INFLUENCE OF RAINFALL ON RED-LIGHT RUNNING AT SIGNALISED INTERSECTIONS AND THE SERVICE QUALITY IMPLICATIONS

By

Janet Kemunto Oyaro

217079425

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Prof. Johnnie Ben-Edigbe

PREFACE

The research contained in this Thesis was completed by the candidate while based in the Department of Civil Engineering, College of Agriculture, Engineering and Science, University of Kwa-Zulu Natal, Howard Campus in Durban South Africa.

The contents of this work have not been submitted in any form to another University 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: 25 August 2021

DECLARATION

I, **Janet Kemunto Oyaro** declare that

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As candidate's supervisor I agree to the submission of this thesis

Signed by Supervisor: Prof Johnnie Ben-Edigbe

Date: 25 August 2021

PUBLICATIONS, CONFERENCES, AND TRAINING

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

Publications

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Conferences

1. Oyaro J, Ben-Edigbe J (2021). Influence of rainfall on the probability of Red-light Running at Signalised intersections. Presented at the Canadian Society for Civil Engineering Conference (CSCE) held from 26th -29th May 2021. (Presented by Oyaro J)

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Training

1. 28th International course on Transportation Planning and Safety in India Institute of Technology (IIT), Delhi held from 26th November to 05th December 2018.

Signed by candidate: Janet Kemunto Oyaro

Date: August, 2021

DEDICATION

To all the women in my family, this is for you.

and

To my late grandmother, born at a time it was considered a waste to educate girls but grew to support and encourage girl child education, you bought me my first school uniform, supported me throughout my many years of schooling, I wish you lived long enough to witness this.

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ABSTRACT

When drivers approach a signalised intersection stop-line, they must decide whether to stop or proceed and clear the intersection before the end of a green phase. The driver behaviour at the intersection affects signalised intersections' performance especially in terms of safety (red-light violation) and efficiency (throughput and delay). Drivers are affected by the state of the traffic lights, prevailing traffic conditions, road conditions and prevailing yellow light laws. When rain falls driver behaviour is affected, and this could, in turn, affect the performance of signalised intersections. This study aims to determine the impact rainfall has on red-light violations and what implications that could have on intersection service quality. In Durban South Africa, four (4) signalised intersections were selected for a "dry" versus "rainy" study carried out using traffic, and rainfall data collected over eight weeks covering the rainfall season. The probability of red-light running (RLR) was found to decrease with an increase in rainfall intensity. The probability reduced from about 42% on average under dry conditions to 17% under light, 7% under moderate and 2.5% for heavy rainfall intensity. It implies that it becomes nearly impossible to violate a red light under rainfall conditions due to speed reduction and hence increase in travel time. The average time needed to safely cross the stop line at the onset of yellow time interval also increased from 3s during dry weather to 3.6s for light rain, 3.9s for moderate rain, and 4.5s for heavy rain. Thus, approaching vehicles cannot safely enter the signalised intersection and must wait at the stop line for a green signal. Therefore, it can be summarised that rainfall has a mitigating effect on red light violations especially under heavy rain where it is near impossible to run a red light. South Africa does not have a highway capacity manual (HCM) and relies on USA-HCM for signalised intersection assessment. HCM uses delay as the sole determiner of signalised intersection quality of service; this was found inadequate and not a complete reflection of driver perception of the level of service; this study proposed a criterion that incorporated degree of saturation in addition to the delay. With the developed criteria, analysis was done to determine the rainfall influence on signalised intersection performance. Through and right-turning traffic were considered separately in this study. For through traffic, saturation flow rate reduced by 3.9% under light rainfall, 8.68% under moderate and 10.88% under heavy. It led to a capacity loss of 4.25% under light, 9.18% under moderate and 11.5% under heavy. For right-turning traffic, saturation flow rates decreased by 7.07% under light rainfall intensity, 13.44% under moderate and 17.88% under heavy. The capacity loss was also recorded where light rainfall caused a 7.38% loss, moderate 14.5% and heavy 19.15%. For the degree of saturation, delay, and queue length, all three increased. The degree of saturation increased by 1.55% under light, 7.23% under moderate and 9.4% under heavy. The overall impact on service quality was mixed; for through traffic lanes, few instances where heavy rainfall caused a deterioration in SQ by one level were recorded. For right-turning traffic lanes, the results were more consistent with expectations. There was an increase in both degrees of saturation and delay. Overall, the SQ level deteriorated especially under heavy rainfall conditions. Right-tuning lanes showed higher SQ deterioration attributed to their higher saturation level.

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LIST OF ABBREVIATIONS USED

AD	Approach Distance
AMS	America meteorological Society
ASCE	American Society of Civil Engineers
ATC	Automatic Traffic Counters
BS	Braking Section
CBD	Central Busines District
CCTV	Closed-Circuit Television
CSCE	Canadian Society for Civil Engineering
DZ	Dilemma Zone
ETA	eThekwini Transport Authority
FHWA	Federal Highway Administration
FVZ	free flow Zone
HCM	Highway Capacity Manual
IIT	India Institute of Technology
LOS	Level of Service
PCE	Passenger Car Equivalent
PCU	Passenger Car Unit
PRT	Perception Reaction Time
PVZ	Posted Speed Zone
RLR	Red-Light Running
RTMC	Road Traffic Management Corporation
SARTSM	South African Road Traffic Signal Manual
SATC	Southern Africa Transportation Conference
SNMI	Spanish National Meteorological Institute
SQ	Service Quality
SSD	Stopping Sight Distance
SULT	Start-up lost time
TDE	Traffic Data Extractor
TVZ	Transition Zone
UKZN	University of KwaZulu Natal
WHO	World health Orgainzation
WMO	World Meteorological Organization
YLO	Yellow Light Overrun

LIST OF SYMBOLS AND UNITS

Symbol	Definition	Units
v	Approach speed	m/s
δ	Perception reaction time	s
a	Deceleration	m/s^2
ϕ	Dummy variable	
β	Regression coefficient factors	
λ	Ratio of effective green to cycle length	
h	Headway	s
s	Saturation flow rate	pcu/hr
X	Degree of saturaion	
C	Cyle length	s
c	Capacity	pcu/hr/ln
i	Rainfall intensity	Mm/hr

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CHAPTER 1

INTRODUCTION

1.1 Overview

Traffic lights are signalling devices placed at road intersections to control traffic flows. They play a crucial role in road network performance in terms of safety and congestion (Miguel S'anchez and Kim, 2006). Traffic lights can be useful at intersections with a history of crashes between turning vehicles because they facilitate orderly turning movements with three light signals (red, yellow, and green). The red and green traffic lights indicate stop and go, respectively. However, a yellow signal light warns that the red signal is about to appear. Signal timing is an important element of traffic signal control systems. Signal timing is used to distribute right-of-way at a signalised intersection by appropriating timing values per traffic phase.

In South Africa, the law states that *when a vehicle enters the intersection during the entire yellow interval, it is permitted to proceed and clear the intersection safely, however in a situation where the vehicle can neither enter nor be in the intersection on red, it must stop upon receiving the yellow interval* (SARTSM, 2012a). This rule is ambiguous and may be partially held accountable for red-light running. When a vehicle enters the intersection during the entire yellow light, permission to proceed and clear the intersection is inconsequential because the vehicle is already in the intersection. The unclear yellow light rules governing driver's behaviour at signalised intersections may be partially responsible for red-light running. Red-light running occurs when drivers violate the red light either because they cannot stop (unsafe or uncomfortable) or choose not to stop. It is a persistent problem in many countries.

Driving in the rain is deft and challenging. Rainfall has been shown to have effect on various elements of a transportation network (Alhassan and Ben-Edigbe, 2010),(Alhassan and Ben-Edigbe, 2011, Mashros et al., 2014). Previous studies have in addition shown that rainfall causes capacity loss and increases travel time on roads irrespective of classification (Prevedouros and Chang, 2004),(Perrin et al., 2001, Shin and Choi, 1998). How would road users adapt to traffic signal instructions under rainy conditions? It may be queried. Due to the spatiotemporal nature of rain, it poses a major hazard to drivers. They are forced to reduce speed and maintain increased space to minimise shunting in the event of an abrupt stop by the lead vehicle. Red-light running occurs when drivers violate the red light either because they cannot stop (unsafe or uncomfortable) or choose not to stop (Bonneson and Son, 2000).

This study investigated the influence of rainfall on red-light running and the implications for service quality. It is based on the hypothesis that service quality reductions due to rainy conditions are significant. This exercise aimed to establish whether red-light running violation and service quality can be sustained during rainfall and the relationship between these variables and rainfall intensities.

It is clear from the preceding that initiatives and measures that include research into the influence of rainfall on red-light running and service delivery at signalised intersections have to be undertaken to tackle issues on red-light running violations in South Africa. Consequently, the remainder of this chapter is structured as follows; section 1.2 gives the background to the study; section 1.3 the study aim and objectives are presented; section 1.4 discusses the method used in the study, while section 1.5 highlights the scope and limitations of the study. Section 1.6 discusses the significance of the study and in section 1.7 the thesis organisation is presented.

1.2 Background to the study

The South Africa Road network comprises approximately 754,600km of roads and streets. The road system is divided into three categories: National, Provincial, and municipal roads. Considering that South African operates on the left-hand drive rule, signalised intersections are commonly used on the urban metropolitan / municipal roads. At signalised intersections, traffic light phases are set so that, at the posted speed limit, vehicles should be able to pull up safely when the light turns yellow without having to drive through the intersection while the light is red. At the onset of red traffic lights, users must stop if safe, then wait behind the line until the light turns green.

South Africa is a subtropical country with the coldest months between June and August. The average annual rainfall is 450mm, but large and unpredictable variations are common. Overall, rainfall is greatest in the east and gradually decreases westward. Wet weather conditions, as mentioned in SARTSM, are not defined in terms of actual rainfall quantities. According to the world meteorological Organisation (WMO), the intensity of rainfall at any given time and place may be classified as light rain intensity (i) $\leq$ 2.5mm/h, moderate rain (2.5 – 10 mm/h), and heavy rain (10 – 50 mm//h). The Spanish National Meteorological Institute, on the other hand, delineates 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, the rain classification for these countries is presented below 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. The world meteorological organisation (WMO) classification was used for this study.

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	10 - 50	15 - 30
Very heavy	-	>50	30 - 60
Torrential rain	-	-	>60

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

Passenger car equivalent value is an essential parameter. It is defined in (HCM, 2016) 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 prevailing weather conditions also constrain the passenger car equivalent value. Thus, it could be argued that the passenger car equivalent value would also vary, given rainy 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. 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. That postulation cannot be allowed to stand. In this thesis, passenger car equivalent values were investigated, appraised, and modified as required.

Whilst capacity is clearly defined and well understood in many studies, quality of service or definition is often intertwined with the level of service. Quality of service has often been used interchangeably with the level of service, even though each has a unique definition. According to the Highway Capacity Manual (HCM), the level of service (LOS) is a measure of effectiveness, whereas the quality of service is defined as a measure of performance based on the perceptions of the service provider and service user. 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 the desired result, bearing in mind that if effectiveness is a measure of road providers' perception, one can assume that the desired result of road providers would be capacity utilisation of signalised intersection facilities. After all, signalised intersections are designed for capacity utilisation and not delay.

In this study service quality (SQ) delivery at the signalised intersection is defined as a measure of road users' and providers' perception of quality. It can be argued that road users are most likely concerned with a delay at signalised intersections, whereas road provider's quality assessments are often tied to the instrument of intersection geometric design. Service quality is an ephemeral, somewhat complex concept and descriptive where customers' perceptions form the foundation on which service quality is partially assessed. Service quality and delay at signalised intersections are intertwined. Generally, good quality of service to road users could be construed as a minimum delay, while poor service means excessive delay. On the other hand, good quality of service to road providers means a minimum degree of saturation also called volume capacity ratio (v/c), and minimum delay at intersections among others. According to (Ben-Edigbe et al., 2018), the road providers and road users are the main parties in quality-of-service assessment; hence the quality of service must depict their perceptions. Bearing in mind that the service quality from road providers is tied to road design specifications, whereas the service perceived by the road users is a statement of satisfaction or dissatisfaction experienced by the road users.

According to (Ben-Edigbe, 2011), the definition of quality of service or service quality as the case may be is presumed to be more encompassing than that of HCM 2016 level of service, which recognises only delay at intersections. Using delay as a determinant of the level of service has been argued against in the past. The Canadian capacity guide (S. Teply., 2008) argued that there is more logic in basing LOS classification on the degree of saturation (v/c ratio) than on delay. Because delay is a subjective measure that changes depending on individuals, situations, and locations, also delay was found to give conservative results of LOS. It can also be argued that the delay values used in the HCM evaluation of LOS are meant to be based on driver perception of delay but that is not necessarily the case, and secondly it is unlikely that delay is the only parameter that could influence user perception of quality of service.

Volume to capacity ratio (v/c) and the average back of queue was also important. Is it possible that both v/c and queue length could be used to determine the quality of service experienced at signalised intersections? Since capacity and quality of service performance are tied to prevailing weather conditions, how will delay, degree of saturation and intersection capacity be affected? Further, the inherent red light running implications are associated with service delivery and capacity reduction at signalised intersections. What are the lessons in context to be learnt from the study? If rainfall influences driver behaviour at signalised intersections and impacts the quality of service, what possible implication would this have on driver propensity to run red lights? Previous studies (Lu et al., 2016, Lu et al., 2019, Maki, 1999, Perrin et al., 2001) have shown that rainfall impacts driver behaviour and traffic stream characteristics. However, at signalised intersections, the influence of rainfall on capacity and service delivery reduction and their implications for red-light running has yet to be studied.

1.3.1 Research Aim

This study aimed to investigate the influence of rainfall on red-light running and the implication for service quality reduction at signalised intersections.

1.3.2 Objectives

The study objectives were:

i. To determine the extent of red-light running violations at signalised intersections during dry, rainy, and daylight conditions.

ii. To model the relationship that exists between red-light running violations and rainfall intensities at signalised intersections.

iii. To determine the probability of red-light running violation under dry, rainy weather and daylight conditions.

iv. To develop a service quality criteria table with delay and degree of saturation parameters.

v. To use the novel criteria table to assess service quality at signalised intersections during dry and rainy weather conditions.

vi. To determine the relationship that exists between service quality and rainfall at signalised intersections.

1.4 Method of the study

This study was carried out using both empirical and analytical methods. Empirical data was collected at selected study sites and the collected data was analysed then used to develop models for both red-light running and service quality. Models were developed for two scenarios (dry and rainfall) under daylight conditions. Empirical data collected at selected signalised intersections reflected the study objectives. The acceptability of passenger car equivalent values was investigated, and values were adjusted where necessary.

Traffic data were collected mainly using video footage obtained at the selected study intersections. Data extraction was done using mainly manual means, except for the speed for which software was employed. Manual on-site data were also collected for short periods and used mainly for verification purposes. Rainfall data was collected from rain gauge stations located within the study sites.

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 the 'dry and rainy' 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. The collected and analysed data were fed into relevant software to fit the required models used to explain the impact of rainfall on red-light running and the implication on service delivery at signalised intersections.

1.5 Research scope and limitations

This study was limited to four arms signalised intersections in the city of Durban, South Africa. All study sites had similar signal information as well as posted speed limits. The traffic data was manually collected from obtained video footage, and the manual data collection had its limitations. The video footage was obtained from a third party; hence verification had to be done at every stage. The number of study sites was also limited to the intersections where video footage could be obtained in a format that could be used to extract useful information.

The scope of the study was also limited to daylight conditions only; both dry and rainy weather conditions considered were during the day. It was to isolate any influence that darkness could have on driver behaviour. Based on the method of data collection (video cameras), it was not possible to properly analyse night-time data. For red-light running data analysis, only two intersections could be analysed because of limitations with the camera's angle.

The research limitations also included the rainfall period in Durban, which was between August and March. Peak rainfall with an average of 134mm is in January, meaning that data collection was restricted to the rainfall period. Only motorised vehicles were considered. Non-motorised transport was beyond the scope of this research. The total number of survey sites was constrained by funding, equipment, and human resources.

1.6 Significance of the study

The study contributes to the body of literature on red-light running violations at signalised intersections. At signalised intersections, drivers can be proactive or responsive as their behaviour is mainly determined by traffic signal compliance. They can choose to be normal, aggressive, or conservative, especially in response to adverse weather conditions. When approaching a signalised intersection, drivers must decide at the onset of yellow light whether to stop or proceed into the intersection. The unclear yellow light rules governing driver behaviour at a signalised intersection may be partially responsible for red-light running. Red-light running occurs when drivers violate the red light either because they cannot stop (unsafe or uncomfortable) or choose not to stop. It is a persistent problem in many countries. Driving in the rain is deft and challenging due to the spatiotemporal nature of rain.

The study contributes to the body of red-light running and service quality modelling at signalised intersections during rainfall. It uses a unique criteria table developed for the surveyed sites and enriches literature with this method. The novel criteria table uses delay and degree of saturation as key indicators of service delivery performance at signalised intersections. It is a clear departure from the HCM singular approach where the 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 perspectives in assessing signalised intersection service delivery enriches the existing literature. The influence of rainfall on red-light running and their implications for service quality delivery at signalised intersections using the empirical method is a novel approach. In previous studies, passenger car equivalent values were broadly applied to all conditions, a questionable approach.

There are very few studies on the influence of rainfall on red-light running at signalised intersections. Thus, this study is significant is in its attempt to show that by mapping out specific areas where the action is needed, delay at signalised intersections induced by rainfall can be minimised. The issue of red-light running, the factors that lead to it, and how it is impacted by rainfall are also significant to South Africa with a high road accident rate. The information could be used by traffic management of speed limits and the need to provide relevant information to motorists as they approach a signalised intersection. The research has relevance for traffic management policy and the decision-making process. 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 rainfall conditions. The study explains how signalised intersections perform under rainfall which could be useful in road intersection management and planning.

1.7 Organization of Thesis

The thesis is organised in an orderly manner to enable the reader to follow the arguments presented. Each chapter is structured to address the issues aimed at strengthening the hypothesis that rainfall, irrespective of intensity, affects red-light running and leads to an adverse effect on the quality of service delivery at signalised intersections. Note that chapter numbers precede figures and tables in the thesis for ease location; for example, figure 2.1 or table 4.2 shall be chapters 2 and 4. The layout of the remainder of the thesis is:

Chapter 2: the literature review is discussed.

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. South African passenger car equivalent values were investigated and tested for model fitness under rainy conditions.

Chapter 4: the empirical results per surveyed sites are presented.

Chapter 5: The influence of rainfall on red-light running at signalised intersections is discussed.

Chapter 6: Implications for service quality at signalised intersections are presented. A novel criteria table was developed and used to determine the service delivery at each selected study site.

Chapter 7: The study summary of findings, conclusions, recommendations, and the way forward are presented.

CHAPTER 2

LITERATURE REVIEW

2.0 Overview

In this chapter, the theoretical framework presented will support arising arguments in the later chapters. The central theme of this study is red light running at signalised intersections during rainfall and what this means for service quality. Red light running depicts a vehicle entering a signalised intersection any time after the traffic signal has turned red. It has a known correlation with vehicle acceleration. Rainfall, on the other hand, correlates with deceleration. Thus, when it rains, to what extent rainfall will dampen red-light running? At signalised intersections, drivers can be proactive or responsive; they can be aggressive or conservative in response to yellow lights. Will service quality deteriorate given the complexity of drivers' behaviour at signalised intersections, especially at the onset of yellow light? Highway Capacity Manual uses the level of service and quality of service (service quality) interchangeable. The manual defined the signalised intersection level of service as a weighted average control delay for the entire intersection. However, a delay is not an instrument of intersection design. So why appraise signalised intersection solely with a delay parameter. Intersection design is principally concerned with converting concepts to physical dimensions that include speed, sight constraints, traffic volume, and turning radius among orders. Once traffic signal control is at an intersection, the design role of parameters like sight constraints and speed depends on traffic signal control. Traffic volume relative to capacity (degree of saturation) is also an important measure of service quality and delay. Interestingly both road users and providers are interested in delay minimisation at signalised intersections. However, it would be useful for traffic management to be warned of possible capacity over-utilisation, hence the need to include the degree of saturation as a service quality assessment parameter.

This study is concerned with the influence of rainfall on red-light running and their service quality implications at signalised intersections. In light of the discussion so far, the chapter has eight sections. Section 2.1 presents roads and traffic in South Africa, and section 2.2 deals with rainfall in South Africa. In section, 2.3 road intersection design and operations presented. In section, 2.4 influence of signalised intersections on red-light running presented, and yellow traffic light concepts presented in section 2.5. In section, 2.6 service quality concepts presented. Section 2.7 discusses service quality implications, and the chapter is summarised in 2.8

2.1 Roads and traffic in South Africa

The South Africa Road network comprises approximately 754,600km of roads and streets. The South African National Roads Agency SOC Ltd (SANRAL) manages the national roads. The provincial roads are under provincial governments' management, metropolitan/ municipal roads, and concerned municipal council authorities. An intersection is an at-grade junction where two or more roads or streets meet or cross. The number of road segments, traffic controls, or lane design may classify intersections. They are a critical element of a road section and maybe a major bottleneck to the smooth flow of traffic and a major accident spot. Table 2.1 below gives a summary of the South Africa road network.

Table 2. 1: Summary of South Africa Road network

Road	Length	Percentage
Surfaced National toll and non-toll roads	15,600	2.10%
Surfaced Provincial roads	348,100	46.10%
Unproclaim rural roads	222,900	29.50%
Metropolitan, Municipal and other	168,000	22.30%
Total	**754,600**	**100%**

Source: National Department of Transport, SANRAL

The national roads are a class of trunk roads and freeways that connect major cities. They form the highest category in the country's route numbering scheme. They are designated with route numbers beginning with N. Provincial roads are the second category of roads and are designated with the letter R. they serve as feeders to the national roads. The metropolitan/ municipal roads designated with the letter M are important intra-city routes. All traffic in the country follows the keep left rule. South Africa has nine Provincial administrative centres, including Eastern Cape, Free State, Gauteng, KwaZulu-Natal, Limpopo, Mpumalanga, Northern Cape, North West, and the Western Cape. Each province then has several municipalities under it.

KwaZulu-Natal is in the Southeast of South Africa, bordering the Indian Ocean. It covers 94,361km^2 and has a population of 11,065 240; divided into 1 Metropolitan Municipality (eThekwini Metropolitan municipality that covers the city of Durban) and 10 district municipalities, further subdivided into 43 local municipalities. The entire paved road network within the eThekwini municipality consists of approximately 8,200Km of formally maintained roads. The total number of registered vehicles in South Africa as of December 2018 was 12, 462,979 out of which about 90% were light vehicles. The largest shares of the vehicle's population are in Gauteng, Western Cape, and KwaZulu-Natal. For the eThekwini Municipality, as per the 2016 report, the total traffic volume stood at 474,828 out of which 86% were cars, 8% Taxis (14-seater public transport vehicles), 5% heavies and 1% buses. The traffic volume trend for the eThekwini municipality for 2001 to 2016, as reported by the eThekwini Transport Authority (ETA), presented in figure 2.1 below.

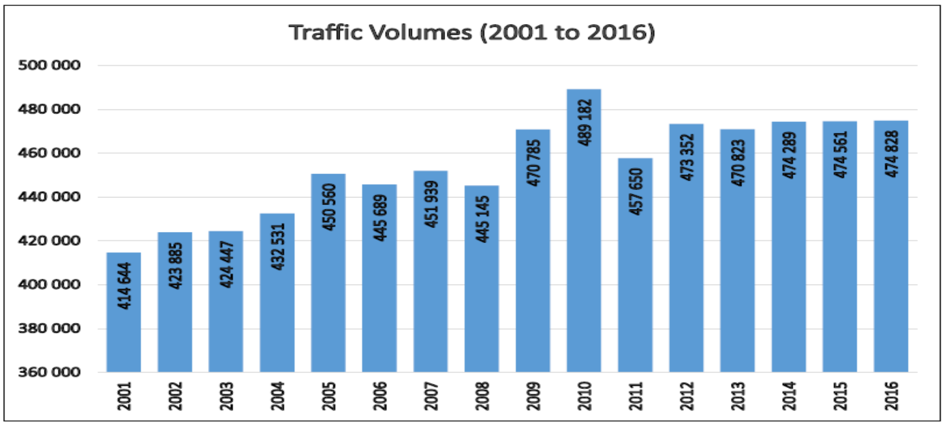

Figure 2. 1: Traffic volume trend for eThekwini municipality (ETA, 2016)

2.1.1 Road Traffic Accidents

The 2018 global status report on road safety (WHO, 2018) recorded that in 2016 the estimated number of deaths due to road traffic accidents was at 1.35 Million. The report further notes that road traffic injuries are the eighth leading cause of death for all age groups and the leading cause of death for children and young adults aged 5-29 years. Whereas the global rate of road traffic death is 18.2 per 100,000 population, there is significant variation across the World's regions, where the rate ranges from 9.3 to 26.6 per 100,000 population. Africa has the highest at 26.6, followed by South-East Asia at 20.7 deaths per 100,000 population; the trend illustrated in figure 2.2 below. The report notes that pedestrians, cyclists, and those using motorised 2 and 3 wheelers are the most affected when it comes to the types of road users most affected by road accident deaths. According to the report, Africa has the highest proportion of pedestrians and cyclist mortalities, with 44% deaths. 12,921 fatalities recorded for the period January – December 2018 from 10,564 fatal crashes in South Africa. The Road Traffic Management Corporation (RTMC) report released in 2019 (RTMC, 2019) recorded some contributory factors to road crashes. The contributory factors classified as either human, vehicle, or environment. The results indicated that human factors contributed to about 89% of the crashes, vehicle factors (mechanical failures) contributed about 4.2%.

In contrast, environmental factors (visibility, poorly marked roads, missing road signs, weather conditions, among others) contributed about 6.5%. Further analysis of the contributory factors showed that jaywalking contributed about 33% to road crashes hit and run about 14%. Wet and slippery road surfaces estimated to contribute about 12.6% of the total road crashes.

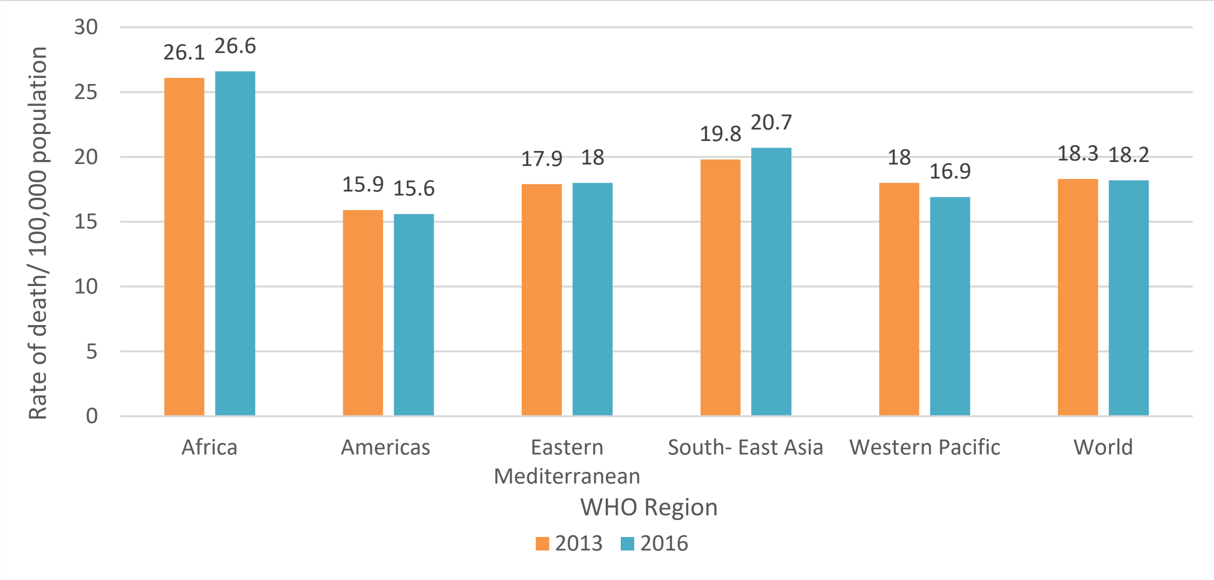

Figure 2. 2: World Road safety trend in 2013 and 2016 (WHO, 2018)

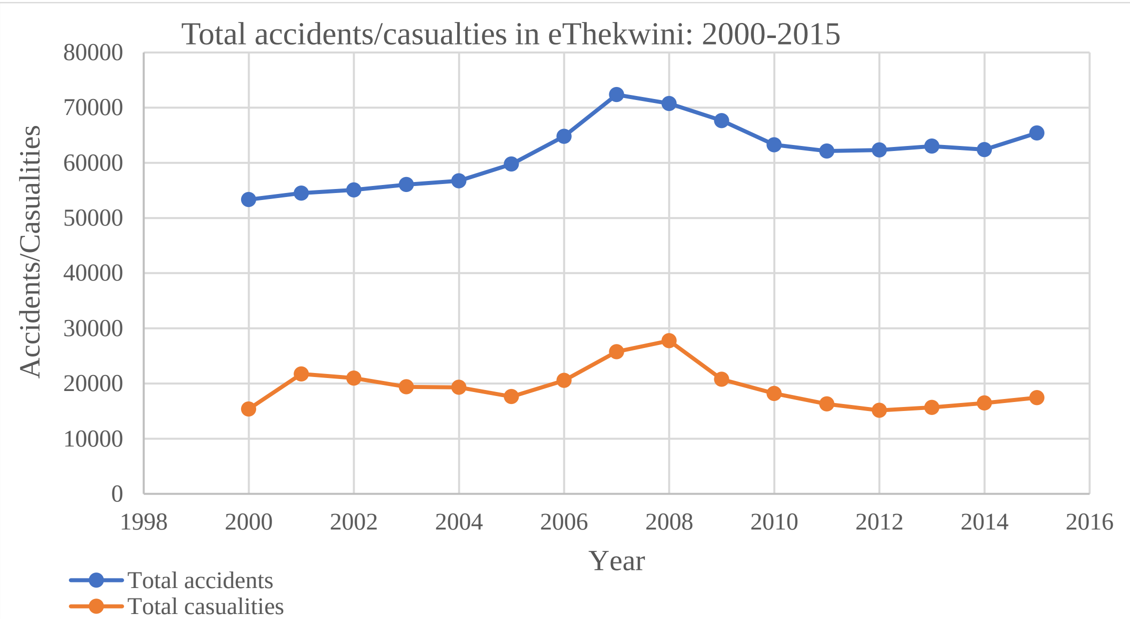

Figure 2. 3 : Road Accident trends for eThekwini municipality 2000-2015 (ETA, 2015)

Among the human factors accountable for road accidents, the following were found to be because drivers: failed to keep a proper lookout, failed to keep the vehicle under control, unsafe /illegal U-turns, disregard for stop sign/ traffic light, intoxication, driving in the wrong lane, fatigue, turning in front of oncoming traffic and overtaking in the face of oncoming traffic. In KwaZulu-Natal, the number of fatalities in 2018 was 2, 473 which was 261 below the 2017 number of 2800. For the eThekwini municipality, the accident report from 2016 showed 571 fatalities out of 65,414 accidents, 2,650 were serious, 9,719 were slight, and 52,474 were damage only. Figure 2.3 above summarises the trend in accidents for the city of Durban.

Records obtained from the eThekwini Transport Authority (ETA), the body in charge of traffic and transport in Durban for some of the sites used in the study, indicated that over 150 accidents were recorded per site. There was one pedestrian fatality involved. The accidents recorded were 67% of the accidents involved vehicles in the same direction, 14% vehicles in the opposite direction, 15% due to right-angle collisions, and about 2% for vehicle-pedestrian collisions. From these statistics, most accidents are rear-ended collisions, which could be due to sudden braking; secondly, though vehicle-pedestrian collision has a low percentage, it is the most serious resulting in a fatality in almost all cases.

2.2 Rainfall in South Africa

South Africa is a sunny country with an average of 8-10 hours of daily sunshine in most regions. It classified as a semi-arid country with an average rainfall of about 464mm while the world average is about 860mm. Most parts of the country get rain only in summer. The rainfall patterns vary from one province to another. The amount of precipitation received is higher in the Eastern parts of the country. 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 the city of Durban in the KwaZulu-Natal Province, where the rainfall intensity and frequency are high during October to March. There is a rainfall pattern, with rainfall increasing from West to East; this is illustrated in Table 2.2 below, which shows the city of Durban (Eastern part) having an annual rainfall of about 828mm while the City of Cape Town (Western part) having 475mm (Olurotimi et al., 2017). Rainfall usually occurs during the summer months of November to March, with some thunderstorms in the afternoon. In the Western Cape, the rainfall occurs in the winter months of May to September. The intensity of rainfall varies from province to province. Light rain falls throughout the year in Durban, but the wet season occurs from October to March. The wettest period occurs in January, while June is the driest month. The driest part of South Africa is Richards Bay in the KwaZulu-Natal province, with an average annual rainfall of 46mm.

Table 2. 2 Rainfall distribution in South Africa

Location	Latitude (^{o}S)	Longitude (^{o} E)	Annual Rainfall (mm)	Climatic Region
Durban	29^{o} 97'	30^{o} 95'	828	Coastal Savannah
Johannesburg	26^{o} 12'	28^{o} 20'	543	Subtropical Highland
Cape Town	33^{o} 58'	18^{o} 36'	475	Mediterranean
Kimberly	28^{o} 44'	24^{o} 46'	350	Continental

Source (Olurotimi et al., 2017)

Rainfall is the quantity of water falling within a given area over a given duration of time. There are three main types of rainfall: relief, frontal, and convectional, with description by quantity, frequency, distribution over an area, time of occurrence, or intensity. Rainfall intensity is defined as the ratio of the total amount of rain (rainfall depth) falling during a given period to the duration of the period. Expressed in-depth units per unit of time and classified in line with the World Meteorological Organization (WMO) as light, moderate or medium, heavy, and very heavy (violent), as shown in Table 2.3 below. There are different ways of measuring the amount of rainfall. They include the use of a rain gauge, weather radar, and satellite imagery. The weather radar and satellite imagery measure the rain precipitation at the earth's surface and above the earth's surface. Rain gauge collects the rain data at the earth's surface where traffic interacts directly with rainfall hence the preferred collection method for this study.

Table 2. 3: Rainfall intensity classification

Type of rain	**Intensity (mm/hr)**
Light rain	< 2.5
Moderate rain	$2.5 < I \leq 10$
Heavy rain	$10 < I \leq 50$
Very heavy (violent)	> 50

Source (WMO)

2.3 Road Intersection Design and Operations

An intersection is an at-grade junction where two or more roads converge, diverge, meet or cross. The three basic intersections are the three-leg or T-intersection (with variations in the angle of approach), the four-leg inter-section, and the multi-leg intersection. Each intersection can vary greatly in scope, shape, use of channelisation, and other types of traffic control devices. It can be a priority or signalised intersection. Irrespective of type, intersection crashes are one of the most common crash problems, particularly in urban areas. In rural areas, or where vehicle speeds are high, the consequence of collisions at intersections can be particularly severe. Intersections may be signalised to address road safety, efficiency, or operational issues or improve crossing opportunities for pedestrians and cyclists. Traffic signal controls installed at road intersections are due to the temporal component. Delay and queue are the main characteristics of signalised intersections. The South African Road Traffic Signal Manual-(SARTSM, 2012a) defines road traffic signals as "*one of the most common and widely accepted forms of traffic control and affect the daily lives of virtually all road users. Traffic signals can be very effective in improving traffic flow and facilitating access. However, traffic signals can also cause significant disbenefit and possible danger to road users when installed inappropriately.*" The definition suggests that traffic signals are good for traffic control, improving traffic flow, and facilitating access.

As defined in SARTSM, the most noted minimum requirements to appropriate installation of traffic signals are:

(a) Speed limit – the approach speed at signalised intersections must not exceed 80km/h; (b) Visibility requirements – traffic signal faces must be visible as drivers approach the traffic signal. In South Africa, five types of intersections are highlighted in SARTSM:

- Case A intersection where controls are not required;
- Case B intersections where the priority control system is used;
- Case C intersections where yield control is adopted;
- Case D intersections where traffic is signal controlled irrespective of the number of nodes;
- Case E is the all-way stop control system

2.3.1 Signalised Road Intersections Geometric Design

Several factors such as traffic volumes, human factors, vehicle factors, topography, and economic considerations influence road intersection geometric design. It must accommodate peak hour travel efficiently and safely, even if additional lanes may be required to accommodate the traffic. There are considerations for human factors with uniform and proper design across all intersections to avoid surprises, as humans tend to act according to habit. About topography and environment, often some compromises may be required. The geometric factors affecting the capacity of a traffic signal approach include:

- degree of saturation on the opposing arm
- number of storage spaces available inside the intersection which right turners can use without blocking straight-ahead vehicles;
- proportion of cycle time effectively green for the phase under consideration or effective green divided by cycle time; cycle time (s).

In South Africa, lane widths for straight-through movements cannot be narrower than 3.3m, although 3m can be resorted to where the 3.3m would mean a right turn lane, with high heavy vehicle volumes, the lane width increased to 3.5m. Lane widths for left and right turn should preferably not be narrower than 3m, with significant heavy vehicle volumes increased to 3.3m. The widths of exit lanes should be sufficient to accommodate paths of turning vehicles, where only one lane is available, it should be at least 3.5m, but widths of 4 to 4.5m may be required, especially to accommodate heavy goods vehicles. Where a right-turn movement turns into a two-way road, the stop-line on the exit may be set back to provide space for the movement. With a median provided, the exit lane widened. (SANRAL, 2011) (SARTSM, 2012a)Figure 2.4 shows some of the options that could be adapted to handle turning movements.

