainy conditions.

1.7 Organisation of Thesis

The thesis is organised in an orderly fashion to enable the reader to follow the arguments presented therein with ease. Each chapter is structured to address issues aimed at strengthening the hypothesis that rainfall, irrespective of intensity, has adverse effects on the quality of service delivery at roundabouts. Note that **Figures** and **Tables** in the thesis are preceded by chapter number for ease of location; for example, **Figure 2.1** or **Table 4.2** shall be in chapters 2 and 4 respectively. The layout of the remainder of the thesis is as follows:

Chapter 2: Literature review on quality of service at roundabout is presented.

Chapter 3: Research methodology, the research framework, the site selection criteria and setup, the traffic and rainfall data collection, data processing and hypothesis testing are discussed.

Chapter 4: The empirical results per surveyed sites are discussed.

Chapter 5: The quality of service assessment is presented. A novel quality of service criteria table is developed and employed to determine the quality of service delivery at each surveyed site.

Chapter 6: The time headway implications of rainfall at roundabouts are presented.

Chapter 7: The summary of the findings, conclusions, and the way forward for future research works are presented.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

In the previous chapter the objectives of this study were set out. It is imperative that literature on relevant previous works and the theoretic framework is reviewed in order to support arising arguments in the later chapters. The study is concerned with the influence of rainfall on service delivery at multilane roundabouts and their implications for time headway. Service delivery in the context of highway engineering defines the interaction between road providers and users, where the provider offers a service and the road users either find time value or loses value as a result. Good roundabout service delivery provides road users with an increase in the value of time. It can be argued that research into the influence of rainfall on roundabouts' functional quality of service delivery has not been undertaken before this study, as there is no evidence of literature on previous works. Probably the closest research works are on the level of service and the entry capacity of roundabouts under dry weather conditions. They are usually in the form of measuring the extent of entry capacity and their associated delays under dry weather conditions. It should be mentioned in passing that literature on the influence of rain precipitation on roundabout performance is limited, if at all existing. In light of the aforementioned, the remainder of this chapter is devoted to the discussion of the interrelationship between functional quality of service delivery reduction and rainfall and their implications for time headway. Roundabouts' service delivery is constrained by factors that include traffic, road and ambient conditions. Traffic conditions refer to the characteristics of the traffic stream and the stream components that use the facility, such as traffic composition, directional distribution, proportion of different types of vehicles and their performance capability. Ambient conditions are usually weather, visibility, levels of pedestrian activity, number of parked vehicles, and frontage activity, among others. Road conditions which include road surface and geometric parameters are, number and direction of lanes, lane widths, shoulder widths, lateral clearances from edge of pavement, design speed, type of intersections, horizontal and vertical alignments.

The review of literature is divided into six sections. Section 2.2 deals with South Africa and rainfall, whilst section 2.3 is on roundabouts in South Africa. Section 2.4 addresses roundabouts' traffic characteristics, whilst section 2.5 is on the qualitative assessment of roundabout performance. In section 2.6, a novel quality of service assessment concept is discussed. In section 2.7, the impact of rainfall on quality of service delivery is presented, whilst section 2.8 addresses driver behaviour and time headway at roundabouts. Section 2.9 presents a summary of the chapter.

2.2 South Africa and Rainfall

South Africa is made up of nine administrative provincial centres which include: Eastern Cape, Free State, Gauteng, KwaZulu-Natal, Limpopo, Mpumalanga, Northern Cape, North West and the Western Cape. The rainfall pattern varies in each province. Rainfall of high intensity usually occurs during the summer months of November to March with some thunderstorms in the afternoon. In the Western Cape with the capital city of Cape Town, the rainfall occurs in the winter months of May to September. The intensity of rainfall varies from province to province in South Africa. The amount of precipitation in South Africa varies tremendously, which makes it difficult to predict the variation in the rainfall amount.

The amount of rainfall is higher in the eastern parts of the country and decreases towards the western parts. Generally, rainfall in South Africa occurs throughout the year with a varying amount of precipitation in each month in different parts of the country. The intensity of rainfall in the other parts of South Africa is lower when compared to rainfall intensity in city of Durban in the KwaZulu-Natal Province, where the rainfall intensity and frequency are high during the months of October to March. The highest amount of rainfall in South Africa occurs in the city of Durban in KwaZulu-Natal province with an annual average precipitation of 828mm (Olurotimi et al., 2017). In Durban, a light rain falls throughout the year, but the wet season occurs from October to March. The wettest period occurs in January, while June is the driest month in Durban. The average amount of precipitation in South Africa is 450mm (Otieno and Ochieng, 2004) compared to the global average amount of rainfall of 860mm. The driest part of South Africa is Richards Bay in the KwaZulu-Natal province with an average annual rainfall of 46mm.

Rainfall is most likely to affect the traffic flow rate at the roundabouts. This is because it impairs visibility more compared to other factors that associated with precipitation which includes the temperature and humidity. Most of the cars in South Africa have air conditioners which the drivers use to regulate the temperature inside the vehicle to a comfortable condition as well as keep the humidity out of effect. Rainfall varies in intensity and the intensity has a great influence on how it affects traffic flow rate.

Rainfall is the quantity of water, always expressed in millimeter (mm), precipitated as rainfall in a specific area at a given time interval (NRMAE, 1986). It has many characteristics which are; the amount of rainfall, the frequency, the distribution over the area, the time of occurrence, and intensity. Rain intensity is an important characteristic that affects the traffic flow rate, usually measured in mm per hour (mm/hr). Rainfall is classified into light, medium, heavy and very heavy rainfall, according to rainfall intensity (Jarraud, 2008), as shown in Table 2.1. The classification is in line with the World Meteorological Organisation’s (WMO) rainfall classification.

Table 2.1: Rain Classification

Type of rain	Intensity (mm/hr)
Light rain	< 2.5
Moderate rain	2.5 - 10
Heavy rain	10- 50
Very heavy or violent rain	More than 50

Source: Jarrud, 2008

There are different ways of measuring amount of rainfall which includes the use of a surface rain gauge, the weather radar and satellite imagery. The weather radar and satellite imagery do not measure the rain precipitation at the earth’s surface but above the earth’s surface. They are very useful in the prediction of the occurrence of rainfall from cloud movements. The surface rain gauge is relevant to this study because it collects the rain data at the earth’s surface where traffic

interacts directly with rainfall. The discussion in this chapter will be limited to the use of surface rain gauges for rain measurement.

2.3 Roundabouts in South Africa

The roads in South Africa are classified into national (N), provincial (R) and municipal (M) respectively. The national roads are the roads and freeways that connect the major cities in South Africa. These roads are maintained by South Africa National Agency Limited (SANRAL) and are designated by N followed by an assigned number. For example, N2, where N represents national road and assigned number 2 indicates the road designation. National roads are designated from N1 to N19. Provincial roads are next to the national roads; they are numbered by the designation R followed by an alphanumeric as an example R21, where R represents provincial road and assigned number 21 indicates the road designation. These roads serve as feeder roads to the national roads and as trunk roads where there is no national road. The roads are maintained by the provincial government road authority. They also vary in quality from gravel roads to freeways. Municipal roads are next to the provincial roads. They are street and township roads and maintained by the local or municipal road authority. Irrespective of the road type, they connect or meet each other at interchanges or intersections in the form of roundabouts, signalized and priority intersections. This study is on roundabouts as a form of intersection. Hence, **discussions in this section are limited to roundabouts in South Africa.**

The use of roundabouts in South Africa has not gained much traction, but in recent times the use of roundabouts is increasing. Intersections at some newly developed areas are built with roundabouts and some signalized intersection are being replaced with roundabouts in some locations. Typical examples are where three signalized intersections at the University of Zulu land, in the KwaZulu-Natal Province, were replaced with roundabouts in 2013 (Kendal and Reutener, 2014, Moodley, 2013), and another was carried out at a new market in Alberton, Gauteng in 2011. The main reasons for the replacement as noted by SANRAL was due to the safety at the roundabout, elimination of traffic signal cable theft and the need for a reduction in vehicle hijacking at the intersections. This is because total stop of vehicles at roundabout are not necessary compared to a signalised intersection where vehicles are to stop when the signal light is on red.

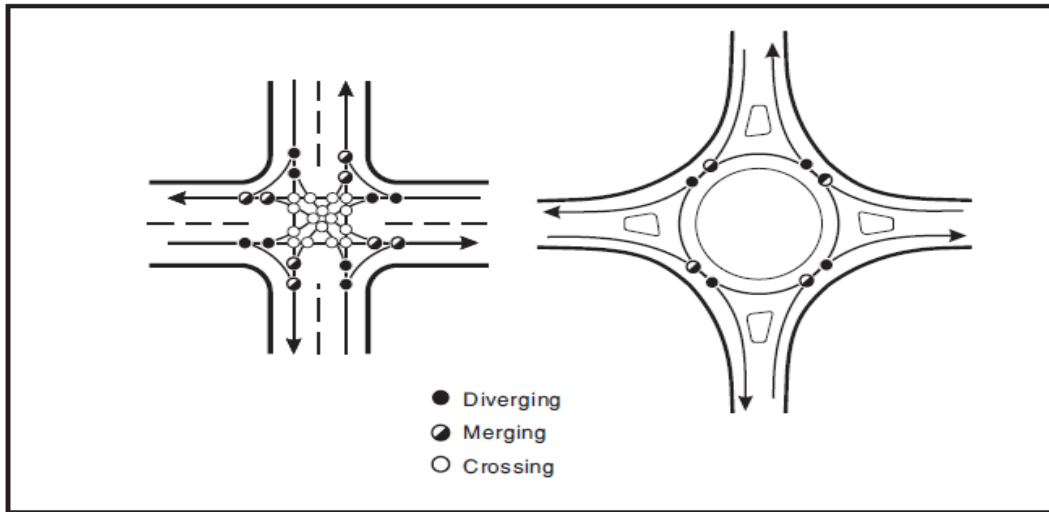
Kendal and Reutener add that the reasons for the upgrading of these intersections to roundabouts was to reduce the conflict of heavy and light vehicles, danger to pedestrians and reduction in the frequency of static traffic Kendal and Reutener (2014). This shows that the awareness of the quality of roundabouts is becoming recognized in South Africa. Some intersections in newly developed areas in KwaZulu-Natal and some other provinces are being made as roundabouts. A typical example is in Umhlanga which is located within Durban city in the KwaZulu-Natal Province at the coordinates of $29^{\circ} 43' 09''$ S $31^{\circ} 05' 09''$ E and Western Blvd, Cape Town in Western Cape Province located at $-33^{\circ} 54' 24.24''$ S $18^{\circ} 24' 44.57''$ E.

The SANRAL geometric design guidelines does not state the categories of the roundabouts in South Africa, but the roundabouts in South Africa have the geometric and descriptive features similar to roundabouts in other parts of the world. Roundabouts are divided into three categories (mini, single and multilane) according to the National Cooperative Highway Research Program (NCHRP) Report 672 of 2010, based on the size and the number of lanes in the roundabout (Robinson *et al.*, 2000). Construction of mini-roundabouts is inexpensive because of its small size and the pavement widening at the curb corner which is minimal. The central island is usually made up of road markings as it does not require raised Central Island. This type of roundabout is suitable for a low operating speed urban environment and in areas with an insufficient right of way. It is designed in such a way that it can accommodate passenger cars without traversing the central island, but larger vehicles can only traverse the central island and is usually of a single lane.

Conventional roundabouts have single or multiple entry lanes with a large, raised, inscribed diameter and non-traversable central island which distinguishes it from the mini roundabout. They are usually designed with a truck apron and an entry design speed of 40 to 50km/h. The size of this category of the roundabout depends on the available right of way. The approaching road is 3.4m - 3.7m and the entry width, a minimum of 5m for a single lane roundabout. The entry width of a two-lane roundabout is within the range of 8m. The design speed is 40km/h to 50km/h. The multilane roundabout is the category of roundabout having an approach of two or more lanes with a raised splitter island, non-traversable central island, and truck apron. The number of entry lanes is not necessarily the same for all approaches. The circulatory roadway is always wider to accommodate the vehicles travelling side by side. The circulating road width for two-lane

roundabouts is in the range of 8m - 16m, with a maximum radius or vehicle path of not more than 100m. The speed at the entry, exit and within the circulatory roadway is always the same or slightly higher than the single lane roundabout. In describing roundabouts, the following descriptive features are used in order to gain a clearer understanding: Central Island, Splitter Island, circulatory roadway, apron, yield line, accessible pedestrian crossings, and landscaping buffer.

A central island is an area around which vehicular traffic circulates and is usually raised at the centre of the roundabout. A splitter island is a speed controlling feature that deflects and reduces the entry speed of vehicles at the entry to the roundabout. It also separates the entry traffic from the exit traffic and provides a safe crossing for pedestrians. It could be raised or painted (Robinson *et al.*, 2000). A circulatory roadway is the roadway around the central island through which traffic travels and is usually a curved road. The apron is usually a mountable portion of the central island which is usually 50mm to 75mm in height and with a slope of 2 percent and a width of 1m to 4m. It is usually provided on small roundabouts to accommodate the large vehicles' wheel tracking. A yield line is a road marking along the inscribed circle usually at the entry lane of a roundabout into the circulatory roadway. The entry vehicles usually yield to vehicles in the circulatory roadway before crossing the marked line into the circulatory roadway. An accessible pedestrian crossing is usually set back from the yield line. It is cut at the minimum width of 3m within the splitter island to allow pedestrians, strollers, and wheelchairs to pass through. A landscaping buffer is the feature provided to separate the pedestrians from the vehicular traffic and allow pedestrians to cross at the designated locations. Roundabout operates on the yield rule, it can be argued that roundabouts operate more efficiently than signalized intersections in the sense that drivers do not have to stop completely at the intersection. Moreover, another important advantage of a roundabout as an intersection is conflict minimization. As shown below in Figure 2.1, roundabouts have **fewer** conflict points than priority intersections. Conflicts at roundabouts can be divided into three classes (queuing, crossing and merging). Queuing conflicts are caused when entry traffic queues while waiting for a sufficient gap to merge in the circulating traffic flow rate. Crossing conflicts are caused by the interaction of two traffic streams, whereas, merging conflicts are caused by the joining or separating (diverging) of two traffic streams.



Source: Robert Z (2014)

Figure 2.1: Conflict points reduction at four-arm roundabouts.

2.4 Roundabout Traffic Characteristics

The operating characteristics of multilane roundabouts are influenced by their geometric elements and have often led to separate capacities. Entry width is the width at the point where the entry road meets the circle, usually measured perpendicularly from the left edge to the right edge intersection line and the inscribed circle. The entry angle is a geometric angle that represents the entering and circulating traffic stream's conflicting angle. Entry radius is the minimum radius of curvature of the outside curb of the entry. Approach width is the width of the approaching road through which the traffic stream travels towards the entry of the roundabout. The inscribed circle diameter is the diameter of the outer curb to the opposite outer curb in which the central island diameter, the apron and the circulating roadway are inclusive. Circulatory roadway width is the width of the roadway around the central island through which the circulating flow rate travels.

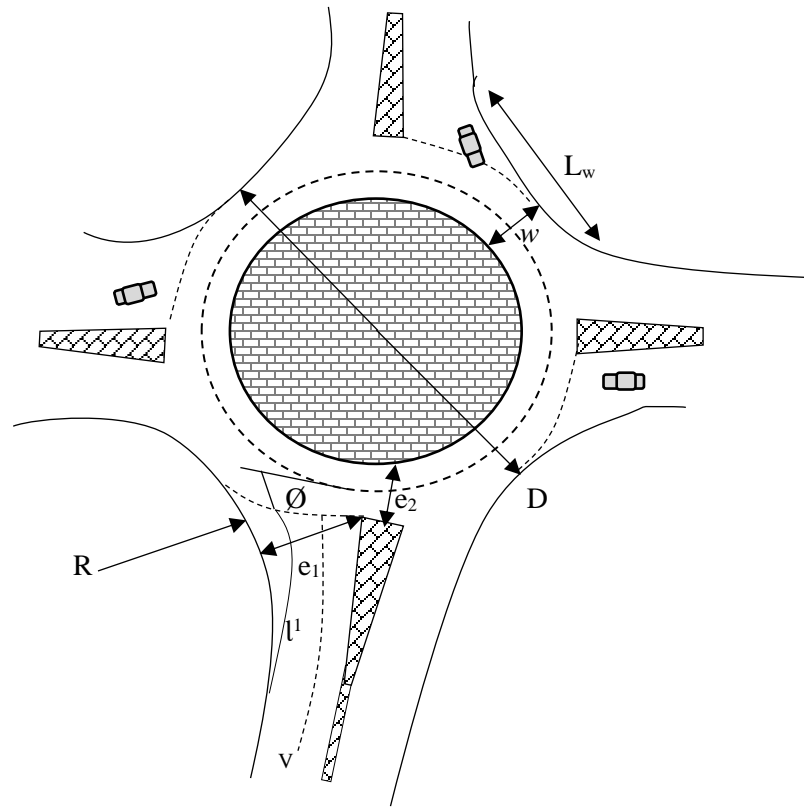


Figure 2.2: Typical Geometric Parameters

As shown above in Figure 2.2 the geometric influences include:

- Entry width (e_1), circulating entry width (e_2), hence average $e = (e_1 + e_2) / 2$
- Weaving width (w) and weaving length (L_w)
- Entry angle (\emptyset) and entry radius (R),
- Inscribed diameter (D),
- Approach width (v),
- Flare length (l^1)

Brilon argues that capacity equations should not be transferred internationally, instead each country should find a solution of its own because of the difference in driver behaviour in different countries (Brilon *et al.*, 1991). The off-side rule was introduced in 1966, consequently, weaving

movement is no longer the main determinant of capacity but rather the number of lanes, approach entry width and critical gaps in the circulating traffic stream. The off-side rule also prompted geometric changes and facilitated the introduction of smaller circles with a flared approach. Roundabout sizes and the yield rule have decreased the tendency of drivers to weave at roundabouts, according to Troutbeck (1984). Horman and Turbull found that less than one percent of drivers in the circulating stream give way to motorists entering the roundabouts when circulating flow rates are greater than entry flow rates (Horman and Turnbull, 1974). Interestingly, a uniform yield rule has never resulted in a uniform method of capacity estimation. Instead different estimation methods have emerged and broadly classified as theoretical (gap acceptance) and empirical (linear or exponential).

2.4.1 Roundabout Capacity Estimation Using Weaving – based on a design approach

Regarding weaving capacity, there are those who postulate that traffic volume per weaving section per time factor, is the determinant of practical capacity, while others prefer to use the number of lane change operations performed within the given weaving section. One thing is clear though, weaving can cause traffic stream disturbance that may lead to a bottleneck. According to HCM (1985), the weaving section will operate satisfactorily, only if traffic on the approach road is well below the practical capacities of these approaches and the weaving section has one more lane than would normally be required for the combined traffic from both approaches. When a merge area is closely followed by a diverge area, weaving segments are formed. Weaving segments require intense lane-changing manoeuvres because drivers jockey to access lanes appropriate to their desired exit points. The most critical aspect of a weaving segment is lane changing. Hence, the practical capacity of a roundabout can be estimated with a weaving-based equation 2.1 (CSS, 1972)

$$Q_p = \frac{280w(1+e/w)(1-p/3)}{1+(w/l)} \quad [2.1]$$

Where; Q_p = practical capacity;
 p = proportion of weaving vehicles

e = average entry width;

w = weaving width and l = weaving length.

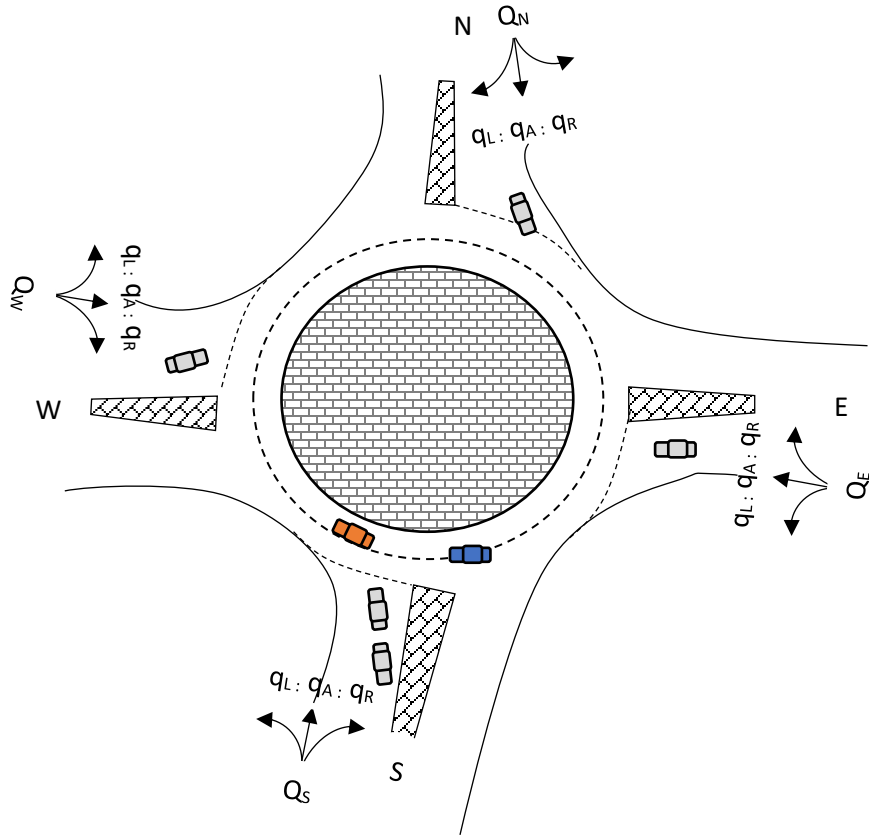


Figure 2.3: Typical yield rule movement at roundabouts

Assuming no U-turn, where A denotes ahead, L denotes left turning vehicles, and R denotes right turning vehicles, it can be seen from figure 2.3 that:

$$\text{Entry flow rate rate per arm, } Q_E = q_L + q_A + q_R \quad [2.2]$$

$$\text{Circulating flow rate rate per arm, } Q_C = q_A + 2q_R$$

[2.3]

From the definition in HCM (2010), some parameters are described as follows:

q_K^{De} denotes flow rate rate of turning vehicles at approach D

$D \in \{\text{South, East, North, West}\}$ represents either the south, east, north, or west approach

$K \in \{\text{A, L, R}\}$ represents through, left-turning, or right-turning vehicles, respectively

q^{Dc} denotes flow rate rate of the conflict stream of approach D

q_K^e denotes total flow rate rate of turning vehicles on all approaches at the roundabout

q^e denotes total flow rate rate of all approaches at the roundabout

Entry flow rate of roundabouts can be divided into three directions (straight, left and right). However, assuming the probabilities of through vehicles entering the inner and outer circulatory lanes are p_{in}^{De} and p_{out}^{De} respectively; $p_{in}^{De} + p_{out}^{De} = 1$. For the entry flow rate rate q^{De} , there will be $q_L^{De} + q_A^{De} \cdot p_{in}^{De}$ entering the inner lane, crossing the two streams. Meanwhile $q_R^e + q_A^e \cdot p_{out}^{De}$ select the outer lane and only need to pass through one stream, the relationship can be written as (provided U-turn is not allowed):

$$q_{in}^{Sc} = q_L^{Ne} + q_L^{We} + q_A^{We} \cdot p_{in}^{We} \quad [2.4]$$

$$q_{out}^{Sc} = q_A^{We} \cdot p_{out}^{We} \quad [2.5]$$

$$q_{in}^{Ec} = q_L^{We} + q_L^{Se} + q_A^{Se} \cdot p_{in}^{Se} \quad [2.6]$$

$$q_{out}^{Ec} = q_A^{Se} \cdot p_{out}^{Se} \quad [2.7]$$

$$q_{in}^{Nc} = q_L^{Se} + q_L^{Ee} + q_A^{Ee} \cdot p_{in}^{Ee} \quad [2.8]$$

$$q_{out}^{Nc} = q_A^{Ee} \cdot p_{out}^{Ee} \quad [2.9]$$

$$q_{in}^{Wc} = q_L^{Ee} + q_L^{Ne} + q_A^{Ne} \cdot p_{in}^{Ne} \quad [2.10]$$

$$q_{out}^{Wc} = q_A^{Ne} \cdot p_{out}^{Ne} \quad [2.11]$$

For the inner flow rates,

$$q_{in}^{Sc} + q_{in}^{Ec} + q_{in}^{Nc} + q_{in}^{Wc} = 2q_L^e + q_A^{We} \cdot p_{in}^{We} + q_A^{Se} \cdot p_{in}^{Se} + q_A^{Ee} \cdot p_{in}^{Ee} + q_A^{Ne} \cdot p_{in}^{Ne} \quad [2.12]$$

For the inner flow rates,

$$q_{out}^{Sc} + q_{out}^{Ec} + q_{out}^{Nc} + q_{out}^{Wc} = q_A^{We} \cdot P_{out}^{We} + q_A^{Se} \cdot P_{out}^{Se} + q_A^{Ee} \cdot p_{out}^{Ee} + q_A^{Ne} \cdot p_{out}^{Ne} \quad [2.13]$$

If $p_{in}^{We} = p_{in}^{Se} = p_{in}^{Ee} = p_{in}^{Ne} = p_{in}^e$ then equation 2.12 becomes

$$q_{in}^{Sc} + q_{in}^{Ec} + q_{in}^{Nc} + q_{in}^{Wc} = 2q_L^e + q_A^e \cdot P_{in}^e \quad [2.14]$$

If $p_{out}^{We} = p_{out}^{Se} = p_{out}^{Ee} = p_{out}^{Ne} = p_{out}^e$ then equation 2.13 becomes

$$q_{out}^{Sc} + q_{out}^{Ec} + q_{out}^{Nc} + q_{out}^{Wc} = q_A^e \cdot P_{out}^e \quad [2.15]$$

In addition, combining equations 2.11, 2.12 and 2.13, one can obtain:

$$\begin{aligned} q^{Sc} + q^{Ec} + q^{Nc} + q^{Wc} &= 2 \cdot q_L^e + q_A^{We} \cdot p_{in}^{We} + q_A^{Se} \cdot p_{in}^{Se} + q_A^{Ee} \cdot q_{in}^{Ee} + p_A^{Ne} \cdot q_{in}^{Ne} + q_A^{We} \cdot p_{out}^{We} \\ + q_A^{Se} \cdot p_{out}^{Se} + q_A^{Ee} \cdot p_{out}^{Ee} + q_A^{Ne} \cdot p_{out}^{Ne} &= 2q_L^e + q_A^e \end{aligned} \quad [2.16]$$

2.4.2 Gap-acceptance roundabout capacity estimation method

Gap acceptance method is the theoretical approach of estimating roundabout capacity. It operates on two main principles which are; the availability of gaps within the circulating or opposing traffic streams, and the usefulness of the gap by entry traffic. It depends on the driver's reaction and response time, the acceleration of the vehicle and the vehicle length (SANRAL, 2011). This approach is a probabilistic approach that takes headway, follow-up time, critical gaps and the traffic flow rate into consideration, but does not consider the geometry (AL-MADANI and Pratelli, 2014). The gap acceptance capacity estimation considers, first, the critical gap which is identified as the minimum headway between successive vehicles in the circulating approach that entering vehicles can accept to enter the circulating approach. Secondly, the follow-up time headway which is the difference in time between a departure vehicle and the immediate following vehicle at the roundabout entry if the two vehicles accept the same gap in the circulating stream under queuing conditions, and thirdly, the distribution of gaps in the circulating traffic flow rates, which depend on the Poissonian bunched vehicles or random arrivals. The follow-up time and headway change with geometry but are highly influenced by the drivers' behaviour and traffic composition. The gap acceptance models have not been able to address the inconsistency in the form of the real traffic gap acceptance because the gap acceptance of different vehicles varies compared with the fixed critical gap and follow-up headway stipulated for use in the models (Akçelik, 2003). Drivers reject the large gap and accept smaller gaps in some cases which was not addressed in these models. The vehicle in the circulating roadway gives right of way to entry vehicles, while the entry

vehicles force their way into the circulating road way at the saturation period (Mark Lenters PE, 2010, AL-MADANI and Pratelli, 2014, Russell and Rys, 2000, Ersoy and Çelikoğlu, 2014, Hagring, 1998), this makes the evaluation of critical headway a difficult task (NCHRP, 2006). Ersoy and Çelikoğlu (2014) discovered that entry capacity has a very sharp change when the accepted follow-up headway is small, this shows that the stipulated critical headway value used for the estimation of capacity may not give the accurate entry capacity in some situations. Moreover, most of the gap acceptance models are exponential models which were discovered not to describe the platooning, they predict short headways which are unrealistic and become more distorted with an increase in flow rate rate, and cannot deal realistically with a high traffic flow rate rate (Vasconcelos *et al.*, 2012). Tanner (1967) developed, and (Troutbeck, 1986, Troutbeck, 1988) refined a roundabout entry capacity as shown below in equation 2.17.

$$q_e = \frac{q_c(1-\Delta q_c)e^{q_c(t_a-\Delta)}}{1-e^{-q_c t_f}} \quad [2.17]$$

Where: q_e = Enter capacity (veh/s); and q_c = Circulating flow rate (veh/s)

t_a = Critical gap (s); t_f = Follow-up time (s);

Δ = Minimum headway in the circulating streams (1s for multilane and 2s for single lane)

Tanner's equation for the capacity of priority intersection forms the fundamental basis for the development of the gap acceptance method. Tanner's equation was adjusted by Troubeck to relate the equation to the observed field data and adopted with modifications in Australia. All the gap acceptance models are based on the distribution of gaps in the circulating flow rate and acceptance of the gap by the entering traffic (Vasconcelos *et al.*, 2012). This method relies on parameters that have different approach measurements and as to be expected these different ways do not give the same result.

2.4.3 HCM roundabout capacity estimation method

The highway capacity manual, HCM (2010) introduced exponential regression which is a mixture of empirical and theoretical methods. HCM estimates the entry capacity based on three main parameters; critical gap, follow-up time and the circulating flow rate. Critical gap which is the minimum gap within the circulating traffic that is safe for an entry vehicle to be willing to accept for merging with circulating traffic, the follow-up headway, and the circulating flow rate. The HCM (2010) capacity model was developed as an exponential regression model with parameter estimates based on gap acceptance theory with inherent weaknesses. For example, choosing a negative exponential equation to define the capacity of a roundabout entry, particularly one that is gap-acceptance based. The equation becomes nearly asymptotic to the x-axis making it unreliable to model small entry traffic flow rate when circulating traffic volume is high. It is easier to record the direct measurement of entry and circulating flow rates and more difficult to collect gap data at a roundabout. The absence of a Y-intercept means that the geometric influence of a roundabout is unexplained. A significant advantage of empirical model is sensitivity to roundabout geometric design. Without the capability to predict different capacities for a variety of configurations or number-of-lane-based designs, the designer runs the risk of overdesigning, decreasing safety, and increasing cost (Mark Lenters PE, 2010). Consequently, geometrically-sensitive design methods are sought after by clients, agencies, and owners to achieve required capacity targets while minimising right-of-way impacts, avoiding high construction costs, and balancing the safety of all users (Mark Lenters PE, 2010). Nevertheless, the HCM 2010 model equation is shown below;

$$Q_T = Ae^{(-\beta q_c)} \quad [2.18]$$

Where; Q_T denotes theoretical capacity and q_c = circulating flow rate

$$A = \frac{3600}{t_f}; \beta = \frac{t_c - 0.5t_f}{3600}; t_f = 3.19s \quad \text{and} \quad t_c = 4.11s$$

It can be rewritten as (multilane roundabouts);

$$Q_T = 1130e^{(-0.0007)Q_c} \quad [2.19]$$

2.4.4 Empirical roundabout capacity estimation method

Many countries including the UK, Germany, France, and Switzerland have been using the empirical method of estimating roundabout capacity for many years. The empirical method relies on linear or exponential regressions. It is postulated that direct empirical linear and exponential regression without predetermined theoretical values can be explored when estimating roundabout entry capacity. After all, linear regression is a modelling approach whereby the relationship between a scalar dependent variable and one or more independent variables is explained with a best fit line. Exponential regression is a modelling approach in which a constant change in the independent variable gives the same proportional change in the dependent variable. The empirical linear regression method is a deterministic approach based on the roundabout geometry, entry and the circulating flow rate (AL-MADANI and Pratelli, 2014). In any case, the UK empirical linear model is simple and widely used. The UK model is being used by several road authorities in the US for the estimation of roundabout capacity. According to Mark Lenters PE (2010) it gives the most accurate results for the calibration of the specific site because it uses the collected data from the site. It does not work on the dominant lane and this makes it possible to be used for a multilane roundabout with more than one saturation lane (Mark Lenters PE, 2010). Flaring is an important part of the roundabout geometry that has an influence on entry capacity, this aspect of geometry is not covered by the gap acceptance models and the other empirical models. The UK model has a correction factor which makes it suitable for a roundabout with different geometry and it has a wide application in different countries. It is more useful because of its ability to relate geometry to entry capacity (Mark Lenters PE, 2010), although this approach of using regression requires a large amount of data (Russell and Rys, 2000). The determination of entry capacity from the UK model is based on research carried out by Kimber (1980) where roundabout entry capacity is presented as:

$$Q_e = k(F - f_c Q_c) \quad [2.20]$$

$$\text{Where; } k = 1.151 - 0.00347\phi - 0.978/r \quad [2.21]$$

Q_e = Entry Capacity (pce/h); and Q_c = Circulating Flow rate (pce/h),

$$F = 329e_1 + 35e_2 + 2.4D - 135 \quad [2.22]$$

$$f_c = 0.29 + 0.116e_1 = \frac{F}{Q_c} \quad [2.23]$$

k denotes the correction factor for entry angle (φ) and entry radius (r)

e_1 = entry width (m); e_2 = circulating width (m), D is roundabout size factor

φ denotes entry angle ($^\circ$), r is the entry radius (m), f_c is the slope gradient and F is the y intercept.

Philbrick (1977) concludes that the optimal equation for the y-intercept involves only the entry width (e_1) and the radius (r_1) and can be determined by equation 2.24

$$F = 233e_1 \left(1.5 - \frac{1}{\sqrt{r_1}}\right) - 255 \quad [2.24]$$

Also, that the optimal equation for the slope (f_c) involves only two variables; entry width and weaving width (w) as shown in equation 2.25.

$$f_c = 0.0049 (2e_1 - w) + 0.282 \quad [2.25]$$

In any case, the key entry capacity parameter is the y-intercept (F). It contains the major capacity influences of entry width, flare length and approach width. If the y-intercept can be adjusted, then the slope of the linear equation that also contains the major capacity geometry relationships, can be preserved. Çalişkanelli *et al.* (2009) postulates that regression analysis and gap acceptance-based models are the most used estimation methods for predicting traffic circle capacity. According to Hale (2015) methods of indirect capacity measurement require various model adjustments to prevent capacities from increasing unrealistically in response to increasing congestion levels. Gazzarri *et al.* (2013) suggest that the choice of roundabout capacity model depends on driving behaviour and the roundabout design, and that countries with similar roundabout driving habits can use the same model applications. They assert that countries with left-hand traffic could not use the same model application as countries with a right-hand traffic rule. Roundabouts tend to operate well in Australia, which has a default yield-to-right rule, because yielding to clockwise traffic is natural to drivers. However, experience in deployment of modern roundabouts in the USA has been mixed, with drivers approaching on the right side of the splitter

island required to yield to anti-clockwise traffic in contrast to the default yield-to-right rule. In South Africa, entry vehicles yield to circulating vehicles which drive in a clockwise direction. This is contrary to USA but similar to United Kingdom mode of roundabout operation. Consequently, South Africa can use the United Kingdom modelling approach because they are both countries with left-hand traffic. In any case, it is postulated that direct empirical linear and exponential regression without predetermined theoretical values can be explored when estimating roundabout capacity in this thesis.

2.4.5 Traffic Delays and Queues Concepts

Traffic delay and queue length are also key parameters used to assess the effectiveness of roundabout performance. Delay is an important parameter that is used in the performance evaluation of intersections. It is influenced by many variables and hence its determination is somewhat complex. HCM prescribes delay as the primary measure of effectiveness for roundabouts and intersections, with the level of service determined from the delay estimate. Delay in a roundabout can be defined as the time spent on traversing the roundabout in excess of traffic-free flow rate at the roundabout and it is the primary service delivery for the roundabout (Rodegerdts, 2010). The Highway Capacity Manual only includes control delay, which is the delay attributable to the control device. Control delay is the time which a driver spends queuing and then waiting for an acceptable gap in the circulating flow rate while at the front of the queue. Control delay can also be defined as the time a driver takes to decelerate into a queue, stay in the queue, while at the front of the queue wait for an acceptable gap and accelerate out of the queue (Rodegerdts, 2010, HCM, 2000). Queuing occurs when the entry vehicles are waiting for an appropriate and safe gap in the circulating traffic (Sofia *et al.*, 2012). Control delay comprises of both the geometric delay and the stop line delay (Yap *et al.*, 2013, Akçelik, 2005). The mode of operation of a roundabout does not necessarily make the entry vehicle to have total stop before entering the roundabout, but rather yield to the circulating vehicles, look for a safe gap within the circulating vehicle, accept the gap and enter the roundabout. The total stop of a vehicle at entry, unlike at a signalized intersection, is not always necessary which enables it to have a better service delivery and higher entry capacity than a signalized intersection (Kakooza *et al.*, 2005, Sisiopiku and Oh, 2001). Delays still occur at the roundabout, though they might be reduced compared to other forms of at-grade intersections. Collins (2008) states that if the vehicular delay is to be an

objective of any intersection, then the roundabout should be given a consideration because it will significantly reduce the total vehicle delay. HCM control delay is given as:

$$d = \frac{3600}{c} + 900T \left(\left[\frac{v}{c} - 1 \right] + \sqrt{\left(\frac{v}{c} - 1 \right)^2 + \frac{\left[\frac{3600}{c} \right] \frac{v}{c}}{450T}} \right) + 5 \quad [2.26]$$

Most stochastic delay model equations are derived from Tanner (1962) average delay equation:

$$d_t = \frac{1/2 * \frac{E(y^2)}{Y} + q_e Y e^{-\beta_2 q_c} * \frac{(e^{\beta_2 q_c} - \beta_2 q_c - 1)}{q_c}}{1 - q_e Y (1 - e^{-\beta_2 q_c})} \quad [2.27]$$

$$\text{With, } Y = \frac{e^{q_c} (\tau - \beta_1)}{q_c (1 - \beta_1 q_c)} \quad [2.28]$$

$$E(y^2) = \frac{2Y}{q_c} \left\{ e^{q_c(\tau - \beta_1)} - \tau * q_c (1 - \beta_1 q_c) + \beta_1 q_c - \beta_1^2 q_c^2 - \frac{1}{2} * \frac{\beta_1^2 q_c^2}{1 - \beta_1 q_c} \right\} \quad [2.29]$$

Where: d_t = average delay on approach (s/veh)

q_{ae} = arrival rate on entering approach (veh/s)

q_c = arrival rate on circulating flow rate (veh/s)

τ – critical gap (s); β_1 – headway (s); β_2 – follow-up time (s)

Assuming random arrivals and no queues, Tanner shows that the average delay for isolated minor road vehicles is:

$$d_m = \frac{e^{\lambda(\alpha - \Delta)}}{\varphi * v_c} - \alpha - \frac{1}{\lambda} + \frac{\lambda \Delta^2 - 2\Delta + 2\Delta\varphi}{2(\lambda\Delta + \varphi)} \quad [2.30]$$

Now, if $\varphi = 1$ and the minimum gap in the circulating traffic is set to zero, then:

$$d_m = \frac{e^{\lambda(\alpha - \Delta)}}{\varphi * v_c} - \alpha - \frac{1}{\lambda} - \frac{\beta_2^2 v_c}{2} \quad [2.31]$$

Where: $\varphi = 0.75(1 - \Delta * v_c)$ and $= \frac{\varphi * v_c}{1 - \Delta * v_c}$; $\Delta = 2s$

Troutbeck (1989) recommends that when estimating average delay (d_t), a steady state model equation 2.32 be used:

$$d_t = d_m * \left\{ 1 + \frac{ex}{1-x} \right\} \quad [2.32]$$

Where;

x denotes the degree of saturation

e is a form factor which can be set to 1 or 0, if no other value is available

Akcelik and Chung (1994b) follows up Troutbeck's model equation with an allowance for variation over time, as seen in the equation shown below:

$$d = d_m + 900T \left(\left[\frac{v}{c} - 1 \right] + \sqrt{\left(\frac{v}{c} - 1 \right)^2 + \frac{[8k] \frac{v}{c}}{c * T}} \right) \quad [2.33]$$

Where; $k = \frac{c * d_m}{3600}$; $d_m = \frac{e^{\lambda(\alpha - \Delta)}}{\varphi * v_c} - \alpha - \frac{1}{\lambda} + \frac{\lambda \Delta^2 - 2\Delta + 2\Delta\varphi}{2(\lambda\Delta + \varphi)}$

$$\varphi = 0.75(1 - \Delta * v_c) \text{ And } \lambda = \frac{\varphi * v_c}{1 - \Delta * v_c}; \Delta = 2s$$

$$c = \frac{3600 \varphi * v_c * e^{-\lambda(\alpha - \Delta)}}{1 - e^{-\lambda\beta}};$$

$$\beta = 2.819 - 3.94 * 10^{-4} * v_c; \text{ and } \alpha = \{1.641 - 3.137 * 10^{-4}\}\beta$$

Figure 2.4 shows the two curves obtained by the two queue length model functions of which the first term is deterministic (Equation 2.34) and the second term is in a steady state condition (Equation 2.35):

$$L = (\rho - 1)q_e t + L_0 \quad [2.34]$$

$$L = \rho + C\rho^2(\rho - 1) \quad [2.35]$$

$$L = \frac{\rho}{(1-\rho)} \text{ For } C = 1$$

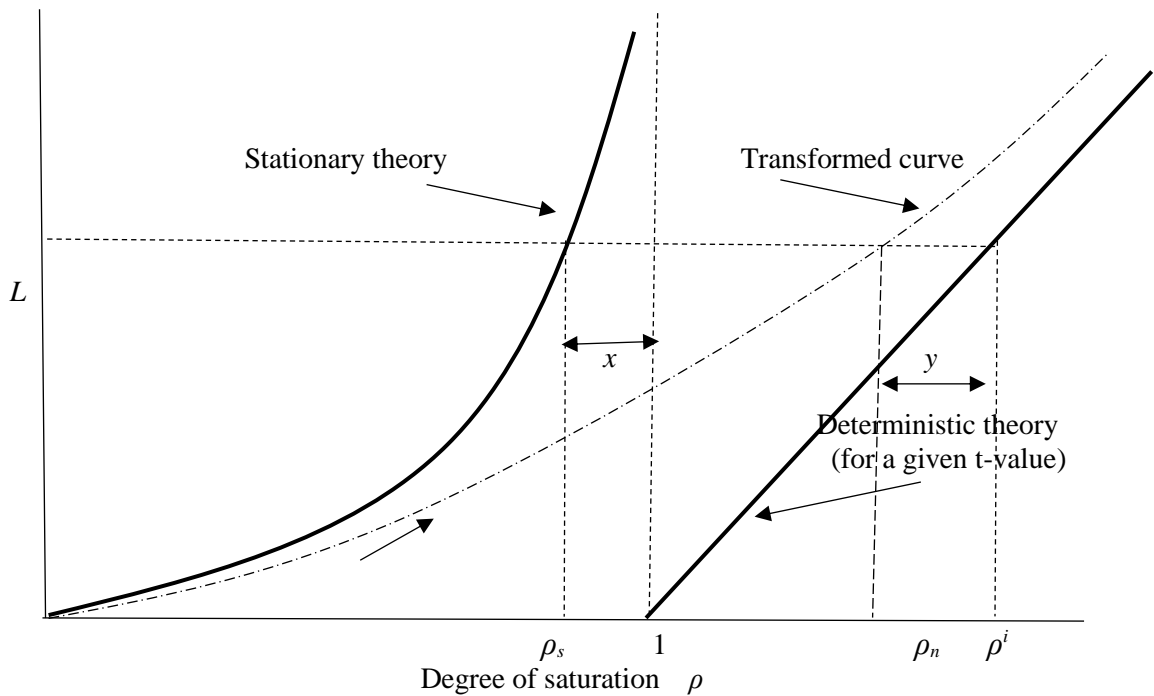
Where:

L_0 denotes initial queue length; and t denotes any time interval;

ρ denotes traffic intensity (x); and q_a denotes demand flow rate; q_m denotes capacity;

C denotes constant to describe arrival and service patterns.

For regular arrivals, $C=0$; and for random arrivals, $C=1$



Source: Kimber and Hollis (1979)

Figure 2.4: Coordinate transformation for average delay estimation

Generally, the basic equation for traffic delay at a roundabout is:

$$d = d_1 + d_2 + d_3 \quad [2.36]$$

Where: d_1 = The yield line delay or the follow-up time (s)

d_2 = Queueing delay (s) and d_3 = geometric delay (s)

According to Guo and Wang, delay in a roundabout is of two main categories, namely the control and the geometric delay (Guo and Wang, 2011). The geometric delay is the reduction in speed due to the effect of the roundabout geometry in the course of traversing the roundabout (HCM, 2000). Geometric delay can further be defined as any delay experienced when a vehicle is traversing the roundabout in the absence of any other vehicle at the roundabout if the driver could identify that he is traversing the roundabout in isolation (Akcelik, 2009, Sofia et al., 2012, Kimber et al., 1986). The value is usually small for a small roundabout, but for large diameter roundabouts it could be significant. The value is usually high for a stopping vehicle because of the time it takes to accelerate to the design speed of the roundabout (Rodegerdts, 2010). This delay is always present

at the roundabout whether there is the presence of a vehicle or not. Geometry is one of the major factors that have an influence on delay (Al-Omari et al., 2004). The geometric delay excludes the queueing time at the roundabout entry, it could be more than the delay at a congestion with the exception of when the traffic approaches capacity (Kimber *et al.*, 1986). All other elements being equal, delay (and queue) would increase under rainy conditions due to more conservative car following and gap acceptance behaviour. While this seems to be apparent in Figures 2.5 and 2.6. The delay (queue)- volume capacity curves rise rapidly as the volume capacity increases at some points as pointed out in the relationship of delay and volume capacity ratio in HCM (2010). The interest in this study is not to build a different relationship for delay and volume capacity ratio but to determine the effect of rainfall and make some modifications if need be.

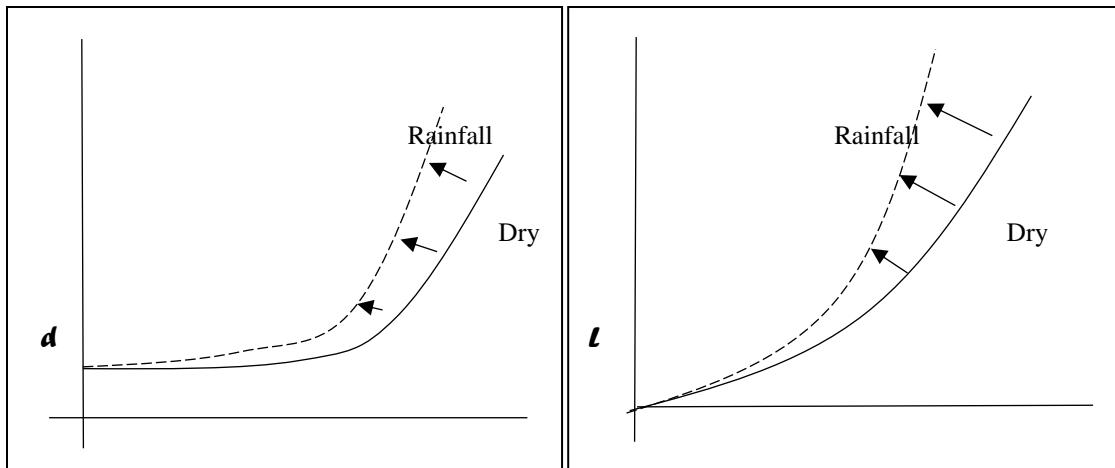


Figure 2.5: Delay vs. volume capacity ratio

Figure 2.6: Queue length vs. volume capacity ratio

2.5 Qualitative Assessment of Roundabout Performance

A road network is made up of segments (links) and intersections (junctions), each with its own characteristics and uniqueness. For the purpose of designing a roadway, capacity is a key parameter and also a key performance indicator when assessing the roadway. Quality of service has been an elusive, confusing and ambiguous roundabout assessment parameter because it is related to capacity and often confused with level of service. Quality of service and level of service (LOS) have been used interchangeably in many studies and the Highway Capacity Manual (HCM). Level of service, according to the HCM (2010), is a useful measure of effectiveness of a

roadway, unlike capacity which is a quantitative measure. The quality of service is defined by the Florida Department of Transportation (2013) as “*users’ perception of the performance of transportation facility*”. This definition limits the assessment of the quality of service to the road users’ perception. LOS is important because it points the road providers in the right policy and management direction. However, effectiveness is defined as the capability of producing a desired result; bearing in mind that effectiveness is measured by road providers, one can assume that the desired result would be capacity utilization of roundabout facilities. The HCM (2010) defines LOS for roundabouts as a function of the average control delay which is not the same as capacity utilization. Previous studies have been content with measuring travel delay as a substitute for quality of service, when in fact travel delay is not the main concern of road providers. The primary concern of road providers is capacity utilization. Why are previous studies content with defining quality of service loosely and depicting quality of service as level of service when they are in fact different? The answer probably lies with the assumption that the term quality of service may have a different meaning to different people. The road providers and road users are the principal parties in quality of service assessment, hence quality of service must correctly depict their perceptions. The service quality from road providers is tied to road design specifications. Whereas, the service perceived by the road users is a statement of the satisfaction or dissatisfaction experienced by the road users. It is usually expressed as travel time for road segments and travel time delay for intersections.

Nevertheless, the principal operating characteristic of the traffic control device is degree of saturation also known as volume/capacity ratio. Average delay and (percentile) queue lengths can be estimated from degree of saturation, as can the amount of degree of saturation available in reserve. All these measures are relevant to the traffic engineer as service provider, while average delay and queue length are directly experienced by road users. Most commercial traffic analysis software (e.g. SIDRA) includes tables of all of these outputs to provide the traffic engineer with a complete perspective of the operation of the traffic control device.

Therefore, if the road provider is interested in measuring the effectiveness of roundabout performance, reserve capacity is recommended. If the road provider is interested in measuring road users’ level of satisfaction or otherwise, then average delay at roundabout is the parameter required, it can be argued.

Many factors influence the quality of service (Hostovsky *et al.*, 2004). The quality of service is influenced by factors related to traffic, pavement conditions, environment and ambient conditions (Corporation *et al.*, 2003, Flannery *et al.*, 2006). These factors are categorised under the operational, geometric and other factors (Roess and Prassas, 2014). The operational factors include the delay experienced within the travel time, the congestion and queue length. The geometric factors include the visibility, clear signs and pavement markings, design related factors such as the lane width, entry width, entry radius, entry angle, flare length, inscribed circle diameter and other geometric elements in the roundabout. These features have been reported to have an effect on the capacity of roundabouts (Kimber, 1980). Since the main concern of the road user is the delay, safety and comfort when driving, it implies that the shorter it takes to traverse the roundabout with good comfort, the better the quality of service from the users' point of view. The mentioned geometric features of a roundabout have been proven to have an effect on the flow rate of vehicles at the roundabout. Kimber (1980) shows that a small increase in the entry radius reduces the delay at the roundabout, while the increase in entry angle minimizes the occurrence of an accident at a roundabout. The increase in the inscribed circle diameter improves the traffic flow rate at the circulating roadway. Hence, it provides a more available safe gap for the entry vehicle and reduces delay.

The quality of service is significant in the roundabout because it is a factor that is considered throughout the life of the roundabout. This is from the inception where the need for the roundabout is considered, the expectation of the expected users is taken into consideration during the design and construction process as well. After construction, the quality of service is then measured to ascertain the agreement in service performance expectations by the users and the **provider**. The quality of service is also useful in the determination of the need for roundabout maintenance. A major consideration in the roundabout's quality of service includes the safety and travelling comfort i.e. the reduction in inconvenience and increase in the travel reliability as well as service to the users.

2.5.1 Roundabout Level of Service Assessment Methods

Although level of service has been used interchangeably with quality of service in many studies, in this section level of service assessment methods are discussed. South Africa does not have a

specific parameter used in the assessment of the road traffic performance. South Africa relies on the Highway Capacity Manual (HCM) for level of service assessment. HCM (2010) incorporates TRB's National Cooperative Highway Research Programme (NCHRP) Report's 572 methodologies (with enhancements and extensions) of lane-by-lane analysis of multilane roundabouts. Note that HCM uses the concept of level of service (LOS) as a qualitative measure to describe operational conditions of vehicular traffic at roundabout facilities, "*based on service measures such as control delay, freedom to manoeuvre, traffic interruptions, comfort and convenience.*" The NCHRP Report 572 states that, "*perceived differences in driver behaviour raises questions about how appropriate some international research and practices are for the United States*". On operational performance, the report concludes that,

"...currently, drivers in the United States appear to use roundabouts less efficiently than models suggest is the case in other countries around the world. In addition, geometry in the aggregate sense (number of lanes) has a clear effect on the capacity of a roundabout entry; however, the fine details of geometric design (lane width, for example) appear to be secondary and less significant than variations in driver behaviour at a given site and between sites"

Furthermore, the report proposes exponential models of capacity for single-lane and two-lane roundabouts and recommends that level of service (LOS) **criteria be the same as** those currently used for unsignalised intersections. The NCHRP Report 572 recommends that "*because driver behaviour appears to be the largest variable affecting roundabout performance, calibration of the models to account for local driver behaviour and changes in driver experience over time is highly recommended to produce accurate capacity estimates*" and also states that "*these models have been incorporated into an initial draft procedure for the Highway Capacity Manual (2010), which the TRB Committee on Highway Capacity and Quality of Service will continue to revise until its eventual adoption*". It appears that the HCM LOS is still a work in progress. In any case, the HCM (2010) level of service methodology has twelve steps as listed below:

1. Convert movement demand volumes to flow rate rates,
2. Adjust flow rate rates for heavy vehicles,
3. Determine circulating and exiting flow rate rates,
4. Determine entry flow rate rates by lane,
5. Determine capacity of each entry lane and bypass lane in passenger car equivalents,
6. Determine pedestrian impedance to vehicles,
7. Convert lane flow rate rates and capacities into vehicles per hour,
8. Compute volume to capacity ratio (v/c) ratio for each lane,
9. Compute average control delay for each lane with equation 2.37,

$$d = \frac{3600}{c} + 900T \left(\left[\frac{v}{c} - 1 \right] + \sqrt{\left(\frac{v}{c} - 1 \right)^2 + \frac{3600}{450T} \frac{v}{c}} \right) + 5 \quad [2.37]$$

Where: $c = A * \exp(-\beta * v_c)$

d = delays (s); C = entry capacity; v = demand flow rate; and v_c = circulating flow rate

$$A = \frac{3600}{t_f} :$$

$$\beta = \frac{t_c - 0.5t_f}{3600} = 0.0007 \text{ for multilane roundabouts}$$

$T = 0.25$ for 15min; 0.1667 for 10min; 0.08333 for 5 min and 1 for 1-hr traffic analysis

10. Determine LOS for each lane on each approach,

The HCM uses six divisions, A to F, for levels of service for unsignalised intersections. The delay under this condition is mainly due to control delay. HCM does not give specific criteria for assessment of roundabout service delivery but a combined criterion for unsignalised intersections and roundabouts. The criteria table for assessment of unsignalised intersections which is applicable to roundabouts in the HCM is presented in Table 2.2. The HCM recognises the need for further research into roundabouts' quality of service.

Table 2.2: HCM Level of service for unsignalised intersections

Level of service $v/c \leq 1$	Average control delay (s/vehicle)	Level of service $v/c > 1$
A	0 - 10	F
B	> 10 - 15	F
C	> 15 - 25	F
D	> 25 - 35	F
E	> 35 - 50	F
F	> 50	F

Source: HCM 2010

11. Compute average control delay and LOS for each approach and the entire roundabout,
12. Compute 95th percentile queues for each lane,

$$Q_{95} = 900T \left(\left[\frac{v}{c} - 1 \right] + \sqrt{\left(\frac{v}{c} - 1 \right)^2 + \frac{\left[\frac{3600}{c} \right] \frac{v}{c}}{450T}} \right) \frac{c}{3600} \quad [2.38]$$

Where: Q_{95} is queue length (veh).

The Australian level of service method is based on the SIDRA intersection model. The performance assessment categorizes level of service into six categories A to F. The LOS for roundabouts considers both the control delay and degree of saturation as shown below in table 2.3. The degree of saturation threshold is 0.85, hence the degree of saturation within the threshold is classified as A, B, C or D, depending on the average delay. Both SIDRA and HCM (2010) recognize that delay is a key indicator of effectiveness and that the degree of saturation is a key management tool. **However, reserve capacity is not mentioned in both HCM (2010) and SIDRA bearing in mind that reserve capacity and degree of saturation are dependent variable.** It can be argued that the degree of saturation, reserve capacity, delay and queue length are the key parameters that can be used to assess the quality of service. Each measure provides a unique perspective on the quality of service of roundabouts under observations.

Table 2.3: SIDRA Level of service for roundabout

Level of service	Average control delay (s/vehicle)	Degree of Saturation (x)
A	$d < 10$	
B	$10 < d \leq 20$	$0 < x \leq 0.85$
C	$20 < d \leq 35$	
D	$35 < d \leq 50$	$0 < x \leq 0.85$
	$0 < d \leq 50$	$0.85 < x \leq 0.95$
E	$50 < d \leq 70$	$0 < x \leq 0.95$
	$0 < d \leq 70$	$0.95 < x \leq 1.0$
F	$70 < d$	$1.0 < x$

Source: Rahmi Akelic, 2009

2.6 Novel Quality of Service Assessment Concept

The proposed quality of service assessment method presented in this thesis uses delay as a proxy for road users' perception of service delivery and reserve capacity as a measure of concern for road and traffic management service. The thesis recognizes that quality of service can either be structural or functional. Structural quality of service deals with highway infrastructure issues. Whereas, functional quality of service addresses traffic flow rate and control issues. The proposed functional quality of service (FQS) in this thesis is divided into two phases. Phase one focuses on criteria table development bearing in mind that only peak performance traffic data can be used to construct a criteria table. Phase two deals with FQS determination given prevailing traffic, road and ambient conditions as shown below in figure 2.7. When developing a criteria table, the following parameters are important; entry capacity, circulating capacity, reserved capacity, demand flow rate, volume to capacity ratio (degree of saturation) and control delay. It is important that a criteria table be developed before computing FQS. The FQS criteria table presented in this thesis is not going to be different from the HCM (2010) or SIDRA design wise and number of classes, but the contents therein will not be the same. The functional quality of service criteria table is summarized in Figure 2.7.

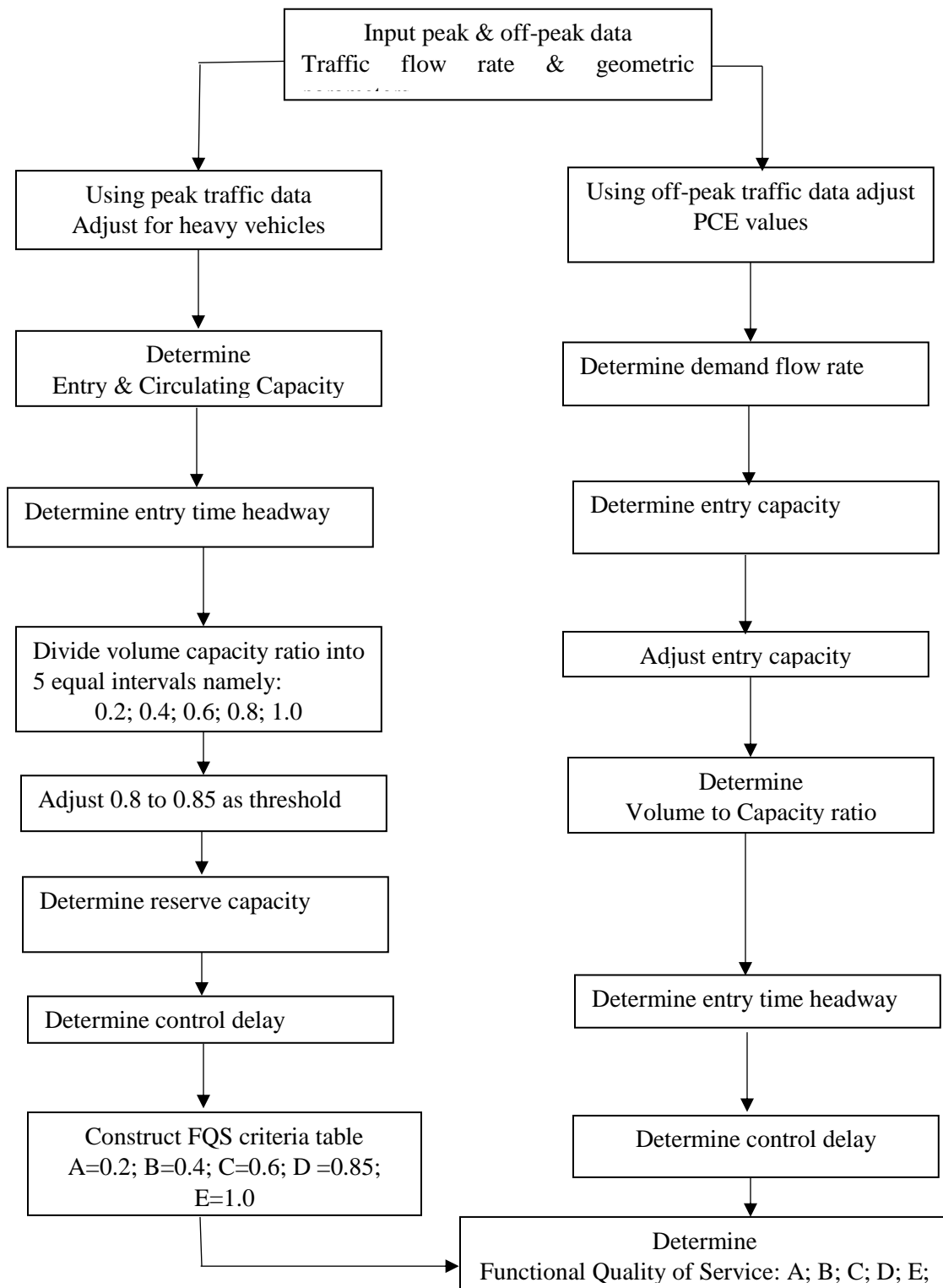


Figure 2.7: Functional Quality of Assessment Flow rate chart.

Phase 1 Step 1: Input of peak traffic flow rate and geometric data. Note the percentage of heavy vehicles

Phase 1, Step 2: Adjust for heavy vehicles using HCM (2010) equation below:

$$V_i = V_d * \left(\frac{1}{1 + P_T [E_T - 1]} \right) \quad [2.39]$$

Where:

f_{hv} is factor adjustment for heavy vehicles

E_T is PCE value for heavy vehicles

P_T is the proportion of demand volume that consists of heavy vehicles

V_d demand flow rate (veh/h) and V_i is demand flow rate (pcu/h)

Phase 1, Step 3: Model entry and circulating flow rate rates using the **linear** regression equation.

Note that the key parameter F is the y-intercept that allows the influence of geometrical parameters such as entry width, flare length and approach width to be determined. **By adjusting F, the slope of the linear equation** that also contains the major capacity, geometrical relationships can be preserved.

Phase 1, Step 4: Test model equation for statistical fitness.

Note that the coefficient of determinant must be > 0.5 which indicates that the model's equations are reliable, the t-test result must be > 2.0 at 95 percent level of confidence which shows that the parameters used are significant. The F test values > 4 , which indicates that the model's equation did not occur by chance.

Phase 1, Step 5: Determine entry and circulating capacity.

Note that entry and circulating capacity can only be estimated with the empirical linear method. Weaving and gap acceptance methods can be used to estimate practical and theoretical entry capacity respectively.

Phase 1, Step 6: Compute correction factor (k) for the capacity model equation.

Use the model equation 2.21 to estimate the correction factor, bearing in mind that entry angle and entry radius have limitations as expressed by Kimber (1980).

Phase 1, Step 7: Adjust computed model equation with the correction factor in step 6 and test the entry capacity for validation using the HCM (2010) model equation.