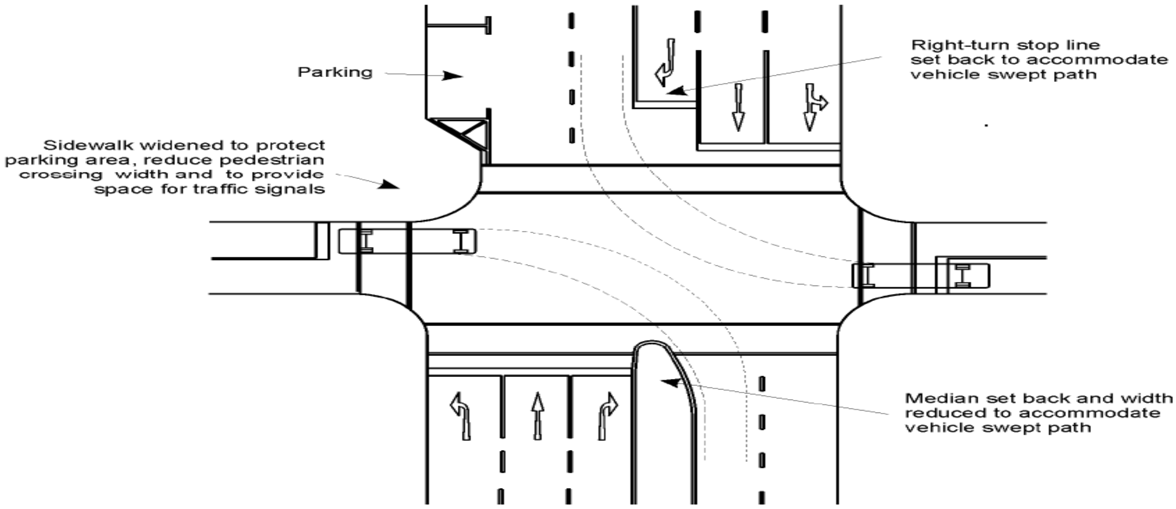

Figure 2. 4 : Handling of turning movements at signalised intersections (SARTSM, 2012a)

Summary of Road Markings for Signalised Junctions and Crossings shown in table 2.4. For the design of the medians, about 2m minimum recommended; this will give allowance to accommodate the traffic signals and pedestrians and pedal cyclists. In places with high pedestrian volumes, a wider median is used. Exclusive right turn lanes warranted in most locations for safety as well as capacity purposes.

The need for right-turn lanes depends on the speed of the roads considered and the probability of conflict; on high-speed roads, even with low volumes, exclusive right-turn lanes recommended for safety purposes. As a rule, therefore, right turn lanes provided at all traffic signals except where the operating speeds are 50km/h or less. On capacity grounds, where right-turn traffic volume exceeds 300 vehicles in the peak hour, the provision of a right-turn lane is considered. When it comes to the traffic signal faces and poles and positioning, there are principal traffic signal faces and supplementary traffic signal faces (for visibility and conspicuity). Traffic signal faces mounted on a standard post, extended post, or overhead cantilever post. Principal traffic signal faces should preferably be post mounted at the side of the road. Supplementary signal faces may be post-mounted or mounted above the road surface on a cantilever (gantry). In some cases, the signal faces used in conjunction with other road signs like street names.

Table 2. 4 : Summary of Road Markings for Signalised Junctions and Crossings (SARTSM, 2012a)

Marking number		Urban areas		Urban areas	
	Description	**Width**	**Length**	**Width**	**Length**
Regulatory and road markings					
RTM 1 (white) [1,2]	STOP Line	300mm Min	Full approach	500 mm Min	Full approach
RTM 2 (white) [1,3]	YIELD Line	300mm Min	600mm Line 300mm Gap	500mm Min	100mm Line 500mm Gap
RTM 3 (white) [2,3]	Pedestrian crossing line	100mm Min [4]	Full roadway	100mm Min [4]	Full roadway
RTM4 (white)	Block pedestrian crossing	[4]	600mm Block 600mm Gap	[4]	600mm Block 600mm Gap
RM 1 (White)	No-overtaking line	100mm Min	9m Minimum 18 m Preferred 27m Multi-lane	100mm Min	12mm Minimum 26 to 60m Preferred
RM 3 (white)	Channelising / Stacking line	100mm Min same as GM 1 or WM2	9m Minimum 18 m Preferred 27m Multi-lane	100mm Min same as GM 1 or WM2	12mm Minimum 24m Preferred 60m High speed
RM4:1 (yellow)	Left edge line	100mm Min		100mm Min	
RM4.2 (white)	Right edge line	100mm Min		100mm Min	
RM5 (yellow with white borders)	Painted island marking	100mm White boundary lines 150mm to 1000mm Yellow lines sloped at 30/60 degrees or 200mm to 600mm Continuous yellow line between two white lines			
RM8 (yellow)	Mandatory direction arrows	Approximately 1 m in advance of stop line			
RM 10 (yellow)	Box junction	100mm Min		100mm Min	
RM11 (white)	Zig Zag Zone	100mm Min	2.0 m Line 150mm gap 30 m length	100 mm Min	2.0 m Line 150mm gap 30 m length
RM12 (red)	No-stopping line	100/150mm Min		100/150mm Min	
Warning road markings					
WM2 (white) [5]	Continuity line	20mm Min	1.5m Line, 1.5/3.0/ 7.5m gap	200mm Min	2m Line 2/4/6m Gap
WM3 (white)	Dividing line	100mm Min	3.0 m Line 6.0m Gap	100mm Min	4.0 m Line 8.0m Gap
Guidance road markings					
GM1 (white) [5]	Lane line	100mm Min	1.5m Line, 1.5/3.0/ 7.5m gap	100mm Min	2m Line 2/4/6 m Gap
GM2 (white)	Guidelines	100mm Min	0.5 Line 1.5M Gap	100mm Min	0.5 Line 1.5M Gap
GM3 (white)	Bifurcation arrows				
GM4 (white)	Information arrows				

2.3.2 Traffic Signal Operations Concepts

As contained in the literature, traffic signal operation is the active prioritisation of objectives and collection of information to manage traffic signal infrastructure and control devices to maximise safety and throughput while minimising delays. In the design of signalised intersections, some elements influence the safety and operation of the intersections. Those elements include; the number of approach lanes, length of turn bays, presence of additional through lanes near the intersection, the size and location of detectors, presence of turning lanes, among other geometric features. Typically, a traffic controller mounted inside a cabinet controls the traffic signal. The traffic controller uses the concept of phases, which are directions of movement grouped. A phase is a green interval plus the change and clearance intervals that follow it. Thus, during the green interval, non-conflicting movements assigned to each phase. It allows a set of movements to flow and safely halt the flow before the phase of another set of movements start. A stage is a group of non-conflicting phases that move at the same time.

The signal design procedure involves six major steps. They include the (1) phase design, (2) determination of amber time and clearance time, (3) determination of cycle length, (4) apportioning of green time, (5) pedestrian crossing requirements, and (6) the performance evaluation of the above design. The objective of phase design is to separate the conflicting movements in an intersection into various phases so that movements in a phase should have no conflicts. If all the movements separated with no conflicts, then many phases would be required. In such a situation, the objective is to design phases with minimum or less severe conflicts. There is no precise methodology for the design of phases. Except guided by the geometry of the intersection, flow pattern, especially the turning movements and the relative magnitudes of flow. Therefore, a trial and error procedure adopted. However, phase design is very important because it affects further design steps. In addition, it is easier to change the cycle time and green time when the flow pattern changes, whereas a drastic change in the flow pattern may cause considerable confusion to the drivers. To illustrate various phase plan options, consider a four-legged intersection with through traffic and right turns where the Left turn ignored, as shown in figure 2.5. The first issue is to decide how many phases are required. It is possible to have two, three, four, or more phases.

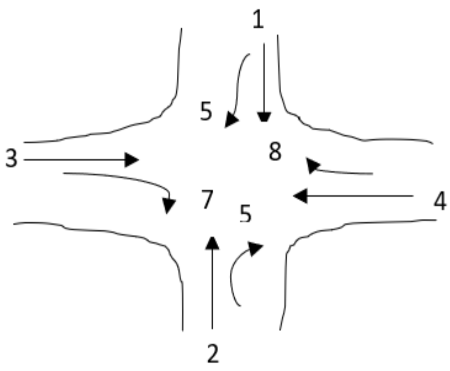

Figure 2. 5 : Four-legged intersection (SARTSM, 2012a)

A two-phase system is usually adopted if the through traffic is significant compared to the turning movements. For example, in figure 2.6, non-conflicting through traffic 3 and 4 grouped in a single phase. None conflicting through traffic 1 and 2 are grouped in the second phase. However, flow 7 and 8 offer some conflicts in the first phase and are called permitted right turns. This kind of phasing is possible only if the turning movements are relatively low. On the other hand, if the turning movements are significant, then a four-phase system adopted.

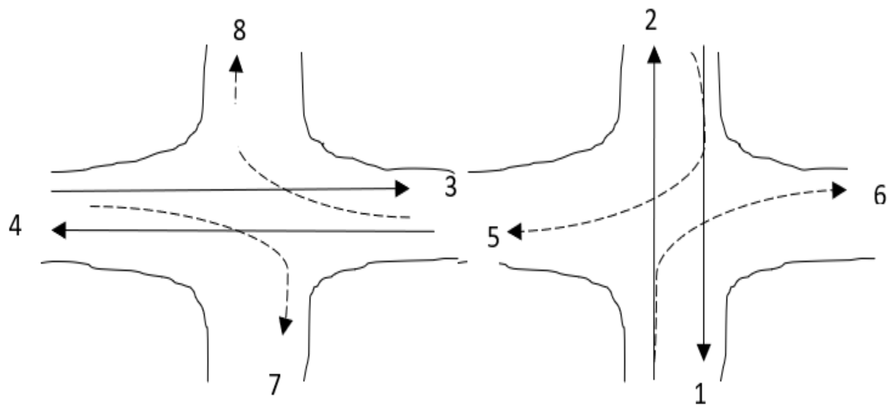

Figure 2. 6: Two-phase signal (SARTSM, 2012a)

For four-phase systems, there are at least three possible phasing options. For example, figure 2.7 shows the most simple and trivial phase plan where the flow from each approach is put into a single phase, avoiding all conflicts. This type of phase plan is ideally suited in urban areas where the turning movements are comparable with through movements and when through traffic and turning traffic need to share the same lane. This phased plan could be very inefficient when turning movements are relatively low.

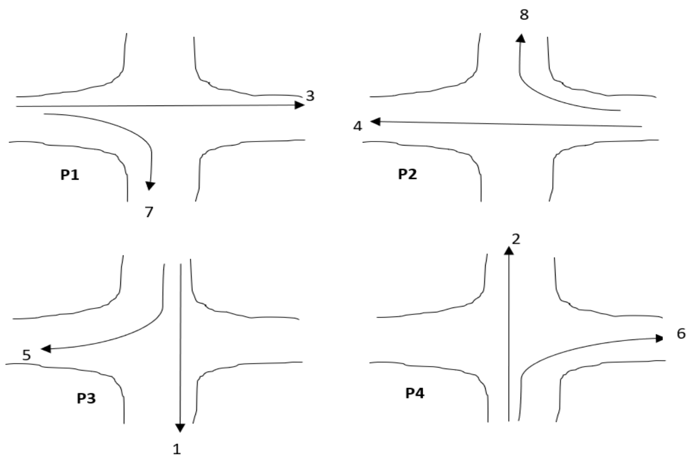

Figure 2. 7 : One way of providing four-phase signals (SARTSM, 2012a)

Figure 2.8 shows a second possible phase plan option where opposing traffic put into the same phase. The non-conflicting right turn flows 7 and 8 grouped into a third phase. Similarly, flows 5 and 6 grouped into a fourth phase. This phrasing is very efficient when the intersection geometry permits at least one lane for each movement, and the through traffic volume is significantly high.

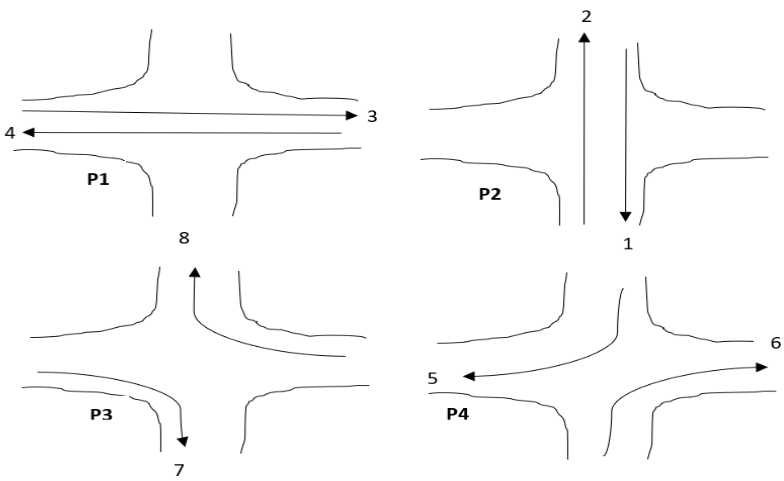

Figure 2. 8: Second possible way of providing a four-phase signal (SARTSM, 2012a)

2.3.3. Traffic signal timing

In establishing traffic signal settings, determining phasing requirements is fundamental; some of the phases that could implemented include; the main phase, which provides for straight-through and permitted left and right-turn movements (Akcelik et al., 1999). A single right-turn phase provides a movement to the right with or without a parallel left-turn phase. A double right-turn phase that provides for right-turn movements from two approaches, with or without left-turn phases. A protected turning phase allows one movement direction to turn while another on the same approach stopped (used with exclusive turning lanes). Right-turn phases could either be lagging or leading. The leading right-turn phase appears with or before the main phase on the same approach. The lagging right-turn phase appears after or during the final part of the main phase interval. A right-turn phase provided in two modes; protected/permitted mode, where a leading or lagging protected turn phase provided, but the turning movement is permitted during the main phase. Protected only mode is where the vehicles can only turn during a leading or lagging protected phase. The primary objective of signal timing settings is to move people through an intersection safely and efficiently(FHWA, 2008). Achieving this objective requires a plan that allocates the right of way to the various users and accommodates fluctuations in demand for each day, week and year. Many signal timing parameters affect intersection efficiency, including the cycle length, movement green time, and clearance intervals. The relationship between signal timing and safety addressed with specific timing parameters; for example, the yellow change interval intends to facilitate the safe transfer of the right of way from one movement to another. The safety benefit of this interval is realised when its duration is consistent with the needs of drivers approaching the intersection at the onset of the yellow indication. Proper signal timing and phasing is necessary for efficiency and safety at signalised intersections. In designing signal timing and phasing, aspects such as cycle lengths, green splits, yellow time and all-red intervals designed. To this end, SARTM defines some of the signal timing and phasing parameters as below:

- Cycle – the time required for one complete sequence of lights
- Wintergreen- the yellow signal interval plus the all-red. It is a safety interval; between the end of one green and the start of another.
- Offset- time difference between the start of a signal at one traffic signal and the start of a stage at another signal.
- Phase- an interval of the signal cycle during which a particular green signal displayed.
- Stage – an interval of the signal cycle during which any combination of the vehicular green signal is displayed (pedestrian and pedal cyclists excluded).
- Signal group: a group of traffic signals that always display the same sequence of light signals simultaneously.

A signal-timing plan determines the cycle time, sequence of phases and stages, and the timing characteristics of each stage. Often traffic signals may require multiple timing plans to cope with variations in traffic demand throughout the day and different days of the week. In typical setups, several signal-timing plans provided, including but not limited to the following:

i. The weekday morning (AM) peak period plan operated for 30 minutes before and after the morning peak.
ii. Off-peak (midday) period plan, operated between morning and afternoon peak plans.

iii. Weekday afternoon (PM) peak period plan, typically operated for 30 minutes before and after the afternoon peak period
iv. Evening period plan that follows the PM peak period
v. Night flow period plan
vi. Weekend and holiday period plan.

Correct timing and phasing are fundamental to the proper functioning of traffic signals. Traffic signals operate in either pre-timed (fixed) time, actuated mode, or some combination of the two (S. Teply., 2008). It is possible to have different timing plans operated at different times of the day or days of the week. In the pre-timed mode, it consists of series of intervals that fixed in duration. The cycle length, green time, yellow and red are fixed, and the sequence is deterministic. Pre-timed control is ideally suited for closely spaced intersections where traffic volumes and patterns are consistent on a day or day of the week basis. This control mode used to provide coordination with adjacent pre-timed signals. The drawback of pre-timed control is the inability to handle unexpected fluctuations in traffic flow and could be inefficient in isolated junctions where traffic arrivals are random. Actuated mode or control consists of intervals called or extended in response to vehicle detectors. The detectors provide information about traffic demand. The duration of each phase is determined by detector input and corresponding parameters. Actuated control characterized as either fully actuated or semi-actuated. The semi-actuated control uses detection only for the minor movements of an intersection. The phases associated with the major road (through road) operated as non-actuated. In this mode, the controller dwells on the non-actuated phase (with sustained green indication). The minor movement phases are services after a call for the service in-vehicle presence received. All phases actuated in the fully actuated control; therefore, detectors are required on all approaches. It is ideally suited for isolated junctions where the traffic demand patterns vary widely during the day. Fully actuated control has an advantage over pre-timed in that it could reduce delay because it is responsive to traffic demand and changes in traffic flow patterns.

A time-space diagram can represent the flow of vehicles approaching and travelling through a signalised intersection. The diagram shows the position of each vehicle at any point in time. The slopes of the trajectories show the speed of a vehicle at any point in time. Figure 2.9 below is an example of the time-space diagram; signal display intervals of red and green. During the first cycle, three vehicles arrive during green and drive through without stopping, the arrival headway (h_a) is constant, and the flow pattern called uniform. During the second cycle, vehicles queued during the red interval are discharged; when the discharge headway becomes constant between the departing vehicles, it is called saturation headway (h_s).

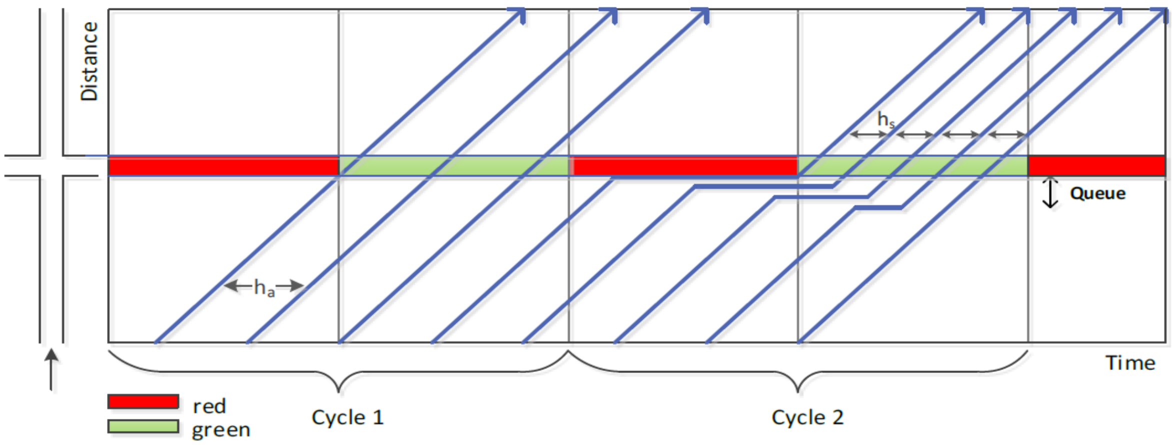

Figure 2. 9 : Example of time-space diagram representing flow through a signalised intersection(Kyte and Tribelhorn, 2014)

2.3.4. Traffic signal cycle length

At any signalised intersection, the key parameter is cycle length settings. One of the main advantages of traffic signals is the facilitation of orderly and timely traffic flow movements. The cycle length is the time in seconds that it takes a signal to complete one full cycle of indications. In designing cycle times, it noted that shorter cycle lengths are desirable because delay reduced (except when traffic volumes are high). The signal timing manual(Koonce, 2008) recommends cycle lengths between 50 and 100s because cycle lengths shorter than the 30s would not meet minimum requirements for a green time. While longer ones (longer than 120 seconds) would be unsuitable because drivers assume the lights are not working and may move through red. The optimum cycle length estimated as per the SARTSM manual based on the Webster method using equation 2.1:

$$C_0 = \frac{1{\cdot}5 * L + 5}{1 - \Sigma Y_i} \qquad 2.1$$

Where: *Co* = Optimum cycle length (s), *L*= Total lost time per cycle (s)

Y_i = volume/ saturation flow ratio per critical movement in stage *i*

Note that in equation 2.1 optimum cycle length can be verified linearly (C_L) and non-linearly (C_C) by way of regressions where;

$$C_L = \frac{aL+b}{1-cy} \qquad 2.2$$

$$C_c = aLe^{bYa} + c \qquad 2.3$$

Where: *Co* = Optimum cycle length (s), *L*= Total lost time per cycle (s)

Y_i = volume/ saturation flow ratio per critical movement in stage *i*

a, b, and c are regression parameters

Note also that lost time indicates when the intersection is not effectively utilised for any movement. For example, when the signal for an approach turns from red to green, the vehicle's driver in the front of the queue will take some time to perceive the signal (usually called reaction time), and some time will be lost here before the driver moves. The flow ratio (*Y*) is the proportion of an hour required to serve a traffic movement. The flow ratio determines the minimum green ratio required to serve that movement. As shown below in figure 2.10, it is clear that the regression formulas, search algorithm, and other methods of determining the optimum cycle length give approximately the same performance (Zakariya and Rabia, 2016).

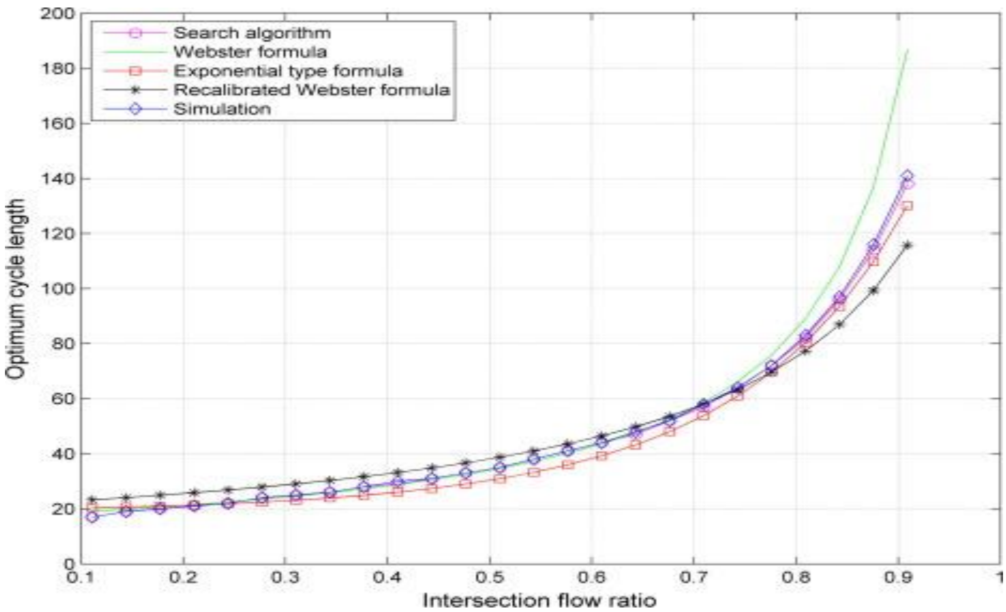

Figure 2. 10 : Comparison between different models of determining optimum cycle length (Zakariya and Rabia, 2016)

2.3.5 Lost time and Effective green

Lost time is the time in seconds during which any movement does not effectively use an intersection. It comprises the time lost at the start of a phase and the end of it. Start-up lost time (SULT) is the additional time in seconds consumed by the first few vehicles in a queue caused by the initial driver reaction to the green light and the need to accelerate. When a traffic light signal changes from red to the green, time elapses between the signal changing and the first vehicle crossing the stop line, lost time may also occur at the end of the phase where some traffic may continue into the intersection during the yellow time. However, a portion of the yellow time and all of the all-red clearance intervals can not be effectively sued by traffic. It is referred to as the clearance lost time. The clearance lost time added to the start-up lost time makes the total last time per phase.

The (HCM, 2016) uses a value of 2s for SULT and 2s for clearance lost time, giving the total lost time of 4s per phase. These values are used in cases where specific site/ field estimation of the values is not done. The definition of the SULT points to the fact that driver behaviour has an impact on the value assigned. Since driver behaviour changes from one country to another and even from one region to another, assuming an authoritative figure may not be accurate. Secondly, SULT is an important parameter in calculating other parameters like capacity and even signal timing; it is important that the value used be accurate and representative of the region. Field measurement of the SULT is therefore adopted. The determination of SULT in the field involves determining discharge headways of vehicles in a queue and then subtracting the observed headways of the first up to 4 vehicles from the saturation headway. Summing up the calculated differences gives the SULT. Figure 2.11 gives a graphical representation of the SULT measurement, and equation 2.4 gives the formula used to obtain it.

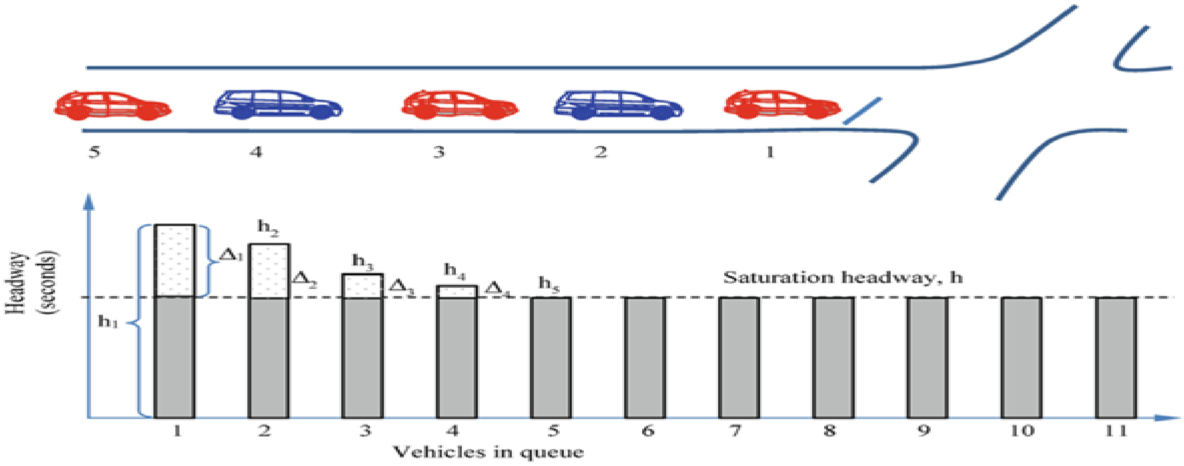

Figure 2. 11: SULT field measurement

$$SULT = \sum_1^4 \Delta n \qquad 2.4$$

Where Δn = headway difference between nth vehicle and saturation headway

A previous study (Shawky and Ghafli, 2016) found the SULT to vary between 1s and 2s. Further, they found that the SULT is affected by turning movements, queue length, geometry, location, time of day, weather conditions, and traffic light visibility. It can be concluded that the actual field determination of the SULT gives more accurate values and should be preferred over the HCM, 2016 method of assuming a general value of 2s.

Effective green is the amount of usable time available to serve vehicular movements during a cycle phase. The effective green time for any movement is the sum of the displayed green and yellow times less the lost time. If the green time provided is too little, vehicle queues will not clear the intersection, and cycle failures will occur. If too much green time is provided, portions of the cycle will be unused, resulting in inefficient operations and frustration of drivers on the adjacent approaches. The effective green time for a given movement is the sum of displayed green, yellow, and all-red clearance intervals less the total lost time. The equation 2.5 below is used to determine effective green.

$$g = G + Y + AR - t_L \quad 2.5$$

Where : g = Effective green time in seconds

G = displayed green time in seconds

Y = Yellow time in seconds

AR = All red interval

t_L = total lost time (sum of SULT and clerance lost time)

2.4 Influence of Signalised Intersection timing on Driver's Behaviour

At signalised intersections, drivers can be proactive or responsive as their behaviour is mainly determined by traffic signal compliance. They can choose to be normal, aggressive, or conservative, especially in response to the invisible and undetermined dilemma zone. Vehicles either stop safely with comfortable deceleration or run through the intersection with their current speed or a comfortable acceleration rate without violating a red light. Drivers facing these different options and in different zones are expected to make the right decisions; inappropriate decisions could cause serious safety concerns for the intersection. Problems arose when different drivers caught up in these zones make conflicting decisions. These decisions could cause rear-end collisions or red-light running, all of which are unsafe actions. There is no clear agreement among researchers/ practitioners about the definition and demarcation of these zones, especially option and dilemma. Secondly, these zones are dynamic, and the drivers are unaware of them as they approach the intersection. The concept of dilemma zone was proposed by (Gazis et al., 1960)in what is referred to as the GHM model. They defined a dilemma zone as a zone where a driver can neither safely stop nor go through the intersection before the lights turn red. The GHM model defines the critical minimum(safe) stopping distance (X_c) given by equation 2.6 as well as the maximum distance a vehicle can travel during amber (X_o) given by equation 2.7

$$X_C = v_0\delta_1 + \frac{v_0^2}{2a_1} \quad 2.6$$

Where; X_C is the defined distance at the onset of amber light to the stop line,

v_0 is the approach speed, δ_1 is the perception reaction time for stopping, and

a_1 is the maximum deceleration rate of a vehicle(Gazis et al., 1959).

At a distance closer to the intersection stop-line than X_c, a vehicle cannot safely stop.

$$X_0 = v_0\tau + 0.5a_2(\tau - \delta_2)^2 - W - L \qquad 2.7$$

Where: X_0 is the maximum distance from the stop line at the onset of amber light,

τ is the amber duration, δ_2 is the perception reaction time for proceeding,

a_2 is the maximum acceleration rate of a vehicle,

W is the curb-to-curb intersection width, and

L is the length of the vehicle (Gazis et al., 1960).

According to GHM, a dilemma zone is then described as the zone between X_c and X_o when Xc >Xo. The option zone is also between X_c and X_o and occurs when $Xo > Xc$, the maximum distance travelled during amber, is greater than the safe stopping sight distance. Figures 2.12 and 2.13 show the dilemma of a signalised intersection, and equation 2. 8 gives its formula.

$$DZ = X_C - X_O \qquad 2.8$$

Where; DZ is the dilemma zone

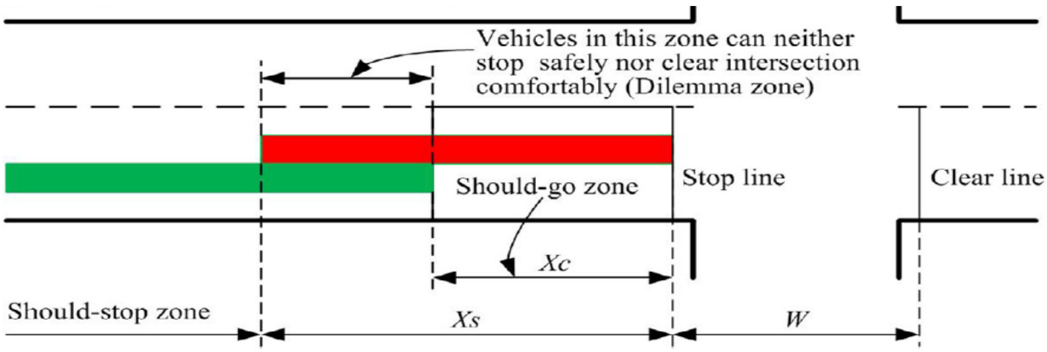

Figure 2. 12 : Representation of Dilemma Zone at signalised intersections(Li et al., 2016)

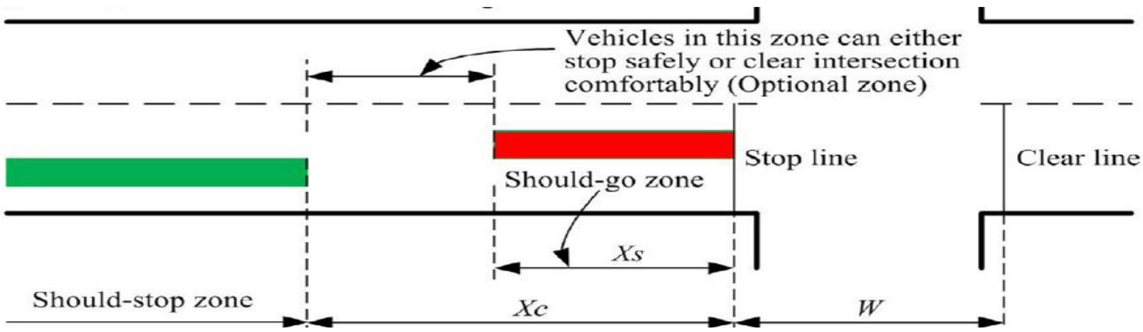

Figure 2. 13: Representation of Option zone at signalised intersections (Li et al., 2016)

Among scholars, there is no clear agreement on the dilemma zone and option zone definition and demarcation. Some studies have also defined dilemma zone as Type I and II. (Si et al., 2007) stated that the dilemma zone and option zone are fundamentally different. According to their study dilemma zone eliminated by appropriate yellow and red clearance times. However, the option zone always exists because of varied decisions made by drivers depending on approach speed. The model developed by (Gazis et al., 1960) has been improved upon and used to determine appropriate yellow and all-red clearance intervals. However, it has not solved the challenge.

(Jahangiri et al., 2016) Their study identified time to an intersection, distance to the intersection, the speed at the onset of yellow and vehicle acceleration as factors contributing to red-light running (RLR). Other studies have also considered driver characteristics like age, gender, aggressiveness, and cell phone use. In a bid to solve these issues (Elmitiny et al., 2010) undertook a study using observational data from videos and observed driver behaviour at signalised intersections; the finding was that there was a positive relationship between speed and vehicle RLR. Some measures like law enforcement using cameras were shown to reduce RLR's frequency (Ko et al., 2017). It concluded that motorists were approximately 3.4 times prone to RLR when cameras were removed compared to being present. The reduction of RLR by red camera law enforcement resolved the issue of RLR but introduced the occurrence of rear-end collisions. Other measures that have been implemented include; count down timers (Zhaosheng et al., 2014). However, these were also not very effective because of driver perception and competition, especially among risk-averse drivers who have to judge the time left and, in most cases, make all attempts to cross the stop-line leading to over speeding and unsafe situations at the intersections.

In South Africa, the law permits drivers approaching traffic signal lights to stop at (restrictive rule) or proceed beyond (permissive rule) the stop-line as the case may be at the onset of yellow lights. Red light running is not a violation, provided the driver be still within the all-red interval. In studying driver behaviour during the yellow interval, different methods used. The collection of data has been done using various means, most notably videos. From the videos, relevant data about speed, vehicle position, and time among others, extracted. Simulation used in cases of modelling where empirical data may not be available. The collected/ extracted data analysis then depends on the desired output. Most researchers use binary regression models to analyse the collected/ extracted data to understand driver stop/running behaviour during the yellow interval (Gates et al., 2007a, Köll et al., 2004, Long et al., 2013). Other models like binary probit models, ordered probit models, agent-based behavioural models, and fuzzy logic models. With empirical data, statistical models are easy to compute, and the modelled results are easy to understand. However, agent-based behavioural models and fuzzy sets and logic models are a better fit (Zhang et al., 2014). The preceding yellow interval and the laws around it are not specific, and drivers have to be aware of what laws apply and behave accordingly.

Secondly, a yellow interval designed as a warning, and action left to the driver's discretion though subject to the applicable laws. In cases where drivers make conflicting decisions at onset and during the yellow interval, it compromises the safety of signalised intersections. Needless to add that, the yellow period should be adequate to give drivers warning and allow them to stop safely even under adverse weather conditions. Ideal requirements for the yellow interval could result in relatively long yellow periods and, as such SARTSM recommends an approach where the yellow period reduced. The all-red period correspondingly increased while effectively retaining the wintergreen period. It is because, with a long yellow period, there is a tendency by drivers to abuse it, using it as an extension of the green period, and this can result in unsafe conditions. Following this design choice, the manual recommends that for purposes of red-light law enforcement, prosecution of red-light violations should start during the last one second of the all-red interval. Equations 2.9 and 2.10 are the yellow and all-red interval equations, respectively (SARTSM, 2012a)

$$Yellow = t_y + \frac{1}{2}\left[\frac{v}{A_y + g.^{G}/_{100}}\right] \quad 2.9$$

$$All - red = t_r + \frac{1}{2}\left[\frac{v}{A_r + g.^{G}/_{100}}\right] + \frac{W}{v} - Yellow \quad 2.10$$

Where: t_y, t_r~Reaction time (0.75s for yellow, 1s for all-red); v ~posted speed

A_y, A_r - Deceleration rates (3.7m/s^2 for yellow, 3.0m/s^2 for all-red)

g - Gravitational acceleration constant

G - Gradient on approach to signal

W - Clearance width or intersection width

SARTSM mentioned that the calculated intervals from equations 2.9 and 2.10 must be subject to the following minimum values: at 60km/h or less 3s yellow interval; at 70km/h 3.5s yellow interval and 80km/h 4s yellow interval. Note that the yellow interval calculated using the above formula is inadequate for wet weather conditions or drivers requiring longer reaction time. The manual recommends a longer all-red period given by equation 2.11. The longer all-red period used to extend the yellow interval in such conditions (wet).

$$All - red = t_r + {}^{1}/_{2}\frac{V/3.6}{A_r + g \cdot {}^{G}/_{100}} + \frac{W}{V/3 \cdot 6} \quad 2.11$$

Where: t_r= reaction time take as 1s, A_r= deceleration rate taken as 3m/s^2, W = clearance width (m)

At signalised intersections, driver behaviour influenced by traffic flow, road conditions, pedestrian crossings, and ambient conditions (especially adverse weather). The dilemma zone is invisible to drivers. It is variable as it is a function of approach vehicle speed and is thus undefined. The dilemma zone is indeed a signalised intersection dilemma. Driver action during the yellow interval differs per country/ region/ state depending on applicable yellow laws. Typically two yellow laws could be applied:

permissive yellow laws or restrictive yellow laws. The permissive yellow law allows drivers to enter the intersection during the entire yellow interval and be in the intersection during the red indication as long as the vehicle entered the intersection during the yellow interval. Under permissive yellow law, an all-red clearance interval must exist as a timing parameter to ensure the safe right of way transfer at the intersection. Restrictive yellow law has two variations; in variation one, a vehicle may not enter an intersection when the indication is yellow unless the vehicle can clear the intersection by the end of yellow. A vehicle may not enter an intersection in variation two unless it is impossible or unsafe to stop. With the restrictive yellow law, the presence of an all-red interval is optional. Drivers have to know which law is applicable in their region, and their behaviour should then correspond to the applicable law.

The red clearance interval (all-red) is an interval at the end of the yellow interval during which the phase has a red signal display before the display of green for the following phase. The purpose of the all-red is to allow time for vehicles that entered the intersection during the yellow interval to clear the intersection before the next phase. The use of this interval is optional (Koonce, 2008), and there is no consensus on its application and duration. A disadvantage of the all-red interval; is that there is a reduction in available green time for other phases. The manual for urban control devices (FHWA, 2014) advises that the yellow interval should last approximately 3s to 6s, with higher intervals used on higher speed roads. The same manual recommends that the all-red interval should not exceed 6s.

Whatever yellow light laws are applicable in an area, it is evident that drivers faced with a decision to make at the onset of the yellow interval depending on their positioning and the approach to the intersection stop-line. Figure 2.14 below gives a schematic representation of the area and an intersection approach at the onset of the yellow interval. The figure shows the area before the intersection stop-line, the intersection width W and the clearing line, which is W plus vehicle length L. The area before the stop line has a 'choose to go' distance, which is the distance approximately equal to the operating speed/ speed limit by the yellow interval. When conditions are permissive, drivers in that region can make it through the stop-line at the onset of the yellow interval. The drivers in the 'choose to stop' region cannot safely make it through the intersection without accelerating beyond acceptable limits. All the drivers in the yellow influence area have a decision to make. Differences in behaviour and the multiplicity of options and decisions made are factors that could lead to safety-related problems of the yellow interval.

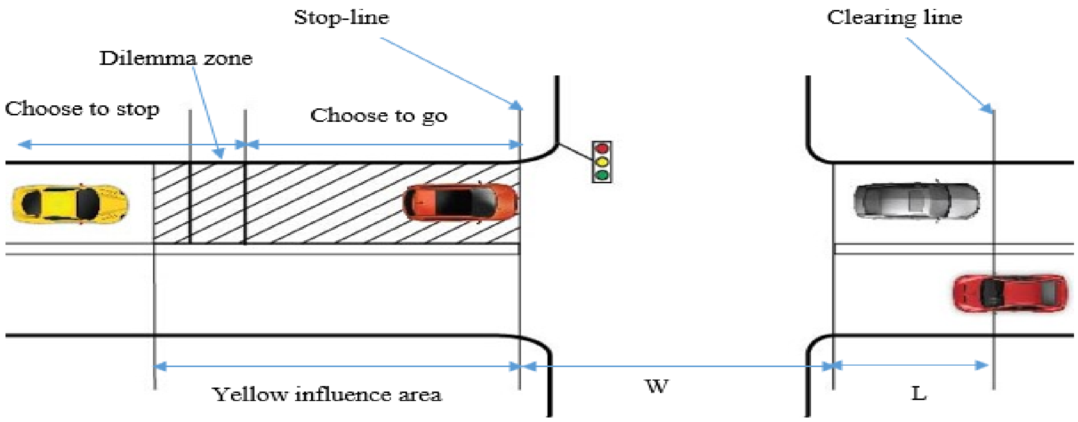

Figure 2. 14 : Representation of yellow influence area at a signalised intersection

(Lu et al., 2015) described four positions a vehicle may be found in at the onset of the yellow interval; "should go zone", "should stop zone", "dilemma zone", and "optional zone". In the "should go" zone, a vehicle can successfully run through the intersection with current speed or a comfortable acceleration rate before the signal turns red but cannot safely stop at the stop-line without sudden deceleration. In the "should stop" zone, a vehicle cannot run through the intersection before the signal turns red but can safely stop behind the stop-line with a comfortable deceleration rate. In the "optional" zone, a vehicle can either fully stop with a comfortable deceleration rate or run through the intersection safely with its current speed or a comfortable acceleration rate before the signal turns red.

The evaluation discussed in the previous sections covers dry weather conditions. When the weather changes, driver behaviour also changes, which could impact the performance of signalised intersections. When rain falls, and the pavement becomes wet, there is reduced friction between the wet surface and the tread. The windscreen and windows of vehicles are covered by raindrops which could lead to poor visibility. Splash and spray from other vehicles worsen the visibility problem by adding a film of dirt. As a result of all these, drivers may tend to maintain longer distances between vehicles and drive at slower speeds due to the possibility of increased perception reaction times. The methods and models used to evaluate the various parameters discussed in the previous section remain the same under wet weather conditions. However, the effect of the weather needs to be evaluated, and its implication on signalised intersection efficiency and safety quantified.

2.5 Yellow Traffic Light Concepts

At the onset of red, some drivers accelerate to clear the intersection, and others decelerate to stop at the stop-line. Red-light violation at signalised intersection occurs when a vehicle enters the intersection during the red phase. Despite attempts by researchers and practitioners to minimise red-light running, safety at signalised intersections is still a challenge in South Africa. Traffic signal timing is the technique used to distribute right-of-way at signalised intersections. The process allows traffic signal lights to emit intermittently, by which each intersection movement serviced without allowing conflicting movements to enter the intersection simultaneously. The amount of time required to display all phases for each directional movement of an intersection before returning to the starting point is sacrosanct to signalised intersection performance.

Whether fixed or actuated, traffic signal timings have three traffic lights; red, yellow, and green. Red and green lights give specific instructions. Whilst the yellow light merely warns drivers that 'it is permitted to proceed and clear the intersection safety, however in a situation where the vehicle can neither enter nor be in the intersection on red, it must stop upon receiving the yellow interval. As illustrated below in figure 2.15, when a vehicle approaches a signalised intersection at the onset of the yellow traffic light, assuming a posted speed limit of 60km/h, if the vehicle is at d_1 and the approach speed is greater than 60km/h, the driver can choose between stopping and clearing the intersection. If the vehicle is at d_2 or d_3 and the approach speed exceeds 60km/h, the driver clears the intersection safely. Note that DZ denotes the dilemma zone, and BZ is the braking zone.

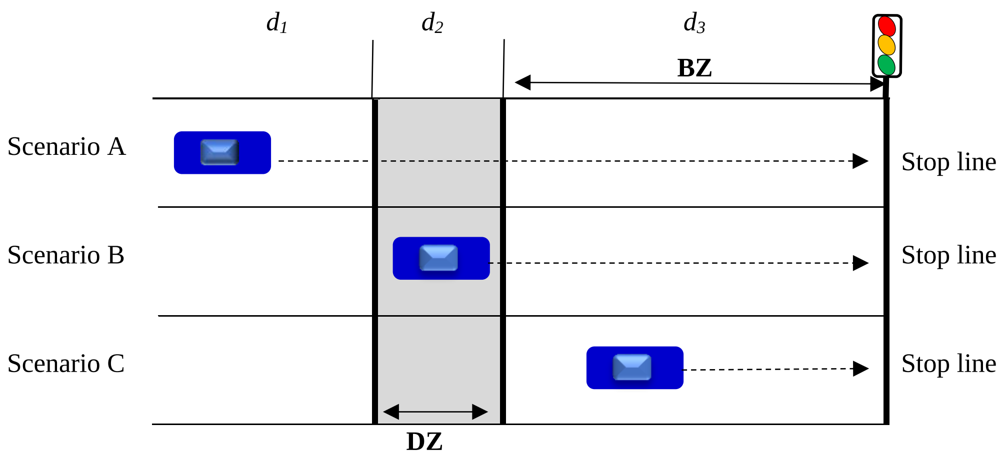

Figure 2. 15: Possible scenarios at signalised intersections at the onset of yellow light

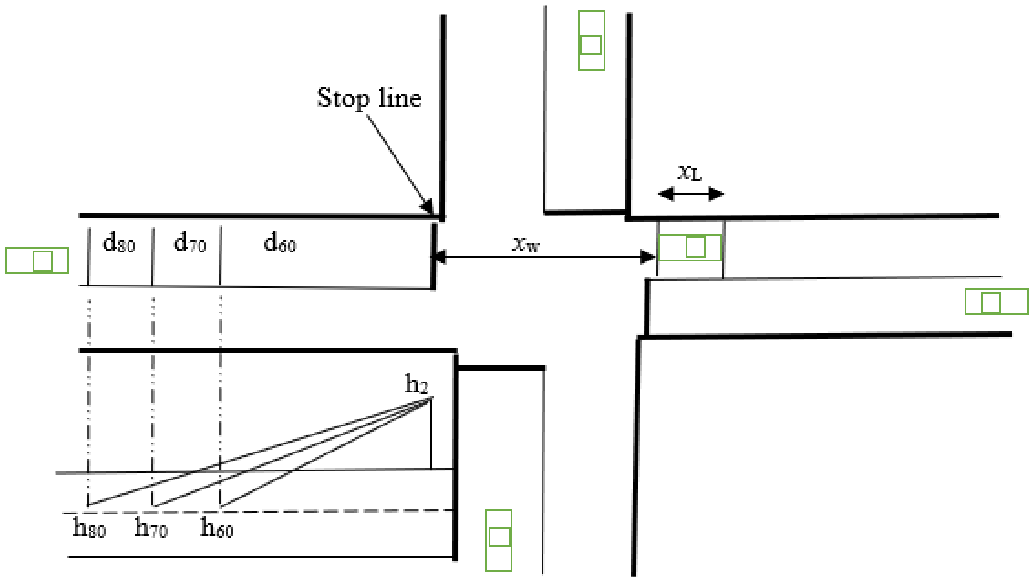

Figure 2. 16: Total Yellow interval influence area

The concept is based idea that drivers have intrinsic passive communication with traffic signal emitting stations. As illustrated in Figure 2.16 above, the total influence area hinges on the idea that drivers have intrinsic passive communication with traffic signal emitting stations (h_2). The influence area has two components (yellow phase and all-red phase). The yellow phase terminates at the stop-line, while the all-red phase comprises the intersection width and vehicle length. The communication distance is dependent on drivers' approach speed and time to stop line, among others, for example, where the approach speed is 60km/h and the yellow light is 3s. The communication distance is (16.67 x 3) 50m. It is further assumed that in most cases, drivers decelerate on approach to the stop-line if they intend to stop, accelerate or maintain approach speed if they intend not to stop. In the yellow interval novel concept presented in figure 2.16 above, the communication distance (d_{60}) is dependent on the driver's approach speed and pre-set yellow time where d_{60} represents distance relative to 60km/h and the communication point denoted as h_{60} taken into consideration driver's eye height. The total yellow influence area assumes that stopping at or proceeding beyond the stop line is a function of approach speed, yellow phase time, and stopping sight distance.

In most cases, drivers decelerate on approach to the stop-line if they intend to bring the vehicle to a halt and accelerate if they want to proceed beyond the stop-line. As illustrated in Figure 2.17, the total influence area can be divided into two chambers (yellow chamber ($X_0 \sim X_R$) and the all-red chamber ($X_R \sim X_L$)). The yellow chamber is made up of dilemma sub-zone ($X_0 \sim X_d$) and braking sub-zone ($X_d \sim X_R$), whereas the all-red chamber has an intersection sub-zone ($X_R \sim X_w$) and the vehicle occupancy sub-zone (X_w ~ X_L). The expressions for the yellow influence distance and time given in equations 2.12 and 2.13.

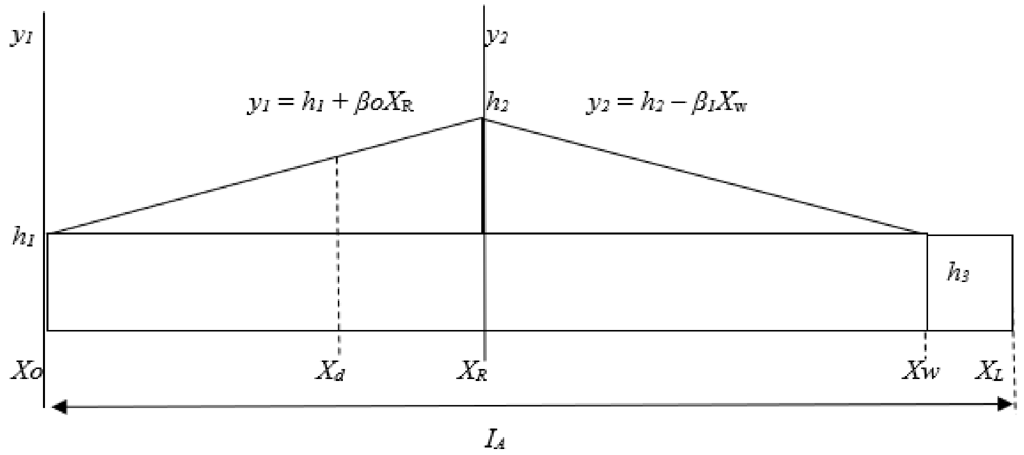

Figure 2. 17: Total Yellow Interval Influence Area

Where:

h_1 ~ driver's eye height (1.05m); h_2 ~ traffic signal pole height; h_3 ~ vehicle height

X_d ~ dilemma distance and ($X_R - X_d$) ~ Braking distance

X_R~ yellow phase sub-zone; Xw ~ Intersection sub-zone and

X_L ~ vehicle sub-zone; β_o and β_1 are coefficients; I_A is the influence area

The total yellow interval distance is: $X_L = \sum X_d, X_R, X_w, X_L$ 2.12

The time needed for total yellow influence distance is: $\tau_c = \frac{1}{v}(\sum X_d, X_R, X_w, X_L) - \left(\frac{v}{2a}\right)$ 2.13

Where: v ~ approach speed and a ~ Deceleration/acceleration

2.5.1 Effect of Rainfall on Red-light Running

Red-light running has been identified as one of the prominent factors involved in signalised intersection crashes(Zhang et al., 2018). Yellow traffic lights warn drivers of an upcoming red light, which means an end to the right of way. At the onset of yellow traffic lights, depending on the driver's position, they decide whether to slow down and stop, continue at their speed and pass or accelerate to make it through the intersection. If a driver decides to either maintain their speed to pass or accelerate to clear the intersection and fails, this leads to red-light running.

Traffic accidents during the yellow time account for more than half of the traffic accidents at signalised intersections(Yang et al., 2014). There are studies carried out to understand driver behaviour at the onset of the yellow time, and their contribution to intersection safety. (Elmitiny et al., 2010) in their study, the distance from the intersection at the start of yellow, operating speed, and traffic flow are among the important factors determining driver decision to either stop or cross the intersection. Acceleration and deceleration during yellow time have also been found to contribute to red-light running(Arash et al., 2016). From these studies, driver behaviour during yellow time can be a key contributor to red-light running violations.

Driver behaviour affected by several factors, including road infrastructure, vehicle type, and prevailing weather and traffic conditions. Some of the factors affected by rainfall at signalised intersections include; saturation headway, saturation flow rate, and start-up lost time(Yang et al., 2014). Rainfall affects the driver's perception and reaction time. In a study carried out to determine the weather effect on perception reaction time (PRT) at the onset of the yellow interval, it concluded that PRT increased as the time to the intersection increased. It was found to be longer when the driver was travelling along an upgrade approach section (El-Shawarby et al., 2013). Overall, PRTs under wet conditions were higher than in dry weather conditions. According to the South African Road Traffic Signs Manual(SARTSM, 2012a), deceleration used to determine the yellow time is $3m/s^2$ and a perception reaction time of 0.75 seconds applied. The signal timing manual (Koonce, 2008), however, recommends a PRT of 1 second.

The other factor affecting driver behaviour during the yellow interval contributing to red-light running was speed and acceleration. Prevailing weather conditions affect speed. A study conducted in Salt Lake City found that start-up delay increased by 5-23% (the highest for snowy conditions) (Perrin et al., 2001). (LI et al., 2012) concluded that compared to clear weather conditions, driver dilemma zone boundaries started farther away from the stop-line by approximately 0.2 seconds, driver PRT increased by about 0.11 seconds, and driver deceleration levels decreased by about 8%. Thus, it is fair to conclude that rainfall negatively affects signalised intersection performance in terms of quality of service. However, the effect on red-light running has yet to be determined.

The yellow time interval is generally recognised as a key factor that affects the frequency of red-light-running and researchers recommend setting the yellow time interval based on the probability of stopping and the 85th percentile driver's travel time (Gates, 2012, Rakha et al., 2008, Perrin et al., 2001). Previous studies have also indicated that a driver's decision to stop at yellow onset based partly on an estimate of speed and distance to the stop line (Rakha et al., 2008, Long et al., 2013, Liping Zhang, 2008, Mashros, 2014, Moore and Hurwitz, 2013, Oyaro, 2020). Many researchers have measured driver response to the yellow light in terms of travel time to the intersection (Gates, 2012, Rakha et al., 2008, Long et al., 2013, Liping Zhang, 2008, Mashros, 2014). The decision-making section called the dilemma zone. Research has concluded that the dilemma zone contributes to red-light running behaviour especially for high-speed intersections(Liping Zhang, 2008). However, evasive problems of red light running associated with the yellow traffic light are dynamic and seem not to go away. The issue of red-light running studied for over five decades and is still significant today. Previous studies have instituted many commendable mitigation measures to douse red-light running occurrences signalised intersections. This study argues that visual perception is a key part of driving tasks where cognitive processing plays an important role. It postulated that RLR would be far too risky to execute under rainy conditions given a rainfall scenario; visual perception would be poor, accompanied by speed reduction.

2.5.2 Modelling Influence of Rainfall on Red-light Running

As shown in previous papers, the logit model is suitable for many applications. However, it also has limitations, resulting in erroneous parameter estimates if the basic assumptions are not satisfied. Random Parameter Logit Model (*RPLM*) is a possible antidote for logit model limitations. RPLM addresses several weaknesses of the traditional logit model by allowing parameter values to vary across observations (S. P. Washington, 2011). Modelling yellow traffic signal-induced problems at signalised intersections is paramount to understanding driver behaviour. It is upon a driver's discretion to either stop or proceed at the onset of the yellow time interval. Based on observations and discussion with drivers at the four surveyed sites, limited postulations:

- Drivers not versed with posted speed limits on approach to signalised intersections.
- Drivers not versed with yellow time intervals.
- Drivers not versed with permissive or restrictive rules governing signalised intersections.
- The absence of a marked decision-making section is problematic.

In previous studies (Rakha et al., 2008, Long et al., 2013, Liping Zhang, 2008), driver behaviour was assessed with binary logistic regression or fuzzy logics. (Gates et al., 2007b) recommend the logit regression model. When conducting a field investigation of RLR in Shanghai, China, (Sharma et al., 2011) used the random effects logistic regression model. When modelling driver behaviour during the onset of yellow intervals at signalised intersections (Amer et al. (2011) used the agent-based behavioural model. (Liu et al. (2011) conducted an empirical study of modelling driver response during yellow signal intervals based on the probit model. The model successfully predicted yellow light running occurrences. The fuzzy logistic model was used by (Hurwitz et al. (2012) to describe driver behaviour at high-speed signalised intersections. (Moore and Hurwitz (2013) used the fuzzy logic model when conducting a driving simulator study for dilemma zone identification to assist drivers when confronted with yellow signal lights. (Wu (2014) used the logistic regression model proved to be a good fit probabilistic method of predicting red-light running. Wu (2014) logistic regression model proved to be a good fit probabilistic method of predicting red-light running. Other researchers used machine-learning techniques. Liu et al. (2011) analysed the red light running (RLR) at the signalised intersection using a regression model development. The model showed a good fit between model (with R^2 of 0.953) and data. The model was successful in predicting red-light running occurrences. Sharma et al. (2011) used the random effects logistic regression model, when conducting field investigation of RLR in Shanghai, China. The model was successful for the study. Substitute to the normal distribution is the standard logistic distribution, where the expression curve is similar to the normal distribution having a distinguishable advantage of closed-form expression (Rodriguez, 2007). The utility function (z_i) ranges from - ∞ to + ∞. The logistic distribution is perfectly symmetrical, with a zero mean. The probit distribution has the normal error distribution (Rodriguez, 2007). The probit model defines the standard normal cumulative distribution (c. d. f.) function in its expression. The complementary log-log transformation (C-log-log) is similar to the logit. However, as probability increases, the transformation approaches infinity slower than either logit or probit. Figure 2.18 shows the distribution of probit, logit, and c-log-log curves on the probability/link axes. The logit and probit have similar distribution as the curves aligned in the distribution. The c-log-log, however, shows similar distribution but outlies from the probit and logit distributions.

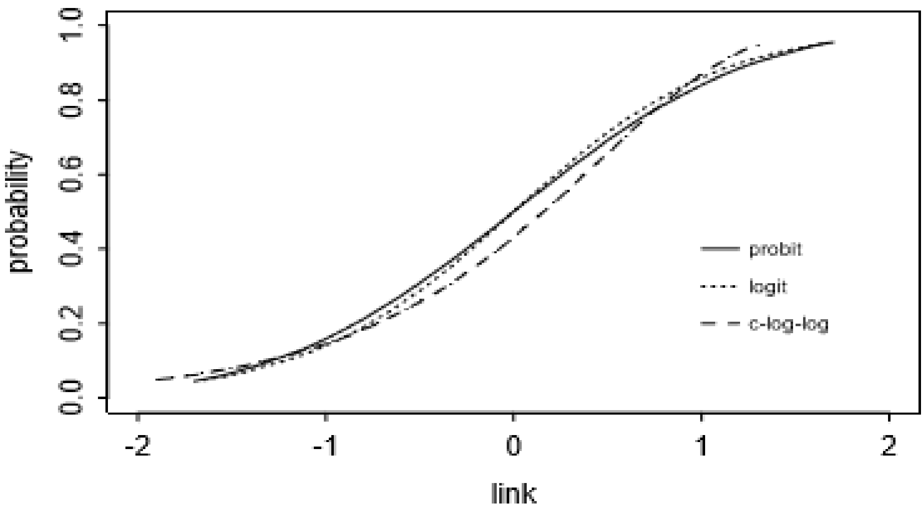

Figure 2. 18: Typical Standard for Probit, Logit, and c-log-log Distribution Links (Rodriguez, 2007)

When drivers receive the yellow light signal, they have only two response choices: pass or stop. Given this situation, a binary logistic regression model is suitable to model the drivers' stopping probability as a function of multiple related factors, as expressed by equations 2.14 and 2.15.

$$p_i(i = 1) = \frac{e^{[\alpha+\sum_k b_k X_{ik}]}}{1+e^{[\alpha+\sum_k b_k X_{ik}]}} \qquad 2.14$$

$$p_i(i = 0) = \frac{1}{1+e^{[\alpha+\sum_k b_k X_{ik}]}} \qquad 2.15$$

Where the dependent variable is the driver’s choice, Pi (i =1) = probability of red-light violation and Pi (i =0) = probability of stopping; ά = constant of the linear function,

X_{ik} = k^{th} variable affecting driver behaviour and b_k = coefficient of the k^{th} variable

Several factors influence the drivers' behaviour when approaching intersections. These include the vehicle speed, time and distance to the intersection at the onset of yellow light, and rainfall. Travel time is a function of free-flow speed and saturation; however, the degree of saturation is set at zero because only lead vehicles approaching the signalised intersection considered in the study. In the logistic regression model, the dependent variable is the driver’s choice behaviour. At the onset of yellow time light, where y = 1 represents red-light running (RLR) and y = 0 means the driver stops. A dummy variable introduced to depict drivers' behaviour before generating the logistic regression models. The dependent variable was the driver's choice behaviour at the onset of the yellow traffic signal, represented by φ; where φ =1 means the driver brings the vehicle to a halt at the stop-line and φ =0 denotes that the driver proceeded beyond the stop-line into the intersection. The regression model was of the form:

$$L_p = \ln\left(\frac{P}{1-P}\right) = \beta_o + \beta_1 x_R + \beta_2 v_2 + \beta_3 \varphi_3 \quad \text{For } 0 < P < 1 \qquad 2.16$$

$$\text{Thus, } L_p = \frac{P}{1+ e^{(-\beta_o+\beta_1 x_R+\beta_2 v_2+\beta_3 \varphi_3)}} \qquad 2.17$$

Where; P - denotes the probability of a driver’s choice;

βo - a model constant; X_R denotes the distance to the stop-line;

v - approach speed at the onset of yellow signal;

φ - dummy variable; and β_1, β_2 are regression coefficient factors.

The probability of a vehicle stopping estimated by

$$P(\varphi = 1) = \frac{e^{\beta_o+\beta_1 x_R+\beta_2 v_2+\beta_3 \varphi_3}}{1+ e^{\beta_o+\beta_1 x_R+\beta_2 v_2+\beta_3 \varphi_3}} \qquad 2.18$$

The probability of the vehicle not stopping estimated by

$$P(\varphi = 0) = \frac{e^{\beta_o+\beta_1 x_R+\beta_2 v_2+\beta_3 \varphi_3}}{1+ e^{\beta_o+\beta_1 x_R+\beta_2 v_2+\beta_3 \varphi_3}} \qquad 2.19$$

The study approached the yellow interval's evasive issues by viewing the influence of communication between the driver and the emitting traffic signal. Even though yellow interval times prescribed in the traffic signal system creates dilemma area, drivers are not aware of the physical influence area; hence, decisions made by the drivers are merely intuitive and poorly formed. Studies conducted on the hypothesis that rainfall irrespective of intensity will reduce red light running at signalised intersections. A binary logistic regression model used to determine the probability of stopping or proceeding beyond the stop-line under dry and rainy weather conditions at four Durban, South Africa locations under dry and rainy weather.

2.5.3 Passenger car equivalency modification due to rainfall

The concept of the passenger car equivalent (PCE) was first introduced in the Highway Capacity Manual (HCM) to account for the effect of trucks and buses in the traffic stream. HCM defines PCE 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". Passenger Car Equivalent (PCE) or Passenger Car Unit (PCU) is a metric used to assess the traffic-flow rate on a highway. Based on the premise that rainfall slows vehicles, passenger car equivalents were modified. Heavy vehicles occupy more space on the road. They are characterised by longer reaction time and slower acceleration at the onset of a traffic light. During rainfall, vehicle speeds are generally lower than in dry weather conditions.

Notwithstanding, heavy vehicles, whether they are lead vehicles or not, make the traffic stream travel even slower. Since saturation flow is the maximum constant departure rate of queues from the stop line of an approach lane during the green signal period, PCE values must be modified to account for prevailing weather conditions (rainfall). For South Africa, the Geometric Design manual (SANRAL, 2011) gives values as presented in Table 2.5.

Table 2. 5: Passenger Car Equivalent values as per South African Geometric Design Manual

Vehicle type	Passenger car Units			
	Rural roads	Urban streets	Roundabouts	Traffic Signals
Cars and light vans	1.0	1.0	1.0	1.0
Commercial vehicles	3.0	1.8	2.8	1.8
Buses and coaches	3.0	3.0	2.8	2.3
Motorcycles	1.0	0.8	0.8	0.3
Pedal cycles	0.5	0.3	0.5	0.2

In urban areas, traffic flow is often heterogeneous. Consequently, it is necessary to convert heterogeneous traffic into a stream of homogenous traffic by using appropriate passenger car equivalents (PCE). In addition, appropriate PCE values are also used for capacity analysis as well as service quality appraisal. The appropriate PCE values depend on the region and could be estimated for a particular study or local guidelines. The saturation flow rate can be converted to the corresponding value with appropriate PCE values. In cases where the PCE values have to be determined for a specific study, one of the common methods is the headway method. In this case, the average headway between two successive vehicles of a similar kind is obtained, as are headways between two successive passenger cars cross the stop line. The PCE value for the vehicle type is then obtained as the ratio of the average headway between vehicle type i and average headway for passenger cars. It is represented in equation 2.20

$$PCE = \frac{\bar{h}_i}{\bar{h}_c} \qquad 2.20$$

Where: h_c= average time headway for passenger car

PCE = Passenger Car Equivalent, and h_i= average time headway for vehicle type *i*

2.6 Service Quality Concepts

The terms 'level of service and quality of service' used interchangeably in many studies without distinction has implications. The level of service is a qualitative measure that defines how well a transportation facility operates. The quality of service (term used in this study) is a qualitative measure that defines how well a transportation facility operates from road users and providers perspectives. Service quality is inherently subjective and driven by road user's expectations. Service delivery or quality of service is a two-way performance measurement (road provider and users); however, at signalised intersections, road users and providers are interested in measuring delay. That is fine. Several studies have shown how the sole use of delay may not be a true representation of service quality.

South Africa relies on the Highway Capacity Manual (HCM) for the level of service assessment. HCM, 2016 incorporates the Transporation Research Board (TRB) National Cooperative Highway Research Programme (NCHRP) Report's 572 methodologies (with enhancements and extensions) lane-by-lane analysis of signalised intersections. Note that HCM uses the concept of level of service (LOS) as a qualitative measure to describe operational conditions of vehicular traffic at signalised intersections, "*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 signalised intersections less efficiently than models suggest the case in other countries around the world."*

Furthermore, the report proposes exponential models of capacity for single-lane and two-lane roundabouts and recommends that level of service (LOS) criteria are 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 intersection 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*. Also 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*. HCM 2016 defines the level of service as to how well a transportation facility operates from the travellers perspective, a departure from HCM 2010 definition. It appears that the HCM LOS is still a work in progress.

At a signalised intersection, LOS is a measure of the delay incurred by motorists, according to HCM 2016. For signalised intersections, control delay is used as the performance measure. The HCM provides a table with a range of values for the delay and the corresponding level of service. The average delay values apply to both single approaches and entire intersections. If the volume to capacity ratio is greater than 1, then the level of service is taken to be F regardless of the estimated delay.

In some cases, the delay will be high even when the v/c ratios are low. In these situations, poor progression or inappropriately long cycle length, or both may be the cause. An intersection may have unacceptably high delays without there being a capacity problem. Therefore, traffic analysts should consider both the capacity and delay (LOS) results to obtain a complete picture of intersection operations. The level of service classification as per the HCM range is shown in Table 2.6.

Table 2. 6 : HCM Level of Service ranges for signalised intersections

Level of Service	Average control delay per vehicle (sec)
A	≤ 10
B	>10 and ≤ 20
C	>20 and ≤ 35
D	>35 and ≤ 55
E	>55 and ≤ 80
F	>80

The Canadian Capacity Guide for signalised intersections (S. Teply., 2008) notes that the term level of service implies a qualitative measure of traffic flow at an intersection dependent on the intersection's vehicle throughput. The guide further notes that the HCM bases the level of service on control delay but mentions that it is more logical to base the LOS primarily on volume to capacity ratio (degree of saturation).

Delay is a subjective entity that can vary from different individuals, situations, and locations. In addition, the relationship between driver discomfort and delay may not be linear. Delay is also a time-specific parameter. A delay acceptable under today's increasing congestion may not have been acceptable in the past when there were fewer vehicles, and a similar trend expected in the future.

Volume to Capacity ratio or degree of saturation represents the extent of traffic capacity utilisation. Compared to delay, the v/c value is discrete and definitive and gives a clearer picture of the available capacity in an intersection independent of time, user, and location. The v/c is a fixed quantity that speaks to a logical assessment of the relationship between traffic volumes and the approach capacity. A level of service based on control delay can produce conservative results; a small number of vehicles turning from a minor street may have a disproportionate impact on LOS because of the high delay.

There needs to be a consideration in the range of typical operating conditions at signalised intersections. The guide further notes that a saturated condition does not necessarily imply a long delay and vice versa. It concludes that it is important to assess both the v/c ratio and delay to evaluate the operation of a signalised intersection. A well-designed intersection should have an acceptable v/c ratio and delay for all movements. In the Canadian LOS guide presented below in Table 2.7, with delay and v/c ratios used as measures of effectiveness.

Table 2. 7: Canadian levels of service at signalised Intersection (S. Teply., 2008)

LOS	**^{v}/c ratio**	**delay (s/veh)**	**Features**
A	0.0- 0.59	≤ 10 ≥	Almost no signal phase is fully utilized
B	0.60- 0.69	>10 and ≤ 20	An occasional signal cycle is fully utilized
C	0.70- 0.79	>20 and ≤ 35	Operation stable though with more frequently fully utilized signal phases
D	0.80- 0.89	>35 and ≤ 55	Motorist experiences increasing restriction and instability of flow. Substantial delays
E	0.90- 0.99	>55 and ≤ 80	At capacity. Long queues and delays may extend to several cycles
F	1.0 or >	> 80	Vehicle demand exceeds available capacity

A study by (JOVANIS, 2008)looked at the HCM level of service procedure and noted some concerns. Although the concept of LOS meant reflecting, the operational conditions perceived by a motorist, the HCM levels of service grades for signalised intersections not based on studies of driver perception.
Secondly, although the HCM specifies control delay as the measure of effectiveness for signalised intersection LOS analysis, it is unlikely that delay is the only factor that influences user perception of service quality. The study notes that some problems are associated with the HCM delay-based LOS. The delay conditions are not the same for level of service based on observed delay values. Suppose the With observed delays maintained, the delay thresholds should be continuously adjusted to reflect increasing delays observed due to increased traffic, population, number of trips taken, motorisation, and even car ownership.

Another study (Cameron, 1997) recommended extending the LOS criteria by adding more levels to quantify existing conditions and address future conditions. It was based on the reasoning that in some cases increase of congestion beyond the level of F could be accepted (congestion tolerance could increase due to public expectation of higher congestion levels) but is not quantified. Secondly, level F does not play a significant part in a measuring system choosing where major improvements needed. The study recommended adjusting the LOS from A to F; or expanding to A to J. It noted that it is usual to wait 2 to 3 minutes at an intersection in some cases. The additional levels from G to J range in value from 120s to above 300s. (Zhang, 2004) carried out a web-based study to identify driver perception of signalised intersection level of service. The conclusion was that delay-only based LOS was unable to reflect user perception of intersection LOS. The study revealed that drivers considered signal efficiency, left turn safety, clarity of pavement markings, and pavement quality have impact beyond delay. For example, drivers were willing to wait a long time at the intersection in exchange for a protected left-turn movement. South Africa does not have a capacity manual and therefore relies on the HCM. The HCM 2016 LOS method expanded to come up with the new functional quality of service criteria. With the new evaluation, the expansion of parameters considered to include the degree of saturation. This new parameter is a good measure of intersection performance and can give road traffic managers a good indication of the performance. The basis of the new functional quality of service evaluation is the HCM, 2016 criteria; the classification also remains the same with the extra parameters added. The criteria table developed for dry weather conditions and evaluated the same sites under different rainy weather conditions. Figure 2.19 gives the summary of the procedure adapted to come up with the proposed criteria table.