Phase 1, Step 8: Determine volume to adjusted capacity ratio.

From the definitions of volume to capacity ratio (v/c) and capacity, it is clear that volume capacity ratio depends on the flow rate at the entry and circulating roadway because entry capacity is a function of the vehicles at the circulating roadway (Akçelik, 2003). Entry capacity is the maximum sustainable flow rate that an entry can have under the prevailing conditions and is dependent on the magnitude of the circulating flow rate. The volume to capacity ratio (v/c) is a measure of traffic demand compared to its total capacity. It is an accepted practice to use $v/c = 0.85$ as the threshold of degree of saturation for roundabouts (HCM, 2010). Values over 85 percent are typically regarded as suffering from traffic congestion, with vehicle queues beginning to form. The volume capacity ratio (v/c) is the ratio of traffic flow rate at the entry of a roundabout to the capacity of the same entry and is one of the parameters used in measuring the performance of a roundabout (Robinson *et al.*, 2000). In many studies it has been suggested that roundabouts should not operate more than 0.85 degree of saturation or volume to capacity ratio (Robinson and Rodegerdts, 2000, Pande and Wolshon, 2016), which implies that no approach of the roundabout should have a vehicular traffic flow rate more than 85 percent of the capacity. HCM (2010) specifies 0.9 as the degree of saturation threshold.

Phase 1, Step 9: Determine reserve capacity.

Reserve capacity is a measure of sufficiency. Reserve capacity values alert road providers to areas where traffic mitigation measures should be considered for deployment. Once capacity is reached congestion sets into the traffic stream. Reserve capacity is a measure of the overall of the physical design features of the intersection. Not to be confused with practical reserve capacity, which is commonly used to measure available spare capacity at a traffic signal junction, reserve capacity

indicates whether the entry flow rate is greater than the capacity of the facility. Reserve capacity is one of the parameters for the measurement of unsignalised intersection performance (Salter, 1989, Wong and Yang, 1997a). The concept of reserve capacity has long been used as a useful measure of the operational performance of individual signal-controlled junctions. Reserve capacity is a factor that could be used to determine the total number of vehicles that could enter the roundabout before reaching the saturation condition. This could also be used when traffic is to be diverted during construction, accidents and road closures. The traffic flow rate at the road of diversion could be compared to the reserved capacity of the roundabout to determine if the roundabout can accommodate the diverted traffic comfortably. Reserve capacity can also be used in the determination of the effect of the traffic on the roundabout at a glance and to determine how well a roundabout can handle traffic fluctuation and future traffic growth. Reserve capacity is defined in the HCM as the unused capacity of movement, or the difference between the actual capacity for movement and the flow rate for the movement. The equation for the estimation of reserve capacity is shown in Equation 2.40.

$$Q_R = \left(\frac{Q_e - q_d}{Q_e} \right) * 100 \quad [2.40]$$

Where;

Q_R denotes normalized reserve capacity, q_d denotes demand flow rate and Q_e is entry capacity,

The HCM establishes a level of service for each range of reserve capacity as; A \pm 400vph; B 300-390vph; C 200-299vph; D 100-199vph; E 0-99vph and F < 0 (Kyte *et al.*, 1992).

Phase 2, Step 1: Determine entry time headway (t_h) with equation 2.41 below:

$$F = \frac{3600}{t_h} \quad [2.41]$$

Where F is the intercept of linear regression equation, $Q = F - fcQc$.

Phase 2, Step 2: Determine delay and queue length using the delay equations shown below:

$$d = \frac{3600}{c} + 900T \left(\left[\frac{v}{c} - 1 \right] + \sqrt{\left(\frac{v}{c} - 1 \right)^2 + \frac{\left[\frac{3600}{c} \right] \frac{v}{c}}{450T}} \right) + 5 \quad [2.42]$$

$$L = d * q_d \quad [2.43]$$

Where; d denotes delay (s); $T = 0.25$; v/c denotes volume to capacity ratio;

L denotes queue length (veh); q_d denotes demand flow rate (veh/s)

Phase 2, Step 3: Determine criteria table.

The criteria table for this study will be developed with the users' and providers' perspective for double-lane roundabouts. The existing methods of assessment consider control delay and/or degree of saturation in the assessment of the roundabout quality of service, but none consider the reserve capacity (Q_R). Note that the reserved capacity is not the practical reserved capacity, which is the ratio at which the capacity can be increased before reaching the capacity but is the ratio of the capacity reserved to the entry capacity. The delay model equation 2.42 will be used in the estimation of the control delay. Reserved capacity is an important parameter because it gives the roundabout provider the extent to which vehicles can still be accommodated by the roundabout. The peak period traffic data will be used in developing the criteria table. The entry capacity will be estimated, the time headway will be calculated. The time headway is the yield line delay which will be used in estimating the delay at different v/c . The initial v/c will be estimated at $v/c=0$, and 1 to know the boundary limit for the assessment criteria. The queue length will be estimated and the increase in the queue length will be used in forming the classes of the FQS. The delay, Q_R and v/c at $v/c= 1$ will form FQS E, the threshold will be FQS D and other FQS classes will be formed based on the queue length (L). These criteria for roundabout FQS assessment are not different from the HCM and [Signalised and Unsignalised intersection Design and Research Aid \(SIDRA\)](#) criteria for assessment of roundabout service delivery, but the content is different.

Phase 2, Step 4: Determine prevailing quality of service with table 2.4.

Table 2.4 shows the proposed quality of service classifications and parameters. The functional quality of service (FQS) for roundabouts will be divided into six classes from FQS A to FQS F. Note that FQS A is the best class and FQS F is the worst. FQS A - This is a situation where there is a free flow rate of vehicles into the roundabout and it can be taken as the situation where the

circulating flow rate is so small and the effect on entry flow rate is insignificant. FQS B - This is a situation with little interruption on entry vehicles by circulating flow rate and minimal delay at the roadway. FQS C - This is a situation when there is little, but bearable delay at the roundabout. FQS D - This is a threshold class that alerts that the roundabout is operating close to the capacity. FQS E - This is a situation where the roundabout is operating at capacity, and any little increase in the demand flow rate may cause an excessive delay, long queue and traffic jams. FQS F - At this class, the roundabout is operating above the roundabout capacity. The delay might be lengthy, and long queues will start to form and there might be traffic jams at the entry.

Table 2.4: Proposed (FQS) criteria table

FQS	Q_R	v/c (χ)	d (s)	L
A	$Q_R \leq A$	$\chi \leq A$	$d \leq A$	$L = 0$
B	$A \leq Q_R < B$	$A < \chi \leq B$	$A \geq d \leq B$	$A < L \leq B$
C	$B \leq Q_R < C$	$B < \chi \leq C$	$B < d \leq C$	$B < L \leq C$
D	$C \leq Q_R < \tau$	$C < \chi \leq \tau$	$C < d \leq \tau$	$C < L \leq \tau$
E	$\tau \leq Q_R \leq 0$	$\tau < \chi \leq 1$	$\tau < d \leq E$	$\tau < L \leq E$
F	$Q_R < 0$	$\chi > 1$	$d > E$	$L > E$

Note: Q_R reserved capacity, v/c is volume to capacity ratio, d is delay, L is queue length, τ denotes threshold

2.7 Impact of Rainfall on Quality of Service Delivery

Weather is an ambient condition that can influence quality of service at the roundabout. The weather, which has an effect on the quality of service, includes windstorms, fog, snowfall, smog, extremely high temperatures, hail, dust storms, and tornadoes (Agarwal et al., 2005, Koetse and Rietveld, 2009, Andrey and Olley, 1990, Bartlett et al., 2013). These weather conditions vary from country to country and the effect cannot be controlled like other factors. Rainfall is the major weather condition that affects the traffic in all the nine provinces of South Africa. Rainfall records show that rain falls throughout the year, but the rainy period starts in July and increases in frequency through December to March. Rainfall of high intensity occurs in October to March.

During rainfall, driver visibility is affected, and sight distance is reduced (Ben-Edigbe *et al.*, 2013), anxiety develops in some drivers in certain rainy conditions, reduction in pavement friction between the vehicle tyre and road pavement occurs (Mashros *et al.*, 2014) at both circulating and entry roadways. These factors could result in discomfort, a decrease in speed with an apparent increase in delay, the queue may start forming in some situations, and may reduce the safety due to an increase in the accident rate as a result of rainfall. All these factors may affect the quality of service at the roundabouts in South Africa. Any changes in any of the factors at the roundabouts might have an influence on the roundabout's quality of service. Though rainfall is one of the factors that influences the traffic flow rate, driver behavior, and highway performance, it also has an effect on other factors that contribute to the quality of service at the roundabout. Delay is a major parameter that is used in the determination of the quality of service at the roundabout, and entry capacity is also vital in the determination of the entry delay. Traffic flow rate at both the circulating and the entry roadway are needed in the estimation of the entry capacity. Rainfall effect has been investigated to influence traffic demand volume, speed, density, percentage car equivalent, safety, traffic flow rate, and capacity (Alhassan and Ben-Edigbe, 2011, Shi *et al.*, 2011, Bergel-Hayat *et al.*, 2013, Cools *et al.*, 2010). The major effect of rainfall is the frictional reduction between tyres and the road surface, and visibility reduction due to vehicle windscreens becoming covered by drops from rain, splashes from other vehicles may add dirt to the windscreen, induce anxiety, anger and reduce drivers' visibility (Prevedouros and Chang, 2005, Goodwin and Pisano, 2003, Mashros *et al.*, 2014b). The effect of rainfall makes drivers reduce speed, increase the perception time and longer stopping distance. The influence of rainfall increases the operational costs at intersections and in general has increased travel time, fuel consumption and tyre wearing, the cost of repair of vehicular accidents and economic losses due to the delay in travelling time.

When rain falls, it falls on both the circulating and entry roadway, so no one has undue advantage over the other, bearing in mind that the entry vehicle must yield to the circulating vehicles. When rain falls, the entry vehicles might not be able to judge the critical gap correctly, this might make some drivers reduce their speed and at times force themselves into the available gap. Rainfall causes crashes at the intersections (Shi *et al.*, 2011) in which a roundabout is a typical example, despite the high level of safety at roundabout compared to other forms of intersections. The crashes occur due to a reduction in friction between the pavement and vehicle tyres and the reduction in visibility during heavy rain.

Traffic volumes reduce during winter, (Knapp, 2000, McBride et al., 1977, Hanbali, 1994, Nixon, 1998, Keay and Simmonds, 2005, Maze et al., 2006). Keay and Simmonds (2005) discovered that there is a negative relationship between traffic volume and rainfall intensity which implies that traffic volume reduces as the rainfall intensity increases. The increase in traffic flow rate can be reasoned to affect the capacity, volume capacity ratio (v/c) and the reserved capacity (Q_R), which are key parameters in the quality of roundabout service delivery. It will be expected that the vehicular speed and the traffic flow rate will increase with a reduction in traffic volume, but studies have discovered that these parameters reduce with an increase in the intensity of rainfall despite the reduction in traffic volume due to the effect of rainfall. Mashros *et al.* (2014c) discovered that the average traffic volume for daylight off peak hours for all cases of rainfall intensity shows no pronounced effect under rainfall conditions. Mashros et al. (2014a) investigated the impact of rainfall on travel speed and discovered that travel speed decreases with an increase in rainfall intensity, and they noticed that there were noticeable changes in travel speed due to rainfall especially during heavy rainfall. Akin *et al.* (2011) worked on the impact of weather on two roads in Istanbul, and they discovered that rainfall reduces speed by 8 to 12 percent and capacity by 7 to 8 percent. Smith *et al.* (2004) discovered that regardless of the rain intensity it decreases the operating speed by 5.0 to 6.5%. Ibrahim and Hall (1994) studied adverse weather effects and discovered that free flow rate speed decreases by 2km/hr with light rain and 30 to 50km/hr with heavy rainfall. In research carried out on the effect of weather on traffic, it was also concluded that there is a reduction in traffic speed due to rainfall (Pham *et al.*, 2007).

With the reduction in speed due to rainfall, it would be expected that this would have an effect on the capacity. Smith *et al.* (2004) investigated the impact of rainfall on traffic flow rate and collected traffic and rainfall data on two roads in Hampton and discovered that light rainfall decreases capacity by 4 to 10 percent, while heavy rain decreases the capacity by 25 to 30 percent. Mashros et al. (2014c) discovered that capacity reduces by 2 to 32 percent, free flow rate decreases with increase in rain intensity by 3 to 14 percent and they attributes the changes in the capacity to drivers' reactions under rainy conditions. Maze *et al.* (2006) discovered there is a reduction in speed and an increase in headway with flow rate reduction, which results in a reduction in capacity.

Hashim (2012) discovered that passenger car equivalent (PCE) decreases in value with an increase in rainfall intensity, and he attributes the decrease in value to a low percentage of heavy vehicles in the traffic and an increase in headway of small vehicles under the prevailing conditions of rainfall. Nordiana (2012) discovered that the average traffic stream gap for dry weather is higher than that of rainfall under free flow rate conditions and concluded that rainfall intensity does not have a significant impact on the mean gap acceptance of traffic stream under free flow rate conditions because all vehicles are under the same rainfall conditions and therefore no one has an undue advantage over the others. The rate of occurrence of accidents was also found to increase under rainfall (Chung *et al.*, 2005). It can also be stated that an increase in rainfall intensity influences speed, capacity, increases the headway and reduces the traffic demand volume. If the travel demand is affected, the entry demand at the roundabout entry might also be affected.

2.7.1 Impact of rainfall on passenger car equivalent values

There are different methods of estimating passenger car equivalent (PCE) based on delay, density, speed, platoon formation, vehicle hours, travel time, and average headway (Shalini and Kumar, 2014). Seguin (1981) uses the headway method for estimation of PCE as the ratio average headway for the same vehicle type to average headway for passenger cars under the same conditions (see Equation 2.44). Seguin's method is effective, and the application is simple to use, so there is no need to develop a new model.

$$PCE_{ij} = \frac{H_{ij}}{H_{pcj}} : \quad [2.44]$$

$$\text{for: } H_{ij} = \frac{n}{(n-1)} \quad [2.45]$$

Where: PCE_{ij} = passenger car equivalent of class i vehicle under condition j .

H_{ij} = average headway of class i vehicle under condition j .

H_{pcj} = average headway of passenger car under condition j .

It may be useful to adjust PCE values to reflect prevailing rainy conditions before determining roundabout capacity. Passenger car equivalent (PCE) or passenger car unit (PCU) is a measurement used to assess the traffic-flow rate on a highway. It is essentially the impact that a mode of transport has on traffic variables (such as headway, speed, density) compared to a passenger car. Note that passenger car equivalent and passenger car units are often used interchangeably. Due to their design characteristics especially weight and length, operational performance of heavy vehicles differs from passenger cars and light vans. Hence, the HCM (2010) proposes an adjustment factor for heavy vehicles entering a roundabout. Interestingly, no adjustment is made for heavy circulating vehicles even though one can expect circulating heavy vehicles to have an impact on gap acceptance. In order to allow for the effect of traffic composition, ambient and road conditions, it is proposed that the South African passenger car equivalent values for roundabout shown in Table 2.5 be investigated for fitness and adjusted if necessary to reflect rainy conditions.

Table 2.5: South Africa Passenger Car Equivalent

Vehicle type	Rural roads	Urban roads	Roundabout	Traffic signal
Car and light vehicles	1.00	1.00	1.00	1.00
Commercial vehicles	3.00	1.75	2.80	1.75
Buses and coaches	3.00	3.00	2.80	2.25
Motorcycles	1.00	0.75	0.75	0.33
Pedal cycles	0.50	0.33	0.50	0.20

Source: SANRAL (2011)

2.7.2 Anomalous roundabout capacity shrinkage

Road network reserve capacity is defined as the maximum additional demand that can be accommodated by a road network without exceeding a prescribed degree of saturation while taking the users' route of choice into account (Wong and Yang, 1997b). A roundabout has three capacity determinants namely, entry capacity, circulating capacity and reserve capacity. Each has unique

functions and characteristics that have been described earlier in the thesis. Based on evidence from previous studies on rainfall impact on traffic flow rate, it is correct to postulate that rainfall would also cause anomalous capacity shrinkage at roundabouts. It is equally correct to state that entry capacity and reserve capacity are dependent variables whereas circulating capacity is an independent variable. It is also worth noting that circulating capacity by default could be written as volume to capacity ratio or the so-called degree of saturation. Reserve capacity is an indispensable management parameter in that it describes the amount of capacity remaining or available. It correlates well with the volume/capacity ratio. Kyte et al. (1992) have shown that reserve capacity is a useful measure and correlates well with the expected delay. Although Kyte et al. (1992) presented reserve capacity as an independent variable with delay as the dependent variable, it can be argued that delay and reserve capacity are dependent variables that can be used in turn as functions of degree of saturation or volume/capacity ratio, bearing in mind that degree of saturation is a derivative of roundabout circulating capacity. Wong (1996) estimates the reserved capacity for a roundabout by considering the approaches and the turning movements. Wong also presents a matrix of the turning movements that contribute to the circulating flow rate. According to Wong, $a_{ij} = 1$ if the turning movement j will form part of the circulation flow rate that crosses approach i and 0 otherwise.

b is a parameter that represents a matrix $n_a \times n_t$ for the turning movement entering the roundabout and is defined as:

$b_{ij} = 1$ if the turning movement j enter the roundabout through approach i and 0 otherwise.

The arrival rate is represented by q_j , where $j = 1, 2, 3, \dots, n_t$

The total arrival rate (q_{ai}) at the entry roadway i ,

$$q_{ai} = \sum_{j=1}^{n_t} b_{ij} q_j \quad [2.46]$$

The total arrival rate at (q_{ci}) the circulating roadway i ,

$$q_{ci} = \sum_{j=1}^{n_t} a_{ij} q_j \quad [2.47]$$

Wong (1996) introduces a multiplier factor (μ) to the Kimberly's model for roundabout capacity as shown in Equation 2.48.

$$Q(\mu) = k_i(F_i - \mu f_i q_{ci}), i = 1, 2, 3, \dots, n_a \quad [2.48]$$

Where;

f_i , F_i and k_i are the geometric parameters

q_{ci} is the circulating flow rate across the entry i

Wong then estimates the maximum value of μ as:

$$\mu = k_i \left(\frac{p_i k_i F_i}{\sum_{j=1}^{n_i} b_{ij} q_j + p_i k_j f_{ci} \sum_{j=1}^{n_i} a_{ij} q_i} \right) \quad [2.49]$$

Based on the hypothesis that roundabout capacity would shrink under rainy conditions, it is postulated that reserve capacity would shrink, and the curve would shift backward as shown in Figure 2.8 below. The curve area reduction because of backward shift is estimated with Equation 2.50. Note that D denotes dry weather and R denotes rainfall. It is postulated that capacity would shrink, and the curve would shift backward as shown below in Figure 2.9, the curve area reduces as a result of the backward shift is estimated with Equation 2.51. Note that D denotes dry weather and R denotes rainfall.

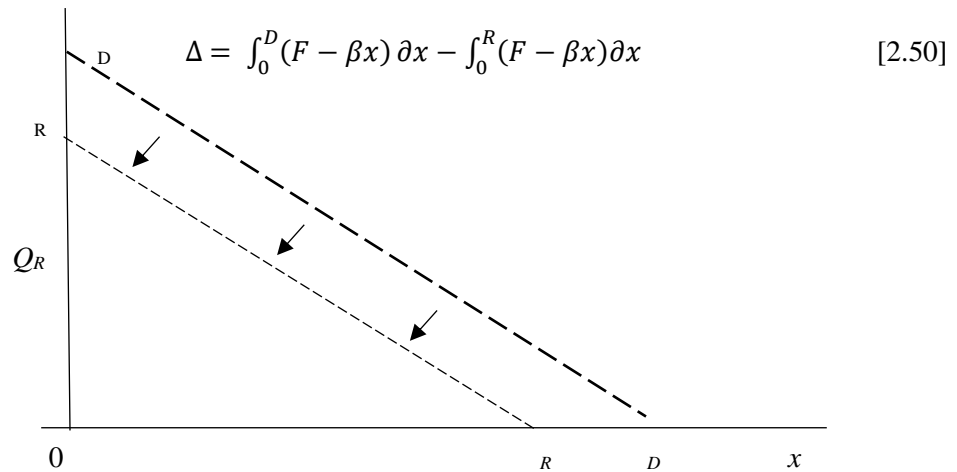


Figure 2.8: Reserve capacity (Q_R) vs degree of saturation (x)

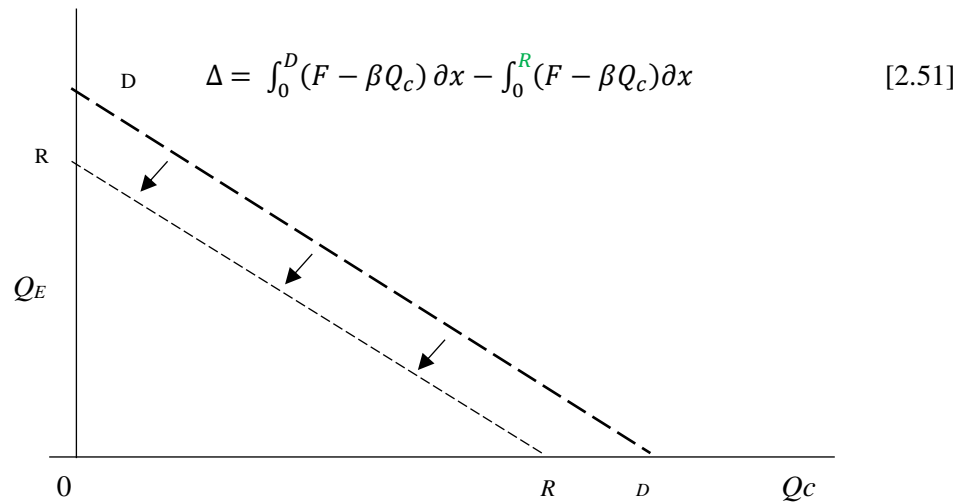


Figure 2.9: Entry capacity vs circulating capacity

2.8 Time Headway Concepts

Vehicles arrive at a roundabout either at the drivers' desired speed and deceleration or by following a lead vehicle. If there is a queue, the vehicles will join the queue and proceed at the queue's pace. Otherwise, the vehicle will join the circulating flow rate when an acceptable gap presents itself. If the available gap in the circulating stream is not acceptable, the driver waiting at the yield line must wait for the next gap. Successive gaps are then evaluated until a gap greater than the waiting driver's minimum acceptable gap is available in the circulating traffic stream. In any case the key aspects to be considered when modelling the queuing and entering process at the roundabout include among others:

- i. How the queue is formed,
- ii. When vehicles join the queue,
- iii. How vehicles move up the queue,
- iv. How the gap acceptance decision is made.

In summary, the queuing process affects the way in which delays are estimated, while the move-up times between successive vehicles and the gap acceptance decisions of drivers affect the capacity of an approach and hence the delay on the approach.

Time headways are time intervals between successive vehicles past a point on the highway. Because the inverse of the mean time headway is the rate of traffic flow rate, headways have been described as one of the traffic flow rate characteristics. It can be used to describe the stochastic arrival and departure process at roundabouts. Time headway varies between a random and regular state. According to May (1990) the time headway distribution in practice occurs between these two boundary conditions. Time headway is important because many traffic operations (such as passing, merging, and crossing) depend on the availability of large headways in the traffic flow. The time headway between vehicles is an important microscopic flow rate characteristic that affects the safety, level of service, driver behaviour and capacity of the transportation systems. The capacity of the system is governed primarily by the minimum time headway and the time headway distribution under capacity flow rate conditions. The elapsed time between pairs of vehicles is defined as the time headway. Figure 2.10 below illustrates the meaning of gap and headway.

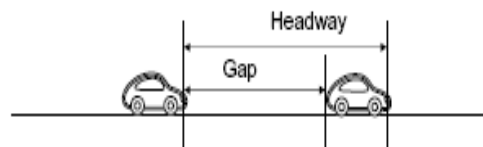


Figure 2.10: Gap and Headway

The distribution of headways has an effect on platoon formation and delays. In the Highway Capacity Manual (HCM, 2000), the level of service on two-lane rural highways is approximated by the proportion of headways less than five seconds, thus making a connection between the headway distribution and the level of service. In the traffic theory time headway is a microscopic variable that has been studied since the 1930s. This theoretical variable is fundamental because it describes the arrival traffic pattern of vehicles. Time headways are the time intervals between the

passage of successive vehicles past a point on the highway. In the generalized mathematical time headway model, the theoretical distributions such as the displaced negative exponential distribution, the gamma distribution and the log-normal distribution, are used to model the headways. Furthermore, time headway is also easy to obtain with a high precision simply by using, for example, inductive loops on a traffic lane. In this time headway model, three main approaches have been proposed: the probabilistic model, the analytical model of Heidemann and Wegmann (1997), and the cellular automaton model. Suffice to say that several models have been proposed for the distribution of headways, for example, Buckley (1968), Cowan (1975) M3 distribution model, the log-normal distribution (Mei and Bullen, 1993), and the double displaced negative exponential distribution (Sullivan and Troutbeck, 1994). The gamma and shifted exponential distributions were found by Al-Ghamdi (1999) and he described headways for low and medium traffic flow rate conditions. In this distribution he suggests that headway data cannot be combined from different sites to find the proper distribution. Where the Pearson Type III family of distribution is used to represent headway distributions, the Probability Distribution Function for Pearson Type III (h must be greater than α) as per the Equation (2.52) below:

$$f(h) = \frac{\lambda}{\Gamma(k)} [\lambda(h - \alpha)]^{(k-1)} e^{-\lambda(h-\alpha)} \quad [2.52]$$

Where; α , is the shift parameter generally taken as 0.5s, $k = \left[\frac{\hat{h}-\alpha}{h_s} \right]^2$

\hat{h} represent the mean of the observed headway data,

h_s represents standard deviation of the observed headway data and $\lambda = \frac{k}{\hat{h} - \alpha}$

The Probability Distribution Function for Erlang (but k is an integer) as per Equation (2.53) below:

$$f(k) = \frac{\lambda^k}{(k-1)!} [\lambda h]^{(k-1)} e^{-\lambda h} \quad [2.53]$$

Where; $k = \text{nearest integer to } \left[\frac{\hat{h}}{h_s} \right]^2$ and $\lambda = \frac{k}{\hat{h}}$

When an 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;

$$f(t) - \phi g(t) + (1 - \phi)h(t) \quad [2.54]$$

Where ϕ denotes the proportion of following vehicles (Van As, 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 headways (Van As, 1993). Akcelik and Chung (1994a) suggest that the bunched exponential distribution is much more realistic than the other two distributions and they recommend its usage. The bunched exponential 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). According to Akcelik and Chung (1994a), the cumulative distribution function $F(t)$ of the bunched exponential distribution, representing headway in a multi-lane traffic stream and the probability that a headway is less than or equal to t_1 is stated as:

$$F(t) = 1 - \alpha_2 e^{-\lambda(t - \beta_1)} \text{ for } t \geq \beta_1 \quad [2.55]$$

$$= 0 \quad \text{for } t \leq \beta_1$$

Where; $\lambda = \alpha_2 q_t / (1 - \beta_1 q_t)$ and the entering capacity from a minor approach in vehicles per second is computed (Troutbeck, 1989) thus:

$$q_e = \frac{q_c \alpha_2 e^{-\lambda(\tau - \beta_2)}}{1 - e^{-\lambda \beta_2}} \quad [2.56]$$

Where;

q_c is the sum of the flow rates in all circulating lane,

τ = critical gap, β_2 = average move up time

q_t denotes arrival flow rates on all the lanes

α_2 denotes proportion of free vehicles = $e^{-b\beta_1 q_t}$

b is the bunching factor; β_1 is the minimum arrival headway

2.8.1 Drivers' behavior and time headway at roundabouts

Driver behaviour at roundabouts is mostly affected at the roundabout entry. This is because the circulating vehicles have continuous movement, while the entry vehicle must judge the gap to be accepted before entering the roundabout. The driver's behaviour influences the number of vehicles that accept the same critical gap with the circulating vehicle, which is termed the follow-up headway. Follow-up headway is defined as the difference in time between a departure vehicle and the immediately following vehicle at the roundabout entry if the two vehicles accept the same gap in the circulating stream under queuing conditions (Tracz et al., 2004). The follow-up headway is influenced by many factors which include roundabout geometry, traffic movement at the roundabout, the circulating traffic volume, the pedestrian volume, vehicle type and speed, waiting time, driver age and gender (Lord-Attivor and Jha, 2012b, Zong et al., ND). Rain is a factor that affects the driver's visibility at the entry, and when visibility is affected the entry driver may not be able to judge the gap to be accepted nor the speed of the circulating vehicle correctly. It can therefore be reasoned that it will have an effect on the entry vehicle's waiting time and the follow-up time as the entry drivers will be more cautious before entering the roundabout. The effect of the waiting time might have a knock-on effect on the number of vehicles that enter the roundabout, affect the capacity, cause delay and decrease the quality of service of the roundabout. This may depend on the rain intensity, although the extent of rain intensity on follow-up headway is yet to be determined.

The follow-up time at roundabouts varies in places around the world. Rodegerdts *et al.* (2007) show that the average follow-up time in the US is 3.2 seconds, while Dahl and Lee (2012) found the average follow-up time in Canada to be 3.30 seconds. There are many factors that contribute to drivers' behaviour at roundabouts. They include road infrastructure, vehicle type, traffic and ambient conditions as well as the ability to estimate the circulating vehicle speed (Lord-Attivor and Jha, 2012a, Johnson, 2013, Ben-Edigbe, 2016). **The critical gap is the safe time headway of the circulating vehicle for entrance of the entry vehicle. In general, critical gap is a parameter that depends on local conditions such as geometric layout, driver behaviour, vehicle characteristics, and traffic conditions.** Follow-up time is the minimum time headway between two successive vehicles entering the roundabout if the available gap is big enough. Follow-up time is a time headway.

The roundabout is an at-grade intersection that operates on the yield rule where the entry vehicles give priority to the circulating vehicles. The yield rule operates on the availability of a gap within the circulating traffic. Whenever a gap is available, the entry vehicle will look for a safe gap in the circulating traffic before accepting and entering the roundabout. Sometimes when safe gaps appear in the circulating traffic stream, they are not taken by drivers entering the roundabout. Others may select to enter the roundabout when it is deemed unsafe. After all, driver behaviour at roundabouts is what the driver actually does, not what the driver can or is expected to do at roundabouts. It raises the question of what exactly can be construed a safe gap. One thing is clear, the available gap determines the number of the vehicles that enters the roundabout.

Critical gap is one of the factors that determines the number of vehicles that enter the roundabout. HCM (2000) define “critical gap as the minimum time between successive major street vehicles where the street vehicles make a maneuver.” This is a generalised critical gap definition for intersection but the definition for the critical gap at a roundabout can be defined as the minimum acceptable time headway between successive circulating vehicles by the entry vehicle. Any changes in the critical gap will have an effect on the entry vehicles and could also affect the service provided by the roundabout. Critical gap is influenced by the drivers’ behaviour, entry angle, sight condition, pavement markings and vehicles’ performance (Xu and Tian, 2008, Lord-Attivor and Jha, 2012b, Raff, 1950, Kang et al., 2012).

The effect of rainfall on traffic flow rate and driver visibility as discussed shows that drivers’ visibility is affected, and it can therefore be reasoned that the circulating vehicles may be affected by the poor visibility due to rainfall. Impaired visibility might make drivers reduce speed and affects the time headway at the circulating stream, this might affect the availability of acceptable gap by the entry vehicles. The waiting time of the entry vehicles at the entry while taking time to cautiously judge correctly the circulating vehicle speed and the safe gap under rainfall might lead to a reduction in critical gap availability. It could be reasoned that rainfall might affect the critical gap, but the extent of the effect with the varying intensity is yet to be investigated. Given a rainfall scenario at roundabouts, it is necessary to know the interaction of vehicle entering and circulating the central island. The key question is whether established and probable critical gaps and follow-up time headway under dry weather are the same under rainy conditions? Should there be

differentials, to what extent can be a concern. Would there be a significant differential along the rainfall precipitation line, it may be asked. The extent of driver behavioural changes and critical gap characteristics under rainy conditions has not been studied, hence the procedure adopted in this thesis is novel. There is a consensus among researchers that follow-up time and critical gap are two important roundabout performance measures, even though their values vary depending on the computation method used.

The best way to describe this stochastic process of vehicle arrivals is by using the time headways between vehicles. As the estimation of headways is important to several applications in traffic engineering, it has been researched and documented by (Van As, 1993, Akcelik and Chung, 1994a, Ashalatha and Chandra, 2011, Jenjiwattanakul and Sano, 2011), among others and will be discussed in this thesis. According to May (1990), time headways vary considerably between two boundary conditions. Under low traffic flow rate, time headways can be considered random, when flow rate is near capacity time headways are constant.

In previous studies, a maximum likelihood Raff (1950), Ashworth (1968), Siegloch (1973), and Wu (2012) methods among others were used to estimate critical gap. The Raff model is based on cumulative density function of the accepted and rejected headway used at the intercept as the critical headway. Rodegerdts (2007) show that critical gap in the US is in the range of 3.7 to 5.5 seconds, Dahl and Lee (2012) found the critical gap to be between 3.5 to 6.1 seconds in Canada, Manage et al. (2003) showed that the critical gap in Japan ranges between 3.26 to 4.90 seconds, while Qu et al. (2014) mentioned that the critical gap is in the range of 2.6 to 3.2 seconds in China. So it is postulated that time headway has no fixed value. It varies relative to prevailing conditions.

Raff's model is widely used in many countries owing to its simplicity and practicality. Wu (2012) mentions that the critical headway based on the Raff model does not consider the average critical headway. The Raff method of estimating the critical gap uses the cumulative probability of the rejected and accepted gap (F_r and F_a). Raff's threshold method is one of the earliest methods of gap acceptance estimation and is adopted in many studies for its simplicity. It can be expressed as:

$$1 - Fr(t) = Fa(t) \quad [2.57]$$

Raff's consideration that the number of rejected gaps larger than the critical gap is equal to the number of accepted gaps smaller than critical gap which can be expressed as;

$$N_a F_a(\hat{t}_c) = N_r [1 - F_r(\hat{t}_c)] \quad [2.58]$$

Then;

$$F_a(\hat{t}_c) = \frac{\gamma_r}{\gamma_a} [1 - F_r(\hat{t}_c)] \quad [2.59]$$

The proportion of rejected gaps larger than the critical gap is equal to the proportion of accepted gaps smaller than the critical gap because N is fixed. Two proportions can be counteracted, so that the total accepted coefficient is equal to the accumulated probability of the headway (t) larger than the critical gap as illustrated below;

$$\gamma_a = P(T \geq \hat{t}_c) + \gamma_a F_a(\hat{t}_c) - \gamma_r [1 - F_r(\hat{t}_c)] \quad [2.60]$$

$$= P(T \geq \hat{t}_c) = \int_{\hat{t}_c}^{\infty} f(t) dt = 1 - F_r(\hat{t}_c) = \alpha e^{-\lambda(\hat{t}_c - t_m)} \quad [2.61]$$

Then,

$$\hat{t}_c = t_m - \frac{1}{\lambda} \ln \left(\frac{\gamma_a}{\alpha} \right) \quad [2.62]$$

Where,

t denotes headway; $Fr(t)$ is cumulative probability of the rejected gap; $Fa(t)$ is the cumulative probability of the accepted gap; $P(\cdot)$ denotes the probability of gap interval; $f(t)$ denotes the probability function of headway in a major stream; $F(t)$ denotes the cumulative probability function of headway in a major stream; λ denotes decay constant, $= \alpha q / (1 - qt_m)$; t_m is the minimum headway, and α denotes the proportion of free vehicles.

Ashworth (1968) assumes that the headway of a major stream follows the negative exponential distribution and critical gap and the accepted gap follows normal distribution, Ashworth gives the calculation formula of critical gap as follows:

$$\bar{t}_c = \bar{t}_a - q_a \sigma_a^2 \quad [2.63]$$

Where; q is the flow rate rate of the major stream (veh/s), \bar{t}_c is the average critical gap (s), \bar{t}_a is the average accepted gap (s), and σ_a^2 is the variance of accepted gaps (s^2). The standard deviation of the accepted gaps (s) is shown below in equation 2.64 where s is the standard deviation of the accepted gap (s), x is the accepted gap (s), \bar{x} is the mean of the accepted gap (s) and n is the total number of accepted gap.

$$s = \sqrt{\sum \frac{(x - \bar{x})^2}{n - 1}} \quad [2.64]$$

Ashworth (1970) estimates critical gap using standard deviation of the accepted gaps within the circulating traffic stream, the circulating traffic flow rate and the mean accepted gap. However, Miller (1974) modified Ashworth's equation on the hypothesis that critical gap follows distribution.

$$\bar{t}_c = \bar{t}_a - v_p \sigma_c^2 \quad [2.65]$$

$$\sigma_c = \sigma_a \frac{\bar{t}_c}{\bar{t}_a} \quad [2.66]$$

Where, σ_c^2 is the variance of critical gap (s^2).

Wu (2012) did not require any assumptions concerning the distribution function of critical gaps and the driver behaviour, rather probability density function was used to estimate critical headways as illustrated below in equation 2.67.

$$F_{tc}(t) = \frac{F_a(t)}{F_a(t)+1-F_r(t)} \quad [2.67]$$

Where: $F_{tc}(t)$ = PDF of the critical headway; $F_{ta}(t)$ = PDF of an accepted gap t , and

$F_r(t)$ = PDF of a maximum rejected gap t .

If a time gap is sorted in ascending order, critical headway is estimated with the equation 2.68.

$$t_c = \sum_{j=1}^N [P_{tc}(t_j) \cdot \frac{t_j + t_{j-1}}{2}] \quad [2.68]$$

Where $P_{tc}(t_j)$ is the frequency of the calculated critical headways between j and j-1.

Logit's method is a weighted regression model to estimate the critical gap as shown in equation 2.69.

$$p = \frac{1}{1 + e^{-(\beta_0 + \beta_1 x)}} \quad [2.69]$$

Where:

p = Probability of gap acceptance by the entry vehicle; x = gap duration.

β_0 = Regression coefficient; β_1 = Regression coefficient

Solving for x by assuming that the probability of accepting a gap is 50 percent, then substitute 0.5 for P to obtain the critical gap. This method is adopted for critical gap estimation in many studies (Gattis and Low, 1999, Ashalatha and Chandra, 2011, Vasconcelos et al., 2013). Brilon *et al.* (1999) and Vasconcelos *et al.* (2013) discovered that this method underestimates critical gap when compared to other methods of estimating critical gap. The Logit method was not recommended for critical gap estimation by Brilon *et al.* (1999) because it was found to be dependent on conflicting traffic volume. The Probit method of critical gap estimation uses a best-line fit to a weighted linear regression of gap data. The interval is divided into suitable portions, and the proportion of accepted gap is determined. The process of gap acceptance is a binomial response and is dependent on the size of the gap. On the assumption that the critical gap is normally distributed, the probit of the proportion accepting a gap is shown in equation 2.70.

$$Y = 5 + \frac{(x - \mu)}{\sigma} \quad [2.70]$$

Where: x = Accepted gaps proportion; μ and σ = normal distribution parameters.

Y = Probit of x ; 5 is added to the equation to keep the probit value.

Plotting probit versus gap size logarithm with a best-fit line, the critical gap can be obtained from the best-fit line as the x corresponding value of probit is 5. Siegloch (1973) uses the average number of entry vehicles and the accepted gaps to determine the critical gap and follow-up headway. He developed a linear relationship with the number of vehicles that accept gaps. He determined the accepted gap by plotting the number of vehicles accepting the gap as the dependent variable, and average accepted gap as the independent variable. The reciprocal of the gradient of the positive linear relationship gives the follow-up headway. He used the x -intercept and the estimated follow-up time to estimate the critical gap as:

$$t_c = t_0 + 0.5t_f \quad [2.71]$$

Where:

t_c = Critical gap (s);

t_0 = X-axis intercept (s) = 2.51s (HCM 2000)

t_f = Estimated follow-up headway (s).

According to HCM (2010) follow-up time (t_f) and critical gap (t_c) can be estimated with the following equations:

$$Q_E = Ae^{(-Bq_c)} \quad [2.72]$$

Where;

$$A = 1130 = \frac{3600}{t_f} \rightarrow t_f = 3.19s \quad [2.73]$$

$$B = 0.0007 = \frac{t_0}{3600} = \frac{t_c - 0.5t_f}{3600} \rightarrow t_c = 4.11s \quad [2.74]$$

Note that in equations 2.73 and 2.74, the parameters have fixed values for A and B. By implication, if the values of A and B can be computed by any valid method, follow-up time and critical gap can be estimated along the HCM (2010) line, it can be argued. If Kimber's equation is modified to include a dummy variable as shown in equation 2.75, and simple substitution of F for A and fc for B, then the prevailing follow-up time and critical gap can be computed thus;

$$Q_E = k \{(F - fcQc) + \epsilon\} \rightarrow fcQc \leq F \quad [2.75]$$

$Q_E = 0$ when $fcQc > F$

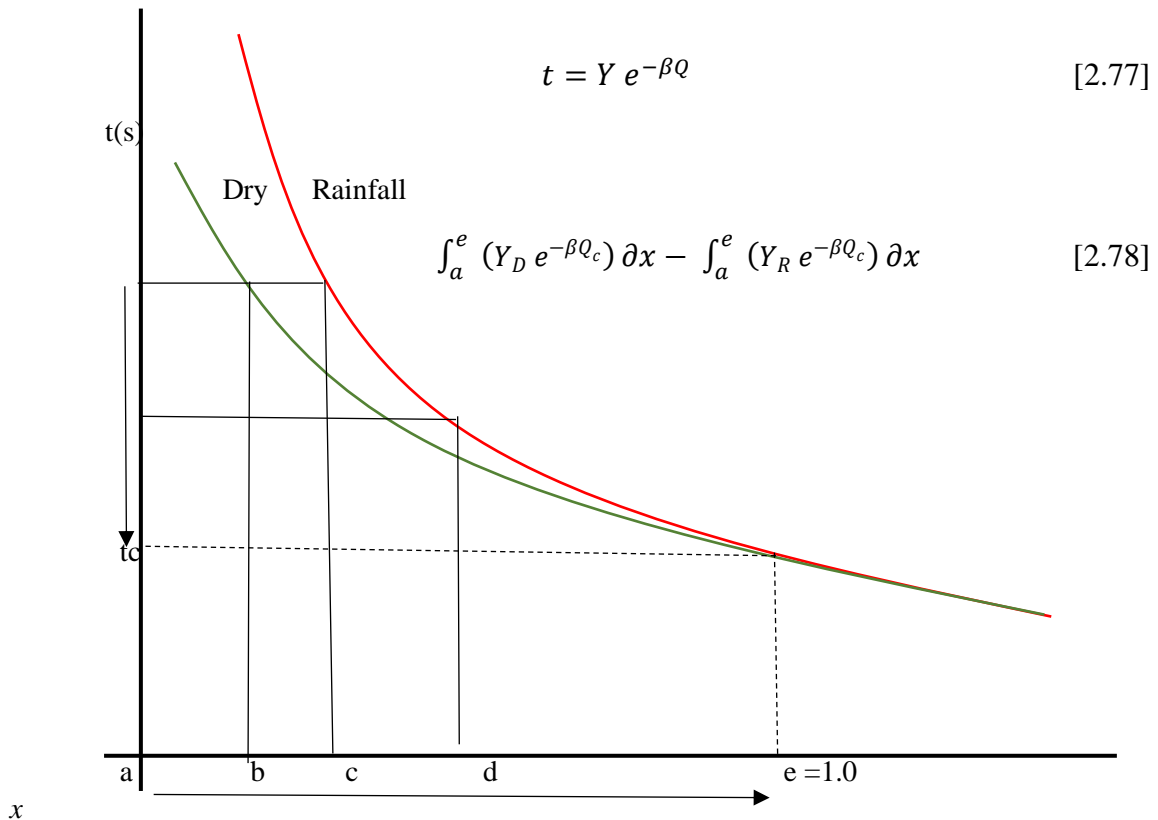
$$k = 1 - 0.00347(\phi - 30) - 0.978\left(\frac{1}{r} - 0.05\right) \quad [2.76]$$

Where;

Q_e = entry flow rate (pcu/h) and

Q_c = circulating flow rate (pcu/h)

Consider equation 2.75 again, when it rains, a dummy variable (ϵ) is introduced to depict that condition, hence one (1), otherwise zero (0) for dry weather.



a is the volume capacity ratio of 0, b, c and d are the volume capacity ratio increase towards 1.

Figure 2.11: Hypothetical time headway changes caused by rainfall

The key parameter F allows the influence of geometrical parameters like entry width, flare length and approach width to be determined. By adjusting F , the slope of the linear equation that also

contains the major capacity geometrical relationships is preserved. Average time headway for vehicles circulating the roundabout can be estimated and adjusted to critical gap by considering the average vehicle length given that the difference between headway and gap is the lead vehicle length. When converting time headway to gap, the average circulating vehicle speed is also needed. For example, assuming F is 1000 veh/h, the average follow-up time headway will be $3600/1000 = 3.6$ seconds. Assuming the average travel speed is 10m/s and the average vehicle length is 5m then the average gap time = $3.6 - [5/10] = 3.1$ seconds. So, there is no need to build a new model. What is needed is a modification of the relevant existing methods to accommodate rainy conditions. Shown below in figure 2.11 is a hypothetical time headway (t) relationship with degree of saturation. It is postulated that once the optimum flow rate is reached, the influence of rainfall is nullified. Thereafter, the peak traffic conditions set in and control time headway (t).

2.9 Summary

The main objective of roundabout design is to minimise delay whilst maintaining the safety for all road users. Entry width and sharpness of flare are the most important determinants of capacity, whereas entry deflection is the most important factor for safety as it governs the speed of vehicles through the roundabout. A conventional roundabout has a kerbed central island at least four metres in diameter. Usually, it has flared entries and exits to allow two or three vehicles to enter or leave the roundabout on a given arm at the same time. A roundabout is unique because of the yield rule.

In this chapter eight important issues were raised and discussed namely; roundabout characteristics, rainfall intensity, traffic flow rate, passenger car equivalency, service delivery, delay, follow-up time and critical gap characteristics. The influence of rainfall on functional service delivery at multilane roundabouts and their implications for driver behaviour and critical gap characteristics was the principal issue. As mentioned in the chapter, service delivery can either be functional or structural. Structural service delivery at roundabouts deals with infrastructure performance, whereas functional service delivery deals with traffic operations. Functional service delivery was combined with quality of service and renamed functional quality of service in this thesis. Functional quality of service uses grade classifications of class A to F to describe the effectiveness of traffic operations.

In South Africa there are no guidelines on roundabout quantitative and qualitative assessment approaches. Consequently, an empirical model is used to describe the relationship between entry and circulating flow rate rates under dry and rainy weather conditions. The empirical multiple regression model was employed in the estimate entry capacity because of the suitability for dry and rainy conditions. The ensuing model functions were modified with appropriate correction factors relative to their entry angles and entry widths. The influence of rainfall on passenger car equivalent values is significant to entry capacity estimation. Passenger car equivalent values are measures of vehicle performance relative to various types of terrain, usually level, rolling and mountainous, under prevailing conditions. Many scholars have attempted to provide realistic values, but the issues have yet to be resolved. In this chapter, a simplistic headway method was used to at least point passenger car equivalency for multilane roundabouts.

The implications of rainfall at roundabouts on driver behaviour and critical gap characteristics will be investigated. Follow-up time and critical gap are the proxies for driver behaviour in this study. As for delay, there is a consensus among scholars that modified (HCM, 2010) delay would be adequate. Gap acceptance and empirical estimation methods' appropriateness are still debated by researchers all over the world. One thing is clear though, there are no fixed values for roundabout capacity, delays, follow-up time and critical gap. Those discussed in this chapter are effective approximations since there are no fixed values for these parameters. Now whether the values are fixed or **dynamic**, one thing is obvious, functional quality of service deterioration at multilane roundabouts would result from rainfall irrespective of the intensity.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Overview

This study is centered on how rainfall influences roundabout quality of service delivery. Roundabout service delivery has been discussed in the preceding chapter to be two dimensional (structural and functional). The functional quality of service is the main consideration of this study. The parameters for the assessment of functional quality of service are multi-dimensional and consider both user and provider's perspective. The parameters for the assessment of functional quality of service are the volume capacity ratio (v/c) and reserve capacity (Q_R) for the provider's perspective, and the control delay and queue length for the user's perspective. To determine any of these parameters, the entry capacity must be determined. Hence, a model was developed for the prediction of roundabout entry capacity under rainy and dry conditions; entry capacity is an essential parameter in determining any of the parameters for assessing the roundabout functional quality of service. There are different methods of estimating the entry capacity, which includes the gap acceptance and the empirical method. The empirical method will be adopted in this study because the study is an empirical study where gap acceptance might not be appropriate because of its theoretical approach. The empirical method also considers the roundabout geometry, which is one of the major factors that contributes to roundabout performance (Kimberly, 1990). The volume capacity ratio and reserve capacity were estimated.

The HCM delay model does not take into consideration the roundabout geometry, despite the fact that geometry contributes to the control delay. The empirical capacity model takes into consideration the geometry; hence, was used in the modification of the HCM control delay model. The queue length and the delay were determined under dry and rainy conditions. The volume capacity ratio and the reserved capacity was used for the providers' perspectives while the control delay and queue length were used for the users' perspectives of roundabout performance. These parameters were estimated at peak period for the development of functional quality of service

criteria, as discussed earlier in chapter 2. These parameters were used in the assessment of the functional quality of service under dry and rainy conditions. In addition to these, the passenger car equivalent, critical gap, and follow-up headway were investigated under dry and rainy conditions.

The roundabouts considered for the study met some key selection requirements. The site selection criteria and traffic data collection methods were discussed in this chapter. The methodology for the study and the procedure for entry and circulating traffic data collection with the use of automatic traffic counters (ATC), which includes the setup of the device, uploading of traffic data with the reconnaissance survey, and precautionary measures taken during the device installation, were explained. The pilot data assessment has been assessed analytically. This is important because it forms the prerequisite for the survey that are presented in subsequent chapters.

This chapter is arranged as follows: methodology adopted for the study is discussed Section 3.2; the roundabout selection criteria is discussed in section 3.3; followed by coding of the selected roundabouts for the survey in section 3.4; the assessment of the selected roundabout is presented in section 3.5; the survey method used in the study is discussed in section 3.6; and the sample data for the pilot study are appraised analytically in section 3.7. The chapter summary is presented in section 3.8.

3.2 Research Methodology

The traffic data and rain intensity survey samples were taken from various sites in Durban city, South Africa. Empirical and analytical methods were used in this thesis. The functional quality of service considers the roundabout providers' and users' perspective, unlike the level of service that considers only one of either the user or providers' perspectives in the assessment of roundabout service delivery. The volume capacity ratio and the reserved capacity were two parameters used for the roundabout providers' perspectives of roundabout performance assessment while the control delay and queue length are the parameters used in the users' assessments of roundabout performance. Capacity estimation is an important parameter for determination of any of the parameters assessing the functional for quality of service. There are many empirical methods of predicting the entry capacity, but the method that could estimate the entry capacity at the free flow

rate and peak period **was** be adopted because off-peak traffic under rainy conditions is always free flow rate, except where rain falls at peak period. The rainy conditions **were** categorized into light, moderate, and heavy rain according to rainfall intensity in accordance with the World Meteorological Organization (WMO) rain classification. Very heavy rain with rain intensity more than 50mm per hour **was** not considered in this study because of the drag force effect, aquaplaning, and reduction in visibility due to splash that may occur from other vehicles in the traffic stream which might induce aggression and anger on drivers and further affects the drivers' behaviour.

A predictive model **was** developed for the entry capacity for rainy and dry weather conditions. In previous research by Kimberly (1980), he modeled the roundabout capacity under dry conditions. However, he did not consider the influence of weather change like rainfall in the empirical model. In this research, the effect of rainfall **was** considered in the entry capacity model. The capacity model will be tested statistically at a 95 percent level of confidence. The developed capacity model **was** used in estimating the volume capacity ratio.

Empirical data for entry and circulating traffic flow rates **were** collected continuously for six weeks at four selected multilane roundabouts using automatic traffic counters (ATC). The roundabouts were selected based on the proximity to surface rain gauges. Geometric data **were** collected by way of direct roundabout measurement and this was checked against design drawings for authentication.

The collected traffic data corresponding to the time of **every** rainfall was selected for each of the rain classes. The collected traffic volume **data** were converted to traffic flow rate with the application of passenger car equivalent (PCE) values. The South Africa National Road Agency Limited (SANRAL) geometric design guidelines provided for roundabout PCE, but the PCE was estimated under dry weather conditions. The application of the SANRAL PCE values in this study might affect the roundabout functional quality of service under rainy conditions. The effect of PCE value on vehicles at both the entry and circulatory roadway is investigated by estimating the PCE values under different categories of rainfall, based on the rain intensity and compared to the SANRAL PCE values. The framework of the functional quality of service assessment method is in Figure 3.1.

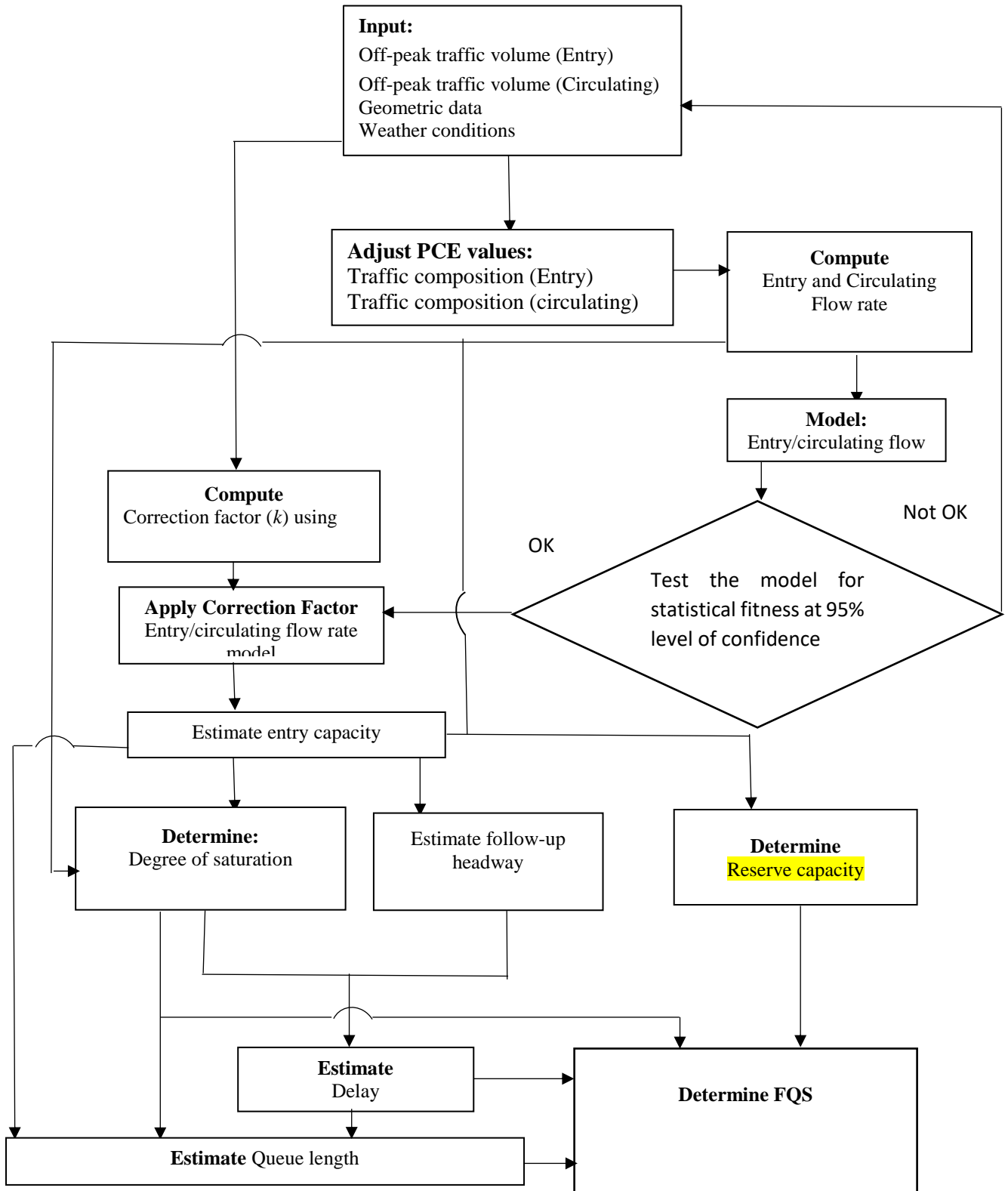


Figure 3.1: Framework of Functional Quality of Service assessment method.

3.3 Conditions for Site Selection

Proximity to a rain gauge **was** an important factor considered in the selection of the roundabout location **since information on** rainfall intensity was **germane to** the study. **Since** rain gauges have different capacities in terms of the time in which they record the amount of rainfall, **those** that belong to eThekweni municipality were used in this study. **The** rain gauges are modern-day **types** that can record the amount of rainfall over a period of five-minute intervals daily. The five-minute intervals **were** considered adequate for this study **due to** the rain intensity fluctuation. If a time of more than five minute is considered, the rain fluctuation might affect the rain data.

The roundabouts within the catchment area of the rain gauges were considered for the study. The geometry of **each roundabout** was checked to make sure that **it** conformed to the required roundabout geometry specifications of SANRAL. The roundabouts were examined to be free **from all** factors that may influence traffic, like closeness to tourist centers, commercial places, and other intersections. The roundabouts' pavements were **assessed** and found to be free of any functional and structural defects because the presence of these defects is another **factor** that **could affect** drivers' behaviour, which might be difficult to separate from the rainfall effect. The roundabout with very low pedestrian volume was considered because high pedestrian volume has an effect on the traffic flow rate and drivers' behaviour, which might be difficult to separate from rainfall effect. Road markings and signs were properly checked to make sure they were adequate at the selected roundabouts. Roundabouts with very low traffic were not considered because there might be no delay other than the geometric delay. The double lane roundabout was considered suitable for the study **because** the traffic volume would be sufficient to give vehicles queuing on the approaches. Properly drained roundabouts were considered for the study. The typical geometric selection criteria parameters considered in this study are shown in Table 3.1.

Table 3.1: Conditions for Site Selection

Geometry	Selection criterial
Entry lane number	3
Entry lane width	3.7 to 4.6m
Entry width	7.3 to 9.1m
Entry angle	20 - 60deg
Entry radius	20 - 60m
Effective flare length	On all approaches
Number circulating roadway lane	2
Circulating roadway width	8.5-9.8m
Central Island	Circular in shape
Inscribed circle diameter	30 - 80m
Road signs and marking	Adequate road marking and signs.
Traffic condition	Moderate
Splitter Island	Presence of deflection Island in all the approaches

3.4 Coding of the Surveyed Site

The surveyed sites were coded for easy data presentation and description of each surveyed site. The coding was in the form of SS “###” and PST “###”. The SS, PST, and ### represent the surveyed site, pilot surveyed site, and the serial number respectively. As an example, SS01 means

survey site with serial number 01. Five roundabouts were selected for the study, one for pilot site, and others for the study sites. The code for each site is presented in Table 3.2. The assessment of these roundabouts is discussed in section 3.5.

Table 3.2: **Site Code for the Surveyed Sites**

Site number	Site name	Site code
Site 01	Armstrong – Umhlanga rock drive roundabout SS001	SS01
Site 02	Umhlanga Rock – Douglas Saunders roundabout	SS02
Site 03	Millennium – Jubilee roundabout	SS03
Site 04	Gateway roundabout	SS04
Site 05	Torsvale roundabout	PST01

3.5 Assessment of Selected Roundabout Sites

3.5.1 Site 01 - Armstrong Roundabout

The roundabout is located in the Umhlanga area of Durban. It is a multilane roundabout, the circulating and entry roads are double lanes, and there are four approaches to the roundabout. The design speed is 50km/h. The central Island is raised and circular in shape. The entry and circulating roadway pavement is an asphaltic concrete with a design life of 20 years. The roundabout links Armstrong, La Lucia, Umhlanga rocks, and Durban North roads, which are municipality roads. Drivers' visibility is not impaired, adequate road marking and signs were also well placed. The pavement is free from any functional and structural defects. The geometry meets the geometry requirement for double lane roundabouts in SANRAL geometric guidelines. The geometry features of the roundabout are shown in Table 3.3. The two approaching roads on Umhlanga Rocks and Durban North roads are separated by a median while the approach roads on Armstrong and

La Lucia roads are separated with road markings. This separation does not affect drivers' behaviours. The four approaches have Splitter Island, and they are provided with a flare at the entry. The roundabout is properly drained. The distance to the closest rain gauge is 0.95km, as shown in Figure 3.2, the roundabout is within the catchment area of this rain gauge which belongs to the **EThekwini** Municipality. The rain gauge station ID is Crawford (see Appendix C), and the rain gauge catchment area is Crawford school-Armstrong and La-Lucia (EThekwini Municipality, 2016). The rain gauge is within a 10km radius of the site at 50m above or below ordnance datum (AOD), this is considered for the study (Jarraud, 2008). Rain data **were** supplemented with manual rain gauge data and **were** compared **with** the rain data from the website of the eThekwini Municipality. The automatic traffic counter was set up to collect traffic data at the entry and circulating roadway continuously for six weeks. The traffic data for the period between 07:00 and 17:00 was considered for this site, the time was selected to remove the effect of darkness and/or road lighting from the study. The site layout at Armstrong roundabout is shown in Figure 3.3. **Two hours manual counting was carried out under dry weather condition daily to authenticate the traffic volume recorded by the automatic traffic counter.** The rainfall between 07:00 and 17:00 was used as the control time for the traffic volume.

Table 3.3: Summary of the Geometric Data of the Surveyed Roundabouts

Roundabout features	Site 01	Site 02	Site 03	Site 04
Name of roundabout	Armstrong	Millennium	Douglas	Gateway
Class of roundabout	Double lane roundabout	Double lane roundabout	Double lane roundabout	Double lane roundabout
Entry pavement surface type	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement
Circulating pavement surface type	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement
Number of entry lanes	2	2	2	2
Number of circulatory roadway lanes	2	2	2	2
Entry width (m)	8.5	7.90	8.20	8.40
Entry angle (°)	50	45	45	50
Entry radius (m)	40	50	50	45
Effective flare length (m)	16	18	15	13
Inscribed circle diameter (m)	50.00	58.10	49.50	48.00
Approach road half width (m)	7.30	6.80	6.90	6.80
Circulating road width (m)	9.40	9.30	9.10	8.80
Central Island shape	Circular	Circular	Circular	Circular
Road signs and marking	OK	OK	OK	OK
Distance from rain gauge	0.95km	1.18km	0.82km	0.75km



 Rain gauge location  Armstrong Roundabout location

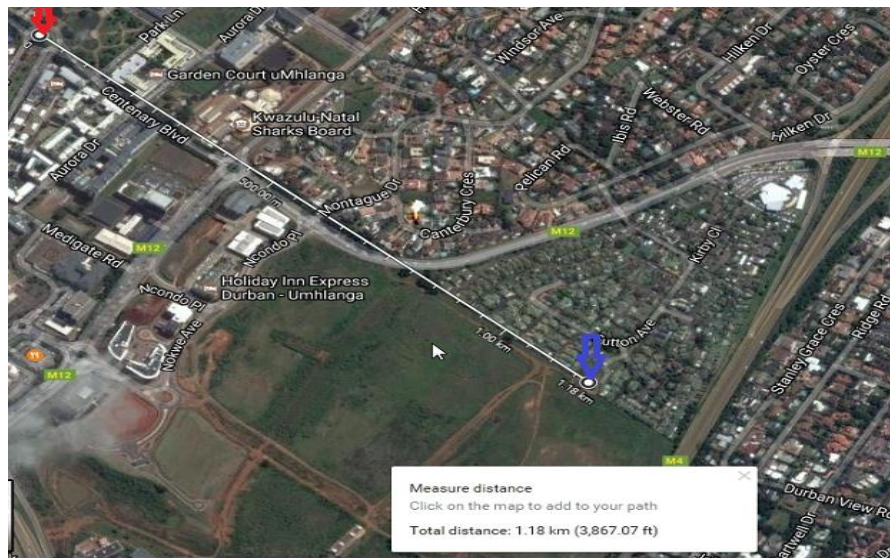
Figure 3.2: Distance of rain gauge from Armstrong roundabout



Figure 3.3: Site set-up for data collection at Armstrong roundabout.