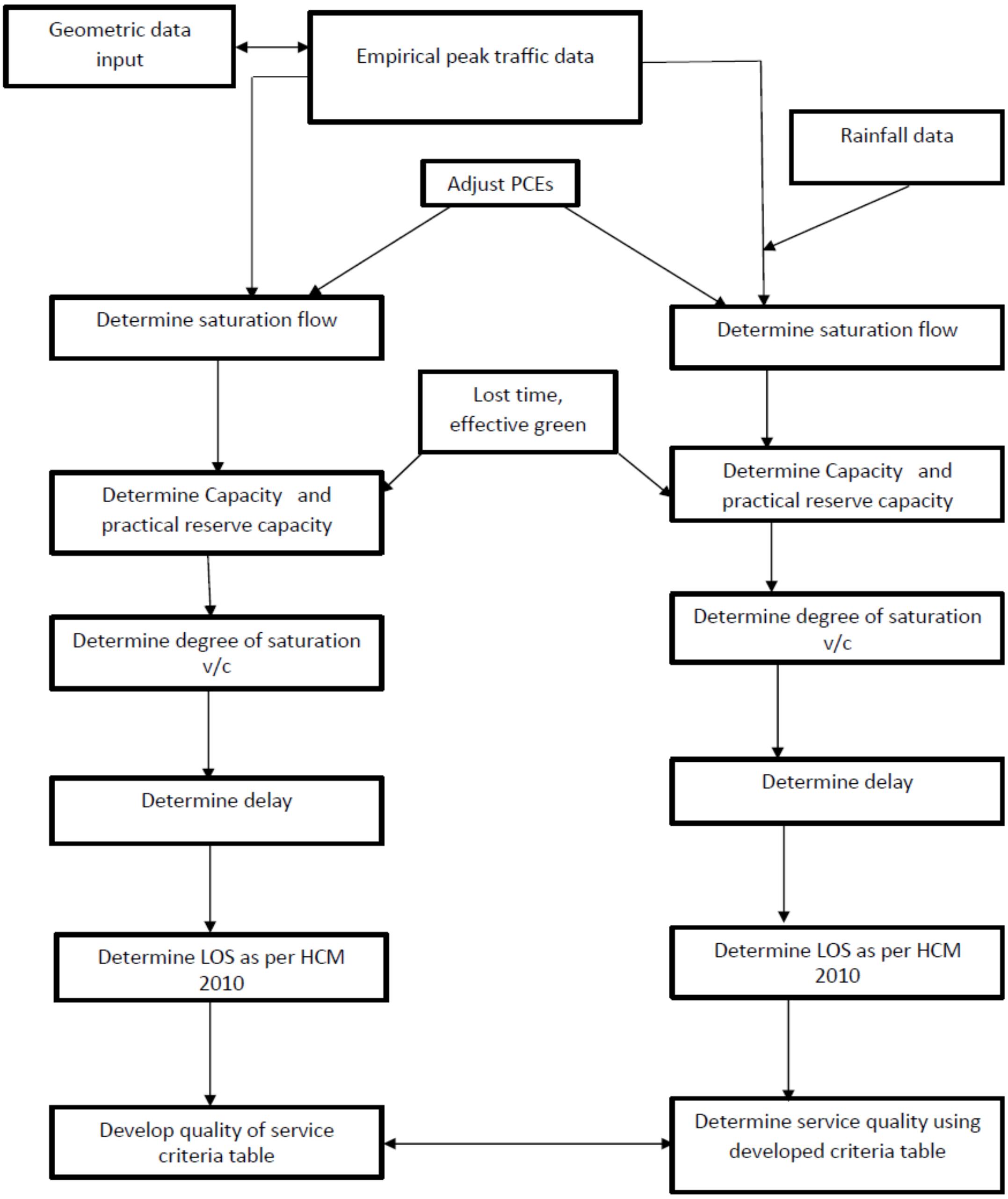

Figure 2. 19 : Schematic representation of the approach to quality-of-service determination

2.7 Service Quality Implications

The study also investigated the service quality implications of rainfall. In many studies, a delay is a proxy for the level of service (quality of service). Delay at signalised intersections is computed as the difference in a vehicle's departure and arrival times or the total extra time spent than what is required if the vehicle was allowed to pass the intersection without any hindrance. Delay is the additional travel time experienced by a driver, passenger or pedestrian. Intersection delay is the additional travel time experienced by road users due to control measures and interaction with other users. Delay is considered an effective way to measure intersection performance because it can be felt /quantified by drivers. Delay is, however, difficult to measure in the field (Dion et al., 2004) because of the random arrival pattern of vehicles at the intersection, lost time, and oversaturated traffic flow conditions. Manually collecting delay data from intersections through observation is difficult because vehicle delay includes many parameters such as traffic volumes, car following, queued vehicles, bus stops near intersections, roadside parking, and pedestrians (Murati). Delay can be quantified in different ways; stopped time delay, approach delay, travel time delay, time-in-queue delay, and control delay. The stopped time delay is when a vehicle is stopped in a queue while waiting to pass through the intersection. Approach delay includes stopped delay but adds the time loss due to deceleration from the approach speed to a stop and the time loss due to acceleration back to the desired speed. Travel time delay is the difference between the driver's expected travel time through the intersection and the actual time taken. Time-in-queue delay is the total time a vehicle joins an intersection queue to its discharge across the stop line on departure.

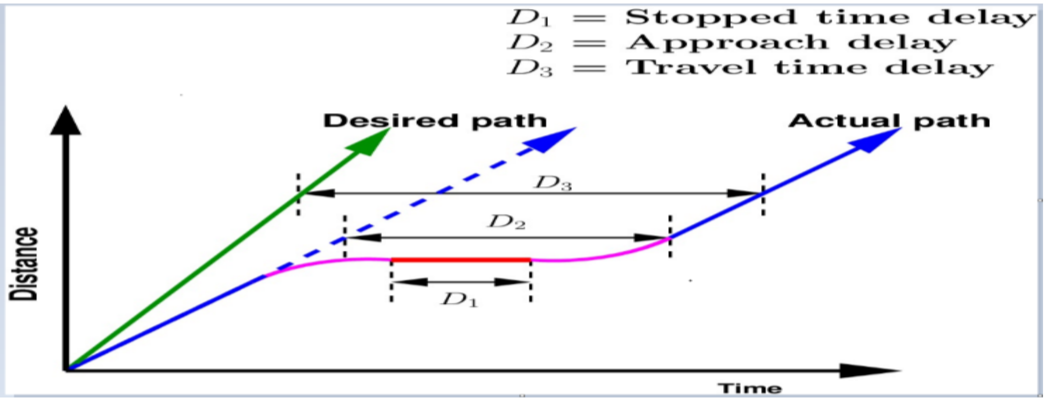

Figure 2. 20: Components of delay at signalised intersections(Kyte and Tribelhorn, 2014)

Control delay is caused by a control device and is approximately equal to the time-in-queue delay plus the acceleration-deceleration delay components. In analytical models used to estimate delay, there are three components of the delay; uniform delay, random delay, and overflow delay. Uniform delay is based on the assumption of uniform vehicle arrivals and stable flow with no individual cycle failures (cycle failure is when a vehicle waits for more than one green phase to be discharged). The random delay is above and beyond the uniform component that comes about because the flow is randomly distributed. Overflow delay is the additional delay that occurs when the capacity of an individual phase or series of phases is less than the demand or arrival rate. Several models are used to estimate signalised intersection delay; Webster, Ackelik and HCM are described below.

2.7.1 Delay Models

Early models focused on the performance of a single intersection experiencing unexpected arrivals and deterministic service times emulating fixed-time control. The thrust of these models has been to produce point estimates that are expectations of delay and queue length used for timing design and quality of service evaluation. The steady-state queue theory approach breaks down at high degrees of saturation. The model form typically includes a deterministic component to account for the red-time delay and a stochastic component for queue delays—the latter term derived from a queue theory approach. Delay models are; deterministic queuing, steady state stochastic and time-dependent stochastic.

2.7.1.1 Deterministic queuing models – They assume that vehicles arrive at a uniform and constant flow rate; that vehicles decelerate and accelerate instantaneously, and that vehicles queue vertically at the intersection stop line. Average delays due to under-saturation (du) and over-saturation (d*o*) flow using deterministic models shown below in equations 2.21 and 2.22

$$d_u = \frac{C(1-\lambda)^2}{2(1-X\lambda)} \qquad 2.21$$

$$d_o = \frac{C-g_e}{2} + \frac{T}{2}(X-1) \qquad 2.22$$

Where λ fraction of effective green to cycle length ($\frac{g_e}{C}$); X denotes the degree of saturation ($\frac{v}{Q}$) and X ≤1, Q denotes capacity, *v* is traffic volume, T denotes the time for which flow is oversaturated.

2.7.1.2 Steady-state stochastic models: It accounts for the randomness of arrivals and make it closer to reality. It characterises traffic delays based on statistical distributions of the arrival and departure processes. First, considering that the number of arrivals in a given time interval follows a known distribution, typically a Poisson distribution, and this distribution does not change over time. Second, they all assume that the headway between departures from the stop line follows a known distribution with a constant mean or are identical. Third, while it recognised that temporary over-saturation might occur due to the randomness of arrivals, it assumed that the system remains unsaturated over the analysis period. Fourth, the system is assumed to be running long enough to allow it to have settled into a steady state. Fifth, all these models still consider that vehicles decelerate and accelerate instantaneously and thus, all drivers behave the same way. Equation 2.23 gives the formula for the delay.

$$d = \frac{C(1-\lambda)^2}{2(1-X\lambda)} + \frac{X^2}{2v(1-X)} - 0.65\left(\frac{c}{v^2}\right)^{1/3} X^{2+\lambda} \qquad 2.23$$

Note that cycle length, $c = \frac{1.5L+5}{1-\sum y_i}$ where L is lost time per cycle and Σy_i sum of volume/saturation ratio of critical movement i. X is the degree of saturation, λ is g/c, g is effective green and v traffic volume

The first term in the steady-state model represents delay when traffic arrives at a uniform rate, while the second term makes some allowance for the random nature of the arrivals. The third term, calibrated based on simulation experiments, is a corrective term to the estimate, typically in the range of 10% of the first two terms. The stochastic equilibrium assumed in steady-state models requires an infinite period of stable traffic conditions. At low flow to capacity ratios, equilibrium is reached reasonably; thus, the equilibrium models are an acceptable approximation of the real-world process. When traffic flow approaches signal capacity, the time to reach statistical equilibrium usually exceeds the period demand sustained. The problem lies in the assumption of sustained arrival flows needed to reach stochastic equilibrium.

2.7.1.3 Time-dependent stochastic models are a compromise approach, using the coordinate transformation methods, which overcomes some steady-state model problems. The time-dependent stochastic model assumes steady-state traffic conditions. It estimates delay under stochastic equilibrium conditions when the arrival and departure flow rates have been stationary for an indefinite period. It also assumes that the number of arrivals in a given interval follows a Poisson distribution. The headway between departures has a known distribution with a constant mean value. Similar to the earlier models, it also assumes instantaneous acceleration and deceleration to simplify data estimation. The three time-dependent stochastic models are; HCM delay model, Australian delay model, and Canadian delay model. The HCM delay model consists of three components; uniform, incremental, and initial queue delay. The uniform delay component gives the delay assuming uniform arrivals, stable flow, and no initial queue and is based on Webster's uniform delay formula. The incremental delay component is used to estimate the delay due to the non-uniform arrival of vehicles and temporary cycle failures. The initial queue delay (similar to overflow delay above) component is the delay that occurs when a residual queue from a previous period causes vehicles arriving in the period under consideration to experience an extra delay as they wait for the initial queue to clear. The HCM gives a procedure for determining the initial queue; the value is zero if there is no initial queue. The average control delay per vehicle is then given as a summation of the two key components. The three time-dependent stochastic model equations given below, starting with the HCM model in equation 2.24

$$d = 0.5C\frac{(1-\lambda)^2}{1-\lambda * x_1} + 900T\left\{(x-1) + \sqrt{(X-1)^2 + \frac{8kX}{cT}}\right\} \qquad 2.24$$

The Australian time-dependent stochastic model,

$$d = \frac{C(1-\lambda)^2}{2(1-X\lambda)} + 900T\left\{(x-1) + \sqrt{(X-1)^2 + 12\left(\frac{X-X_o}{cT}\right)}\right\} \text{ for } X_o = 0.67 + \frac{sg}{600} \text{ Eqn 5} \qquad 2.25$$

Where *sg* denotes capacity per cycle time

The Canadian time-dependent stochastic model,

$$d = \frac{C(1-\lambda)^2}{2(1-X\lambda)} + 900T\left\{(x-1) + \sqrt{(X-1)^2 + \frac{4X}{cT}}\right\} \qquad 2.26$$

Where: d_2 = incremental delay, k = incremental delay factor that is dependent on controller settigs

Average number of vehicles in queue, $Q = d_x * v_x$ 2.27

Where; Q = average number of vehicles in the queue for movement x (veh)

d_x = average control delay for movement x (hrs.); v_x = volume attempting to complete movement x (vph)

Under a steady-state stochastic delay model, the average approach delay becomes discontinuous when capacity is reached. Approach delay is continuous under fitted time-dependent delay and the deterministic oversaturation delay models. It is clear that irrespective of the model employed, curve upturn at capacity is imperative irrespective of the degree of bendiness. Since delay is a function of flow and rainfall affects flow adversely, a curve shift upward expected because of rainfall irrespective of the model used to derive the delay. In a study conducted in Noblesville, Indiana (Bullock, 2008) that compared different methods used to estimate intersection delay, the study concluded that the HCM method performed satisfactorily in estimation accuracy. (Dion et al., 2004) did a comparison study for some selected delay estimation models and compared them with simulation results. They concluded that in the v/c ratio range of 0.1 to 1.4, the models used in the Australian Capacity guide, Canadian Capacity guide, and HCM and the simulation were generally in agreement. It can be concluded thus that if the intersection operation is not oversaturated, the use of HCM delay models could yield satisfactory results.

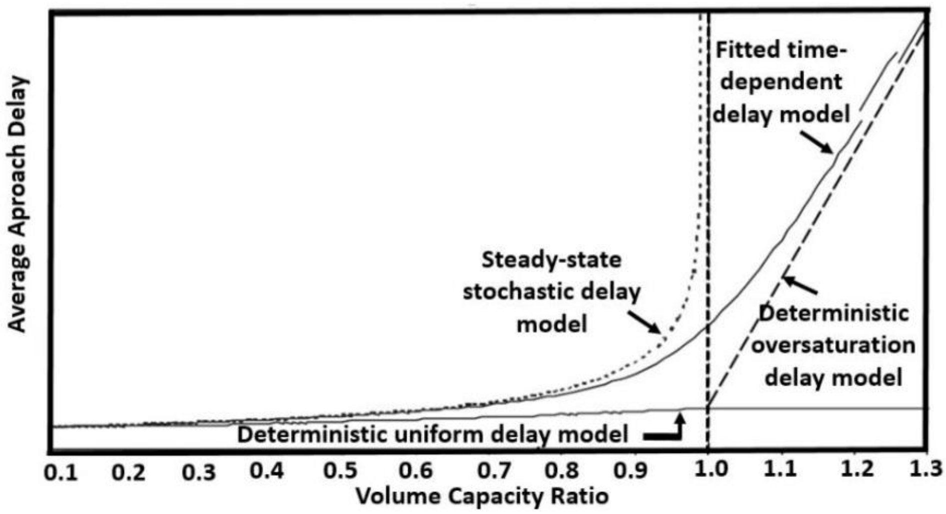

Figure 2. 21: Stochastic Time-Dependent Delay Model Concept (Rodriguez, 2007)

2.7.2 Modification of cycle length

Cycle length comprises the total signal time to serve all the signal phases, including the green time plus any change interval. Longer cycles will accommodate more vehicles per hour, but that will also produce higher average delays. The best way is to use the shortest practical cycle length that will serve traffic demand. The cycle length includes the green and lost time for each phase totalled to include all signal phases. The Webster method, the most common method for cycle time determination, overestimates cycle length when the degree of saturation is closer to zero. In a study by (Zakariya and Rabia, 2016), the researchers used the regression method to estimate optimum cycle length. The results indicated that Webster and their method produced similar results for the lower values of saturation flow, and at higher levels, the regression method gave better results. They generated charts based on flow ratio and the lost time used to estimate optimum cycle length that minimises delay. Figure 2.22 shows some of the charts developed by the study.

Past studies have investigated both start-ups lost time and effective green. The values obtained for start-up lost time vary from one to two seconds (Shawky and Ghafli, 2016). Factors that affect the value of start-up lost time have also been investigated, and among others, the following have been determined; turning movements, queue length, geometry and location, time of day, weather condition, traffic light visibility, and phase timing and sequencing (Shawky and Ghafli, 2016). One of the factors noted that could affect start-up lost time is weather conditions. A study in Beijing, China (Weng, Wang, Liu, & Qiao, 2014) considered rainfall and snow effects on saturation flow and start-up lost time. The results indicated that start-up lost time increased by 51.35% for a rainfall intensity of 5mm/hr. While for the intensity of about 32mm/hr, the increase was 75%. In another study carried out by (Maki, 1999) that looked at adverse weather impact on traffic volume, speed, saturation flow, and start-up lost time, it was found that the start-up lost time increased from 2 to 3 seconds.

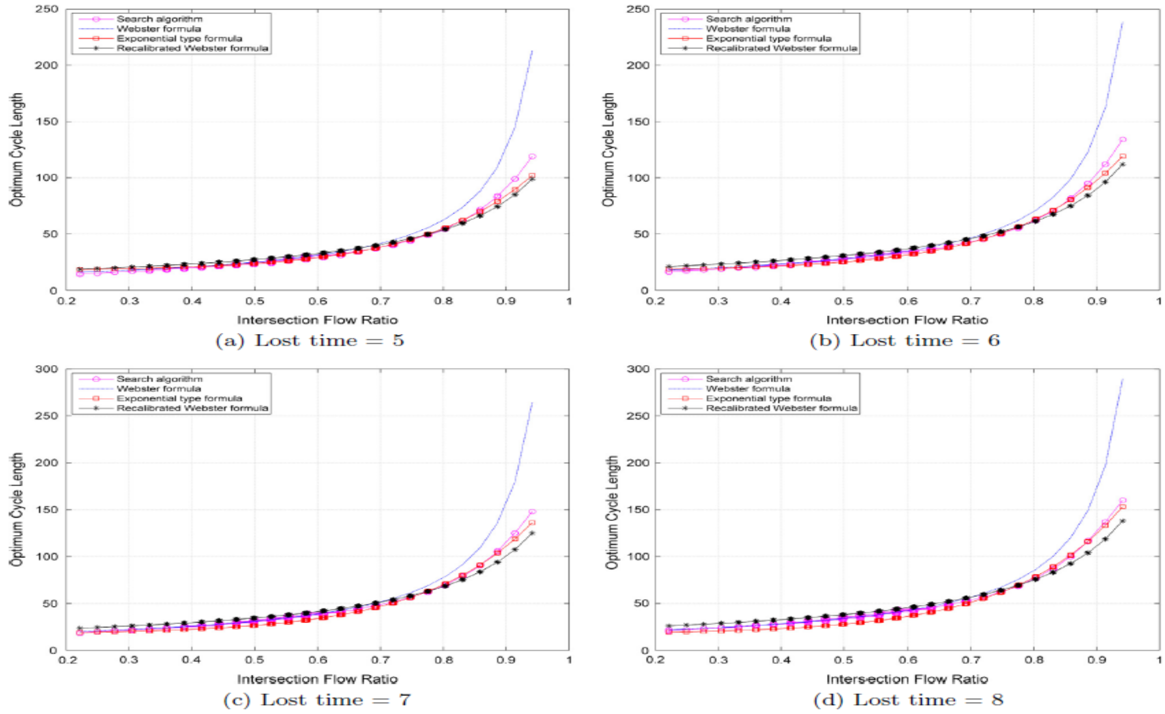

Figure 2. 22 : Determination of optimum cycle length using regression method (Zakariya and Rabia, 2016)

2.7.3 Lost time relative to light/ moderate/heavy rainfall

In a study in Salt Lake City, Utah (Perrin et al., 2001), it was recorded that the start-up lost time increased by 23% from 2 seconds to 2.5 seconds and the speed reduced by about 30%. (Sun et al., 2013a) carried out an investigation in China which found that the start-up lost time increased by between 21% and 33%, differences being whether it was an outer or inner lane. The conclusion of the above discussion is that rainfall has an impact on start-up lost time. That impact varies from one region to another because it is based on, among other factors, driver reaction, location, and geometry. The SARTSM mentions two seconds as the value to be used for SULT, and there is no mention of weather conditions.

2.7.4 Saturation flow

The Signal timing manual (Koonce, 2008) defines saturation flow as the equivalent hourly rate at which vehicles can traverse an intersection approach under prevailing conditions, assuming constant green indication and no loss time in vehicles per hour or vehicles per hour per lane. Saturation headway is the average headway between vehicles occurring after the fourth vehicle in the queue and continuing until the last vehicle in the initial queue clears the intersection. Saturation flow rate is a direct function of vehicle speed and separation distance and varies by traffic movement, time of day, and intersection location. Saturation flow is the cornerstone of the analysis and design process for signalised intersections; it is therefore recommended that the values used for it be accurate and specific. The most commonly used method for saturation flow measurement is the one (HCM, 2016) Over time, researchers have been modified to suit prevailing conditions, different regions, and locations. The HCM 2016 provides two methods, an estimation method based on an assumed base saturation flow rate and a field measurement procedure. The first method uses the equation 2.28 below;

$$S = S_0 N f_w f_{Hv} f_g f_p f_a f_{Lu} f_{LT} f_{RT} f_{Lpb} f_{Rpb} f_{bb} \qquad 2.28$$

Where: S = saturation flow rate for subject lane group in hev/h

S_o = Base saturation flow rate in veh/h/ln, N = number of lanes in lane group

f = Adjustment factors for various parameters where the subscripts denote the particular parameter (w- lane width, Hv- heavy vehicles, g- grade of approach, plane parking, a- area type, Lu- lane utilisation, LT- left turns in lane group, RT- right turns in lane group, Lpb- pedestrian adjustment factor for a left turn, Rob- pedestrian- bicycle adjustment factor for right turns, bb- blocking effect of local buses that stop within intersection area)

This estimation's base saturation flow rate is taken as 1900 vehicles per hour per lane (veh/h/ln). The adjustment for lane width accounts for the negative impact of narrow lanes and allows for an increase in wider lanes, where the standard lane width is taken as 3.6m. The heavy vehicle adjustment factor accounts for these vehicles' additional space and their different operating capabilities. The adjustment for approach grade recognises the fact that heavy vehicles are affected by the gradient of approach. The adjustment for parking accounts for the frictional effect of a parking lane on flow in the adjacent lane group and lane blocking by vehicles moving in and out of the parking lane. The adjustment for bus blockage accounts for local transit buses that discharge or pick up passengers at stops within 75m of the stop-line. The adjustment for area type accounts for the location of the intersection with Central Busines District (CBD) areas where CBDs show inefficiencies that may not be in other non-CBD areas. Adjustment for lane utilisation accounts for the unequal distribution of traffic among the lanes in the lane group in case of more than one lane. Adjustment for right turns accounts for geometry with different factors for pedestrians and bicycles, and the same applies to left turn adjustments. Considering the variations in driver behaviour per region and the assumption of a base saturation flow rate, the HCM estimation method is not regarded as accurate and, as such, not recommended unless as a preliminary measure.

The HCM field measurement method involves estimating saturation headway; the saturation headway is usually achieved after about 10s to 14s of green time, which corresponds to the front axle of the fourth passenger car passing the stop line. Vehicles are recorded when the front axle crosses the stop line; recording starts when the first vehicle in the queue crosses the stop line, the fourth and the 14th or last vehicle are also recorded. The time difference between the 14th(or last) vehicle and the fourth divided by vehicles yields the saturation headway (h_s). The saturation flow rate in vehicles per hour (veh/h) or passenger car units per hour per lane (pcu/h/ln) is then obtained by diving 3600 by the average saturation headway. Equations 2.29 and 2.30 below give the expressions for saturation headway and saturation flow, respectively.

$$h_s = \frac{T_n - T_4}{n} \tag{2.29}$$

$$S = \frac{3600}{h_s} \tag{2.30}$$

Where: $T_{n,4}$= time nth and 4th vehicle clears the stop line, *n* = number of vehicles

S = saturation flow in veh/h/ln or pcu/h/ln, h_s= Saturation headway (s)

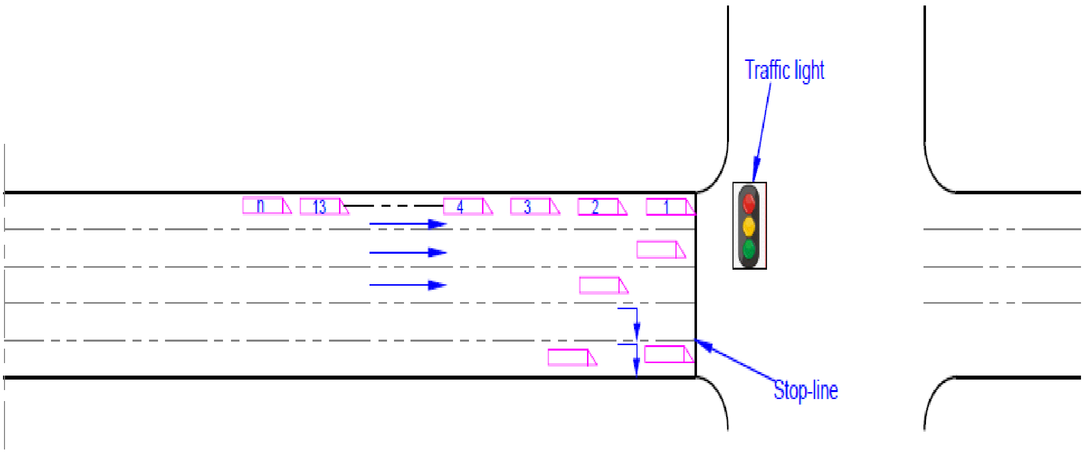

Figure 2. 23 : Set up for field measurement of saturation headway

Figure 2.23 diagrammatically shows how to measure saturation headway in the field; this is then used to calculate the saturation flow rate. According to the HCM, a minimum of 15 cycles with more than 8 vehicles in the initial queue is typically required to obtain statistically significant results. The percentage of heavy vehicles and turning vehicles should also be noted for reference. Variations of the HCM method were noted from past researches in cases where the prevailing conditions were different. For example (Saha et al., 2017), in their study based in India, noted that traffic flow was heterogeneous and that lane discipline was not observed. They proposed a method that used volume and effective green time to estimate the saturation flow. A correction was done for lane width and traffic composition by coming up with regression equations. Another study conducted in China (Shao and Liu, 2012) investigated the accuracy of using average discharge headway to estimate the saturation flow rate. The study found that the average discharge headway was greater than the median value. More than 50% of the drivers kept headways smaller than the average headway, and as such use of average values underestimated the saturation flow rate. The study recommended using median values (in the case of normal distribution) or lognormal transformed values for the saturation flow rate estimation. Therefore, the choice of which method to use to estimate saturation headway/saturation flow rate depends on the prevailing conditions. In case lane discipline is not an issue, the HCM method, where the vehicle position to determine saturation headway is empirically obtained, is considered sufficient to give accurate results.

In the United Kingdom approach, the factors that affect the maximum discharge rate of vehicles across the stop line are considered. The factors approach width, traffic flow composition, length of green period per cycle length and lost time during a green period are included in the saturation flow equations 2.31 to 2.33.

$$S_1 = \frac{S_o - 140d_n}{1 + \frac{1.5f}{r}} pcu/hr. \tag{2.31}$$

$$S_o = 2080 - 42d_G * G + 100(W - 3.25) pcu/hr \tag{2.32}$$

Where: G – Gradient; f – the proportion of turning vehicles, r – radius of curvature of vehicle path dn - nearside lane and gradient dummy variables, o for nearside lane, 1 for non-nearside lane d_G - nearside lane and gradient dummy variables, o for downhill, 1 for uphill

For streams, containing opposed right turning vehicles in individual lanes saturation flow;

$$S_2 = S_g + S_c \ pcu/hr \tag{2.33}$$

Where,

$$S_g = \frac{S_o - 230}{\{1 + (T-1)f\}}$$

$$T = 1 + \frac{1.5}{r} + \frac{t_1}{t_2}$$

$$t_1 = \frac{12{x_o}^2}{\{1 + 0.6(1-f)N_s\}} \text{ and } t_2 = 1 - (fx_o)^2$$

$$S_c = p(1 + N_s)(fx_o)^{0.2} \frac{3600}{\lambda c}$$

$$x_o = \frac{q_o}{\lambda n_l s_o}$$

Where: Sg and Sc are the saturation flows in lanes of opposed mixed turning traffic during and after the effective green period. Xo - the degree of saturation on the opposite arm, Ns - storage spaces available inside the intersection which right turners can use without blocking following straight-ahead vehicles, So and Sn are the saturation flow and the number of lanes of the opposing entry respectively, p - a ratio of passenger car units to vehicles for the stream being considered.

2.7.4.1 Capacity at Signalised Intersections

The HCM, 2016 defines capacity as the maximum hourly rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a roadway during a given period under prevailing roadway, environmental, traffic and control conditions. In other words, the capacity is the maximum flow rate that would be observed based on the amount of green time available. Given the saturation flow rate, effective green, and the cycle length, the capacity at a signalised intersection is then determined using equation 2.34 below:

$$c = s * \left({}^{g}/_{c_o}\right) \qquad 2.34$$

Where: c = Capacity in veh/h, g = effective green time in seconds, C_o = cycle time (s)

In designing a new intersection or evaluating the operation of an existing one, capacity is a key consideration. Determination of the adequacy of the capacity viz a vis the expected traffic volume is usually done; the measures used in this determination include; volume to saturation flow (flow ratio), volume to capacity ratio, also called the degree of saturation, and the reserve capacity. The volume to capacity ratio (X) or degree of saturation is the volume to the capacity per given lane of approach. If the degree of saturation is less than 1, there is sufficient capacity to serve the traffic demand. If it is greater than 1, then it means the traffic demand exceeds capacity. The degree of saturation X is given by equations 2.35 and 2.36 below:

$$X = \frac{v}{c} \qquad 2.35$$

Since $c = s * \left({}^{v}/_{GC}\right)$ and v, which is the traffic demand volume, can also be expressed as; $v = \sum_{j=1}^{n} v$ X can also be expressed as $X = \left(\frac{C}{C-L}\right) \sum_{i=2}^{n} \left({}^{v}/_{S}\right)_j$ 2.36

Where; X= degree of saturation, C = cycle length, L= lost time per cycle

The degree of saturation represents the sufficiency of an intersection to accommodate traffic demand. Generally, when the v/c ratio is less than 0.85, it indicates adequate capacity is available, and vehicles are not expected to experience significant delays and queues. At the v/c ration approaches, 1.0 traffic flow becomes less stable and queueing, and delay may be experienced by vehicular traffic. Under such conditions, vehicles may require more than one signal cycle to clear the intersection (cycle failure).
The concept of reserve capacity has long been used as a useful measure of the operational performance of individual signal-controlled junctions (Bell, 2004). The greatest common multiplier of the existing Origin-Destination (OD) flows accommodated by a given cycle time, green time, saturation flows, and other constraints commonly measures reserve capacity.

The concept of reserve capacity has two interpretations: the spare capacity available to accommodate the inevitable day-to-day fluctuations in demand and, secondly, the spare capacity available to accommodate the steady growth of demand in the future. In the past, methods that have been used to calculate the reserve capacity include linear programming and optimisation algorithms like TRANSYT (Yang, 1996) (Bell, 2004). Above a certain saturation level, signalised intersection ceases to work efficiently; that threshold for cessation of effective operation is at 90% of saturation.

2.7.5 Effect of rainfall on service quality

According to (Prevedouros and Chang, 2004), three factors may be affected: wet weather, saturation flow, effective green time, and progression. Saturation headway may lengthen, effective green time may shrink, and progression may worsen in wet conditions, and as a result, signalised intersection operations become less efficient. Based on an internet survey (Zhang and Prevedouros, 2005), drivers drove 2.7mph (4.9%) slower on wet roads and 6.1 mph (11.1%) slower during rainfall. It reduced travel by about 3%. (Lu et al., 2016) found that optimal traffic signal timing settings are affected by prevailing traffic flow patterns and driving behaviour. For example, longer cycle lengths are generally required to increase road capacity when saturation flow rates are lower, and the red clearance intervals should be increased when drivers drive slower.

A study conducted in Waterloo, Canada, considered road surface conditions and not rainfall intensity. The road surface was categorised as either dry, wet, wet, and slushy, slushy, and snowy and sticky. The results showed a decrease of 3% for wet conditions, 19% for wet and slushy, and 27% for snowy and sticky. (Perrin et al., 2001) noted that drivers are cautious during adverse weather and accelerate more slowly. In a study conducted in Salt Lake City, Utah, the categorisation is based on road surface conditions; results showed a 6% reduction in saturation flow for rainy conditions and up to 20% for snowy and humid conditions. (S.J. and A.W, 2004) in a study conducted in Vermont and considering both rainfall and road surface conditions. The study recorded a 3% reduction in saturation flow for wet conditions and a 16% reduction for snowy and humid conditions. (Shin and Choi, 1998) conducted a study in South Korea and looked at the influence of rainfall found that with rainfall intensity of between 17-28mm/h, saturation flow was between 87% and 96% of the dry weather condition one.

With rainfall affecting saturation flow and SULT, there is an impact on the capacity of signalised intersections. (Chunxiao et al., 2012) a study conducted in China using video data to investigate the effect of slight snow (less than 3cm in 24 hours) on signalised intersection capacity found that the capacity reduced by 7% and the level of service reduced by one level. (Edward et al., 2014) in a study carried out in Tokyo, Japan, the intersection capacity decreased by 4-7% under light rainfall and by a maximum of 14% under heavy rainfall. Concerning the impact that rainy conditions have on driver behaviour during the yellow light, and all-red clearance interval (LI et al., 2012) carried out a study on the impact rainfall has on signalised intersection performance and whether the yellow times have to be redesigned for the wet weather conditions. The results conclude that during rainy weather conditions, yellow intervals should be longer than during day conditions.

As shown below in figure 2.24, service quality deterioration would result from rainy conditions. Service quality is measured on 0 to 1, where 0 denotes level F, and 1 denotes level A. Delay measured on scale 0 to 1 where 0 denotes no delay, and 1 denotes maximum delay; queue also measured on scale 0 to 1 where 0 denotes no queue, and 1 denotes maximum queue. Degree of saturation measured on a scale 0 to 1 where 0 denotes free flow, and 1 denotes jam.

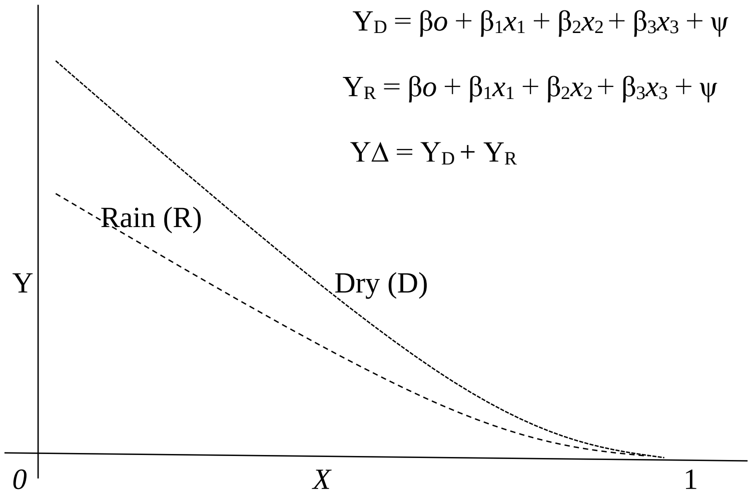

Figure 2. 24 : Service quality v delay/degree of saturation/queue

Based on the hypothesis that service quality deterioration would result from rainy conditions, relationships between service quality as independent variable and delay, queue and degree of saturation as independent variables developed as:

Service quality under dry weather, $Y_D = \beta o + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \psi$ 2.37

Service quality under rainfall, $Y_R = \beta o + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \psi$ 2.38

Differential service quality, $Y\Delta = Y_D + Y_R$

Where, Y is service quality, βo is the intercept and $\beta_1, \beta_2, \beta_3$ Are model coefficients, ψ is dummy variable (D=1 and R = 0), x_1 is delay (s); x_2 is degree of saturation and x_3 is queue length

Note that x_3 computed from x_1, in order to prevent multicolinearity x_3 removed from the equations so that the model equations becomes:

Service quality under dry weather, $Y_D = \beta o + \beta_1 x_1 + \beta_2 x_2 + \psi$ 2.39

Service quality under rainfall, $Y_R = \beta o + \beta_1 x_1 + \beta_2 x_2 + \psi$ 2.40

Note that delay, $x_1 = d_1 + d_2 - d_3$ for steady state and $x_1 = d_1 + d_2 + d_3$ for time dependent method

For steady-state, $y_s = \beta_o + \beta_1[d_1 + d_2 - d_3] + \beta_2\left[q\left(\frac{1}{s\langle g/c\rangle}\right)\right] + \psi$ 2.41

Regular arrival, $d_1 = \frac{c\left(1-\langle g/c\rangle\right)^2}{2\left(1-(g/c)*q\left(\frac{1}{s\langle g/c\rangle}\right)\right)}$ 2.42

Random arrival, $d_2 = \frac{q\left(\frac{1}{s\langle g/c\rangle}\right)^2}{2q\left(1-q\left(\frac{1}{s\langle g/c\rangle}\right)\right)}$ 2.43

Correction factor, $d_3 = 0.65\left(\frac{c}{q^2}\right)^{1/3} q\left(\frac{1}{s\langle g/c\rangle}\right)^{(2+5\langle g/c\rangle)}$ 2.44

$C = \frac{1.5L+5}{1-\sum y_m}$

Where: q - Demand flow, s – Saturation flow, g - Effective green, ψ - Dummy variable (dry = 1 and rainfall = 0)

For time-dependent, $y_t = \beta_o + \beta_1[d_1 + d_2 + d_3] + \beta_2\left[q\left(\frac{1}{s\langle g/c\rangle}\right)\right] + \psi$ 2.45

Regular arrival, $d_1 = \frac{c\left(1-\langle g/c\rangle\right)^2}{2\left(1-(g/c)*q\left(\frac{1}{s\langle g/c\rangle}\right)\right)}$ 2.46

Random arrival, $d_2 = 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{4x}{(s\langle g/c\rangle T)}}\right]\frac{x^2}{2q(1-x)}$ 2.47

Correction factor, $d_3 = 0$

$C = \frac{1.5L+5}{1-\sum y_m}$

Where: q - Demand flow, s – Saturation flow, g - Effective green, ψ - Dummy variable (dry = 1 and rainfall = 0)

2.8 Summary

In this chapter, literature on the evaluation of driver behaviour at a signalised intersection under dry and wet weather conditions was done. Driver behaviour during yellow and all red (wintergreen) intervals was discussed as this study's major focus. Factors that contribute to red-light running were discussed and how they are affected by rainfall. The evaluation of driver behaviour during the wintergreen period was done to measure intersection safety. In this regard, both yellow and all-red intervals are designed to minimise any safety issues brought about by the interval being either too short or too long. When drivers behave in a way that disregards the yellow and all red lights, that amounts to aberrant behaviour, which is unsafe.