3.5.2 Site 02: Millennium Roundabout

The Millennium roundabout is a multilane roundabout with four approaches. The approaches and the circulating roads are double lanes. The roundabout is maintained by the eThekwiwini Municipality. The central island is circular, and all the entries are flared. The geometry of the roundabout is shown in Table 3.3 (subsection 3.5.1). The rain gauge that covers the location of the roundabout is 1.18km away from the roundabout as shown in Figure 3.4. The rain gauge station ID is Umhnth (see Appendix C), located at coordinates -29.730142 latitude and 31.077661 longitude. The time of rainfall, and traffic data, between the period of 07:00 and 17:00 is used as the control for traffic volume. The data collection method and the site layout at millennium roundabout is shown in Figure 3.5.



Rain gauge location



Millennium Roundabout location

Figure 3.4: Distance of rain gauge from millennium roundabout



Figure 3.5: Site set-up for data collection millennium roundabout

3.5.3 Site 03: Douglas Roundabout

This roundabout has four approaches; each of the approaches and the circulating roads are double lanes. This means the roundabout is a multilane roundabout. The geometry of this site is shown in Table 3.3 (subsection 3.5.1). The circulating and entry roads' pavements are asphaltic concrete. The design life of the pavement is 20 years. The four roads connected by the roundabout are municipal roads. The eThekweni Municipality maintains the roundabout. The pavement is free of both functional and structural defects. The closest rain gauge, whose catchment area covers the roundabout, is located 0.82km away from the roundabout with station ID Umhnth (see Appendix C) as shown in figure 3.6. Pneumatic tube laying process at the circulating roadway at this site is shown in Figure 3.7.



↓ Rain gauge location ↓ Douglas Roundabout location

Figure 3.6: Distance of rain gauge from Douglas roundabout



Figure 3.7: Pneumatic tube laying process at the circulating roadway at Douglas roundabout

3.5.4 Site 04: Gateway Roundabout

The roundabout is in the Gateway area of Durban and it is a standard double lane roundabout that connects four municipalities' double lane roads; the circulating roadway is a double lane road. The distance of the roundabout from the rain gauge that has the catchment area that covers the roundabout is 0.75km. The rain gauge station ID is Umhnth (see Appendix C) and it is located at coordinates 29.730142 latitude and 31.077661 longitude. The road markings and signs are adequate and the entry and circulating pavement are free of structural and functional distress. The central island is circular. The geometry of the roundabout is shown in Table 3.3 (subsection 3.5.1). The driver visibility at the roundabout is acceptable. The entry and circulating roadways have asphaltic pavements. The location of the roundabout and the distance from rain gauge is shown in Figure 3.8. The survey was carried out at 07:00 to 17:00. The rainfall within this period was used as a control for the traffic data. All the entries were flared, and the design speed is 50km/h. The site set up process is shown in Figure 3.9.



↓ Rain gauge location ↓ Gateway roundabout location

Figure 3.8: Distance of rain gauge from site 004



Figure 3.9: Site set-up for data collection at gateway roundabout

3.6 The Survey Method Adopted in the Study

Traffic data is germane to this study. The data were collected after the approval by the eThekweni Transportation Authority (ETA) for installation of automatic traffic counters (ATC) to collect traffic data at the selected roundabouts (see Appendix B). The approval was communicated to the metro police division for traffic control during the installation of the devices. The ATC were set-up according to the manufacturer's procedure. This included the pneumatic tube installation, connection of the pneumatic tube to the automatic traffic detector and setting up of the traffic detector for data logging. Manual counting of vehicles was carried out at a selected time and compared to the ATC traffic data to check the accuracy of the ATC. The department of engineering services and records of eThekweni municipality generated a username and password to access the website for downloading the rain precipitation data. The manual rain gauge was also used to collect rain data which was compared to the eThekweni municipality website rain data.

3.6.1 Sampling

Durban has an estimated yearly average rainfall precipitation amount of 828mm which is the highest in South Africa (Climtemp, 2009). *Despite the variation in annual rainfall across South Africa, Durban has the highest average annual rainfall precipitation of 860mm compare to South Africa Average annual rainfall of 480mm amount.* This shows that rainfall in Durban can represent South Africa's rainfall. The history of rainfall in Durban, South Africa is considered to know the rainy months, which is used in deciding the appropriate time for traffic data collection. The roundabout's usage is also taken into consideration. South Africa is a subtropical region; the climate is warm and temperate. The rainy weather occurs throughout the year. The season with frequent rainfall and high intensity is between the months of September and January. Although the rain precipitation is high in the months of February and March, the frequency is lower than ten rainy days in a month during February and March (Climtemp, 2009).

The traffic data collected during national public holidays, which included the reconciliation, heritage, Christmas, good will, and New Year days were not considered for the study. The traffic data on Monday following a public holiday that falls on Sunday was as well not considered because such holidays were observed on Monday following the public holiday. This data was not considered because of associated traffic from schools, governmental and private organizations during public holidays. Only daylight traffic data was considered for the study to remove the effect of darkness and road lighting from the rainfall effect. The pilot study was carried out in the months of July and August. South Africa National Road Agency (SANRAL) geometric design guidelines stated that roundabouts could be considered when the cumulative traffic flow rate is about 4000pce/h with four approaches to the roundabout. This is considering single lane roundabouts, which implies that the double lane roundabouts should be in the range of 2000pce/h per approach road. the assumption of 2000veh/h was assumed for the entry flow rate.

The level of confidence adopted for this research is 95 percent with 5 percent error margin. Using the inferential statistical Equation 3.1, to estimate the sample size.

$$S = \frac{X^2NP(1-P)}{d^2(N-1)+X^2P(1-P)} \quad [3.1]$$

Where:

S = Sample size required

N = Size of sample population

P = Sample proportion,

X = Z value which is constant as 1.69 for 95% level of confidence.

d = margin of error (5%).

Without going through calculations, **with sample size of 2000veh/h**, the required minimum sample size required at each of entry and circulating roadways per site is determined to be ≥ 322 veh/h, using the Krejcie and Morgan (1970) table (see Appendix F). Irrespective of the rain intensity, this value is applicable to dry and rainy weather conditions. Irrespective of the minimum sample size, the automatic traffic counter was installed at the entry and circulating roadway of the surveyed sites to record traffic data continuously for the period of six weeks at each selected site. This was considered as enough sample size for the study as it covers rainy periods at the selected roundabout sites.

3.6.2 Automatic Traffic Counter and Survey Team

The study was carried out under rainfall, the use of manual counting **was considered unsuitable** because no one could **predict** the exact time of rainfall. The rainfall time, on most internet weather sites, is approximate and depends on probability. In addition to this, the manual counting method **would be** good for short duration surveys and single traffic data collection. The video camera was not used for the traffic data collection because rainfall might impair video visibility as well damage the camera. The Automatic Traffic Counter (ATC) **which is** one of the modern devices for the collection of traffic data was used to collect traffic data at the selected roundabouts with the use of a pneumatic air sensor device. This device **was used to record the traffic volume**, vehicle type,

axle number, headway, speed, gap, the date and time the vehicle moved to the observed points, **the device** has a capacity of collecting traffic data under dry and rainy weather effectively and can collect data continuously for a long period.

The survey team was made up of six people. Four persons were controlling traffic at the roundabout approaches and the circulating roadways for safety reasons. The other two persons carried out the setup of the device at the entry and circulating roadways of the selected roundabouts. The components and the installation of the device are described in sections 3.6.2.1 to 3.6.2.3.

3.6.2.1 Components of Automatic Traffic Counter

The ATC used in collecting traffic data for the research work is a MC5600 vehicle classifier system. The device is a dual air-sensor data logging unit powered by an alkaline battery which gives 290 days continuous use when fully charged. The device collects the traffic volume, speed, headway, gap, vehicle type, and axle number. The component of the device consists:

- **Pneumatic tubes:** This is a black durable and hard wearing pneumatic tube usually installed perpendicular to traffic flow rate direction. Whenever any vehicle struck or hit the tube, it senses the axle by emitting an air pulse and transfers the emitted air pulse to the air sensor device. This tube is shown in Figure 3.10.
- **Air sensor device:** This is the device that senses the air pulse from the pneumatic tube axle sensor and records the details of the individuals' vehicles. This device is shown in Figure 3.11.
- **Steel case:** This case houses the air sensor device, is made of steel, and it provides mechanical protection for the air sensor.

- USB communication cable: This cable allows communication regarding the setup of the device and the downloading of traffic data from the air sensor device to a computer system.
- Road nails: This is a super strength nail. It is less prone to bending when used on the road surface, is 70mm in length. It is used to nail the cleat and center lane flap to the road surface.
- Cleats: These are made of steel and are used for tensioning the road tube.
- Flap: The flap is nailed with the road nail to protect the pneumatic tube from shifting from the observed point, hence providing lateral stability and serving to reduce the tube slap.
- Heavy duty bitumen: This is a super adhesive bituminous surface used in securing the pneumatic tube to the bituminous road surface effectively for the further protection of the pneumatic tube from shifting from the observed point.



Figure 3.10: Axle sensor Pneumatic Tube



Figure 3.11: Air sensor device

3.6.3 Precautionary Measures during Installation

Precautionary measures were taken to avoid damage of the ATC and to ensure accurate data collection. The precautionary measures are:

- The choosing of the observed point for the laying of the pneumatic tube was done in such a way that vehicles could not stop on the tube to avoid tube damage. This precaution was taken at the roundabout entry because vehicles are not expected to stop within the circulatory roadway.
- The tubes were laid where no U-turn could be made on the tube to avoid the effect of a tyre turning on the tube. This measure was taken by laying the tubes at the entry yield line and not at the approach roadway.
- The tubes were laid perpendicular to the traffic flow rate direction to avoid errors in data collection.
- The roadside unit (air sensor device) was placed in plastic to avoid water damaging the device; further it was placed in a wooden box as shown in Figures 3.12 and 3.13. This is to avoid device theft because the manufactured mechanical protection is made of steel. Unofficial information tells that steel is a common interest to thieves in Durban city. This made the steel case not appropriate for protecting the air sensor.
- The roadside unit was placed at 600mm above the ground surface to avoid water penetrating and damaging the device during very heavy rainfall.
- The tube and the roadside unit were not placed across the pedestrian pathways to avoid a potential hazard to pedestrians.



Figure 3.12: Alternative mechanical protection for ATC



Figure 3.13: Alternative mechanical protection for ATC.

3.6.4 Device Installation

The device installation includes the tube installation and the automatic counter set up. The installation is discussed in sections 3.6.4.1 and 3.6.4.2

3.6.4.1 Tube installation

Two sets of pneumatic tubes each were laid for traffic data collection at the entry and circulating roadway. The first pneumatic tube was laid perpendicular to traffic flow rate; this was achieved by making sure that the tube was laid perpendicular to the side kerb of the entry and circulating roadway. End plugs were fixed on the end of the tubes and they were further tightly tied in two knots to avoid the leakage of air pulse emissions from the tube, which could give an error in the traffic data collected by the roadside unit. The end of the tube was tightened with a flap and nailed with a road nail into the road pavement surface, at the two edges of the road, with the use of heavy duty hammer. The tube was stretched at about 10 to 15% of the road width covered, to minimise lateral movement as recommended by the ATC provider. The 1m calibrated wooden rod was placed at the two ends and at the middle of the tubes to separate the tubes at the 1m interval to achieve parallel tubes. After tensioning, the tube was tested by pulling the tube from the road's surface, the tubes pulled back the hand which showed that it was properly tensioned. The 1m interval was checked after tensioning to make sure that the tension did not shift the position of the tubes. The length of the tubes was checked to make sure they were the same length, to avoid errors in speed accuracy and wheelbase results.

Centerline flips were fixed with road nails at two more points each on the tube to further secure the tubes laterally. These points were at the center of each lane of the double lanes, at the entry and circulating roadway, to avoid the flap being in vehicle tire path. Heavy duty bitumen tape was cut at about 200mm by 200mm and fixed on the tube at the interval to further secure tubes against lateral movement. The tubes were connected to the automatic counter device. The first tube to be hit by a vehicle was connected to terminal A of the device while the second tube was connected to the terminal B of the device. The devices were set at the selected roundabout, one at the entry and the other at circulating roadway. The typical sensor configuration is shown in Figure 3.14. There is a little warning that the use of one logger on multi-lane uni-directional may result in an error in the traffic data collection. This may not be so, it can be argued that the first vehicle in the first lane will send the air pulse to the sensor before the vehicle in the second lane due to the air pulse distance travel. The air pressure at the first lane will be stronger and travel faster than the air pressure from a vehicle in the second lane. The speed at the entry and circulating roadways are low, if the vehicle hit the tube simultaneously, the traffic data will be very good. Though, it can

be argued as well that frequent occurrence of simultaneous hits on the pneumatic tubes can reduce the data quality. This can be checked by assessing the single unit convenience for the data, acceptable for the survey site.

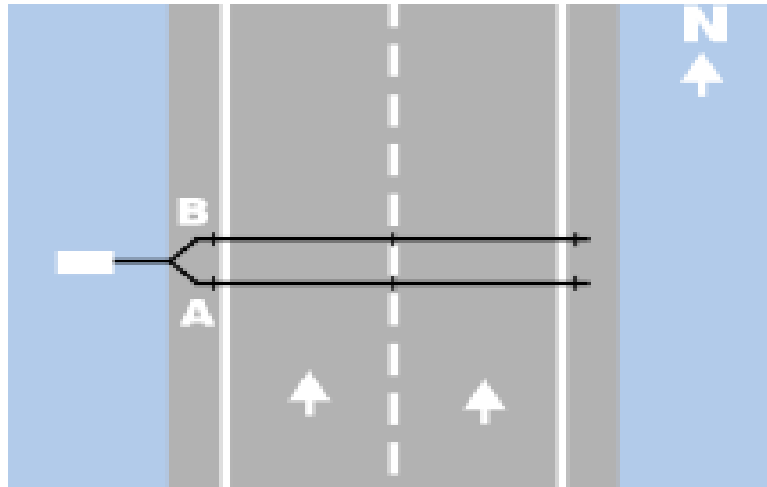


Figure 3.14: Typical sensor configuration

The air sensor device was placed in plastic and, further, in a wooden box and tied to a permanent structure like a signpost or a street light pole. The device was further configured with a laptop to record data as shown in Figure 3.15.



Figure 3.15: Automatic Traffic Counter set-up and configuration.

3.6.4.2 Automatic Traffic counter setup

It is very important to know the status of the ATC before setting up the device for traffic data collection. The status of the battery was checked. Firstly, the continuous blinking light which comes up at every 8 seconds, when the device is in an idle state, shows that the battery is active. The tubes were connected to the air sensor device in the automatic counter. Care was taken to connect the first tube to be hit by a vehicle to the terminal A and the second to terminal B of the sensor device. After the two tubes had been plugged into the terminals, the USB communication cable was used to connect the air sensor device to the laptop. The setup software was activated and the status icon was opened. In the status box, the level of energy and the lifetime of the energy in the battery were checked. The battery status was checked to avoid a shortage in power supply to the device during data collection. The second most important checking carried out was the memory status of the device in order avoid memory shortage during the traffic survey, which might lead to the device cutting off during data collection. The next step was the activation of the setup icon and filling in the site description, which included the site name, the site number, and the direction of traffic flow rate as the vehicle hits the tube as 'AB'. The spacing between the tube, the data collection starting time, and sensor configuration were inputted into the setup dialog box. The setup dialog box is shown in Figure 3.16.



Figure 3.16: Roadside unit dialog box.

Data collection started during the fixed time after the setup box was closed. The measure taken to make sure the device was functional was checking the display of vehicles that hit the tube on the laptop by the selection of the view icon on the setup software. Whenever a vehicle hits the tube, rolling time mode will be displayed on the laptop screen. The device was set to start counting immediately from the date of setup but the data for the first two days was not used in the study to allow drivers on the roundabout to get familiar with the installed tubes, on both the entry and circulating roadway to avoid driver reactions to the tubes. Individual vehicle data was recorded in the air sensor memory unit. The memory status was continually checked at intervals of five days to avoid full usage of the memory, which might make the device stop data collection until unloading the data.

3.6.5 Data Unloading from the Air Sensor

Data unloading is the process of transferring data from the air sensor device to the computer. The traffic data was uploaded from the device through the connection of the air sensor device to the laptop with the use of a USB communicating cable. The data unloading was carried out at the site. The battery blinking light and memory level were always checked before each unloading. The memory of the device was 2 megabytes which could take traffic data for seven to ten days on either the circulating or entry road, depending on the traffic flow rate. The data upload was carried out weekly to know the data quality, avoid full usage of the device memory, and to check the device status.

The process of unloading involved the connection of a laptop to the device with the communication cable. The ATC setup toolbar was selected after the connection; device status was checked as described in section 3.6.4.2. The data upload dialog was selected from the ATC setup page. There were two options for unloading from the device, which included either to unload the data and stop the device, then make another setup to continue with data logging, or to unload the data while the device continues taking the data without stopping. The weekly data unloading was carried out with the option of stopping the device and setting the device up to continue with the data logging. The weekly upload was carried out in order to check the data quality, the device status, and to avoid losing data.

The storage file location was selected, and file names were allocated to each file for easy assessment of the file, before proceeding with the unloading. The proceed button was clicked to continue the uploading of the data. The process of unloading the data was displayed on the computer ATC setup page by showing the plot of each hit on tube A and B. The blue profile line shows there is a good match between A and B, while red or red and blue shows a poor match, and hence poor data quality. Whenever this occurs, there was a need to check the setup, tube leakage, and make a necessary correction if need be.

The individual vehicle data logging information was accessed with ATC report. The logging information of the individual vehicles is displayed with the characteristics as:

- Axle Num: Data file index
- Ht: The Axle hit number in the vehicle
- YYYY-MM-DD: Year, month and day of vehicle axle hit
- hh: mm: ss: Hour, minutes and second of vehicle axle hit
- Dr: Direction of travel on the pneumatic tube (e.g. from tube A to tube B will be shown as AB)
- Speed: Speed of the vehicle
- Wb: Vehicle wheelbase
- Hdwy: Headway
- Gap: Gap between the two successive vehicles
- Rho: Correlation factor of the sensor
- Cl: Vehicle class
- Nm: Not defined
- Vehicle: Name of the class and picture of the vehicle wheel

3.6.6 Problems Encountered During Setup and Data Collection

The installation of the device was unfamiliar to the author; therefore a study of the installation process on the software had to take place. On completion of the study, a trial set up was carried

out within the university campus. The trial set up was used for collecting campus traffic data for five weeks.

The authority to give the approval for the installation on the roundabouts within South Africa could not be easily reached, which led to a delay in getting the approval for the installation. The authority in charge of the approval of installation (eThekweni Transport Authority) requested a demonstration of the setup of the device before approval, because the use of ATC for data collection is not a common practice in South Africa. The demonstration set up was carried out before the final approval.

The other problem that was encountered was the securing of the device at the site because of the unofficial information that steel material attracts theft in Durban city. The manufactured mechanical protection for the device is made of steel, hence it could be a target to the thieves. The device was secured in a plastic box and further secured in a self-made wooden box. The wooden box was padlocked and chained to the nearest permanent structure as shown in Figure 3.12 and 3.13 (subsection 3.6.3). Each time a set-up was to be carried out, the Metro and Crime-stop police were always adequately informed.

The traffic volume was moderate at the selected roundabouts, this meant that traffic control was not an easy task during the device installation, without affecting the flow rate of traffic. After the first installation at the pilot site, the setup in other sites was carried on at very early hours of the morning and on Sundays when the traffic was low to avoid the disturbance of the flow rate of traffic.

The heavy bitumen tape was stolen after the trial test within the campus. It was difficult to replace the bitumen tape on time because of unforeseen logistic delays. Getting the survey assistance was not easy because all efforts to get people employed for the period did not yield any useful results. The undergraduate students in the civil engineering department within the university were used as survey assistants, this was helpful because they needed little training as they already had an idea of traffic data collection.

The washer to hold the flap was out of place after the first usage; a bottled drink metal cover was flattened and used as a washer to hold the nail in place on the flap. The bitumen tape was wearing out within a short period of time due to a frictional effect between tyres and the bitumen tube; the bitumen tape was checked regularly and replaced whenever it was worn. At one of the sites, data logging was not set up after uploading. An additional week's data was collected to cover for the week lost in data collection.

3.6.7 Typical Site Layout

The pneumatic tubes were laid and connected to the air sensor device of the automatic traffic counter (ATC) at the entry and the circulating roadway as shown in Figure 3.17.

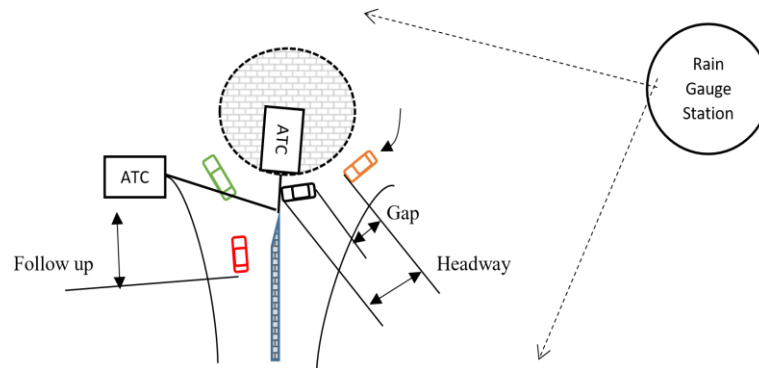


Figure 3.17: Typical site layout.

3.6.8 Traffic volume survey

Traffic volume is the total number of vehicles that passes through an observed point at any given period. This parameter is one of the useful parameters of the study. It was collected at both the entries and circulating roadways of the selected roundabouts. Whenever a vehicle hit the installed pneumatic tubes air pulse was created which were sensed by the ATC and the vehicle was counted. The device assigns the vehicles to each vehicle class, taking into consideration the wheelbase of the vehicle. The surveyed traffic vehicles were converted to traffic flow rate with the use of passenger car equivalent values.

3.6.9 Headway Survey

The headway is the difference in time it takes the front axle of the preceding vehicle and the front axle of the vehicle following in the same direction to hit the pneumatic tube. The sensor measures it by the difference in the time between the first axle hit on the first pneumatic tube A of a leading vehicle and the immediately following vehicle. This was processed by the ATC and recorded by the device until it was unloaded into the computer.

3.6.10 Vehicle Classification survey

There are many types of vehicle classifications around the world. The vehicle classifications in the automatic traffic count software are ten in number. South Africa classify vehicles into passenger cars, trucks, and buses. Passenger cars include light vehicles, utility vehicles for recreation, subcompact and compact vehicles, and light delivery vehicles. The truck includes the single unit trucks, trucks, tractors with trailers attached, and trucks or tractors with semi-trailer combined. The buses classification includes single unit buses and intercity buses. South African vehicle classifications are not among the enlisted vehicle classification schemes in the device software. However, the main aim of classifying vehicles for this research work is to classify vehicles into the light, medium, and heavy vehicles for the estimation of PCE for the conversion of heterogeneous traffic volume to homogeneous traffic flow rate. Irrespective of the type of classification used, the vehicles can still be classified as light, medium, and heavy vehicles. Where light vehicles are passenger cars and motor vehicles with or without a trailer but this excludes heavy vehicles. Medium vehicles are a heavy vehicle with a minimum of one heavy axle which is designed for the conveyance of minimum of 16 passengers or freight by the design of adaptation and with a maximum of 3 axles. Heavy vehicles are large or extra-large vehicles with three to five axles or more. Typical vehicle classification that was adopted in this research work was the AXL as shown in Table 3.4.

Table 3.4: **AXL Vehicle Classification.**

Axles	Description	Class		Aggregate
2	Very Short - Bicycle or Motorcycle	MC	1	
2	Short - Sedan, Wagon, 4WD, Utility, Light Van	SV	2	1 (Light)
3, 4 or 5	Short Towing - Trailer, Caravan, Boat, etc.	SVT	3	
2	Two axle truck or Bus	TB2	4	
3	Three axle truck or Bus	TB3	5	2 (Medium)
>3	Four axle trucks	T4	6	
3	Three axles articulated vehicle or Rigid vehicle and trailer	ART3	7	
4	Four axles articulated vehicle or Rigid vehicle and trailer	ART4	8	
5	Five axles articulated vehicle or Rigid vehicle and trailer	ART5	9	
>=6	Six (or more) axle articulated vehicle or Rigid vehicle and trailer	ART6	10	3 (Heavy)
>6	B-Double or Heavy truck and trailer	BD	11	
>6	Double or triple road train or Heavy truck and two (or more) trailers	DRT	12	

Source: Metro count software, 2013.

3.6.11 Rainfall Survey

The rainfall data was collected from the eThekweni website (MCSysstem, 2009) which belongs to the eThekweni Municipality. This governmental agency has 43 rain gauges in Durban city and its environment (see appendix C). The rain gauges were internet-connected and upload the amount of precipitation on the website for easy accessibility. The department generated a username and a password to the researcher for access to the website.

The coordinates of the roundabout were determined with the use of Google Maps. The rain gauges' locations were visited, and the coordinates were recorded with Google Maps. These coordinates were used in determining the distance of the rain gauges from the roundabouts with the use of Google Earth. The roundabouts within the catchment area of the closest rain gauge were considered for the survey. The amount of rainfall for each of the investigated roundabouts was collected from the nearest rain gauge that has a catchment area that covers the surveyed roundabouts. The amount of precipitation was measured in mm for every five minutes to take care of fluctuation in rainfall amount during a specific rainfall period. The amount of rainfall collected was converted to intensity by dividing the rainfall amount with the period covered. This rain data was synchronized with the traffic data to obtain the rain-traffic data. The distance of the rain gauges from the surveyed roundabouts is shown in Table 3.5.

Table 3.5: Rain gauge distance from roundabout sites

Site	Site name	Distance from rain gauge (km)
01	Armstrong roundabout	0.95
02	Millennium Roundabout	1.18
03	Douglas Roundabout	0.82
04	Gateway roundabout	0.75

3.6.11.1 Rainfall Classification for the Study

The rainfall intensity data collected was classified into light rainfall (LR), with rain intensity < 2.5mm/h; moderate rainfall (MR), with intensity 2.5 – 10mm/h; and heavy rainfall (HR), with the intensity of 10 – 50mm/h, in accordance with the World Meteorological Organisation (WMO). Very heavy rainfall with intensity > 50mm/h was not considered in this research work because of drag force effect on tires, aqua planning, and splash on windscreen which might induce anger and anxiety into drivers. Hence, influencing drivers' behaviour and this effect would be difficult to separate from the rainfall effect.

3.6.12 Geometric Data Survey

Geometry is one of the factors that contribute to the performance of a roundabout and is one the factors that can influence the roundabout functional quality of service. It has to conform to the SANRAL geometric design guideline for acceptability in the study. The geometry of the surveyed roundabouts were collected on the surveyed sites by direct measurement. The collected geometry was the entry width (m), entry lane width (m), approach road width (m), entry radius (m), entry angle ($^{\circ}$), effective flare length (m), inscribe circle diameter (m), the number of the entry lanes, and the number of the circulating lanes.

These parameters were collected at each of the surveyed roundabouts. The collected geometric data was cross-checked with the design drawing provided by the eThekweni municipality planning department and a further check was carried out with Google Maps measurements. The geometry of the surveyed roundabouts is shown in Table 3.3 (subsection 3.5.1).

3.7 Sample Data Appraisal and Analytical Method

It is important to carry out a pre-study, which is known as a pilot study, before proceeding to the large-scale study. This is important to test the model statistically, which may give an insight into the model behaviour. It will help in making an adjustment to the model if need be and assist in

giving approaches and ideas that might not have been known before conducting the pilot study. If any of these ideas occur, it will assist in getting a clearer outcome in the main study and serve as an opportunity to check the statistical and the analytical procedure that would be used in the study. The pilot study will also assist in judging the effectiveness of the statistical and analytical procedure for the data, which makes the data analysis in the main study more efficient. It will help in reducing unforeseen problems because, during the pilot study, there is an opportunity to adjust the approach to the study. It allows room to test as many alternative measures before concluding on the measure to be used in the study. It serves as a medium of knowing if there is any problem with the ATC's functionality and the site layout before proceeding to the main study.

One pilot study was carried out. The geometric, traffic, and rain data was collected as described in section 3.6. Motorcycles were not considered for traffic in the study because hardly any motorcycle would be captured under heavy rainfall conditions. Motorcycles have little or no effect on vehicle headway under the traffic free flow rate condition. In addition, the study considered the off-peak period where there will be free flow rate to avoid the additional influence of peak period, which might be difficult to separate from rainfall.

The traffic data was processed with the use of ATC report and it gives the traffic characteristics of each vehicle presented in Table 3.4. The data was unloaded into Microsoft excel for further processing. The procedures for processing the data in Microsoft excel are:

- The night traffic data was separated from the daylight traffic. The daylight traffic was taken in between 07:00 and 17:00 The traffic data between 17:01 and 06:59 was not considered because the darkness has an influence on traffic behaviour which might be difficult to separate from rainfall effect.

- The rainfall amount of precipitation from the Department of Engineering and Records Department of the eThekweni municipality's website was converted in to rainfall intensity. The rainfall intensity was used to group the rainfall into the light, medium, and heavy rainfall, in line with the World Meteorological Organization's (WMO) classification.

- The traffic flow rate at the time of each group of rainfall classification was synchronized. The traffic flow rate corresponding with the same time and day of rainfall in each week, under consideration, were collated. Traffic flow rate, with the corresponding rainfall class and dry weather, was separated in different sheets in excel workbook.
- The traffic flow rate for dry weather and rainy conditions was grouped into five-minute intervals to match with the rainfall data. The PCE value was applied to each of the vehicles as shown in the Figure 3.18.
- The macroscopic traffic flow rate was obtained for both entry and circulating roadways under dry and rainy conditions.

	B	C	D	E	F	G	H	I	M	N	P
187	YYYY-MM-DD	hh:mm:ss	Dr	Speed	Wb	Hdwy	Gap	Ax	PCE	Vehicle\par	
188	01/10/2016	14:10:31	AB	15.5	2.8	3.2	2.7	2	1	SV	
189	01/10/2016	14:10:33	AB	15.9	2.5	1.5	0.8	2	1	SV	
190	01/10/2016	14:10:52	AB	11.4	4.5	9.1	7.9	2	1	SV	
191	01/10/2016	14:10:54	AB	12	9.3	2.7	0.8	2	2.8	TB2	
192	01/10/2016	14:11:10	AB	11.7	2.3	15.3	12.6	2	1	SV	
193	01/10/2016	14:11:39	AB	17.6	15.2	29.2	27.8	3	2.8	ART3	
194	01/10/2016	14:11:44	AB	20.8	2.8	5	1.9	2	1	SV	
195	01/10/2016	14:11:47	AB	21.7	2.7	3.6	3.1	2	1	SV	
196	01/10/2016	14:11:53	AB	37.6	2.7	5.9	5.5	2	1	SV	
197	01/10/2016	14:11:57	AB	26	3.1	4	3.7	2	1	SV	
198	01/10/2016	14:12:00	AB	20.1	2.8	2.3	1.8	2	1	SV	
199	01/10/2016	14:12:04	AB	15	2.5	2.4	1.7	2	1	SV	
200	01/10/2016	14:12:06	AB	14.5	2.5	2.4	1.8	2	1	SV	
201	01/10/2016	14:12:08	AB	15.9	1.4	2.2	1	2	1	SV	
202	01/10/2016	14:12:14	AB	12.1	2.7	3.7	2.4	2	1	SV	
203	01/10/2016	14:12:16	AB	13.5	3	2.1	1.3	2	1	SV	
204	01/10/2016	14:12:19	AB	12.8	2.7	2.5	1.7	2	1	SV	
205	01/10/2016	14:12:24	AB	10.8	2.8	5.1	4.3	2	1	SV	
206	01/10/2016	14:12:26	AB	16.6	2.7	2.4	1.5	2	1	SV	
207	01/10/2016	14:12:37	AB	23.8	2.6	3.9	3.3	2	1	SV	

Figure 3.18: Excel worksheet sample for data processing.

3.7.2 Sample Appraisal for Pilot Study at Site PST01

The pilot study was carried out at the Torsvale roundabout. Traffic data was collected for a total period of six weeks, from July 2016 to August 2016. The survey was carried out on rainfall, where the rainfall precipitation amount was collected from the eThekweni website (MCSysyem, 2009), the rain gauge with station ID umhnth (see appendix C) has the catchment area that covers the roundabouts' locations. The rainfall was classified into light rain (LR), with intensity (i) less than 2.5mm; moderate rainfall, with an intensity greater than 2.5mm/h and less than or equal 10mm/h;

and heavy rainfall, with an intensity greater than 10mm but less or equal 50mm/h; as well as very heavy rainfall with an intensity greater than 50mm/h. Though, very heavy rain was not considered in the study. Traffic data was surveyed continuously for a total period of six weeks by collecting traffic volume, vehicle type, speed, and headway of individual vehicles.

3.7.3 Analytical Method

The major consideration of this study is centered on functional quality of service. Functional quality of service has been shown to be a multi-parameter that considers both the user's and provider's perspective. The user's perspective parameters are the delay and queue length while the provider's perspective parameters are the degree of saturation and reserve capacity (Q_R), as discussed in chapter 2.

In the estimation of any of these parameters for the assessment of the roundabouts' functional quality of service (FQS), entry capacity needs to be estimated.

It was mentioned in chapter 2 that entry capacity could be estimated using gap acceptance and the empirical method. This study is an empirical study; hence the gap acceptance will not be adopted because is theoretic approach but rather an empirical method. The stepwise method is followed for simplicity.

Step 1: The collected entry and circulating traffic data was converted to traffic flow rate with the application of SANRAL PCE values of 1.0 for passenger cars, 2.8 for medium vehicle (MV) and 2.8 for heavy vehicles (HV). The traffic flow rate was grouped into twelve periods and the highest hourly entry flow rate with the corresponding circulating flow rate was considered for the study. The hourly peak entry and circulating flow rate data is as shown in table 3.6 for the development of a criterial table for functional quality of service assessment.

Table 3.6: Peak entry and circulating traffic flow rate

Flow rate (pce/h)	Period											
	1	2	3	4	5	6	7	8	9	10	11	12
Circulating	712	712	492	501	897	619	679	463	741	969	888	979
Entry flow rate	1276	1288	1360	1360	1360	984	1300	1301	1397	1276	936	888

Step 2: There are two options that could be used for empirical regression analysis which includes the linear and exponential regression. Both options were tested to decide which the best fit for this study was. The scatter diagram, and the best fit line graph for the linear and exponential functions along with the model equations for the selected data is shown in figure 3.19.

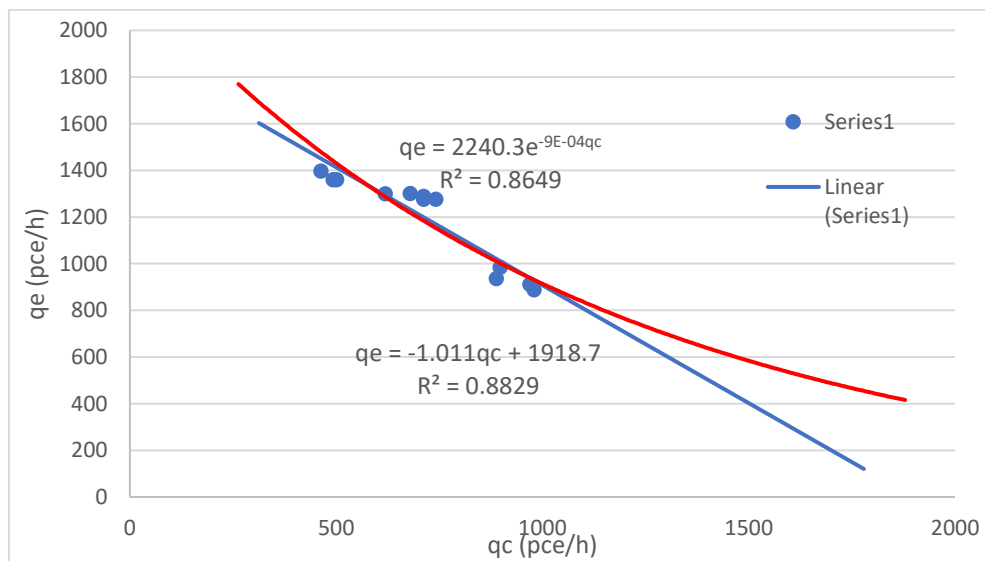


Figure 3.19: Circulating flow rate versus entry flow rate.

The exponential model equation is:

$$q_e = 2240e^{-9 \times 10^{-4} q_c} \quad R^2 = 0.87 \quad [3.2]$$

The linear model equation is:

$$q_e = 1919 - 1.01q_c \quad R^2 = 0.88 \quad [3.3]$$

The coefficient of the determinant (R^2) for the two model equations is more than 0.5 which shows that the two model equations are reliable because more 50% of the data are significant. The R^2 for the exponential model equation is 0.87 while that of linear model equation is 0.88.

Step 3: The other statistical testing conducted for the linear regression includes the p-value, t-test, and f-test. This was carried out at a 95 percent level of confidence; the result is presented in Table 3.7.

Table 3.7: Summary of ANOVA result

SUMMARY OUTPUT								
<i>Regression Statistics</i>								
Multiple R	0.939631108							
R Square	0.882906619							
Adjusted R Square	0.871197281							
Standard Error	70.53626091							
Observations	12							
<i>ANOVA</i>								
	<i>df</i>	<i>SS</i>	<i>MS</i>	<i>F</i>	<i>Significance F</i>			
Regression	1	375152.0256	375152	75.40192	5.7031E-06			
Residual	10	49753.64104	4975.364					
Total	11	424905.6667						
	<i>Coefficients</i>	<i>Standard Error</i>	<i>t Stat</i>	<i>P-value</i>	<i>Lower 95%</i>	<i>Upper 95%</i>	<i>Lower 95.0%</i>	<i>Upper 95.0%</i>
Intercept	1918.729472	86.37542943	22.21383	7.67E-10	1726.27302	2111.1859	1726.273022	2111.18592
qc (pce/h)	-1.010951649	0.116423103	-8.68343	5.7E-06	-1.27035849	-0.7515448	-1.27035849	-0.75154481

The result in table 3.7 shows that the R^2 is more than 0.5 which shows that the variability between the variables is good and that they are statistically significant. The t-test is more than 2.2 which shows that the parameters are significant, and the f-test is greater than 4.84 which shows that the model equation did not occur by chance. The p-value is less than 0.05 which shows that the variance is equal. The linear model equation is statistically satisfactory and could be used for prediction.

Step 4: The entry capacity occurs when the circulating flow rate (q_c) = 0, substituting for this in equations 3.2 and 3.3,

$$q_e = 2240e^{-9 \times 10^{-4} q_c} = 2240e^{-9 \times 10^{-4}(0)} = 2240 \text{ pce/h} \quad (\text{from equation 3.2})$$

$$q_e = 1919 - 1.01q_c = 1919 - 1.01(0) = 1919 \text{ pce/h} \quad (\text{from equation 3.3})$$

The difference between the entry capacity estimated from the linear and exponential models = 249pce/h

In order to test for significant differences between modified, linear and exponential capacity values two hypotheses are made between the linear and exponential capacity values.

The hypotheses are:

- (i) Null hypothesis (H_1): The entry capacity values are the same.
- (ii) Alternate hypothesis (H_2): The entry capacity values are not the same.

The test is carried out with a chi-square using the chi-square equation 3.4

$$X^2 = \frac{(o-e)^2}{e} \quad [3.4]$$

Where:

X^2 = chi-square, o = observed value, e is the expected value

The test is carried out at a 95 percent level of confidence, where $X^2 < 3.84$ means there is no significant difference between the two variables.

Assuming the linear capacity value is the expected value = 1919 pce/h, the exponential capacity value is the observed = 2240 pce/h

$$X^2 = \frac{(2240-1919)^2}{1919} = 53.70 > 3.84$$

Hence the null hypothesis is rejected, and the alternative hypothesis is accepted whereby the entry capacity values are not the same.

Figure 3.19 shows that at a capacity lower than 400 pce/h, the circulating flow rate is discontinuous with an exponential relationship which suggests it has become nearly asymptotic to the circulating flow rate axis. This makes the exponential relationship unreliable in estimating the entry flow rate with high circulating flow rate and it over-estimates the entry capacity by 249 pce/h. Very low entry flow rate is a traffic scenario that can occur under rainy conditions, this makes the exponential regression unfit for this study. These two models only consider the entry and circulating flow rate, but the roundabout geometry is an important factor which has an influence on the roundabout entry capacity.

Step 5: The key parameters are estimated using the HCM capacity model in this step for comparison of the linear and exponential models to the HCM capacity model.

The HCM capacity model is:

$$Q_e = 1130e^{-0.007q_c} \quad [3.5]$$

$$\text{The follow-up time } (t_f) = \frac{3600}{A} \quad [3.6]$$

Where A is the y-intercept of the exponential equation.

The intercept in the HCM model equation = 1130

$$\text{Hence, the } t_f = \frac{3600}{1130} = 3.19\text{s}$$

Using equation 3.5, the entry capacity occurs when there is no circulating flow rate.

Substituting 0 for q_c in equation 3.6.

$$Q_e = 1130e^{-0.007(0)} = 1130\text{pce/h/lane}$$

Step 6: The empirical exponential model is compared with the HCM model.

The entry capacity using the empirical exponential model equation = 2240pce/h = 1120pce/h/lane < 1130pce/h/lane using the HCM model.

The follow-up headway (t_f) = $\frac{3600}{1120} = 3.21$ s

This shows that the t_f using the empirical exponential model > 3.19s (the t_f using the HCM model).

Figure 3.19 in step 2 shows that when the entry flow rate is < 400pce/h, the circulating flow rate is discontinuous which suggests that the empirical exponential regression is nearly asymptotic to the x-axis. As it does not have an x-intercept. This also shows that the model is unreliable for the estimation of low entry capacity when the circulating flow rate is high.

Step 7: The empirical linear regression model is compared to the HCM capacity model in this step.

The linear model is corrected with correction factor for geometry effect. The correction factor k value was estimated with equation 3.7, Where, $r = 35$ m, $\varphi = 45$ deg.

$$k = 1.151 - 0.00347\varphi - \frac{0.978}{r} \quad [3.7]$$

Substituting for φ and r , $K = 0.93$

The correction factor was applied to the model equation 3.3 and the model equation becomes;

$$q_e = 0.93(1919 - 1.01)q_c \quad [3.8]$$

$$q_e = 1785 - 0.94 q_c \quad [3.9]$$

The entry capacity is estimated by assuming that the circulating flow rate is zero, though this rarely occurs in nature. Substituting for $q_c = 0$ in equation 3.9.

Then entry capacity, $Q_e = 1785 - 0.94(0) = 1785$ pce/h = 893pce/h/lane < 1130pce/h (HCM, 2010).

The follow-up time headway is estimated using equation 3.6.

$$t_f = \frac{3600}{893} = 4.03s > 3.19s \text{ (HCM, 2010)}$$

Step 8: The exponential and linear regression models are compared.

The follow-up headway using the exponential model = 3.21s > 3.19s

The follow-up headway using the linear model = 4.03s > 3.19s

The entry capacity using the exponential model = 1120pce/h < 1130pce/h

The entry capacity using the linear model = 893pce/h < 1130pce/h

The linear and exponential models predict the entry capacity lower than the HCM capacity and the follow-up time headway is higher than the HCM follow-up time headway.

The circulating capacity is predicted when there is no entry flow rate.

Using the empirical linear model equation 3.9.

$$0 = 1785 - 0.94 q_c$$

$$Q_c = \frac{1784}{0.94} = 1899 \text{ pce/h for two lanes} = 949 \text{ pce/h/lane}$$

Using the exponential model equation 3.2.

$$q_e = 2240e^{-9 \times 10^{-4} q_c}$$

When there is no entry flow rate,

$$Q_c = \frac{\ln\left(\frac{0}{2400}\right)}{0.0009} = \infty$$

This shows that the exponential model cannot be used to predict the circulating capacity as it shows that the circulating capacity has no limit, which is never the case. More so, the scenario of very low or no entry vehicles may occur under rainy condition.

The linear model is the preferred model because it can be used in the estimation of very low entry capacity, circulating capacity and the geometry effect also makes it suitable for the prediction of entry capacity.

Step 9: The delay and queue length are estimated for the development of a criterial table and a sensitivity test is conducted using the estimated time headway as:

$$d = \frac{3600}{k(F-f_c Q_c)} + 900T \left[(x-1) + \sqrt{(x-1)^2 + \frac{\left(\frac{3600}{k(F-f_c Q_c)}\right)^x}{450T}} \right] + 5 \quad [3.10]$$

$$L = d \left(\frac{q_e}{3600} \right) \quad [3.11]$$

Note that:

$$\frac{3600}{k(F-f_c Q_c)} = 4.03 \text{ sec (from step 7), } T = 15 \text{ min} = 0.25 \text{ hr, Hence } 900T = 225 \text{ sec, } 450T = 112.5 \text{ sec.}$$

Substituting for $\frac{3600}{k(F-f_c Q_c)}$, $900T$ and $450T$ in Equation 3.10

$$d = 4.03 + 225 \left[(x-1) + \sqrt{(x-1)^2 + \frac{4.03x}{112.5}} \right] + 5$$

A sensitivity test is conducted for this model equation by setting the degree of saturation to zero and 1.0. Table 3.8 shows that at degree of saturation of zero when there is no entry vehicle, the delay under this condition is a purely geometric delay with the value of 9.03s. At capacity, when the degree of saturation is 1, the queue length is 13 vehicles and the delay 51.88 s.

Table 3.8: Summary of sensitivity test result for pilot test

Q_e (pce/h)	Q_e (pce/h/lane)	x	d (s)	L (veh)
1785	893	0.00	9.03	0
1785	893	1.00	51.88	13

Note: Q_e is entry capacity, x is degree of saturation, d is the delay and L is the queue length.

The degree of saturation is divided into ten equal parts of 0.1 each. The delay and corresponding queue are estimated. The summary is presented in table 3.9.

Table 3.9: Summary of degree of saturation, and delay

x	d (s)	L (veh)
0	9.03	0
0.1	9.48	0
0.2	10.05	1
0.3	10.76	1
0.4	11.71	1
0.5	13.01	2
0.6	14.92	2
0.7	17.91	3
0.8	23.07	5
0.9	32.93	7
1.0	51.88	13

Note: x is degree of saturation, d is the delay and L is the queue length.

The division of degree of saturation into ten divisions might be unrealistic in forming the FQS class because of the closeness in delay values and there might be an overlap in values of the delay parameter in each class. There is no method of checking the overlapping of parameter values in each class if it occurs because of a single unit division. In view of this, the division for the FQS table of five equal divisions of degree of saturation of 0.2 each is adopted.

To avoid an overlap of each division, the standard deviation is estimated for each class. The standard deviation is applied to determine the extent of the deviation that could be within the lower and the upper limit of each class. σ and μ are the mean and standard deviations of each division. Taking the data boundary number of 1 for simplicity, then -1σ and 1σ are the upper and lower boundaries of each division.

The class of degree of saturation of to 0.2 with the corresponding delay and queue length are shown in table 3.10.

Table 3.10: The parameter for class of degree of saturation 0 to 2

x	d (s)	L (veh)
0	9.03	0
0.1	9.48	0
0.2	10.05	1

Note: x is degree of saturation, d is the delay and L is the queue length.

The mean (σ) delay = $\frac{9.03+9.48+10.05}{3} = 9.52$ s

The standard deviation **(G)** = 0.51s (Estimated with Microsoft excel)

The lower limit is 9.01s and the upper limit is 10.03s. The upper limit does not overlap the delay at a degree of saturation of 0.3, but the same queue length of 1 vehicle occurs at a degree of saturation of 2 to 4. which is the second division. Hence these two divisions are merged together as one class.

The class of degree of saturation of 0 to 0.4 with the corresponding delay and queue length are shown below in table 3.11.

Table 3.11: The parameter for class of degree of saturation 0 to 0.4

x	delay (s)	L (veh)
0	9.03	0
0.1	9.48	0
0.2	10.05	1
0.3	10.76	1
0.4	11.71	1

Note: x is degree of saturation, d is the delay and L is the queue length.

$$\text{Mean } (\sigma) \text{ delay} = \frac{9.03+9.48+10.05+10.76+11.71}{5} = 10.21\text{s}$$

$$\text{Standard deviation } (\sigma) = 1.06\text{s}$$

$$\text{The lower boundary} = 10.21 - 1.06 = 9.15 \approx 9\text{s}$$

$$\text{The upper boundary} = 10.21 + 1.06 = 11.27 \approx 11 \text{ s}$$

This division is taken as FQS A class

The next division is the class of degree of saturation of 0.5 and 0.6 where the corresponding delays are 13.01s and 14.92s.

$$\text{Mean } (\sigma) \text{ delay} = \frac{13.01+14.92}{2} = 13.97\text{s}$$

$$\text{Standard deviation } (\sigma) = 1.35\text{s}$$

$$\text{The lower boundary} = 13.97 - 1.35 = 12.61 \approx 13\text{s}$$

$$\text{The upper boundary} = 13.97 + 1.35 = 15.32 \approx 15\text{s}$$

The lower boundary does not overlap with the upper boundary of class FQS A and the upper boundary does not overlap with the delay at the degree of saturation of 0.7. The queue length is two vehicles (from table 3.9). This division is taken as FQS B.

The next division is volume capacity ratio of 0.7 and 0.8 with the delay values of 17.91s and 23.07s. The mean value = 20.49s and the standard deviation = 3.65s.

$$\text{The lower boundary} = 20.49 - 3.65 = 16.84\text{s} \approx 17\text{s}$$

$$\text{The upper boundary} = 20.49 + 3.65 = 24.14\text{s} \approx 24\text{s}$$

The lower limit is more than 15s which is the upper limit of FQS B and the upper limit is less than 32.93s with a degree of saturation of 0.9. The queue length does not overlap. This class is taken as FQS C.

The threshold is set at the degree of saturation of 0.9. This class is to alert that the roundabout is operating close to the capacity and the delay is $32.93s \approx 33s$ with a corresponding queue length of seven vehicles (from table 3.9). This class is taken as FQS D.

When the roundabout is operating at capacity, the degree of saturation is one and the delay is $51.88s \approx 52s$ with a queue length of 13 vehicles. This division is taken as FQS E.

When the roundabout is operating above the capacity, then the delay is more than 52s and the degree of saturation is > 1 . This is set as a class of FQS F.

The reserve capacity for the lower and upper limit is estimated using equation 3.12.

$$Q_R = \frac{Q_e - q_e}{Q_e} \quad [3.12]$$

The corresponding delay, degree of saturation, reserve capacity, and queue length of the lower and upper boundary of each division are used in the development of a functional quality of service criterial table in table 3.12.

Table 3.12: The functional quality of service criterial table

FQS	d (s)	Q_R	x	L (veh)
A	$d \leq 11$	$Q_R \geq 0.6$	$x \leq 0.4$	1
B	$11 < d \leq 15$	$0.4 \leq Q_R < 0.6$	$0.4 < x \leq 0.6$	2
C	$15 < d \leq 23$	$0.2 \leq Q_R < 0.4$	$0.6 < x \leq 0.8$	$2 < L \leq 5$
D	$23 < d \leq 33$	$0.1 \leq Q_R < 0.2$	$0.8 < x \leq 0.9$	$5 < L \leq 7$
E	$33 < d \leq 52$	$0.1 \leq Q_R < 0$	$0.9 < x \leq 1$	$7 < L \leq 13$
F	$d > 52$	$Q_R < 0$	$x > 1$	$L > 13$

Note: d = delay, x is degree of saturation, Q_R denotes normalized reserve capacity, and L is the queue length

Step 10: The criterial table is compared to other methods of service delivery assessment as shown in table 3.13. The table shows that the SIDRAL delay at capacity is 70s, The HCM delay at

capacity is 50s which has a close value to the empirical delay at capacity (52s). The empirical delay is in-between the SIDRAL and HCM delay at capacity. At the other classes the upper limit of the empirical delay is in-between the HCM and the SIDRAL delay. **This shows delays values estimated using different models are different from each other**

Using the degree of saturation, the class A of SIDRA is divided into two classes in the empirical method (Class A and B). The threshold class in SIDRA is class C with a degree of saturation of 0.85 and the empirical method is class D with a degree of saturation of 0.9. There is no basis for a reserve capacity comparison because of its novelty.

Table 3.13: Criteria of roundabout quality of service compared

Class	d (s)			Reserve Capacity (Q_R)	x	
	Empirical	HCM 2010	SIDRA	Empirical	Empirical	SIDRAL
A	$d \leq 11$	$d \leq 10$	$d \leq 10$	$Q_R \geq 0.6$	$0 < x \leq 0.4$	$0 < x \leq 0.6$
B	$11 < d \leq 15$	$10 \geq d \leq 15$	$10 \geq d \leq 20$	$0.6 > Q_R \geq 0.4$	$0.4 < x \leq 0.6$	$0.6 < x \leq 0.7$
C	$15 < d \leq 23$	$15 \geq d \leq 25$	$20 \geq d \leq 35$	$0.4 > Q_R \geq 0.2$	$0.6 < x \leq 0.8$	$0.7 < x \leq 0.85$
D	$23 < d \leq 33$	$25 \geq d \leq 35$	$35 \geq d \leq 50$	$0.2 > Q_R \geq 0.1$	$0.8 < x \leq 0.9$	$0.85 < x \leq 0.95$
E	$33 < d \leq 52$	$35 \geq d \leq 50$	$50 \geq d \leq 70$	$0.1 > Q_R \geq 0$	$0.9 < x \leq 1.0$	$0.95 < x \leq 1.0$
F	$d > 52$	$d > 50$	$d > 70$	$Q_R < 0$	$1 < x$	$1 < x$

Note: d is delay, Q_R is normalized reserve capacity, x is degree of saturation or volume capacity ratio

Step 11: The degree of saturation or volume to capacity ratio (x), the reserve capacity (Q_R) and delay at the roundabout are estimated as illustrated in step 7, the values are:

$$x = \frac{q_e}{Q_e} = \frac{1552}{1785} = 0.86, Q_R = \frac{1785-1552}{1785} = 0.13 \text{ and delay} = 28.06s \approx 28s, \text{ queue length} = 6\text{veh. This}$$

roundabout is operating at FQS D at the peak period under dry weather conditions.

Step 12: To test the hypothesis set in chapter 2, the off-peak traffic data for dry, light, moderate, and heavy rain is used for this purpose.

The entry capacity is calculated as described in step 2 to 4. The rainy and dry weather traffic data were combined to see the effect of rainfall on capacity. The linear multiple regression is used to develop the model equations in the form of:

$$q_e = k(F - f_c q_c - \epsilon) \quad [3.13]$$

Where; k , F , f_c , q_c are as defined in equation 2.44 and 2.50 in chapter 2 and ϵ is the dummy variable, $\epsilon = 1$ under the rainy conditions and otherwise. The rain and dry traffic data were combined with dummy variable as, dry and light rain, dry and moderate rain, and dry and heavy rain.

The developed model equations are:

$$q_{eL} = 1983.5 - 0.99q_c - 110\epsilon \quad R^2=0.88 \quad [3.14]$$

$$q_{eM} = 2038 - 1.04q_c - 270\epsilon \quad R^2=0.84 \quad [3.15]$$

$$q_{eH} = 1900 - 0.92q_c - 379\epsilon \quad R^2=0.85 \quad [3.16]$$

L, M, and H denote the light, moderate, and heavy rain conditions.

The model equations were tested statistically at a 95% level of confidence. The R^2 is more than 0.5, the t-test is more than 2.2, the F-test is more than 4.84 for all the model equations. This shows that the equation is statistically satisfactory, and it can be used for predictions.

The model equations after the application of $K = 0.93$ as estimated in step 4 is presented in equation 3.17 for dry and light rain conditions.

$$q_{eL} = 1846 - 0.92q_c - 102\epsilon \quad [3.17]$$

The entry capacity is estimated by setting $q_c = 0$, as an example for dry and light rain conditions.

For dry weather, $Q_{edry} = 1846 - 0.92(0) - 102(0) = 1846 \text{ pce/h}$.

For light rain, $Q_{eL} = 1846 - 0.92(0) - 102(1) = 1744 \text{ pce/h}$.

The circulating capacity, Q_{cdry} is estimated by setting $q_e = 0$ in equation 3.17.

$0 = 1846 - 0.92q_c - 102(0)$, $Q_{cdry} = 1744 \text{ pce/h}$.

This same procedure was used for the estimation of capacity for dry, light, moderate and heavy rain. The summary of the results is shown in table 3.14.

Table 3.14: Summary of entry and circulating capacity

Model equation without modification	Entry capacity (Q _e) pce/h		ΔQ _e	Circulating capacity (Q _c) pce/h		ΔQ _c
	Dry	Rain		Dry	Rain	
	$Q_E = 1985 - 0.99Q_C - 110\epsilon_L$	1846	1744	102	2005	1894
$Q_E = 2038 - 1.04Q_C - 270\epsilon_M$	1895	1644	251	1960	1700	260
$Q_E = 1900 - 0.92Q_C - 379\epsilon_H$	1767	1414	353	2065	1653	412

Note: Q_e is entry capacity, Q_C is circulating capacity.

There are three values for entry and circulating capacity under dry weather condition. There is need to test if there is a significant difference in these values. To test this, two hypotheses are made between the capacity under dry weather conditions.

The hypotheses for capacity are:

- (iii) Null hypothesis (H₁): The values of the capacity are the same.
- (iv) Alternate hypothesis (H₂): The capacity values are not the same.

$$X^2 = \frac{(o-e)^2}{e}$$

Where:

X^2 = chi-square, o = observed value, e is the expected value

The test is carried out at a 95% level of confidence, where $X^2 < 3.84$ means there is no significant difference between the two variables.

The sample calculation using entry capacity under dry/ light rain, and dry/ heavy rain models, taking the capacity under dry/light rain as the expected entry capacity and the capacity under dry/heavy rain as the observed capacity, then:

$$O = 1767\text{pce/h}, e_1 = 1846\text{pce/h}$$

Substituting for O and e, the chi-square is:

$$X^2 = \frac{(1767-1846)^2}{1846} = 3.38 < 3.84$$

The null hypothesis (H_1) is accepted and the alternate hypothesis (H_2) is rejected, these show there is no significant difference between the estimated entry capacity under dry condition in the models. The test was carried out for circulating capacity and the chi-square shows that there is no significant difference in the circulating capacity values.

Since there is no significant difference between the values, then the capacity value under each condition will be used.

Table 3.14 shows that both entry and circulating capacity reduces with an increase in rain intensity. To have a clear understanding, the plot of the Q_e and Q_c for dry and rainy conditions are shown in figures 3.20 to 3.22.

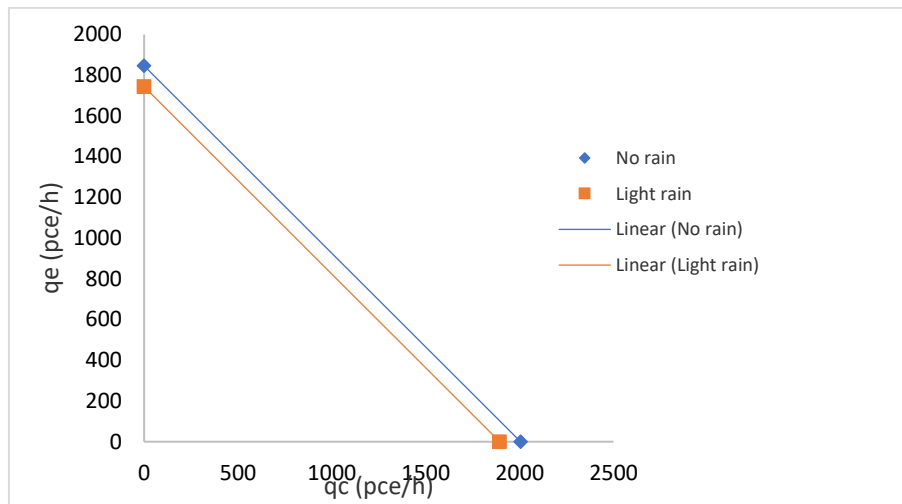


Figure 3.20: Effect of light rain on entry and circulating capacity

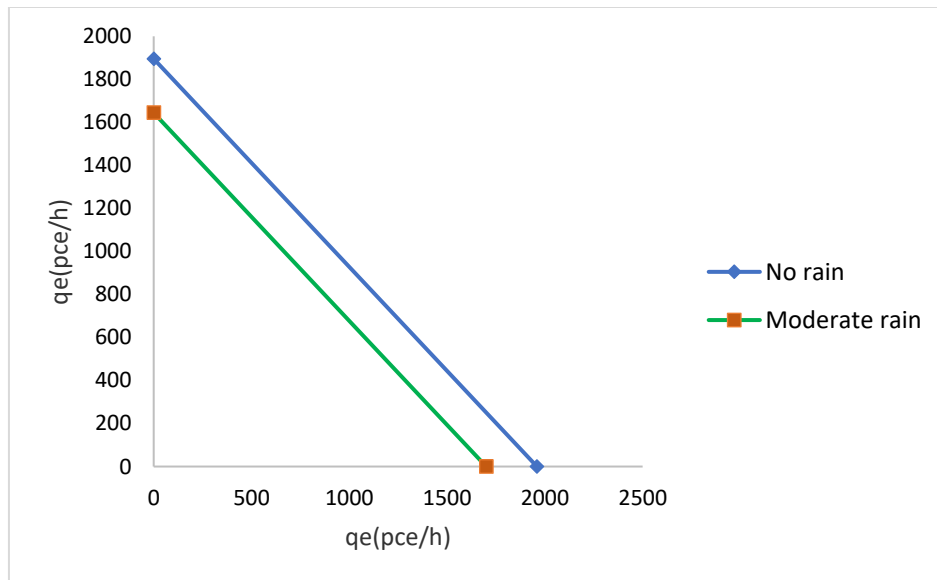


Figure 3.21: Effect of moderate rain on entry and circulating capacity

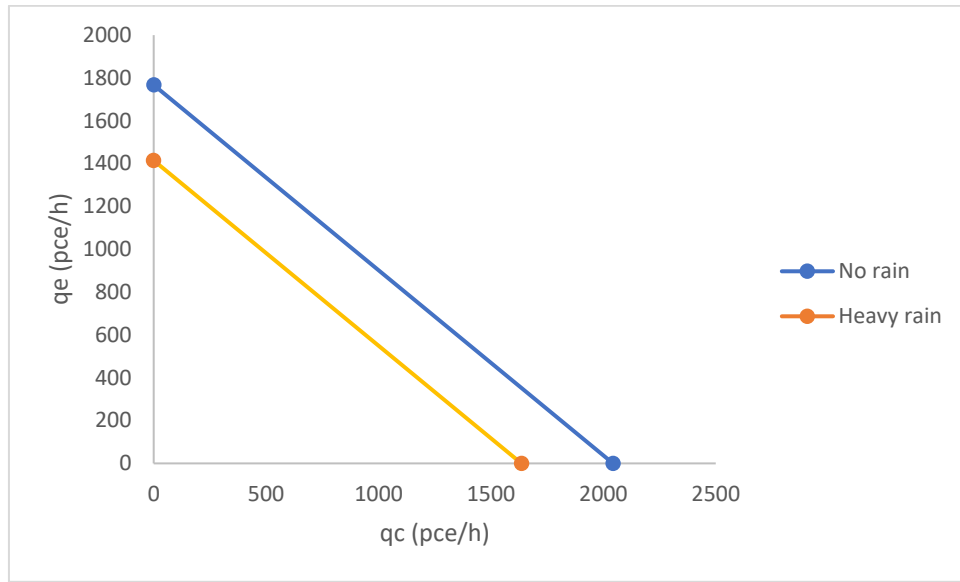


Figure 3.22: Effect of heavy rain on entry and circulating capacity

Figure 3.20 to 3.22 shows that there is a negative differential shift in both entry and circulating capacity under light, moderate and heavy rain conditions respectively.

Step 13: In this step, the effect of PCE value is investigated to see if there will be a need for modification of the PCE value.

The Segium PCE model is adopted in this study using the headway method as:

$$PCE = \frac{H_{ij}}{H_{pcj}} \quad [3.18]$$

PCE estimation under light rain weather conditions at the entry is used as working example as follows:

$$H = \frac{3600}{q} \quad [3.19]$$

The capacity = 1744pce/h, this capacity is for two lanes, on the assumption that the two lanes have the same capacity, the capacity of a single lane = 872pce/h, then the headway (H) is calculated as:

$$H = \frac{3600}{872} = 4.13s$$

Estimate the spacing using the average speed of all the vehicles:

$$H = \frac{S}{U} \quad [3.20]$$

Where: S = spacing (m), H = Headway (s), U = speed (m/s)

Then, $S = HU$

The spacing is estimated using the average speed of all vehicles as:

$$S = H \left(\frac{1}{N} \sum_{i=1}^N U \right) \quad [3.21]$$

Since all vehicles are assumed to be performing as passenger cars when the PCE values were applied in estimation of capacity then:

$$\frac{1}{N} \sum_{i=1}^N U = \text{the average speed of all vehicles (m/s)}$$

The average speed for all entry vehicles (passenger cars) under light rain = 18.26km/h = 5.07m/s (from ATC data).

Hence, spacing = $4.13 \times 5.07 = 20.94\text{m}$

Next is to determine the spacing of individual vehicles using:

$$S_i = S_{pc} + \left(\frac{1}{N} \sum_{i=1}^N L_i - \frac{1}{N} \sum_{i=1}^N L_{pc} \right) \quad [3.22]$$

Where:

S_i = Spacing of class i vehicles (m)

S_{pc} = spacing of passenger cars (m)

$\frac{1}{N} \sum_{i=1}^N L_i$ = The average length of class i vehicles

$\frac{1}{N} \sum_{i=1}^N L_{pc}$ = The average length of passenger cars

The length of a vehicle = $WB + O_F + O_R$

Where: WB = wheel base; O_F = front overhanging; O_R = rear overhanging

Average wheel base of a passenger car = 2.63m (from ATC data).

The front and rear overhanging of a passenger car are 1.03 and 1.53m respectively (SANRAL).

The length of passenger car = $1.05 + 2.63 + 1.53 = 5.21\text{m}$

Average wheel base of medium vehicles = 4.21m (from ATC data)

The front and rear overhanging of medium vehicles are 2.10 and 2.10m respectively (SANRAL).

Length of medium vehicles = $2.1 + 4.21 + 2.1 = 8.41\text{m}$

Average wheel base of heavy vehicles = 8.89m (from ATC data)

The front and rear overhanging of heavy vehicles are 0.9 and 0.6m respectively (SANRAL).

Length of heavy vehicle = $0.9 + 8.89 + 0.6 = 10.39\text{m}$

The spacing for an individual vehicle is estimated using equation 3.22 as:

Spacing for passenger cars = $20.94 + (5.21 - 5.21) = 20.94\text{m}$

Spacing for medium vehicles = $20.94 + (8.41 - 5.21) = 24.14\text{m}$

Spacing for heavy vehicles = $20.94 + (10.39 - 5.21) = 26.12\text{m}$

The headway for class i vehicles is estimated as:

$$H_{ij} = \frac{S_i}{U_i}$$

Where:

H_{ij} is the headway of vehicle class i under condition j.

S_{ij} is the spacing of vehicle class i under condition j.

U_{ij} is the speed of vehicle class i under condition j.

Average speed for a passenger car = 18.79km/h = 5.22m/s (from ATC data).

Average speed for a medium vehicle = 17.93km/h = 4.85m/s (from ATC data).

Average speed for a heavy vehicle = 15.05.km/h = 4.18m/s (from ATC data).

Headway for passenger cars = 20.94/5.22 = 4.01s

Headway for medium vehicles = 24.14/4.85 = 4.98s

Headway for heavy vehicles = 26.12/4.18 = 6.25s

Then the PCE can be estimated under light rain using equation 3.18.

PCE for a passenger car (PC) = 4.01/4.01 = 1

PCE for a medium vehicle (MV) = 4.98/4.01 = 1.2

PCE for a heavy vehicle (HV) = 6.25/4.01 = 1.6

The same procedure is used in estimation of PCE value for each vehicle category under moderate and heavy rain conditions within the entry and circulating traffic. The results summary is presented and compared to SANRAL PCE values in tables 3.15 and 3.16.

Table 3.15: Modified entry and SANRAL PCE values

Vehicle class	Weather condition			SANRAL
	LR	MR	HR	
PC	1.00	1.00	1.00	1.00
MV	1.24	1.18	1.24	2.88
HV	1.56	1.45	1.46	2.88

Note: PC is passenger car, MV is medium vehicle, HV is heavy vehicle,
LR is light rain, MR is moderate rain, HR is heavy rain.

Table 3.16: Modified circulating and SANRAL PCE values

Vehicle class	Weather condition			SANRAL
	LR	MR	HR	
PC	1.00	1.00	1.00	1.00
MV	1.32	1.17	1.16	2.88
HV	1.66	1.41	1.41	2.88

Note: PC is passenger car, MV is medium vehicle, HV is heavy vehicle, LR is light rain, MR is moderate rain, HR is heavy rain.

Table 3.15 and 3.16 shows that the modified PCE values for medium vehicle and heavy vehicles are less than the prescribed SANRAL PCE values under the light, moderate and heavy rain conditions. It must be noted that SANRAL pce values were estimated under daylight and dry weather conditions, consequently, it not surprising that the PCE values under rainy conditions are different.

Step 14: Test for significant difference between modified and SANRAL PCE values. To test this, two hypotheses are made between the modified PCE and SANRAL PCE values.

The hypotheses for MV are:

- (v) Null hypothesis (H_1): The values of the PCE are the same.
- (vi) Alternate hypothesis (H_2): The PCE values are not the same.

The test is carried out with a chi-square using the chi-square equation in 3.23.

$$X^2 = \frac{(o-e)^2}{e} \quad [3.23]$$

Where:

X^2 = chi-square, o = observed value, e is the expected value

The test is carried out at a 95% level of confidence, where $X^2 < 3.84$ means there is no significant difference between the two variables.

The sample calculation using modified and SANRAL entry medium vehicle PCE values under light rain conditions is:

$O = 1.18$, $e_1 = 2.88$ (these values are from table 3.15).

For modified and SANRAL PCE values:

$$X^2 = \frac{(1.18-2.88)^2}{2.88} = 1.00 < 3.84$$

The null hypothesis (H_1) is accepted and the alternate hypothesis (H_2) is rejected, these show there is no significant difference between the modified entry MV PCE and SANRAL PCE values. The test was carried out for modified entry and circulating medium and heavy vehicles' PCE values in dry and rainy conditions. All showed that there is no statistical difference between the modified and SANRAL PCE values at a 5 level of significance. Since there is no significant difference, it implies that any of the PCE values could be adopted in the study. Therefore the SANRAL PCE value will be adopted in this study.

Step 15: The estimation of the degree of saturation (x) and the reserved capacity (Q_R) are estimated in this step using equation 3.24 and 3.25.

$$x = \frac{q_e}{Q_e} \quad [3.24]$$

$$Q_R = \frac{Q_e - q_e}{Q_e} \quad [3.25]$$

The degree of saturation under dry and light rain conditions:

The maximum entry flow rate (q_e) under dry conditions = 1087 pce/h

The maximum entry flow rate (q_e) under light rain conditions = 1183 pce/h

Entry capacity (Q_e) under dry weather = 1846 pce/h

Entry capacity (Q_e) under light rain = 1744 pce/h

Substitute for q_e and Q_e in equation 3.24 and 3.25, Then x under dry weather = 0.58, $Q_R = 0.42$

The x under light rain = 0.69, $Q_R = 0.31$

The same procedure is used in estimating the degree of saturation and reserved capacity under moderate and heavy rainy conditions. The summary of the results is presented in table 3.17.

Table 3.17 Summary of the degree of saturation and reserved capacity for site PSTS005

Weather condition	x		Δx	Q _R		ΔQ_R
	Rain	Dry		Rain	Dry	
Light	0.68	0.58	0.10	0.31	0.42	0.11
Moderate	0.72	0.57	0.15	0.28	0.43	0.15
heavy	0.71	0.59	0.12	0.63	0.41	0.22

Note: x is the degree of saturation, Q_R is the reserved capacity.

Table 3.17 shows that degree of saturation increases, and the reserved capacity reduces irrespective of the rain intensity.

Step16: The delay is estimated as explained in step 7.

$$d = \frac{3600}{k(F-f_c q_c - D)} + 900T \left[(x - 1) + \sqrt{(x - 1)^2 + \frac{\left(\frac{3600}{k(F-f_c q_c - D)}\right)x}{450T}} \right] + 5 \quad [3.26]$$

As an example, for the light rain condition,

Entry capacity = 1744 pce/h (from table 3.14) = 872 pce/h/lane, $x = 0.68$, $T = 0.25$ hr. Substitute these into equation 3.26,

$$d = \frac{3600}{872} + 900(0.25) \left[(0.68 - 1) + \sqrt{(0.68 - 1)^2 + \frac{\left(\frac{3600}{872}\right)0.68}{450(0.25)}} \right] + 5 = 17.42s$$

The queue length is estimated as:

$$L = \frac{d * q_e}{3600} \quad [3.27]$$

Substituting for a delay of 17.42s, the entry flow rate at the degree of saturation of $0.68 = 872 \times 0.68 = 593\text{pce/h/lane}$ in equation 3.27.

$$L = \frac{17.42 \times 593}{3600} = 2.9 \approx 3\text{veh}$$

The same procedure is used in the estimation of delay and queue length for dry, moderate and heavy rain weather conditions, the result shows that delay increases with an increase in rain intensity as the delay increases by 3.28s, 6.07s and 6.77s under light, moderate, and heavy rain respectively, while the queue length increases from two to three irrespective of the rain intensity. The summary is in table 3.18.

Table 3.18: Summary of delay and queue length site PST005

Weather condition	d (s)		Δd (s)	L (veh)		ΔL (veh)
	Dry	Rain		Dry	Rain	
Light rain	14.14	17.42	3.28	2	3	1
Moderate rain	13.71	19.78	6.07	2	3	1
Heavy rain	14.76	21.53	6.77	2	3	1

Note: d is the delay and L is the queue length.

Step 17: The functional quality of service is determined in this step. There is no need to estimate all the parameters for the assessment of functional quality of service. Once the degree of saturation, reserved capacity or the delay is estimated, other parameters can be read from the FQS criterial table by interpolation. For the pilot study, the delay will be used for the assessment of the functional quality of service with the developed FQS criterial table 3.12 for this site. The assessment of the functional quality of service under dry and rainy conditions are presented in table 3.19.

Table 3.19: Summary of functional quality of service for site PST005

Weather Condition	Delay (s)	FQS
Dry	14.14	B
Light rain	17.42	C
Moderate rain	19.78	C
Heavy rain	21.53	C

Note: FQS is functional quality of service

Table 3.19 shows that light, moderate and heavy rain changes the service delivery from FQS B to FQS C. This shows that irrespective of rain intensity, rainfall influences the roundabout functional quality of service.

Step 18: The follow-up time is estimated to determine rainfall implications on time headway with the use of equation 3.28.

$$t_f = \frac{3600}{F} \quad [3.28]$$

Where:

t_f = follow-up time.

F = entry capacity (pce/h)

This is like the follow-up time using the HCM (2010) as

As an example, when the entry capacity = 1846pce/h = 923pce/h/lane

$$\text{At capacity, } t_f = \frac{3600}{923} = 3.90s$$

The follow-up time headway is estimated at 0.1 to 0.9 degree of saturation to eliminate the effect of peak period on the follow-up time.

At 0.9 degree of saturation, the entry traffic flow rate = $0.9 \times 923 = 831$ pce/h

$$t_f = \frac{3600}{831} = 4.33s$$

The same procedure is used in estimating the follow-up time at 0.1 to 1.0 degree of saturation under dry and rainy conditions.

The result shows that light rain causes an increase in the follow-up time, for example when the degree of saturation is 0.5 the follow-up time increases from 7.8s to 8.26s with an increase of 0.46s, at the degree of saturation of 0.9, it increases from 4.33s to 4.59s with an increase of 0.26s, and at capacity it increases from 3.90s to 4.13s with an increase of 0.23s. This shows that the effect of rainfall decreases as the entry flow rate increases. The estimated follow-up result summary is presented in tables 3.20 to 3.22. To determine the pattern of the effects of rainfall, the plotting of follow-up time against degree of saturation is presented in figure 3.23.

Table 3.20: Summary of follow-up headway under dry and light rain

x	t _f (s)	
	Dry	Light rain
0.1	39.00	41.28
0.2	19.50	20.64
0.3	13.00	13.76
0.4	9.75	10.32
0.5	7.80	8.26
0.6	6.50	6.88
0.7	5.57	5.90
0.8	4.88	5.16
0.9	4.33	4.59
1.0	3.90	4.13

Note: x is the degree of saturation and t_f is follow-up time.

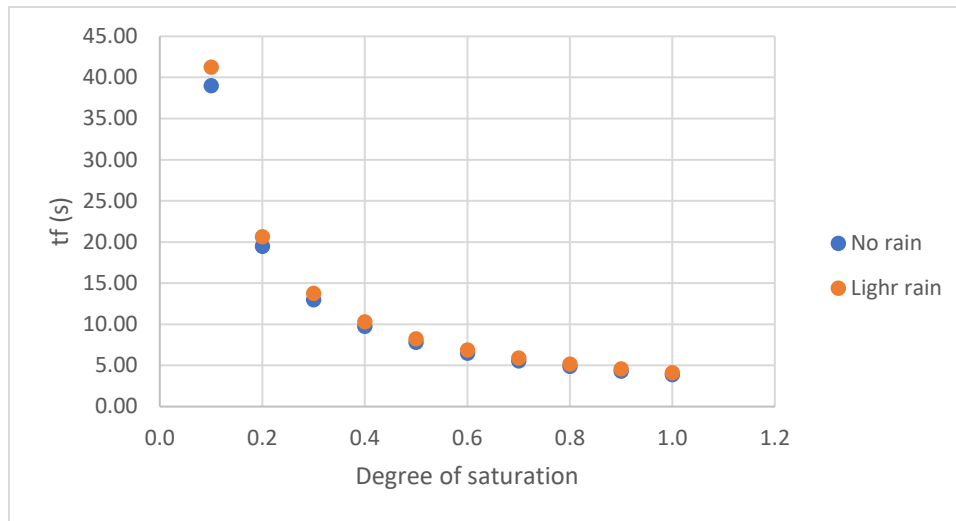


Figure 3.23: Follow-up headway under dry and light rain conditions

Figure 3.23 shows that at free flow rate there is a noticeable effect of light rain, but as the degree of saturation increases the light rain effect reduces, and at capacity the effect is almost nullified by the traffic effect.

The follow-up time under dry and moderate rain shows that as the degree of saturation increases, the follow-up time decreases under dry and moderate rain conditions, the moderate rain has an increasing effect on the follow-up time, for example when the degree of saturation is 0.5, the follow-up time increases from 7.68s to 8.76s with an increase of 1.08s, at 0.9 degree of saturation it increases from 4.22s to 4.87s with an increase of 0.65s, and at entry capacity with a degree of saturation of 1.0, it increases from 3.80s to 4.38s with an increase of 0.58s. This shows that a moderate rain effect reduces as the degree of saturation increases. The summary of the follow-up time under dry and moderate rain is presented in table 3.21. The pattern of the moderate rain effect is shown in figure 3.24.

Table 3.21: Summary of follow-up headway under dry and moderate rain

x	t_f (s)	
	Dry	Moderate rain
0.1	37.99	43.80
0.2	19.00	21.90
0.3	12.66	14.60
0.4	9.50	10.95
0.5	7.60	8.76
0.6	6.33	7.30
0.7	5.43	6.26
0.8	4.75	5.47
0.9	4.22	4.87
1.0	3.80	4.38

Note: x is the degree of saturation and t_f is follow-up time.

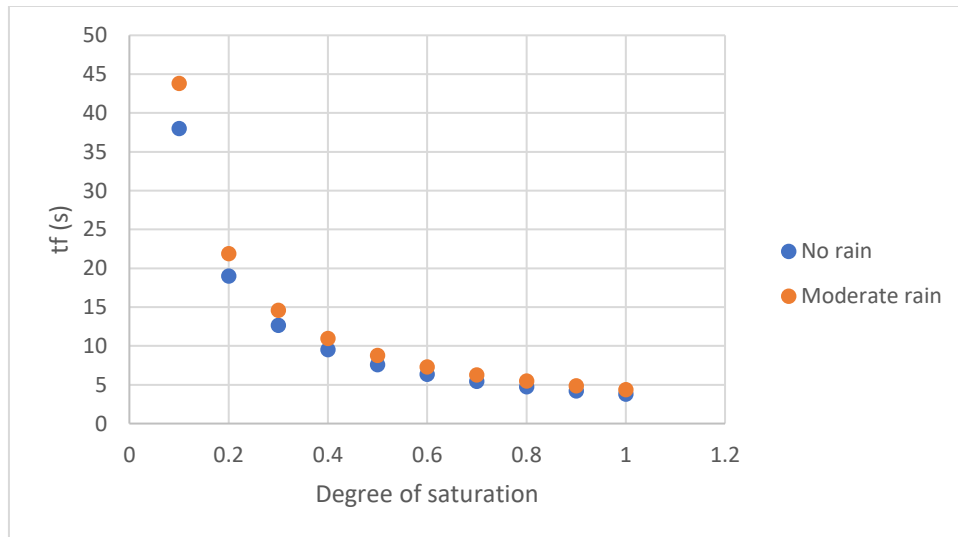


Figure 3.24: Follow-up headway under dry and moderate rain conditions

Figure 3.24 shows that as the flow rate increases towards the capacity with the degree of saturation of 1.0, the effect of the moderate rainfall decreases. At capacity, the moderate rain effect is minimal which shows that the traffic at capacity has more control on the follow-up time.

The follow-up time under dry and heavy rain shows that the heavy rain effect on follow-up time follows the same pattern as with the light and moderate rain effect on follow-up time, as the follow-up time increases from 8.15s to 10.18s with an increase of 2.03s at a 0.5 degree of saturation, 4.53s to 5.66s with an increase of 1.13s at a 0.9 degree of saturation, and 4.07s to 5.09s with an increase of 1.02s at capacity. The effect of heavy rain reduces as the degree of saturation increases. The summary of the follow-up time under dry and heavy rain condition at varying degree of saturation is presented in Table 3.22. The trend of a heavy rain effect is shown in the plotting of follow-up time against the degree of saturation in figure 3.25.

Table 3.22: Summary of follow-up headway under dry and heavy rain

x	t_f (s)	
	Dry	Heavy rain
0.1	40.75	50.92
0.2	20.37	25.46
0.3	13.58	16.97
0.4	10.19	12.73
0.5	8.15	10.18
0.6	6.79	8.46
0.7	5.82	7.27
0.8	5.09	6.36
0.9	4.53	5.66
1.0	4.07	5.09

Note: x is the degree of saturation and t_f is follow-up time.

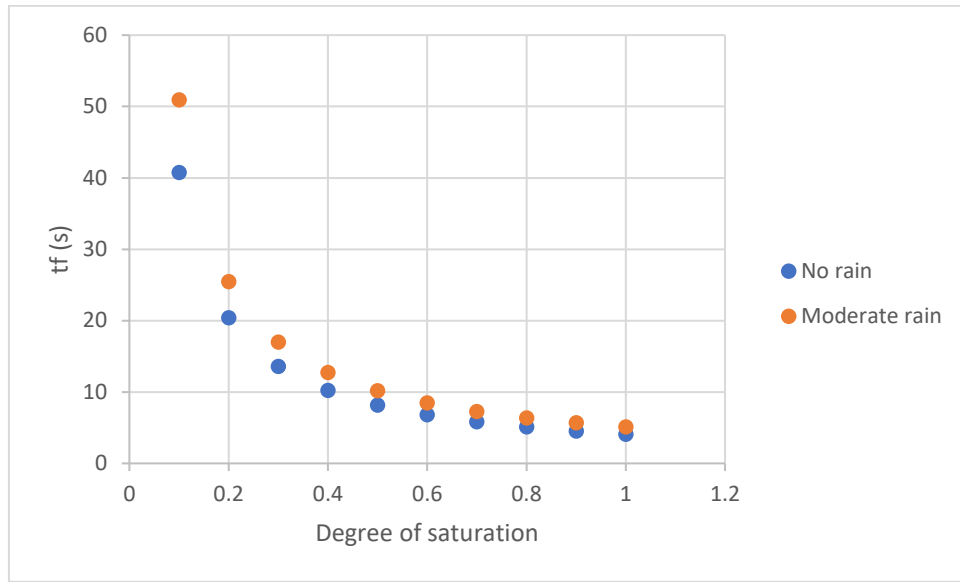


Figure 3.25: Follow-up headway under dry and heavy rain conditions.

Figure 3.25 shows that the heavy rain effect on follow-up decreases with the increases in degree of saturation, and the effect is at a minimum at capacity.

Step 19: The critical gap is estimated to determine rainfall implications. The critical gap is estimated using the empirical method in this step.

The circulating headway is estimated using equation 3.29.

$$H_c = \frac{3600}{Q_c} \quad [3.29]$$

Where:

H_c = circulating headway

Q_c = circulating capacity

As an example, under light rain when $Q_c = 1894$ pce/h, then $Q_c = 947$ pce/h/lane.

$$H_c = \frac{3600}{947} = 3.80s$$

Critical gap can be estimated using equation 3.30.

$$\text{Gap} = H_c - \text{travel time to cover vehicle length} \quad [3.30]$$

Travel time to cover vehicle length is estimated with equation 3.31.

$$T = \frac{\frac{1}{N} \sum_{i=1}^N d_i}{\frac{1}{N} \sum_{i=1}^N u_i} \quad [3.31]$$

Where:

T = Time taken to drive through the length of the vehicle (s)

$\frac{1}{N} \sum_{i=1}^N d_i$ = Average length of all the vehicles under weather condition I (m)

$\frac{1}{N} \sum_{i=1}^N u_i$ = Average speed of all vehicles under weather condition I (m/s)

The circulating vehicles' length are estimated using the procedure in step 12 as:

Passenger cars = 5.21m (from ATC data)

Medium vehicles = 8.3m (from ATC data)

Heavy vehicles = 10.4m (from ATC data)

The average length of all the vehicles are estimated based on the traffic vehicle composition.

The vehicle composition from the data collected are:

Passenger cars - 96.04%

Medium vehicles - 2.13%

Heavy vehicles - 1.31%

Motorcycles - 0.52%

The average length of the vehicles is estimated as the average of the sum of products of vehicles' length and composition proportion. (Motorcycle is not considered in this study).

$$\text{The average length} = \frac{(5.21 \times 0.9604) + (8.3 \times 0.0213) + (10.4 \times 0.0131)}{0.9604 + 0.0213 + 0.0131} = 5.3\text{m}$$

The average speed of all the vehicles is 25.38km/h = 7.05m/s (from ATC data)

$$\text{Then, } T = \frac{5.3}{7.05} = 0.75\text{s}$$

The critical gap is estimated using equation 3.32.

$$t_c = H_c - T \quad [3.32]$$

Substituting for H_c and T in equation 3.32,

$$t_c = 3.80 - 0.75 = 3.05s$$

Using the HCM method,

Where:

$$Q_{e=Ae^{(-Bq_c)}} = 1130e^{-0.0007q_c}$$

$$A = \frac{3600}{t_f}, \text{ then } t_f = \frac{3600}{1130} \text{ 3.19s}$$

$$\text{Then } B = \frac{t_c - 0.5t_f}{3600} = 0.0007, \text{ If } t_f \text{ is substituted then } t_c = 4.11s$$

This shows that the HCM overestimates the critical gap as the empirical method yields $3.05s <$ HCM value of $4.11s$ (It was stated in the literature review that the critical gap varies from place to place).

This procedure is used in estimating the critical gap at a circulating degree of saturation of 0.1 to 1.0 under dry and rainy conditions.

The result of critical gap under dry and light rain shows that the critical gap reduces as the circulating flow rate increases under dry and light rain conditions. The light rain causes an increase in the critical gap, for example when the degree of saturation is 0.5, the critical gap increases from 6.8s to 7.00s causing an increase of 0.22s. At 0.9 degree of saturation, the critical gap increases from 3.59s to 3.62s causing an increase of 0.03s. When the roundabout is operating at capacity, the degree of saturation is 1.0, the critical gap increases from 3.19s to 3.20s with an increase of 0.01s, which is insignificant. The summary of the estimated critical gap is presented in table 3.23.

Table 3.23: Summary of the critical gap under dry and light rain

x	t _c (s)	
	Dry	Light rain
0.1	35.51	37.41
0.2	17.56	18.64
0.3	11.57	12.07
0.4	8.58	8.90
0.5	6.78	7.00
0.6	5.59	5.73
0.7	4.73	4.83
0.8	4.09	4.15
0.9	3.59	3.62
1.0	3.19	3.20

Note: x is the degree of saturation and t_c is the critical gap.

The result of critical gap under dry and moderate rain shows that the effect of moderate rain reduces as the circulating flow rate increases, for example when the degree of saturation is 0.5, the critical gap increases from 6.95s and 7.75s with an increase of 0.80s, when the degree of saturation is 0.9, the critical gap increases from 3.68s to 3.98s causing an increase of 0.30s, and at capacity, when the degree of saturation is 1.0, it increases from 3.27s to 3.51s causing an increase of 0.24s. The summary of the estimated critical gap under dry and moderate rain at varying degree of saturation is presented in table 3.24.

Table 3.24: Summary of the critical gap under dry and moderate rain

x	t _c (s)	
	Dry	Moderate rain
0.1	36.33	41.63
0.2	17.97	20.45
0.3	11.84	13.40
0.4	8.78	9.87
0.5	6.95	7.75
0.6	5.72	6.34
0.7	4.85	5.33
0.8	4.19	4.57
0.9	3.68	3.98
1.0	3.27	3.51

Note: x is the degree of saturation and t_c is the critical gap.

The result of critical gap under dry and heavy rain shows that under heavy rain, at a degree of saturation of 0.5, the critical gap increases from 6.57s to 7.74s with an increase of 1.17s. At a 0.9 degree of saturation, it increases from 3.49s to 3.86s with an increase of 0.30s, and at capacity the degree of saturation is 1.0, and the gap increases from 3.09s to 3.38s causing an increase of 0.29s. The results show that the critical gap increases with the increase in rain intensity and the effect of rainfall diminishes as the degree of saturation increases. The summary of the estimated critical gap under dry and heavy rain at varying degree of saturation is presented in table 3.25.

Table 3.25: Summary of the critical gap under dry and heavy rain

x	t _c (s)	
	Dry	Heavy rain
0.1	34.47	42.58
0.2	17.03	20.88
0.3	11.22	13.51
0.4	8.32	9.91
0.5	6.57	7.74
0.6	5.41	6.28
0.7	4.58	5.25
0.8	3.96	4.47
0.9	3.49	3.86
1.0	3.09	3.38

Note: x is the degree of saturation and t_c is the critical gap.

The pattern of the rainfall effect is presented by plotting the critical gap against the degree of saturation under dry and rainy weather conditions as shown in figures 3.26 to 3.28. It is shown that when the roundabout circulating traffic is operating at capacity, the effect of light, moderate and heavy rain is reduced, and the optimum traffic controls the critical gap. The trend of effect or rainfall on critical gap follows same pattern under light, moderate and heavy rain.

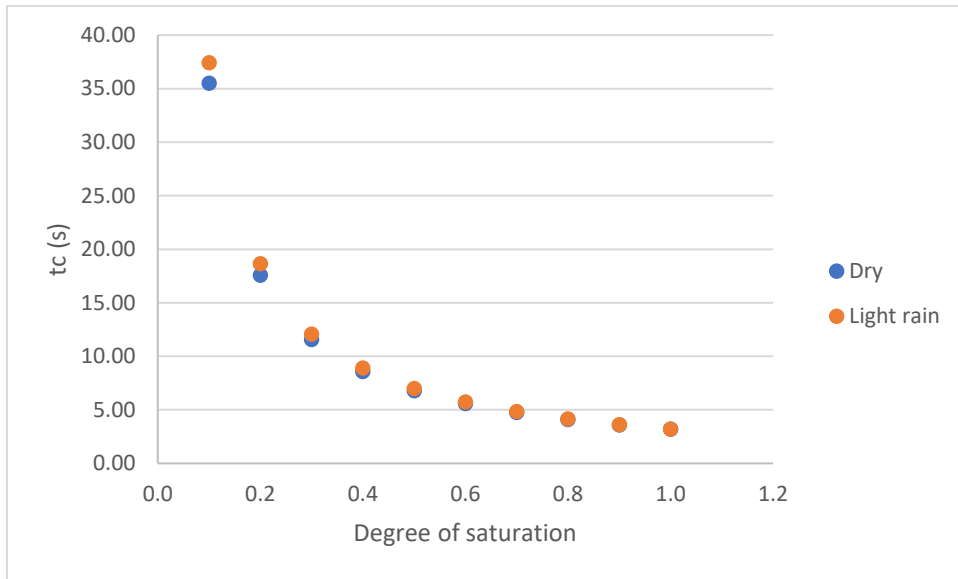


Figure 3.26: Critical gap and degree of saturation under dry and light rain

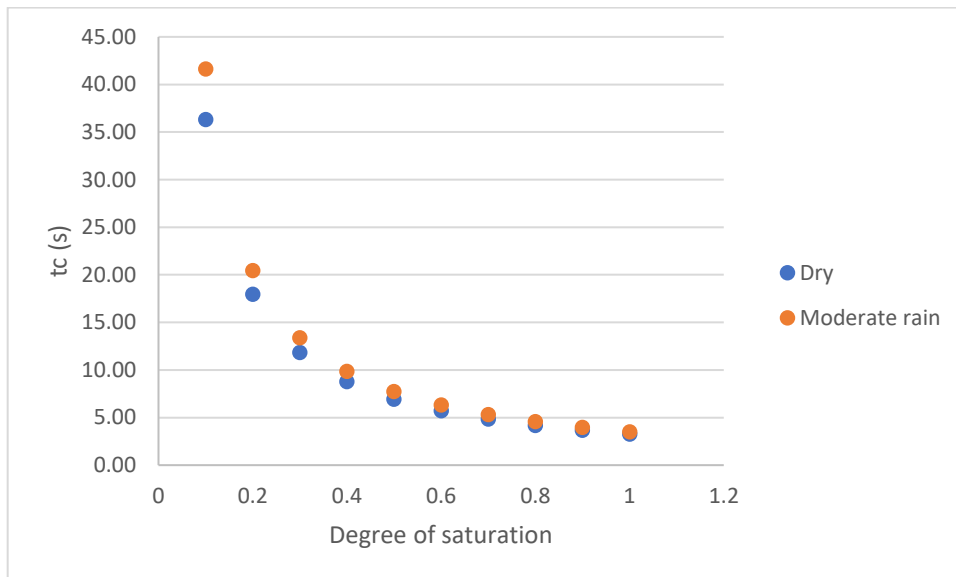


Figure 3.27: Critical gap and degree of saturation under dry and moderate rain

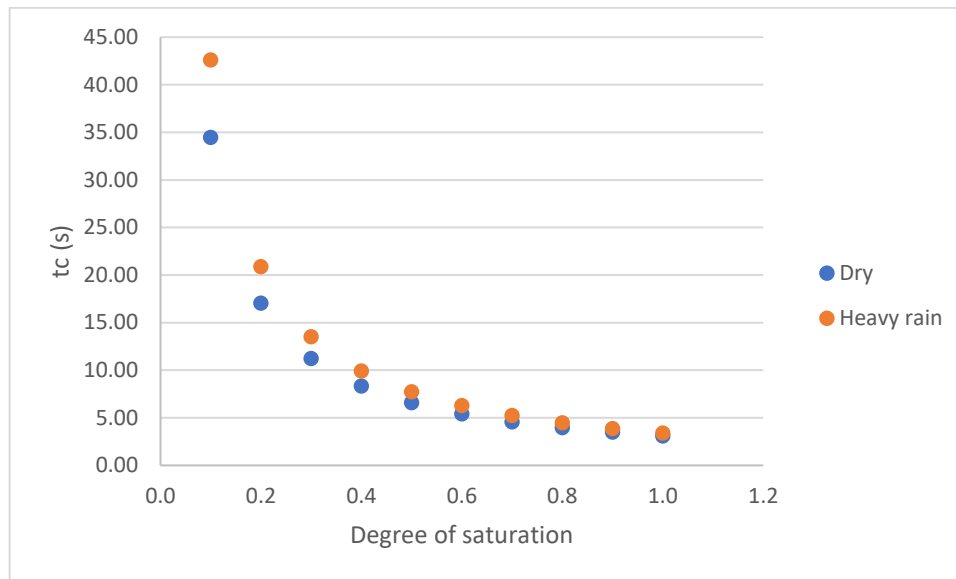


Figure 3.28: Critical gap and degree of saturation under dry and heavy rain

3.8 SUMMARY

In this chapter the data required for this study and the methods of collecting and analysing them were presented. This chapter has explained how study sites were selected, how empirical data collections were carried out and later how they were analysed. In terms of data collection, it has described the sample size of the data, the survey team and equipment, the traffic survey and the rainfall survey in order to accomplish the objectives of this study. It presented a typical survey site layout. Traffic data were collected using an automatic traffic counter that provided vehicle volume, vehicle type, and headway information. While rainfall data was obtained from the nearest rain gauge station, it was also supplemented by local survey data. The main survey involved four sites. In terms of data analysis, it has described the steps used to assess the quality of service delivery at roundabouts. This chapter has also discussed the pilot test that was carried out to test the data collection tools and procedures. Sample data from the pilot test were assessed based on traffic flow rate performance at the selected roundabout. The validity of the capacity estimation method and analytical procedure were also tested. The impact of rainfall on passenger car equivalent values was also explored and found to be inconsequential. By the isolated nature of the data on which the preliminary investigations were based, the results described in this chapter are broadly

suggestive. Consequently, the data in this chapter begs several questions about the influence of rainfall on the quality of service at roundabouts. The questions include:

- To what extent has rainfall affected the degree of saturation?
- To what extent has rainfall affected the reserved capacity?
- To what extent has rainfall affected delay and queue?
- What is the relationship between rainfall and quality of service?
- What is the effect of rainfall on time headway?

In the next chapter, results from the samples surveyed at the four selected sites will be investigated and used in chapters 5 and 6 for multilane roundabouts' quality of service assessment and their time headway implications.

CHAPTER 4

SURVEY SITES EMPIRICAL RESULTS

4.1 Overview

The empirical data which includes the geometric, rainfall, traffic data and the data collection methods are described in chapter 3 of this thesis. This research is focused on how rainfall affects the roundabout service delivery. The data needed in achieving the aim of this study includes, rain, the entry and circulating traffic data. These data are presented in this chapter. The peak traffic data under dry daylight conditions is required for the development of a criterial table for the roundabout assessment, and off-peak traffic data under dry and rainy weather conditions are also required for the roundabout assessment.

This chapter is organised as follows: The following section (4.2) describes the empirical data for the surveyed sites which includes the geometrical data, the rain data, and the empirical entry and the circulating traffic data. Data were collected at all the approaches of the roundabouts, but for the purpose of this study the approach with the highest traffic flow rate is considered. The chapter summary is presented in section 4.3.

4.2 Empirical Data for Surveyed Sites

Four roundabouts were surveyed in Durban city in the KwaZulu-Natal Province of South Africa. This study is focussed on how rainfall affects the roundabout service delivery. Based on this, the study was carried out during the rainy season in South Africa, which is in the months of August to March, as discussed in chapter 3 of this thesis. The traffic data was collected continuously from the months of August 2016 to March 2017. However, December to March is the summer period with a high rain intensity. All the surveyed roundabouts are standard double lane roundabouts with asphaltic pavement on entry and circulating roadways. The geometry is within the South African roundabout geometry specifications in [SANRAL,2011](#). They are well marked with adequate road

signs and the pavement is free of any defects. The summary of the geometric data of the surveyed roundabouts is shown in Table 4.1. The geometric data was compared to the design drawings for each roundabout from the eThekweni Municipality's planning department and was also compared to the measurements from Google Maps.

Table 4.1 Summary of the geometric data of the surveyed roundabouts

Roundabout features	Site 01	Site 02	Site 03	Site 04
Name of roundabout	Armstrong	Millennium	Douglas	Gateway
Class of roundabout	Double lane roundabout	Double lane roundabout	Double lane roundabout	Double lane roundabout
Entry pavement surface type	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement
Circulating pavement surface type	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement	Asphaltic pavement
Number of entry lanes	2	2	2	2
Number of circulatory roadway lanes	2	2	2	2
Entry width (m)	8.5	7.90	8.20	8.40
Entry angle ($^{\circ}$)	50	45.00	45.00	50.00
Entry radius (m)	40	50.00	50.00	45.00
Effective flare length (m)	16.00	18	15	13
Inscribed circle diameter (m)	50.00	58.10	49.50	48.00
Approach road half width (m)	7.30	6.80	6.90	6.80
Circulating road width (m)	9.40	9.30	9.10	8.80
Central Island shape	Circular	Circular	Circular	Circular
Road signs and marking	OK	OK	OK	OK
Distance from rain gauge	0.95km	1.18km	0.82km	0.75km

Rainfall data was collected at the surveyed sites. Measurement of rainfall is usually carried out with rain gauges which can be either a manual rain gauge or the modern rain gauges which collect rain data automatically and transmit the data to the server or email at the defined time intervals through a global system for mobile telecommunication or general packet radio service. The resolution of the modern rain gauge can be from one to ten minutes, and high-resolution rainfall data is needed for traffic flow rate because it varies considerably with time. The rain gauge takes a measurement of the rain data at a certain point and it is assumed that the point of measurement is uniform over an area, which is the catchment area of the rain gauge. The standard rain gauge is made up of a 20cm diameter and 50cm length of cylindrical barrel into which a funnel is placed with a graduated 2cm in radius cylinder which empties into the barrel. If the 2cm cylinder overflow rates, the outer barrel will be able to catch the overflow rate water. The rain gauge is usually placed in an open place and where it is not under any form of cover. The rain gauge is usually placed above the ground level; this is to avoid errors in rain data collection, because if placed on the ground the water splash might enter the rain gauge thereby increasing the volume of water in the rain gauge more than the actual rain data. Figure 4.1 below shows a typical modern rain gauge station.



Figure 4.1: Typical modern rain gauge station

The rainfall data was collected from the website of the eThekweni Municipality. The municipality uses modern rain gauges to collect rain data. Standard rain gauges were also used to collect rainfall intensity manually, and the manually collected rain data was compared to the data collected from the eThekweni Municipality website to avoid error.

Traffic flow rate was collected automatically and continuously at entry and circulating roadways for six weeks under dry and rainy conditions, and it was also collected manually at intervals daily under dry weather conditions. The manually collected traffic data was compared to the ATC data of the same period. Data collected at each of the selected roundabouts will be considered in the subsequent subsections.

4.2.1 Site 01: Armstrong Roundabout

Armstrong roundabout description has been presented in section 3.5.1. The site set-up for data collection is as shown in Figure 4.2.



Figure 4.2: Site 01 Armstrong Roundabout

The Armstrong roundabout entry width is 8.5m, entry angle is 50° , entry radius is 40m, effective flare length is 16m, the inscribed circle diameter is 50m, approach half width and the circulating road width are 7.30m and 9.40m respectively as presented in Table 4.1. The entry and circulating roadway are asphaltic concrete pavement and the roundabout is marked with appropriate road signs. The distance from the rain gauge is 0.95km.

4.2.1.1 Site 01: Armstrong Roundabout Rain Data

The rain data was collected from the eThekweni Municipality rain gauge station. The rain data captured within the area of the Armstrong roundabout was used for this site. The manual rain gauge was used for the collection of rain data at this site. The collected rain data was compared to the rain data collected from the eThekweni Municipality rain gauge station. The closest rain gauge to site 01 is the Crawford rain gauge with the station ID Crawford (see appendix C) located at Crawford School along Crawford school – Armstrong Avenue – La Lucia road at 0.95km from the Armstrong roundabout. The rain data at this site was collected from 31 July 2016 to 15 September 2016. There was rain during ten days within the surveyed period as shown in the daily rainfall precipitation chart in Figure 4.3. Rain precipitation was recorded in 5-minute intervals at this site. A typical example of five minutes' rain precipitation at the Crawford rain gauge is shown in Figure 4.4. The collected rain precipitation was converted to intensity by dividing the value of rain precipitation with the rain duration. For example, rain precipitation of 0.2mm has an intensity of 2.40mm/h as shown below:

Rain precipitation = 0.2mm: Rain period = 5 minutes = 0.0833hr

The rain intensity = $\frac{0.2}{0.0833} = 2.40\text{mm/hr}$.

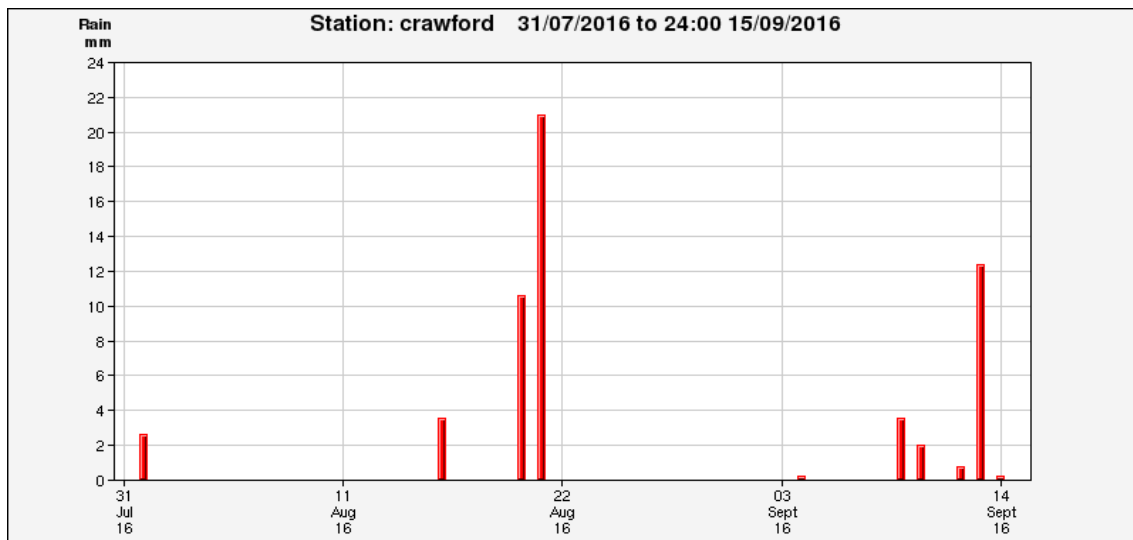


Figure 4.3: Daily rain precipitation at the Crawford rain gauge.

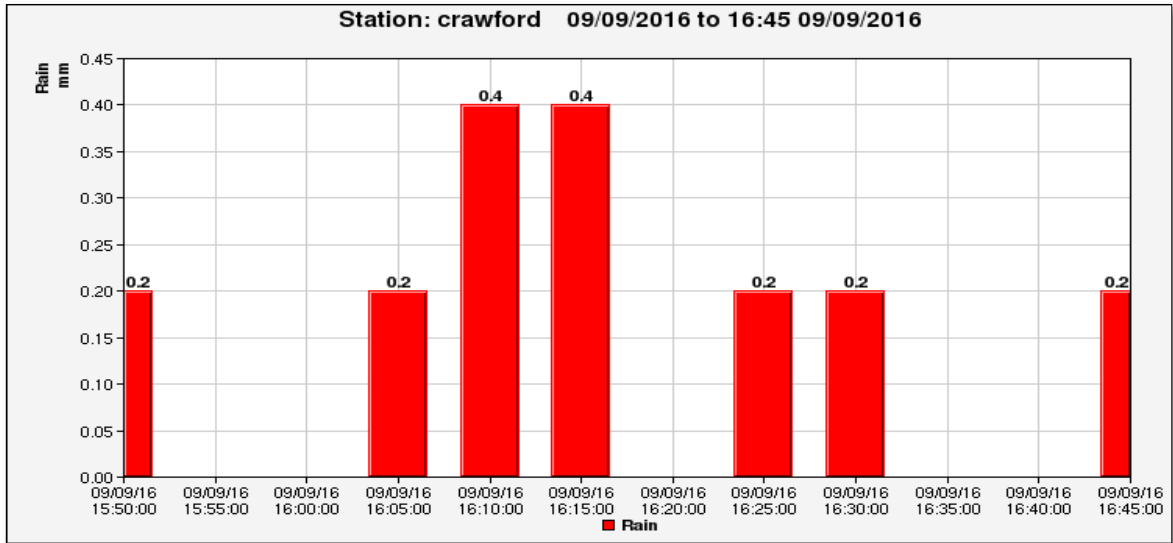


Figure 4.4: Typical five minutes' precipitation at the Crawford rain gauge station

Note that, rainfall is classified into three categories; light rain (LR) with intensity (i) $< 2.5\text{mm/h}$, moderate rain (MR) with intensity $> 2.5\text{mm/h}$ but $\leq 10\text{mm/h}$, heavy rain (HR) with intensity $> 10\text{mm/mm/h}$ but $\leq 50\text{mm/h}$.

4.2.1.2 Site 01: Armstrong Roundabout Traffic Flow Rate Profile Data

The entry flow rate pattern at this site fluctuates which shows that the entry flow rate is not a continuous flow rate because is dependent on the gap in the circulating flow rate to access the roundabout. The circulating flow rate is almost uniform which depicts that the circulating flow rate is continuous and is independent of the entry flow rate. In addition, the vehicles at entry reduce speed to access the roundabout. The rate of speed reduction depends on the existing queue and the type of vehicle at the entry roadway awaiting the safe gap in the circulating flow rate. The peak traffic flow rate occurs from Monday to Friday, which are the weekdays, while the low traffic flow rate occurs during Saturdays and Sundays (weekends) at both the entry and circulating roadways. The entry and circulating flow rate profile for the data collection period of 31 July to 15 September 2016 is shown in Figures 4.5 and 4.6 respectively.

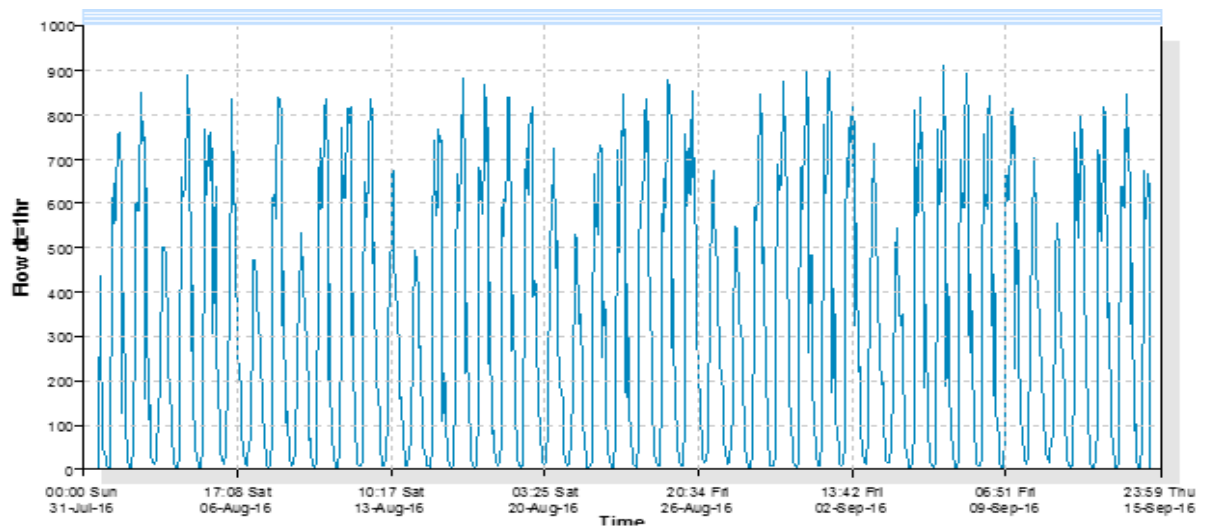


Figure 4.5 Entry traffic flow rate profile for site 01

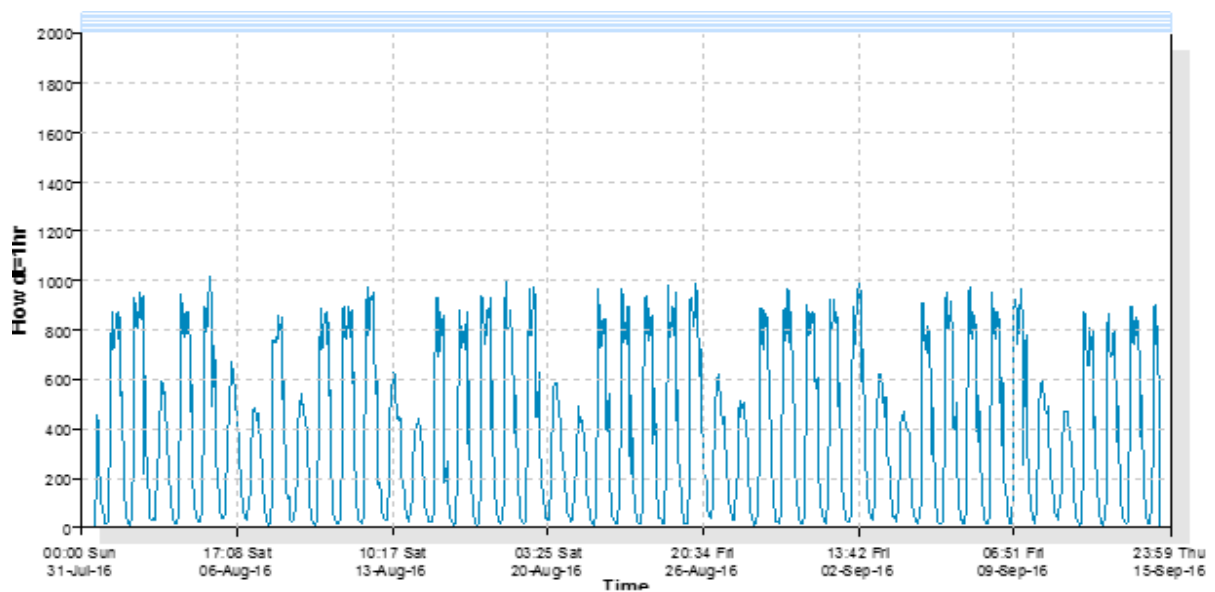


Figure 4.6: Circulating traffic flow rate profile for site 01

4.2.1.3 Site 01: Armstrong Roundabout Traffic Volume Data

The macro traffic data observed at this site was divided into 12 intervals for both the entry and circulating roadways. The vehicles were classified into four main categories in accordance with

the South Africa National Road Agency Limited (SANRAL) geometric design guide, which were; the car and light vans, commercial vehicles, buses and coaches, and motorcycles. These are more simply grouped as passenger cars, for cars and light vans, medium vehicles for commercial vehicles, and heavy vehicles for buses and coaches, and motorcycles, for this study.

The vehicular traffic data was collected continuously for six weeks under different traffic and weather conditions. This ensured that it covered both dry and rainy weather conditions. The entry and circulating peak period data under dry weather was collected to ascertain how the roundabout functions during peak periods, and for functional quality of service criterial table development.

The total vehicle volume collected at this site was 805,159, of which 377,878 was collected at the circulating roadway, while the total vehicle volume of 427,287 was collected at the entry roadway for a period of six weeks continuously. The hourly peak entry and circulating traffic volume under dry weather condition is shown in Table 4.2. The hourly off-peak entry and circulating traffic flow rate under the dry and rainy weather is shown in Tables 4.3 and 4.4 respectively. The traffic composition at the peak period for site 01 is made up of 94.30 percent passenger cars, 2.93 percent medium vehicles, 2.93 percent heavy vehicles and 0.65 percent motorcycles. The circulating traffic volume composition is 90.82 percent passenger cars, 4.97 percent medium vehicles, 3.19 percent heavy vehicles and 1.02 percent motorcycles as presented in Table 4.6. The passenger cars constitute the highest vehicle volumes for both the entry and circulating traffic at the peak period, while motorcycles are the least.

Table 4.2: Site 01 hourly peak traffic flow rate under dry weather

Flow rate (pce/h)	Period											
	1	2	3	4	5	6	7	8	9	10	11	12
Circulating	828	607	852	1054	1070	787	797	864	1049	1202	607	979
Entry rate	1106	1534	1190	967	967	1018	1282	1258	1150	914	1430	967

Table 4.3: Hourly off-peak entry traffic flow rate at site 01 (off-peak)

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	499	1006	924	1052
2	1044	912	912	768
3	1006	1265	924	626
4	1018	1123	1020	789
5	1255	1325	1054	709
6	972	972	1128	792
7	972	982	1161	1063
8	926	912	1162	796
9	936	1017	972	811
10	794	1157	962	663
11	1017	1114	1017	787
12	948	1039	1032	818

Table 4.4: Hourly off-peak circulating traffic flow rate at site 01 (off-peak)

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	1459	1039	861	777
2	1063	1109	871	1094
3	1137	744	876	1317
4	1128	938	777	1130
5	905	670	765	1154
6	1164	1015	672	1041
7	1255	1113	624	778
8	1212	1094	643	1001
9	1341	998	850	1106
10	1334	864	744	1164
11	1123	984	818	999
12	1190	996	717	934

T test is used to determine the difference in the entry flow rate under dry and rainy conditions. Comparing the entry flow rate under dry and light rain conditions. Two hypotheses are set, the hypotheses are:

- i Null hypothesis (H_1): The entry flow rate values are the same under dry and rainy Conditions.
- ii Alternate hypothesis (H_2): The entry capacity values are not the same under dry and rainy conditions.

Hypothesis i holds when the t-test is less than t-critical, otherwise hypothesis ii holds.

The result of the t-test is presented in Table 4.5

Table 4.5: Summary of t-test result

	Entry flow rate Dry	light rain
Mean	948.916667	1068.66667
Variance	31234.2652	17316.7879
Observations	12	12
Hypothesized Mean Difference	0	
df	20	
t Stat	-1.882638	
P(T<=t) one-tail	0.03718974	
t Critical one-tail	1.72471824	
P(T<=t) two-tail	0.07437948	
t Critical two-tail	2.08596345	

t-two tail test statistics is used, the t stat is 1.88 which is less than t critical of 2.10, hence, the null hypothesis is accepted, and alternate hypothesis is rejected. There is no significant difference between the entry flow rate under dry and light rain weather conditions. This test is used for determining the difference in dry, light, moderate and heavy rain entry and circulating flow rate. It was discovered that there is no significant difference between the traffic flow rate under dry and rainy conditions irrespective of the rain intensity.

The off-peak traffic composition at the entry under dry and rainy conditions shows that the passenger cars increase from 92.14 percent (dry) to 93.44 percent in light rain (LR), and 95.35 percent in moderate rain (MR) but reduces to 91.57 percent in heavy rain (HR); and the medium vehicle reduces from 4.05 percent (dry) to 3.48 percent (LR), 3.07 percent (MR) and increases to 5.57 percent (HR); and the heavy vehicles reduce from 2.20 percent (dry) to 2.36 percent (LR), 1.48 percent (MR) and increases to 2.86 percent (HR) as shown in Table 4.7.

At the circulating roadway, the passenger cars increase from 93.36 percent (dry) to 92.76 percent (LR) and increases to 93.24 percent (MR) and 96.09 percent (HR); the medium vehicles increase from 3.42 percent (dry) to 3.56 percent (LR), 4.32 percent (MR) and reduces to 2.47 percent (HR). The heavy vehicles increase from 2.13 percent (dry) to 2.87 percent (LR), and reduces to 1.87 percent (MR), and 1.44 percent (HR) as presented in Table 4.8. Despite the changes in weather conditions, passenger cars are the dominant entry and circulating vehicles at both peak and off-peak periods in dry and rainy conditions. The dry traffic flow rate was taken as the control.

Table 4.6: Peak traffic composition at site 01

Type of vehicles	Composition (%)	
	Entry	Circulating
Passenger cars	94.3	90.82
Medium vehicles	2.93	4.97
Heavy vehicles	2.12	3.19
Motorcycles	0.65	1.02

Table 4.7: Off-peak entry traffic composition at site 01

Type of vehicles	Composition under weather conditions (%)			
	Dry	Light rain	Moderate rain	Heavy rain
Passenger cars	92.14	93.44	95.35	91.57
Medium vehicles	4.05	3.48	3.06	5.57
Heavy vehicles	2.20	2.36	1.48	2.86
Motorcycles	1.62	0.72	0.11	0.00

Table 4.8: Off-peak circulating traffic composition at site 01

Type of vehicles	Composition under weather conditions (%)			
	Dry	Light rain	Moderate rain	Heavy rain
Passenger cars	93.36	92.76	93.24	96.04
Medium vehicles	3.42	3.56	4.32	2.47
Heavy vehicles	2.13	2.87	1.87	1.44
Motorcycles	0.46	0.80	0.58	0.00

4.2.2: Site 02: Millennium Roundabout

The Millennium roundabout features have been described in section 3.5.2. The site set up for data collection is shown in Figure 4.7.



Figure 4.7. Site 02 Millennium Roundabout

The geometry of the roundabout is: entry width is 7.9m, the entry angle is 45° , entry radius is 50m, effective flare length is 18m, the inscribed circle diameter is 58.10m, approach half width and the circulating road width are 6.80m and 9.30m respectively as presented in Table 4.1.

4.2.2.1 Site 02: Millennium Roundabout Rain Data

The rain gauge that covered site 02 is a rain gauge with the station ID Umhnhth (see appendix C) located along Umhlanga North Reservoir – Umhlanga Rocks Drive which is 1.18km away from site 02. The daily rain precipitation was collected from 15 September 2016 to 8 November 2016. The total occurrence of rainy days at this site was 42 days as shown in Figure 4.8. There was light, moderate, and heavy rainfall at this location at the time of the survey. The typical five minutes' daily rain precipitation is shown in Figure 4.9.

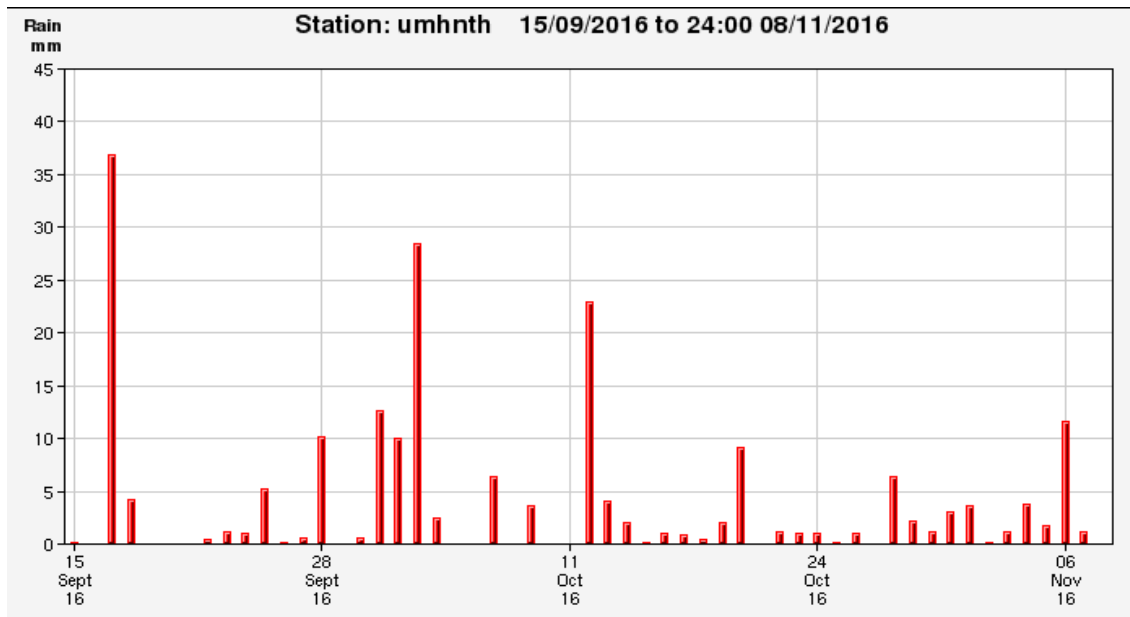


Figure 4.8: The daily rain precipitation at Umhnth rain gauge station

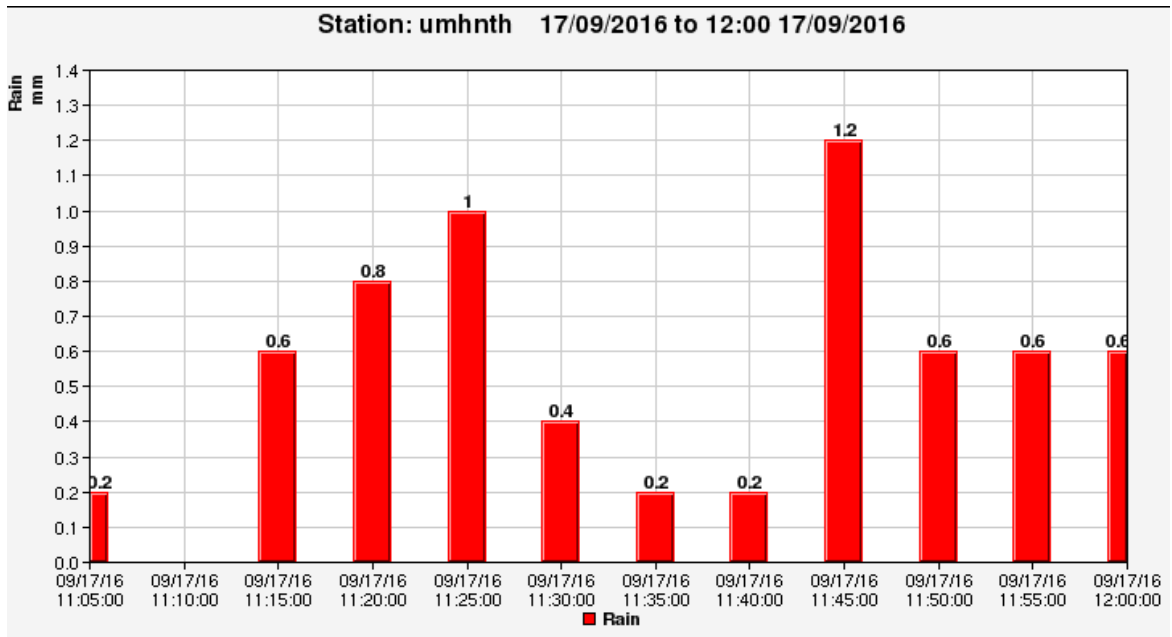


Figure 4.9: Typical five minutes' rain precipitation at Umhnth rain gauge station

4.2.2.2 Site 02: Millennium Roundabout Traffic Flow rate Profile Data

The traffic data was collected simultaneously at the entry and circulating roadways from 15 September 2016 to 08 November 2016. The peak flow rate occurs during the weekdays and the flow rate during the weekends are low with the lowest occurring on Sunday at both the entry and circulating roadways. There is almost a uniform flow rate at the entry which depicts a noncontinuous flow rate because the entry vehicles obey the give way rule of the roundabout whereby the entry vehicle has to reduce speed and at times stop at the yield line to look for a safe gap within the circulating flow rate before entering the roundabout (see Figure 4.10). The circulating flow rate is more uniform because circulating vehicles have a continuous flow rate and are independent of the entry traffic flow rate (see Figure 4.11). As at the time of collecting the data at entry, the ATC was not set up for data collection for the period from 23 to 30 September 2016 after downloading on 22 September 2016. An additional one week's traffic data was collected at both the entry and circulating roadways from 01 to 08 November 2016 to make up the lost period in the data collection.