Evaluation of driver behaviour during the green time was also discussed to evaluate intersection performance under different weather conditions and determine what implication rainfall may have on service quality. In this regard, the level of service(LOS) classification is a qualitative measure used to gauge intersection performance. LOS is based on a quantitative measure of control at the intersection under consideration. The use of only delay to do LOS evaluation of intersections has been questioned because the delay is rather subjective and depends on prevailing circumstances. Secondly, in the determination of service quality, it is important to include Traffic Managers who, in addition to delay, may be more interested in capacity utilisation. It led to the proposal to replace LOS with Service quality, to include expanded parameters like degree of saturation. With the new definition of service quality, quantifying driver behaviour change during wet weather conditions was deemed important as a guide to determine the adequacy of key intersection design parameters, including; cycle time, green time, yellow and all-red time. It is based on the fact that normal intersection parameters are designed based on dry weather conditions.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Overview

This chapter describes the method used to achieve the study's aim and objectives set out in chapter 1, whilst chapter 2 focussed on the theoretical background and literature review to support arguments arising later in the study. In the literature review, delay estimation method with saturation flow and discharge headway considered suitable for signalised intersection during rainfall. It has been hypothesised that, 'rainfall produces service delivery reduction and increases discharge headway at signalised intersections. To determine the influence of rainfall on service delivery data is needed to: i) to establish the extent of delay per light, moderate and heavy rainfall per surveyed site, ii) for determination of vehicle types, speeds, and traffic flows on signalised intersections with and without the influence of rainfall, and iii) estimate the rainfall implications for red-light running.

To that effect, the survey data and information collected were both qualitative and quantitative. Direct measurements obtained at four selected signalised intersections in Durban, South Africa, and supplemented with records from the government offices, eThekwini Municipal Authority, and the South African Meteorological Society. The qualitative data covered details about the physical features of the roads, details about design traffic, road maintenance activities, and road management, and road traffic trends. The traffic data refer to multi-lane signalised intersections with functional side drains. In the analysis, only data per carriageway lane used, therefore the estimated delays are per carriageway lane. The aggregated database contained the following variables used for signalised intersection delay analysis: (i) flows per vehicle type per turning movement, (ii) average approach speed, (iii) discharge headway, (iv) cycle time, and (v), the degree of saturation (derived from volume and capacity). Approach speeds, acceleration, cycle time data used to determine rainfall impact on red light running. Note that traffic volume data, vehicle speeds, cycle time and rainfall are central to the study. The remainder of this chapter has of four sections: Section 3.2 discusses the research methodology giving the adopted framework. Section 3.3 study site selection criteria is discussed, followed by coding of the selected signalised intersection sites, section 3.4 details pilot study test for equipment, and the analytical method including some of the challenges encountered. The chapter concluded in section 3.5

3.2 Research methodology framework

A systematic approach employed to carry out this study and a systematic process developed and followed to achieve the study objectives. The process started with identifying sites for the study, collecting data about the sites, and then combining that with traffic and rainfall data to evaluate the sites during dry and rainy weather conditions. The evaluation was done for both safety (red-light running) and operational efficiency (quality of service) of the intersections and a new approach for determining service quality was developed and used to evaluate the intersections under rainy weather conditions. Figure 3.1 below gives a schematic that shows the process used in executing the methodology. The research methodology summarised as:

Step 1. Collect geometric design data, rainfall intensity data, and traffic signal data at survey sites

Step 2. Collect traffic volume and turning movement, discharge headway, and speeds at survey sites

Step 3. Collate volume, speed, turning movement, and discharge headway data into dry and rainfall intensity

Step 4. Determine the extent of RLR violations during dry and rainy weather conditions

Step 5. Determine the regression models for RLR under dry and rainy weather conditions

Step 6. Then determine the probability for variable rainfall on RLR

Step 7. Determine traffic composition and modify passenger car equivalent to account for rainfall

Step 8. Compute traffic flow and turning movements for dry and rainfall conditions

Step 9. Compute discharge headways for dry and rainfall conditions

Step 10. Compute cycle time and effective green time for dry and rainfall conditions

Step 11. Compute saturation flow and volume/capacity ratio for dry and rainfall conditions

Step 12. Compute service quality criteria table based on dry weather data

Step 13. Compute delay and functional service quality for dry and rainfall conditions

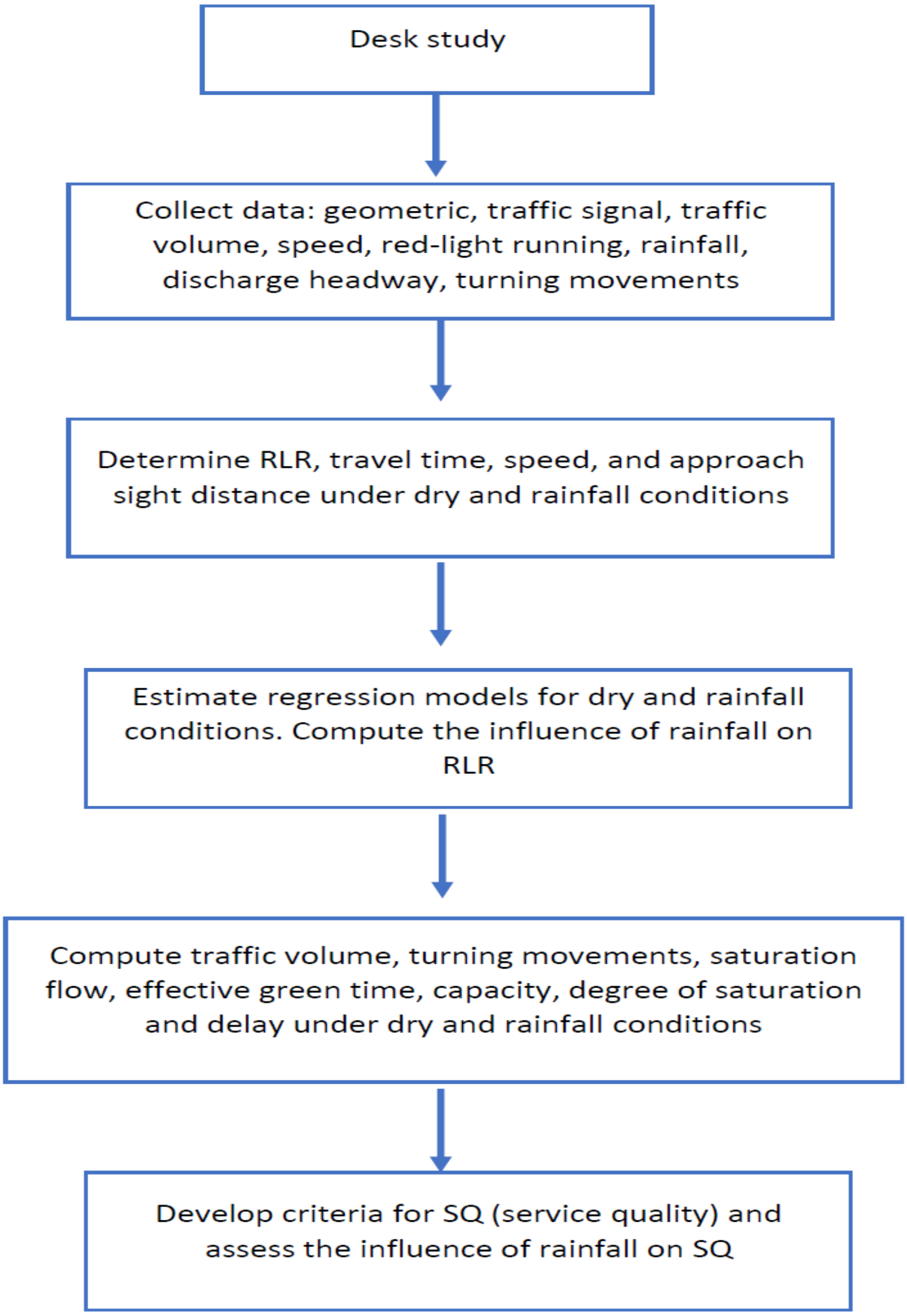

Figure 3. 1 : Framework for research methodology

3.3 Study Site

In this section information about study site selection, coding of selected sites, determination of appropriate sample size, survey equipment, and the type of data to be collected are discussed.

3.3.1 Location and selection criteria

In selecting an appropriate site for this study, certain factors were considered important. The study was conducted in the city of Durban using video data, therefore identification of sites under the city authority and served with appropriate video coverage was important. There are different types of signalised intersections depending on the number of arms, four-armed intersections were selected. This is because they were found to be robust with different turning movements; through, exclusive turning, leading phases and having both minor and major roads. Traffic volume was also considered important, with care being taken to ensure that the selected study sites had adequate volumes, because some of the field data measurements like saturation headway are dependent on the number of vehicles in a queue.

The study concentration was on evaluating the influence of rainfall on red-light running and the implication for performance of signalised intersections and to this end, other variables were to be kept to a minimum, and among them was traffic control. Study sites were therefore selected based on the availability of fixed time traffic control and those on actuated control throughout the day were not considered. The main mode of data collection was video and therefore it was key that the study site selected be served by a good resolution Closed-Circuit Television (CCTV) camera and preferably a static camera, to ensure that the footage obtained was focused on the same spot for the required duration. Rainfall was another a key input, and therefore finding a rain gauge station near the selected study site was considered important. To ensure that the rainfall data recorded and used in data analysis was representative of the study location, the rain gauge stations were selected to be within a maximum of 2km radius of the selected site. Listed below is a summary of the criteria.

[i]. Survey intersection be within eThekwini municipality and covered by CCTV camera
[ii]. Location be on a flat and geometrically straight road section.
[iii]. Section before and after the survey site should be straight
[iv]. Selected intersection should not be within CBD to avoid interference from parked cars and public transport drop and pick-up points.
[v]. Section should be well drained to avoid water pooling and the consequent effects.
[vi]. The road surface at the selected site be in good condition free from any surface defects.
[vii]. The distance between survey site and the rain-gauge station should be less than 2 km: and
[viii]. Survey site should have daily traffic volume more than 10,000 for both direction

To ensure rainfall data collected was useful to the study, the selected rain gauge stations were to meet the following:

[i]. Be located within a 2km radius of the selected survey site.
[ii]. Be active
[iii]. One that uses data logger to record rainfall data with an accuracy of at most 1 minute

Information regarding potential survey site and rain-gauge stations were recorded on survey sheet summary as shown in Table 3.1.

Table 3. 1: Example of Survey Summary Sheets

SURVEY SUMMARY SHEET	
GENERAL INFORMATION	
Road allocated number	
Location	
SURVEY SITE	
Type of carriageway	
Length of road segment	
Distance from ends of road segment to selected road section	
Road section geometry	
Carriageway width	
Road surface condition	
Type of pavement	
Approximate Annual daily traffic volume	
Design speed	
Posted Speed limit	
Stopping sight distance	
Distance to nearest rain gauge station	
RAINGAUGE STATION	
Owner	
Station number	
Station name	
Status	
Type of data collection	
SKETCH OF LOCATION	
SUMMARY OF SURVEY SITE	

3.3.2 Site coding and assessment

For ease of data presentation and analysis, the selected study sites were coded. For signalised intersections, traffic and signal conditions change per the times of the day. As such the morning peak and evening peak could be quite different. All selected sites were given a code which was then used for the rest of the study, the summary of all sites with their respective codes is given in table 3.2.

Table 3. 2: Coding for study sites

S/No	Site Name	Highway	Flow direction	No of arms	Site code
1	Umgeni Road / Alpine road	M19	South	4	001
2	Umgeni Road / Alpine road	M10	North	4	002
3	Sandile Thusi Road / Stalwart Simelane Street	M17	South	4	003
4	Sandile Thusi Road / Stalwart Simelane Street	M4	North	4	004

3.3.2.1 Site 001: Umgeni Road / Alpine Road intersection (South)

The intersection is located within the eThekwini Central Business District (CBD). It is a four-arm multilane signalised intersection, with slip roads for left-turning traffic on all approaches. There are three through traffic lanes and two right turning traffic lanes. On the major road approaches, through and right-turning traffic have separate lanes. The minor roads have all mixed lanes, through and right-turning traffic. The intersection is 34m wide (for major roads) and 40 wide for minor roads. The approach to the intersection is clear of any obstructions giving adequate sight distance on all approaches. The road geometry meets the requirements in terms of lane width. As at the time of the study, the pavement covering the intersection area was devoid of any defects or failures and proper lane marking was provided. Figure 3.2 below gives an overview of the intersection, showing all the approaches.

Figure 3. 2: View of study sites 001 and 002

The approaching roads from all sides are separated by medians, the ones of the major roads being bigger than the minor ones. The separation between the through and right turning lanes is achieved through use of proper lane marking. All the right turning lanes are flare lanes that are provided at the approach to the intersections. Rainfall was also a key input parameter and rain gauges used were within a 1.5 km radius of the intersection. Figure 3.3 below shows the location of the rain gauge used for this site relative to the location of the intersection.

Figure 3. 3: Rain gauge station location for sites 001 and 002

The video footage used, covered the whole intersection area, showing all the approaches and all vehicle movements including the turning movements. Data was extracted from these videos using both manual observations and software where applicable as will be described later in the section. The signal information for the site was such that the morning and evening peaks were on fixed time and the rest of the day on semi-actuated control. For the AM and PM peaks, the cycle time was 120 seconds. The difference in the signal information arose out of the way the time was split between the different phases, for example in the AM peak the leading green phase on the major road was 30 seconds while in the PM peak it was 24 seconds. Data collection was therefore done for only the AM peak and PM peak, considering an hour in each case. Corresponding rainfall data was obtained from the rain gauge stations that were used in the rainy weather condition data analysis. The rainfall data collected by the University of KwaZulu Natal Electrical Engineering department was used to supplement and confirm the data obtained from these rain gauge stations that are maintained by eThekwini municipality.

3.3.2.2 Site 002: Umgeni Road / Alpine Road intersection (North)

The intersection is located within the eThekwini Central Business District (CBD). It is a four-arm multilane signalised intersection, with slip roads for left-turning traffic on all approaches. There are three through traffic lanes and one right turning traffic lane. The intersection is 34m wide (for major roads) and 40 wide for minor roads. The approach to the intersection is clear of any obstructions giving adequate sight distance on all approaches. The road geometry meets the requirements in terms of lane width. As at the time of the study, the pavement covering the intersection area was devoid of any defects or failures and proper lane marking was provided. Rainfall was also a key input parameter and rain gauges used were within a 1.5 km radius of the intersection. Figure 3.3 above shows the location of the rain gauge used for this site relative to the location of the intersection.

3.3.2.3 Site 003: Sandile Thusi road / Stalwart Simelane Street (South)

The intersection is located within the eThekwini Central Business District. It is a four-arm multilane signalised intersection, with slip roads for left-turning traffic on all approaches. On the major road approaches, through and right-turning traffic have separate lanes. The minor roads have one through lane and two mixed through and turning lanes. The intersection is 37 m wide (for major roads) and 45m wide for minor roads. The approach to the intersection is clear of any obstructions giving adequate sight distance on all approaches. As at the time of the study, the pavement covering the intersection area was devoid of any defects or failures and proper lane marking was provided. Figure 3.4 below gives an overview of the intersection, showing all the approaches.

Figure 3. 4 : View of Study site 003 and 004

The approaching roads from all sides were separated by medians, the ones of the major roads being bigger than the minor ones. The separation between the through and right turning lanes is achieved using proper lane marking. All the right turning lanes are flare lanes that are provided at the approach to the intersections. Rainfall was also a key input parameter and rain gauges used were within a 1 km radius of the intersection. Figure 3.5 below shows the location of the rain gauge used for this site relative to the location of the intersection.

Figure 3. 5 : Rain gauge station location for study sites 003 and 004

The signal information for the site was such that the morning and evening peaks were on fixed time and the rest of the day on semi-actuated control like the one described above. For the AM and PM peaks, the cycle time was 100 seconds. The difference in the signal information arose out of the way the time was split between the different phases, for example in the AM peak the major rad traffic got 52 seconds out of 100 while PM peak that proportion changed to 39 seconds with 71 seconds going to the minor road traffic. Data collection was done for only the AM peak and PM peak, considering an hour in each case. Corresponding rainfall data was obtained from the rain gauge stations and used in the rainy weather condition data analysis. The rainfall data collected by the University of KwaZulu Natal Electrical Engineering department was used to supplement the data obtained from these rain gauge stations that are maintained by eThekwini municipality.

3.3.2.4 Site 004: Sandile Thusi road / Stalwart Simelane Street (North)

The intersection is located within the eThekwini Central Business District. It is a four-arm multilane signalised intersection, with slip roads for left-turning traffic on all approaches. There are three through traffic lanes and one right turning traffic lane. On the major road approaches, through and right-turning traffic have separate lanes. The intersection is 37 m wide (for major roads) and 45m wide for minor roads. The approach to the intersection is clear of any obstructions giving adequate sight distance on all approaches. As at the time of the study, the pavement covering the intersection area was devoid of any defects or failures and proper lane marking was provided. Figure 3.4 above gives an overview of the intersection, showing all the approaches while figure 3.5 gives the location of the site relative to the rain gauge station.

The road geometry details are as summarized in table 3.3, the summary is for all selected survey sites.

Table 3. 3 : Summary of basic site features

Feature	Intersection Name			
	Umgeni / Alpine- Major	Umgeni / Alpine- minor	Sandile Thusi / Stalwart Simelane- Major	Sandile Thusi / Stalwart Simelane- Minor
Pavement surface	Asphalt	Asphalt	Asphalt	Asphalt
Pavement Surface condition	No defects	No defects	No defects	No defects
Lane marking	Proper	Proper	Proper	Proper
Number of exclusive through lanes	3	0 -1	3	1
Number of exclusive right-turning lanes	1 - 2	0 - 1	1 - 2	0 - 1
Number of mixed lanes	0	2	0	2
Length of flare lane (m)	60 - 130	40 - 60	60-90	40- 60
Lane width (m)	3.3 - 3.8	3.3 - 3.8	3.3 - 3.8	3.3 - 3.8
Intersection width (m)	34	40	37	45

3.3.4 Data collection

Discharge headway data were collected at 4 signalized intersections in Durban. Surveys were conducted at the morning and evening peak periods from 7:00 Am to 8:00 Am and 5:00 Pm to 6:00 Pm by video cameras and the data were manually collected from the videotapes. Peak period dry weather condition data were used to develop service quality (SQ) criteria and the peak data wet weather data were used to determine the impact of rainfall on SQ and Red-Light Running (RLR). The time for each vehicle rear bumper passing the stop line was recorded and the time headway was calculated. Data was collected in conjunction with the eThekwini Transport Authority (ETA) and Durban Metro police especially the Disaster Management section that oversees all Closed-Circuit Television (CCTV) cameras installed around the city. The ETA gave the authority to carry out the study and provided some of the signalization information data that were used for verification purposes. The actual CCTV footage were obtained by permission from the Durban Metro Police who are the custodians of the data on the condition that it would be used for research purposes only. To eliminate the effect of start-up and acceleration on the saturation flow rate, the first 5 headways in each signal cycle were removed from the data. Also, data that were anomalous because the drivers were distracted that caused the headway to exceed the normal ranges were eliminated. A proper signing protocol was followed with each footage that was collected. Rainfall data was also obtained from the eThekwini Municipality website under the department of Engineering Services. Login details were obtained and used each time to log in and check the relevant data.

3.3.4.1 Sampling

When carrying out empirical studies, having enough sample size is imperative because it makes it possible to draw conclusions that can be applied to the general population being studied. For this study, therefore, it was important to calculate the required sample size, and equation 3.1(Krejcie and Morgan, 1970) below was applied.

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

Where:

S = Sample size required

N = Size of sample population

P = Sample proportion,

X = the table value of chi-square for 1 degree of freedom at the desired confidence level(95%)= 3. 8416

d = margin of error (5%).

The formula was applied based on the assumption of the flow rate of 1900/veh/hr/, the sample proportion of 50% would be adequate but for this case 100% was assumed at 95% confidence level. The sample size obtained using the equation and reading off a table provided by Krejcie and Morgan was 320 veh/hr. This was achieved and exceed in all stidy sites.

In terms of the location of the study, from chapter 2, for the nation of South Africa, the province of KwaZulu Natal was shown to have on average higher rainfall amounts than other parts of the country. In KwaZulu Natal, Durban is the biggest city and has the highest traffic volume. Rainfall occurs throughout the year with the highest amounts recorded in the months ranging from September to March, these were the months sampled for this study. The intersections studied were also carefully selected as earlier mentioned and because the study was based on video data, any day that was observed to have any incidents/ accidents or anomalies of any kind were eliminated from the study. This was done to ensure that only normal regular traffic was considered.

3.3.4.2 Survey team and equipment

The study was conducted to evaluate the impact of rainfall and this meant that manual observation data collection would not entirely work but some amount of manual data was collected for purposes of verification. The study was based on video data, where the videos were collected through a third party and any verification that was deemed important was done. To collect the manual data, a survey team was constituted, and equipment was used.

The first part of the survey work involved carrying out speed, headway, and volume surveys to obtain data that would be used to verify the data collected using the video footage. To conduct these surveys a team of two people was used. Traffic volume surveys were simply done by manually counting and tarrying the vehicles passing considering a 15-minute interval in each case. Having two people was for accuracy purposes because the study intersections had quite high traffic volumes during the peak periods. To carry out the saturation headway surveys, a simple stopwatch was used. Vehicles were recorded as they crossed the intersection stop-line (front bumper). These considered queues that had more than 10 vehicles and the headways were then recorded. To further check the data, a simple portable video camera was used to record short video clips on all the study sites. These videos were recorded first-hand and were used to verify and countercheck the information that was obtained from the CCTV footage videos. For the speed surveys, a radar gun was used at different locations on the approach to the stop line. The approach speed was determined at the upstream point, the person recording was stationed at a designated point, the point was beyond the point where the flare lane started. Average speed during the green, yellow and all-red times were recorded by standing near the intersection stop-line. Figure 3.7 shows the different positions where speeds were recorded from.

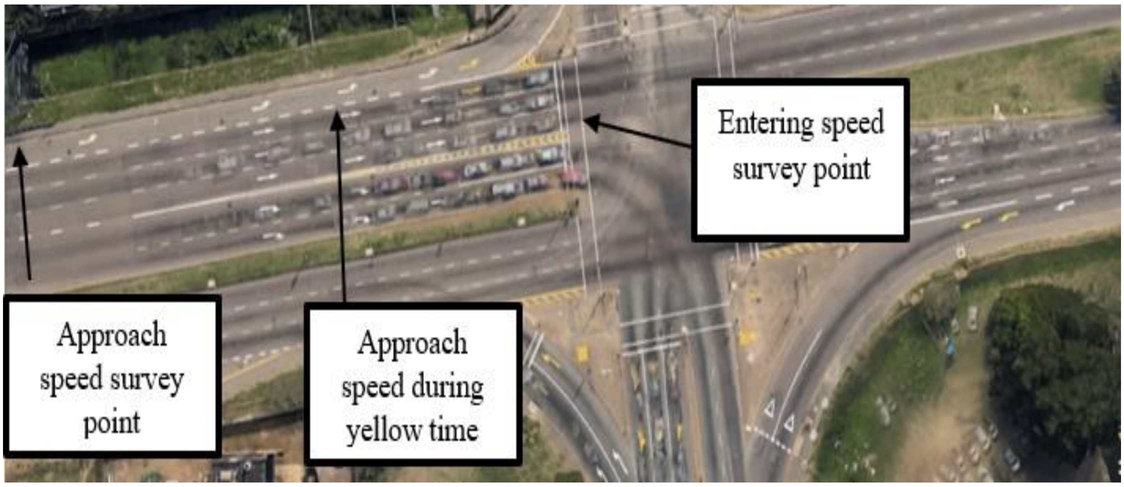

Figure 3. 6: Location points for speed survey

The first survey point was used to determine the approach speed and the operating speed for the intersection. The second and third points were used to determine speed especially during the yellow time for determination of yellow and red-light running cases. The point for the determination of approach speed during yellow was obtained by multiplying the yellow time with the posted speed limit. Entering speed was collected at the intersection stop-line, the difference between the entering speed and the approach speed during the yellow time, coupled with the time difference was then used to determine acceleration and deceleration rates.

3.3.3.2.1 Challenges encountered with survey team and equipment.

In carrying out these survey work and using the equipment as described some challenges were encountered and include:

1. Using a radar gun to collect speed data, in most cases when the motorists noticed the radar gun, they tended to alter their behaviour which gives slightly biased results. To correct for this, the survey team tried as much as possible not to be openly exposed.
2. In using CCTV footage for data collection, as much as care was taken to use static camera locations, there was still camera adjustment and movement (because the cameras are used mainly for safety purposes and in some cases zooming in and out to capture an incident happened) and this lead to loss of information in some cases. This meant that more data days were required to fill in any gap
3. Most of the traffic data were collected using manual means which could introduce individual bias. This was mitigated by maintaining the same person for all data recordings and increasing the volume of data collected so that the calculated average values would have better accuracy.

3.3.3.3 Layout of typical survey site

Except in the case of setting up the pilot study and collection of survey data for verification, all other data for the study were obtained through desk extraction of data from the provided video footage. The desk extraction involved mostly manual observation and recording of the data. Data on red-light running, traffic volume, turning movements, and headways were collected and recorded manually by watching videos and counting the vehicles as they crossed the intersection stop-line. The screenshots in figures 3.7 and 3.8 below show a typical video display showing vehicle movement for the two study sites.

Figure 3. 7: Screenshot of a playing video for sites s001 and 002

Figure 3. 8: Screenshot of a playing video for sites 003 and 004

In the case of collecting data for evaluation of driver behaviour during the yellow and all-red durations, software was used. In this case, it was important to get the site layout marked to show points to collect speed data. The intersection safe sight stopping distance (SSD), as well as distance travelled during the yellow interval (speed multiplied by yellow time) were demarcated and speed data collected from those points as well as at the intersection stop-line. To extract this speed from the obtained videos, software was used. The software used was Traffic Data Extractor (TDE). Figure 3.9 below shows a screenshot from the TDE software showing speed extraction

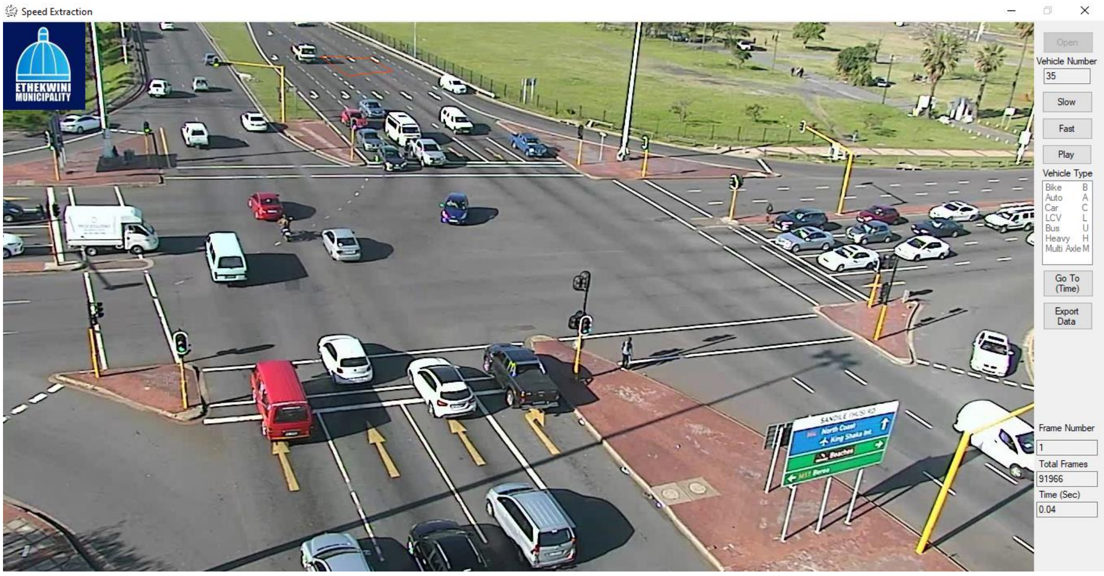

Figure 3. 9 : Photo showing speed extraction using TDE software

From figure 3.9 above, the red box in the photo shows the point that had been selected for speed determination. As vehicles moved through, manual punching of the computer keyboard as the vehicle hit the two points was done and the software used that along with the recorded timestamp to determine the speed which was then stored in an output file that was exported to a Microsoft Excel file.

3.3.4.4 Geometric data

In studying the influence of rainfall on the performance of signalised intersections, road geometry was found to also have an impact, and as such geometrical data was collected and recorded. Geometrical data was collected on the surveyed sites by on-site direct measurement and was counter-checked with measurements on Google earth. The data collected here were lane width, intersection width, number of lanes, and the number and length of turning lanes. A summary of collected geometric data is presented in table 3.4 below.

Table 3. 4 : Summary of geometric data for all study sites

Signalised Intersection features	Sites 001 and 002	Sites 003 and 004
Name of Intersection	Umgeni road / Alpine road	Sandile Thusi road / Stalwart Simelane Street
Number of lanes on major road	3	3
Number of exclusive right- turning lanes on major road	2	2
Number of lanes on minor road	2	2
Number of exclusive right-turning lanes on minor road	1	2
Lane width (m)	3.3	3.5
Width of intersection (m)- major road	34	37
Width of intersection (m) - minor road	40	45
Distance from rain gauge	1.5 Km	1.0 Km

3.3.3.5 Traffic data

Traffic data collected included: red-light running, traffic volume, vehicle composition, turning movements, discharge headway, lost time, and speed. Traffic volume was considered a key parameter for this study as it plays a role in the determination of other parameters like capacity and delay. Speed and acceleration data were collected using data extraction software. These were important in the evaluation of driver behaviour during the yellow and all-red durations. Speed was collected at a designated point on the intersection approach lane and second at the stop-line. The difference between the two speeds and the time difference was then used to determine the acceleration. Volume indicated the traffic demand being served and was extracted by counting the number of vehicles passing the stop-line of the intersection. In close conjunction with the volume were the turning movements which indicated the direction the vehicles moved. Through traffic volumes and all turning movements were counted and recorded. The data collection included an hour for both morning and evening peak. Vehicle composition was considered important too because different vehicle types have different operational capabilities and behaviour. Vehicle composition considered three vehicle types; passenger cars which were all two-axle vehicles, heavy goods vehicles (vehicles with more than three axles except buses), and medium commercial vehicles (buses). Manual observation and counting were used and therefore when any of the different vehicle types were encountered, they were recorded, and the data was presented in terms of percentage of the total volume. Headways and lost time data were also manually recorded. Headway is used in the determination of saturation flow rate and saturation flow rate and lost time are important in the determination of capacity. A stopwatch was used to record headways at the vehicles crossed the intersection stop-line at the onset of the green time. Headway was recorded from front bumper to front bumper. The lost time and saturation headway were then obtained as outlined in chapter 2.

3.3.3.6 Rainfall data

Rainfall data was a key input because the study was centred around determining the influence rainfall has on red-light running and signalised intersection performance. Rain gauge stations were marked out and the rainfall data was obtained from the eThekwini website and counterchecked with one maintained by the university's Electrical Engineering department. The rainfall data obtained was in terms of the amount and duration, this was then converted to rainfall intensity (mm/h) and then classified depending on the quantity. Figure 3.10 below shows a sample of the data that was obtained eThekwini website, and which was used for this study.

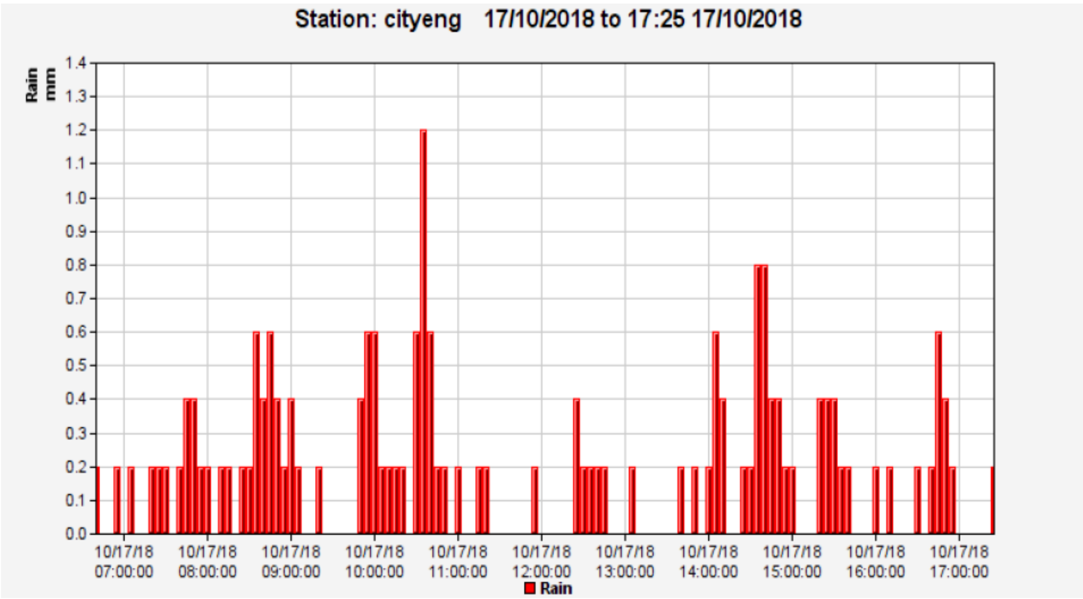

Figure 3. 10 : Sample of rainfall data for rain gauge station for site 003 and 004

Rainfall intensity was obtained by dividing the amount recorded by the time duration. The rainfall intensity data obtained was then classified into light rainfall (L), with rain intensity < 2.5mm/h; moderate rainfall (M), with intensity 2.5 – 10mm/h; and heavy rainfall (H), with the intensity of 10 – 50mm/h, per the World Meteorological Organization (WMO). Very heavy rainfall with intensity > 50mm/h was not considered in this research work because of drag force effect on tyres and for the duration and the specific times studied this rainfall intensity was not recorded.

3.4 Pilot test of equipment and analytical method

In the process of choosing the most appropriate equipment and analytical method to use for this study, several steps were taken, different equipment and methods were explored. The first equipment that was explored involved the use of BS (ATC). The ATC uses pneumatic air sensors to collect data and can be used to collect such traffic data as traffic volume, vehicle type, vehicle speed, gap, headway, and axle numbers among others. From these collected data other derived traffic parameters can then be estimated.

For this pilot study, the ATC used was the MetroCount 5600 series. The component of the device consists of the following: pneumatic tubes, the tubes are installed on the selected study point in a way that they are perpendicular to the direction of vehicle movement. When a vehicle tyre hits the tube, a pulse is generated and transferred to the roadside sensing device. The second component is a roadside air sensing device which is the device that is installed at the roadside and connected to the pneumatic tubes and it records the pulses transmitted from the tubes. Another component of the ATC is the steel case that houses the roadside air sensing device and is meant for protection of the sensor device.

A USB communication cable is also among the components and is used to connect the sensing device to a computer system to set up data collection, check on the status from time to time and to download the recorded data. In addition, some accessories are required to install the ATC equipment and include bitumen tapes to keep the tubes flat of the road surface, cleats and nails to tightly fasten the tubes (at the end, away from bitumen surface) to the road and to keep them tight through the data collection period. Any changes on the tubes could have an impact on the accuracy of the collected data.

A survey was done and identified a signalised intersection that was used as a pilot study site. The site was selected for its proximity to the University of Kwa-Zulu Natal. Proximity was important to guarantee the safety of the equipment as well as ensure that constant monitoring was done. The pilot study setup involved the installation of ATC.

The setup involved laying two sets of pneumatic tubes across all lanes at all approaches of the intersection. The tubes were laid past the intersection stop-line and were set to be were 1m apart. The intersection chosen had three lanes on each approach, two lanes for each entry, and one lane for exit. The tubes were laid and fixed as per the guidelines provided in the manuals. Care was taken to ensure that the road surface was not interfered with and as such all nailing was done at the shoulders. On the bitumen surface, heavy-duty bitumen tape was used to fasten the tube. In addition to the steel casing to protect the ATC equipment, additional precaution was taken. This involved protecting the sensor device complete with the steel casing in a wooden box. The wooden box was secured with a lock and chained onto an immovable roadside post for extra security. Figures 3.11 and 3.12 below show a typical setup of the plot study sites, showing all the tubes, roadside units covered, and everything well secured.

Figure 3. 11: Pilot Study site setup

Figure 3. 12: Pilot study site setup showing tube layout

With the physical installation complete, the ATC was fixed, and the data collection started. Proper coding of all the approach lanes was done. This was done in a way to enable the identification of all the different movements that take place at the intersection. Data were collected continuously on the site for five days. In coming up with the appropriate tube layout that would give the most accurate data, different tube layouts were tested. Figures 3.12 above show the first and typical tube layout that was first used. This involved laying two tubes across the entire width of the intersection, covering two entry lanes and one exit lane.

The tube layout presented challenges, on inspection of the generated data, it was noted that in most cases some vehicles were double-counted (multiple entries for the same vehicle). This was especially noted at the entry and was attributed to the fact that because of two lanes, when vehicles move over the tubes at the same time, the pulse sent to the roadside sensor had problems where it was likely recorded as one vehicle or in other cases recorded as two vehicles but with zero headway. With this challenge, different tube layouts were tried out to mitigate it. One solution is shown in figure 3.13 below involved the second tube covering only one lane and the one in figure 3.14 where only one tube was laid across all three lanes. These new layouts did however not yield any improvements in the results recorded.