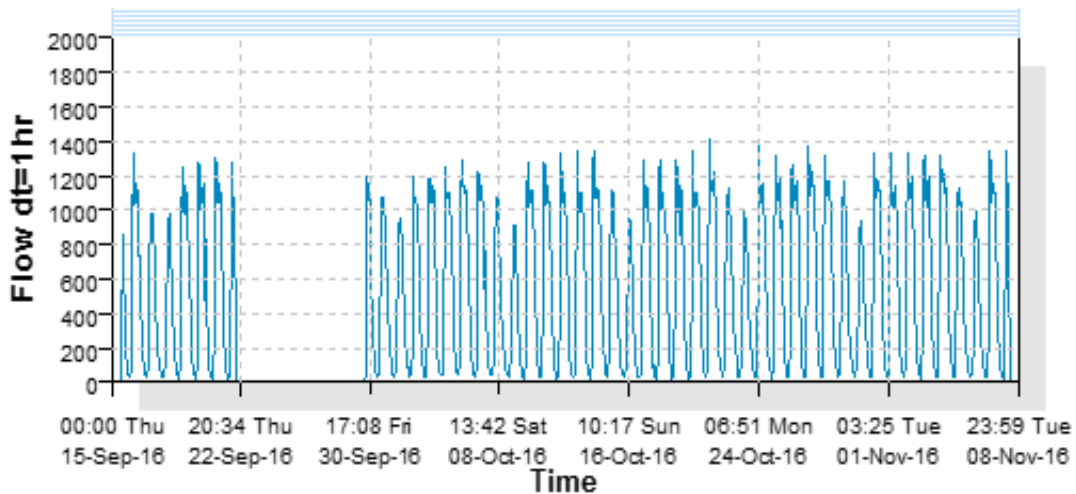


Figure 4.10: Entry traffic flow rate profile for site 02

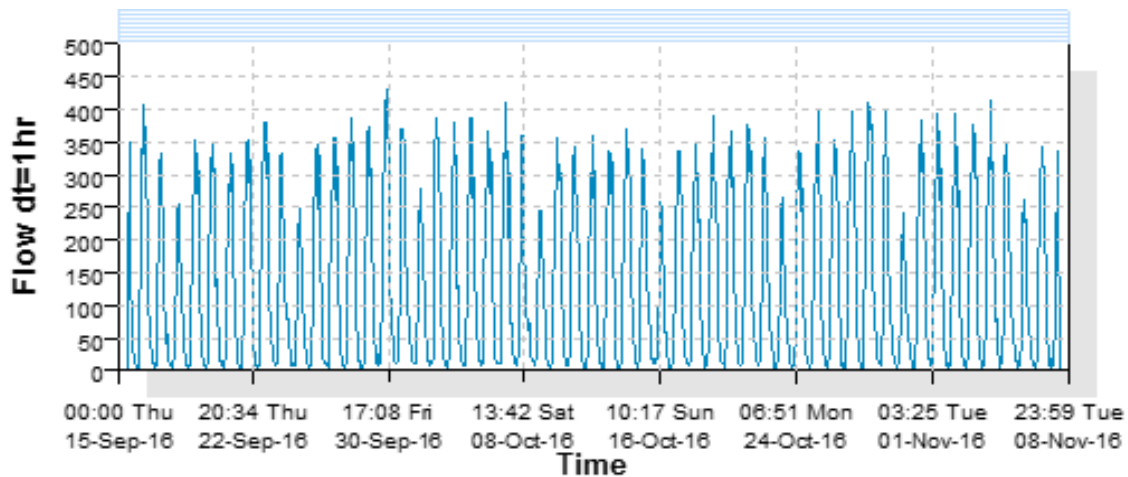


Figure 4.11: Circulating traffic flow rate profile for site 02

4.2.2.3 Site 02: Millennium Roundabout Traffic Volume Data

The total entry and circulating vehicle data collected at site 02 for the period 15 September 2016 to 8 November 2016 was 821,264, of which the circulating vehicles were 315,870 and entry vehicles were 505,394. This data was collected continuously under different weather and traffic conditions for six weeks.

The morning hourly peak period is generally between 08:00 to 09:00 when people are going to work in the morning while the afternoon peak period generally occurs during 13:00 to 14:00, which is the general break time in Durban when most people leave their duty post for lunch and other activities. The hourly peak circulating, and entry traffic flow rate is shown in Table 4.8 and the hourly off-peak entry and circulating traffic flow rate under dry and rainy conditions is shown in Tables 4.10 and 4.11 respectively.

Table 4.8 Hourly peak traffic flow rate under dry weather at site 02

Flow rate (pce/h)	Period											
	1	2	3	4	5	6	7	8	9	10	11	12
Circulating	697	655	731	330	353	493	545	561	538	386	413	521
Entry	1271	1159	1123	1588	1596	1434	1310	1462	1327	1478	1663	1344

Table 4.10: Hourly off-peak entry traffic flow rate at site 02

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	1400	1381	1224	1152
2	1350	1349	1296	737
3	1421	1397	1058	1164
4	1365	1351	1032	1068
5	1333	1333	1378	998
6	1305	1305	1224	950
7	1320	1321	1200	1020
8	1350	1349	1248	1188
9	1384	1393	1176	1188
10	1301	1320	1224	1116
11	1333	1323	1308	1104
12	1327	1350	1246	1116

Table 4.11: Hourly off-peak circulating traffic flow rate at site 02

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	565	568	571	360
2	606	600	475	734
3	560	557	658	430
4	580	580	715	569
5	615	620	418	588
6	646	636	547	667
7	620	600	562	63
8	579	601	370	427
9	564	564	598	406
10	626	621	492	456
11	635	626	480	523
12	605	602	516	502

The entry and circulating traffic consist of heterogeneous traffic. The peak traffic at the entry consists of 93.86 percent passenger cars, 3.54 percent medium vehicles, 2.05 percent heavy vehicles, and 0.55 percent motorcycles, while the circulating peak traffic consists of 78.70 percent passenger cars, 10.91 percent medium vehicles, 9.09 percent heavy vehicles and 1.30 percent motorcycles as presented in Table 4.12. The off-peak entry vehicle composition shows that the passenger cars increase with light and moderate rain but reduce in heavy rain. Medium vehicles and motorcycles reduce with the increase in rain intensity while the heavy vehicles increase with an increase in rain intensity as presented in Table 4.13. At the circulating roadway, passenger cars constitute the dominant circulating vehicles, with passenger cars and motorcycles reducing as the rain intensity increases but the medium and heavy vehicles increase with an increase in rain intensity as shown in Table 4.14. Despite the changes in the variation in vehicle composition, passenger cars constitute the dominant vehicle, and this suggests that it can affect the traffic behaviour under rainy condition.

Table 4.12: Peak traffic composition at site 02

Type of vehicles	Composition (%)	
	Entry	Circulating
Passenger cars	93.86	78.70
Medium vehicles	3.54	10.91
Heavy vehicles	2.05	9.09
Motorcycles	0.55	1.30

Table 4.13: Off-peak entry Traffic composition at the site 02

Type of vehicles	Composition under weather conditions (%)			
	Dry	Light rain	Moderate rain	Heavy rain
Passenger cars	94.29	95.04	95.05	93.43
Medium vehicles	3.14	2.24	2.12	2.09
Heavy vehicles	1.93	2.30	2.39	4.28
Motorcycles	0.64	0.40	0.44	0.20

Table 4.14: Circulating traffic composition at site 02

Type of vehicles	Traffic Composition under weather conditions (%)			
	Dry	Light rain	Moderate rain	Heavy rain
Passenger cars	90.58	86.09	88.86	87.44
Medium vehicles	5.00	6.95	6.01	5.53
Heavy vehicles	3.85	5.93	4.64	6.98
Motorcycles	0.58	1.02	0.45	0.23

4.2.3: Site 03: Douglas Roundabout

Douglas roundabout features were described in section 3.5.3. The data collection set-up is shown in Figure 4.12.



Figure 4.12: Site 02 Douglas Roundabout

The geometry of the roundabout consists of the entry width of 8.2m, the entry angle is 45°, entry radius is 50m, the effective flare length is 15m, the inscribed circle diameter is 49.50m, approach half width and the circulating road width are 6.90m and 9.10m respectively as presented in Table 4.1.

4.2.3.1: Site 03: Douglas Roundabout Rain Data

The closest rain gauge to site 03 is the rain gauge with station ID Umhnth (see appendix C) at 0.81km from the Douglas roundabout. The rain data was collected from 08 November 2016 to 07 December 2016. There was a total of 24 days' rainfall of varying precipitation within the survey period as shown in Figure 4.13. There was light, moderate, and heavy rainfall at this location at the time of the survey as shown in the typical five minutes' daily rain precipitation in Figure 4.14.

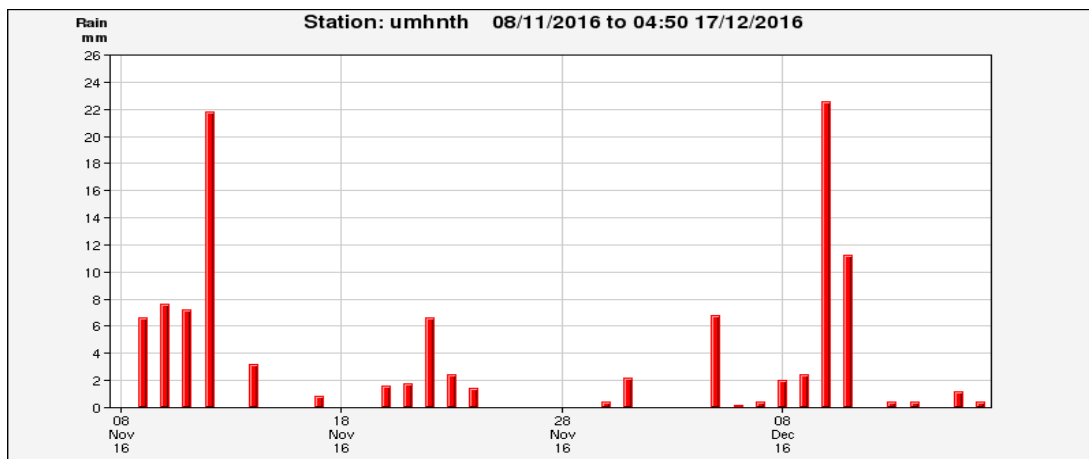


Figure 4.13: The daily rain precipitation at Umhnth rain gauge station

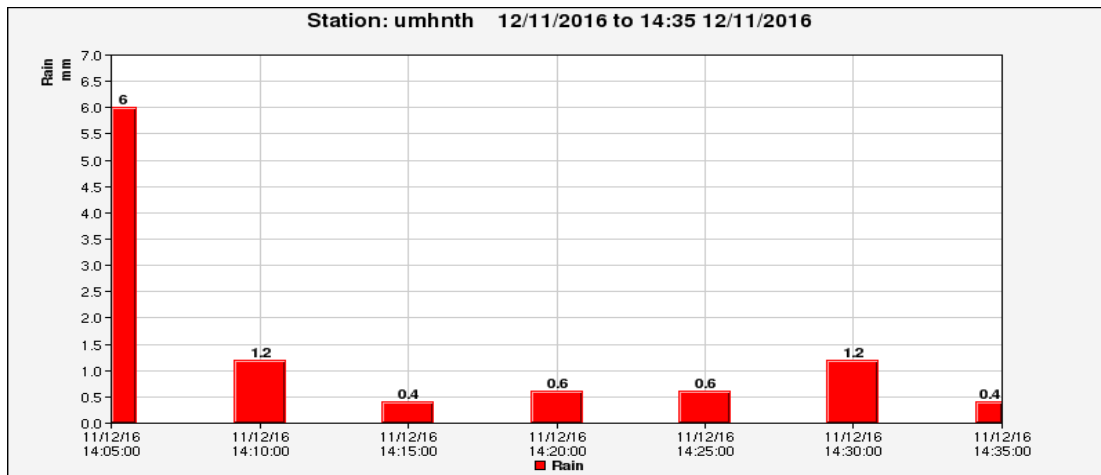


Figure 4.14: Typical five minutes' rain precipitation at Umhnhth rain gauge station

4.2.3.2 Site 03: Douglas Roundabout Traffic Flow Rate Profile Data

The entry and circulating traffic data at this site were collected from 08 November 2016 to 21 December 2016. The peak traffic flow rate occurrence of entry and circulating traffic was on weekdays, which is Monday to Friday, while the low traffic flow rate occurs during the weekend, which is Saturday and Sunday, but the lowest occurred on Sunday. The entry flow rate pattern fluctuation shows that the flow rate at the entry roadway at site 03 was not continuous because of the give way rule of the roundabout operation, whereby entry vehicles yield to the circulating vehicles (see Figure 4.15). The circulating flow rate pattern is almost uniform, which depicts that the flow rate at the circulating roadway is a continuous flow rate and independent of entry traffic flow rate (see Figure 4.16).

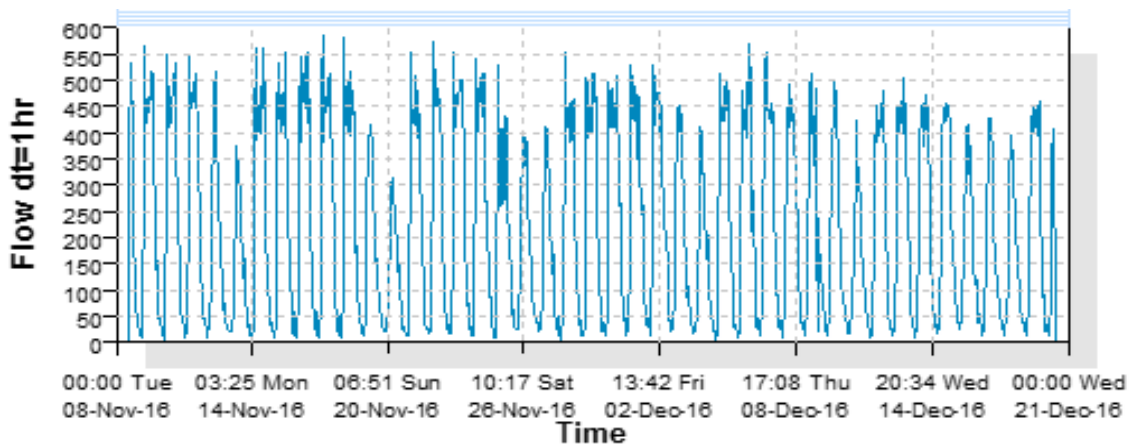


Figure 4.15: Entry traffic flow rate profile for site 03

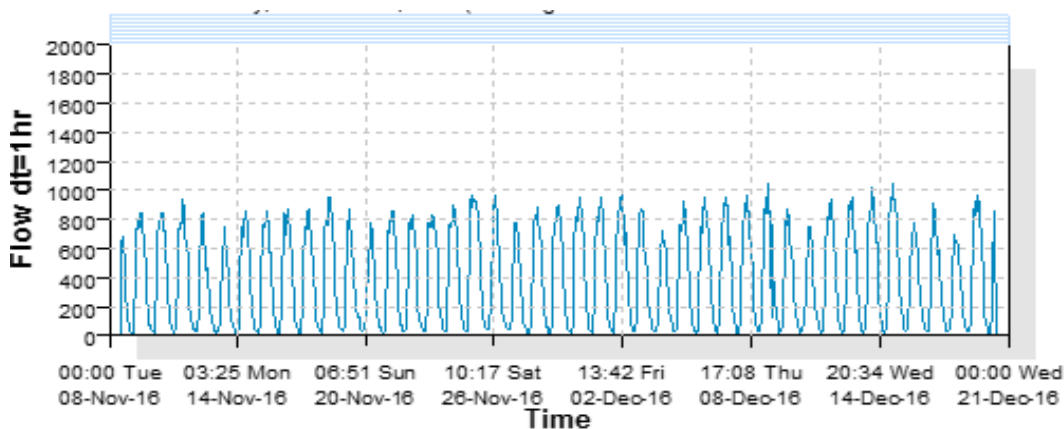


Figure 4.16: Circulating traffic flow rate profile for site 03

4.2.3.3 Site 03: Douglas Roundabout Traffic Volume Data

The traffic volume is made up of different types of vehicles and the data was collected continuously for six weeks. The vehicles' data were collected under dry and rainy conditions as well as peak and off-peak traffic conditions because of the continuous collection of data for six weeks (24 hours on each day). The total volume of vehicles collected at this site was 695,495, of which the entry vehicles were 261,412 and the circulating vehicles were 398,083. The data was collected continuously under varying weather and traffic condition for six weeks.

The hourly peak circulating flow rate (q_c) and entry flow rate (q_e) under dry weather condition are shown in Table 4.15. The off-peak entry and circulating traffic flow rate under dry and rainy conditions are presented in Tables 4.16 and 4.17 respectively. The rainy conditions were classified according to the World Meteorological Organisation's. (WMO) rainfall classification into light rain with intensity (i) $< 2.5\text{mm/h}$, moderate rain ($2.5 < i \leq 10\text{mm/h}$) and heavy rain ($10\text{mm/h} < i \leq 50\text{mm/h}$).

Table 4.15: Site 03 Hourly peak traffic flow rate under dry weather condition.

Flow rate (pce/h)	1	2	3	4	5	6	7	8	9	10	11	12
Circulating	718	712	492	502	897	619	679	463	741	969	888	979
Entry	1278	1289	1360	1362	984	1300	1310	1397	1278	912	936	888

Table 4.16: Hourly off-peak entry traffic flow rate at site 03

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	1260	1248	1176	1056
2	1200	1248	1248	684
3	1284	1308	1008	1080
4	1188	1308	972	1092
5	1116	1176	1284	960
6	1140	1164	1164	900
7	1188	1308	1152	1008
8	1140	1200	1176	1104
9	1212	1224	1128	1092
10	1224	1236	1188	1092
11	1128	1224	1224	1128
12	1152	1236	1140	1056

Table 4.17: Hourly off-peak circulating traffic flow rate at site 03

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	612	588	600	432
2	600	576	480	684
3	552	588	684	480
4	624	648	696	576
5	576	612	444	588
6	648	660	600	636
7	660	612	588	684
8	576	624	420	408
9	540	516	636	468
10	624	624	564	528
11	636	624	492	516
12	600	612	564	528

The passenger cars form the dominant entry and circulating flow rate irrespective of the peak or off-peak traffic and the weather conditions. The peak traffic composition under dry weather is shown in Table 4.18, the off-peak entry and circulating traffic composition is shown in Tables 4.19 and 4.20.

Table 4.18: Peak traffic composition at site 03

Type of vehicles	Composition (%)	
	Entry	Circulating
Passenger cars	94.20	86.08
Medium vehicles	2.21	7.10
Heavy vehicles	2.08	5.97
Motorcycles	0.91	0.85

Table 4.19: Entry traffic composition at the site 03

Type of vehicles	Composition under weather conditions (%)			
		Light	Moderate	Heavy
	Dry	rain	rain	rain
Passenger cars	94.00	94.37	94.00	93.47
Medium vehicles	3.32	2.28	2.67	2.78
Heavy vehicles	2.22	2.46	3.05	3.75
Motorcycles	0.46	0.35	0.29	0.00

Table 4.20: Circulating traffic composition at site 03

Type of vehicles	Composition under weather conditions (%)			
		Light	Moderate	Heavy
	Dry	rain	rain	rain
Passenger cars	87.55	87.43	86.84	86.24
Medium vehicles	6.22	5.79	6.58	6.65
Heavy vehicles	5.82	6.19	6.58	7.11
Motorcycles	0.40	0.60	0.00	0.00

4.2.4 Site 04: Gateway Roundabout

Gateway roundabout was described in detail in section 3.5.4. The roundabout set up for traffic data collection is shown in Figure 4.17.



Figure 4.17: Site 02 Gateway Roundabout

The geometry is made up of 8.4m entry width, 50° entry angle, 40m entry radius, 13m effective flare length, 48m inscribed circle diameter, approach half width and the circulating road width are 6.80m and 8.80m respectively as presented in Table 4.23.

4.2.3.1 Site 04: Gateway Roundabout Rain Data

The rain station catchment that covers site 04 is the rain gauge with the station ID Umhnh (see appendix C) along Umhlanga North Reservoir – Umhlanga Rocks Drive. The rain gauge is 0.75km away from the surveyed roundabout. There were 16 rainy days at the surveyed period from 20 December 2016 to 30 January 2017, as shown in the daily rain precipitation in Figure 4.18. There was light, moderate and heavy rainfall during the surveyed period. A typical five minutes' rain precipitation amount is shown in Figure 4.19.

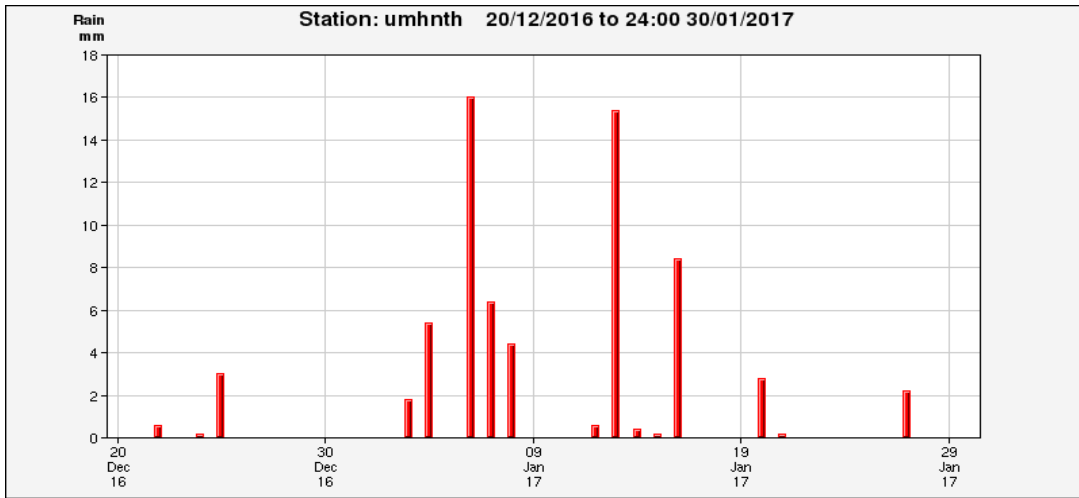


Figure 4.18: The daily amount of rain precipitation at Umhnth rain gauge station

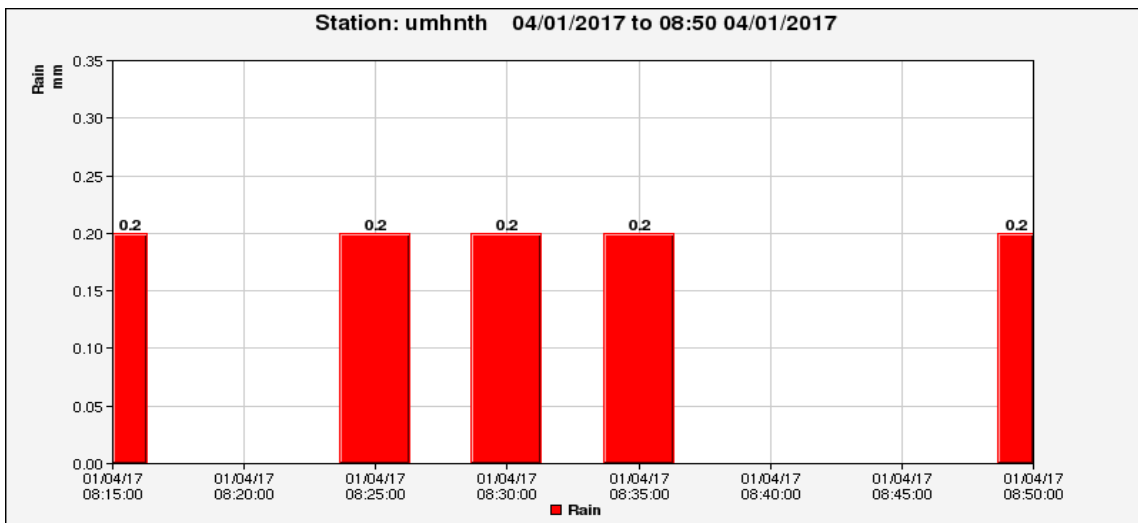


Figure 4.19: Typical five minutes' rain precipitation at Umhnth rain gauge station

4.2.3.2 Site 04: Gateway Roundabout Traffic Flow Rate Profile Data

The entry and circulating traffic data were collected for six weeks from 20 December 2016 to 01 February 2017. The peak entry and circulating traffic flow rate occur at weekdays, which are Monday to Friday while the low traffic flow rate occurs during the weekend, which is the Saturday and Sunday.

The flow rate behaviour at this site follows the same trend as the other surveyed sites as the entry flow rate fluctuates, which shows that it is dependent of the circulating flow rate and the circulating flow rate is more of a uniform flow rate than the entry flow rate. The entry and circulating flow rate profile is presented in Figures 4.20 and 4.21 respectively.

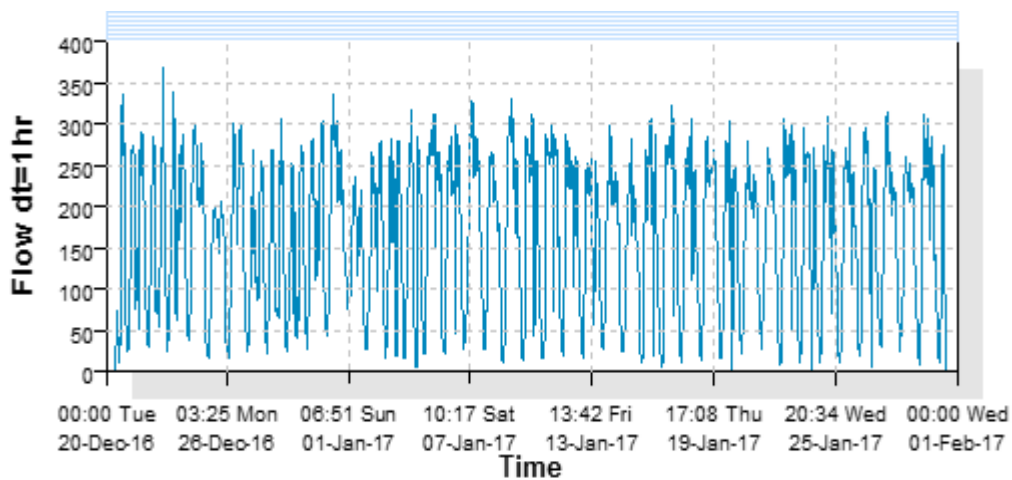


Figure 4.20: Entry traffic flow rate profile for site 04

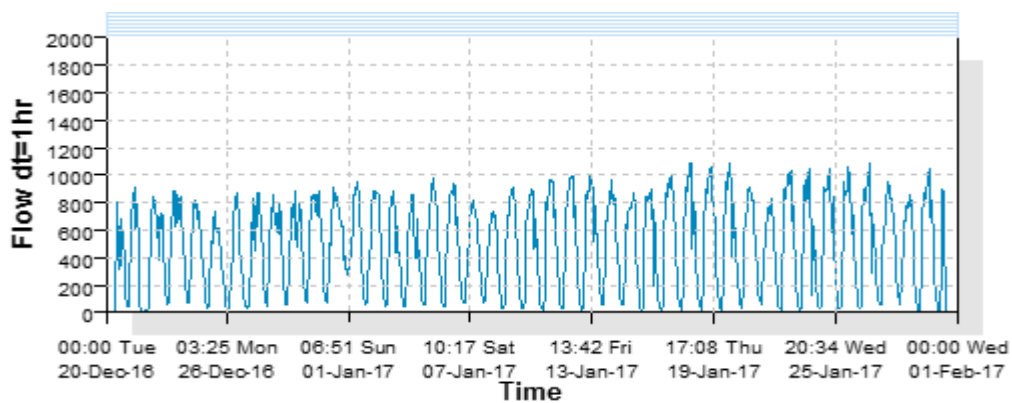


Figure 4.21: Circulating traffic flow rate profile for site 04

4.2.3.3 Site 04: Gateway Roundabout Traffic Volume Data

The total vehicle volume collected at the site 04 was 768,933. The entry vehicles were 247,802 and circulating vehicles were 516,131. The hourly peak traffic entry and circulating flow rate under dry weather conditions are shown in Table 4.21. The off-peak entry and circulating traffic flow rate under dry and rainy conditions are shown in Table 4.22 and 4.23 respectively. The rainy conditions were classified into light rain, moderate rain, and heavy rain according to the World Meteorological Organisation's rainfall classification.

Table 4.21: Hourly peak traffic flow rate under dry weather condition at site 04

Flow rate (pce/h)	Period											
	1	2	3	4	5	6	7	8	9	10	11	12
Circulating	1190	1128	1099	936	1056	986	802	958	835	1130	1255	1258
Entry	734	910	897	1005	921	1029	1054	960	1006	746	569	650

Table 4.22: Hourly off-peak entry traffic flow rate at site 04

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	468	888	840	984
2	984	852	804	672
3	888	1188	840	552
4	912	1020	900	744
5	1104	1176	936	624
6	888	864	1020	720
7	876	864	1032	960
8	876	816	1056	768
9	840	864	900	720
10	732	1044	876	636
11	888	984	948	708
12	864	960	936	744

Figure 4.23: Hourly off-peak circulating traffic flow rate at site 04

Period	Dry	Light Rain	Moderate Rain	Heavy Rain
	pce/h	pce/h	pce/h	pce/h
1	1104	936	732	744
2	828	1020	852	1008
3	948	780	792	1140
4	840	936	684	1068
5	756	720	636	972
6	912	900	588	948
7	948	924	576	708
8	936	984	600	984
9	900	804	756	1092
10	984	816	660	1032
11	924	864	744	984
12	852	984	708	900

The entry and circulating traffic volume is made up of different types of vehicles under different weather and traffic conditions because the traffic data was collected continuously for six weeks. Irrespective of the period and weather conditions passenger cars form the dominant vehicles at entry and circulating traffic. The peak traffic composition is presented in Table 4.24, and off-peak entry and circulating traffic composition is presented in Tables 4.25 and 4.26 respectively.

Table 4.24: Peak traffic composition at site 04

Type of vehicles	Composition (%)	
	Entry	Circulating
Passenger cars	93.64	90.02
Medium vehicles	2.82	5.87
Heavy vehicles	2.92	3.91
Motorcycles	0.60	0.20

Table 4.25: Entry off-peak Traffic composition at the site 04.

Type of vehicles	Composition under weather conditions (%)			
	Dry	Light rain	Moderate rain	Heavy rain
Passenger cars	91.10	92.20	92.53	90.48
Medium vehicles	4.90	4.77	4.90	6.03
Heavy vehicles	2.19	2.21	2.45	3.49
Motorcycles	1.81	0.81	0.12	0.00

Figure 4.26: Circulating traffic composition at site 04

Type of vehicles	Composition under weather conditions (%)			
	Dry	Light rain	Moderate rain	Heavy rain
Passenger cars	91.96	91.50	89.15	93.00
Medium vehicles	4.95	3.81	6.44	3.97
Heavy vehicles	2.48	3.81	3.73	3.03
Motorcycles	0.62	0.89	0.68	0.00

4.3 Summary

The empirical results from the four surveyed roundabouts have been presented in this chapter. The geometry of all the roundabouts fall within South Africa's roundabout specifications and all the roundabouts are standard double lane roundabouts.

The rain data was presented, and the number of rainy days varies from site to site. The rainfall intensity **varied** with time and location. The highest number of rainy days was recorded at site 02 with 42 rainy days, and the lowest was recorded at 01 with 10 rainy days. The light, moderate and heavy rainfall were recorded at four surveyed sites. The rain **was** classified according to the rain

intensity as light rain (LR) with intensity (i) $< 2.5\text{mm/h}$, moderate rain (MR) with intensity $> 2.5\text{mm/h}$ but $\leq 10\text{mm/h}$, heavy rain (HR) with intensity $> 10\text{mm/mm/h}$ but $\leq 50\text{mm/h}$.

The entry and circulating traffic flow rate under the dry and rainy weather of different intensities were presented. The highest vehicle volume was recorded at Millennium roundabout with a volume of 315,870 circulating vehicles and 505,394 entry vehicles making a total of 821,264 vehicles, and the lowest at Douglas roundabout with the volume of entry vehicles being 261,412 and 398,083 circulating vehicles making a total of 695,495 vehicles.

Passenger cars **were** the dominant vehicles at both the entry and circulating traffic at all the four surveyed sites. The average entry traffic composition was 93.34 percent passenger cars, 3.59 percent medium vehicles, 2.60 percent heavy vehicles, and the average circulating traffic consisted of 90.13 percent passenger cars, 5.10 percent medium vehicles, and 4.28 percent heavy vehicles at off-peak periods. The average entry and circulating vehicles at peak periods were 94.00 percent passenger cars, 2.88 percent medium vehicles, 2.29 percent heavy vehicles at entry roadway. At the circulating roadway the average vehicle volume **was** 86.41 percent passenger cars, 7.19 percent medium vehicles and 5.54 percent heavy vehicle. The circulating vehicles **had** higher speeds than the entry vehicles at all the sites. This shows that the circulating vehicles have a continuous flow rate while the entry vehicles obey the yield rule. The data presented in this chapter **are** analysed in the next chapter.

CHAPTER 5

FUNCTIONAL QUALITY OF SERVICE ASSESSMENT

5.1 Overview

In the previous chapter, empirical survey data were presented. The data were used in this chapter to determine the quality of service criteria table, delay and reserve capacity per site under dry and rainy weather conditions and is reported in this chapter. The ensuing criteria table was used to assess the prevailing multilane roundabout service delivery per site and further discussed. Roundabouts are designed to carry traffic loads; therefore, it is appropriate for road providers to check from time to time their prevailing reserve capacity. Road users, on the other hand, are more interested in prevailing delays and queues. Consequently, it can be argued that the quality of service encompasses road providers' and users' perceptions of roundabout service delivery. Note that quality of service in this thesis has an added appellation, 'functional'; hence a functional quality of service as opposed to a structural quality of service. In any case, the remainder of the chapter has been divided into five sections. In sections 5.2 to 5.5, the roundabout functional quality of service (FQS) is determined for site 01, 02, 03 and 04 respectively. FQS criteria table is developed for each site using the peak day-light traffic data, the dry and rainy off-peak traffic data is used to determine the operational performance at each roundabout using the users' and providers' perception of the roundabout's operational measure. The reserved capacity and volume capacity ratio are used as the providers' perception parameter, while delay and queue length are used for the users' performance perception. The roundabout service delivery is assessed under dry and rainy conditions with the FQS criteria table for each site. In section 5.6, the summary of functional quality of service under dry and rainy weather conditions at all sites are compared and analysed. The chapter summary is in section 5.7.

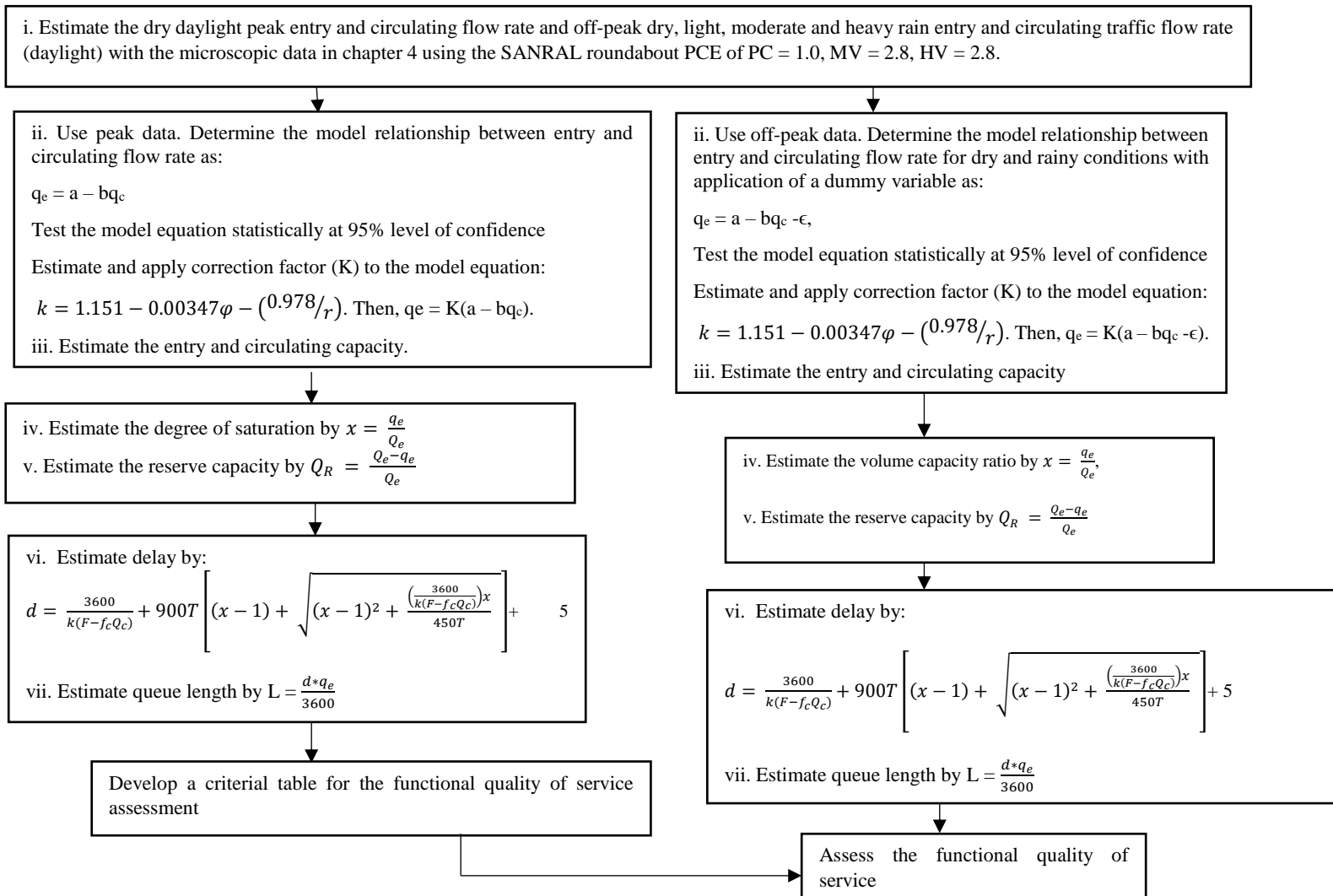


Figure 5.1 Procedure for determining roundabout functional quality of service delivery

5.2 Roundabout Functional Quality of Service (FQS) Determination

As shown above in figure 5.1, the procedure used to determine prevailing roundabout functional service delivery is made up of two stages. At stage one, the site criterial table is developed from peak traffic and geometric data during dry weather conditions. At stage two the prevailing roundabout operational performance is estimated. Note that stage two traffic data is based on off-peak traffic performance during dry, and rainy conditions. For ease of explanation, a stepwise analytical procedure is used for all sites. Only the analytical procedures for site 01 (Armstrong roundabout) are fully explained, in order to minimise repetitive explanations, the analysis for the remainder sites are summarised and discussed.

5.2.1 Criterial table development for site 01 (Armstrong Roundabout)

The procedure and the techniques adopted in the development of the functional quality of service assessment criterial table is presented in figure 5.1. The peak traffic data are used for the criterial table development. The functional quality of service has been shown to be multiparameter in chapter 2. These parameters represent the users' and the roundabout providers' perceptions. The reserved capacity and delay are the main parameters that represent the users' and providers' perceptions in the criterial table. The volume capacity ratio and queue length are also in the criterial table because they were used in the assessment table development and are also considered as the providers' and users' perceptions. The reserved capacity forms the uniqueness of the table because it has not been used for assessment of roundabouts before now. These parameters will be estimated using peak period data to develop the criterial table of FQS assessment. The criterial table of assessment will be developed for each site. The reason is because each site has its traffic and environmental conditions. To have a clear understanding of the influence of rainfall on each site the criterial table developed for each site will be used for the assessment at the sites.

The stepwise procedure method is used for multilane roundabout functional quality of service (FQS) criterial table development for simplicity. Site 01 traffic is used to demonstrate the steps and the same procedure is used in the development of the criterial table for the remaining three sites.

Step 01: The five-minute peak vehicle volume collected at this site is converted to traffic flow rate with the use of SANRAL PCE values, which are: passenger cars (1.00), medium vehicles (2.80) and heavy vehicles (2.80) for the roundabout. The estimated PCE for each class of vehicles are added together to determine the flow rate per five minutes and further multiplied by 12 to convert to flow rate per hour.

As an example, for the computation of traffic flow rate:

The number of collected vehicles in five minutes are:

Passenger cars = 81 veh.

Medium vehicles = 2 veh.

Heavy vehicles = 2 veh.

Convert the heterogeneous traffic volume to homogeneous traffic flow rate by application of the **SANRAL PCE** value of 2.8 for medium and heavy vehicles.

Passenger cars = 81 pce/5 min

Medium vehicles = $2 \times 2.8 = 5.6$ pce/5min

Heavy vehicles = $2 \times 2.8 = 5.6$ pce/5min

Total traffic flow rate in five min = $81 + 5.6 + 5.6 = 92.2$ pce/5 min

Traffic flow rate per hour = $92.2 \times 12 = 1106.4 \approx 1106$ pce/h

The computed peak entry and circulating traffic flow rate under dry, light, moderate and heavy rain weather conditions for site 01 are presented in tables 5.1 to 5.2.

Table 5.1: Computed peak entry flow rate under dry daylight at site 01

Col. 1	Col. 2	Col. 3	Col. 4 Col. 2*2.8	Col. 5 Col.3*2.8	Col. 6 Σ col. 1,4,5	Col. 7 Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
81	2	2	5.6	5.6	92.2	1106
125	1	0	2.8	0	127.8	1534
88	3	1	8.4	2.8	99.2	1190
61	5	2	14	5.6	80.6	967
68	3	3	8.4	8.4	84.8	1018
90	4	2	11.2	5.6	106.8	1282
102	1	0	2.8	0	104.8	1258
79	5	1	14	2.8	95.8	1150
65	3	1	8.4	2.8	76.2	914
66	2	2	5.6	5.6	77.2	926
108	1	3	2.8	8.4	119.2	1430
61	4	3	11.2	8.4	80.6	967

Note: Col = Column, 2.8 = South Africa PCE value, MV = Medium vehicle, HV= Heavy vehicle

Table 5.2: Computed peak circulating flow rate under dry daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4 Col. 2*2.8	Col. 5 Col. 3*2.8	Col. 6 Σ col. 1,4,5	Col. 7 Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
55	3	2	8.4	5.6	69	828
31	3	4	8.4	11.2	50.6	607
57	4	1	11.2	2.8	71	852
85	1	0	2.8	0	87.8	1054
78	2	2	5.6	5.6	89.2	1070
46	4	3	11.2	8.4	65.6	787
44	5	3	14	8.4	66.4	797
58	4	1	11.2	2.8	72	864
76	3	1	8.4	2.8	87.2	1046
89	3	1	8.4	2.8	100.2	1202
31	3	4	8.4	11.2	50.6	607
62	4	3	11.2	8.4	81.6	979

Note: Col = Column, 2.8 = South Africa PCE value, MV = Medium vehicle, HV= Heavy vehicle

Step 2: The entry and circulating flow rate are analysed with linear regression where the entry flow rate is taken as the dependent variable because it depends on the circulating flow rate to

enter the roundabout. The entry and circulating flow rate for the analysis is presented in table 5.3. The peak entry – circulating flow rate relationship is shown in figure 5.2 and with the model equation.

Table 5.3: Peak circulating and entry flow rate at site 01.

Period	1	2	3	4	5	6	7	8	9	10	11	12
q_c (pce/h)	828	607	852	1053	1070	787	796	864	1048	1202	607	979
q_e (pce/h)	1106	1534	1190	967	967	1018	1282	1258	1150	914	1430	967

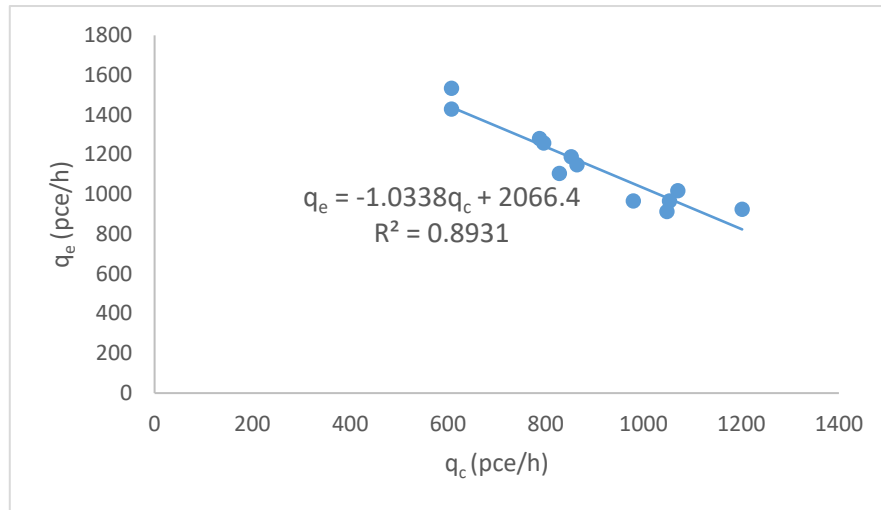


Figure 5.2: Peak entry and circulating traffic empirical relationship for site 01

$$q_e = 2066 - 1.034 q_c \quad [5.1]$$

The statistical testing was at a 95% level of confidence, the statistical testing shows that the coefficient of the determinant (R^2) is 0.89 which is more than 0.5 which shows that the model equation is reliable, the t-test is more than 2.2 which shows that variables are significant, the F-test is more than 4.84 which shows that the model equation did not occur by chance, and the P-value is less than 0.05. The model equation could be used for prediction because it is statistically fit. The summary of the ANOVAL analysis output is presented in table 5.4.

Table 5.4: Summary of ANOVA output for peak entry and circulating flow rate relationship at site 01.

SUMMARY OUTPUT								
<i>Regression Statistics</i>								
Multiple R	0.945049							
R Square	0.8931177							
Adjusted R Squ	0.8824295							
Standard Error	69.234308							
Observations	12							
<i>ANOVA</i>								
	<i>df</i>	<i>SS</i>	<i>MS</i>	<i>F</i>	<i>Significance F</i>			
Regression	1	400539.7723	400539.7723	83.56087	3.59692E-06			
Residual	10	47933.89436	4793.389436					
Total	11	448473.6667						
	<i>Coefficients</i>	<i>Standard Error</i>	<i>t Stat</i>	<i>P-value</i>	<i>Lower 95%</i>	<i>Upper 95%</i>	<i>Lower 95.0%</i>	<i>Upper 95.0%</i>
Intercept	2066.3757	102.7386664	20.11293124	2.03E-09	1837.459719	2295.291747	1837.459719	2295.291747
qc (pce/h)	-1.033808	0.113093701	-9.141163273	3.6E-06	-1.285796455	-0.781819517	-1.285796455	-0.781819517

Step 3: Kimberly (1980) specifies an entry angle (ϕ) of 0 - 77⁰ and an entry radius (r) of 3.4m - ∞ geometry for roundabouts. Provided the parameters are within the specified range, there is no need to have a new correction factor model. The Kimberly equation will be adopted for the estimation of the correction factor (k).

$$k = 1.151 - 0.00347\phi - (0.978/r) \quad [5.2]$$

Where:

k = the correction factor

ϕ = entry angle (degree)

r = entry radius (m)

The entry angle at site 01 is 50⁰, and the entry radius is 40m (these two parameters fall within Kimberly's specification).

Substitute the values of entry angle and radius in equation 5.2,

k = 0.95.

Apply the correction factor k to equation 5.1, then equation 5.1 becomes:

$$q_e = 0.95(2066 - 1.034 q_c) = 1963 - 0.98q_c \quad [5.3]$$

Step 04: The entry capacity (Q_e) occurs when there is no circulating flow rate i.e. when $q_c = 0$, Substitute for $q_c = 0$ in equation 5.3. Then, $Q_e = 1963 - 0.98(0) = 1963$ pce/h for two lanes, on the assumption that the two lanes have the same capacity, $Q_e = 0.5 \times 1963 = 982$ pce/h/lane.

Step 05: The sensitivity test is conducted estimating the delay and the queue length by setting the volume capacity ratio = 0 and 1 using equations 5.4 and 5.5.

$$d = \frac{3600}{k(F-f_cQ_c)} + 900T \left[(x - 1) + \sqrt{(x - 1)^2 + \frac{\left(\frac{3600}{k(F-f_cQ_c)}\right)x}{450T}} \right] + 5 \quad [5.4]$$

Where:

d = control delay (s)

$k(F-f_cQ_c)$ = entry capacity (Q_e) per/ lane

x = volume capacity ratio

T is the time of observation = 0.25hr

Substitute for $K (F-f_cQ_c) = 982$ pce/h for a single lane, $T = 0.25$ hr, and $x = 0$ in equation 5.4.

Then, $d = 8.67$ s.

The queue length (L) is estimated using equation 5.5.

$$L = \frac{d * q_e}{3600} \quad [5.5]$$

Where:

L = queue length (veh)

d = control delay (s)

q_e = entry flow rate (pce/h)

At $d = 8.67s$, there is no entry flow rate, $x = 0$, then $q_e = 0$

substituting for d and q_e in equation 5.5.

$L = \frac{8.66 \times 0}{3600} = 0$, this shows that there is no entry vehicle at the roundabout at $x = 0$ and depicts that the estimated delay is the geometric delay.

When $x = 1$, $k(F-fcQc) = 982pce/h$ (from step 04), $T = 0.25hr$, substitute for x , $k(F-fcQc)$, and T in equation 5.4, then, $d = 49.29s$. This shows that the entry vehicle will experience 49.29s delay when the roundabout is operating at peak.

The queue length at peak = $\frac{49.29 \times 932}{3600} = 13 veh$. At capacity, the total number of vehicles on queue is 13 vehicles per lane.

The delay, queue length and the reserve capacity is estimated at a volume capacity ratio of 0 to 1 which is a division of ten equal parts of the volume capacity ratio.

The functional quality of service deteriorates as the reserve capacity decreases, delay increases, the queue length increases, and the volume capacity ratio increases. As the service delivery deteriorates, the queue length at the entry increases, these parameters are considered in the division of the functional quality of service classes. The results summary is presented in table 5.5.

Table 5.5: Summary of parameters for development of FQS criterial table at site 01

x	Q_R	d (s)	L (veh)
0	1.0	8.67	0
0.1	0.9	9.07	0
0.2	0.8	9.58	1
0.3	0.7	10.23	1
0.4	0.6	11.09	1
0.5	0.5	12.28	2
0.6	0.4	14.01	2
0.7	0.3	16.74	3
0.8	0.2	21.51	5
0.9	0.1	30.80	8
1	0.0	49.29	13

Note: x is volume capacity ratio, Q_R is reserved capacity, d is delay, L is queue length

The division of volume capacity ratio into ten divisions might be unrealistic in forming the FQS classes because of the closeness in delay values and there might be an overlap in the values of the delay parameter in each class. There is no method for checking the overlapping of parameter values in each class if it occurs because of a single unit division. In view of this, the division for the FQS table of five equal divisions of volume capacity ratio of 0.2 each is adopted. To avoid overlap, the standard deviation is estimated for each class. The standard deviation is applied to determine the extent of the deviation that could be within the lower and the upper limits of each class. σ and G are mean and standard deviations of each class where -1σ and 1σ are the upper and lower boundaries of each class.

For example, where volume capacity ratio (x) is 0, 0.1 and 0.2,

the delay values are 8.67s, 9.07s and 9.58s respectively, the mean value = 9.11s and the standard deviation is 0.5s.

Hence, the lower limit = $9.11 - 0.5 = 8.61s$

The upper limit = $9.11 + 0.5 = 9.61s$

The upper limit is above the delay value at a volume capacity ratio of 0.2. Comparing the queue length, it is discovered that the queue length at volume capacity ratio of 0.2 to 0.4 is the same as with the queue length of one vehicle. Thus, these classes can be grouped together as a volume capacity ratio of 0 to 0.4 because of the overlap in queue length.

The same procedure is used for the determination of the upper and lower limits of the class of volume to capacity ratio of 0 to 0.4. The delay values from table 5.5 are 8.67s, 9.07s, 9.58s, 10.23s and 11.09s.

The mean value = 9.73s

The standard deviation = 0.96s

Then, the lower limit = $9.73 - 0.96 = 8.77s$

The upper limit = $9.73 + 0.96 = 10.70 \approx 11s$

The upper limit is within the volume capacity ratio of 0.4 and does not overlap with the delay value of 12.28s at the volume capacity ratio of 0.5. This class is taken as FQS A.

The next division is a volume capacity ratio of 0.5 and 0.6.

The delay values are 12.28s and 14.01s

The mean delay value = $\frac{12.28+14.01}{2} = 13.15s$

The standard deviation estimated = 1.42s

The lower delay limit = $13.15 - 1.42 = 11.73s$ (this does not overlap the upper boundary of class of FQSA which has 11s delay).

The upper delay boundary = $13.15 + 1.42 = 14.57s \approx 15s$ (this does not overlap the volume capacity ratio of 0.7 with a delay value of 16.74s). This is taken as class of FQS B.

The next division is at a volume capacity ratio of 0.7 and 0.8. The delay values are 16.74s and 21.51s.

The mean delay value = $= \frac{16.74+22.50}{2} = 19.12s$

The standard deviation = 3.37s (estimated with Microsoft Excel)

The lower delay limit = $19.12 - 3.37 = 15.76s$

The upper delay limit = $19.12 + 3.37 = 22.50s$

The lower delay limit does not overlap the upper delay limit of class FQS B with a delay value of 14s and does not overlap the delay value of 30.08s for the volume capacity ratio of 0.9. This forms a class of FQS C.

The next division is a volume capacity ratio of 0.9 and 1. This is not taken as a class because of the wide range in the delay values and there is a need for an alert that the roundabout is operating close to the capacity. This is set at a volume capacity ratio of 0.9. This threshold class is the class of FQS D.

When the roundabout is operating at capacity, the volume capacity ratio = 1 and the delay = 49.29s. This is the class FQS E.

Class FQS F is when the roundabout is operating above the capacity, then the volume capacity ratio > 1 .

The corresponding reserve capacity, delay, queue length and volume capacity ratio for each class are put together to form the FQS assessment criteria for site 01.

The FQS assessment criteria shows FQS A; at this class, the reserve capacity is 0.6, a delay of less than or equal to 11s, the volume capacity ratio is less than or the same as 0.4, there is only one vehicle in the queue, and this occurs when there is free entry flow rate of traffic into the roundabout.

The next class is FQS B where the reserve capacity is 0.4 to 0.6, a delay of 11s – 15s, the volume capacity ratio is 0.4 – 0.6 with a queue length increase to two. FQS C is the division of a reserve capacity of 0.1 to 0.2, with a delay of 15s to 22s, a volume capacity ratio of 0.6 to

0.8 and a queue length of two to five vehicles. FQS D is the threshold that serves as the warning that the roundabout is operating close to the capacity with a reserved capacity of 0.1 to 0.2, a delay of 22s to 31s, a volume capacity ratio of 0.8 to 0.9, and a queue length of 5 to 7 vehicles. FQS E is when the roundabout is operating at capacity with a reserve capacity of 0 to 0.1, a delay of 31s to 49s, a volume capacity ratio of 0.9 to 1.0, and a queue length of 7 to 13 vehicles. FQS F occurs when the roundabout is operating above the capacity, with a reserve capacity of less than 0 (no capacity is reserved), a delay greater than 49s, a volume capacity ratio greater than 1.0, and a queue length greater than 13 vehicles. The FQS criterial table for site 01 is presented in table 5.6.

Table 5.6: FQS assessment criterial for site 01

FQS	d (s)	Q_R	x	(L) (veh)
A	$d \leq 11$	$Q_R \geq 0.6$	$x \leq 0.4$	1
B	$11 < d \leq 15$	$0.4 \leq Q_R < 0.6$	$0.4 < x \leq 0.6$	2
C	$15 < d \leq 22$	$0.2 \leq Q_R < 0.4$	$0.6 < x \leq 0.8$	$2 < L \leq 5$
D	$22 < d \leq 31$	$0.1 \leq Q_R < 0.2$	$0.8 < x \leq 0.9$	$5 < L \leq 7$
E	$31 < d \leq 49$	$0.1 \leq Q_R < 0$	$0.9 < x \leq 1$	$7 < L \leq 13$
F	$d > 49$	$Q_R < 0$	$x > 1$	$L > 13$

Note: FQS is Functional Quality of Service, Q_R is reserved capacity, d is delay, x is volume capacity ratio, L is queue length

5.2.2 Prevailing operational performance at site 01 (Armstrong Roundabout)

The roundabout user is more concerned with the time it takes to traverse a roundabout and perhaps the queue length, these parameters are the performance measure by users. The roundabout providers are concern with the utilisation of the roundabout in which the reserved capacity and the volume capacity ratio are the performance measures. The estimation of these parameters under dry, light, moderate and heavy rain is carried out in this section to know the effect of rainfall on these parameters. The off-peak data presented in chapter 4 for site 01 is used for the analysis to eliminate the effect of the peak period which might be difficult to separate from the rainfall effect. The stepwise procedure is used for simplicity and clarity.

Site 01 data is used in describing the stepwise procedure for estimation of the operational performance which includes the reserved capacity, volume capacity ratio, delay, and queue length under dry and rainy conditions. The steps are:

Step 1: Convert the vehicle volume to passenger car equivalent (PCE) as described in subsection 5.2.1. This is achieved by using the PCE factors of SANRAL which are: passenger cars (PC) = 1.00, medium vehicles (MV) = 2.80 and heavy vehicles (HV) = 2.80 for the roundabout. It was observed that the percentage difference in vehicle type might give a vehicular interaction within the traffic flow rate. The speed and headway measured by the ATC for passenger cars, medium and heavy vehicles under rainy conditions suggest that passenger cars, medium and heavy vehicles have different performance patterns.

As an example, for the computation of traffic flow rate:

The number of collected vehicles in five minutes are:

Passenger cars = 22 veh.

Medium vehicles = 4 veh.

Heavy vehicles = 3 veh.

Convert the heterogeneous traffic volume to homogeneous traffic flow rate by application of the SANRAL PCE value of 2.8 for medium and heavy vehicles.

Passenger cars = 22 pce/5min

Medium vehicles = $4 \times 2.8 = 11.2$ pce/5min

Heavy vehicles = $3 \times 2.8 = 8.4$ pce/5min

Total traffic flow rate in 5mins = $22 + 11.2 + 8.4 = 41.2$ pce/5min

Traffic flow rate per hour = $41.6 \times 12 = 499.2 \approx 499$ pce/h

The computed off-peak entry and circulating traffic flow rate under dry, light, moderate and heavy rain weather conditions for site 01 are presented in tables 5.7 to 5.10.

Table 5.7a: Computed off-peak entry flow rate during dry daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4 Col. 2*2.8	Col. 5 Col. 3*2.8	Col. 6 Σ col. 1,4,5	Col. 7 Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
22	4	3	11.2	8.4	41.6	499
73	3	2	8.4	5.6	87.0	1044
81	1	0	2.8	0.0	83.8	1006
82	1	0	2.8	0.0	84.8	1018
85	4	3	11.2	8.4	104.6	1255
67	4	1	11.2	2.8	81.0	972
67	3	2	8.4	5.6	81.0	972
66	2	2	5.6	5.6	77.2	926
64	3	2	8.4	5.6	78.0	936
43	5	3	14	8.4	65.4	785
82	1	0	2.8	0.0	84.8	1018
65	4	1	11.2	2.8	79.0	948

Note: Col is Column, 2.8 is South Africa PCE value, PC is passenger car, MV is Medium vehicle, HV is Heavy vehicle.

Table 5.7b: Computed off-peak circulating flow rate during dry daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4 Col. 2*2.8	Col. 5 Col. 3*2.8	Col. 6 Σ col. 1,4,5	Col. 7 Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
102	5	2	14.0	5.6	121.6	1459
69	4	3	11.2	8.4	88.6	1063
92	1	0	2.8	0.0	94.8	1138
80	3	2	8.4	5.6	94.0	1128
56	4	3	11.2	8.4	75.6	907
83	4	1	11.2	2.8	97.0	1164
85	5	2	14.0	5.6	104.6	1255
87	2	3	5.6	8.4	101.0	1212
109	0	1	0.0	2.8	111.8	1342
100	2	2	5.6	5.6	111.2	1334
74	4	3	11.2	8.4	93.6	1123
88	3	1	8.4	2.8	99.2	1190

Table 5.8a: Computed off-peak entry flow rate during light rain daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7
Col. 1	Col. 2	Col. 3	Col. 2*2.8	Col. 3*2.8	∑col. 1,4,5	Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
81	1	0	2.8	0.0	83.8	1006
62	3	2	8.4	5.6	76.0	912
83	5	3	14.0	8.4	105.4	1265
74	4	3	11.2	8.4	93.6	1123
88	4	4	11.2	11.2	110.4	1325
67	4	1	11.2	2.8	81.0	972
82	1	0	2.8	0.0	84.8	1018
62	3	2	8.4	5.6	76.0	912
82	1	0	2.8	0.0	84.8	1018
74	5	3	14.0	8.4	96.4	1157
90	0	1	0.0	2.8	92.8	1114
67	3	4	8.4	11.2	86.6	1039

Note: Col is Column, 2.8 is South Africa PCE value, PC is passenger car, MV is Medium vehicle, HV is Heavy vehicle,

Table 5.8b: Computed off-peak circulating flow rate during light rain daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7
Col. 1	Col. 2	Col. 3	Col. 2*2.8	Col. 3*2.8	∑col. 1,4,5	Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
67	4	3	11.2	8.4	86.6	1039
70	6	2	16.8	5.6	92.4	1109
48	3	2	8.4	5.6	62.0	744
67	3	1	8.4	2.8	78.2	938
53	0	1	0.0	2.8	55.8	670
65	4	3	11.2	8.4	84.6	1015
90	1	0	2.8	0.0	92.8	1114
80	2	2	5.6	5.6	91.2	1094
72	3	1	8.4	2.8	83.2	998
58	2	3	5.6	8.4	72.0	864
68	2	3	5.6	8.4	82.0	984
69	1	4	2.8	11.2	83.0	996

Note: Col is Column, 2.8 is South Africa PCE value, PC is passenger car, MV is Medium vehicle, HV is Heavy vehicle.

Table 5.9a: Computed off-peak entry flow rate during moderate rain daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7
Col. 1	Col. 2	Col. 3	Col. 2*2.8	Col. 3*2.8	∑col. 1,4,5	Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
63	3	2	8.4	5.6	77.0	924
62	4	1	11.2	2.8	76.0	912
63	3	2	8.4	5.6	77.0	924
71	3	2	8.4	5.6	85.0	1020
85	1	0	2.8	0.0	87.8	1054
80	4	1	11.2	2.8	94.0	1128
94	0	1	0.0	2.8	96.8	1162
94	0	1	0.0	2.8	96.8	1162
67	3	2	8.4	5.6	81.0	972
69	1	3	2.8	8.4	80.2	962
70	2	3	5.6	8.4	84.0	1008
72	2	3	5.6	8.4	86.0	1032

Note: Col is Column, 2.8 is South Africa PCE value, PC is passenger car, MV is Medium vehicle, HV is Heavy vehicle.

Table 5.9b: Computed off-peak circulating flow rate during moderate rain daylight at site 01

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7
Col. 1	Col. 2	Col. 3	Col. 2*2.8	Col. 3*2.8	∑col. 1,4,5	Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
69	1	0	2.8	0.0	71.8	862
53	3	4	8.4	11.2	72.6	871
59	3	2	8.4	5.6	73.0	876
62	0	1	0.0	2.8	64.8	778
61	1	0	2.8	0.0	63.8	766
42	4	1	11.2	2.8	56.0	672
38	5	0	14.0	0.0	52.0	624
34	5	2	14.0	5.6	53.6	643
68	1	0	2.8	0.0	70.8	850
48	4	1	11.2	2.8	62.0	744
57	2	2	5.6	5.6	68.2	818
57	1	0	2.8	0.0	59.8	718

Note: Col is Column, 2.8 is South Africa PCE value, PC is passenger car, MV is Medium vehicle, HV is Heavy vehicle.

Table 5.10a: Computed off-peak entry flow rate during heavy rain daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7
Col. 1	Col. 2	Col. 3	Col. 2*2.8	Col. 3*2.8	∑col. 1,4,5	Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
71	4	2	11.2	5.6	87.8	1054
50	3	2	8.4	5.6	64.0	768
41	2	2	5.6	5.6	52.2	626
63	1	0	2.8	0.0	65.8	790
45	3	2	8.4	5.6	59.0	708
52	4	1	11.2	2.8	66.0	792
69	5	2	14.0	5.6	88.6	1063
55	3	1	8.4	2.8	66.2	794
48	5	2	14.0	5.6	67.6	811
44	2	2	5.6	5.6	55.2	662
46	4	3	11.2	8.4	65.6	787
57	3	1	8.4	2.8	68.2	818

Note: Col is Column, 2.8 is South Africa PCE value, PC is passenger car, MV is Medium vehicle, HV is Heavy vehicle.

Table 5.10b: Computed off-peak circulating flow rate during heavy rain daylight at site 01.

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7
Col. 1	Col. 2	Col. 3	Col. 2*2.8	Col. 3*2.8	∑col. 1,4,5	Col. 6*12
PC	MV	HV	MV*2.8	HV*2.8	Flow rate/5min	Flow rate/h
62	0	1	0.0	2.8	64.8	778
80	2	2	5.6	5.6	91.2	1094
107	1	0	2.8	0.0	109.8	1318
83	3	1	8.4	2.8	94.2	1130
85	1	3	2.8	8.4	96.2	1154
84	1	0	2.8	0.0	86.8	1042
62	1	0	2.8	0.0	64.8	778
61	5	3	14	8.4	83.4	1001
81	3	1	8.4	2.8	92.2	1106
83	3	2	8.4	5.6	97.0	1164
72	3	1	8.4	2.8	83.2	998
75	1	0	2.8	0.0	77.8	934

Note: Col is Column, 2.8 is South Africa PCE value, PC is passenger car MV is Medium vehicle, HV is Heavy vehicle.

Step 2: The entry capacity (Q_e) is determined as explained in chapter 3 by combining the entry and circulating flow rate under rainy and dry weather conditions with the introduction of a dummy variable (ϵ) to distinguish the capacity under dry and rainy conditions. $\epsilon = 1$ under rainy conditions and 0 otherwise. Multiple linear regression is used to develop a model for the dry and rainy conditions. The combined off-peak entry and circulating traffic flow rate under light rain and dry weather are presented in table 5.11, moderate rain and dry weather in table 5.12 and heavy rain and dry weather in table 5.13.

Table 5.11: Entry and circulating flow rate during light rain and dry conditions.

Weather condition	q_e (pce/h)	q_c (pce/h)	ϵ
Light rain	1006	1039	1
	912	1109	1
	1265	744	1
	1123	938	1
	1325	670	1
	972	1015	1
	982	1113	1
	912	1094	1
	1017	998	1
	1157	864	1
	1114	984	1
Dry	1039	996	1
	499	1459	0
	1044	1063	0
	1006	1137	0
	1018	1128	0
	1255	905	0
	972	1164	0
	972	1255	0
	926	1212	0
	936	1341	0
	794	1334	0
1017	1123	0	
948	1190	0	

Note: q_e is entry flow rate, q_c is circulating flow rate, ϵ is dummy variable.

Table 5.12: Entry and circulating flow rate during the moderate rain and dry conditions.

Weather condition	q_e (pce/h)	q_c (pce/h)	ϵ
Moderate rain	924	861	1
	912	871	1
	924	876	1
	1020	777	1
	1054	765	1
	1128	672	1
	1161	624	1
	1162	643	1
	972	850	1
	962	744	1
	1017	818	1
	1032	717	1
	Dry	499	1459
1044		1063	0
1006		1137	0
1018		1128	0
1255		905	0
972		1164	0
972		1255	0
926		1212	0
936		1341	0
794		1334	0
1017		1123	0
948		1190	0

Note: q_e is entry flow rate, q_c is circulating flow rate, ϵ is dummy variable.

Table 5.13: Entry and circulating flow rate during heavy rain and dry conditions.

Weather condition	q_e (pce/h)	q_c (pce/h)	ϵ	
Heavy rain	1052	777	1	
	768	1094	1	
	626	1317	1	
	789	1130	1	
	709	1154	1	
	792	1041	1	
	1063	778	1	
	796	1001	1	
	811	1106	1	
	663	1164	1	
	787	999	1	
	818	934	1	
	Dry	499	1459	0
		1044	1063	0
1006		1137	0	
1018		1128	0	
1255		905	0	
972		1164	0	
972		1255	0	
926		1212	0	
936		1341	0	
794		1334	0	
1017		1123	0	
948	1190	0		

Note: q_e is entry flow rate, q_c is circulating flow rate, ϵ is dummy variable.

Using multiple linear regression for the analysis, the model equations during light, moderate and heavy rain conditions in combination with dry weather traffic data are shown in equations 5.6 to 5.8.

$$q_e = 2157 - 1.014q_c - 112.3\epsilon_L, \quad R^2 = 0.88 \quad [5.6]$$

$$q_e = 2215 - 1.06q_c - 337.5\epsilon_M, \quad R^2 = 0.84 \quad [5.7]$$

$$q_e = 2064 - 0.94q_c - 284.4\epsilon_H, \quad R^2 = 0.86 \quad [5.8]$$

Where: L, M, and H stand for light, moderate and heavy rain in the model equations and they are the same in subsequent sections.

The output of the analysis of variance (ANOVA) for the light, medium, and heavy rain traffic data in combination with dry weather traffic data and the dummy variable shows that all the model equations have expected signs of a negative linear regression which shows that entry flow rate reduces with an increase in circulating flow rate. This suggests that the entry vehicle yields to the circulating vehicles. The coefficient of the determinant (R^2) is more than 0.5 for all the model equations, this suggests that the relationship between the variables is strong. The P-value is less than 0.05 in all the results which shows that the variables are significant. The F-stat at a 95% level of confidence is more than the F-critical (4.84) for all the model equations, this suggests that the model equations did not occur by chance. The t-test at a 95% level of confidence is more than 2.2 for all the model equations, which suggests that the variables are significant, and the model equation could be used for prediction. The ANOVA summary report outputs are shown in tables 5.14 to 5.16.

Microsoft Excel was used for the multiple regression. q_c is the circulating flow rate, q_e is the entry flow rate while L, M, and H denote light, moderate and heavy rain, and D is used for the dummy variable in the ANOVA summary output. Note that D is used in place of ϵ because regression analysis with Microsoft Excel does not accept symbols.

Table 5.14: ANOVA report of off-peak circulating and entry flow rate during dry and light rain at site 01.

SUMMARY OUTPUT								
<i>Regression Statistics</i>								
Multiple R	0.9371264							
R Square	0.8782059							
Adjusted R Square	0.8666065							
Standard Error	59.97013							
Observations	24							
ANOVA								
	<i>df</i>	<i>SS</i>	<i>MS</i>	<i>F</i>	<i>Significance F</i>			
Regression	2	544577.2111	272289	75.7111	2.50654E-10			
Residual	21	75524.74726	3596.42					
Total	23	620101.9583						
	<i>Coefficients</i>	<i>Standard Error</i>	<i>t Stat</i>	<i>P-value</i>	<i>Lower 95%</i>	<i>Upper 95%</i>	<i>Lower 95.0%</i>	<i>Upper 95.0%</i>
Intercept	2157.9534	108.4652926	19.8953	4.2E-15	1932.387483	2383.519	1932.387483	2383.519331
qc (pce/h)	-1.0137964	0.089783936	-11.2915	2.2E-10	-1.200512359	-0.827081	-1.200512359	-0.82708053
D	-112.3249	31.96607864	-3.51388	0.00206	-178.802002	-45.8478	-178.802002	-45.8478026

Table 5.15: ANOVA report of off-peak circulating and entry flow rate during dry and moderate rain at site 01.

SUMMARY OUTPUT								
<i>Regression Statistics</i>								
Multiple R	0.9179432							
R Square	0.8426197							
Adjusted R Square	0.8276311							
Standard Error	59.009357							
Observations	24							
ANOVA								
	<i>df</i>	<i>SS</i>	<i>MS</i>	<i>F</i>	<i>Significance F</i>			
Regression	2	391509.4357	195755	56.2174	3.6981E-09			
Residual	21	73124.18929	3482.1					
Total	23	464633.625						
	<i>Coefficients</i>	<i>Standard Error</i>	<i>t Stat</i>	<i>P-value</i>	<i>Lower 95%</i>	<i>Upper 95%</i>	<i>Lower 95.0%</i>	<i>Upper 95.0%</i>
Intercept	2214.9792	125.8182694	17.6046	4.7E-14	1953.32575	2476.6326	1953.32575	2476.63258
qc (pce/h)	-1.0616134	0.104529201	-10.1561	1.5E-09	-1.27899381	-0.8442331	-1.2789938	-0.8442331
D	-377.14977	50.48276392	-7.47086	2.4E-07	-482.134426	-272.16512	-482.13443	-272.16512

Table 5.16: ANOVA report of off-peak circulating and entry flow rate during dry and heavy rain at site 01.

SUMMARY OUTPUT								
<i>Regression Statistics</i>								
Multiple R	0.924870948							
R Square	0.855386271							
Adjusted R Square	0.841613535							
Standard Error	67.27470601							
Observations	24							
<i>ANOVA</i>								
	<i>df</i>	<i>SS</i>	<i>MS</i>	<i>F</i>	<i>Significance F</i>			
Regression	2	562180.3509	281090	62.10721	1.5212E-09			
Residual	21	95043.60746	4525.89					
Total	23	657223.9583						
	<i>Coefficients</i>	<i>Standard Error</i>	<i>t Stat</i>	<i>P-value</i>	<i>Lower 95%</i>	<i>Upper 95%</i>	<i>Lower 95.0%</i>	<i>Upper 95.0%</i>
Intercept	2064.870069	114.8453482	17.9796	3.12E-14	1826.03609	2303.704	1826.03609	2303.704045
qc (pce/h)	-0.93574459	0.094912794	-9.85899	2.48E-09	-1.1331266	-0.738363	-1.13312655	-0.73836263
D	-284.3593479	30.99392848	-9.17468	8.56E-09	-348.81475	-219.9039	-348.814751	-219.903945

Step 4: Apply the correction factor ($k = 0.95$) estimated in subsection 5.2.1 for site 01 to the model equations 5.6 to 5.8. The modified model equations after the application of the correction factor are shown in equations 5.9 to 5.11.

$$q_e = 0.95(2157 - 1.014q_c - 112.3\epsilon_L) = 2050 - 0.963q_c - 107\epsilon_L \quad [5.9]$$

$$q_e = 0.95(2215 - 1.06q_c - 337.5\epsilon_M) = 2104 - 1.009q_c - 358\epsilon_M \quad [5.10]$$

$$q_e = 0.95(2064 - 0.94q_c - 284.4\epsilon_H) = 1962 - 0.889q_c - 270\epsilon_H \quad [5.11]$$

Step 5: In this step, the entry capacity (Q_e) and circulating capacity (Q_c) under the dry weather and raining conditions are estimated. The entry capacity occurs when there is no flow rate at the circulating roadway, though this situation is not a common traffic occurrence, setting $q_c = 0$, and $\epsilon = 1$ under rainy conditions and 0 otherwise. The circulating capacity also occurs when the entry flow rate (q_e) = 0. The entry and circulating capacity under dry and rainy weather conditions is estimated with the equations 5.9 to 5.11.

Capacity under light rain and dry weather conditions.

Estimating the Q_e and Q_c from equation 5.9 setting $\epsilon_L = 0$, $q_c = 0$ for the estimation of $Q_{e(\text{dry})}$ and $\epsilon_L = 0$ and $q_e = 0$ for estimation of $Q_{c(\text{dry})}$

$$Q_{e(\text{dry})} = 2050 - 0.963(0) - 107(0) = 2050 \text{ pce/h}$$

$$\text{Setting } Q_{e(\text{dry})} = 0 \text{ and } \epsilon_L = 0, \text{ the } Q_{c(\text{dry})} = \frac{2050}{0.963} = 2129 \text{ pce/h.}$$

The capacity under light rain is estimated using equation 5.9 by substituting $\epsilon_L = 1$, $q_c = 0$,

$$Q_{eL} = 2050 - 0.963(0) - 107(1) = 1943 \text{ pce/h}$$

$$\text{Setting } Q_{eL} = 0 \text{ and } \epsilon_L = 1, \text{ the } Q_{cL} = \frac{1943}{0.963} = 2018 \text{ pce/h.}$$

Capacity under moderate rain and dry weather conditions.

Equation 5.10 is used to estimate the capacity under the dry weather and the moderate rain.

setting $\epsilon_M = 0$, $q_c = 0$ for estimation of $Q_{e(\text{dry})}$, and $\epsilon_M = 0$ and $q_e = 0$ for estimation of $Q_{c(\text{dry})}$

$$Q_{e(\text{dry})} = 2104 - 1.009(0) - 358(0) = 2104 \text{ pce/h}$$

$$\text{Setting } Q_{e(\text{dry})} = 0 \text{ and } \epsilon_M = 0, \text{ then } Q_{c(\text{dry})} = \frac{2104}{1.009} = 2085 \text{ pce/h.}$$

The capacity under moderate rain is estimated using equation 5.10 by substituting $\epsilon_M = 1$, $q_c = 0$

$$Q_{eM} = 2104 - 1.009(0) - 358(1) = 1746 \text{ pce/h}$$

$$\text{Setting } Q_{eM} = 0 \text{ and } \epsilon_M = 1, \text{ then } Q_{cL} = \frac{1746}{1.009} = 1730 \text{ pce/h.}$$

Capacity under heavy rain and dry weather conditions.

Equation 5.11 is used to estimate the capacity under the dry weather and heavy rain.

Setting $\epsilon_H = 0$, $q_c = 0$ for estimation of $Q_{e(\text{dry})}$ and $\epsilon_H = 0$ and $q_e = 0$ for estimation of Q_{cD}

$$Q_{e(\text{dry})} = 1962 - 0.889(0) - 270(0) = 1962 \text{ pce/h}$$

$$\text{Setting } Q_{e(\text{dry})} = 0 \text{ and } \epsilon_H = 0, \text{ then, } Q_{c(\text{dry})} = \frac{1962}{0.889} = 2207 \text{ pce/h.}$$

The capacity under heavy rain is estimated using equation 5.11 by substituting $\epsilon_H = 1$, $q_c = 0$

$$Q_{eH} = 1962 - 0.889(0) - 270(1) = 1692 \text{ pce/h}$$

Setting $Q_{eH} = 0$ and $\epsilon_H = 1$, then. $Q_{cH} = \frac{1692}{0.889} = 1903 \text{ pce/h}$.

The results of the entry and circulating capacity under dry and rainy conditions show that the entry capacity reduces from 2050 pce/h to 1943 pce/h with an entry capacity shift of 107 pce/h or 5.22% under light rain, from 2104 pce/h to 1746 pce/h with a capacity drop of 358 pce/h or 16.17% under moderate rain, and from 1962 pce/h to 1691 pce/h with a capacity drop of 268 pce/h or 13.66%. The lowest entry capacity was under the heavy rainfall, but the highest entry capacity shift was 16.17% which occurs under moderate rainy conditions. The circulating capacity reduces from 2119 pce/h to 2018 pce/h with a capacity shrinkage of 111 pce/h or 5.21% under light rain, from 2085 pce/h to 1730 pce/h with a shrinkage of 355 pce/h or 17.03% under moderate rain, and from 2207 pce/h to 1903 pce/h with a capacity shrinkage of 304 pce/h or 13.77% under heavy rain. The lowest circulating capacity occurs under moderate rain and the influence of rainfall on capacity has the highest reduction of 17.03% under moderate rainfall.

Capacity depends on prevailing conditions and as the rainfall intensity varies the prevailing condition, which is the rainfall, changes and this is the reason for changes in both the entry and circulating capacity under the dry and rainy conditions and is prove that capacity is not static but dynamic. The entry and circulating capacity reduce irrespective of the rain intensity at site 01. The reduction in the entry capacity is because of the reduction in entry flow rate due to the effect of rainfall on visibility which makes entry drivers increase headway from the leading vehicles. This reduces the number of vehicles that accept the same gap, and even the caution taken in judging the safe gap within the circulating gap may reduce the number of vehicles that enter the roundabout, hence a reduction in entry capacity.

The circulating capacity reduces under rain because of the caution the circulating vehicles take due to the rain's effect on visibility and reduction in friction between the vehicle tyres and the road pavement. This makes the circulating vehicles reduce speed and maintain a bigger gap from the leading vehicles, hence resulting into a reduction in circulating capacity under rainy conditions irrespective of the rain intensity. The entry and circulating scenario show that neither the entry nor circulating vehicles have undue advantage over the other under rainfall,

irrespective of the rain intensity at site 01. The summary of the entry and circulating capacity is presented in table 5.17.

Table 5.17: Summary of entry and circulating capacity under dry and rainy conditions at site 01.

	Q_e (pce/h)		ΔQ_e (pce/h)	Q_c (pce/h)		ΔQ_c (pce/h)
	Rainfall	Dry		Rainfall	Dry	
Light	1943	2050	107	2018	2129	111
Moderate	1746	2104	358	1730	2085	355
Heavy	1691	1962	268	1903	2207	304

Note: Q_e is entry capacity, Q_c is circulating capacity, Δ is the difference.

In order to have a clear picture of the extent of rain effect on both the entry and circulating capacity, and to determine the direct model equation for each weather scenario, the entry capacity is plot against the circulating capacity for dry and rainy conditions. The plots show that rainfall causes a negative differential shift irrespective of the rain intensity. The plots are presented in figures 5.3 to 5.5.

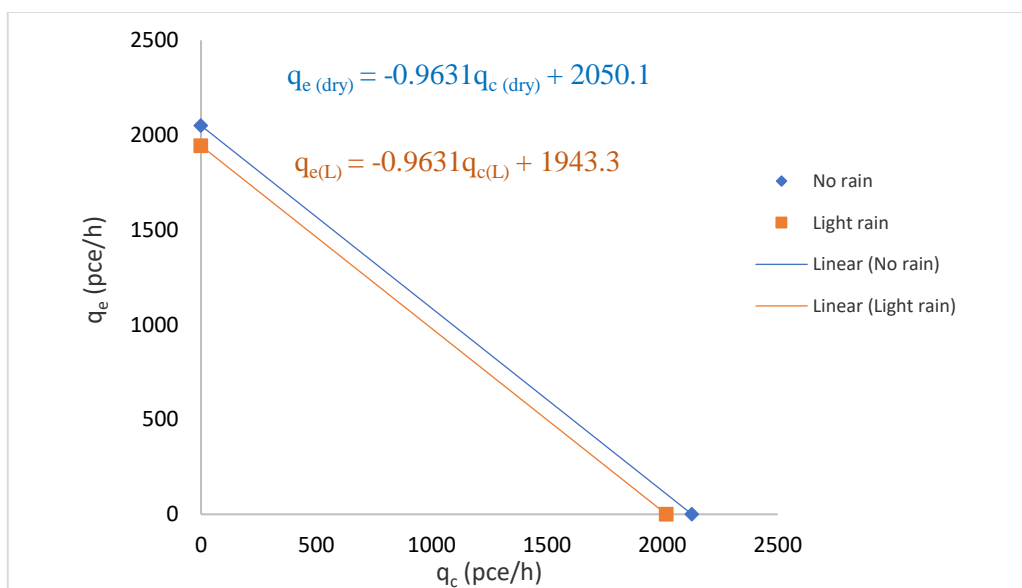


Figure 5.3: Entry - circulating capacity plot for dry and light rain conditions for site 01.

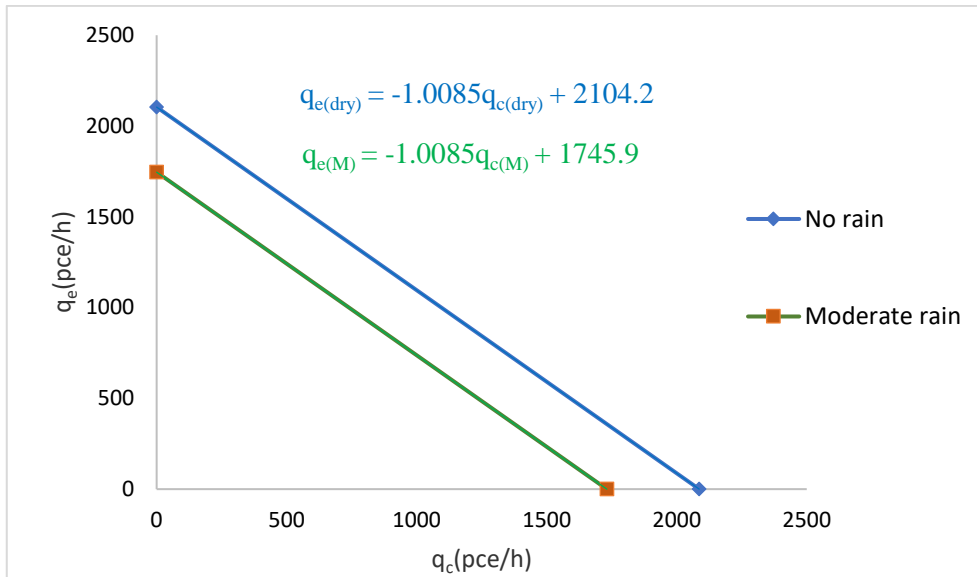


Figure 5.4: Entry - circulating capacity plot for dry and moderate rain conditions for site 01.

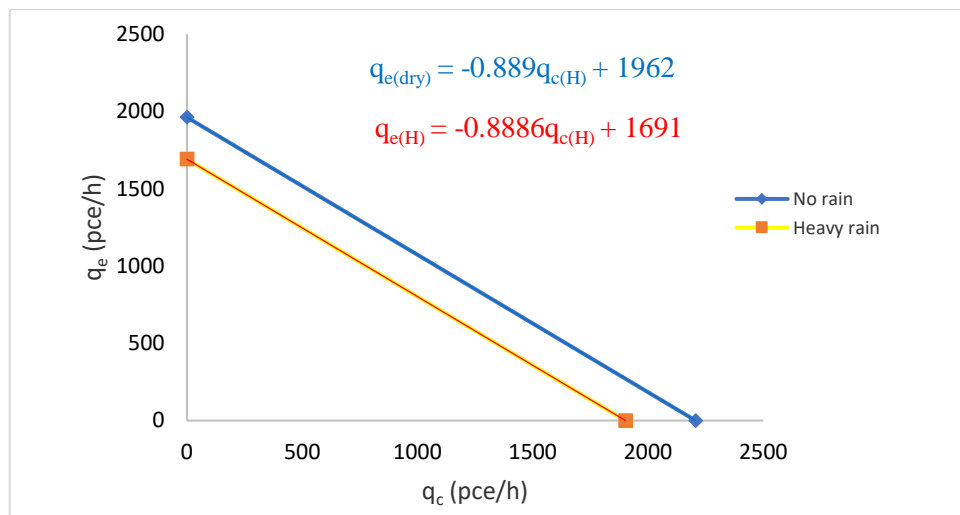


Figure 5.5: Entry - circulating capacity plot for dry and heavy rain conditions for site 01.

Step 6: Estimate reserved capacity, the reserved capacity (Q_R) is a parameter that represents the roundabout providers' perspective in the operational performance of roundabout. The reserved capacity (Q_R) will be estimated with equation 5.12.

$$Q_R = \frac{Q_e - q_e}{Q_e} \quad [5.12]$$

The entry capacity (Q_e) and maximum entry flow rate (q_e) under dry and rainy conditions for site 01 are:

$$Q_{e(\text{dry})} = 2050 \text{ pce/h (from table 5.17)}$$

$$q_{e(\text{dry})} = 1255 \text{ pce/h (from table 5.11)}$$

Substituting for Q_e and q_e in equation 5.12,

$$Q_{R(\text{dry})} = \frac{2050-1255}{2050} = 0.4$$

Under light rain:

$$Q_{eL} = 1943 \text{ pce/h (from table 5.17)}$$

$$q_{eL} = 1325 \text{ pce/h (from table 5.11)}$$

Substituting for Q_e and q_e in equation 5.12,

$$Q_{R(L)} = \frac{1943-1325}{1943} = 0.32$$

Under dry and moderate rain weather conditions:

$$Q_{e(\text{dry})} = 2104 \text{ pce/h (from table 5.17)}$$

$$q_{e(\text{dry})} = 1055 \text{ pce/h (from table 5.12)}$$

Substituting for Q_e and q_e in equation 5.12,

$$Q_{R(\text{dry})} = \frac{2104-1255}{2104} = 0.4$$

Under moderate rain:

$$Q_{eM} = 1746 \text{ pce/h (from table 5.17)}$$

$$q_{eM} = 1162 \text{ pce/h (from table 5.12)}$$

Substituting for Q_e and q_e in equation 5.12,

$$Q_{R(M)} = \frac{1746-1162}{1746} = 0.33$$

Under dry and heavy rain weather conditions:

$$Q_{e(\text{dry})} = 1962 \text{ pce/h (from table 5.17)}$$

$$q_{e(\text{dry})} = 1255 \text{ pce/h (from table 5.13)}$$

Substituting for Q_e and q_e in equation 5.12,

$$Q_{R(\text{dry})} = \frac{1962-1255}{1962} = 0.36$$

Under heavy rain:

$$Q_{eM} = 1692 \text{ pce/h (from table 5.17)}$$

$$q_{eM} = 1063 \text{ pce/h (from table 5.13)}$$

Substituting for Q_e and q_e in equation 5.12,

$$Q_{R(H)} = \frac{1692-1063}{1692} = 0.37$$

Step 7: The volume capacity ratio is estimated in this step. The maximum entry flow rate is used in the estimation of the volume capacity ratio to adequately represent the entry flow rate. The use of any other value might be inadequate when the maximum off-peak traffic is to be analysed. For example, if the average entry flow rate is used, it will not be applicable when the maximum entry flow rate occurs. Moreover, in many traffic analysis studies, the maximum traffic flow rate is always used. The volume capacity ratio is estimated using equation 5.13.

$$x = \frac{q_e}{Q_e} \quad [5.13]$$

The volume capacity ratio (x) in dry and light rain is estimated as:

The maximum entry flow rate under dry weather = 1255 pce/h (from table 5.11),

Entry capacity (Q_e) = 2050 pce/h (from table 5.17)

The maximum entry flow rate under light rain weather = 1325 pce/h (from table 5.11),

$Q_e = 1943$ pce/h (from table 5.17)

$$\text{Then, } x_{\text{dry}} = \frac{1255}{2050} = 0.6 \quad x_L = \frac{1325}{1943} = 0.68$$

The volume capacity ratio in dry and moderate rain is estimated as:

The maximum entry flow rate in dry weather = 1255 pce/h (from table 5.12),

$Q_e = 2104$ pce/h (from table 5.17)

The maximum entry flow rate in moderate weather = 1162 pce/h (from table 5.12),

$Q_e = 1746$ pce/h (from table 5.17)

Then, $x_{dry} = \frac{1255}{2104} = 0.59$, $x_M = \frac{1162}{1746} = 0.67$

The volume capacity ratio in dry and heavy rain is estimated as:

The maximum entry flow rate in dry weather = 1255 pce/h (from table 5.13),

$Q_e = 1962$ pce/h (from table 5.17)

The maximum entry flow rate in heavy weather = 1063 pce/h (from table 5.13),

$Q_e = 1692$ pce/h (from table 5.17)

Then, $x_{dry} = \frac{1255}{1962} = 0.63$, $x_H = \frac{1063}{1692} = 0.63$

The results of reserved capacity and volume capacity ratio estimated under dry and rainy weather conditions at site 01 show that the reserved capacity reduces from 0.40 to 0.32 under a light rain with a reduction of 0.08, from 0.4 to 0.33 under moderate rain with a reduction of 0.07, and from 0.36 to 0.37 under heavy rain with a reduction of 0.01. **Despite the usage of the capacity under light rain, the light rain and moderate rain have the highest reserve capacity reductions on the reserved capacity and heavy rain has little effect. This shows that under heavy rain this roundabout reserve more capacity than under light and moderate rain.** The volume capacity ratio increases from 0.60 to 0.68 during a light rain with an increase of 0.08, from 0.59 to 0.67 during moderate rain with an increase of 0.08 and shows no difference under heavy rain. The light and moderate rain has a higher effect than heavy rain on the volume capacity ratio. This shows that the capacity usage under light rain and moderate rain is higher than under heavy rain at this site. The volume capacity ratio increases, and the reserved capacity reduces at this site irrespective of the rain intensity. The results summary for the reserved capacity and volume capacity ratio at site 01 is presented in table 5.18.

Table 5.18: Summary of reserved capacity and volume capacity ratio at site 01.

	Q _R		Δ Q _R	x		Δx
	Rainfall	Dry		Rainfall	Rainfall	
Light	0.32	0.40	0.08	0.68	0.60	0.08
Moderate	0.33	0.40	0.07	0.67	0.59	0.08
Heavy	0.37	0.36	0.01	0.63	0.63	0.00

Note: Q_R is reserve capacity, x is volume capacity ratio, Δ is the differential.

Step 8: The control delay and queue length are estimated using equation 5.14 and 5.15 respectively.

$$d = \frac{3600}{k(F-f_c q_c - \epsilon)} + 900T \left[(x - 1) + \sqrt{(x - 1)^2 + \frac{\left(\frac{3600}{k(F-f_c q_c - \epsilon)}\right)x}{450T}} \right] + 5 \quad [5.14]$$

$$L = \frac{d * q_e}{3600} \quad [5.15]$$

Where:

d = control delay (s)

L = queue length (veh)

q_e = entry flow rate (pce/h)

k(F-f_cq_c-ε) = entry capacity (Q_e) pce/h/lane

x = volume capacity ratio

T is the time of observation = 0.25hr

The delay for dry and light rain are:

Delay (dry)

$Q_{e(\text{dry})} = 2050$ pce/h, on the assumption that the two lanes have the same traffic flow rate,

$Q_{e(\text{dry})}$ for single lane = $0.5 \times 2050 = 1025$ pce/h, $x = 0.60$,

Substituting for $Q_{e(\text{dry})}$ and $x_{(\text{dry})}$ in equation 5.14, $d_{\text{dry}} = 13.68\text{s}$

Queue length (dry)

Entry flow rate (q_e) at x of 0.60 = 1255 pce/h for a double lane,

$q_e = 0.5 \times 1255 = 627$ pce/h/ lane

Substitute for d and q_e in equation 5.15, $L = 2.3 \approx 2$ veh.

Delay (light rain)

$Q_{eL} = 1943$ pce/h,

Q_{eL} for a single lane = $0.5 \times 1943 = 972$ pce/h, $x = 0.68$,

Substituting for Q_{eL} and x_L in equation 5.14, $d_L = 16.25\text{s}$

Queue length (light rain)

$q_e = 1325$ pce/h for a double lane at a volume capacity ratio of 0.68 (from step 6),

On assumption that the two lanes have equal traffic flow rate, $q_e = 0.5 \times 1325 = 663$ pce/h per lane

Substitute for d and q_e in equation 5.15, $L = 3$ veh.

The delay for dry and moderate rain are:

Delay (dry)

$Q_{e(\text{dry})} = 2104$ pce/h,

$Q_{e(\text{dry})}$ for a single lane = $0.5 \times 2104 = 1052$ pce/h/lane, $x = 0.59$,

Substituting for $Q_{e(\text{dry})}$ and $x_{(\text{dry})}$ in equation 5.14, $d_{\text{dry}} = 13.22\text{s}$

Queue length (dry)

$q_e = 1255$ pce/h for a double lane at a volume capacity ratio of 0.59 (from step 6)

$$q_e = 0.50 \times 1255 = 627 \text{ pce/h/lane}$$

Substitute for d and q_e in equation 5.15, $L = 2.3 \approx 2$ veh.

Delay (moderate rain)

$$Q_{eM} = 1746 \text{ pce/h,}$$

$$Q_{eM} \text{ for single lane} = 0.5 \times 1746 = 873 \text{ pce/h/lane, } x = 0.68,$$

Substituting for Q_{eM} and x_M in equation 5.14, $d_M = 16.93\text{s}$

Queue length (moderate rain)

$q_e = 1162$ pce/h for a double lane at a volume capacity ratio of 0.67 (from step 6)

$$q_e = 0.5 \times 1162 = 581 \text{ pce/h/lane}$$

Substitute for d and q_e in equation 5.15, $L = 2.8 \approx 3$ veh.

The delay for dry and heavy rain is:

Delay (dry)

$$Q_{e(\text{dry})} = 1962 \text{ pce/h,}$$

$$Q_{e(\text{dry})} \text{ for a single lane} = 0.5 \times 1962 = 981 \text{ pce/h/lane, } x_{\text{dry}} = 0.63,$$

Substituting for $Q_{e(\text{dry})}$ and x_D in equation 5.14, $d_{\text{dry}} = 14.70\text{s}$

Queue length (dry)

$$q_e = 1255 \text{ pce/h for a double lane, } q_e = 0.5 \times 1255 = 522 \text{ pce/h/lane}$$

Substitute for d and q_e in equation 5.15, $L = 2.5 \approx 3$ veh.

Delay (heavy rain)

$$Q_{eH} = 1692 \text{ pce/h,}$$

Q_{eM} for a single lane = $0.5 \times 1692 = 846$ pce/h/lane, $x = 0.63$,

Substituting for Q_{eM} and x_M in equation 5.14, $d_M = 16.21$ s

Queue length (heavy rain)

$q_e = 1063$ pce/h for a double lane, $q_e = 0.5 \times 1063 = 532$ pce/h per lane

Substitute for d and q_e in equation 5.15, $L = 2.5 \approx 3$ veh.

The result of delay under dry and rainy conditions shows that delay increases from 13.68s to 16.25s with an increase of 2.57s with light rain, 13.22s to 16.93s with an increase of 3.71s with moderate rain and 14.70s to 16.17s with an increase of 1.51s with heavy rain. The delay under moderate rain is highest because the medium and heavy vehicles are highest under moderate rain and entry drivers take more caution in accepting the available gap within the circulating traffic. Irrespective of rain intensity, there is an increase in entry delay. This is because under rainfall, the ability to judge the safe gap between the circulating traffic becomes more difficult, the car following at entry keeps a bigger gap from the leading vehicles, hence increases the waiting time and the follow-up time. The delay increases under rainfall at this site irrespective of the rain intensity.

The result of queue length shows that under dry weather conditions, the queue length is 2 vehicles and it increases to 3 vehicles during light to moderate rainfall with an increase of 1 vehicle. However, the queue length remains unchanged under heavy rain. The queue length at this site increases under of rainfall. The summary of the delay and queue length at site 01 is presented in table 5.19.

Table 5.19: Summary of delay and queue length at site 01.

	d (s)		Δd (s)	L (veh)		d (s)
	Rainfall	Dry		Dry	Rainfall	
Light	16.25	13.68	2.36	2	3	1
Moderate	16.93	13.22	3.71	2	3	1
Heavy	16.21	14.70	1.51	3	3	0

Note: d is delay, L is queue length, Δ is difference.

5.2.3 Functional quality of service delivery at Site 01 (Armstrong Roundabout)

To assess the effect of rainfall intensity on the functional quality of service delivery at a roundabout, the estimated provider and user parameters of assessing the FQS in section 5.2.2, which are the reserved capacity, delay, volume capacity ratio, and queue length under dry and rainy weather of varying intensity are used in the assessment. Any of the parameters can be used for the assessment of the FQS while the value of the other parameters can be determined from the FQS criterial table by an interpolation method. For the purpose of this study, the delay will be used but the value of the other parameters can be determined using interpolation.

The criterial table 5.6 developed in section 5.2.1 for the assessment of the functional quality of service for site 01 will be used in the assessment of FQS at this site. The estimated delay under dry and light, moderate and heavy rain is used in the assessment of the FQS during dry and rainy conditions. The results show that rainfall, irrespective of intensity, influences the functional quality of service as the light, moderate and heavy rain shift the FQS from FQS B to FQS C. The FQS assessment at site 01 under dry and rainy weather conditions is presented in table 5.20.

Table 5.20: FQS assessment under dry and rainy conditions at site 01.

	Delay (s)	FQS
Dry	13.68	B
Light Rain	16.25	C
Moderate Rain	16.93	C
Heavy rain	16.21	C

Note that: FQS is Functional quality of service

5.3 Functional Quality of Service Delivery at Site 02 (Millennium Roundabout)

5.3.1 Site 02 Criterial table development for site 02 (Millennium Roundabout)

The peak period entry and circulating traffic data for site 02 presented in chapter 4 is used for the estimation of the parameters for the criterial table development for site 02. A linear

relationship developed for the peak entry and circulating flow rate is presented in figure 5.6 and the developed model is equation 5.16.

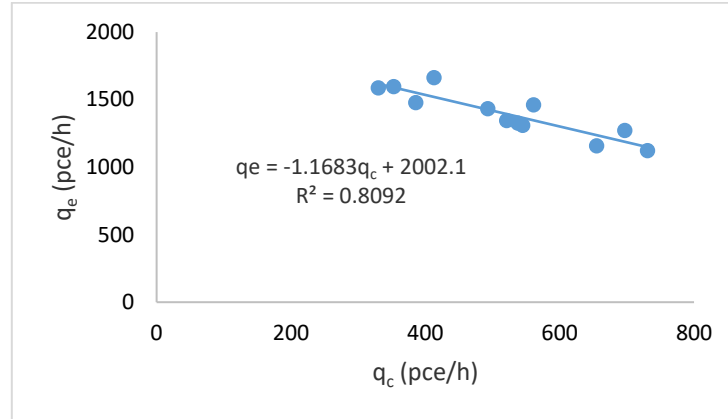


Figure 5.6: Peak entry – circulating flow rate relationship for site 02.

$$q_e = 2002.1 - 1.168 q_c \quad R^2 = 0.81 \quad [5.16]$$

The statistical testing shows that the R^2 is more than 0.5 which shows that the variables are significant, the t-test is more than 2.2, the F-test is 42.40 which is more than 4.84 which shows that the model equation did not occur by chance, the P-value is 0 which is less than 0.05, this shows that the model equation could be used for prediction. At this site, the entry angle is 45° , and the entry radius is 50m which are within Kimberly’s geometric limits for entry angle and radius. These are used for estimation of the correction factor (k) to be 0.98. The correction factor is applied to the model equation. The modified model is equation 5.17.

$$q_e = 1902 - 1.11 q_c \quad [5.17]$$

The capacity for the single lane is estimated as 951 pce/h/lane. The sensitivity test carried out by an estimation of delay and queue length at volume capacity ratio of 0 and 1 shows that when the volume capacity ratio = 0, delay is 8.79s and the queue length = 0 vehicle. The delay at a volume capacity ratio = 1 is 50.06s and the queue length is 13 vehicles. This is the delay and queue length when the roundabout is operating at capacity. The delay, reserved capacity, and the queue length are estimated for ten divisions of volume capacity ratio from 0 to 1. The summary result is shown in table 5.21.

Table 5.21: Summary of parameters for the development of a FQS criterial table at site 02.

x	Q_R	d (s)	L (veh)
0	1.0	8.79	0
0.1	0.9	9.21	0
0.2	0.8	9.73	1
0.2	0.8	9.73	1
0.3	0.7	10.40	1
0.4	0.6	11.29	1
0.5	0.5	12.51	2
0.6	0.4	14.30	2
0.7	0.3	17.11	3
0.8	0.2	21.99	5
0.9	0.1	31.44	7
1	0	50.06	13

Note: x is volume capacity ratio, Q_R is reserved capacity, d is delay, L is queue length.

The criterial table is developed as explained in section 5.2.1. The developed FQS criterial table for site 02 is in table 5.22. This table will be used for the functional quality of service assessment of site 02.

Table 5.22: The FQS criterial table for site 02.

FQS	d (s)	Q_R	x	(L) (veh)
A	$d \leq 11$	$Q_R \geq 0.6$	$x \leq 0.4$	1
B	$11 < d \leq 14$	$0.4 \leq Q_R < 0.6$	$0.4 < x \leq 0.6$	2
C	$14 < d \leq 22$	$0.2 \leq Q_R < 0.4$	$0.6 < x \leq 0.8$	$2 < L \leq 5$
D	$22 < d \leq 31$	$0.1 \leq Q_R < 0.2$	$0.8 < x \leq 0.9$	$5 < L \leq 7$
E	$31 < d \leq 50$	$0.1 \leq Q_R < 0$	$0.9 < x \leq 1$	$7 < L \leq 13$
F	$d > 50$	$Q_R < 0$	$x > 1$	$L > 13$

Note: FQS is Functional Quality of Service, d is delay, Q_R is reserved capacity, x is volume capacity ratio, L is queue length

The delay values in this table has slight difference from the FQS table at site 01. The reason is because the geometry, traffic and other environmental factors are not the same for the sites.

5.3.2 Prevailing operational performance at site 02 (Millennium Roundabout)

The off-peak entry and circulating flow rate under dry condition, and each class of rainy conditions at site 02 as presented in chapter 4 were combined with an introduction dummy variable to distinguish the capacity under rain from the dry conditions with the procedure in

section 5.2.2. Multiple regression was used for the analysis, the reserved capacity, volume capacity ratio, the control delay (d) and queue length (L) are estimated for dry and rainy conditions with off-peak data. The developed model equation for dry and each rainy condition are presented in equations 5.18 to 5.20.

$$q_e = 2019 - 1.116q_c - 3.84\epsilon_L, \quad R^2 = 0.84 \quad [5.18]$$

$$q_e = 1885 - 0.8969q_c - 190.88\epsilon_M, \quad R^2 = 0.87 \quad [5.19]$$

$$q_e = 1965.37 - 1.026q_c - 360.19\epsilon_H, \quad R^2 = 0.95 \quad [5.20]$$

The statistical testing of the model equations at a 95% level of confidence shows that the P-value is less than 0.05, the t-test is more than 2.2 and the F-test is more than 4.84 for all the model equations. This shows that the model equations are statistically fit and could be used for prediction.

The entry angle is 45° , and the entry radius is 50m. These are used for estimation of the correction factor (k) to be 0.98. The correction factor is applied to equations 5.18 to 5.20 and the modified model equations are shown in equations 5.21 to 5.23.

$$q_e = 1978 - 1.09q_c - 3.77 \epsilon_L \quad [5.21]$$

$$q_e = 1848 - 0.88q_c - 187 \epsilon_M \quad [5.22]$$

$$q_e = 1926 - 1.01q_c - 353 \epsilon_H \quad [5.23]$$

The entry and circulating capacity were estimated under dry and rainy conditions, the results show that the entry capacity reduces from 1978 pce/h to 1974 pce/h with a capacity shift of 4 pce/h or 0.20% under light rain, from 1848 pce/h to 1661 pce/h with a capacity shift of 187pce/h or 10.12% during moderate rain, and from 1926 pce/h to 1572 pce/h with a capacity shift of 400 pce/h or 20.77% during heavy rain. The entry capacity under light rain is almost the same as the capacity under dry conditions, the reason being that the light rain might be a rain shower of very low intensity since the light rain class is below the intensity of 2.5mm/h. Very low rain intensity might not have much effect on the driver's reaction. Heavy rain has the lowest capacity and the highest effect on the entry capacity shift at this site.

The circulating capacity reduces from 1810 pce/h to 1806 pce/h with a capacity shrinkage of 4 pce/h or 0.22% during light rain, from 2110 pce/h to 1896 pce/h with a capacity shift of 214 pce/h or 10.14% during moderate rain, and 1914 pce/h to 1563 pce/h with a capacity shift of 351 pce/h or 18.34% during heavy rain. Heavy rain has the lowest circulating capacity with the highest reduction effect on the circulating capacity. The light rain has very little effect on the circulating capacity, the reason is because of low rain intensity. The moderate and heavy rain affects visibility more than light rain and this makes the driver at the circulating road take more caution of the leading vehicles. Hence, the reduction in circulating capacity. However, the wide range in the entry and circulating capacity shift between the moderate and the heavy rain is due to the wide range in the intensity class of heavy rain (10 – 50 mm/h). The heavy rain intensity might be close to the upper limits of the heavy rain class which might make the rain class closer to the very high rain where the visibility is adversely affected. The entry and circulating capacity increases with an increase in rain intensity at site 02 irrespective of rain intensity. The summary of the entry and circulating capacity is presented in table 5.32.

Table 5.23: Summary of entry and circulating capacity under dry and rainy conditions at site 02.

	Q_e (pce/h)		ΔQ_e (pce/h)	Q_c (pce/h)		ΔQ_c (pce/h)
	Rainfall	Dry		Rainfall	Dry	
Light	1974	1978	4	1806	1810	4
Moderate	1661	1848	187	1896	2110	214
Heavy	1573	1926	353	1563	1914	351

Note: Q_e is entry capacity, Q_c is circulating capacity, Δ is the difference.

The plot of entry against circulating capacity under the dry and rainy conditions for each class of rain shows that there is an overlap on the plot of entry and circulating capacity during dry and light rain weather conditions. There is an entry and circulating capacity shift because of the moderate and heavy rain effect. The pattern for the shift under light, moderate and heavy rainfall is the same as they all have a negative capacity differential shift. The light rain effect on differential shift is small and insignificant because the rain could be a rain shower. The wide range of the effect of moderate rain and heavy rain is because of the wide range in the

class of heavy rain (rain intensity of 10 – 50mm). The heavy rain might be close to the upper limit of the heavy rain class and this makes drivers' visibility to be more affected. The three rain conditions show that both entry and circulating traffic are affected by rain and no one has an undue advantage over the other because there is an entry and circulating capacity loss irrespective of the rain intensity. The plot is presented in figures 5.7 to 5.9.

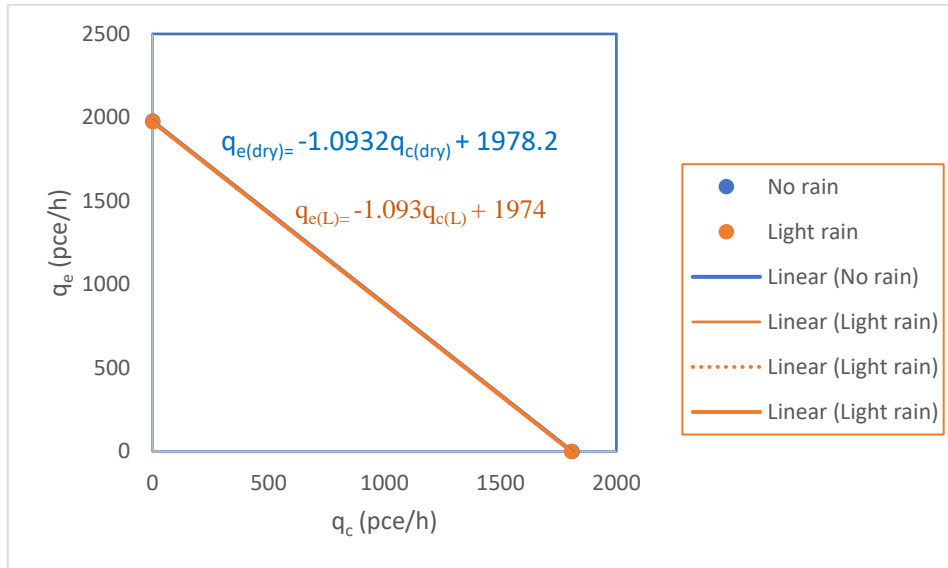


Figure 5.7: Entry - circulating capacity plot for dry and light rain conditions for site 02,

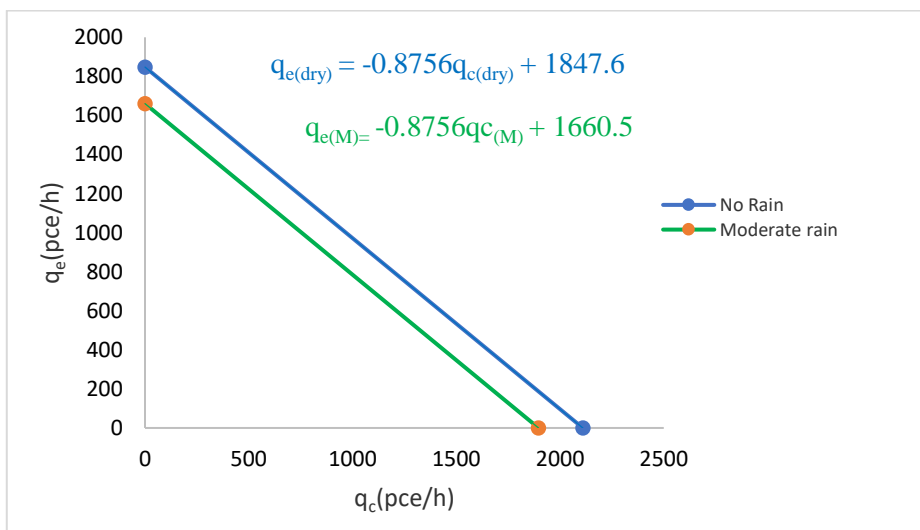


Figure 5.8: Entry - circulating capacity plot for dry and moderate rain conditions for site 02.

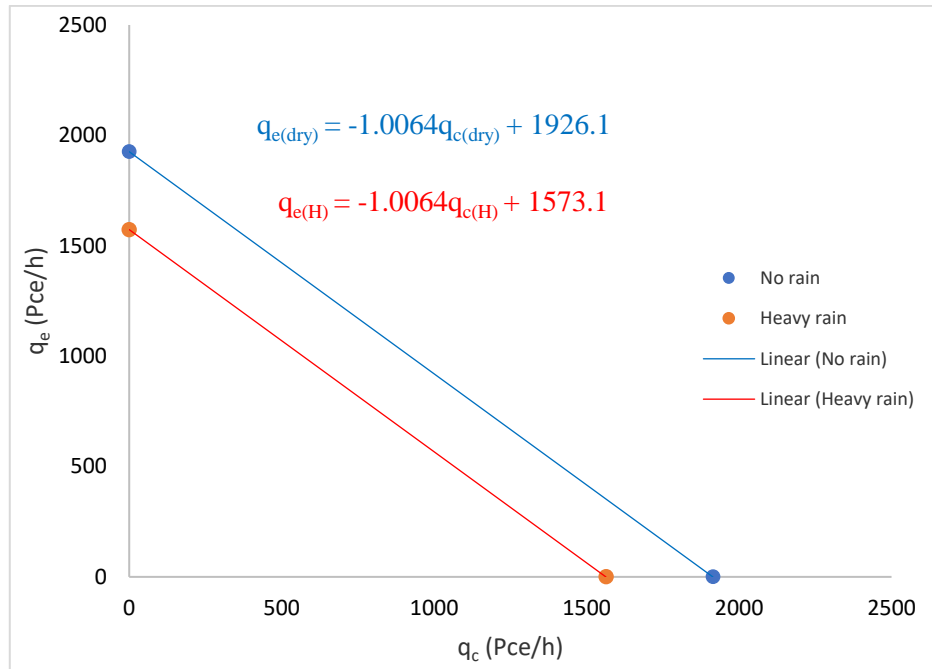


Figure 5.9: Entry - circulating capacity plot for dry and heavy rain conditions for site 02.

The entry capacity reserved (Q_R) and volume to capacity under dry weather and each class of rain are estimated following the procedures of subsection 5.2.2. The reserved capacity and volume capacity ratio results show that light rain has no effect on the reserved capacity and volume capacity at this site. This is expected as there is an almost similar value in the entry capacity under dry and light rain with values of 1978 pce/h and 1974 pce/h. The moderate and heavy rain reduces the reserved capacity from 0.29 to 0.2 and 0.27 to 0.2 respectively. This shows that as the capacity usage increases, the capacity reserved reduces under rainfall irrespective of the rain intensity at this site.

Moderate rain increases the volume capacity ratio from 0.74 to 0.8 with an increase of 0.06 and heavy rain from 0.73 to 0.80 with an increase of 0.07. Heavy rain has the highest increasing effect on the volume capacity ratio at this site. The reserved capacity reduces and the volume to capacity ratio increases with an increase in rain intensity at this site. The summary of the reserved capacity and volume to capacity ratio results under dry and rainy conditions is presented in table 5.24.

Table 5.24: Summary of the ratio of flow rate to capacity and reserve capacity at site 02.

	Q_R		ΔQ_R	x		Δx
	Rainfall	Dry		Rainfall	Dry	
Light	0.29	0.29	0.00	0.71	0.71	0.00
Moderate	0.20	0.26	0.06	0.80	0.74	0.06
Heavy	0.20	0.27	0.07	0.80	0.73	0.07

Note: Q_R is reserve capacity, x is volume capacity ratio and Δ is the difference.

The estimated volume capacity ratio under dry and rainy weather conditions are used in the estimation of delay and queue length. The results show that light rain increases the delay from 16.93s to 17.04s with an increase of 0.11s, and moderate rain has an increase effect from 19.10s to 24.22s with an increase of 5.12s, and heavy rain increases the delay from 17.99s to 25.18s with an increase of 7.19s. This shows that the delay at this site increases with an increase in rain intensity and heavy rain has the highest effect on the delay at this site. The reason for the increase in delay under rainfall is that the entry drivers take more caution of the leading vehicle and take caution in accepting the available gap within the circulating vehicles due to impaired visibility. The results also show that light rain does not influence the queue length at this site, while moderate and heavy rain increases the queue length from 4 vehicles to 5 vehicles. At this site, the delay and queue length increase with an increase in rain intensity. The summary of the delay and queue length results is presented in table 5.25.

Table 5.25: Summary of delay and queue length under dry and rainy conditions at site 02.

	d (s)		Δd (s)	L (veh)		ΔL (veh)
	Rainfall	Dry		Dry	Rainfall	
Light	17.04	16.93	0.11	4	4	0
Moderate	24.22	19.10	5.12	4	5	1
Heavy	25.18	17.99	7.19	4	5	1

Note: d is delay, L is queue length, Δ is the difference.

5.3.3 Functional quality of service delivery at site 02 (Millennium Roundabout)

The developed criterial table 5.40 for assessment of the functional quality of service developed in section 5.3.1 for site 02 will be used in the assessment of FQS under the dry and rainy weather of varying intensity to determine the effect of rain intensity on the service delivery at site 02. The users' and providers' parameters for this site have been estimated in section 5.3.2. The delay will be used in the assessment of the functional quality of service under dry and rainy weather conditions at site 02. Other parameters can be estimated from the criterial table using the interpolation method.

The assessment results show that light rain does not influence the roundabout service delivery at this site because the FQS under dry weather is FQS C and it remains unchanged during light rain, while moderate and heavy rain does influence the functional quality of service as it deteriorates from FQS C to D. The service delivery at site 02 deteriorates under rainfall irrespective of the rain intensity. The summary of the FQS assessment under light and rainy weather conditions is presented in table 5.26.

Table 5.26: FQS assessment under dry and rainy conditions at site 02??????.

	d (s)		FQS	
	Rainfall	Dry	Rainfall	Dry
Light	17.04	16.93	C	C
Moderate	24.22	19.10	D	C
Heavy	25.18	17.99	D	C

Note: d is delay, FQS is Functional quality of service.

5.4 Functional Quality of Service Delivery at Site 03 (Douglas Roundabout)

5.4.1 Criterial table development for site 03 (Douglas Roundabout)

The peak entry and circulating traffic data under dry and rainy conditions for site 03 as presented in chapter 4 is used for the estimation of the parameters for the development of a criterial table of assessment for site 03. A linear relationship is developed for the peak entry and circulating flow rate at this site using the procedure in subsection 5.2.1. The relationship between the entry and circulating flow rate is shown in figure 5.10 and the model is equation 5.24.

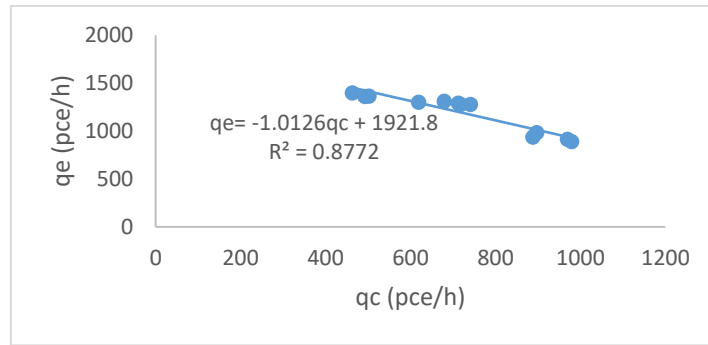


Figure 5.10: Peak entry – circulating flow rate relationship for site 03.

$$q_e = 1921.8 - 1.013 q_c \quad R^2 = 0.88 \quad [5.24]$$

The model equation is tested statistically at a 95% level of confidence, the R^2 is more than 0.5 which shows that the variables are significant, the t-test is greater than 2.2, the F-test is 71.44 which is more than 4.84 which shows that the model equation did not occur by chance, the P-value is 0 which is less than 0.05, this shows that the model equation could be used for prediction. The correction factor was estimated with the entry radius (50m) and the entry angle (45°) to be 0.98 and applied to equations 5.24. The modified model equation is shown in equation 5.25.

$$q_e = 1882 - 0.99 q_c \quad [5.25]$$

The capacity for the single lane is estimated as 941 pce/h/lane. The sensitivity test carried out by estimation of the delay and queue length at a volume capacity ratio of 0 and 1 shows that when the volume capacity ratio = 0, delay is 8.82s and the queue length = 0 vehicle. When the roundabout is operating at capacity, the volume capacity ratio = 1, the estimated delay and queue length are 50.30s and 13 vehicles respectively. These values form the upper and lower limits. The delay, reserved capacity, and the queue length corresponding to ten divisions of volume capacity ratio of 0 to 1 are estimated and the summary is presented in table 5.27.

Table 5.27: Summary of parameters for development of FQS criterial table at site 03.

x	Q_R	d (s)	L (veh)
0	1	8.82	0
0.1	0.9	9.25	0
0.2	0.8	9.78	1
0.3	0.8	10.45	1
0.4	0.7	11.35	1
0.5	0.6	12.58	2
0.6	0.5	14.39	2
0.7	0.4	17.22	3
0.8	0.3	22.14	5
0.9	0.2	31.65	7
1	0.1	50.30	13

Note: x is volume capacity ratio, Q_R is Reserve Capacity, d is Delay, L is Queue length.

The criterial table of FQS assessment is developed as explained in section 5.2.1. The developed criterial table of FQS assessment for site 03 is presented in table 5.28. The criterial table 5.28 will be used for the assessment of the functional quality of service for site 03.

Table 5.28: The criterial of FQS assessment for site 03.

FQS	d (s)	Q_R	x	(L) (veh)
A	$d \leq 11$	$Q_R \geq 0.6$	$x \leq 0.4$	1
B	$11 < d \leq 14$	$0.4 \leq Q_R < 0.6$	$0.4 < x \leq 0.6$	2
C	$14 < d \leq 22$	$0.2 \leq Q_R < 0.4$	$0.6 < x \leq 0.8$	$2 < L \leq 5$
D	$22 < d \leq 32$	$0.1 \leq Q_R < 0.2$	$0.8 < x \leq 0.9$	$5 < L \leq 7$
E	$32 < d \leq 50$	$0.1 \leq Q_R < 0$	$0.9 < x \leq 1$	$7 < L \leq 13$
F	$d > 50$	$Q_R < 0$	$x > 1$	$L > 13$

Note: FQS is Functional Quality of Service, d is delay, Q_R is reserve capacity, x is volume capacity ratio, L is queue length

5.4.2 Prevailing operational performance at site 03 (Douglas Roundabout)

The procedure in section 5.2.2 is followed to estimate the reserve capacity, volume capacity ratio, delay and queue length at this site. The dry and rainy off-peak data presented in chapter 4 for site 02 is used for determining the operational performance at this site. Combining the off-peak dry and rainy weather together with the introduction of a dummy variable following the procedure in section 5.2.2, and the multiple linear regression is adopted in the analysis. The models developed for dry and each rainy condition are presented in equations 5.26 to 5.28.

$$q_e = 1716 - 0.877q_c - 148.78\epsilon_L, \quad R^2 = 0.67 \quad [5.26]$$

$$q_e = 1902 - 1.186q_c - 61.11\epsilon_M, \quad R^2 = 0.74 \quad [5.27]$$

$$q_e = 1556 - 0.613q_c - 182.72\epsilon_H, \quad R^2 = 0.76 \quad [5.28]$$

The correction factor was estimated with the entry radius (50m) and the entry angle (45^0) to be 0.98 9 (using Kimberly's model) and applied to equations 5.26 to 5.28. The modified model equations are shown in equations 5.29 to 5.31.

$$q_e = 1682 - 0.86q_c - 149\epsilon_L \quad [5.29]$$

$$q_e = 1864 - 1.162q_c - 60\epsilon_M \quad [5.30]$$

$$q_e = 1525 - 0.601q_c - 179\epsilon_H \quad [5.31]$$

The entry and circulating capacity are estimated with the procedure in subsection 5.2.2. The results of the estimated entry and circulating capacity under dry and rainy conditions show that rainfall reduces the entry capacity, as light rain reduces the entry capacity from 1682 pce/h to 1536 pce/h with a reduction of 146 pce/h or 8.68%, moderate rain reduces the capacity from 1864 pce/h to 1804 pce/h with a reduction of 60 pce/h or 3.22%, and heavy rain reduces the entry capacity from 1525 pce/h to 1346 pce/h with a reduction of 179 pce/h or 11.74%. The lowest entry capacity of 1346 pce/h and the highest capacity increase of 11.74% occur under the heavy rainfall conditions. Moderate rain has the lowest effect, the reason for this is the rainfall distribution, because the moderate rain might just be fluctuating within the borderline of light and moderate rain.

The circulating capacity also reduces from 1956 pce/h to 1786 pce/h with a capacity drop of 170pce/h or 8.69% during light rain, 1604 pce/h to 1552 pce/h with a drop of 52 pce/h or 3.24% under moderate rain, and 2538 pce/h to 2240 pce/h with a capacity drop of 298 pce/h or 11.74% during heavy rain. The highest capacity drop occurs during heavy rain. The summary of the entry and circulating capacity is presented in table 5.29.

Table 5.29: Summary of entry and circulating capacity under dry and rainy condition at site 03.

	Q _e (pce/h)		ΔQ _e (pce/h)	Q _c (pce/h)		ΔQ _c (pce/h)
	Rainfall	Dry		Rainfall	Dry	
Light	1536	1682	146	1786	1956	170
Moderate	1804	1864	60	1552	1604	52
Heavy	1346	1525	179	2240	2358	298

Note: Q_e is entry capacity, Q_c is circulating capacity, Δ is the differential

Plotting entry and circulating capacity under dry and rainy conditions, shows that both entry and circulating capacity reduces under rainfall irrespective of the rain intensity at this site. Heavy rain has the highest effect on the capacity shift at both the entry and the circulating roadways. This capacity shift under dry and rainy conditions has the same trend as site 01 and 02, because there is a reduction in both the entry and circulating capacity irrespective of the rain intensity. Heavy rain has the highest effect on the capacity differential shift. However, light, moderate and heavy rain have the same trend of a negative shift effect on the capacity differential. This shows that both the entry and circulating capacity are affected by rainfall at site 03 irrespective of the intensity. The entry – circulating capacity graph plot under dry and rainy conditions are shown in figures 5.11 to 5.13.

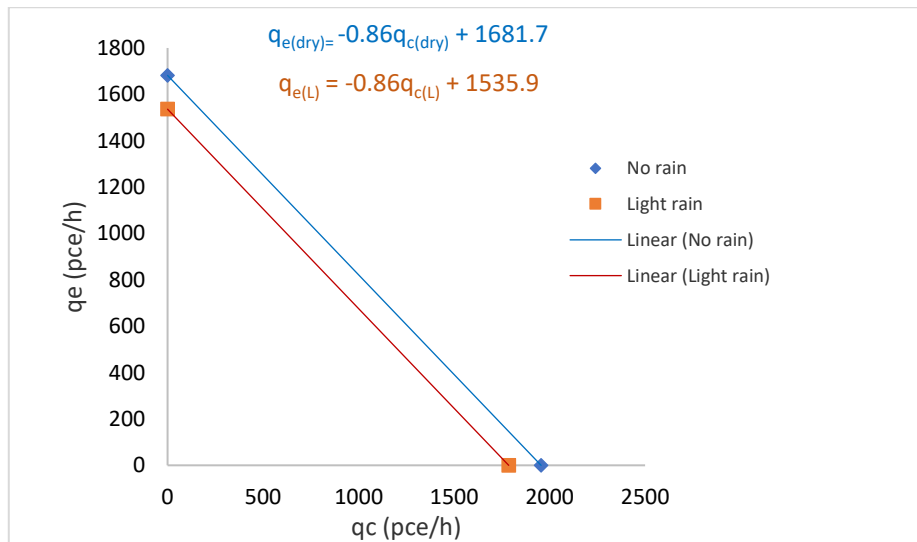


Figure 5.11: Entry - circulating capacity plot for dry and light rain conditions for site 03.