Figure 3. 13 : Pilot study site tube layout across one lane

Figure 3. 14: Pilot study site showing one tube layout

Each of the tube layouts shown above was tested for a few days, where the tubes were laid, data collected, and preliminary analysis done to extract any useful information out of it. During each setup, the ATC roadside sensor unit was configured and coded to reflect the changes. The data recorded on the first day after the installation was discarded, this was to allow drivers to get used to the presence of the tubes. During the first day, drivers may have adjusted their behaviour because of the presence of the tubes. Permission was obtained from ETA to do the installation and the Durban Metro Police Department was also informed. At the end of each data collection period, the data was unloaded from the roadside unit. The unloading was done by using the USB cable to connect the unit to the laptop and following the given steps to unload. It should be noted that the ATC equipment was accompanied by the corresponding software that is used for the installation and unloading of data. The unloaded data was then checked for relevance and application.

3.4.1 Challenges encountered with pilot setup

Some challenges were noted with this equipment and setup for data collection. Some of the installed tubes were found on several occasions to have been cut, whether because of failure or vandalism could not be established, this meant the loss of data on some days and very close monitoring of the site.

The second and biggest challenge came in analyzing the collected data. Logically it is expected that the headway between two successive vehicles should be more than 0 seconds and each vehicle entry to have a unique combination. This was however not the case, several vehicles were shown to have 0 seconds of headway and one vehicle had multiple entries, the screenshot in figure 3.15 below highlights this problem. This was attributed to the fact that at each intersection approach, the entry had two lanes in the same direction. This means that at the start of green when the vehicles started to move, vehicles on the two lanes moved at the same time but as the roadside unit as the pulse came from around the same spot and more so in the same direction it was recorded as one. Tube layouts were changed as earlier mentioned but each layout yielded similar results where it was not possible to isolate the movements. This major challenge coupled with the fact that signal information had to be obtained differently and matching of the two done to make analysis it was decided to seek another approach.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W
16	\b	Direction:\b	1	-	North	bound.	\b	Lane:\b0	0\par														
17	\b	Survey	Duration:	11:52	Monday	July	23	2018	=>	15:44	Wednesd	August	8	2018	\par								
18	\b	Zone:\b0\tab\par																					
19	\b	File:\b0\tab	2	PILOT	STATION	C_MK	0	8/8/2018	1544.EC(	(Plus	)\par												
20	\b	Identifier:\b	NA3342	MC5900-X11	(c)MetroCount	15-Aug-16	\par																
21	\b	Algorithm:\	Factory	default	axle	(v5.02)\par																	
22	\b	Data	type:\b0\	Axle	sensors	-	Separate	(Count)\par															
23	\par																						
24	\ul\b	Profile:\ulnone\b0\par																					
25	\b	Filter	time:\tab	11:53	Monday	July	23	2018	=>	15:44	Wednesd	August	8	2018	(16.1607)\par								
26	Included	classes:\b0	1	2	3	4	5	6	7	8	9	10	11	12	13\par								
27	\b	Speed	range:\b	10	-	160	km/h.\par																
28	\b	Direction:\b	North	East	South	West	(bound)	P	=	\ul	North\ulr	Lane	=	0-16\par									
29	\b	Separation	Headway	>	0	sec	Span	0	-	100	metre\par												
30	\b	Name:\b0\t	Default	Profile\par																			
31	\b	Scheme:\b	Vehicle	classification	(Scheme	F3)\par																	
32	\b	Units:\b0\ta	Metric	(metre	kilometre	m/s	km/h	kg	tonne)\f1\fs16\par														
33																							
34	\pard\page\f0\par																						
35	\f1\fs18\par																						
36	Trig	Num	Ht	YYYY-MM-D	hh:mm:ss	Dr	Speed	Wb	Hdwy	Gap	Ax	Gp	Rho	Cl	Nm	Vehicle\par							
37	1	00003d70	13	7/23/2018	14:27:52	NO	41.46	0.82	17.6	11.6	2	2	0.55	2	118	PC	o	o	-	Coerced	sequenc	3\cf1\par	
38	1	00003d70	13	7/23/2018	14:27:52	NO	41.46	0.82	0	0	2	2	0.55	2	118	PC	o	o	2\par				
39	1	00003d70	13	7/23/2018	14:27:52	NO	41.46	0.82	0	0	2	2	0.55	2	118	PC	o	o	1\par				
40	1	3.00E+95	6	7/23/2018	14:31:09	NO	16.34	1.3	197	196.2	2	2	0.67	2	0000414	PC	o	o	-	Coerced	sequenc	2\par	
41	1	3.00E+95	6	7/23/2018	14:31:09	NO	16.34	1.3	0	0	2	2	0.67	2	0000414	PC	o	o	1\par				
42	1	00003f19	15	7/23/2018	14:32:38	NO	19.54	5.43	88.9	88.1	2	2	0.33	2	000071c	PC	o	o	-	Coerced	sequenc	2	'\par
43	1	00003f19	15	7/23/2018	14:32:38	NO	19.54	5.43	0	0	2	2	0.33	2	000071c	PC	o	o	1	'\par			

Figure 3. 15: Pilot study results showing data anomalies

The second approach involved seeking out intersections that have one entry lane and one exit lane on each approach. Those intersections were however found to have very low traffic volumes even during the peak periods and to be on semi-actuated traffic control throughout the day. Traffic volumes were deemed important because a queue of more than 10 vehicles is needed to estimate saturation flow rate values. The fixed traffic control was chosen over the actuated control because the main goal was to determine the influence of rainfall and was thus important to keep all other variations to a minimum so that the rainfall impact can be isolated.

3.4.2 Video data for the pilot study

The third approach was to explore the use of video data. For this approach, the eThekwini Transport Authority and the Durban Metro Police (Disaster Management Unit) played a key role. Signalised intersections were identified that fit the selection criteria as mentioned above. A check was done alongside the ETA to find which of the intersections had CCTV coverage and were on fixed time during the morning and evening peak. With the selection narrowed down, another challenge came, where most of the installed cameras were dynamic (because they were mainly used for safety purposes), a further narrowing down of those that had static cameras was done. The Disaster Management Unit was then approached with the final list of selected intersections to provide footage for the selected days and times. With the obtained footage, the next step involved seeking methods to extract the required data. The first step involved running the video footage through video editing software (VSDC was used for this study). With the software, the footage was cut down to an hour each for the morning and evening. Further editing included adjusting light and other features where some of the footage had problems associated with lighting or the sun glares.

Data extraction was done using two means. Manual observation was the main one, this was used to obtain data like red-light running(frequency), headway (for saturation flow rate), traffic volume, turning movements, and vehicle composition. The second method was the use of software, Traffic Data Extractor (TDE) developed by the India Institute of Technology (IIT) Mumbai. This was used to determine speed, acceleration, red-light running, and yellow light running cases. For determination of speed, a box was drawn covering the area where the speed was to be determined, and then as the vehicle moved through the box, manual pressing of keys on the computer recorded the vehicle and because the actual real-life dimensions of the boxed area were known, speed was recorded. The data recorded was then exported to excel sheets and were further analyzed. The recorded data included a timestamp for each recorded vehicle. Figure 3.16 below shows a screenshot of the software (showing speed collection) while figure 3.17 shows a portion of extracted data in an excel sheet.

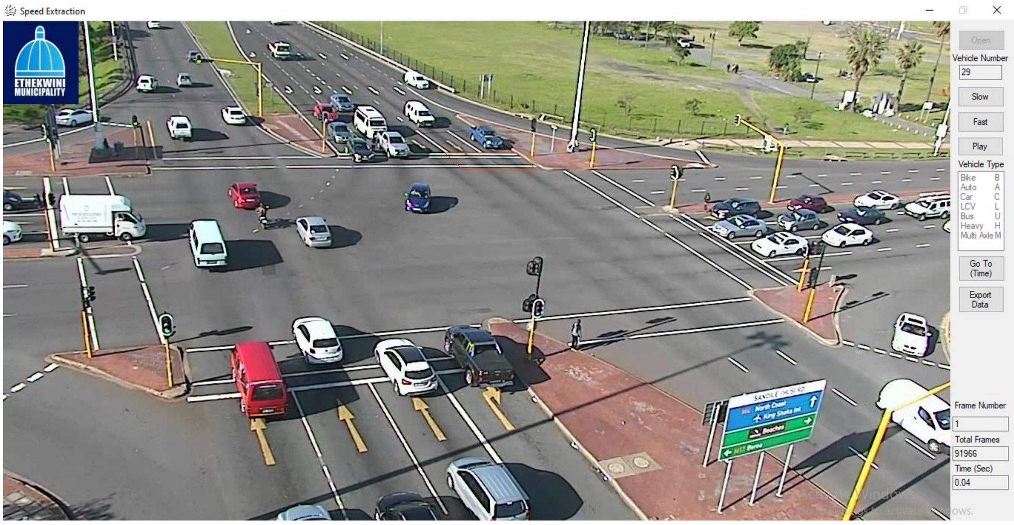

Figure 3. 16: Traffic data collection at intersection stop-line

	A	B	C	D	E	F	G
1	Veh No	Frame No	Start Time(s)	End Time	Veh Type	Speed(m/s)	
2	1	779	30.96	31.16	Car	22.5	
3	2	3271	130.56	130.84	Car	16.07143	
4	3	18313	732.36	732.52	Car	28.125	
5	4	18313	732.36	732.52	Car	28.125	
6	5	20777	830.84	831.08	Car	18.75	
7	6	20829	832.96	833.16	Car	22.5	
8	7	23292	931.4	931.68	Car	16.07143	
9	8	48299	1931.76	1931.96	Car	22.5	
10	9	55779	2230.92	2231.16	Car	18.75	
11	10	58301	2331.76	2332.04	Car	16.07143	
12	11	58315	2332.32	2332.6	Car	16.07143	
13	12	78297	3131.68	3131.88	Car	22.5	
14	13	80806	3232.12	3232.24	Car	37.5	
15	14	80812	3232.4	3232.48	Car	56.25	
16	15	80808	3232.12	3232.32	Car	22.5	
17	16	90794	3631.6	3631.76	Car	28.125	

Figure 3. 17: Sample of speed data collected from TDE software

The site that was selected for the pilot study was Sandile Thusi road / Stalwart Simelane Street. The pilot was done in August and September 2018 before the onset of the actual rainfall season. One-week data was obtained as mentioned above and was used to test the data extraction methods that have been mentioned above. The data extracted for the pilot study included: red-light running, saturation headway, traffic volume, turning movements, and vehicle composition. From the extracted data saturation flow rate was estimated and the influence of light rainfall intensity (only light rainfall was recorded during the pilot study duration) was calculated and it was shown that rainfall does have an impact on the saturation flow rate. Further lost time, effective green, and capacity were estimated using the extracted data and all were shown to be influenced by rainfall.

3. 4.3 Impact of rainfall on red-light running

Pilot site (Sandile Thusi road / Stalwart Simelane Street) was used. A binary logistic regression model was used to predict the probability of red-light running. Speed, sight distance based on yellow light influence length concept,, and travel time were used in the prediction as shown in Table 3.5. Speed was obtained using the traffic data extractor software and acceleration was calculated using the speeds measured at two designated points (one at a distance equal to yellow time multiplied by the speed limit and the second point at the intersection stop-line). Red-light running (RLR) was the dummy variable, 1 for RLR, and 0 for stopping on time. To determine the probability of driver stopping or RLR the regression model is:

$$Logit\ (P) = ln\ (P/1 - P) \qquad 3.2$$

Which can be rewritten as:

$$L_p = ln\left(\frac{P}{1-P}\right) = \beta_o + \beta_1\tau + \beta_2 v + \beta_3 d \ for\ 0 < \mathrm{P} < 1 \qquad 3.3$$

Table 3. 5: Dry W Travel time, Speed and Sight distance at Pilot site

RLR y = 0	Travel Time (s)	Speed (m/s)	AD (m)	RLR y = 1	Travel Time (s)	Speed (m/s)	AD (m)
0.00	3.08	16.25	45.78	1.00	2.56	23.21	78.50
0.00	3.23	15.48	42.63	1.00	3.11	19.12	58.30
0.00	3.00	16.67	47.51	1.00	3.11	19.12	58.30
0.00	3.25	15.38	42.26	1.00	2.75	21.67	70.55
0.00	4.25	11.76	28.87	1.00	2.75	21.67	70.55
0.00	3.27	18.18	54.07	1.00	2.01	29.55	115.07
0.00	4.00	12.50	31.41	1.00	3.11	19.12	58.30
0.00	4.25	11.76	28.87	1.00	2.65	22.50	74.78
0.00	5.00	10.00	23.11	1.00	2.38	25.00	88.16
0.00	3.75	13.33	34.41	1.00	2.20	27.08	100.08
0.00	4.50	11.11	26.67	1.00	2.73	21.76	71.04
0.00	3.57	14.00	36.89	1.00	3.36	17.69	51.91
0.00	4.38	11.41	27.66	1.00	2.98	20.00	62.42
0.00	3.31	15.11	41.17	1.00	2.88	20.67	65.62
0.00	3.03	16.50	46.82	1.00	3.43	17.33	50.35
0.00	3.44	14.53	38.89	1.00	3.26	18.29	54.53
0.00	3.50	14.29	37.97	1.00	3.11	19.12	58.30
0.00	3.25	15.38	42.26	1.00	2.68	22.22	73.36
0.00	4.75	10.53	24.77	1.00	2.65	22.50	74.78
0.00	4.25	11.76	28.87	1.00	3.27	18.18	54.07
0.00	5.00	10.00	23.11	1.00	3.27	18.18	54.07
0.00	3.75	13.33	34.41	1.00	2.98	20.00	62.42
0.00	3.18	15.71	43.59	1.00	2.98	20.00	62.42
0.00	4.25	11.76	28.87	1.00	3.27	18.18	54.07
0.00	3.50	14.29	37.97	1.00	2.68	22.22	73.36

*Note that RLR denotes Red-light running

From table 3.5, the 85th percentile speed of red-light running drivers is 82km/h (2.78m/s) and for drivers stopping at the traffic light is 57km/h (15.93m/s), thus suggesting that drivers complying with posted speed limits are more likely to stop at the traffic light line. Whereas drivers with speed over the legal limit are more like to violate red lights at the signalised intersections. Of course, these are preliminary but useful findings, the question is; will drivers violating red lights be able to maintain their speeds during rainfall. It has been shown in many studies that rainfall causes speed reduction.

Note that travel time to the stop line was measured as the time taken to cover 50m length to the stop line (the distance covered in 3 seconds of yellow time at the posted speed of 16.67m/s). At the onset of yellow light, the dependent variable was represented by y, where y = 0 means the driver brings the vehicle to a halt at the stop-line and y = 1 denotes that the driver proceeded beyond the stop-line into the intersection at the onset of red light. The probability of a driver's choice at the onset of a yellow signal can be calculated by:

The probability of RLR estimated by $P(Y = 1) = \frac{e^{\beta_0+\beta_1\tau+\beta_2 v+\beta_3 d_3}}{1+ e^{\beta_0+\beta_1\tau+\beta_2 v+\beta_3 d_3}}$ 3.4

The probability of stopping estimated by $P(Y = 0) = \frac{1}{1+ e^{\beta_0+\beta_1\tau+\beta_2 v+\beta_3 d_3}}$ 3.5

Where:

P represents the probability of a driver's choice, d is stopping distance

β_0 represents the constant of the model and β_1, β_2, β_3 are model coefficients

τ is travel time to the stop line, $t_f\left(1 + \partial\left[\frac{T_v}{Q}\right]^{\mu}\right)$ for ∂=0.4 and μ = 10

T_v denotes traffic volume and Q is signalised intersection capacity

v is the vehicle's approach speed at the onset of a yellow time interval

By fitting binary regression models, analysis was done for dry weather and under light rainfall for the pilot site. From table 3.6, the *t'* values all have an absolute value greater than 2.5; therefore, all the variables used in the regression equation are useful in predicting red-light running violation. The R^2 value (0.92) is greater than 0.5, and the F value is greater than 4 suggesting the model equation did not happen by chance. The model equation for dry weather is; $y = -11.62 + 1.245t + 0.865v - 0.135d$ [1]

Table 3. 6: Summary of model coefficients for dry weather

t' values	-10.2418	12.40353	11.11224	-12.9764
Coefficients	**-0.13504**	**0.864955**	**1.24456**	**-11.6185**
Std. Error	0.013186	0.069735	0.111999	0.895357
R^2	**0.917503**	0.149725	#N/A	#N/A
F *df*	170.5326	46	#N/A	#N/A
Residuals	11.46879	1.031209	#N/A	#N/A
Variables	*d*	*v*	*t*	constant

The posted speed limit at the pilot test site was 60km/h (16.67m/s) and the yellow light time was 3s. Assuming that the distance to stop line, d = 50m (16.67m/s *x* 3s), given the model equation 1 above.

Probability of RLR, $p(y = 1) = \frac{e^{\beta_0+\beta_1 t+\beta_2 v+\beta_3 d_3}}{1+ e^{\beta_0+\beta_1 t+\beta_2 v+\beta_3 d_3}} = \frac{e^{\langle -11.62\, +\, 1.24t+0.86v-0.14d\, \rangle}}{1+ e^{\langle -11.62\, +\, 1.24t+0.86v-0.14d\, \rangle}}$ = 44.6%

Probability of stopping, $p(y = 0) = \frac{1}{1+ e^{\beta_0+\beta_1 t+\beta_2 v+\beta_3 d_3}} = \frac{1}{1+ e^{\langle -11.62\, +\, 1.24t+0.86v-0.14d\, \rangle}}$ = 55.4%

Based on the model equation prediction, the probability of red-light running during dry weather conditions at the pilot site was 44.6% whereas the probability of stopping was 55.4%. Assuming that the speed limit rule is rigorously enforced at the pilot site and the distance to stop line clearly marked, what is the best yellow time and distance to stop line combination it may be queried. To estimate the best combination, the distance values 60m, 70m, and the yellow time values 2s, 3s, and 4s were used iteratively with results shown below in table 3.7.

Table 3. 7: Dry Weather Sensitivity Analysis for Pilot Site

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	18.8%	81.2%
	3s	44.6%	55.4%
	4s	73.6%	26.4%
60	2s	5.7%	94.3%
	3s	17.2%	82.8%
	4s	42.0%	58.0%
70	2s	1.5%	98.5%
	3s	5.1%	94.9%
	4s	15.8%	84.2%

Based on the outcomes in Table 3.7, increasing yellow light time from 3s to 4s will increase red-light running whilst decreasing yellow light time from 3s to 2s will decrease red-light running under dry weather conditions. In any case, a regression model equation was developed for the same pilot site under light rainfall conditions, and the results are shown below in Table 3.8. The light rainfall regression model for the pilot site takes the form:

$$y = -2.92 + 0.8679t + 0.0069v - 0.0052d \qquad [2]$$

Table 3. 8: Summary of model coefficient for light rainfall at Pilot Site

Variables	*d*	*v*	*t*	Constant
t' values	2.675171	6.633226	17.14881	-16.8021
Coefficients	**-0.00521**	**0.006859**	**0.867858**	**-2.92193**
Std. Error	0.002112	0.01034	0.028386	0.173903
R^2	0.918898	0.088952	#N/A	#N/A
F df	139.7383	37	#N/A	#N/A
Residuals	3.316997	0.292759	#N/A	#N/A

Note: d – stopping distance, v-speed, and t – travel time

From table 3.8, the *t'* values all have an absolute value greater than 2.5; therefore, all the variables used in the regression equation are useful in predicting red-light running violation. The R^2 value is greater than 0.5, and the F value is greater than 4 suggesting the model equation did not happen by chance.

Probability of RLR, $p(y=1) = \frac{e^{\beta_0+\beta_1 t+\beta_2 v+\beta_3 d_3}}{1+e^{\beta_0+\beta_1 t+\beta_2 v+\beta_3 d_3}} = \frac{e^{\langle -2.92\,+\,0.8679t+0.0069v+0.0052d\rangle}}{1+e^{\langle -2.92\,+\,0.8679t+0.0069v+0.0052d\rangle}}$ = 33.8%

Probability of stopping, $p(y=0) = \frac{1}{1+e^{\beta_0+\beta_1 t+\beta_2 v+\beta_3 d_3}} = \frac{1}{1+e^{\langle --2.92\,+\,0.8679t+0.0069v+0.0052d\rangle}}$ = 66.2%

The probability of red-light running during light rainfall is 33.8%, there is a reduction in the probability of red-light running during rainfall because of the speed reduction caused by rainfall. It reduces from 44.6% to 33.8%. Summary of the regression model equations for dry and light rainy conditions are shown below in Table 3.9.

Table 3. 9: Summary of Probability RLR during dry and light rainfall

Weather	***Model Equations***	R^2	***RLR, y = 1***	***RLR, y = 0***
Dry	$y = -11.62 + 1.245t + 0.865v - 0.135d$	0.91	44.6%	55.4%
LR	$y = -2.92 + 0.868t + 0.0069v + 0.0052d$	0.92	33.8%	66.2%

Note: LR denotes red-light running, d – stopping distance, v-speed and t – travel time

The probability of RLR decreased from about 45% under dry weather conditions to 34% under light rainfall conditions. The probability of stopping during light rainfall increased from 55% to 66%.

3.4.4 Implications for Service Quality delivery at Signalised Intersections

To determine the implications for service quality delivery at signalised intersections:

Step 1: involved determination of discharge headway, saturation headway, and thus saturation flow rate. The discharge headway was determined manually from the videos by replaying the videos and using a stopwatch to manually record the vehicles as they crossed the intersection stop-line. This was done for vehicles in the queue at the onset of the green time and up to the 14^{th} vehicle. Figure 3.18 below shows the differences in discharge headway values between the dry and light rainfall conditions.

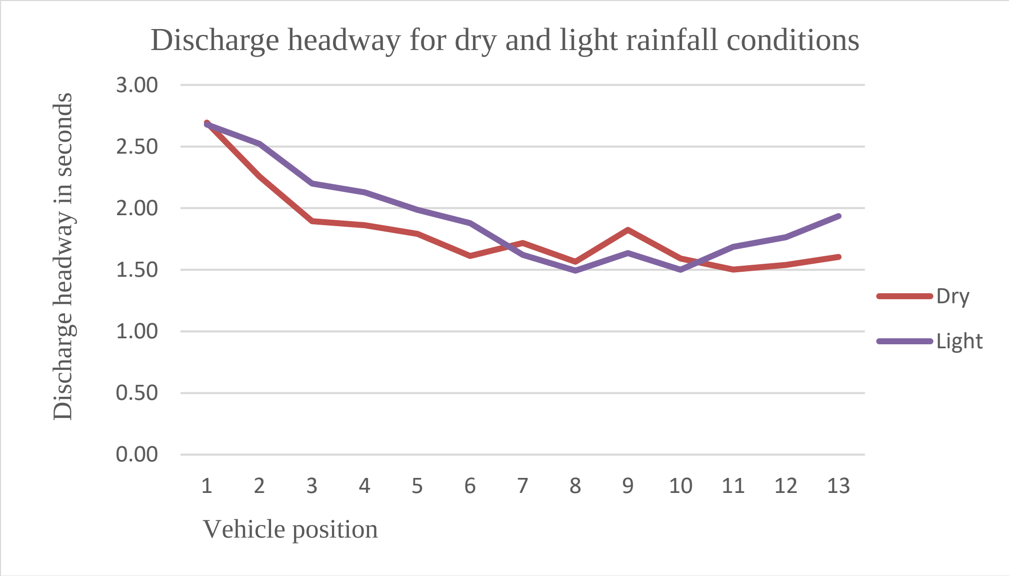

Figure 3. 18: Discharge headway comparison for dry and light rainfall conditions

From the discharge headway values obtained and as displayed in figure 3.18 above, the headway values started to be constant around the 4th vehicle position. This was then used (5th to the last vehicle) to determine the saturation headway and then the saturation flow rate. For dry conditions, the average saturation headway was 1.60 seconds while for light rainfall conditions it was 1.70 seconds. The discharge headway was recorded and saved on an excel sheet where the averages were obtained from. The recordings were done for over 15 cycles as recommended in the HCM, 2016, and equation 3.6 below was used to determine individual headways and the average was then obtained using excel.

$$h_s = \frac{T_n - T_4}{n} \qquad 3.6$$

Given the saturation headway, the saturation flow rate was determined using equation 3.7

$$S = \frac{3600}{h_s} \qquad 3.7$$

For dry conditions = $S = \frac{3600}{1.60}$ = 2250 veh/h

For light rainfall conditions = $S = \frac{3600}{1.70}$ = 2118 veh/h

Light rainfall led to an increase in saturation headway and caused the saturation flow rate to decrease.

Step 2: this involved determination of passenger car equivalent (PCE) values. The traffic composition showed that apart from passenger cars, there were medium and heavy goods vehicles in the traffic. To determine the PCE values, the average headway method was used. The average headway between two cars following each other was determined as was the average between two medium vehicles and two heavy vehicles, expression 3.4 was then used to determine the PCE value for each vehicle type. The determination was done for both dry and light rainfall conditions since it is based on headway values which are affected by weather conditions.

$$PCE = \frac{\bar{h}_i}{\bar{h}_c} \qquad 3.8$$

Where: h_i is the average headway of vehicle under consideration, h_c is average headway for passenger car

For the dry weather conditions, the PCE values were:

For Passenger cars = $PCE = \frac{1.65}{1.65}$ = 1

For medium vehicles = $PCE = \frac{2.51}{1.65}$ = 1.52

For heavy vehicles = $PCE = \frac{3.29}{1.65}$ = 1.99

For light rainfall intensity:

For Passenger cars = $PCE = \frac{1.70}{1.70}$ = 1

For medium vehicles = $PCE = \frac{2.68}{1.70}$ = 1.58

For heavy vehicles = $PCE = \frac{3.42}{1.70}$ = 2.01

The results are summarised in Table 3.10 below. The table also gives the PCE values in the South African Geometric Design manual (SANRAL, 2011) that apply to the country.

Table 3. 10: PCE values

Vehicle type	Weather condition		
	Dry	Light rain	SANRAL
PC	1	1	1
MV	1.52	1.58	1.8
HV	1.99	2.01	2.3

The last part involved a test for significant difference between determined PCE values and those provided by SANRAL. To test this, two hypotheses are made between the determined PCE and SANRAL PCE values.

The hypothesis for Medium vehicles and dry weather are:

(i) Null hypothesis (H_1): The Values of the PCE are the same

(ii) Alternate hypothesis (H_2): The PCE values are not the same.

The test is carried out with chi-square using chi-square Equation 3.9 below:

$$X^2 = \frac{(o-e)2}{e} \tag{3.9}$$

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

The test is carried out at 95% level of confidence, where $X^2 < 3.841$ means there is no significant difference between the two variables.

O = 1.52 e_1 = 1.8

$$X^2 = \frac{(1.52-1.80)^2}{1.80} = 0.04 < 3.841$$

Null hypothesis (H_1) is accepted, and alternate hypothesis (H_2) is rejected, this shows there is no significant difference between the estimated PCE values and those provided by SANRAL. The test was carried out for heavy vehicles as well as under light rainfall conditions. All showed no statistical difference between the estimated and SANRAL PCE values at 5% level of significance. Given no statistical difference, in this case, the SANRAL values were adopted.

With the determined PCE values and the vehicle composition that was recorded, the saturation flow values obtained were modified to passenger cars per hour. This conversion was done using an excel sheet and the results are:

For dry conditions S= 2195 pc/h

For light rainfalll conditions S= 2095 pc/h

Step 3: determination of start-up lost time and effective green. Start-up lost time (SULT) is the time lost at the beginning of the phase. It was determined during the recording of the discharge headway where the difference between the saturation headway and the first four discharge headways was summed up as shown in equation 3.10 below. This was done using excel where the discharge headway values recorded.

$$SULT = \sum_{1}^{4} \Delta n \qquad 3.10$$

Where Δn = headway difference between n[th] vehicle and saturation headway

For dry conditions SULT = 2.15 seconds (from excel)

For light rainfall conditions SULT = 2.64 seconds (from excel)

The clearance lost time was taken as 2 seconds in each case per the HCM

The effective green time for a given movement is the sum of displayed green, yellow, and all-red clearance intervals less the total lost time as given by equation 3.10 below.

$$g = G + Y + AR - t_L \qquad 3.10$$

Where: g = Effective green time in seconds, G = displayed green time in seconds, Y = Yellow time in seconds, AR = All red interval, t_L = total lost time (sum of SULT and clearance lost time)

For dry conditions g = 36 +3+ 3 – 4.15 = 37.85 s
For light rainfall g = 36 +3+ 3 – 4.64 = 37.36 s

Step 4: in this step, the capacity and degree of saturation for the two weather conditions were determined. To calculate the capacity the equation 3.11 was used. The saturation flow rate and effective green time were as determined in the previous steps and the cycle time was 100 for this pilot case. The cycle time for the period considered was fixed.

$$c = s * \left({}^{g}/_{c_o} \right) \qquad 3.11$$

Where: c = Capccity in veh/h, g = effective green time in seconds, C_o = cycle time (s) , s= saturation flow rate

For dry conditions $c = 2195 * \left({}^{37.85}/_{100} \right)$ = 831 veh/h

For light rainfall $c = 2093 * \left({}^{36.36}/_{100} \right)$ = 782 veh/h

Rainfall causes a reduction in capacity as can be seen from the above values. In line with capacity, another parameter considered important and used in the determination of delay is the degree of saturation X. The degree of saturation is the ratio of the volume to capacity and is determined as per equation 3.12

$$X = \frac{v}{c} \qquad 3.12$$

Where: v= traffic volume (veh/h), c= capacity (veh/h)

To calculate the degree of saturation, therefore, traffic volume had to be measured. Volume was measured manually by replaying the videos and manually counting the vehicles as they passed the stop-line, care was also taken to record the direction of movement. For this pilot, for example, only the through movement was considered. The volume was measured for one hour however the recording was in intervals of five minutes. Figure 3.19 below highlights the difference in 5-minute traffic distribution for the two weather conditions.

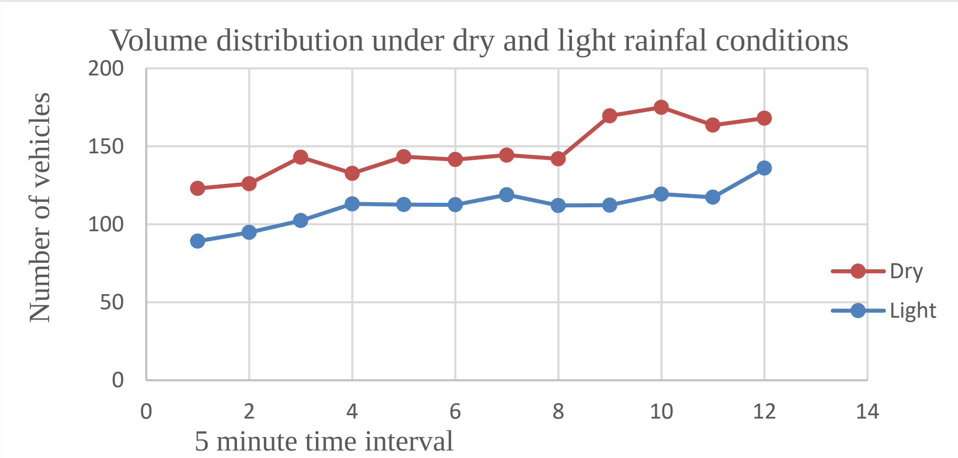

Figure 3. 19: Volume distribution for dry and light rainfall conditions

The calculated degree of saturation (X) values are:

Dry conditions $X = \frac{587}{831} = 0.71$

For light rainfall conditions $X = \frac{408}{782} = 0.52$

The light rainfall degree of saturation is lower than that of dry weather and this is because of the differences in volume. In both cases, however, the degree of saturation was less than 1, showing that these through traffic lanes are not oversaturated.

Step 5: estimation of delay values for the two cases. To estimate delay, a comparison of UK method and HCM was done, for the UK Method equations 3.13 to 3.16 below were used**:**

$$\text{Delay} = d_1 + d_2 + d_3 \qquad 3.13$$

$$\text{Regular arrival, } d_1 = \frac{c(1-\lambda)^2}{2(1-\lambda x)} \qquad 3.14$$

$$\text{Random arrival, } d_2 = \frac{x^2}{2q(1-x)} \qquad 3.15$$

$$\text{Correction factor, } d_3 = 0.65\left(\frac{c}{q^2}\right)^{1/3} x^{(2+5\lambda)} \qquad 3.16$$

Where: C – Cycle length, s note that $C = \frac{1.5L+5}{1-\sum y_m}$, q – Demand flow, veh/s, x = degree of saturation $\lambda = \frac{g_e}{C}$ and that g_e is effective green

While the Highway Capacity Manual (HCM, 2016) method used equations 3.17 to 3.20 below were used.

$$d = d_1(PF) + d_2 + d_3 \qquad 3.17$$

Where : d= control delay per vehicle in seconds per vehicle (s/veh), d_3- initial queue delay

$$d_1 = \frac{0.5\left(1-{}^{g}/_{C}\right)^2}{1-[min(1,X)^{g}/_{C}]} \qquad 3.18$$

$$PF = \frac{(1-P)fpA}{1-({}^{g}/_{C})} \qquad 3.19$$

Where: PF = Progressions adjustment factor, P = proportion of vehicles arriving on green

f_{pA} = supplemental adjustment factor for platoon arriving during green

$$d_2 = 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{8klX}{cT}}\right] \qquad 3.20$$

Where:

d_2 = incremental delay, k = incremental delay factor that is dependent on controller settigs

l = upstream filtering adjustment factor

PF = 1, k= 0.5, l= 1, X= 0.71

For dry weather conditions (HCM) $d_1 = \frac{0.5\left(1-{}^{37.83}/_{100}\right)^2}{1-[0.71*{}^{37.83}/_{100}]}$ = 26.42 s

$$d_2 = 900 * 1\left[(0.71-1) + \sqrt{(0.71-1)^2 + \frac{8*1*0.5*0.71}{831*1}}\right] = 5.25\text{s}$$

Total delay: d= 19.33*1+ 5.25+ 0 = 31.67 s

For dry weather conditions (UK)

Delay = $d_1 + d_2 + d_3$ = 26.42+5.33= 31.75s

Regular arrival, $d_1 = \frac{100(1-37.83/100)^2}{2\left(1-\frac{37.83}{100}*0.71\right)}$= 26.42s

Random arrival, $d_2 = \frac{0.71^2}{2*587/3600(1-0.71)}$= 5.33 s

The two methods gave very similar results. The HCM method was adopted for this study because in South Africa the HCM is used generallly and refernced in the South Africa Geometric design manual. Secondly as the aim of the study was to compare dry and rainy weather conditions and as long as one method was used to determine delay in all instances a comparison could be made.

For light rainfall conditions:

PF = 1, k= 0.5, l= 1, X= 0.52

$$d_1 = \frac{0.5\left(1-{}^{37.36}/_{100}\right)^2}{1-\left[0.52*{}^{37.36}/_{100}\right]} = 24.35 \text{ s}$$

$$d_2 = 900 * 1\left[(0.52-1) + \sqrt{(0.52-1)^2 + \frac{8*1*0.5*0.52}{782*1}}\right] = 2.49\text{s}$$

Total delay: d= 19.33*1+ 5.25+ 0 = 27.14 s

For this case, d_3 was zero but d_1 and d_2 were calculated and the control delay for this lane group was determined.

Step 6: Development of service quality criteria table. The new criteria table is an expansion of the LOS table provided in the HCM. It is expanded to include the degree of saturation(X) in addition to the delay. To do this, a range of X values were used, starting from 0 where the delay is purely geometric and going all the way to X= 1.0 where the intersection is operating at capacity. X values were increased in intervals of 0.1 from 0 to 1.0

For example, for X = 0

$$d_1 = \frac{0.5\left(1-{}^{37.83}/_{100}\right)^2}{1-\left[0*{}^{37.83}/_{100}\right]} = 19.33 \text{ s}$$

$$d_2 = 900 * 1\left[(0-1) + \sqrt{(0-1)^2 + \frac{8*0*0.5}{831*1}}\right] = 0$$

This was done for all values of X and the summary is presented in table 3.11 below

If the criteria table is divided as per the divisions of the X values, there would be overlaps in the delay values would be too close to each other. Dividing into five divisions was therefore found more reasonable. With the overlap in divisions 0 to 0.3, they were put into one class and the rest divided into divisions of 0.2.

For X=0 to 0.3 standard deviation was used to determine the boundaries of the class and to ensure that no overlap occurs with the adjacent classes.