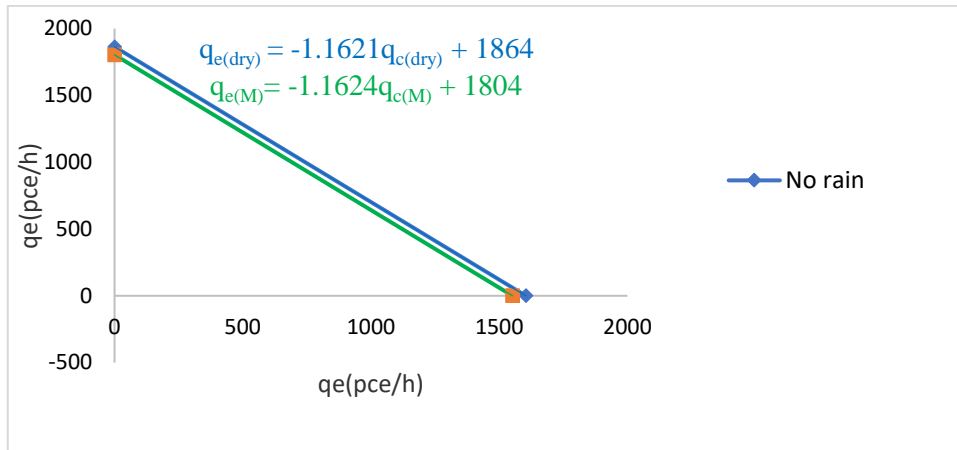


Figure 5.12: Entry - circulating capacity plot for dry and moderate rain conditions for site 03.

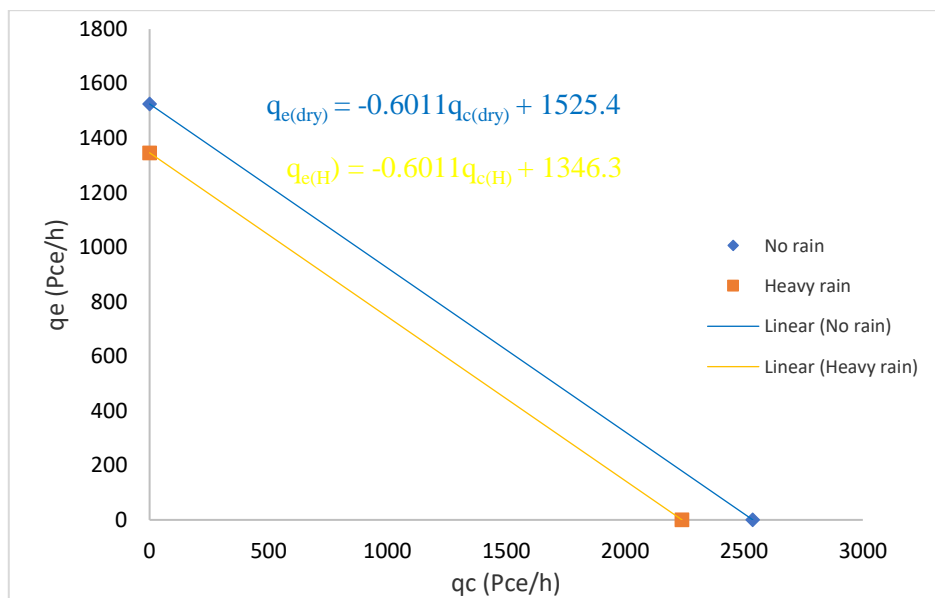


Figure 5.13: Entry - circulating capacity plot for dry and heavy rain conditions for site 03.

The estimated reserved capacity and volume capacity ratio under dry and rainy conditions is estimated using the procedure in step 6 and 7 of section 5.2.2. The results show that reserved capacity is reduced by 0.02 under a light rain, and 0.07 and 0.1 under moderate and heavy rain respectively. The reserved capacity reduces with an increase in rain intensity. The light rain increases the volume capacity ratio by 0.02, moderate rain by 0.07 and heavy rain by 0.10. The rainfall effect shows that the volume capacity ratio increases with an increase in rain intensity. The summary of the estimated reserved capacity and volume capacity ratio under dry, light, moderate and heavy rain are presented in table 5.30.

Table 5.30: Summary of the reserve capacity and volume capacity ratio and at site 03.

	Q_R		ΔQ_R	x		Δx
	Rainfall	Dry		Rainfall	Dry	
Light	0.22	0.24	0.02	0.78	0.76	0.02
Moderate	0.24	0.31	0.07	0.76	0.69	0.07
Heavy	0.16	0.26	0.10	0.84	0.74	0.10

Note: Q_R is reserved capacity, x is volume capacity ratio, Δ is difference.

The delay and queue length are estimated during dry and rainy weather conditions as described in step 8 of section 5.2.2. The results show that there is an increase in the delay at the entry from 21.66s to 23.88s with an increase of 2.22s during light rain, from 16.94s to 20.25s with an increase of 3.31s during moderate rain, and from 21.86s to 31.72s with an increase of 9.83s during heavy rain. Heavy rain has the highest incremental effect on the delay, and light rain has the lowest. The delay at this site increases with the increase in rain intensity, this follows the same pattern as with site 02. The light rain has no effect on the queue length as the queue length of 4 vehicles remains unchanged under dry and light rain. The queue length increases from 3 vehicles to 4 vehicles with moderate rain, and it increases from 4 vehicles to 5 vehicles with heavy rain. The moderate and heavy rain increases the queue length by one vehicle. The queue length increases irrespective of the rain intensity at this site. The summary of the results for delay and queue length is presented in table 5.31.

Table 5.31: Summary of delay and queue length under dry and rainy conditions at site 03.

	d (s)		Δd (s)	L (veh)		ΔL (veh)
	Rainfall	Dry		Dry	Rainfall	
Light	23.88	21.66	2.22	4	4	0
Moderate	20.25	16.94	3.31	3	4	1
Heavy	31.72	21.89	9.83	4	5	1

Note: d is delay, L is queue length, Δ is the difference.

5.4.3 Site 03 Functional quality of service delivery

The developed criterial table 5.28 for the assessment of site 03 functional quality of service in section 5.4.1 is used in the assessment of the FQS during dry and rainy conditions. The FQS assessment under dry and rainy conditions with the delay estimated in section 5.4.2 for site 03 shows that the light and heavy rain influences the service delivery by shifting FQS from FQS C to FQS D. The moderate rain has no effect on the service delivery as the FQS remain unchanged from FQS C. Though the delay during the moderate rain is very close to the upper boundary of class C, which implies that the FQS under moderate rain is almost changing to FQS D. The reason for this unchanged FQS might be that the moderate rain at this site is at the boundary of the light and moderate rain because there is no way a clear boundary could be fixed for rainfall because of the fluctuation in the rain intensity during rainfall. The summary of the FQS assessment under light and rainy conditions at site 03 is presented in table 5.32.

Table 5.32: FQS assessment under dry and rainy conditions at site 03.???????

	d (s)		FQS	
	Rainfall	Dry	Rainfall	Dry
Light	23.88	21.66	D	C
Moderate	20.55	16.94	C	C
Heavy	31.72	21.89	D	C

Note: d is delay, FQS is Functional quality of service.

5.5 Functional Quality of Service Delivery at Site 04 (Gateway Roundabout)

5.5.1 Site 04 Criterial table development

The entry and circulating peak period presented in chapter 4 for site 04 is adopted in this section. A linear relationship is developed with linear regression between peak entry and circulating flow rate as shown in figure 5.22. The model equation is equation 5.32.

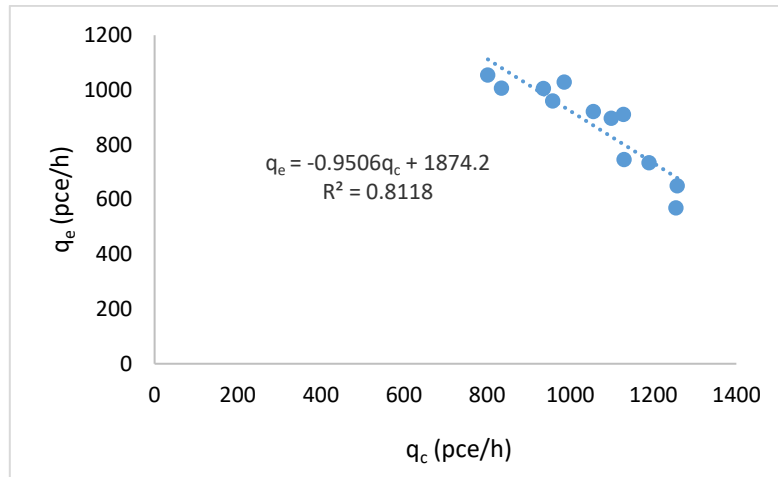


Figure 5.14: Peak entry – circulating flow rate relationship for site 04.

$$q_e = 1874.2 - 0.951 q_c \quad R^2 = 0.82 \quad [5.32]$$

The statistical testing of the model shows that the R^2 is 0.82 which is more than 0.5, this shows that the variables are significant. The t-test is more than 2.2, the F-test is 43.14 which is more than F-critical of 4.84, and this shows that the model equation did not occur by chance. The P-value is 0 which is less than 0.05. The model equation is statistically satisfactory, and it can be used for predictions.

The correction factor was estimated with an entry angle (50°) and entry radius (45 m) to be 0.96, this is used to modify equation 5.32. The modified model equation is shown in equation 5.33.

$$q_e = 1799 - 0.91 q_c \quad [5.33]$$

The entry capacity is estimated as 900 pce/h/lane. The sensitivity test carried out by estimation of the delay and queue length at a volume capacity ratio (x) of 0 and 1. When volume the capacity ratio = 0, delay is 9.00s and the queue length = 0 vehicle. The queue length confirms that this delay is the geometric delay because this delay occurs when there is no vehicle at the roundabout. When the roundabout is operating at capacity, the volume capacity ratio = 1, the delay is 51.44s and the queue length is 13 vehicles. The reserved capacity, delay, and queue

length are estimated for ten equal divisions of the volume capacity ratio from 0 to 1. The summary result is shown in table 5.33.

Table 5.33: Summary of parameters for development of the FQS criterial table at site 04.

x	Q_R	d (s)	L (veh)
0	1	9.00	0
0.1	0.9	9.45	0
0.2	0.8	10.00	0
0.3	0.7	10.71	1
0.4	0.6	11.64	1
0.5	0.5	12.93	2
0.6	0.4	14.82	2
0.7	0.3	17.77	3
0.8	0.2	22.87	5
0.9	0.1	32.62	7
1	0	51.44	13

Note: x is volume capacity ratio, Q_R is reserved capacity, d is delay, L is queue length

The FQS criterial table is developed as explained in section 5.2.1. The developed FQS criterial table will be used in the service delivery assessment at site 04. The developed FQS criterial table for site 04 is presented in table 5.34.

Table 5.34: The criterial table of FQS assessment for site 04.

FQS	d (s)	Q_R	x	(L) (veh)
A	$d \leq 11$	$Q_R \geq 0.6$	$x \leq 0.4$	1
B	$11 < d \leq 15$	$0.4 \leq Q_R < 0.6$	$0.4 < x \leq 0.6$	2
C	$15 < d \leq 23$	$0.2 \leq Q_R < 0.4$	$0.6 < x \leq 0.8$	$2 < L \leq 5$
D	$23 < d \leq 33$	$0.1 \leq Q_R < 0.2$	$0.8 < x \leq 0.9$	$5 < L \leq 7$
E	$33 < d \leq 51$	$0.1 \leq Q_R < 0$	$0.9 < x \leq 1$	$7 < L \leq 13$
F	$d > 51$	$Q_R < 0$	$x > 1$	$L > 13$

Note: FQS is Functional Quality of Service, d is delay, Q_R is reserved capacity, x is volume capacity ratio, L is queue length

5.5.2 Prevailing operational performance at site 04 (Gateway Roundabout)

The off-peak entry and circulating flow rate data under dry and rainy conditions presented in chapter 4 for site 04 were combined with the introduction of a dummy variable following the procedure in section 5.2.2. The combined traffic data was related by multiple linear regression. The model generated for dry and rainy conditions was modified with the correction factor (k). The entry and circulating capacity, reserved capacity, the volume capacity ratio, delay, and queue length were estimated under dry and rainy conditions with the procedure in section 5.2.2. The developed model equations for entry and circulating flow rate under dry and each rainy condition for site 04 are presented in equations 5.34 to 5.36.

$$q_e = 2008 - 1.260q_c - 72.28D_L, \quad R^2 = 0.71 \quad [5.34]$$

$$q_e = 1946 - 1.119q_c - 195D_M, \quad R^2 = 0.75 \quad [5.35]$$

$$q_e = 1842 - 1.078q_c - 265D_H, \quad R^2 = 0.78 \quad [5.36]$$

The statistical testing of the models shows that the coefficient of determinant (R^2) is more than 0.05, the t-test is more than 2.2, the F-test is more than 4.84 (F-critical), the P-value is less than 0.05 for all the model equations which shows that the model equations are statistically fit and could be used for prediction.

The correction factor was estimated with an entry angle (50°) and entry radius (45 m) to be 0.96, this is used in modifying equations 5.34 to 5.36. The modified model equations are shown in equations 5.37 to 5.39.

$$q_e = 1928 - 1.21q_c - 69D_L \quad [5.37]$$

$$q_e = 1869 - 1.15q_c - 187D_M \quad [5.38]$$

$$q_e = 1768 - 1.03q_c - 264D_H \quad [5.39]$$

The entry and circulating capacity are estimated with step 5 of section 5.2.2 procedure. The result of the estimated entry and circulating capacity under dry and rainy conditions shows that light rain reduces entry capacity from 1928 pce/h to 1858 pce/h with a capacity drop of 70 pce/h or 3.63%, while moderate rain reduces the capacity from 1870 pce/h to 1683 pce/h

causing a drop of 187pce/h or 10.00%, and heavy rain causes a reduction from 1768 pce/h to 1504 pce/h with a capacity reduction of 264 pce/h or 14.93%.

The circulating capacity also reduces from 1594 pce/h to 1536 pce/h with a shift of 58 pce/h or 3.64% during light rain, 1626 pce/h to 1464 pce/h with a shift of 162 pce/h or 9.96% during moderate rain, and 1717 pce/h to 1460 pce/h with a shift of 257 pce/h or 14.97% during heavy rain. The entry and circulating capacity reduce with an increase in rain intensity at this site. This also has the same trend as with sites 01 to 03. The summary of the entry and circulating capacity is presented in table 5.35.

Table 5.35: Summary of entry and circulating capacity under dry and rainy conditions at site 04.

	Q_e (pce/h)		ΔQ_e (pce/h)	Q_c (pce/h)		ΔQ_c (pce/h)
	Rainfall	Dry		Rainfall	Dry	
Light	1858	1928	70	1536	1594	58
Moderate	1683	1870	187	1464	1626	162
Heavy	1504	1768	264	1460	1717	257

Note: Q_e is entry capacity, Q_c is circulating capacity, Δ is the difference.

To determine the extent to which rainfall intensity affects the entry and circulating capacity, the entry – circulating capacity are plot under dry and rainy conditions. The light rain has the lowest effect on the capacity differential shift and the highest effect is during heavy rain. However, the light, moderate and heavy rain have the same trend of a negative capacity differential. This shows that both the entry and circulating capacity are affected by rainfall at site 04 irrespective of the intensity. The plotted graphs are shown in figures 5.11 to 5.13.

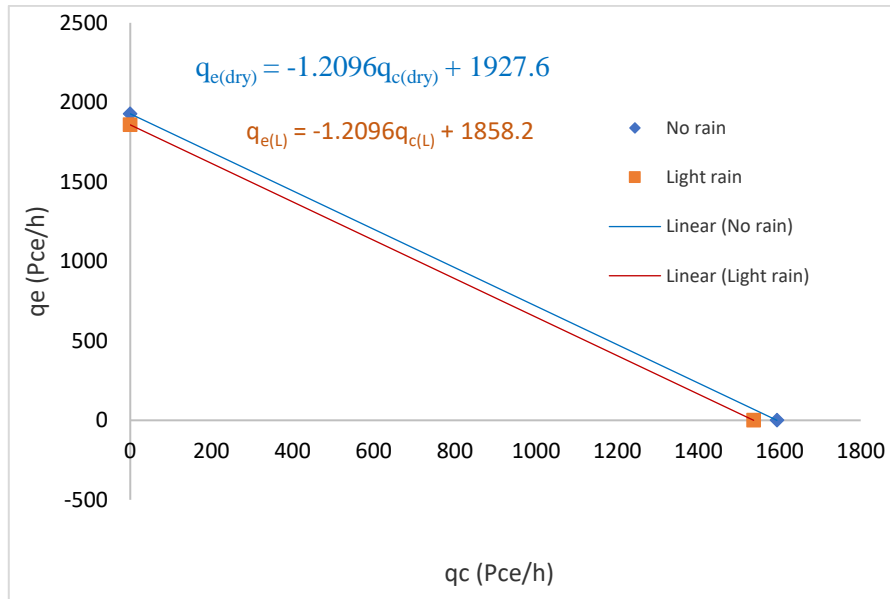


Figure 5.11: Entry - circulating capacity plot for dry and light rain conditions for site 04.

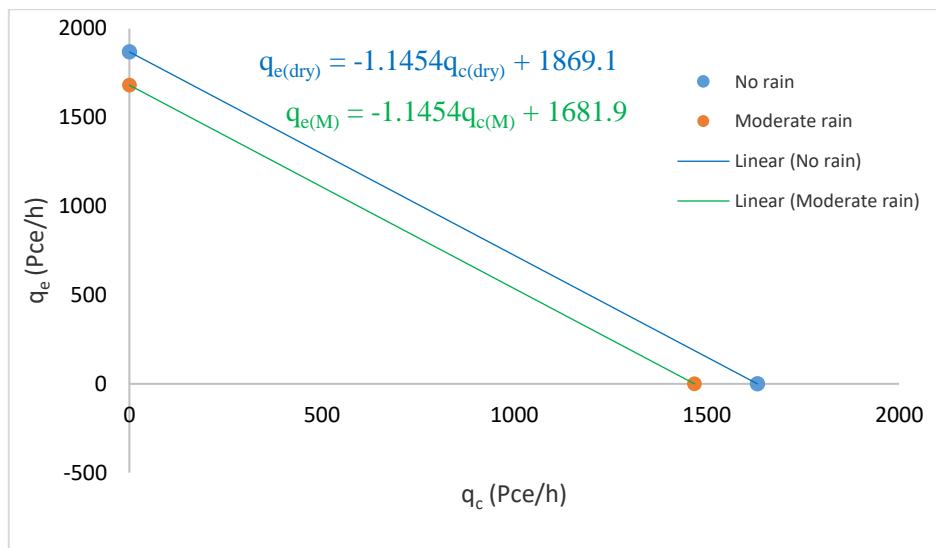


Figure 5.12: Entry - circulating capacity plot for dry and moderate rain conditions for site 04.

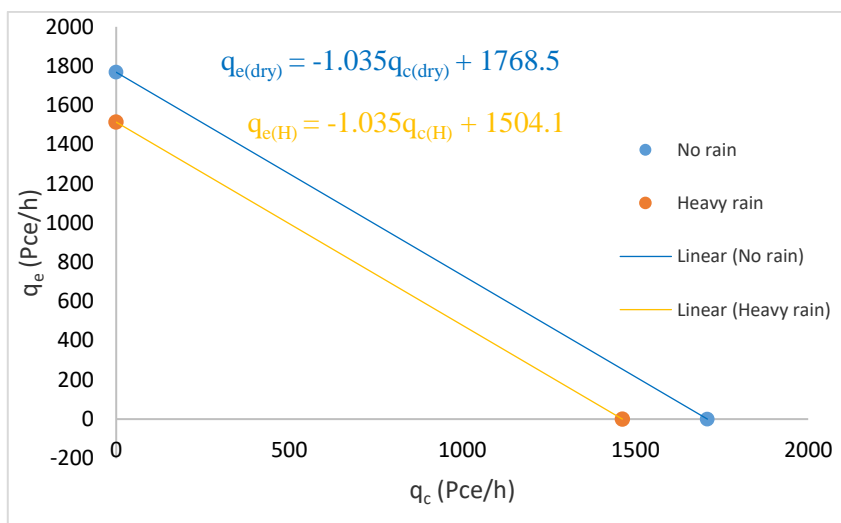


Figure 5.13 Entry - circulating capacity plot for dry and heavy rain conditions for site 04.

The reserved capacity and volume to capacity ratio are estimated with the procedure in section 5.2.2. The results show that the reserved capacity is reduced from 0.43 to 0.36 with a reduction of 0.07 during a light rain, and from 0.41 to 0.37 with a reduction of 0.04 during moderate rain, and from 0.41 to 0.35 with a reduction of 0.04 during heavy rain respectively. Light rain increases the volume capacity ratio from 0.57 to 0.64 with an increase of 0.07, moderate rain causes the capacity ratio to increase from 0.59 to 0.63 with an increase of 0.06, and heavy rain from 0.59 to 0.65 with an increase of 0.06. This shows that the reserved capacity reduces and the volume to capacity ratio increases irrespective of rain intensity at site 04. The results summary for reserved capacity and volume capacity ratio is presented in table 5.36.

Table 5.36: Summary of the degree of saturation and reserve capacity at site 04.

	Q_R		ΔQ_R	x		Δx
	Rainfall	Dry		Rainfall	Dry	
Light	0.36	0.43	0.07	0.64	0.57	0.07
Moderate	0.37	0.41	0.04	0.63	0.59	0.06
Heavy	0.35	0.41	0.04	0.65	0.59	0.06

Note: Q_R is Reserve Capacity, x is volume capacity ratio, Δ is the difference.

The estimated volume capacity ratio is used for estimating the delay and queue length under dry and rainy conditions with the procedure in section 5.2.2. The results show that the delay increases from 13.62s to 15.48s with an increase of 1.86s during a light rain, from 14.25s to 16.22s with an increase of 1.97s during moderate rain, and from 14.76s to 18.14s with an increase of 3.38s with heavy rain. The delay at this site increases with an increase in rain intensity. The queue length increases from 2 vehicles to 3 vehicles irrespective of the rain intensity. The results summary of the delay and queue length are presented in table 5.37.

Table 5.37: Summary of delay and queue length under dry and rainy conditions at site 04.

	d (s)		Δd (s)	L (veh)		ΔL (veh)
	Rainfall	Dry		Dry	Rainfall	
Light	15.48	13.62	1.86	2	3	1
Moderate	16.22	14.25	1.97	2	3	1
Heavy	18.14	14.76	3.38	2	3	1

Note: d is Delay, L is queue length, Δ is the difference.

5.5.3 Functional quality of service delivery at site 04 (Gateway Roundabout)

The FQS criterial table 5.33 for this site is developed in section 5.5.1. The assessment of the FQS during dry and the three rainy conditions with the estimated delay in section 5.5.2 shows that service delivery deteriorates under rainfall irrespective of the rain intensity at this site as the light, moderate and heavy rain shifts the functional quality of service delivery from FQS B to FQS C. The summary of the FQS assessment under dry and rainy conditions at site 04 is presented in table 5.38.

Table 5.38: FQS assessment under dry and rainy conditions at site 04.??????/

	d (s)		FQS	
	Rainfall	Dry	Rainfall	Dry
Light	16.25	12.10	C	B
Moderate	16.93	11.74	C	B
Heavy	16.17	12.77	C	B

Note: d is delay, FQS is Functional quality of service.

5.6 Summary of Functional Quality of Service Assessment

The functional quality of service is multi-parameter that combines the users' and providers' perceptions in the roundabout service delivery. These parameters have been investigated in sections 5.2 to 5.5. The provider is concerned about the roundabout utilisation; hence the reserved capacity and volume capacity ratio is used as the providers' perception of roundabout service delivery. Although, the reserved capacity indicates the traffic flow rate that can be accommodated by the roundabout, the volume capacity ratio is the extent of the roundabout capacity usage. The users are concerned about the time value; hence the delay and queue length are utilised for users' perception of roundabout quality delivery.

5.6.1 Summary of Reserve Capacity and Volume Capacity Ratio at all the Sites

The reserve capacity and volume capacity ratio, which are the parameters that are used by the roundabout providers have been estimated under dry and rainy weather for the four sites in sections 5.2 to 5.5. To have a clear picture of the rainfall effect on these parameters, the results summary for reserve capacity and volume capacity ratios at all the sites shows that the effect of rainfall on reserve capacity and volume capacity ratio at all the sites follows the same pattern, namely that light, moderate and heavy rain reduces the reserve capacity and increases the volume capacity ratio at all the sites. Taking an average of the reserve capacity for all the sites, the reserve capacity reduces by 0.04 or 12.5% under light rain, from 0.35 to 0.29 with a reduction of 0.06 or 17% under moderate rain and from 0.33 to 0.27 with a reduction of 0.06 or 17% under heavy rain.

The volume capacity ratio increases from 0.66 to 0.70 with a reduction of 0.04 or 6.4% under light rain, 0.65 to 0.72 with a reduction of 0.06 or 9.5% under moderate rain, 0.67 to 0.73 with a reduction of 0.06 or 9.0% under heavy rain.

It is evident that the volume capacity ratio increases with an increase in rain intensity. The reason is that as the rainfall intensity increases, the entry capacity reduces, the rate of flow rate into the roundabout is increased in proportion to the capacity at off-peak periods during rainfall. The increase in entry flow rate proportion to entry capacity under rainy periods at off-peak periods can lead to the roundabout operating close to the capacity, this is evident as the reserved capacity reduces under light, moderate and heavy rain. It shows that the amount of capacity reserved is reduced under rainy conditions irrespective of the rain intensity. The summary of the reserved capacity and volume capacity ratio for all the sites is shown in table 5.39.

Table 5.39: Summary of the degree of saturation and reserved capacity under dry and rainy Conditions at all the sites.

Site	Q_R		ΔQ_R	x		Δx	
	Rainfall	Dry		Rainfall	Dry		
01	0.32	0.40	0.08	0.68	0.60	0.08	
02	Light	0.29	0.29	0.00	0.71	0.71	0.00
03		0.22	0.24	0.02	0.78	0.76	0.02
04		0.36	0.43	0.07	0.64	0.57	0.07
01		0.33	0.41	0.07	0.67	0.59	0.08
02	Moderate	0.20	0.26	0.06	0.80	0.74	0.06
03		0.24	0.31	0.07	0.76	0.69	0.07
04		0.37	0.41	0.04	0.63	0.59	0.04
01		0.37	0.36	0.01	0.63	0.63	0.00
02	Heavy	0.20	0.27	0.07	0.80	0.73	0.07
03		0.16	0.26	0.10	0.84	0.74	0.10
04		0.35	0.41	0.06	0.65	0.59	0.06

Note: Q_R is Reserve capacity, x is volume capacity ratio, Δ is the difference.

5.6.2 Summary of Delay and Queue Length at all the Sites

The summary of the delay and queue length at all the sites under dry and rainy conditions shows that delay increases at all the sites irrespective of the rain intensity. The delay behaviour at all sites follows the same pattern as the delay increases under light, moderate and heavy rain respectively.

Taking the average of the increase in delay, the result shows that light rain increases the average delay from 16.47s to 18.16s with an increase of 1.67s or 10.26%, while moderate rain increases the delay from 15.88s to 19.48s with an increase of 3.60s or 22.69%, and heavy rain increases the delay from 17.35 to 22.80s with an increase of 5.45s or 31.44%. Light rain has the least effect on the delay, the reason being that drivers are not adversely affected by the light rain because the impaired visibility increases with an increase in rain intensity. Moderate rain affects the delay more than light rain while heavy rain causes the highest increase in the delay. The effect of heavy rain is greatest because the wide range in the rain class shows that heavy rain might be close to the very heavy rain class. In this situation, the visibility is adversely impaired, and drivers might even find it difficult to judge correctly the safe gap for entry at the yield line. In addition, the time spent in the queue might increase because the entry drivers are more cautious of the leading vehicles, hence they may keep a bigger gap and even reduce speed.

Light rain increases the queue length by 1 vehicle at sites 01 and 04 but has no effect on the queue length at sites 02 and 03. The reason is that the light rain might just be a rain shower at 02 and 03 where the rain intensity is barely up to 1mm/h and the drivers' reactions to this type of rain are not pronounced. Moderate and heavy rain has an increased effect of 1 vehicle on the queue length at all the sites. The reason is that as the delay increases due to a reduction in the speed due to driver caution, and the queue length increases. Taking the average of queue length, the results show that the queue length increases from 3 vehicles to 4 vehicles with an increase of 1 vehicle or 33.33% under dry, light, moderate and heavy rain conditions.

There is a discrepancy between the delay values under the same rainy condition at all surveyed sites. The reason may be because the estimation assumes that the entry and circulating drivers display the same behaviour at all the sites; where the entry vehicles yield to the circulating

vehicles, this may not be so because both the entry and circulating drivers are affected by rainfall, none has an undue advantage over the other, and they are both conscious of the weather conditions. This is to the detriment of the entry vehicles that must yield to the circulating vehicles. The entry drivers might not be able to judge the available gap within the circulating traffic for safe entry due to impaired visibility during rainfall. The entry drivers decelerate and enter the circulating stream cautiously, they may even force themselves into the circulating traffic or the circulating vehicles may slow down to give way to the entry vehicles. Secondly, the distribution of rainfall intensity class may contribute to the entry flow rate overlaps, this is pronounced during heavy rainfall with the class range of 10 to 50mm/h, the rainfall borderline is difficult because of the variation of rain intensity during rainfall. Nevertheless, regardless of the reason for the discrepancy, it is evident that the delay increases with an increase in rainfall intensity and the queue length increases irrespective of the rain intensity. The results summary of delay and queue length under dry and rainy conditions at all the sites is presented in Table 5.40.

Table 5.40: Summary of delay at all the sites.

Site	d (s)		Δd (s)	L (veh)		ΔL (Veh)	
	Rainfall	Dry		Rainfall	Dry		
01	16.25	13.68	4.15	3	2	1	
02	Light	17.04	16.93	0.11	4	4	0
03		23.88	21.66	2.22	4	4	0
04		15.48	13.62	1.86	3	2	1
01	Moderate	16.93	13.22	5.19	3	2	1
02		24.22	19.10	5.15	5	4	1
03		20.55	16.94	3.61	4	3	1
04		16.22	14.25	1.97	3	2	1
01	Heavy	16.17	14.77	3.4	3	3	0
02		25.18	17.99	7.19	5	4	1
03		31.72	21.89	9.83	5	4	1
04		18.14	14.76	3.38	3	2	1

Note: d is Delay, Δ is the difference, L is queue length

5.6.3 Summary of Rain Effect on Functional Quality of Service at all Sites

The functional quality of service delivery by roundabout under dry and rainy conditions for the four sites have been assessed in sections 5.2 to 5.5 with the FQS criterial table developed for each site. Each category of rainfall class is put together for all the sites to have a clear view of how rainfall intensity affects the roundabout service delivery.

The results of the FQS for all sites show that light rain deteriorates the FQS at sites 01 and 04 from FQS B to FQS C, at site 03 from FQS C to FQS D, but there are no changes at site 02 as the FQS remained unchanged from FQS C. The moderate rain downgrades the FQS from FQS B to FQS C at sites 01 and 04, FQS C to D at site 02, but no change at site 03 as the FQS remained unchanged from FQS C. The heavy rain has a deteriorating effect on the FQS at all the sites. At sites 01 and 04, the FQS changes from FQS B to FQS C and at sites 02 and 03 the FQS changes from FQS C to D. With this evidence in the FQS for all sites, it is evident that the functional service delivery deteriorates irrespective of the rain intensity. The summary of the FQS under light and rainy conditions at all the sites is shown in table 5.41.

Table 5.41: Summary of the FQS assessment under dry and rainy conditions at all the sites.

Site	FQS	
	Rainfall	Dry
01	Light	C
02		C
03		D
04		C
01	Moderate	C
02		D
03		C
04		C
01	Heavy	C
02		D
03		D
04		C

Note: FQS = Functional quality of service

5.7 Summary

In this chapter, reserve capacity (Q_R) and volume capacity ratio, delay and queue length at the four sites were estimated using the peak period entry and circulating traffic. These parameters (at peak period) that represent the users' and providers' perspectives were used in the development of the criteria for FQS assessment for each site. The FQS criteria are divided into six classes of FQS A to FQS F and the functional service delivery deteriorates from FQS A to FQS F. FQS A is when there is a free flow rate of entry vehicles into the roundabout and FQS F is when the roundabout is operating above the capacity.

The off-peak data was used to model entry and circulation flow rate under dry and rainy conditions, and the entry and circulating capacity were estimated under dry and rainy conditions. The reserve capacity (Q_R) and volume capacity ratio are the roundabout provider' parameter of assessing the roundabout's functional quality of service, the delay (d) and queue length (L) are the users' parameters of functional quality of service assessment.

These parameters were estimated at the off-peak period under dry, light, moderate and heavy rain at four sites, and the results were compared. The entry and circulating capacity, volume capacity ratio and delay increases with an increase in rain intensity, while the reserve capacity decreases with an increase in rain intensity and the queue length increases irrespective of the rain intensity.

The functional quality of service delivery was assessed under dry, light, moderate and heavy rain at the four sites and the results were compared. The FQS deteriorates irrespective of the rain intensity. The roundabout operates on the give-way principle where the entry vehicle yields to the circulating vehicles. This shows that the time headway at both the circulating and entry vehicles are parameters that can be affected under rainy conditions as the functional quality of service deteriorates irrespective of the rain intensity. Will rainfall have effect on the time headway at the entry and circulating traffic? If there is an effect, then to what extent? Will the headway increase or decrease? This calls for further investigation which is carried out in Chapter 6.

CHAPTER 6

TIME HEADWAY IMPLICATION

6.1 Overview

In chapter 5, the influence of rainfall on the volume to capacity ratio, reserved capacity, delay and queue length were investigated. These parameters are the users' and providers' parameters in the assessment of the effect of rainfall on the roundabout's functional quality of service. As the user and provider parameters change, there could be an implication on the driver's behaviour in terms of time headway and critical gap. The driver's behaviour is influenced by many factors which include the vehicle type, the road conditions, ambient conditions, and the driver's ability to judge the speed of the circulating vehicles. Critical gap is the minimum gap within the circulating traffic that is safe for an entry vehicle to be willing to accept for merging with circulating traffic. Critical gap depends on factors like the geometry layout, the driver behaviour, vehicle characteristics, circulating traffic and ambient conditions. Follow-up time is the minimum time headway between two entry vehicles accepting the same gap within the circulating traffic. The functionality of a roundabout depends on these two parameters because the roundabout operates on the yield rule, whereby a circulating vehicle has the right of way and entry vehicles yield to the circulating vehicle by looking for a safe gap, accepting the gap, and merging with the circulating traffic. There are situations where safe gaps are available within the circulating traffic, but the entry vehicles may decide not to accept the gap, and at times the entry driver accepts gaps smaller than the safe gap. After all the drivers have the right to decide on whether to accept the gap or not, which indicates that driver behaviour is what the driver does and not what the driver can do. Nevertheless, whether the driver accepts the safe gap or a gap that is less than the safe gap, it is evident that gap availability within the circulating traffic determines the number of entry vehicles.

This study is centered around the rainfall implication on both the critical gap and follow-up time. In this chapter, the interaction between circulating and entry vehicles under rainfall of varying intensity will be investigated. This is to determine the implication of rainfall on follow-up time and critical gap.

This chapter is structured as follows: The immediate section 6.2 deals with the schematic diagram of the time headway implication, and in section 6.3 the follow-up time during dry and rainy conditions is investigated at the four sites. The critical gap during dry and rainy conditions is analysed in section 6.4. The implication of rainfall on time headway is discussed in section 6.5 and the summary of the chapter is presented in section 6.6.

6.2 Schematic Diagram of Time Headway Implications

The procedure for the evaluation and analysis of time headway is presented in figure 6.1. Time headway is a parameter that occurs at both the entry and circulating vehicle stream. The interaction between entering and circulating vehicles is measured by the follow-up time and the critical gap. These two parameters need to be evaluated and analysed under dry and rainy weather conditions to determine the extent of the rainfall implication on the vehicle interactions at the roundabout.

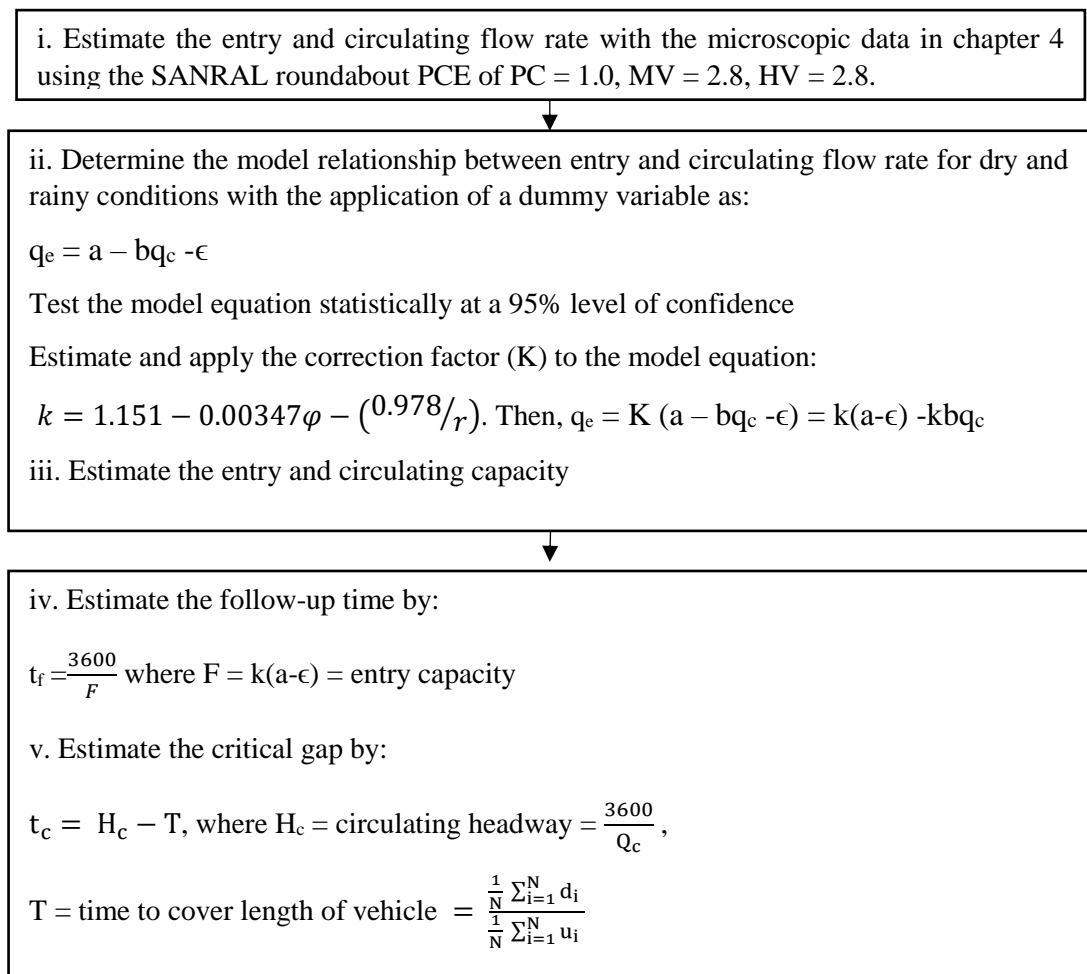


Figure 6.1: Schematic evaluation procedure for the follow-up time and critical gap.

The follow-up time and the critical gap are postulated to decrease with an increase in flow rate under dry and rainy weather conditions, and that they will have a noticeable increase under free flow rate, however as the traffic flow rate increases the effect of rainfall will diminish. At the optimum traffic flow rate, the critical gap and follow-up time will be controlled by the traffic flow rate and rainfall will have little or no effect on the follow-up time and critical gap.

6.3 Follow-up Time Evaluation

6.3.1 Follow-up Time Evaluation at Site 01 (Armstrong Roundabout)

The entry capacity for dry and rainy conditions estimated in chapter 5 for the four sites will be used for the follow-up time headway estimation.

The method of estimation has been demonstrated in chapter 3 where:

$$t_f = \frac{3600}{F} \quad [6.1]$$

t_f = follow-up time headway (s)

$F = Q_e$ = entry capacity (pce/h)

For site 01, the estimated entry capacity under dry and rainy conditions in chapter 5 are recalled and presented in table 6.1.

Table 6.1: Entry capacity at site 01.

Weather condition	Q_e (pce/h)		Q_e (pce/h/lane)	
	Dry	Rainfall	Dry	Rainfall
Light rain	2050	1943	1025	972
Moderate rain	2104	1746	1052	873
Heavy rain	1962	1691	981	845

Note: Q_e is entry capacity

Next, the follow-up time is estimated using equation 6.1

As an example, when the $F = Q_e = 1025 \text{ pce/h/lane}$

$$t_f = \frac{3600}{1025} = 3.51 \text{ s}$$

This procedure is repeated in the estimation of the follow-up time under light, moderate and heavy rain when the entry flow rate is at capacity. The results show that light rain increases the follow-up time headway from 3.15s to 3.7s with an increase of 0.19s, and moderate rain increases the follow-up time headway from 3.42s to 4.12s causing an increase of 0.7s, and during heavy rain it increases from 3.67s to 4.26s with an increase of 0.59s. The summary of the follow-up time at site 01 is presented in table 6.2.

Table 6.2: Summary of the follow-up time at entry capacity at site 01.

Weather condition	t_f (s)		Δt_f (s)
	Dry	Rainfall	
Light rain	3.51	3.7	0.19
Moderate rain	3.42	4.12	0.70
Heavy rain	3.67	4.26	0.59

Note: t_f is follow-up time headway

The results in table 6.2 are insufficient to affirm the response of follow-up headway to rainfall because when the roundabout is operating at capacity, the entry driver reacts to indistinguishable effects of both the traffic and the rainfall effect. To determine the effect of rainfall on the follow-up time, the follow-up time will be estimated at varying traffic flow rates. The follow-up time is estimated under varying traffic flow rate at the volume to capacity ratio of 0.1 to 1.0.

As an example, the follow-up time headway at 0.9 volume to capacity ratio is estimated as:

For light rain and dry conditions:

The entry capacity under dry weather conditions = 1025 pce/h

The entry flow rate at 0.9 volume to capacity ratio (x) = $0.9 \times 1025 = 923$ pce/h

The follow-up time = $\frac{3600}{923} = 3.90$ s

The same procedure is used in estimating the follow-up time under dry and rainy weather conditions with varying rain intensity at a volume to capacity ratio of 0.1 to 1.0.

The results show that the effect of light rain on follow-up time decreases as the volume to capacity ratio increases. Although, the follow-up time also decreases as the volume to capacity ratio increases under dry and light rain condition. This is because as the entry flow rate increases, the vehicle following takes more caution of the leading vehicle and struggles for the safe gap within the circulating traffic to enter the roundabout. The follow-up time increases under light rain, for example, at free flow rate, when the volume to capacity ratio is 0.1, it increases from 35.12s to 37.06s with an increase of 1.94s, while at a volume to capacity ratio of 0.5, it increases from 7.02s to 7.41s with an increase of 0.39s, and at 0.9 volume to capacity ratio, it increases from 3.9s to 4.12s with an increase of 0.22s, and at capacity when the volume to capacity ratio is 1.0, it increases from 3.51s to 3.71s causing an increase of 0.2s. There is a general increase in follow-up time in light rain, the reason being that the entry drivers are more cautious during light rain in accepting the gap within the circulating traffic. Though, the increase is small but more pronounced at free flow rate, this shows that the drivers react mainly to a light rain effect at free flow rate. As the flow rate increases, the drivers gradually become more cautious of the traffic and the effect of the light rain gradually decreases.

The results also show that at capacity when the volume to capacity ratio (x) is 1.0, the light rain effect is almost nullified and insignificant. The reason is because the entry driver reacts to both the light rain and the optimum traffic flow rate, as the effect of the light rain and optimum traffic flow rate cannot be separated. The summary of the follow-up time under dry and light rain conditions at site 01 is presented in table 6.3.

Table 6.3: Summary of the follow-up time under dry and light rain at site 01.

x	t_f (s)		Δt_f (s)
	Dry	Light rain	
0.1	35.12	37.06	1.94
0.2	17.56	18.53	0.97
0.3	11.71	12.35	0.64
0.4	8.78	9.26	0.48
0.5	7.02	7.41	0.39
0.6	5.85	6.18	0.33
0.7	5.02	5.29	0.27
0.8	4.39	4.63	0.24
0.9	3.9	4.12	0.22
1.0	3.51	3.71	0.20

Note: x is volume capacity ratio, t_f is follow-up time headway, Δ is difference

The results of follow-up time under dry and moderate rain show that the follow-up time increases under moderate rain and the moderate rain effect diminishes as the volume to capacity ratio increases. For example, at free flow rate of 0.1 volume capacity ratio, the follow-up time increases from 34.22s to 41.24s with an increase of 7.02s. It also increases from 6.84s to 8.25s at a volume to capacity ratio of 0.5 with an increase of 1.45s. At 0.9 volume to capacity ratio, it increases from 3.8s to 4.58s causing an increase of 0.78s. At optimum traffic flow rate when the volume to capacity ratio is 1.0, it increases from 3.42s to 4.12s causing an increase of 0.7s. The moderate rain effect on follow-up time follows the same pattern as with a light rain effect. The effect of moderate rain is more pronounced at free flow rate. The moderate rain effect diminishes as the volume to capacity ratio increases. The reason is that as the traffic flow rate increases, the entry driver becomes more cautious of the leading vehicles and takes time to judge correctly the gap within the circuiting traffic, hence the effect of moderate rain diminishes. The summary of the follow-up time under dry and moderate rain at site 01 is presented in table 6.4.

Table 6.4 Summary of the follow-up time under dry and moderate rain at site 01.

x	t_f (s)		Δt_f (s)
	Dry	Moderate rain	
0.1	34.22	41.24	7.02
0.2	17.11	20.62	3.51
0.3	11.41	13.35	1.94
0.4	8.56	10.31	1.75
0.5	6.84	8.25	1.41
0.6	5.7	6.87	1.17
0.7	4.89	5.89	1.00
0.8	4.28	5.15	0.87
0.9	3.8	4.58	0.78
1.0	3.42	4.12	0.70

Note: x is volume capacity ratio, t_f is follow-up time headway, Δ is difference

The follow-up time under moderate rain is greater than under light rain. The reason is because the entry vehicle takes more caution under moderate rain and keeps a bigger gap from the leading vehicles. They are also more careful in accepting the gap because as the rain intensity increases from light rain to moderate rain, the visibility becomes more impaired, and the judgement of the critical gap needs more caution. Hence an increase in follow-up time. The results of follow-up time under dry and heavy rain show that the heavy rain effect on follow-up time follows the same pattern as with light and moderate rain, because the effect of heavy rain reduces with an increase in the volume to capacity ratio. The follow-up time increases from 36.7s to 42.58s at free flow rate of 0.1 volume to capacity ratio. At 0.5 volume to capacity ratio, it increases from 7.34s to 8.52s with an increase of 1.18s. It also increases from 4.08s to 4.73s at 0.9 volume to capacity ratio with an increase of 0.65s, and from 3.67s to 4.26s at an optimum entry flow rate when volume to capacity ratio is 1.0 with an increase of 0.59s. The effect of heavy rain on follow-up time is more pronounced at free flow rate and the effect reduces as the volume to capacity ratio increases, but the effect is smallest at an optimum entry flow rate. The summary of the follow-up time under dry and heavy rain at site 01 is presented in table 6.5.

Table 6.5: Summary of the follow-up time under dry and heavy rain at site 01.

x	t_f (s)		Δt_f (s)
	Dry	Heavy rain	
0.1	36.70	42.58	5.88
0.2	18.35	21.29	2.27
0.3	12.23	14.19	1.87
0.4	9.17	10.64	1.47
0.5	7.34	8.52	1.18
0.6	6.12	7.1	0.98
0.7	5.24	6.08	0.84
0.8	4.59	5.32	0.73
0.9	4.08	4.73	0.65
1	3.67	4.26	0.59

Note: x is volume capacity ratio, t_f is follow-up time headway, Δ is difference

Moderate rain has a greater effect on follow-up time than heavy rain at this site, which is expected because the percentage composition of both the circulating medium and heavy vehicles is 4.32% and 1.87% which is higher than 2.47% and 1.44% of circulating medium and heavy vehicles under heavy rain. The entry drivers take more caution in accepting the critical gap with the presence of heavy vehicles in the circulating traffic.

6.3.2 Follow-up Time Evaluation at Site 02 (Millennium Roundabout)

The estimated capacity for site 02 in chapter 5 is recalled, and the entry capacity per lane is estimated and presented in table 6.6. This is for follow-up estimation. The estimated follow-up time at capacity will not be considered separately at this site and subsequent sites because it will form part of the analysis under varying volume to capacity ratios.

Table 6.6: Entry capacity during dry and rainy conditions at site 02.

Weather condition	Q _e (pce/h)		Q _e (pce/h/lane)	
	Dry	Rainfall	Dry	Rainfall
Light rain	1978	1974	989	987
Moderate rain	1848	1661	924	831
Heavy rain	1926	1572	963	786

Note: Q_e is entry capacity

To determine the rainfall effect on the follow-up time, the follow-up time is estimated under varying traffic flow rate. There is no need to consider all the volume to capacity ratios from 0.1 to 1.0 as considered in site 01. Only free flow rate of volume to capacity ratio of 0.1, 0.5 and the warning to alert threshold of 0.9 volume to capacity ratio, and the optimum entry traffic flow rate of volume to capacity ratio of 1.0 will be considered in this site and the subsequent sites.

The summary of the estimated follow-up time under dry and rainy conditions shows that the follow-up time increases irrespective of rain intensity and the effect diminishes as the volume to capacity ratio increases. At the free flow rate of 0.1 volume to capacity ratio, rainfall has a very significant effect on follow-up time because entry drivers react mainly to the rainfall. Under light rain, the follow-up time increases from 36.40s to 39.49s with an increase of 3.09s. It increases from 38.96s to 43.35s with an increase of 4.39s under moderate rain. The increase is from 37.38s to 45.77s with an increase of 8.39s under heavy rain. This shows that at free flow rate the heavy rain has the highest increasing effect. This is because the drivers being more cautious of driving into the roundabout or accepting a gap under heavy rain. The rainfall effects increase through light rain to heavy rain because as the rain intensity increases, the drivers' reactions to rainfall becomes more pronounced. At the 0.5 volume to capacity ratio, the follow-up time increases from 7.29s to 4.04s with an increase of 0.01s under light rain. The moderate rain increases the follow-up time from 7.79s to 8.67s with an increase of 0.88s. Heavy rain causes an increase of 1.97s by increasing the follow-up time from 7.48s to 9.15s. The light rain effect is negligible at 0.5 volume to capacity ratio, the reason is because the light rain might have little effect on the entry driver because the light rain intensity at this site might be very low. As the roundabout is operating close to capacity at a volume to capacity ratio of 0.9, the follow-up time increases from 4.04s to 4.05s under light rain with an increase of 0.01s.

Under moderate rain, it increases from 4.33s to 4.82s with an increase of 0.49s, and from 4.15s to 5.09s with an increase of 0.94s under heavy rain. At optimum flow rate when the volume to capacity ratio is 1.0, it increases from 3.64s to 3.65s with an increase of 0.1s under light rain. It increases from 3.90s to 4.33s with an increase of 0.43s under moderate rain, and 3.74s to 4.58s with an increase of 0.84s under heavy rain.

Heavy rain has the highest effect irrespective of the volume to capacity ratio at this site because as the rain intensity keeps increasing the drivers become more reactive to the rainfall and they take more caution in judging the circulating gap correctly. At this site the follow-up time increases with an increase in rain intensity and the rainfall effect decreases with an increase in volume to capacity ratio, and at entry capacity the rainfall effect has the lowest effect on follow-up time irrespective of rain intensity. The summary of the follow-up time under dry and rainy conditions is presented in table 6.7.

Table 6.7: Summary of the follow-up time under dry and rainy conditions at site 02.

Weather condition	Volume capacity ratio							
	0.1		0.5		0.9		1.0	
	Follow-up time (s)							
	Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	36.40	39.49	7.28	7.29	4.04	4.05	3.64	3.65
Moderate rain	38.96	43.35	7.79	8.67	4.33	4.82	3.90	4.33
Heavy rain	37.38	45.77	7.48	9.15	4.15	5.09	3.74	4.58

6.3.3 Follow-up Time Evaluation at Site 03 (Douglas Roundabout)

The entry capacity estimated during the dry and rainy weather conditions in chapter 5 for site 03 are recalled. The entry capacity per lane is estimated for follow-up time analysis at site 03 as shown in table 6.8.

Table 6.8: Entry capacity during dry and rainy conditions at site 03.

	Q _e (pce/h)		Q _e (pce/h/lane)	
	Dry	Rain	Dry	Rain
Light rain	1682	1536	841	768
Moderate rain	1864	1804	932	902
Heavy rain	1525	1346	763	673

Note: Q_e is entry capacity.

The follow-up time is estimated under dry and rainy conditions with varying volume to capacity ratios, the results show that the follow-up time increases with an increase in rain intensity. As an example, at free flow rate of 0.1 volume to capacity ratio, the effect of rainfall is pronounced as it increases from 42.81s to 46.88s under light rain with an increase of 4.07s. Moderate rain increases the follow-up time from 38.63s to 39.91s with an increase of 1.28s. Heavy rain causes an increase from 47.21s to 53.49s with an increase of 6.28s. At the free flow rate, the driver reacts mainly to the rainfall effect. At a volume to capacity ratio of 0.5, the follow-up time increases from 8.59s to 9.38s with an increase of 0.79s under light rain. It increases from 7.73s to 7.98s with an increase of 0.25s under moderate rain. It also increases from 9.44s to 10.70s with an increase of 1.26s under heavy rain. This trend is followed at a volume to capacity ratio of 0.9. This shows that as the volume to capacity ratio increases, the driver starts to combine the effect of traffic with the rainfall effect and the rainfall effects gradually decrease. At capacity, when the volume to capacity ratio is 1.0, the follow-up time increases from 4.28s to 4.69s with an increase of 0.41s under light rain. It increases from 3.86s to 3.99s with an increase of 0.13s under moderate rain. It also increases from 4.72s to 5.33s with an increase of 0.61s under heavy rain. This shows that at optimum flow rate the effect of rainfall is minimal irrespective of the rain intensity because at this condition, the drivers react more to the traffic than the rainfall. Heavy rain has the highest effect on the follow-up time at this site and the effect of rainfall reduces as the volume to capacity ratio increases. At entry capacity when the volume to capacity ratio is 1.0, the traffic flow rate effect takes control of the follow-up time as the effect of rainfall becomes minimal. This follows the same pattern as sites 01 and 02. The summary of the estimated follow-up time under dry and rainy conditions of varying intensity and volume to capacity ratio is presented in table 6.9.

Table 6.9: Summary of the follow-up time under dry and rainy conditions at site 03.

Weather condition	Volume to capacity ratio							
	0.1		0.5		0.9		1.0	
	Follow-up time (s)							
	Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	42.81	46.88	8.56	9.38	4.76	5.21	4.28	4.69
Moderate rain	38.63	39.91	7.73	7.98	4.29	4.43	3.86	3.99
Heavy rain	47.21	53.49	9.44	10.70	5.25	5.94	4.72	5.33

6.3.4 Follow-up Time Evaluation at Site 04 (Gateway Roundabout)

The entry capacity estimated in chapter 5 under dry and rainy weather conditions at site 04 are recalled. The entry capacity per lane is estimated as presented in table 6.10 for follow-up estimation at site 04.

Table 6.10: Entry capacity during dry and rainy conditions at site 04.

Weather condition	Q _e (pce/h)		Q _e (pce/h/lane)	
	Dry	Rain	Dry	Rain
Light rain	1928	1858	964	929
Moderate rain	1870	1683	935	842
Heavy rain	1769	1514	885	757

Note: Q_e is entry capacity.

The follow-up time is estimated under dry and rainy weather conditions with varying volume to capacity ratios. The results show that follow-up time increases with an increase in rain intensity, the effect is more pronounced at free flow rate and it reduces with an increase in volume to capacity ratio. As an example, at free flow rate when the volume to capacity ratio is 0.1, the light rain increases the follow-up time from 37.34s to 38.75s with an increase of 1.41s. The increase under moderate rain is 4.28s as it increases from 38.50s to 42.78s. Heavy rain causes an increase of 7.60s as the increase is from 40.27s to 47.87s. At a 0.5 volume to

capacity ratio, the follow-up time increases from 7.47s to 7.75s during light rain with 0.28s increase. It increases from 7.70s to 8.56s with an increase of 0.86s under moderate rain. The increase is from 8.14s to 9.57s with an increase of 1.43s under heavy rain. At a volume to capacity ratio of 0.9, the increasing effect caused by light rain is 0.16s as the follow-up time increases from 4.15s to 4.31s. Under moderate rain, it increases by 0.47s as it increases from 4.28s to 4.75s. The effect becomes smaller because the drivers' reactions to the leading vehicles and judgment of the gap within the circulating traffic increases. At a volume to capacity ratio of 1.0 the effect of rainfall becomes minimal as the effect of optimum traffic flow rate is combined with the rainfall effect. The follow-up time increases by 0.15s under light rain as it increases from 3.78s to 3.88s. The increase is 0.43s under moderate rain as it increases from 3.85s to 4.28s. Under heavy rain it increases from 4.07s to 4.79s with an increase of 0.72s. At this site, the follow-up time increases with an increase in rain intensity and the effect reduces with an increase in the volume to capacity ratio. At an optimum entry traffic flow rate, the effect becomes minimal as the traffic conditions become the leading controller of the follow-up time. The estimated follow-up time at varying volume to capacity ratios under dry and rainy weather is presented in table 6.11.

Table 6.11: Summary of the follow-up time under dry and rainy conditions at site 04.

Weather condition	Volume to capacity ratio							
	0.1		0.5		0.9		1.0	
	Follow-up time (s)							
	Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	37.34	38.75	7.47	7.75	4.15	4.31	3.73	3.88
Moderate rain	38.50	42.78	7.70	8.56	4.28	4.75	3.85	4.28
Heavy rain	40.27	47.87	8.14	9.57	4.52	5.32	4.07	4.79

6.4 Critical Gap Evaluation

The critical gap is estimated using the empirical method. The gap is the minimum time between the two circulating vehicles for entry drivers to drive safely into the roundabout. This is different from headway because it can be taken at the headway less the vehicle length in terms of time, as shown in equation 6.2.

$$\text{Gap} = H_c - T \quad [6.2]$$

Where:

$$H_c = \text{circulating headway} = \frac{3600}{Q_c} \quad [6.3]$$

Q_c = circulating capacity (pce/h).

$$T = \text{vehicle length} = \frac{\frac{1}{N} \sum_{i=1}^N d_i}{\frac{1}{N} \sum_{i=1}^N u_i} \quad [6.4]$$

$\frac{1}{N} \sum_{i=1}^N d_i$ = average length of all the vehicles under weather condition i.

$\frac{1}{N} \sum_{i=1}^N u_i$ = average speed of all vehicles under weather condition i.

By fixing equation 6.4 to 6.2, then:

$$t_c = H_c - \frac{\frac{1}{N} \sum_{i=1}^N d_i}{\frac{1}{N} \sum_{i=1}^N u_i} \quad [6.5]$$

Where, t_c = critical gap.

6.4.1 Critical Gap Evaluation for Site 01 (Armstrong Roundabout)

The circulating capacity under dry and rainy conditions at site 01 are recalled from chapter 5 for the purpose of estimating the critical gap at site 01. The recalled capacity under dry and rainy conditions are shown in table 6.12.

Table 6.12: Circulating capacity during dry and rainy conditions at site 01.

Weather condition	Q_c (pce/h)		Q_c (pce/h/lane)	
	Dry	Rain	Dry	Rain
Light rain	2129	2018	1065	1009
Moderate rain	2085	1730	1043	865
Heavy rain	2207	1903	1104	952

Note: Q_c is circulating capacity.

Site 01 will be used for the sample calculation of critical gap estimation. The dry weather conditions in combination with the light weather conditions will be used for the sample calculation.

At dry weather, $Q_c = 2129$ pce/h.

Q_c for a single lane = 1065 pce/h/lane.

The headway is estimated using equation 6.2

$$H_c = \frac{3600}{1065} = 3.38s$$

The average vehicle length is estimated from the average length of all vehicles under dry weather conditions.

The average wheel (W_B) base of passenger cars = 2.65m (from ATC data)

The average wheel base of medium vehicles = 4.11m (from ATC data)

The average wheel base of heavy vehicles = 8.9m (from ATC data)

The front overhanging (O_F) of passenger cars = 1.03m (SANRAL)

The rear overhanging (O_R) of passenger cars = 1.53m (SANRAL)

The front overhanging of medium vehicles = 2.1m (SANRAL)

The rear overhanging of medium vehicles = 2.1m (SANRAL)

The front overhanging of heavy vehicles = 0.9m (SANRAL)

The rear overhanging of heavy vehicles = 0.6m (SANRAL)

$$\text{The length of a vehicle} = W_B + O_F + O_R \quad [6.6]$$

Where: W_B = wheel base

O_F = front overhanging

O_R = rear overhanging

Substitute for wheel base, front and rear overhanging in equation 6.6 to determine vehicle length.

The length of a passenger car = $2.65 + 1.03 + 1.53 = 5.21\text{m}$

The length of a medium vehicle = $4.11 + 2.1 + 2.1 = 8.31\text{m}$

The length of a heavy vehicle = $8.9 + 0.9 + 0.6 = 10.4\text{m}$

The average vehicle length can be estimated by finding the average of all the vehicles' lengths, taking into consideration the percentage composition of the vehicles.

The circulating traffic percentage composition at site 01 under dry weather conditions from chapter 4 are recalled as:

Passenger cars = $94.3\% = 0.943$

Medium vehicles = $2.93\% = 0.0293$

Heavy vehicles = $2.12\% = 0.0212$

The average length of the vehicles is estimated as the average of the sum of the products of the vehicles' length and composition proportion.

$$\text{The average length} = \frac{(5.21 \times 0.943) + (8.31 \times 0.0293) + (10.4 \times 0.0212)}{0.943 + 0.0293 + 0.0212} = 5.4\text{m}$$

The average speed of all the vehicles = $45.37\text{km/h} = 12.60\text{m/s}$ (from ATC data)

Then using equation 6.4, T is estimated as:

$$T = \frac{5.4}{12.60} = 0.43\text{s}$$

The gap is estimated using equation 6.5 as:

$$t_c = 3.38 - 0.43 = 2.95\text{s}$$

The same procedure is used in estimating the gap under light, moderate and heavy rain. The summary of the gap within the circulating traffic at circulating capacity under dry and rainy conditions is presented in table 6.13.

Table 6.13: Summary of the gap at entry capacity at site 04.

Weather condition	t_c (s)		Δt_c (s)
	Dry	Rain	
Light rain	2.95	3.05	0.10
Moderate rain	3.05	3.61	0.56
Heavy rain	2.86	3.14	0.28

Note: t_c is critical gap.

The summary results in table 6.13 show that the gap increases from 2.95s to 3.05s with an increase of 0.1s under light rain, from 3.05s to 3.61s with an increase of 0.56s under moderate rain, and from 2.86s to 3.14s with an increase of 0.28s under heavy rain. The gap at circulating capacity cannot be taken as a critical gap because under this situation, the circulating roadway is operating at capacity and no entry vehicle is expected to enter the circulating roadway. Nevertheless, the results in table 6.13 might not be enough to determine the rainfall effect as the circulating drivers react to indistinguishable effects of both the optimum circulating traffic flow rate and rainfall at the same time.

To determine the effect of rainfall on the critical gap, it is estimated under free flow rate of 0.1 to 0.9 volume to capacity ratio. For example, when the circulating volume to capacity ratio is 0.9 under dry weather conditions, then the traffic flow rate = 0.9 x 1065 = 959 pce/h/lane

$$H_c = \frac{3600}{959} = 3.75s.$$

Then the critical gap = 3.75 – 0.43 = 3.32s.

The same procedure is used to estimate the critical gap for a volume to capacity ratio of 0.1 to 0.9 under dry and rainy conditions.

The results under dry and light rain show that the critical gap increases under light rain conditions, for example, the effect is more pronounced at a free flow rate of 0.1 volume to capacity ratio where the light rain increases the critical gap from 33.42s to 35.16s with an increase of 1.74s. This is because the circulating drivers react mainly to the light rain at free flow rate. Also, at circulating free flow rate, the entry traffic is high, and the circulating drivers are cautious of merging with the entry vehicles under light rain. When the volume to capacity

ratio is 0.5, the critical gap increases from 6.36s to 6.62s with an increase of 0.26s. At a volume to capacity ratio of 0.9, it increases from 3.36s to 3.45s with an increase of 0.09s. At capacity when the volume to capacity ratio is 1.0 and there are no entry vehicles, the gap increases from 2.98 to 3.05s with an increase of 0.1s, which shows that the light rain effect is insignificant because the drivers react more to the traffic situation. The summary of the results under dry and light rain is presented in tables 6.14.