Mean delay for X = 0 to 0.3 = $\frac{19.33 + 20.00 + 21.50 + 22.73}{4}$ = 20.89

Standard deviation = 1.53 (from excel)

Lower boundary = 20.89 – 1.53 = 19.36 ≈ 20.00 s

Upper boundary = 20.89 + 1.53 = 22.43 s ≈ 23.00s

Table 3. 11: Summary of the degree of saturation /delay

X	Delay (s)
0	19.33
0.1	20.00
0.2	21.50
0.3	22.73
0.4	24.21
0.5	26.00
0.6	28.23
0.7	31.30
0.8	40.93
0.9	46.87
1	93.30

The next class considered X = 0.4 and 0.5

Mean delay = $\frac{24.21 + 26.00}{2}$ = 25.11 s

Standard deviation = 1.27

Lower boundary = 25.11 – 1.27 = 24.84 ≈ 25 s

Upper boundary = 25.1 +1.27 = 26.38 ≈ 27 s

Class X = 0.6 and 0.7

Mean delay = $\frac{28.23+ 31.30}{2}$ = 29.77 s

Standard deviation = 2.17

Lower boundary = 29.77 – 2.17 = 27.6 ≈ 28 s

Upper boundary = 29.77 +2.17 = 31.94 ≈ 32 s

Class X = 0.8 and 0.9

Mean delay = $\frac{40.93+ 46.87}{2}$ = 43.90 s

Standard deviation = 4.20

Lower boundary = 43.90 – 4.20 = 39.7 ≈ 40 s

Upper boundary = 43.90 +4.20 = 48.1 ≈ 49 s

For X = 1

Delay = 93.30 ≈ 94 s

Queue length = 37

The classification of A to F as used in HCM was maintained and the boundary values calculated above were used in the formulation of the new functional quality of service table. The full criteria table 3.12 is presented below.

Table 3. 12: New SQ criteria table

SQ class	X	Delay(s/veh)
A	$0 < X \leq 0.3$	$0 < d \leq 22$
B	$0.3 < X \leq 0.5$	$22 < d \leq 27$
C	$0.5 < X \leq 0.7$	$27 < d \leq 32$
D	$0.7 < X \leq 0.9$	$32 < d \leq 49$
E	$0.9 < X \leq 1.0$	$49 < d \leq 94$
F	$X > 1$	$d > 94$

Step 7: the new SQ criteria was compared to that of the HCM and the Canadian capacity guide (S. Teply., 2008). The results of the comparison are in table 3.13

Table 3. 13: Comparison between new SQ and HCM / Canadian guide

Level	**Novel SQ criteria**		HCM LOS	Canadian LOS guide	
	X	Delay (s/veh)	Delay (s/veh)	Delay (s/veh)	X
A	$0 < X \leq 0.3$	$0 < d \leq 22$	≤ 10 ≥	≤ 10 ≥	0.0 - 0.59
B	$0.3 < X \leq 0.5$	$22 < d \leq 27$	>10 and ≤ 20	>10 and ≤ 20	0.60 - 0.69
C	$0.5 < X \leq 0.7$	$27 < d \leq 32$	>20 and ≤ 35	>20 and ≤ 35	0.70 - 0.79
D	$0.7 < X \leq 0.9$	$32 < d \leq 49$	>35 and ≤ 55	>35 and ≤ 55	0.80 - 0.89
E	$0 .9 < X \leq 1.0$	$49 < d \leq 94$	>55 and ≤ 80	>55 and ≤ 80	0.90 - 0.99
F	$X > 1$	$d > 94$	> 80	> 80	1.0 or >

Note X denotes a degree of saturation

From the table above comparing the criteria, for the delay, the HCM upper value is 80 s while for the new criteria it is 94s. This implies that the HCM delay class is underestimated. The lower boundary similarly for new criteria is 22 s while HCM is 10. For the X values, comparing the new table and the Canadian capacity guide, apart from the first two classes, there is a similarity in the divisions. Delay does not appear in the other two guides.

Step 8: the newly developed criteria table was then used to determine the quality of service for the pilot site

under dry and light rainfall conditions. The X and delay values were calculated for the two cases using the equations mentioned in step 6.

For dry conditions: X= 0.72, Delay= 31.67s

For light rainfall conditions: X= 0.52, Delay = 27.14s

For dry conditions, it is in SQ level C. For light rainfall conditions all parameters place it in level C. Light rainfall intensity has an impact on the level of service, however, the impact is not that significant in this case and that could be due to the light intensity of the rainfall, coupled with the volume reduction. It can therefore be concluded that rainfall causes capacity reduction.

3.5 Summary

The chapter described the methods used to undertake this study. The survey that took nine months to complete was conducted in Durban, South Africa between July 2019, and May 2019. Rainfall and dry weather conditions data were collected at selected sites. The criteria used to select an appropriate study site was discussed. The calculated sample size of 15000 vehicles < the empirical population survey of 100,000 vehicles used in this study. On the evidence of empirical data and video recordings taken at sites it could be suggested that several factors can account for service quality reduction and red-light running consequences at signalised intersections in Durban, South Africa; they may include among others, driver behaviour, yellow interval and cycle time setting, absence of road marking disclosure.to road users on approach to stop line, vague yellow interval rule and vehicle approach speed.

Analysis from the preliminary investigation suggests a distinctive pattern of red-light running reduction under the influence of rainfall and as a result, a deterioration in service delivery was recorded. Under the influence of rainfall there was recorded low speed and high discharge headway. In any case, one is not completely sure of the parameters trend till the isolated empirical result is compared against findings from unrelated sample survey data. By virtue of the isolated nature of the data on which the preliminary investigation was based, the results of the service delivery reduction analysis conducted, at best could be described as broadly suggestive. Consequently, the data in this chapter begs several questions about the influence of rainfall red-light running and consequences on service delivery reduction at signalised intersections. For this purpose, the next chapters will address these issues considering light, moderate and heavy rainfall. In addition, their effects on passenger car equivalency values, saturation flow rate, discharge headways, and capacity loss will also be looked into.

CHAPTER 4.

EMPIRICAL SURVEY RESULTS

4.1 Overview

This chapter deals with the empirical data that was collected using the methods that were described in chapter 3. The data collected ranged from geometric, traffic and rainfall for all the selected study sites. Dry weather and different rainfall intensity rainfall data were collected. All the data was important in achieving the study aim of evaluating intersection performance and driver behaviour during the green, yellow, and red-light durations and under dry and wet weather conditions.

The rest of this chapter is organised as follow: section 4.2 details the empirical data for all sites, section 4.2.1 and 4.2.2 details data for sites 001 and 002 respectively (Umgeni road and Alpine road) including site figures and photos. Section 4.2.3 and 4.2.4 presents data collected from sites 003 and 004 respectively (Sandile Thusi road / Stalwart Simelane street) including site figures and photos. Section 4.3 gives the chapter summary

4.2 Empirical Results from Surveyed Sites

The full-scale field data collection activity was conducted on the four selected sites located within the eThekwini municipality. Time schedule of this exercise was as shown in Table 4.1 below and was selected based on the annual rainfall season in South Africa. Data was collected for one hour during the morning and evening peak, continuously for the dry days and as and when it was recorded for the different rainfall intensities. All data collection was restricted to the period indicated in table 4.1. On site, signal timing, geometric data, volume, vehicle speed, headway, and vehicle type were recorded. While nearby rain gauge station provided 5 minutes rainfall resolution data.

Table 4. 1: Data collection schedule

Site Coding	Survey Site	Direction	Highway	Field Data Collection	No. of days **
001	Umgeni road / Alpine road	South	M19	Sept 2018 to March 2019	50
002	Umgeni road / Alpine road	North	M10	Sept 2018 to March 2019	50
003	Sandile Thusi road / stalwart Simelane Street	South	M17	Sept 2018 to March 2019	45
004	Sandile Thusi road / stalwart Simelane Street	North	M4	Sept 2018 to March 2019	45

** Dry weather data was collected for 30 days, and rainfall data was collected when it occurred in the data collection period

Geometric data was also collected on all four study sites and the results are summarised in table 4.2 below. From the table, the intersection geometry shows that the intersection was designed as per the South Africa Geometric design guidelines. Lane widths varied between 3.3m to 3.5m, exclusive lanes were provided for right turning traffic on the major roads. On the minor roads, the lanes were for mixed through and turning traffic. The turning lanes were flare lanes varying in length from 60m to 100m.

Table 4. 2: Data collection sites

Intersection features	Site 001	Site 002	Site 003	Site 004
Name of Intersection	Umgeni road / Alpine road (North)	Umgeni road / Alpine road (South)	Sandile Thusi road / Stalwart Simelane Street (North)	Sandile Thusi road / Stalwart Simelane Street (South)
Pavement surface type	Asphalt	Asphalt	Asphalt	Asphalt
Number of lanes on major road	3	3	3	3
Number of exclusive right-turning lanes on major road	2	1	2	1
Number of lanes on minor road	2	2	2	2
Number of exclusive right-turning lanes on minor road	1	1	2	2
Lane width (m)	3.3	3.3	3.5	3.5
Width of intersection (m)-major road	34	34	37	37
Width of intersection (m) -minor road	40	40	45	45
Road signs and marking	OK	OK	OK	OK
Distance from rain gauge	1.5 Km	1.5 Km	1.0 Km	1.0 Km

4.2.1 Empirical results for site 001

Umgeni road / Alpine road intersection is a busy four arm intersection in the city of Durban. The location is such that Umgeni road connects to the national road while Alpine road connects to a busy shopping complex within the CBD. This means that the intersection is quite busy and especially during the peak periods. The figures 4.1 and 4.2 below shows photos of the intersection, figure 4.1 shows a view of all approaches of the intersection while figure 4.2 shows an aerial view of all the four approaches. Results obtained for both traffic and rainfall data are further described below.

Figure 4. 1: Photograph of site 001 (Umgeni road / Alpine road)

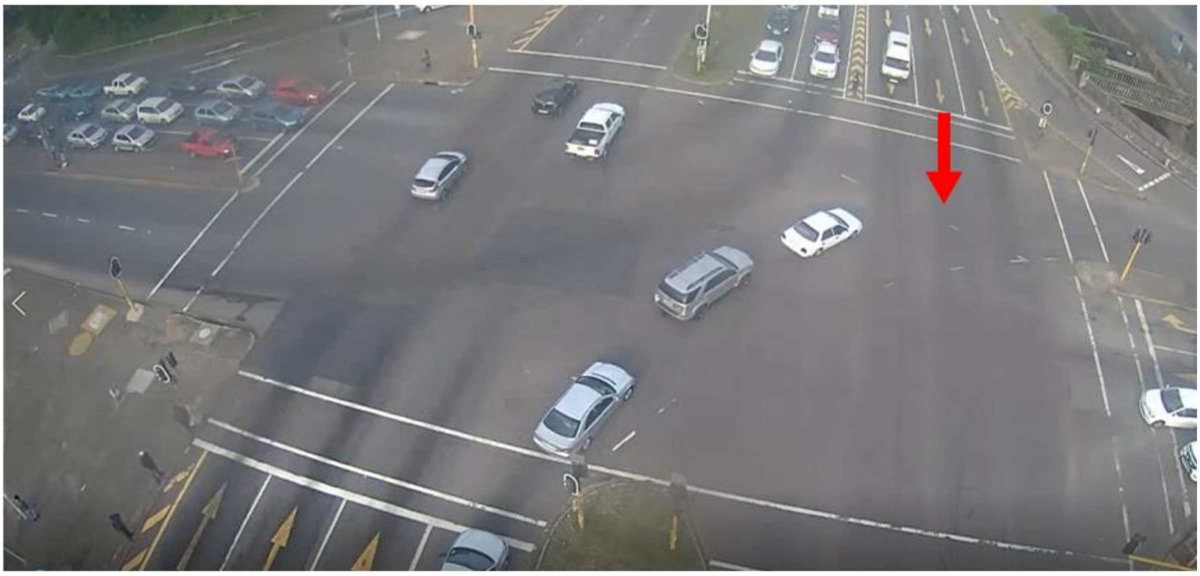

Figure 4. 2: Aerial view of Site 001 showing direction of movement

4.2.1.1 Signalisation data

The intersection operates on different modes of control depending on the time of the day and day of the week. For weekdays, the morning and evening peaks are on fixed time. The rest of the day is on semi-actuated control. Only weekday peak data was used for this study. The cycle time was 120 seconds divided into six phases (also called stages as per SARTSM) an excerpt of the signal information is provided in Appendix B.

4.2.1.2 Rainfall data

To collect rainfall data, as was mentioned earlier the eThekwini department of Engineering services was used as the primary source. The Department provides a website where all the rain gauges within the municipality are controlled and the login details provided were used to log in and access the information for the required days. The rain gauge station used for this site was the Ridgeend location, which was about 1.6 Km from the intersection location. During the study period, all the days that rainfall was recorded (excluding weekends) were noted, and the data was then extracted from the website. An example of the data format obtained for this site is presented in figure 4.3. The figure shows a 20-minute interval of rain which was possible to further break down to 5-minute intervals for purposes of calculating the rainfall intensity. The conversion to intensity was done by dividing the rainfall amount by the duration and the results recorded in mm/h. The obtained rainfall intensity was then used to classify the rainfall into light, moderate or heavy rainfall intensity. In collecting rainfall data care was taken to ensure that for each of the rainfall intensities a minimum of five days was collected.

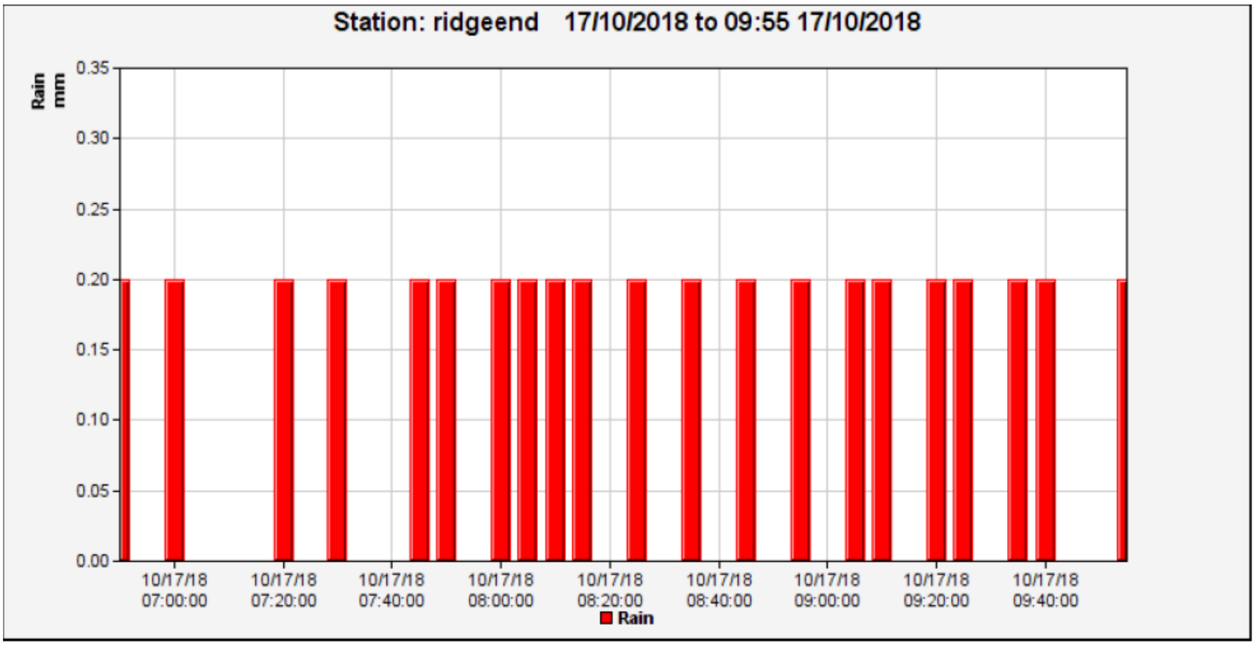

Figure 4. 3 : Sample of rainfall data record for Site 001

4.2.1.3 Traffic data

A range of traffic data were collected and used in this study, as the focus was impact of rainfall on red-light running, speed data was collected, time and frequency (Number of RLR cases) of RLR. In addition, the traffic volume (traffic demand), vehicle composition, and saturation flow data were collected to be used in determining service quality implications of rainfall. The rest of the data required was derived from these major ones. Table 4.3 gives the dry weather time, Speed, and sight distance (AD) data for site 001.

Table 4. 3: Dry Weather Travel time, Speed and Sight distance at site 001

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
3.25	15.38	42.60	0	2.22	22.50	70.31	1
3.88	12.90	34.13	0	2.14	23.33	73.89	1
3.86	12.96	34.33	0	2.29	21.88	67.68	1
4.00	12.50	32.81	0	2.71	18.42	53.81	1
4.00	12.50	32.81	0	2.32	22.60	70.74	1
4.57	10.94	27.86	0	2.14	23.33	73.89	1
3.62	13.79	37.10	0	2.14	23.33	73.89	1
4.50	11.11	28.40	0	2.43	20.59	62.37	1
3.75	13.33	35.56	0	2.57	19.44	57.79	1
5.00	10.00	25.00	0	2.29	21.88	67.68	1
4.86	10.29	25.89	0	2.57	19.44	57.79	1
3.71	13.46	35.98	0	2.43	20.59	62.37	1
3.13	16.00	44.80	0	2.71	18.42	53.81	1
3.13	16.00	44.80	0	1.75	28.57	97.96	1
3.62	13.79	37.10	0				
3.75	13.33	35.56	0				
3.86	12.96	34.33	0				
4.14	12.07	31.42	0				
3.57	14.00	37.80	0				
4.00	12.50	32.81	0				
3.57	14.00	37.80	0				
4.57	10.94	27.86	0				
4.00	12.50	32.81	0				
4.14	12.07	31.42	0				
3.25	15.38	42.60	0				
3.75	13.33	35.56	0				
4.00	12.50	32.81	0				

The same data was collected for the same site under light, moderate and heavy rainfall intensities and summary of the data is presented in tables 4.4 to 4.6 below.

Table 4. 4: Light Rainfall intensity Travel time, Speed and Sight distance at site 001

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.86	12.96	34.33	0	3.29	15.22	42.01	1
4.00	12.50	32.81	0	3.14	15.91	44.47	1
4.00	12.50	32.81	0	3.14	15.91	44.47	1
4.57	10.94	27.86	0	2.14	23.33	73.89	1
3.62	13.79	37.10	0	2.29	21.88	67.68	1
3.75	13.33	35.56	0	2.86	17.50	50.31	1
5.00	10.00	25.00	0	2.32	22.60	70.74	1
4.86	10.29	25.89	0	4.43	11.29	28.95	1
3.71	13.46	35.98	0	3.29	15.22	42.01	1
3.13	16.00	44.80	0	3.14	15.91	44.47	1
3.62	13.79	37.10	0	3.14	15.91	44.47	1
3.75	13.33	35.56	0	2.57	19.44	57.79	1
3.86	12.96	34.33	0	3.71	13.46	35.98	1
4.14	12.07	31.42	0	3.14	15.91	44.47	1
4.57	10.94	27.86	0	2.57	19.44	57.79	1
4.14	12.07	31.42	0	3.29	15.22	42.01	1
4.57	10.94	27.86	0	2.71	18.42	53.81	1
4.14	12.07	31.42	0				
4.29	11.67	30.14	0				
4.14	12.07	31.42	0				
5.00	10.00	25.00	0				
4.86	10.29	25.89	0				
3.75	13.33	35.56	0				
4.00	12.50	32.81	0				

Table 4. 5: Moderate Rainfall intensity Travel time, Speed and Sight distance at site 001

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.86	12.96	34.33	0	2.57	19.44	57.79	1
3.29	15.22	42.01	0	2.71	18.42	53.81	1
4.57	10.94	27.86	0	2.57	19.44	57.79	1
4.57	10.94	27.86	0	2.57	19.44	57.79	1
3.29	15.22	42.01	0	2.86	17.50	50.31	1
3.57	14.00	37.80	0	3.88	17.39	49.91	1
3.43	14.58	39.80	0	2.86	17.50	50.31	1
3.57	14.00	37.80	0	2.57	19.44	57.79	1
3.57	14.00	37.80	0	3.14	15.91	44.47	1
4.57	10.94	27.86	0	2.86	17.50	50.31	1
3.86	12.96	34.33	0	2.86	17.50	50.31	1
4.29	11.67	30.14	0	3.14	15.91	44.47	1
4.86	10.29	25.89	0	2.67	18.75	55.08	1
4.29	11.67	30.14	0	2.86	17.50	50.31	1
3.43	14.58	39.80	0	2.57	19.44	57.79	1
5.00	10.00	25.00	0	2.86	17.50	50.31	1
4.14	12.07	31.42	0	3.75	13.33	35.56	0
4.14	12.07	31.42	0	4.00	12.50	32.81	0
4.29	11.67	30.14	0	3.29	15.22	42.01	0
4.14	12.07	31.42	0	3.57	14.00	37.80	0
3.29	15.22	42.01	0	3.43	14.58	39.80	0
3.43	14.58	39.80	0	3.50	14.29	38.78	0
5.00	10.00	25.00	0				
4.86	10.29	25.89	0				
3.71	13.46	35.98	0				
4.63	10.81	27.47	0				
3.25	15.38	42.60	0				
3.50	14.29	38.78	0				
3.75	13.33	35.56	0				
4.13	12.12	31.59	0				

Table 4. 6: Moderate Rainfall intensity Travel time, Speed and Sight distance at site 001

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
4.63	10.81	27.47	0	3.13	16.00	44.80	1
3.29	15.22	42.01	0	3.29	15.22	42.01	1
3.29	15.22	42.01	0	3.29	15.22	42.01	1
4.57	10.94	27.86	0	3.11	16.07	45.06	1
4.57	10.94	27.86	0	3.29	15.22	42.01	1
3.57	14.00	37.80	0	2.57	19.44	57.79	1
3.43	14.58	39.80	0	3.26	15.34	42.45	1
3.57	14.00	37.80	0	3.29	15.22	42.01	1
3.57	14.00	37.80	0	3.14	15.91	44.47	1
4.57	10.94	27.86	0	3.29	15.22	42.01	1
3.86	12.96	34.33	0	3.14	15.91	44.47	1
4.29	11.67	30.14	0	2.71	18.42	53.81	1
4.86	10.29	25.89	0	3.14	15.91	44.47	1
4.29	11.67	30.14	0	2.67	18.75	55.08	1
3.43	14.58	39.80	0	3.14	15.91	44.47	1
5.00	10.00	25.00	0	3.29	15.17	42.13	1
4.14	12.07	31.42	0	3.29	15.22	42.01	1
4.14	12.07	31.42	0	4.13	12.12	31.59	0
4.29	11.67	30.14	0	3.75	13.33	35.56	0
4.86	10.29	25.89	0	5.80	8.62	20.96	0
3.29	15.22	42.01	0	4.00	12.50	32.81	0
3.43	14.58	39.80	0	3.29	15.22	42.01	0
5.00	10.00	25.00	0	3.57	14.00	37.80	0
4.86	10.29	25.89	0	3.43	14.58	39.80	0
3.71	13.46	35.98	0	3.50	14.29	38.78	0
4.63	10.81	27.47	0				
3.25	15.38	42.60	0				
3.50	14.29	38.78	0				
3.75	13.33	35.56	0				

For the additional traffic data table 4.7 below gives volume and headway data for site 001. The recording of the traffic volume was done for one hour each, but the counting was done per five-minute interval. For the hour, 12 five-minute intervals were recorded. Speed was recorded during the yellow and all-red durations. This was because, speed was mainly used in determination of driver responsiveness to yellow light and red-light running. Therefore, on average the speed values recorded were higher than the posted speed limit of 16.67m/s. Saturation headway was recorded in terms of vehicle position as the vehicles crossed the intersection stop-line at onset of green duration. The headway values were recorded for vehicle queues of above 10 vehicles and up to 14 vehicles.

Table 4. 7: Typical traffic volume and headway data for site 001

Site 001- Dry		
Period/ (Vehicle position for h_s)	Volume (veh/5mins)	Headway (s)
1	188	2.47
2	129	2.38
3	185	1.83
4	132	1.90
5	191	1.56
6	138	1.62
7	184	1.51
8	137	1.32
9	185	1.37
10	138	1.44
11	194	1.39
12	132	1.30

*h_s is headway

The other parameter measured in collecting traffic data was vehicle composition. Traffic composition data was considered important because different vehicle types have different operational capabilities, and the composition affects such things as saturation flow and other parameters all the way to capacity. In recording the different vehicle types, three classes of vehicles were considered: passenger cars (2 axle load vehicles) denoted by the letter C, light trucks, and vehicles with 3 axle loads as well as minibuses were classified as medium vehicles denoted by letter M and heavy truck and lorries with more than 3 axles loads as well as heavy buses were classified as heavy goods vehicles and denoted by letter H. The results for site 001 are presented in table 4.8. It can be noted that passenger cars were the dominant vehicle type, making more than 90% of vehicle type on all recorded days.

Table 4. 8: Traffic composition for site 001

Site 001- Dry							
Date	Volume	C		M		H	
		Number	Percentage (%)	Number	Percentage (%)	Number	Percentage (%)
01/10/2018	1655	1565	94.56	33	2.01	57	3.45
02/10/2018	1589	1517	95.47	29	1.83	44	2.77
04/10/2018	1566	1458	93.10	45	2.87	63	4.02
05/10/2018	1632	1496	91.67	59	3.62	77	4.72
08/10/2018	1403	1319	94.01	33	2.35	51	3.64
10/10/2018	1595	1484	93.04	44	2.76	68	4.26
11/10/2018	1728	1604	92.82	47	2.72	78	4.51
12/10/2018	1700	1572	92.47	51	3.00	77	4.53
15/10/2018	1607	1505	93.65	42	2.61	60	3.73
18/10/2018	1665	1536	92.25	57	3.42	72	4.32
19/10/2018	1776	1619	91.16	74	4.17	83	4.67
22/10/2018	1533	1426	93.02	41	2.67	66	4.31
23/10/2018	1637	1541	94.14	33	2.02	63	3.85
24/10/2018	1892	1775	93.82	42	2.22	75	3.96
25/10/2018	2082	1942	93.28	48	2.31	92	4.42
26/10/2018	1857	1732	93.27	57	3.07	68	3.66

The study was centred around “dry” versus“rainy” and as such, traffic data was also collected during varying intensities of rainfall. The intensities were classified as either light, moderate or heavy. The data collected for volume and headway is summarized in table 4.9. From the table, as rainfall intensity increased, there was a reduction in traffic volume whereas the headway increased.

Table 4. 9: Typical traffic volume and headway for varying rainfall conditions for Site 001

Site 001: Dry and rainy								
Period/ (Vehicle position for headway)	Dry		Light rain		Moderate rain		Heavy rain	
	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)
1	188	2.47	185	1.81	175	2.46	169	2.3
2	129	2.38	129	1.74	120	1.88	116	2.18
3	185	1.83	181	1.85	172	2.02	167	2.08
4	132	1.9	129	1.88	123	1.71	119	1.86
5	191	1.56	189	1.62	178	1.52	172	1.82
6	138	1.62	135	1.67	128	1.71	124	1.59
7	184	1.51	179	1.68	171	1.53	166	1.59
8	137	1.32	136	1.69	127	1.5	123	1.59
9	185	1.37	178	1.74	172	1.6	167	1.55
10	138	1.44	134	1.62	128	1.63	124	1.45
11	194	1.39	188	1.75	180	1.59	175	1.56
12	132	1.3	127	1.66	123	1.63	119	1.44
Total	*1933 veh/hr*		*1890 veh/hr*		*1797 veh/hr*		*1741 veh/hr*	

4.2.2 Empirical results for site 002

The figures 4.4 and 4.5 below shows photos of study site 002, figure 4.4 shows a view of all approaches of the intersection while figure 4.5 shows an aerial view of all the four approaches. Results obtained for both traffic and rainfall data are further described below.

Figure 4. 4: Photograph of site 001 (Umgeni road / Alpine road)

Figure 4. 5: Aerial view of Site 001 showing direction of movement

4.2.2.1 Signalisation data

The cycle time was 120 seconds divided into six phases (also called stages as per SARTSM) an excerpt of the signal information is provided in the Appendix B.

4.2.2.2 Rainfall data

The rain gauge station used for this site was the Ridgeend location, which was about 1.6 Km from the intersection location. During the study period, all the days that rainfall was recorded (excluding weekends) were noted, and the data was then extracted from the website. An example of the data format obtained for this site is presented in figure 4.6. The figure shows a 20-minute interval of rain which was possible to further break down to 5-minute intervals for purposes of calculating the rainfall intensity.

The conversion to intensity was done by dividing the rainfall amount by the duration and the results recorded in mm/h. The obtained rainfall intensity was then used to classify the rainfall into light, moderate or heavy rainfall intensity. In collecting rainfall data care was taken to ensure that for each of the rainfall intensities a minimum of five days was collected.

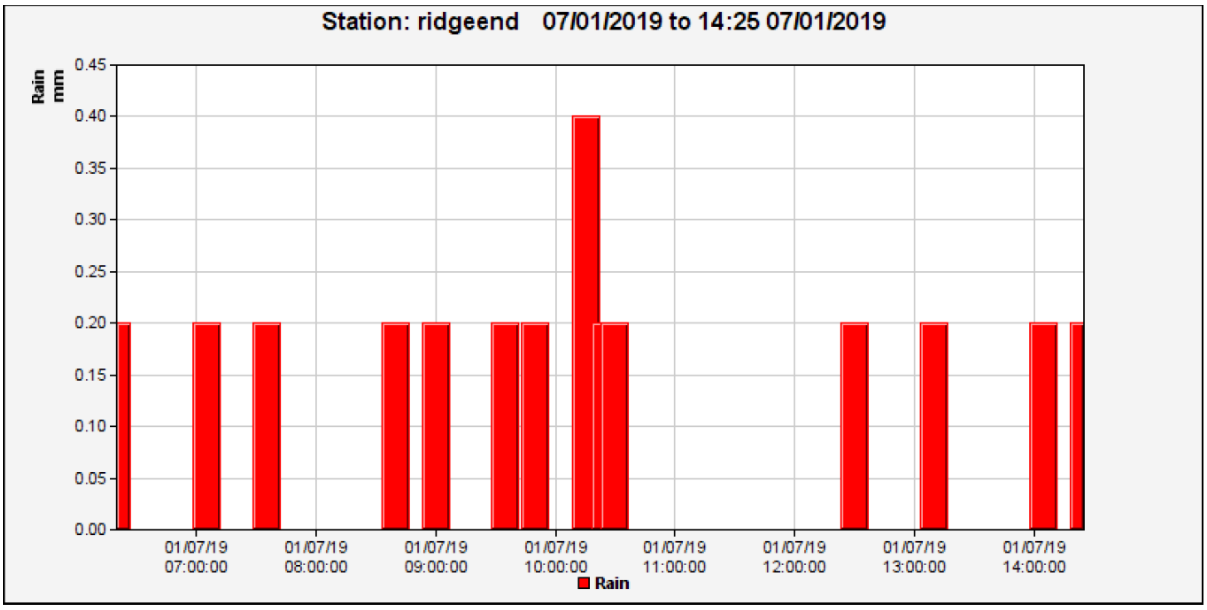

Figure 4. 6: Sample of rainfall data record for Site 002

4.2.2.3 Traffic data

A range of traffic data were collected and used in this study, as the focus was impact of rainfall on red-light running, speed data was collected, time and frequency (Number of RLR cases) of RLR. In addition, the traffic volume (traffic demand), vehicle composition, and saturation flow data were collected to be used in determining service quality implications of rainfall. The rest of the data required was derived from these major ones. Table 4.10 gives the dry weather time, Speed, and sight distance (AD) data for site 001 while tables 4.11 to 4.13 give light, moderate and heavy rainfall intensity data.

Table 4. 10: Dry Weather Travel time, Speed and Sight distance at site 002

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
4.00	12.50	32.81	0	2.29	21.87	67.68	1
3.57	14.00	37.80	0	2.43	20.59	62.37	1
3.87	12.90	34.13	0	2.13	23.29	74.74	1
3.75	13.33	35.56	0	2.79	17.91	49.91	1
4.38	11.43	29.39	0	2.00	25.00	81.25	1
3.62	13.79	37.10	0	2.72	18.38	53.67	1
4.62	10.81	27.47	0	2.00	25.00	81.25	1
6.25	8.00	19.20	0	2.43	20.59	62.37	1
4.00	12.50	32.81	0	2.87	17.39	49.91	1
4.75	10.53	26.59	0	3.00	16.67	47.22	1
3.75	13.33	35.56	0	3.13	16.00	44.80	1
4.13	12.12	31.59	0	2.72	18.38	53.67	1
3.88	12.90	34.13	0	2.43	20.59	62.37	1
2.50	20.00	60.00	0	2.89	17.29	49.51	1
4.25	11.76	30.45	0	2.43	20.88	63.37	1
5.12	9.76	24.27	0	2.94	17.00	48.45	1
3.87	12.90	34.13	0				
3.75	13.33	35.56	0				
3.62	13.79	37.10	0				
5.13	9.76	24.27	0				
5.00	10.00	25.00	0				
5.50	9.09	22.31	0				
3.43	14.58	39.80	0				
4.43	11.29	28.95	0				
4.57	10.94	27.86	0				

Table 4. 11: Light Rainfall intensity Travel time, Speed and Sight distance at site 002

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.40	14.71	40.22	0	3.40	14.71	40.22	0
3.51	14.23	38.59	0	3.51	14.23	38.59	0
3.20	15.63	43.46	0	4.00	12.50	32.81	0
3.60	13.89	37.42	0	3.08	16.23	45.63	0
4.00	12.50	32.81	0	3.40	14.71	40.22	0
3.87	12.90	34.13	0	3.51	14.23	38.59	0
5.25	9.52	23.58	0	3.08	16.23	44.63	1
5.13	9.76	24.27	0	3.00	16.67	46.22	1
4.00	12.50	32.81	0	2.87	17.39	48.91	1
2.88	17.39	49.91	0	2.80	17.86	50.66	1
4.00	12.50	32.81	0	2.82	17.71	50.09	1
4.00	12.50	32.81	0	2.90	17.22	52.26	1
3.08	16.23	45.63	0	3.00	16.67	49.22	1
3.40	14.71	40.22	0	3.02	16.58	45.92	1
3.51	14.23	38.59	0	2.86	17.50	49.31	1
4.00	12.50	32.81	0	2.96	16.91	46.11	1
3.50	14.29	38.78	0	3.00	16.67	45.22	1
3.25	15.38	42.60	0	2.97	16.83	45.92	1
4.12	12.12	31.59	0	2.86	17.50	48.31	1
4.13	12.12	31.59	0	3.11	16.07	43.06	1
4.13	12.12	31.59	0	3.02	16.58	44.92	1
3.88	12.90	34.13	0	2.71	18.42	51.81	1
4.00	12.50	32.81	0				
2.40	20.83	63.37	0				
2.60	19.23	56.95	0				
2.00	25.00	81.25	0				
5.80	8.62	20.96	0				
4.00	12.50	32.81	0				
4.00	12.50	32.81	0				
3.08	16.23	45.63	0				

Table 4. 12: Moderate Rainfall intensity Travel time, Speed and Sight distance at site 002

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
3.43	14.58	39.80	0	3.00	16.67	47.22	1
3.14	15.91	44.47	0	2.86	17.50	50.31	1
4.29	11.67	30.14	0	2.71	18.42	53.81	1
4.14	12.07	31.42	0	3.00	16.67	47.22	1
4.43	11.29	28.95	0	2.86	17.50	50.31	1
4.57	10.94	27.86	0	2.86	17.50	50.31	1
4.14	12.07	31.42	0	3.14	15.91	44.47	1
5.14	9.72	24.17	0	2.43	20.59	62.37	1
3.86	12.96	34.33	0	2.86	17.50	50.31	1
4.00	12.50	32.81	0	3.00	16.57	47.22	1
3.29	15.22	42.01	0	3.14	15.91	44.47	1
4.43	11.29	28.95	0	3.29	15.22	42.01	1
3.43	14.58	39.80	0	2.29	21.88	67.68	1
3.86	12.96	34.33	0	3.29	15.22	42.01	1
3.86	12.96	34.33	0	3.00	16.67	47.22	1
4.00	12.50	32.81	0	2.86	17.50	50.31	1
3.57	14.00	37.80	0	3.00	16.27	47.22	1
5.25	9.52	23.58	0	2.86	17.50	50.31	1
5.25	9.52	23.58	0	3.14	15.91	44.47	1
3.51	14.23	38.59	0	3.14	15.91	44.47	1
3.50	14.29	38.78	0	3.00	16.67	47.22	1
3.60	13.89	37.42	0	2.29	21.88	67.68	1
5.80	8.62	20.96	0	3.14	15.91	44.47	1

Table 4. 13: Heavy Rainfall intensity Travel time, Speed and Sight distance at site 002

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
3.87	12.90	34.13	0	2.71	18.42	53.81	1
3.43	14.58	39.80	0	3.14	15.91	44.47	1
3.14	15.91	44.47	0	3.00	16.67	47.22	1
4.29	11.67	30.14	0	2.86	17.50	50.31	1
4.14	12.07	31.42	0	2.86	17.50	50.31	1
4.43	11.29	28.95	0	3.14	15.91	44.47	1
3.75	13.33	35.56	0	3.14	15.91	44.47	1
3.86	12.96	34.33	0	2.86	17.50	50.31	1
3.86	12.96	34.33	0	3.00	16.67	47.22	1
3.29	15.22	42.01	0	2.71	18.42	53.81	1
4.57	10.94	27.86	0	3.14	15.91	44.47	1
4.14	12.07	31.42	0	2.86	17.50	50.31	1
5.14	9.72	24.17	0	2.71	18.42	53.81	1
3.86	12.96	34.33	0	3.00	16.67	47.22	1
4.00	12.50	32.81	0	2.86	17.50	50.31	1
3.57	14.00	37.80	0	3.00	16.67	47.22	1
5.25	9.52	23.58	0	2.86	17.50	50.31	1
3.43	14.58	39.80	0	3.14	15.91	44.47	1
4.43	11.29	28.95	0	3.14	15.91	44.47	1
4.57	10.94	27.86	0	3.00	16.67	47.22	1
3.29	15.22	42.01	0	3.00	16.67	47.22	1
3.43	14.58	39.80	0	3.13	16.00	44.80	1
3.75	13.33	35.56	0	2.72	18.38	53.67	1
6.14	8.14	19.59	0	2.86	17.50	50.31	1

Table 4.14 gives volume and headway data for site 002. The recording of the traffic volume was done for one hour each, but the counting was done per five-minute interval. For the hour, 12 five-minute intervals were recorded. Speed was recorded during the yellow and all-red durations. This was because speed was mainly used in the determination of driver responsiveness to yellow light and red-light running. Therefore, on average, the speed values recorded were higher than the posted speed limit of 16.67m/s. Saturation headway was recorded in terms of vehicle position as the vehicles crossed the

intersection stop-line at the onset of green duration. The headway values were recorded for vehicle queues of above 10 vehicles and up to 14 vehicles.