Table 6.14: Summary of the critical gap under dry and light rain at site 01.

x	t_c (s)		Δt_c (s)
	Dry	Light rain	
0.1	33.42	35.16	1.74
0.2	16.51	17.32	0.81
0.3	10.87	11.38	0.51
0.4	8.05	8.40	0.35
0.5	6.36	6.62	0.26
0.6	5.24	5.43	0.19
0.7	4.43	4.58	0.15
0.8	3.83	3.94	0.11
0.9	3.35	3.45	0.11
1.0	2.98	3.05	0.10

Note: x is volume capacity ratio, t_c is critical gap, Δ is difference

The results of the critical gap under dry and moderate rainfall show that critical gap increases under moderate rain. For example, at a free flow rate of 0.1 volume to capacity ratio, the critical gap increases from 34.13s to 41.07s with an increase of 6.94s. The effect of moderate rain is as noticeable at the free flow rate. The effect decreases as the circulating flow rate increases. When the volume to capacity ratio is 0.5, the critical gap increases from 6.51s to 7.77s with an increase of 1.26s. Also, at a volume to capacity ratio of 0.9, the critical gap increases from 3.44s to 4.07s with an increase of 0.63s. At circulating capacity, with a volume to capacity ratio of 1.0, the gap increases from 3.10s to 3.60s with an increase of 0.5s. This follows the same pattern as with the light rain effect. Although the circulating traffic has a continuous flow rate, the car following keeps a bigger gap from the leading vehicles because of the rainy

conditions, such as reduction in visibility and friction between tyre and the pavement. Moderate rain has a greater effect than the light rain at this site. The summary of the results under dry and moderate rain is presented in table 6.15.

Table 6.15: Summary of the critical gap under dry and moderate rainfall at site 01.

x	t_c		$\Delta t_c(s)$
	Dry	Moderate rain	
0.1	34.13	41.07	6.94
0.2	16.87	20.26	3.39
0.3	11.11	13.32	2.21
0.4	8.23	9.85	1.62
0.5	6.51	7.77	1.26
0.6	5.36	6.38	1.02
0.7	4.53	5.39	0.86
0.8	3.92	4.65	0.73
0.9	3.44	4.07	0.63
1.0	3.10	3.60	0.50

Note: x is volume capacity ratio, t_c is critical gap, Δ is difference

The results of the critical gap under dry and heavy rainfall shows that heavy rain has an incremental effect on the critical gap. At free flow rate of 0.1 volume to capacity ratio, the critical gap increases from 32.22s to 37.19s with an increase of 4.97s, which is a pronounced effect. When the volume to capacity ratio is 0.5, the critical gap increases from 6.12s to 6.92s with an increase of 0.8s. The result also shows that as the volume to capacity ratio increases, the effect of rainfall diminishes, for example when the volume to capacity ratio is 0.9, the critical gap increases from 3.22s to 3.56s with an increase of 0.34s. At circulating capacity, the volume to capacity ratio is 1.0, the gap increases from 2.90s to 3.10s with an increase of 0.2s. This shows that at capacity, the gap is greatly influenced by the circulating traffic flow rate and rainfall irrespective of the intensity which has very little effect. Though the optimum traffic flow rate and rain effect cannot be separated. The summary of the results under dry and heavy rain is presented in tables 6.16.

Table 6.16: Summary of the critical gap under dry and heavy rainfall at site 01.

x	t_c		$\Delta t_c(s)$
	Dry	Heavy rain	
0.1	32.22	37.19	4.97
0.2	15.95	18.27	2.32
0.3	10.47	11.97	1.50
0.4	7.76	8.82	1.06
0.5	6.12	6.92	0.8
0.6	5.04	5.66	0.62
0.7	4.26	4.76	0.50
0.8	3.68	4.09	0.41
0.9	3.22	3.56	0.34
1.0	2.90	3.10	0.20

Note: x is volume capacity ratio, t_c is critical gap, Δ is difference

The critical gap under heavy rain is less than the moderate rain effect because the number of circulating medium and heavy vehicles under moderate rain are higher than under heavy rain. The circulating vehicles keep a greater distance from these vehicles under moderate rain, hence creating a higher critical gap. At this site, the critical gap increases irrespective of the rain intensity and the rainfall effect irrespective of the intensity decreases as the circulating volume to capacity ratio increases.

6.4.2 Critical Gap Evaluation for Site 02 (Millennium Roundabout)

The critical gap at this site is estimated using the procedure in subsection 6.4.1. The circulating capacity under dry and rainy conditions are recalled from chapter 5 and presented in table 6.17. The critical gap estimation at varying circulating volume to capacity ratios will be limited to 0.1, 0.5, and 0.9 volume to capacity ratios for this site and the subsequent site. The effect of rainfall on gap at 1.0 volume capacity ratio will also be considered.

Table 6.17: Circulating capacity during dry and rainy conditions at site 02.

Weather condition	Q _c (pce/h)		Q _c (pce/h/lane)	
	Dry	Rain	Dry	Rain
Light rain	1810	1806	905	903
Moderate rain	2110	1896	1055	948
Heavy rain	1914	1563	957	782

Note: Q_c is circulating capacity

The results of critical gap under dry and rainy conditions with varying volume to capacity ratios show that light rain has no significant effect on critical gap. At free flow rate of 0.1 volume to capacity ratio, the critical gap increases from 39.25s to 39.28s with an increase of 0.03s. At 0.5 volume to capacity ratio, the critical gap remains unchanged from 7.40s. At a 0.9 volume to capacity ratio, it increases from 3.84s to 3.89s with an increase of 0.05s. This shows that light rain has no significant effect on the critical gap at this site. The critical gap is mainly controlled by the circulating traffic flow rate, the reason is because the light rain at this site might be a rain with very low intensity which does not have a noticeable effect on driver behaviour. At a volume to capacity ratio of 1.0 the gap increases from 3.40s to 3.44s with an increase of 0.1s. The effect of moderate and heavy rain is more pronounced at a free flow rate of 0.1 volume to capacity ratio. When the volume to capacity ratio is 0.1, the critical gap increases from 33.59s to 37.27s with an increase of 3.65s under moderate rain, heavy rain causes an increase of 8.28s by increasing the critical gap from 37.08s to 45.36s. This is because drivers react mainly to rainfall at free flow rate. Heavy rain has the highest effect at free flow rate because as the rain intensity increases, the circulating drivers become more cautious because of the rainfall and try to avoid unnecessary merging of the entry vehicles.

As the circulating volume to capacity ratio increases, the moderate and heavy rain effect keeps decreasing. At a volume to capacity ratio of 0.5, the moderate rain increases the critical gap from 6.29s to 6.89s with an increase of 0.6s. The heavy rain increases the critical gap from 6.99s to 8.51s with an increase of 1.51s. At a 0.9 volume to capacity ratio the moderate rain increases from 3.26s to 3.52s with an increase of 0.26s. Heavy rain causes an increase of 0.77s by increasing the critical gap from 3.65s to 4.42s. This shows that as the circulating traffic increases, there is more vehicular interaction and the drivers become cautious of the interacting traffic.

At capacity the gap increases from 2.88s to 3.09s with an increase of 0.21s under moderate rain. Heavy rain increases the gap from 3.23s to 3.91s with an increase of 0.68s. Despite there being no entry vehicle, the drivers are still more cautious of the circulating vehicles than the rainfall.

At this site the critical gap increases with an increase in rain intensity and the effect of rainfall decreases with an increase in the circulating volume to capacity ratio. The summary of the estimated critical gap under dry and rainy weather conditions is presented in table 6.18.

Table 6.18: Summary of the critical gap under dry and rainy conditions at site 02.

Weather condition	Volume to capacity ratio							
	0.1		0.5		0.9		1.0	
	Critical gap (s)							
	Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	39.25	39.28	7.40	7.40	3.84	3.89	3.40	3.44
Moderate rain	33.59	37.27	6.29	6.89	3.26	3.52	2.88	3.09
Heavy rain	37.08	45.36	6.99	8.51	3.65	4.42	3.23	3.91

6.4.3 Critical Gap Evaluation for Site 03 (Douglas Roundabout)

The recalled circulating capacity for site 03 from chapter 5 is presented in table 6.19. This is used in estimating the critical gap at varying circulating volume to capacity ratios under dry and rainy conditions.

Table 6.19: Circulating capacity during dry and rainy conditions at site 03.

Weather condition	Qc (pce/h)		Qc (pce/h/lane)	
	Dry	Rain	Dry	Rain
Light rain	1810	1806	905	903
Moderate rain	2110	1896	1055	948
Heavy rain	1914	1563	957	782

Note: Q_c is circulating capacity.

The results of the estimated critical gap under dry and rainy weather conditions at this site show that the effect of rainfall on the critical gap follows the same pattern with sites 01 and 02. This is because the critical gap increases under rainfall, but the effect decreases with an increase in the circulating volume to capacity ratio. For example, using heavy rain conditions, the critical gap increases from 30.03s to 32.14s with an increase of 2.11s at a 0.1 volume to capacity ratio. It increases from 5.60s to 5.85s with an increase of 0.25s at a volume to capacity ratio of 0.5. It increases from 2.88s to 2.57s with an increase of 0.11s at a volume to capacity ratio of 0.9. The effect of heavy rainfall is more pronounced on the critical gap at free flow rate of 0.1 volume to capacity ratio, and it decreases with an increase in the circulating volume to capacity ratio. At circulating capacity, the gap increases from 2.57s to 2.63s with an increase of 0.06s. Light and moderate rain effect on the critical gap follows the same pattern at this site. Light rain has the highest effect at this site, this is because critical gap depends on the traffic flow rate, and the circulating capacity under light rain is lower than under moderate rainfall which generates a greater headway and gap. At this site, the critical gap increases irrespective of rain intensity and the rainfall effect decreases with an increase in the volume to capacity ratio irrespective of the rain intensity. The summary of the estimated critical gap is presented in table 6.20.

Table 6.20: Summary of the critical gap under dry and rainy conditions at site 03.

Weather condition	Volume to capacity ratio							
	0.1		0.5		0.9		1.0	
	Critical gap (s)							
	Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	36.30	40.31	6.85	7.59	3.58	4.01	3.21	3.56
Moderate rain	44.38	46.39	8.47	8.72	4.48	4.59	4.01	4.08
Heavy rain	30.03	32.14	5.60	5.85	2.88	2.99	2.57	2.63

6.4.4 Critical Gap Evaluation for Site 04 (Gateway Roundabout)

The circulating capacity is recalled from chapter 5 and presented in table 6.21 for estimation of the critical gap using the procedure in subsection 6.4.1.

Table 6.21: Circulating capacity during dry and rainy conditions at site 04.

Weather condition	Qc (pce/h)		Qc (pce/h/lane)	
	Dry	Rain	Dry	Rain
Light rain	1594	1536	797	798
Moderate rain	1626	1464	813	732
Heavy rain	1717	1460	859	730

Note: Q_c is circulating capacity

The results of the estimated critical gap show that the critical gap increases with an increase in rain intensity. At a 0.1 volume to capacity ratio, light rain increases the critical gap from 44.73s to 46.29s with an increase of 1.56s. Moderate rain increases the critical gap from 43.84s to 48.55s with an increase of 4.71s. The increase under heavy rain is from 41.49s to 48.66s with an increase of 7.17s. The effect of rainfall irrespective of the rain intensity is significant at free flow rate. Heavy rain has the highest effect at this site because as the rain intensity increases, the circulating drivers respond to the rainfall effect such as an increase in impaired visibility. They also become cautious of merging with entry traffic. Hence, they keep a greater distance from the leading vehicle and this increases the critical gap. At a volume to capacity ratio of 0.5, the critical gap increases from 8.59s to 8.79s with an increase of 0.2s under light rain. The increase is from 8.41s to 9.21s with an increase of 0.8s under moderate rain. Heavy rain causes an increase from 7.94s to 9.21s with an increase of 1.27s. As the circulating flow rate increases, the effect of rainfall decreases. At a 0.9 volume to capacity ratio, light rain increases the critical gap from 4.57s to 4.62s with an increase of 0.05s. Moderate rain causes increases from 4.48s to 4.84s with an increase of 0.36s. While heavy rain causes an increase from 3.75s to 4.27s with an increase of 0.52s. At capacity, the volume to capacity ratio is 1.0, and the gap increases from 4.07s to 4.10s with an increase of 0.03s under light rain. Under moderate rain, it increases from 3.98s to 4.29s with an increase of 0.31s. Heavy rain increases the gap from 3.75s to 4.27s with an increase of 0.52s. At this site the critical gap increases with increases in rain intensity and the rain effect diminishes as the circulating flow rate increases. The summary results of the estimated critical gap at varying volume to capacity ratios under dry and rainy weather conditions is presented in table 6.22.

Table 6.22: Summary of the critical gap under dry and rainy conditions at site 04.

Weather condition	Volume to capacity ratio							
	0.1		0.5		0.9		1.0	
	Critical gap (s)							
	Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	44.73	46.29	8.59	8.79	4.57	4.62	4.07	4.10
Moderate rain	43.84	48.55	8.41	9.21	4.48	4.84	3.98	4.29
Heavy rain	41.49	48.66	7.94	9.21	4.22	4.82	3.75	4.27

6.5 Time Headway Implications

The time headway always occurs at both the entry and circulating traffic stream. The vehicle interaction between the entry and circulating traffic flow rate determines the functionality of the roundabout. The time headway of concern at the roundabout is the follow-up time at the entry roadway which is the time between two successive vehicles that accept the same gap within the circulating traffic. The critical gap is the safe gap within the circulating traffic stream for the entry vehicle to drive into the roundabout. These time parameters were evaluated in sections 6.3 and 6.4.

The summary of the follow-up time at volume to capacity ratios of 0.1, 0.5, 0.9 and 1.0 during the dry and three categories of rainy conditions for the four sites as estimated in section 6.3 are put together and rearranged under each rain category class of light, moderate and heavy rain. The results show that the rainfall effect on follow-up time follows the same pattern in all the sites, as the follow-up time increases irrespective of rain intensity. The values of the follow-up time are not the same under the same weather conditions at all the sites because follow-up time is not a fixed value but dynamic. Taking an average of the follow-up time during dry and rainy conditions, the results show that at a free flow rate of entry volume to capacity ratio of 1.0, the light rain increases follow-up time from 37.34s to 38.75s with an increase of 2.63s or 6.93%. Moderate rain increases the follow-up time from 37.58s to 41.82s with an increase of 4.24s or 11.29%. Heavy rain causes an increase from 40.93s to 47.43s with an increase of 7.04s or 17.42%. At a volume to capacity ratio of 0.5, light rain increases the follow-up time from 7.58s to 7.96s causing an increase of 0.38s or 4.95%. Moderate rain causes an increase from 7.52s to 8.37s with an increase of 0.86s or 11.31%. Heavy rain causes an increase of

1.38s or 17.01% by increasing the follow-up time from 8.10s to 9.48s. At a volume to capacity ratio of 0.9, the light rain increases the follow-up time from 4.21s to 4.42s with an increase of 0.21s or 4.99%. Moderate rain increases the follow-up time from 4.18s to 4.65s with an increase of 0.47s or 11.26%. Heavy rain causes an increase of 0.77s by increasing the critical gap from 4.50s to 5.27s or 17.11%.

The effect of rainfall is pronounced on follow-up time at free flow rate because the entry drivers react mainly to the rainfall. The circulating flow rate is always high at a low entry flow rate which makes the entry drivers become more cautious of merging with the circulating traffic under rainfall. There is a general increase in the follow-up time under rainfall because the drivers keep a greater distance from the leading vehicles because of the rainfall. When the leading vehicle accepts the critical gap for merging and driving into the roundabout, the follow-up vehicle will contend with the leading vehicle and the rainfall.

When the entry roadway is operating at capacity, the volume to capacity ratio is 1.0, the follow-up time increases from 3.78s to 3.88s with an increase of 0.02s under light rain, 3.85s to 4.28s with an increase of 0.42s under moderate rain, and 4.05s to 4.74s with an increase of 0.69s under heavy rain.

This summary also shows that the follow-up time increases with an increase in rain intensity, but the rain effect diminishes with an increase in rain intensity, and when the entry roadway is operating at capacity, the follow-up time difference under rainfall and dry condition are insignificant. This is because the optimum entry flow rate at capacity cannot be separated from the rainfall effect. Although at a volume to capacity ratio of 0.9, there is a slight improvement and the effect increases when the volume to capacity ratio is set to 0.5.

It is therefore correct to state that the follow-up time increases with an increase in rain intensity and the effect diminishes with an increase in volume to capacity ratio. At optimum entry flow rate, the influence of rainfall on follow-up time is nullified. Thereafter, the optimum entry traffic conditions control the follow-up time. The summary is presented in table 6.23.

Table 6.23: Summary of the follow-up time during dry and rainy conditions at all the sites.

Weather condition	Site	Volume to capacity ratio							
		0.1		0.5		0.9		1	
		Follow-up time (s)							
		Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	01	35.12	37.06	7.02	7.41	3.90	4.12	3.51	3.71
	02	36.40	39.49	7.28	7.29	4.04	4.05	3.64	3.65
	03	42.81	46.88	8.56	9.38	4.76	5.21	4.28	3.99
	04	37.34	38.75	7.47	7.75	4.15	4.31	3.73	3.88
Moderate rain	01	34.22	41.24	6.84	8.25	3.80	4.58	3.42	4.12
	02	38.96	43.35	7.79	8.67	4.33	4.82	3.90	4.33
	03	38.63	39.91	7.73	7.98	4.29	4.43	3.86	3.99
	04	38.50	42.78	7.70	8.56	4.28	4.75	3.85	4.28
Heavy rain	01	36.70	42.58	7.34	8.52	4.08	4.73	3.67	4.26
	02	37.38	45.77	7.48	9.12	4.15	5.09	3.74	4.58
	03	47.21	53.49	9.44	10.70	5.25	5.94	4.72	5.33
	04	40.27	47.87	8.14	9.57	4.52	5.32	4.07	4.79

The summary of the critical gap estimated at volume to capacity ratios of 0.1, 0.5, 0.9 and 1.0 under dry and rainy conditions at the four sites in section 6.4 are grouped according to the rain categories. The summary of the critical gap at all sites shows that the critical gap in dry weather conditions at all sites is lower than under rainfall, irrespective of the intensity. The reason is because under rainfall, the circulating vehicles keep a greater following distance because of the rainfall and are more cautious of merging entry vehicles. Despite there being a continuous flow rate, the drivers are still cautious because of the rainfall. Taking the average of the critical gap, at a free circulating flow rate of a volume to capacity ratio of 0.1, the critical gap increases from 38.43s to 40.26s with an increase of 1.84s or 4.78%. The moderate rain causes an increase from 38.99s to 43.32s with an increase of 4.34s or 11.12%. Under heavy rain the increase is from 35.21s to 40.82s with an increase of 5.62s or 15.96%. The rainfall effect is more pronounced at circulating free flow rate. At a volume to capacity ratio of 0.5, the critical gap increases from 7.3s to 7.6s with an increase of 0.3s or 4.11% under light rain. It increases from 7.42s to 8.15s with an increase of 0.73s or 9.87% under moderate rain, and 6.61s to 7.62s with an increase of 1.10s or 15.27% under heavy rain. At a 0.9 volume to capacity ratio, the critical gap increases from 3.84s to 4.00s with an increase of 0.16s or 4.17% under light rain. It increases from 3.92s to 4.28s with an increase of 0.33s or 8.49% under moderate rain. Under heavy rain, it increases from 3.49s to 3.95s with an increase of 0.46s or 13.03%. The critical gap increases with an increase in rain intensity

At circulating capacity, the volume to capacity ratio is 1.0, the gap decreases from 3.57s to 3.54s with a difference of 0.03s or 0.77%, this difference is inconsequential, and it shows that at peak traffic flow rate, light rain has no effect, but the circulating flow rate takes control of the gap. Moderate rain causes an increase from 3.49s to 3.77s with an increase of 0.27s or 6.85%, and heavy rain causes the increase from 3.11s to 3.48s with an increase of 0.37s or 11.75%. The summary is presented in table 6.24.

Table 6.24: Summary of the critical gap during dry and rainy conditions at all the sites.

Weather condition	Site	Volume to capacity ratio							
		0.1		0.5		0.9		1	
		Critical gap (s)							
		Dry	Rain	Dry	Rain	Dry	Rain	Dry	Rain
Light rain	01	33.42	35.16	6.36	6.62	3.35	3.45	2.98	3.05
	02	39.25	39.29	7.40	7.40	3.84	3.87	3.40	3.44
	03	36.30	40.31	6.85	7.59	3.58	4.01	3.21	3.56
	04	44.73	46.29	8.59	8.79	4.57	4.65	4.67	4.10
Moderate rain	01	34.13	41.07	6.51	7.77	3.44	4.04	3.10	3.60
	02	33.59	37.27	6.29	6.89	3.26	3.52	2.88	3.09
	03	44.38	46.39	8.47	8.72	4.48	4.59	4.01	4.08
	04	43.84	48.55	8.41	9.21	4.48	4.84	3.98	4.29
Heavy rain	01	32.22	37.19	6.12	6.92	3.22	3.56	2.90	3.10
	02	37.08	45.36	6.79	8.51	3.65	4.42	3.23	3.91
	03	30.03	32.14	5.60	5.85	2.88	2.99	2.57	2.63
	04	41.49	48.60	7.94	9.21	4.22	4.82	3.75	4.27

With the evidence in the summary of the follow-up time and critical gap, it shows that irrespective of the priority rule at the roundabout, the circulating and entry drivers are constrained by rainfall. The drivers keep a distance from the leading vehicle and take caution when merging. This shows that neither the entry nor the circulating drivers have an undue advantage over the other under rainfall. It is also correct to state that the critical gap and follow-up time increases with an increase in rain intensity. At optimum traffic flow rate, the rainfall effect becomes minimal and the traffic flow rate takes over the control of the follow-up time.

While the gap at the optimum circulating flow rate cannot be taken as the critical gap because there is no entry flow rate when the circulating roadway is operating at capacity.

6.6 Summary

The follow-up and critical gap changes during rainfall were investigated in this chapter. The rainy weather conditions were classified into light, moderate and heavy rain. The follow-up time headway was estimated using the entry capacity which was estimated using the empirical method. The headway at capacity showed a lack of evidence to determine the effect of rainfall on the follow-up time because of the peak traffic effect. The follow-up time was estimated at varying entry traffic flow rates under dry and rainy conditions using the volume to capacity ratio. The follow-up time increased irrespective of the volume capacity ratio and rainfall intensity. The rainfall effect was well pronounced at free flow rate and the effect reduced with an increase in the volume capacity ratio. At peak flow rate when the roundabout entry is operating at capacity, the rainfall effect has an inconsequential effect on the follow-up time. At site one, the moderate rain had a higher effect on the follow-up time headway because of the high percentage of medium and heavy circulating vehicles. This was evidence that the follow-up time reacts to the circulating vehicle composition. The follow-up time increased with an increase in rain intensity, but the effect diminished as the entry flow rate increased and it became insignificant at peak entry flow rate.

The critical headway was estimated in this chapter using the estimated circulating capacity. The critical gap was estimated by subtracting the vehicle length from the headway. The estimated gap at capacity was not adequate in determining the rainfall effect on the critical gap because at peak circulating flow rate, there is no interaction between the entry and circulating flow rate. The critical gap was estimated at varying circulating volume capacity ratios. The critical gap increased with increases in rainfall at all the site. Heavy rainfall had the highest effect. The effect of rainfall diminished as the circulating flow rate increased, and at peak the rainfall effect was insignificant.

CHAPTER 7

CONCLUSION

7.1 Overview

The study reports the investigation of the impact of rainfall on the quality of service delivery at multilane roundabouts and their implications for time headway in South Africa. It was based on the hypothesis that rainfall, irrespective of intensity, will have an adverse effect on the quality of service delivery and time headway. The aim behind this exercise was to establish whether the quality of road service can be sustained in the presence of rainfall and the relationship between the two variables. The objectives were to:

- i. develop a quality of service criterial table for multilane roundabouts that would be used to assess roundabout performance under dry daylight and rainy conditions,
- ii. estimate the entry delay for multilane roundabouts under dry daylight and rainy conditions,
- iii. determine the quality of service for dry and rainy conditions from the criterial table developed in subsection i and to compare the outcomes,
- iv. evaluate time headways under dry daylight and rainy weather conditions and to compare the outcomes.

Within the purview of the study objectives, rainfall was classified into three categories; light rainfall with an intensity of less than 2.5mm/h, moderate rainfall with an intensity of between 2.5mm/h and 10mm/hr, and heavy rainfall with an intensity of between 10mm/h and 50mm/h. It portrayed the amount of rain that has occurred at locations during observation periods. The study postulated that the quality of service can be divided into two classes (structural and functional). Structural quality of service deals with the generalised fixed roundabouts' infrastructure like the pavement, drainage, road furniture, markings and signs among others. Whereas the functional quality of service deals with the flow rate entities and control system.

The study focused on the functional quality of service (FQS). Unlike the HCM level of service approach, where a single parameter (delay) is employed to determine service delivery, FQS has two key parameters, delay and reserve capacity, among others. FQS is premised on the concept that quality of service assessment has to take cognisance of road users' and providers' perceptions of quality.

Rainy conditions affect traffic flow rate in a variety of ways that include poor visibility, aquaplaning, poor road surface friction, flooding, and pavement structural damage among others. Rainfall at roundabouts may cause drivers to reduce their vehicle speed, maintain the same carriageway lane, reserve capacity and affect time headway (Cools, M. et. al, 2010, Alhassan, H. & Ben-Edigbe, J. 2011, Ben-Edigbe, J., et. al. 2013). These in turn could promote poor road service and heighten the probability of accident risk. Both South African passenger car equivalent values and modified passenger car equivalent values based on empirical findings were used in turn. Statistical tests confirmed that no significant difference exists between the two values hence the standard South African values were used. A two-stage (assessment criteria and performance measurement) quality of service procedural framework was developed. Guided by the study objectives, roundabouts were surveyed, and their empirical results investigated considering the evidence obtained from the examination of the survey data. The analytical findings for dry and rainfall weather conditions were compared. In passing, it was observed that rainfall affected drivers' visibility irrespective of vehicle type, and it is reasonable to suggest that rainy conditions have an adverse effect on driving conditions irrespective of vehicle type. Based on the synthesis of evidence obtained from the relationship between quality of service and rainfall, it is correct to conclude that there is a significant change in the quality of service delivery and time headways. In summary the study showed that:

- i. a criterial table can be constructed with delay and reserve capacity among others
- ii. quality of service reduction would result from rainfall
- iii. time headway variability would result from rainfall
- iv. heavy rainfall is a significant contributor to poor service delivery

Considering the discussion so far, the remainder of this chapter is organized into five sections.

Section 7.2: Findings based on rainfall intensities are summarized.

Section 7.3: Findings from quality of service criteria table development are summarized.

Section 7.4: Findings from quality of service delivery analyses are summarized.

Section 7.5: Synthesis of evidence from quality of service reduction are summarized.

Section 7.6: The way forward is presented.

7.2 Findings Based on Rainfall Intensities

In South Africa, Durban city has the highest rainfall intensity and frequency with an average yearly precipitation of 828mm compared to the South African yearly average of 450mm. Rainfall portrays the amount of rain that has occurred at locations during observation periods. It was classified into three categories; light rainfall with an intensity of less than 2.5mm/h, moderate rainfall with an intensity of between 2.5mm/h and 10mm/hr, and heavy rainfall with an intensity of between 10mm/h and 50mm/h. It portrays the amount of rain that has occurred at the surveyed sites during the observation periods.

Light, moderate and heavy rain reduced the service delivery at site 01 and 04 roundabouts from B to C, whereas moderate and heavy rain reduced the service delivery at site 02 from C to D. Rainfall reduced the service delivery at site 03 from C to D bearing in mind that light rain had an insignificant effect on the service delivery at site 02 whilst moderate rain had an insignificant effect on the service delivery at site 03. It is often difficult to know with precision the exact intensity of rainfall, for example rainfall on the fringe could have been classified either way. It is possible that some moderate rainfalls are indeed light rainfall, and some are heavy rainfall.

Nevertheless, it is equally important to bear in mind that rainfall changes intermittently, probably explaining the variation in service delivery distribution at the surveyed sites. In the study the quality of service reduction emanating from heavy rainfall intensity was prominent whereas those from light and moderate rainfalls sometimes overlapped. In any case, quality of service and time headway changed significantly due to rainfall at all surveyed sites.

Entry flow rate distribution fluctuated during dry weather, suggesting that drivers were not constrained by rainfall hence could choose suitable gaps in the circulating traffic stream. Whereas the entry flow rate distribution was nearly flat during rainfall, suggesting that drivers were constrained by the rainy conditions. During rainfall, entry and circulating traffic streams' time headways were affected. It was observed that time headway changes were gradual under rainy conditions and fluctuated during dry weather.

Given rainy conditions, reserve capacity values were reduced, consequently control delay time and queue lengths increased. It would have been erroneous if South African passenger car equivalent values were kept without modification. Consequently, South African passenger car equivalent values were modified for study conditions, however, statistical tests suggested that passenger car equivalent value modifications would have a negligible effect on the study outcomes. Therefore, the South African passenger car equivalent values were used. In summary, the study has shown conclusively that rainfall, irrespective of its intensity, affects the effectiveness of traffic stream quality of service delivery.

7.3 Findings from Quality of Service Criteria Development

In this study, it was argued that the level of service is not the same as the quality of service and cannot be used interchangeably. Level of service considers a single parameter whilst quality of service considers two or more parameters that represent road users' and providers' perceptions of service delivery.

A quality of service criterial table was developed in this thesis for ranking service delivery under dry and rainy weather conditions. The table was divided into five grades, being A to F where A is the best service delivery and F is the worst service delivery, where vehicles move at lockstep with the lead vehicle. Typically, for grade A delay~11s, reserve capacity~0.6; for grade B delay~≤14s, reserve capacity~0.4; for grade C delay~≤22s, reserve capacity~0.2; for grade D delay~≤31s, reserve capacity~0.1; for grade E delay~≤49s, reserve capacity~<0.1; for grade F delay~>49s. Grade D is the threshold that serves as a warning to traffic management that the roundabout is operating close to the capacity. From this study, the estimated delays were not significantly different from the delays stated in HCM 2010, however HCM 2010 and previous research studies often relied on delay for service delivery assessment.

This study introduced reserve capacity instead of saturation flow rate as the second criterion that depicts road providers' perception of service delivery. Reserve capacity is an important factor when assessing the effectiveness of roundabout performance. It gives the traffic management team a sense of how much spare capacity the roundabout under observation can deliver, unlike the degree of saturation that merely states the number of vehicles in operation relative to the entry capacity. The inclusion of reserve capacity in the criterial table as one of the quality of service's parameters is a major finding and a clear departure from the HCM 2010 level of service prescriptions that rely solely on delays.

Note that dry weather peak performances at each surveyed site were used to construct their criterial tables because each site had its peculiar environmental and traffic conditions. Furthermore, it allowed each site to be assessed uniquely against their standards. Hence the criterial table varied from site to site. This is a unique development and it is probably the first time that the quality of road service can be assessed against its own set of performance criteria. *It is a clear departure from HCM and SIDRA roundabout performance assessment criterial tables. The departure makes both the road provider and user to be considered in the assessment of roundabout performance. Nevertheless, it was observed that the assessment criterial tables for all sites had very close values because of the difference in traffic, geometry and environmental conditions. Though, these tables can be combined into one table without having much deviation in the lower and upper boundaries of the FQS parameters.*

7.4 Findings from Quality of Service Delivery Analyses

Results of the quality of service delivery at roundabouts can best be summarised as an increase in rainfall intensities relative to a decrease in service delivery. Observed volume to capacity ratios at all sites ranged from 60 to 70 per cent, suggesting that traffic flow rates were not at peak. Two performance measures were used; entry delay and reserve capacity. Each performance measure acted not only as a quality check, but also for checking the trend outcomes. At all sites, both the entry delay and reserve capacity analytical methods showed similar trends in service delivery reduction although not by the same percentage. Interestingly, follow-up time and critical gap reduced with the increase of rainfall intensities, thus suggesting that drivers were constrained by rainfall conditions. That trend was similar for all sites.

For delay and queue length, the generalized delay increase was between 10.26 % and 31.44 % and a queue length increase of 33.33 % under rainfall. However, light rain had the lowest delay increase of 0.19s, 2.22s, and 3.38s at sites 02, 03, and 04 respectively. Heavy rain caused the increase of 7.19s, 9.38s, and 3.38s delays at site 02, 03, and 04 respectively.

Traffic flow rates under dry condition mostly fell in FQS B and C. Generally, when it rains the FQS reduced from B to C at sites 01 and 04, from C to D at sites 02 and 03 as shown on Table 7.1.

Table 7.1: Summarised effect of rainfall on Functional Quality of service.

Site	FQS	
	Dry	Rainfall
01	B	C
02	C	D
03	C	D
04	B	C

The estimated maximum flow rate rates at all investigated sites were generally lower at off-peak than peak travel, suggesting that all traffic flow rate data used for analysis occurred at off-peak periods. Light, moderate and heavy rain caused a reduction in the reserved capacity at all the surveyed sites. The summary of the effect showed that light rain caused a 12.5 % reduction, moderate and heavy rain caused a 17 % reduction respectively. In summary, the findings from this study indicate that the negative impact of rainfall on the quality of service delivery at roundabouts in Durban, South Africa is significant.

7.5 Synthesis of Evidence from Quality of Service Reduction

The study has shown that rainfall, irrespective of its intensity, affects the quality of service delivery at roundabouts in Durban, South Africa. **Quality of service, which is same as functional quality of service, is made up of two key parameters**, delay and reserve capacity. Reserve capacity was used in the thesis as a proxy for the road providers' perception of service whilst delay was used as a proxy for the road users' perception of service delivery. It is obvious that a reduction in quality of service delivery will trigger an increase in travel time and by extension time headway. All the model equations used in the thesis were tested statistically for

fitness. Regression techniques were employed for the development of a capacity model that relates to rainfall intensity. Both linear and exponential techniques were used in this study, but the exponential method was not adopted because of its inability to provide analysis under very low entry flow rates. The ensuing analytical findings were compared and discussed. The study has shown that rainfall has an influence on the functional service delivery at roundabouts and there is no evidence to suggest an undue advantage to either entry or circulating traffic flow rates. Based on the synthesis of evidence from the study, it is correct to state that the significant entry and circulating capacity which was lost at all surveyed sites resulted from rainfall. At all surveyed sites, the reserve capacity at multilane roundabouts in Durban, South Africa decreased during rainfall and it is correct to state that reserved capacity reduction at all surveyed sites were triggered by rainfall.

The effect of rainfall intensity on time headway was investigated in this study. Follow-up time and critical gaps were the key parameters used in the study. Follow-up time increased during rainy conditions at all the surveyed sites. Heavy rainfall had the greatest effect on the follow-up time and critical gap. So, it is also correct to suggest that the time headways are anomalous because the time differentials are inconsistent with rainfall intensity. Time headway differences became smaller in relation to rainy conditions and anomalous under heavy rainfall, thus suggesting that drivers are more cautious. However, once the degree of saturation threshold mark of 0.9 was reached and surpassed, the effect of rainfall as the sole traffic flow rate disturbance gradually diminished. It is also valid to conclude that the effect of rain alone cannot account for peak travel conditions and held responsible for time headway differentials when traffic flow rate is operating at peak without taking into account peak travel conditions.

7.6 Recommendations

The study has shown that rainfall, irrespective of intensity, has a significant impact on the quality of service at multilane roundabouts. It has also shown that the quality of service delivery is made up of at least two principal parameters; road providers' and users' perceptions of quality. The study has also shown that passenger car equivalent values have dynamic properties and that roundabout entry capacity is not static.

It is suggested that the level of service methodology for roundabouts and the passenger car equivalent values prescribed by the South African National Roads Agency Limited (SANRAL) be revisited. The task of effectively managing and operating a roundabout system has never been easy least of all under rainy conditions, nevertheless culling methodology from US HCM 2010 would make the task of management even harder. The US Highway Capacity Manual (HCM) is the most quoted and referred capacity manual in the transportation community worldwide. It was first developed in 1950. Since then, it has undergone significant improvements with major restructuring and rewrites in 1965, 1985, 2000 and 2010. **The models developed and the roundabout FQS criterial table developed in this research can form part of South African HCM when it is developed and hence the need to take a serious look at the development of a South African Highway Capacity Manual (SAHCM) without delay.** It can be argued that the depth of understanding and the experience of a systematic objective approach to roundabout operations is more relevant than borrowed methodology that may be inappropriate socially, economically, as well as culturally. It is often the case with borrowed methodology that the borrower would have to catch up with the lender all the time.

In South Africa, where the capability to manage roundabout systems is still developing, assistance is generally needed. It is believed that a successful outcome will require a fusion of foreign technology, investments and local inputs. The study believes that of far more value to South Africa is an understanding and experience of a systematic approach to variable highway traffic problem-solving than the potential availability of the US Highway Capacity Manual and their problem-solving approach, because of the diversity in driving culture and priority of needs. It is accepted that the US Highway Capacity Manual can be used as a development tool; however, over-reliance on it would be inappropriate to the needs of this country. At the present time, South Africa has no highway capacity manual, its reliance on the US Highway Capacity Manual is near total. This would make traffic management during rainy conditions an audacious task to carry out. At the time of survey, **there is no evidence of coordinated highway traffic data that takes cognizance of the rainfall in South Africa and many developing countries, hence this study makes a significant contribution to the study of rainfall impacts on roundabout operation.** Consequently, poor quality of service would become unavoidable. In any case, this study gave insight into some of the problems inherent in driving under rainy conditions in South Africa.

The conclusions drawn in the study are relevant to multilane roundabouts' traffic streams in South Africa and can be modified for use on other roundabouts. Currently very little is known about drivers' behaviour in South Africa under rainfall conditions and it would be useful if research would be undertaken in this area. **There is a need to comprehensively investigate the effects of rainfall intensity on taxi drivers' behaviour because the Taxi drivers have driving behaviour different from other drivers in South Africa. As this will give insight into behaviour of taxi drivers under rainfall which could be a useful traffic management tool.** There is further concern about the problem of aquaplaning under rainy conditions. This is also an area where research is needed to establish the effect of rainfall on generalised drivers' behaviour in South Africa. Future research should be carried out on singular and multilane lane roundabout entry capacity estimation based on entry speed and traffic volume and the findings compared with other **known** capacity estimation methods. In closing, it is recommended that future studies be conducted to assess the perception of road users' quality of service. It is affirmed that further research works on road users' and providers' perceptions would allow for a quality of road service index to be developed.

Functional quality of service is a management issue because it deals with the traffic operations at roundabouts. Likewise, the rainfall is more of policy and management issue. This indicates that the development, implementation, effectiveness and improvement of the roundabout functional service delivery measures under rainfall in South Africa, is the way forward.

REFERENCES

- AGARWAL, M., MAZE, T. H. & SOULEYRETTE, R. Impacts of weather on urban freeway traffic flow rate characteristics and facility capacity. Proceedings of the 2005 mid-continent transportation research symposium, 2005. 18-19.
- AKÇELIK, R. A roundabout case study comparing capacity estimates from alternative analytical models. 2nd Urban Street Symposium, Anaheim, California, USA, 2003. Citeseer, 28-30.
- AKÇELIK, R. Roundabout model calibration issues and a case study. TRB National Roundabout Conference, Vail, Colorado, USA, 2005. 25.
- AKÇELIK, R. & CHUNG, E. 1994a. Calibration of the bunched exponential distribution of arrival headways. *Road and Transport Research*, 3, 42-59.
- AKÇELIK, R. & CHUNG, E. Traffic performance models for unsignalised intersections and fixed-time signals. Proceedings of the Second International Symposium on Highway Capacity, Sydney, 1994b. 21-50.
- AKIN, D., SISIPIKU, V. P. & SKABARDONIS, A. 2011. Impacts of weather on traffic flow rate characteristics of urban freeways in Istanbul. *Procedia-Social and Behavioral Sciences*, 16, 89-99.
- AKÇELIK, R. 2009. Evaluating Roundabout Capacity, Level of Service and Performance.
- AL-GHAMDI, A. 1999. Entering headway for through movements at urban signalized intersections. *Transportation Research Record: Journal of the Transportation Research Board*, 42-47.
- AL-MADANI, H. M. & PRATELLI, A. 2014. Modeling and calibrating capacity of large roundabouts. *International journal of sustainable development and planning*, 9, 54-73.
- AL-OMARI, B., AL-MASAEID, H. & AL-SHAWABKAH, Y. 2004. Development of a delay model for roundabouts in Jordan. *Journal of transportation engineering*, 130, 76-82.
- ALHASSAN, H. & BEN-EDIGBE, J. 2011. Effect of Rainfall Intensity Variability on Highway Capacity. *European journal of scientific research*, 49, 123-129.

- AMS 2018. Rain in Glossary of meteorology, AMS (America Meteorological Society), MA, USA.
- ANDREY, J. & OLLEY, R. 1990. The relationship between weather and road safety: past and future research directions. *Climatological Bulletin*, 24, 123-127.
- ASHALATHA, R. & CHANDRA, S. 2011. Critical gap through clearing behavior of drivers at unsignalised intersections. *KSCE Journal of Civil Engineering*, 15, 1427-1434.
- ASHWORTH, R. 1968. A note on the selection of gap acceptance criteria for traffic simulation studies. *Transportation Research/UK/*.
- ASHWORTH, R. 1970. The analysis and interpretation of gap acceptance data. *Transportation Science*, 4, 270-280.
- BARTLETT, A., LAO, W., ZHAO, Y. & SADEK, A. W. Impact of Inclement Weather on Hourly Traffic Volumes in Buffalo, New York. Transportation Research Board 92nd Annual Meeting, 2013.
- BEN-EDIGBE, J. 2016. Estimation of Midblock Median Opening U-Turn Roadway Capacity Based on Sectioning Method. *Discrete Dynamics in Nature and Society*, 2016.
- BEN-EDIGBE, J., MASHROS, N. & RAHMAN, R. 2013. Extent of sight distance reductions caused by rainfall on single carriageway roads. *International Journal for Traffic and Transport Engineering*, 3, 291-301.
- BERGEL-HAYAT, R., DEBBARH, M., ANTONIOU, C. & YANNIS, G. 2013. Explaining the road accident risk: weather effects. *Accident Analysis & Prevention*, 60, 456-465.
- BRILON, W., GROSSMANN, M. & STUWE, B. 1991. Towards a new German guideline for capacity of unsignalized intersections. *Transportation Research Record*, 1320, 168-174.
- BRILON, W., KOENIG, R. & TROUTBECK, R. J. 1999. Useful estimation procedures for critical gaps. *Transportation Research Part A: Policy and Practice*, 33, 161-186.
- BUCKLEY, D. 1968. A semi-poisson model of traffic flow rate. *Transportation Science*, 2, 107-133.

- ÇALIŞKANELLI, P., ÖZUYSAL, M., TANYEL, S. & YAYLA, N. 2009. Comparison of different capacity models for traffic circles. *Transport*, 24, 257-264.
- CHUNG, E., OHTANI, O., WARITA, H., KUWAHARA, M. & MORITA, H. Effect of rain on travel demand and traffic accidents. *Intelligent Transportation Systems, 2005. Proceedings. 2005 IEEE, 2005. IEEE, 1080-1083.*
- CLIMTEMP 2009. *Rainfall/precipitation in Durban, South Africa* [Online]. Available: www.durban.climatemps.com/precipitation.php [Accessed 18/11/2015 2015].
- COLLINS, R. R. 2008. Evaluation of a Roundabout at a Five-Way Intersection: An Alternatives Analysis Using Micro-simulation. *M.Sc. thesis, University of Minnesota.*
- COOLS, M., MOONS, E. & WETS, G. 2010. Assessing the impact of weather on traffic intensity. *Weather, Climate, and Society*, 2, 60-68.
- CORPORATION, S. A. I., (SAIC), G. M. U., SYSTEMS, A. V. N. T. & CENTER. 2003. *Quality of service and customers satisfaction on arteril streets: Fianl report* [Online]. Available: http://ntl.bts.gov/lib/jpodocs/repts_te/13849_files/13849.pdf [Accessed 13/12/2016 2016].
- COUNTRY SURVEYOR SOCIETY (CSS), 1972. Mini roundabouts in England and Wales. Essex country council, old court, Chemsford, Essex.
- COWAN, R. J. 1975. Useful headway models. *Transportation Research*, 9, 371-375.
- DAHL, J. & LEE, C. 2012. Empirical estimation of capacity for roundabouts using adjusted gap-acceptance parameters for trucks. *Transportation Research Record: Journal of the Transportation Research Board*, 34-45.
- ERSOY, M. & ÇELIKOĞLU, H. B. 2014. Capacity analysis on multi-lane roundabouts: an evaluation with highway capacity manual 2010 capacity model.
- ETHEKWINI MUNICIPALITY. 2016. *Collection of rain and station gauge data* [Online]. Available:http://www.durban.gov.za/City_Services/engineering%20unit/Coastal_Engineering_Stormwater_Catchment_Management/Engineering_Services_Records/Documents/Rainfall_and_the_Councils_Www_Page_17_Mar_2016.pdf [Accessed 12/04/2016 2016].

- FLANNERY, A., MCLEOD, D. & PEDERSEN, N. J. 2006. Customer-based measures of level of service. *Institute of Transportation Engineers. ITE Journal*, 76, 17.
- FLORIDA-DOT 2013. Quality/Level of Service Handbook. *Tallahassee, FL. Accessed Feb, 27, 2016.*
- GATTIS, J. & LOW, S. T. 1999. Gap acceptance at atypical stop-controlled intersections. *Journal of Transportation Engineering*, 125, 201-207.
- GAZZARRI, A., MARTELLO, M., PRATELLI, A. & SOULEYRETTE, R. 2013. Gap acceptance parameters for HCM 2010 roundabout capacity model applications in Italy. *Intersections Control and Safety. Transportation Systems & Traffic Engineering. WitPress, Southampton. Boston*, 1-16.
- GOODWIN, L. C. & PISANO, P. 2003. Best practices for road weather management. *Road Weather*.
- GUO, R. & WANG, W. A study on delay of single-lane roundabout. *Advanced Forum on Transportation of China (AFTC 2011), 7th, 2011. IET*, 157-160.
- HAGRING, O. 1998. *Vehicle-vehicle Interactions at Roundabouts and their Implications for the Entry Capacity-A Methodological Study with Applications to Two-lane Roundabouts*. Lund University.
- HALE, D. K. 2015. A Case for Geometrically-Based Roundabout Capacity Equation Modeling.
- HANBALI, R. M. 1994. Economic impact of winter road maintenance on road users. *Transportation Research Record*.
- HASHIM, M. A., AND JOHNNIE BEN-EDIGBE 2012. Evaluation of Passenger Car Equivalent Values under Rainfall. *International Conference on Traffic and Transportation Engineering*, 26.
- HCM 1985. Highway Capacity Manual. *Transportation Research Board, Washington, DC*, 1, 985.
- HCM 2000. Highway capacity manual. *Washington, DC*.

- HEIDEMANN, D. & WEGMANN, H. 1997. Queueing at unsignalized intersections. *Transportation Research Part B: Methodological*, 31, 239-263.
- HORMAN, C. & TURNBULL, H. Design and analysis of roundabouts. Australian Road Research Board (ARRB) Conference, 7th, 1974, Adelaide, 1974.
- HOSTOVSKY, C., WAKEFIELD, S. & HALL, F. 2004. Freeway users' perceptions of quality of service: comparison of three groups. *Transportation Research Record: Journal of the Transportation Research Board*, 150-157.
- . *Effect of adverse weather conditions on speed-flow rate-occupancy relationships.*
- JARRAUD, M. 2008. Guide to Meteorological Instruments and Methods of Observation (WMO-No. 8). *World Meteorological Organisation: Geneva, Switzerland.*
- JENJIWATTANAKUL, T. & SANO, K. Effect of waiting time on the gap acceptance behavior of u-turning vehicles at midblock median openings. Proceedings of the Eastern Asia Society for Transportation Studies The 9th International Conference of Eastern Asia Society for Transportation Studies, 2011, 2011. Eastern Asia Society for Transportation Studies, 314-314.
- JOHNSON, M. T. 2013. Synthesis of Roundabout Geometric Capacity Measurement: Calibration and Validation to US Field Measurements.
- KAKOOZA, R., LUBOOBI, L. & MUGISHA, J. 2005. Modeling traffic flow rate and management at un-signalized, signalized and roundabout road intersections. *Journal of Mathematics and Statistics*, 1, 194-202.
- KANG, N., NAKAMURA, H. & ASANO, M. An Empirical Analysis on Critical Gap and Follow-Up Time at Roundabout Considering Geometry Effect. Proc., 46th Infrastructure Planning Conference, 2012.
- KEAY, K. & SIMMONDS, I. 2005. The association of rainfall and other weather variables with road traffic volume in Melbourne, Australia. *Accident analysis & prevention*, 37, 109-124.
- KENDAL, G. & REUTENER, I. 2014. Design and implementation of a turbo roundabout.
- KIMBER, R. 1980. The traffic capacity of roundabouts.

- KIMBER, R. & HOLLIS, E. M. 1979. Traffic queues and delays at road junctions.
- KIMBER, R., SUMMERSGILL, I. & BURROW, I. 1986. Delay processes at unsignalised junctions: the interrelation between geometric and queueing delay. *Transportation Research Part B: Methodological*, 20, 457-476.
- KNAPP, K. K. An Investigation of Volume, Safety, and Vehicle Speeds During Winter Storm Events. Ninth AASHTO/TRB Maintenance Management Conference, 2000.
- KOETSE, M. J. & RIETVELD, P. 2009. The impact of climate change and weather on transport: An overview of empirical findings. *Transportation Research Part D: Transport and Environment*, 14, 205-221.
- KOTLER, P. & ARMSTRONG, G. 2010. *Principles of marketing*, pearson education.
- KREJCIE, R. V. & MORGAN, D. W. 1970. Determining sample size for research activities. *Educational and psychological measurement*, 30, 607-610.
- KYTE, M., KENT LALL, B. & MAHFOOD, N. 1992. Empirical method to estimate the capacity and delay of the minor street approach of a two-way stop-controlled intersection. *Transportation Research Record*, 1-1.
- LLASAT, M. C. 2001. An objective classification of rainfall events on the basis of their convective features: application to rainfall intensity in the northeast of Spain. *International Journal of Climatology*, 21, 1385-1400.
- LORD-ATTIVOR, R. & JHA, M. Modeling gap acceptance and driver behavior at stop controlled (priority) intersections in developing countries. Proceedings of the American Conference on Applied Mathematics (American-Math'12), Harvard, Cambridge, USA, 2012a. 29-38.
- LORD-ATTIVOR, R. & JHA, M. Modeling gap acceptance and driver behavior at stop controlled (priority) intersections in developing countries. Proceedings of the AMERICAN CONFERENCE on APPLIED MATHEMATICS (AMERICAN-MATH'12), Harvard, Cambridge, USA, 2012b. 29-38.
- MANAGE, S., NAKAMURA, H. & SUZUKI, K. 2003. Performance analysis of roundabouts as an alternative for intersection control in Japan. *Journal of the Eastern Asia Society for Transportation Studies*, 5, 871-883.

- MARK LENTERS PE, P. 2010. HCM roundabout capacity methods and alternative capacity models. *Institute of Transportation Engineers. ITE Journal*, 80, 22.
- MASHROS, N. & BEN-EDIGBE, J. Determining the quality of highway service caused by rainfall. Proceedings of the Institution of Civil Engineers-Transport, 2014. Thomas Telford Ltd, 334-342.
- MASHROS, N., BEN-EDIGBE, J., ALHASSAN, H. M. & HASSAN, S. A. 2014. Investigating the Impact of Rainfall on Travel Speed. *Jurnal Teknologi*, 71.
- MASHROS, N., BEN-EDIGBE, J., HASSAN, S. A., HASSAN, N. A. & YUNUS, N. Z. M. 2014c. Impact of Rainfall Condition on Traffic Flow rate and Speed: A Case Study in Johor and Terengganu. *Jurnal Teknologi*, 70.
- MAY, A. D. 1990. *Traffic flow rate fundamentals*. Transportation Reserch Board, US, 476p.
- MAZE, T., AGARWAI, M. & BURCHETT, G. 2006. Whether weather matters to traffic demand, traffic safety, and traffic operations and flow rate. *Transportation research record: Journal of the transportation research board*, 170-176.
- MCBRIDE, J. C., KENNEDY, W., THUET, J., BELANGIE, M., STEWART, R., SY, C. & MCCONKIE, F. 1977. Economic impact of highway snow and ice control.
- MCSYSTEM. 2009. *Server data logging* [Online]. Available: <http://www.dbnrain.co.za/showmodels.php> [Accessed].
- MEI, M. & BULLEN, A. G. R. 1993. Lognormal distribution for high traffic flow rates. *Transportation Research Record*.
- MILLER, A. J. 1974. A note on the analysis of gap-acceptance in traffic. *Applied Statistics*, 66-73.
- MOODLEY, S. 2013. *Traffic Circles design a first in SA, creamer media Engineering news* [Online]. Available: www.engineeringnews.co.za/article/innovation-traffic-circle-a-first-in-sa-2013-97-19 [Accessed 03 2016].
- NCHRP, N. C. H. R. P. 2006. *Appendixes to NCHRP 572: Roundabouts in the United States, NCHRO web-only Document 94, Final report for NCHRP Project 3-65, USA*, Transportation Research Board.

- NIXON, W. A. 1998. The potential of friction as a tool for winter maintenance. Iowa Institute of Hydraulic Research, University of Iowa.
- NORDIANA, M. 2012. Exploring the extent of critical gap acceptance caused by rainfall in Malaysia. *ARPN Journal of Engineering and Applied Sciences*, 7, 1664-1668.
- NRMAE, N. R. M. A. E. D. 1986. Irrigation water management, an introduction to irrigation. www.fao/decrep/r4082e/r4802eo.5htm.
- OLUROTIMI, E., SOKOYA, O., OJO, J. & OWOLAWI, P. 2017. Observation of bright-band height data from TRMM-PR for satellite communication in South Africa. *Journal of Atmospheric and Solar-Terrestrial Physics*, 160, 24-33.
- OTIENO, F. & OCHIENG, G. 2004. Water management tools as a means of averting a possible water scarcity in South Africa by the year 2025. *Water Sa*, 30, 120-124.
- PANDE, A. & WOLSHON, B. 2016. *The Institute of Transportation Engineers, Traffic Engineering Handbook*, Wiley Online Library.
- PHAM, M. H., CHUNG, E., MOUZON, O. D. & DUMONT, A.-G. 2007. Season effect on traffic: A case study in Switzerland. *生産研究*, 59, 214-216.
- PHILBRICK, M. 1977. In search of a new capacity formula for conventional roundabouts.
- PREVEDOUROS, P. D. & CHANG, K. 2005. Potential effects of wet conditions on signalized intersection LOS. *Journal of transportation engineering*, 131, 898-903.
- QU, Z., DUAN, Y., SONG, X., HU, H., LIU, H. & GUAN, K. 2014. Capacity prediction model based on limited priority gap-acceptance theory at multilane roundabouts. *Mathematical Problems in Engineering*, 2014.
- RAFF, M. S. 1950. A volume warrant for urban stop signs.
- RAHMI AKCELIK 2009. Evaluating Roundabout Capacity, Level of Service and Performance, A paper presentation at ITE, USA.
- ROBINSON, B. W., RODEGERDTS, L., SCARBOROUGH, W., KITTELSON, W., TROUTBECK, R., BRILON, W., BONDZIO, L., COURAGE, K., KYTE, M. & MASON, J. 2000. Roundabouts: An informational guide.

- Robert, Z.. 2014. Roundabout as an Effective Tool of Traffic Management. *Journal of Sustainable Architecture and Civil Engineering*.
- ROBINSON, B. W. & RODEGERDTS, L. A. Capacity and performance of roundabouts: A summary of recommendations in the FHWA roundabout guide. Proc., 4th Int. Symp. on Highway Capacity, 2000. National Research Council, 422-433.
- RODEGERDTS, L., BLOGG, M., WEMPLE, E., MYERS, E., KYTE, M., DIXON, M., LIST, G., FLANNERY, A., TROUTBECK, R. & BRILON, W. 2007. Nchrp Web-Only Document 94-Appendixes to Nchrp 572: Roundabouts in the United States." : Transportation Research Board. *Washington, DC*.
- RODEGERDTS, L., BLOGG, M., WEMPLE, E., MYERS, E., KYTE, M., DIXON, M., LIST, G., FLANNERY, A., TROUTBECK, R. & BRILLION, W. 2007. "NCHRP Web-Only Document 94- 572: Roundabouts in the US", Transportation Research Board. Washington, DC.2007.
- RODEGERDTS, L. A. 2010. *Roundabouts: An informational guide*, Transportation Research Board.
- ROESS, R. P. & PRASSAS, E. S. 2014. The Fundamental Concept of Level of Service. *The Highway Capacity Manual: A Conceptual and Research History*. Springer.
- RUSSELL, E. & RYS, M. 2000. Modeling traffic flow rates and capacities at roundabout. MBTC.
- SAKAI, T., YAMADA-KAWAI, K., MATSUMOTO, H. & UCHIDA, T. 2011. New measure of the level of service for basic expressway segments incorporating customer satisfaction. *Procedia-Social and Behavioral Sciences*, 16, 57-68.
- SALTER, R. J. 1989. *Highway traffic analysis and design*, Springer.
- SANRAL 2011. Geometric design guidelines. South African National Roads Agency Limited Pretoria.
- SEGUIN, E. 1981. *Passenger Car Equivalents on Urban Freeways*, U.S. Department of Transportation.

- SHALINI, K. & KUMAR, B. 2014. Estimation of the Passenger Car Equivalent: A Review. *International Journal of Emerging Technology and Advanced Engineering*, 4.
- SHI, L., CHENG, Y., JIN, J., RAN, B. & CHEN, X. 2011. Effects of rainfall and environmental factors on traffic flow rate characteristics on urban freeway.
- SIEGLOCH, W. 1973. Die leistungsermittlung an knotenpunkten ohne lichtsignalsteuerung (Capacity Calculations for Unsignalized Intersections) *STRASSENBAU U STRASSENVERKEHRSTECH.*
- SISIOPIKU, V. P. & OH, H.-U. 2001. Evaluation of roundabout performance using SIDRA. *Journal of Transportation Engineering*, 127, 143-150.
- SMITH, B. L., BYRNE, K. G., COPPERMAN, R. B., HENNESSY, S. M. & GOODALL, N. J. An investigation into the impact of rainfall on freeway traffic flow rate. 83rd annual meeting of the Transportation Research Board, Washington DC, 2004.
- SOFIA, A. P. D. G. G., ISRAA, A. H. A.-H. E. & AL-HAYDARI, S. 2012. Development of Delay Models for Roundabouts. *Journal of Engineering and Development*, 16.
- SULLIVAN, D. & TROUTBECK, R. 1994. The use of Cowan's M3 headway distribution for modelling urban traffic flow rate. *Traffic engineering & control*, 35.
- TANNER, J. 1962. A theoretical analysis of delays at an uncontrolled intersection. *Biometrika*, 49, 163-170.
- TANNER, J. 1967. The capacity of an uncontrolled intersection. *Biometrika*, 54, 657-658.
- TRACZ, M., CHODUR, J. & GACA, S. 2004. *Metoda obliczania przepustowości rond: instrukcja obliczania*, Wydaw. PiT.
- TRB 2010. Highway Capacity Manual. *Transportation Research Board, National Research Council, Washington, DC.*
- TROUTBECK, R. 1984. Does gap acceptance theory adequately predict the capacity of a roundabout? *Australian Road Research*, 12.

- TROUTBECK, R. 1986. Average delay at an unsignalized intersection with two major streams each having a dichotomized headway distribution. *Transportation Science*, 20, 272-286.
- TROUTBECK, R. 1988. Current and future Australian practices for the design of unsignalized intersections. *Intersections Without Traffic Signals*. Springer.
- TROUTBECK, R. 1989. *Evaluating the Performance of a Roundabout*.
- VAN AS, S. C., JOUBERT, H.S., Van As, S.C., & Joubert, H.S., Traffic flow rate theory, VIAED, University of Pretoria, 4th Edition. 1993.
- VASCONCELOS, L., SECO, Á. & SILVA, A. B. 2013. Comparison of procedures to estimate critical headways at roundabouts. *Promet–Traffic&Transportation*, 25, 43-53.
- VASCONCELOS, L., SECO, Á., SILVA, A. B., ABREU, T. & SILVA, J. 2012. A COMPARISON OF ROUNDABOUT CAPACITY MODELS.
- VUILLEMIN, G. 1999. The quality of road service: evaluation, perception and response behaviour of road users. *AIPCR–C4, Routes, Transport et développement regional*.
- WONG, S. 1996. On the reserve capacities of priority junctions and roundabouts. *Transportation Research Part B: Methodological*, 30, 441-453.
- WONG, S. & YANG, H. 1997a. The estimation of reserve capacity in traffic control. *HKIE Transactions*, 4, 21-30.
- WONG, S. C. & YANG, H. 1997b. Reserve capacity of a signal-controlled road network. *Transportation Research Part B: Methodological*, 31, 397-402.
- WU, N. 2012. Estimating distribution function of critical gaps at unsignalized intersections based on equilibrium of probabilities. *Transportation Research Record*, 2286, 49-55.
- XU, F. & TIAN, Z. 2008. Driver behavior and gap-acceptance characteristics at roundabouts in California. *Transportation Research Record: Journal of the Transportation Research Board*, 117-124.
- YAP, Y. H., GIBSON, H. M. & WATERSON, B. J. 2013. An international review of roundabout capacity modelling. *Transport Reviews*, 33, 593-616.

ZONG, Z., TROUTBECK, R., KYTE, M. & BRILON, W. A Further Investigation on Critical Gap and Follow-Up Time. Transportation Research Circular E-C018: 4th International Symposium on Highway Capacity, USA, ND.

APPENDIX

APPENDIX A

**PUBLISHED AND ACCEPTED PEER REVIEWED PAPERS
FROM THIS STUDY**

LIST OF PUBLICATION FROM THIS THESIS

Book Chapter: 1. **Ibijola, S. O.**, & Ben-Edigbe, J. (2016). Effects of Rainfall on Control Delay and Queue at Multilane Roundabout, Proceeding of Canadian Society of Civil Engineer Conference, Resilient Infrastructure, ISBN 978-1-5108-4359-2, vol (4), pp 3118-3127.

Publications:

2 **Ibijola, S. O.** and Ben-Edigbe, J. (2018). Influence of Rainfall on Driver Behaviour and Gap Acceptance Characteristics at Multilane Roundabouts. Open Transportation Journal. (Abstract/Index in Scopus, Manuscript in press; accepted for publication).

3. **Ibijola, S. O.**, & Ben-Edigbe J. (2018). Impact of rainfall on volume/capacity ratio at multilane roundabout, Hong Kong Society for Transportation Studies, Transportation Systems Conference. (Manuscript under review).

4. Ben-Edigbe, J, and **Ibijola, S. O.** (2018). Anomalous Capacity Shrinkage at Multilane Roundabout Caused by Rainfall. ICE-Transport Journal. ISSN 0965-092X, (Abstract/Index in SCI and Scopus; impact factor 0.402, DHET Accredited Journal).
doi.org/10.1680/jtran.17.00092.

5. Ben-Edigbe, J. and **Ibijola, S. O.** (2018). Evaluation of Roundabouts Functional Quality of Service Deterioration Caused by Rainfall, Promet-Traffic and Transportation. (Abstract/Index in SCI and Scopus; impact factor 0.43, DHET Accredited Journal, Manuscript in press; accepted for publication).

6. Ben-Edigbe J., Amir Pakshir and **Ibijola S.O.** (2017). Extent of Entry Capacity Loss at Roundabouts Caused by Rainy conditions, Advances in Civil Engineering. (Abstract/Index in SCI and Scopus; impact factor 0.402, DHET Accredited Journal).
doi.org/10.1155/2018/4192323.

7. **Ibijola, S. O.**, & Ben-Edigbe, J. (2016). Effects of Rainfall on Control Delay and Queue at Multilane Roundabout, Paper presentation at Canadian Society for Engineering Conference on 1st to 4th June 2016 at London, Ontario, Canada. (Presented by Ibijola S. O.).