Table 4. 14: Typical traffic volume and headway data for site 002

Site 002 – Dry		
Period/ (vehicle position for headway)	Volume (veh/5mins)	Headway (s)
1	178	2.48
2	114	2.04
3	170	1.91
4	111	1.78
5	171	1.57
6	118	1.56
7	167	1.55
8	114	1.55
9	166	1.52
10	112	1.44
11	164	1.50
12	108	1.51

Traffic composition data was considered important because different vehicle types have different operational capabilities, and the composition affects such things as saturation flow and other parameters all the way to capacity. In recording the different vehicle types, three classes of vehicles were considered: passenger cars (2 axle vehicles) denoted by the letter C, the light trucks, and vehicles with 3 axle loads as well as minibuses were classified as medium vehicles denoted by letter M and heavy truck and lorries with more than 3 axles loads as well as heavy buses were classified as heavy goods vehicles and denoted by letter H. The results for site 002 are presented in table 4.15. From the table, passenger cars made more than 90% of total vehicle composition. The high passenger car percentage can be attributed to this being peak period as well as the location being close to the central business district.

Table 4. 15: Traffic composition for site 002

Site 002 – Dry							
Date	Volume	C		M		H	
		Number	Percentage (%)	Number	Percentage (%)	Number	Percentage (%)
1/10/2018	1666	1575	94.56	33	2.01	57	3.45
2/10/2018	1599	1527	95.47	29	1.83	44	2.77
4/10/2018	1576	1467	93.10	45	2.87	63	4.02
5/10/2018	1643	1506	91.67	59	3.62	77	4.72
8/10/2018	1412	1328	94.01	33	2.35	51	3.64
10/10/2018	1605	1494	93.04	44	2.76	68	4.26
11/10/2018	1739	1614	92.82	47	2.72	78	4.51
12/10/2018	1711	1582	92.47	51	3.00	77	4.53
15/10/2018	1618	1515	93.65	42	2.61	60	3.73
18/10/2018	1676	1546	92.25	57	3.42	72	4.32
19/10/2018	1788	1630	91.16	75	4.17	83	4.67
22/10/2018	1543	1435	93.02	41	2.67	66	4.31
23/10/2018	1648	1551	94.14	33	2.02	63	3.85
24/10/2018	1904	1787	93.82	42	2.22	75	3.96
25/10/2018	2096	1955	93.28	48	2.31	92	4.42
26/10/2018	1869	1743	93.27	57	3.07	68	3.66

Data was also required to determine the impact rainfall has on the performance of signalised intersections. This data was collected during the dry and rainfall conditions. Table 4.16 gives traffic volume and discharge headway data for dry weather days and for the different rainfall intensities. The volume is given in 5-minute period intervals and discharge headway based on vehicle position. From the table it can be noted that on average the traffic volume reduces with increase in rainfall intensity whereas the discharge headway increases with increase in rainfall intensity.

Table 4. 16: Typical traffic volume and headway for varying rainfall conditions for Site 002

Site 002: Dry and rainy								
Period/ (Vehicle position for headway)	Dry		Light rain		Moderate rain		Heavy rain	
	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)
1	178	2.48	140	2.57	148	2.76	132	2.79
2	114	2.04	124	2.07	106	2.19	125	2.32
3	170	1.91	161	2.07	160	2.05	131	2.08
4	111	1.78	122	1.85	111	2.04	130	1.97
5	171	1.57	159	1.77	159	1.74	160	1.84
6	118	1.56	122	1.64	106	1.73	115	1.75
7	167	1.55	144	1.61	165	1.63	140	1.7
8	114	1.55	138	1.64	110	1.71	120	1.84
9	166	1.52	150	1.6	167	1.62	150	1.73
10	112	1.44	134	1.58	118	1.66	110	1.74
11	164	1.5	153	1.61	165	1.55	149	1.72
12	108	1.51	129	1.55	115	1.52	129	1.61
Total	*1693 veh/hr*		*1676 veh/hr*		*1630 veh/hr*		*1591 veh/hr*	

4.2.3 Empirical results for site 003

Site 003 is the intersection between Sandile Thusi road and Stalwart Simelane street. The intersection is near the city's south beach as well as the Moses Mabidha stadium that hosts sporting as well as entertainment events in the city. The intersection is one of the busiest intersections as shown by traffic volumes obtained from the eThekwini Transport Authority. The intersection is also among those under scrutiny from the ETA because of the number of traffic related accidents and incidents that occur there. Figures 4.7 and 4.8 show some sections of the intersection. Figure 4.7 shows a sectional view of the intersection, showing the Moses Mabidha stadium at a distance while figure 4.8 gives an aerial view of the intersection showing all four approaches.

Figure 4. 7: Photo showing sectional view of Site 003


Figure 4. 8: Photo showing aerial view site 003

4.2.3.1 Signalisation data

Traffic control on the site was dependent on the day of the week and time of the day. Fixed time control was used during the peak periods, both having a cycle time of 100 seconds and divided into six phases, an excerpt of the signal information is provided in the Appendix B.

4.2.3.2 Rainfall data

Rainfall data for this site was collected from rain gauge station located one kilometre away from the location of the intersection. During the study period, all the days that rainfall was recorded (excluding weekends) were noted, and the data was then extracted from the website. An example of the data format obtained for this site is presented in figure 4.9.

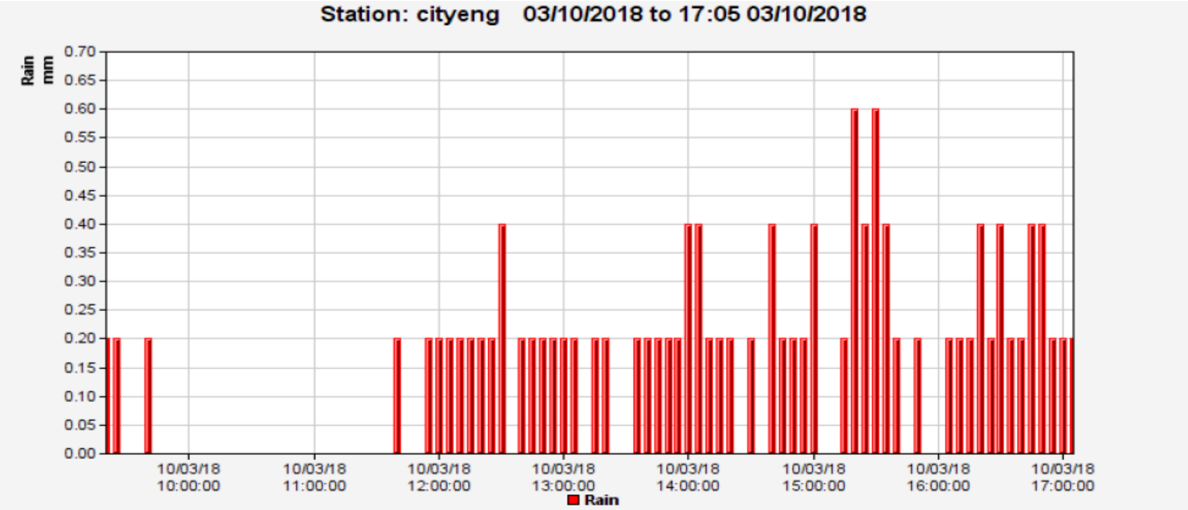

Figure 4. 9: Sample of rainfall data record for Site 003

4.2.3.3 Traffic data

A range of traffic data were collected and used in this study, they include, travel time, speed, sight distance, frequency of red-light running, traffic volume (traffic demand), vehicle composition, and saturation headway. The other data required was derived from these major ones. Table 4.17 gives dry weather data for site 003. The light, moderate and heavy rainfall intensity data are given in tables 4.18 to 4.20. The data in the tables includes travel time, speed, distance and whether the vehicle stopped (Y=0) or violated the red-light (Y=1).

Table 4. 17: Dry weather Travel time, Speed and Sight distance at site 003

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.38	14.81	40.60	0	2.59	19.29	57.17	1
3.50	14.29	38.78	0	1.81	27.70	93.76	1
3.75	13.33	35.56	0	2.47	20.24	60.96	1
3.88	12.90	34.13	0	2.37	21.09	64.43	1
4.00	12.50	32.81	0	2.59	19.29	57.17	1
3.62	13.79	37.10	0	2.59	19.29	57.17	1
4.75	10.53	26.59	0	2.75	18.18	52.89	1
3.00	16.67	47.22	0	2.07	24.11	77.27	1
4.50	11.11	28.40	0	2.22	22.50	70.31	1
3.75	13.33	35.56	0	2.44	20.45	61.83	1
4.12	12.12	31.59	0	2.44	20.45	61.83	1
3.13	16.00	44.80	0	2.52	19.85	59.41	1
3.75	13.33	35.56	0	1.75	28.57	97.96	1
5.00	10.00	25.00	0	2.37	21.09	64.43	1
3.88	12.90	34.13	0	2.59	19.29	57.17	1
4.25	11.76	30.45	0	2.22	22.50	70.31	1
3.00	16.67	47.22	0	2.37	21.09	64.43	1
3.62	13.79	37.10	0	2.67	18.75	55.08	1
2.75	18.18	52.89	0	2.22	22.50	70.31	1
3.50	14.29	38.78	0	2.00	25.00	81.25	1
3.75	13.33	35.56	0	2.53	19.75	59.01	1
3.25	15.38	42.60	0	2.44	20.45	61.83	1
3.75	13.33	35.56	0	4.63	10.81	27.47	0
4.63	10.81	27.47	0	4.13	12.12	31.59	0
2.88	17.39	49.91	0	3.87	12.90	34.13	0
3.25	15.38	42.60	0	3.00	16.67	47.22	0
3.75	13.33	35.56	0	4.12	12.12	31.59	0
4.13	12.12	31.59	0	3.50	14.29	38.78	0
3.75	13.33	35.56	0	3.25	15.38	42.60	0
4.00	12.50	32.81	0	4.75	10.53	26.59	0
3.50	14.29	38.78	0	3.50	14.29	38.78	0
4.87	10.26	25.77	0	4.38	11.43	29.39	0
3.62	13.79	37.10	0	3.88	12.90	34.13	0
3.50	14.29	38.78	0	3.37	14.81	40.60	0
6.00	8.33	20.14	0	4.38	11.43	29.39	0
2.25	22.22	69.14	0	3.50	14.29	38.78	0
3.25	15.38	42.60	0	4.12	12.12	31.59	0
3.75	13.33	35.56	0	3.13	16.00	44.80	0
3.88	12.90	34.13	0				

Table 4. 18: Light rainfall intensity Travel time, Speed and Sight distance at site 003

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
3.46	14.46	39.38	0	2.81	17.76	51.30	1
4.29	11.67	30.14	0	2.86	17.50	49.31	1
4.29	11.67	30.14	0	2.81	17.63	50.30	1
3.43	14.58	39.80	0	3.03	16.52	46.70	1
4.14	12.07	31.42	0	2.96	16.88	47.99	1
4.29	11.67	30.14	0	2.43	20.59	62.37	1
4.14	12.07	31.42	0	2.81	17.76	51.30	1
3.43	14.58	39.80	0	2.38	21.00	64.05	1
2.47	20.26	61.02	0	2.81	17.76	51.30	1
3.15	15.88	44.38	0	2.38	21.00	64.05	1
3.70	13.50	36.11	0	2.96	16.88	47.99	1
3.15	15.88	44.38	0	2.96	16.50	45.99	1
4.57	10.95	27.89	0	2.89	17.31	49.59	1
3.57	14.00	37.80	0	2.96	16.75	47.88	1
5.00	10.00	25.00	0	2.81	17.73	51.30	1
4.14	12.07	31.42	0	2.96	16.88	47.99	1
4.86	10.29	25.89	0	2.90	17.23	49.31	1
3.43	14.58	39.80	0	2.81	17.76	51.30	1
3.86	12.96	34.33	0	2.89	17.31	49.59	1
3.86	12.96	34.33	0	2.81	17.32	51.30	1
3.86	12.96	34.33	0				
3.71	13.46	35.98	0				
3.29	15.22	42.01	0				
3.48	14.36	39.04	0				
3.56	14.06	38.01	0				
4.86	10.29	25.89	0				

Table 4. 19: Moderate rainfall intensity Travel time, Speed and Sight distance at site 003

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
3.00	16.67	47.22	0	3.33	15.00	40.25	1
4.50	11.11	28.40	0	3.26	15.34	42.45	1
4.12	12.12	31.59	0	3.11	16.07	45.06	1
3.13	16.00	44.80	0	3.26	15.34	42.45	1
3.13	16.00	44.80	0	3.39	14.73	40.31	1
3.75	13.33	35.56	0	3.11	16.07	45.06	1
5.00	10.00	25.00	0	2.44	20.45	61.83	1
3.88	12.90	34.13	0	2.52	19.85	59.41	1
4.25	11.76	30.45	0	2.75	18.18	52.89	1
3.62	13.79	37.10	0	2.52	19.85	59.41	1
3.50	14.29	38.78	0	2.44	20.45	61.83	1
2.87	17.39	49.91	0	2.59	19.29	57.17	1
3.25	15.38	42.60	0	2.59	19.29	57.67	1
3.75	13.33	35.56	0	2.61	19.14	57.17	1
4.74	10.55	26.66	0	2.52	19.85	59.41	1
5.41	9.25	22.77	0	2.59	19.29	57.17	1
4.57	10.95	27.89	0	2.37	21.09	64.43	1
4.59	10.89	27.70	0	2.59	19.29	57.94	1
3.75	13.33	35.56	0	2.54	19.72	57.35	1
4.00	12.50	32.81	0	2.37	21.09	64.43	1
2.88	17.39	49.91	0	3.11	16.07	45.06	1
3.50	14.29	38.78	0	2.99	16.71	47.06	1
2.25	22.22	69.14	0	3.10	16.14	45.57	1
3.25	15.38	42.60	0	3.11	16.07	45.40	1
3.75	13.33	35.56	0	3.19	15.70	43.72	1
4.63	10.81	27.47	0	3.19	15.70	43.72	1
3.87	12.90	34.13	0	3.33	15.00	42.25	1
4.12	12.12	31.59	0	3.11	16.07	45.06	1
3.50	14.29	38.78	0	3.19	15.70	43.72	1
3.25	15.38	42.60	0	2.53	19.75	59.01	1
4.75	10.53	26.59	0	3.11	16.07	45.06	1
3.50	14.29	38.78	0	2.59	19.29	57.17	1
4.38	11.43	29.39	0	3.11	16.07	45.06	1
3.88	12.90	34.13	0	3.11	16.07	45.06	1
4.38	11.43	29.39	0	3.33	15.00	43.25	1
3.50	14.29	38.78	0	3.11	16.07	45.06	1

Table 4. 20: Heavy rainfall intensity Travel time, Speed and Sight distance at site 003

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.63	13.78	37.04	0	3.41	14.67	40.11	1
5.73	8.72	21.25	0	2.98	16.78	47.63	1
4.74	10.55	26.66	0	3.33	15.00	41.25	1
4.37	11.44	29.43	0	2.30	21.78	67.27	1
3.65	13.71	36.80	0	3.21	15.59	43.33	1
5.19	9.64	23.93	0	3.17	15.78	44.00	1
4.07	12.27	32.08	0	3.33	15.11	41.25	1
4.52	11.07	28.26	0	3.41	14.67	40.09	1
4.35	11.50	29.61	0	3.48	14.36	39.04	1
3.70	13.50	36.11	0	2.59	19.29	57.17	1
4.82	10.36	26.10	0	3.33	15.02	41.25	1
5.07	9.87	24.61	0	2.66	18.78	55.19	1
4.07	12.27	32.08	0	3.19	15.70	43.72	1
4.11	12.16	31.71	0	2.97	16.86	47.92	1
4.07	12.27	32.08	0	3.19	15.70	43.72	1
3.88	12.89	34.08	0	2.97	16.86	47.92	1
3.65	13.71	36.80	0	2.52	19.83	59.33	1
3.19	15.67	43.61	0	3.33	15.07	41.25	1
3.42	14.63	39.94	0	3.19	15.70	43.72	1
4.59	10.90	27.76	0	3.48	14.36	39.04	1
4.11	12.16	31.71	0	3.39	14.73	40.31	1
5.07	9.87	24.61	0	3.63	13.78	37.04	0
3.42	14.63	39.94	0	3.70	13.50	36.11	0
5.31	9.42	23.27	0	3.65	13.71	36.80	0
3.63	13.78	37.04	0	3.88	12.89	34.08	0
3.78	13.24	35.23	0	3.56	14.06	38.01	0

Table 4.21 gives volume and headway data for site 003. The recording of the traffic volume was done for one hour each, but the counting was done per five-minute interval. For the hour, 12 five-minute intervals were recorded. Saturation headway was recorded in terms of vehicle position as the vehicles crossed the intersection stop-line at the onset of green duration. The headway values were recorded for vehicle queues of above 10 vehicles and up to 14 vehicles.

Table 4. 21: Typical traffic volume and headway data for site 003

Period/ (vehicle position for headway)	**Site 003- Dry**	
	Volume (veh/5mins)	Headway (s)
1	139	2.58
2	143	2.17
3	162	1.97
4	162	1.91
5	164	1.65
6	164	1.58
7	155	1.53
8	159	1.52
9	169	1.49
10	165	1.48
11	152	1.48
12	166	1.49

Traffic composition data was considered important because different vehicle types have different operational capabilities, and the composition affects such things as saturation flow and other parameters all the way to capacity. In recording the different vehicle types, three classes of vehicles were considered: passenger cars (2 axle vehicles) denoted by the letter C, the light trucks, and vehicles with 3 axle loads as well as minibuses were classified as medium vehicles denoted by letter M and heavy truck and lorries with more than 3 axles loads as well as heavy buses were classified as heavy goods vehicles and denoted by letter H. The results for site 003 are presented in table 4.22. From the table, passenger cars made more than 90% of the total vehicle composition. Medium goods vehicles were second and the heavy goods vehicles constituted the lowest percentage. This was the case for all four study sites.

Table 4. 22: Traffic composition for site 003

Site 003 – Dry							
Date	Volume (veh/hr)	C		M		H	
		Number	Percentage (%)	Number	Percentage (%)	Number	Percentage (%)
1/10/2018	1976	1867	94.48	90	4.54	19	0.98
4/10/2018	1622	1529	94.25	70	4.34	23	1.41
8/10/2018	1850	1735	93.77	84	4.56	31	1.66
9/10/2018	1935	1850	95.59	71	3.68	14	0.73
10/10/2018	1841	1771	96.18	58	3.15	12	0.67
12/10/2018	2072	1970	95.12	79	3.82	22	1.06
15/10/2018	2011	1922	95.58	71	3.54	17	0.87
18/10/2018	1947	1851	95.08	83	4.25	13	0.68
19/10/2018	2046	1936	94.63	87	4.25	23	1.12
22/10/2018	2037	1953	95.86	69	3.41	15	0.73
23/10/2018	1831	1745	95.3	70	3.84	16	0.86
24/10/2018	2020	1933	95.73	71	3.53	15	0.74
25/10/2018	1645	1584	96.26	54	3.26	8	0.48
26/10/2018	1817	1728	95.07	78	4.31	11	0.63
29/10/2018	2076	1989	95.81	70	3.35	18	0.85

To determine the implication of rainfall on signalised intersection service quality, data was required to measure intersection performance under dry and rainy weather conditions. Table 4.23 gives traffic volume and discharge headway data for dry weather days and for the different rainfall intensities. The volume is given in 5-minute period intervals and discharge headway based on vehicle position. From the table it can be noted that on average the traffic volume reduced with increase in rainfall intensity whereas the discharge headway increased with increase in rainfall intensity.

Table 4. 23: Typical traffic volume and headway for varying rainfall conditions for Site 003

Site 003: Dry and rainy								
Period/ (Vehicle position for headway)	Dry		Light rain		Moderate rain		Heavy rain	
	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)
1	139	2.58	159	2.37	139	2.52	155	2.65
2	143	2.17	148	2.16	148	2.21	154	2.38
3	162	1.97	151	1.75	163	2.19	152	2.13
4	162	1.91	156	1.71	142	2	139	2.03
5	164	1.65	158	1.86	153	1.91	161	2.06
6	164	1.58	157	1.69	159	1.9	148	1.93
7	155	1.53	165	1.69	158	1.89	149	1.92
8	159	1.52	155	1.75	167	1.84	147	1.96
9	169	1.49	155	1.66	138	1.87	149	1.94
10	165	1.48	156	1.74	151	1.92	160	1.9
11	152	1.48	159	1.63	160	1.86	158	1.9
12	166	1.49	163	1.61	169	1.63	159	1.77
Total	1900 veh/hr		1882 veh/hr		1847 veh/hr		1831 veh/hr	

4.2.4 Empirical results for site 004

Site 004 is the intersection between Sandile Thusi road and Stalwart Simelane street. Figures 4.10 and 4.11 below shows some photographs of the site. Figure 4.10 shows a sectional view while figure 4.11 gives an aerial view of the intersection showing all four approaches.

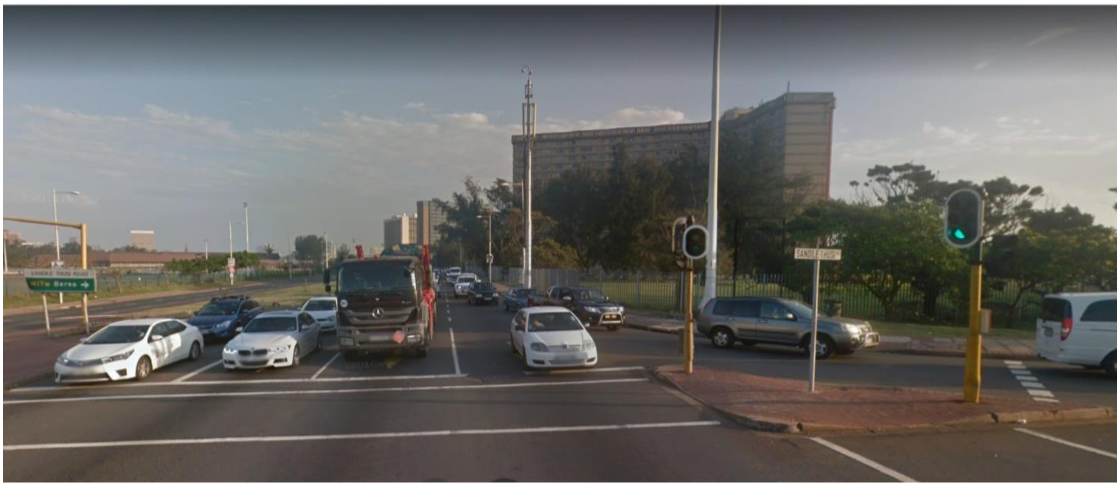

Figure 4. 10: Photo showing sectional view of Site 004

Figure 4. 11: Photo showing aerial view site 004

4.2.4.1 Signalisation data

Traffic control is dependent on the day of the week and time of the day. Fixed time control is used during the peak periods, having a cycle time of 100 seconds, and divided into six phases. An excerpt of the signal information is provided in the Appendix B.

4.2.4.2 Rainfall data

Rainfall data for this site was collected from rain gauge station cityeng, this station was located one kilometre from the location of the intersection. During the study period, all the days that rainfall was recorded (excluding weekends) were noted, and the data was then extracted from the website. An example of the data format obtained for this site is presented in figure 4.12 below.

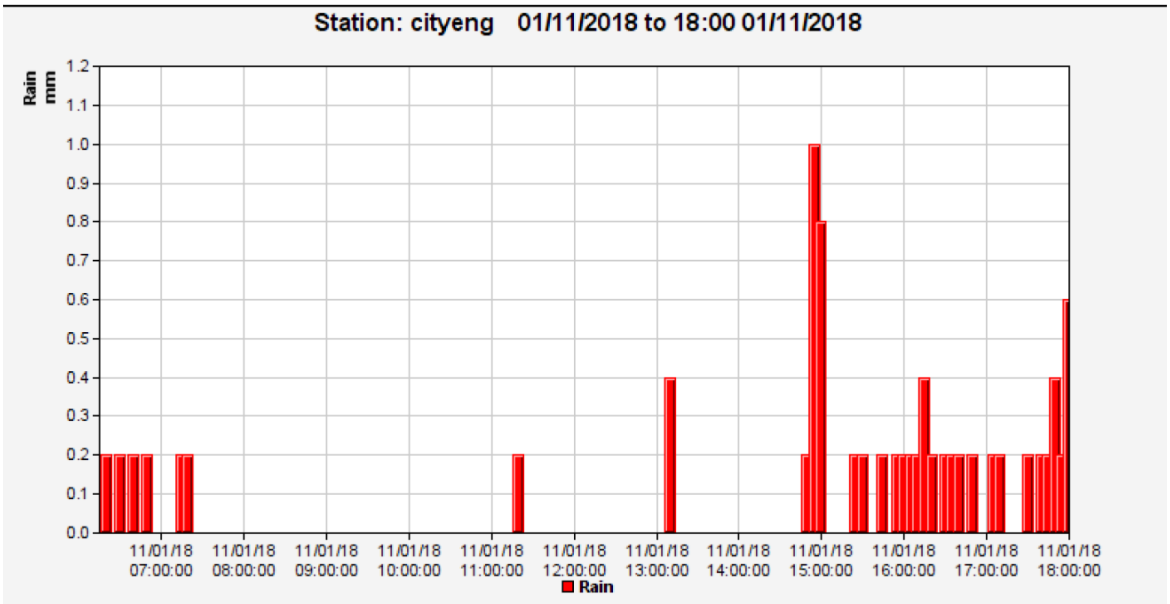

Figure 4. 12: Sample of rainfall data record for Site 004

4.2.4.3 Traffic data

A range of traffic data were collected and used in this study, they include, travel time, speed, sight distance, frequency of red-light running, traffic volume (traffic demand), vehicle composition, and saturation headway. The other data required was derived from these major ones. Table 4.24 gives dry weather data for site 003. The light, moderate and heavy rainfall intensity data are given in tables 4.25 to 4.27. The data in the tables includes travel time, speed, distance and whether the vehicle stopped (Y=0) or violated the red-light (Y=1).

Table 4. 24: Dry weather Travel time, Speed and Sight distance at site 004

Time(s)	Speed (m/s)	AD(m)	Probability of RLR	Time(s)	Speed (m/s)	AD(m)	Probability of RLR
3.19	15.67	43.61	0	2.07	24.11	77.27	1
3.31	15.11	41.62	0	3.23	15.48	42.93	1
3.19	15.67	43.61	0	2.77	18.06	52.41	1
4.50	11.11	28.40	0	1.98	25.25	82.39	1
4.75	10.53	26.59	0	2.77	18.06	52.41	1
3.31	15.53	41.62	0	2.92	17.11	48.84	1
5.00	10.00	25.00	0	2.62	19.12	56.51	1
3.50	14.29	38.78	0	2.06	24.24	77.87	1
3.50	14.29	38.78	0	2.00	25.00	81.25	1
4.00	12.50	32.81	0	2.15	23.28	73.64	1
3.19	15.67	43.61	0	1.78	28.12	95.80	1
3.23	15.48	42.93	0	2.22	22.48	70.31	1
3.19	15.67	43.61	0	2.29	21.83	67.50	1
3.08	16.25	45.70	0	1.83	27.27	91.74	1
3.38	14.77	40.46	0	2.46	20.31	61.25	1
4.31	11.61	29.95	0	2.14	23.31	73.80	1
3.31	15.12	41.66	0	2.46	20.31	61.25	1
3.78	13.21	35.16	0	1.97	25.41	83.09	1
3.31	15.11	41.62	0	2.22	22.50	70.31	1
4.25	11.76	30.45	0	2.07	24.11	77.27	1
4.25	11.76	30.45	0	2.22	22.41	70.50	1
3.25	15.38	42.60	0	1.83	27.27	91.74	1
3.25	15.38	42.60	0	2.44	21.45	62.83	1
5.75	8.70	21.17	0	2.06	24.24	77.87	1
4.25	11.76	30.45	0				
5.25	9.52	23.58	0				
3.50	14.29	38.78	0				
4.75	10.53	26.59	0				
3.50	14.29	38.78	0				
4.38	11.43	29.39	0				
3.88	12.90	34.13	0				
3.37	14.81	40.60	0				
4.38	11.43	29.39	0				
3.50	14.29	38.78	0				
4.12	12.12	31.59	0				
3.13	16.00	44.80	0				
3.88	12.90	34.13	0				

Table 4. 25: Light rainfall intensity Travel time, Speed and Sight distance at site 004

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.20	15.63	43.46	0	3.40	14.71	40.22	0
3.00	16.67	47.22	0	3.51	14.23	38.59	0
3.60	13.89	37.42	0	4.80	10.42	26.26	0
4.00	12.50	32.81	0	3.20	15.63	43.46	0
3.20	15.63	43.46	0	3.08	16.23	45.63	0
4.00	12.50	32.81	0	3.00	16.67	47.22	1
3.08	16.23	45.63	0	2.60	19.23	56.95	1
3.40	14.71	40.22	0	2.40	20.83	63.37	1
3.51	14.23	38.59	0	5.80	8.62	20.96	1
3.20	15.63	43.46	0	3.00	16.67	47.22	1
5.00	10.00	25.00	0	3.00	16.67	47.22	1
3.20	15.62	43.46	0	3.00	16.67	47.22	1
4.40	11.36	29.18	0	2.60	19.23	56.95	1
5.40	9.26	22.81	0	2.40	20.83	63.37	1
2.60	19.23	56.95	0	2.20	22.78	71.50	1
2.00	25.00	81.25	0	2.60	19.23	56.95	1
2.80	17.86	51.66	0	3.00	16.67	47.22	1
4.00	12.50	32.81	0	2.20	22.78	71.50	1
3.08	16.23	45.63	0	2.60	19.23	56.95	1
3.40	14.71	40.22	0	2.40	20.83	63.37	1
3.51	14.23	38.59	0	2.20	22.78	71.50	1
3.20	15.63	43.46	0	2.60	19.23	56.95	1
3.00	16.67	47.22	0	2.40	20.83	63.37	1
3.60	13.89	37.42	0	2.80	17.86	51.66	1
4.00	12.50	32.81	0	4.00	12.50	32.81	1
3.08	16.23	45.63	0	2.20	22.78	71.50	1
3.40	14.71	40.22	0	2.60	19.23	56.95	1
3.51	14.23	38.59	0	2.40	20.83	63.37	1
3.20	15.63	43.46	0				
4.00	12.50	32.81	0				
4.25	11.76	30.45	0				
3.25	15.38	42.60	0				
4.75	10.53	26.59	0				

Table 4. 26: Moderate rainfall intensity Travel time, Speed and Sight distance at site 004

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.20	15.63	43.46	0	2.20	22.38	71.05	1
3.00	16.67	47.22	0	2.60	19.23	56.95	1
3.60	13.89	37.42	0	2.40	20.83	63.37	1
4.00	12.50	32.81	0	2.60	19.23	56.95	1
2.40	20.83	63.37	0	2.20	22.78	71.50	1
2.60	19.23	56.95	0	2.60	19.23	56.95	1
2.80	17.86	51.66	0	2.40	20.33	62.37	1
4.00	12.50	32.81	0	2.20	22.67	71.35	1
3.08	16.23	45.63	0	2.60	19.23	56.95	1
3.40	14.71	40.22	0	3.17	15.78	44.00	1
3.51	14.23	38.59	0	2.20	22.67	71.50	1
3.20	15.63	43.46	0	2.60	19.23	56.95	1
3.00	16.67	47.22	0	2.40	20.83	63.37	1
3.60	13.89	37.42	0	3.39	14.73	40.31	1
4.00	12.50	32.81	0	2.60	19.23	56.95	1
2.40	20.83	63.37	0	2.40	20.83	63.37	1
2.60	19.23	56.95	0	2.20	22.57	71.50	1
2.00	25.00	81.25	0	2.60	19.23	56.95	1
2.80	17.86	51.66	0	2.40	20.83	63.37	1
4.00	12.50	32.81	0	2.20	22.18	71.50	1
3.08	16.23	45.63	0	2.40	20.83	63.37	1
3.40	14.71	40.22	0	4.00	12.50	32.81	0
3.51	14.23	38.59	0	4.25	11.76	30.45	0
3.20	15.63	43.46	0	3.25	15.38	42.60	0
3.00	16.67	47.22	0	4.75	10.53	26.59	0
3.60	13.89	37.42	0				
4.00	12.50	32.81	0				
3.08	16.23	45.63	0				
3.40	14.71	40.22	0				
3.51	14.23	38.59	0				
3.20	15.63	43.46	0				
3.08	16.23	45.63	0				
3.40	14.71	40.22	0				
3.51	14.23	38.59	0				
2.00	25.00	81.25	0				
3.20	15.63	43.46	0				

Table 4. 27: Heavy rainfall intensity Travel time, Speed and Sight distance at site 004

Time(s)	**Speed (m/s)**	**AD(m)**	**Probability of RLR**	**Time(s)**	**Speed (m/s)**	**AD(m)**	**Probability of RLR**
3.75	13.33	35.56	0	3.00	16.67	47.22	1
3.20	15.63	43.46	0	3.40	14.71	40.22	1
3.75	13.33	35.56	0	3.29	15.22	42.01	1
3.60	13.89	37.42	0	3.00	16.67	47.22	1
4.00	12.50	32.81	0	3.14	15.91	44.47	1
3.00	16.67	47.22	0	2.66	18.78	55.19	1
2.00	25.00	81.25	0	2.60	19.23	56.95	1
3.80	13.16	34.97	0	3.00	16.67	47.22	1
3.60	13.89	37.42	0	3.29	15.22	42.01	1
4.00	12.50	32.81	0	2.86	17.50	50.31	1
3.08	16.23	45.63	0	3.00	16.67	47.22	1
3.40	14.71	40.22	0	3.00	16.67	47.22	1
3.51	14.23	38.59	0	3.14	15.91	44.47	1
3.20	15.62	43.46	0	2.86	17.50	50.31	1
4.00	12.50	32.81	0				
3.29	15.22	42.01	0				
3.57	14.00	37.80	0				
3.43	14.58	39.80	0				
3.50	14.29	38.78	0				
4.43	11.29	28.95	0				
3.75	13.33	35.56	0				
4.13	12.12	31.59	0				
3.86	12.96	34.33	0				
3.86	12.96	34.33	0				
3.29	15.22	42.01	0				
3.20	15.63	43.46	0				
4	12.5	32.8125	0				
4.25	11.765	30.44983	0				

Data was also collected to be used in determining the impact of rainfall on signalised intersection service quality. Table 4.28 gives volume and headway data for site 004. The recording of the traffic volume was done for one hour each, but the counting was done per five-minute interval. For the hour, 12 five-minute intervals were recorded. Saturation headway was recorded in terms of vehicle position as the vehicles crossed the intersection stop-line at onset of green duration. The headway values were recorded for vehicle queues of above 10 vehicles and up to 14 vehicles.

Table 4. 28: Typical traffic volume and headway data for site 004

Period/ (vehicle position for headway)	**Site 004- Dry**	
	Volume	Headway
	(veh/5mins)	(s)
1	178	2.59
2	123	2.21
3	176	2.02
4	125	1.91
5	182	1.63
6	131	1.64
7	175	1.58
8	130	1.56
9	176	1.57
10	131	1.50
11	184	1.56
12	125	1.49

Traffic composition data was considered important because different vehicle types have different operational capabilities, and the composition affects such things as saturation flow and other parameters all the way to capacity. In recording the different vehicle types, three classes of vehicles were considered: passenger cars (2 axle vehicles) denoted by the letter C, the light trucks, and vehicles with 3 axle loads as well as minibuses were classified as medium vehicles denoted by letter M and heavy truck and lorries with more than 3 axles loads as well as heavy buses were classified as heavy goods vehicles and denoted by letter H. The results for site 004 are presented in table 4.29. From the table, passenger cars made more than 90% of total vehicle composition. The high passenger car percentage can be attributed to this being peak period as well as the location being close to the central business district.

Table 4. 29: Traffic composition for site 004

Site 004 - Dry							
Date	Volume (veh/hr)	C		M		H	
		Number	Percentage (%)	Number	Percentage (%)	Number	Percentage (%)
2018/01/10	1799	1638	91.05	117	6.50	46	2.56
2018/02/10	1882	1733	92.08	99	5.26	50	2.66
2018/04/10	1850	1705	92.16	100	5.41	45	2.43
2018/05/10	1720	1561	90.76	110	6.40	48	2.79
2018/08/10	1714	1548	90.32	122	7.12	44	2.57
2018/10/10	1876	1725	91.95	98	5.22	53	2.83
2018/11/10	1960	1793	91.48	113	5.77	54	2.76
2018/12/10	1915	1765	92.17	150	7.83	40	2.09
15/10/2018	1831	1691	92.35	140	7.65	43	2.35
18/10/2018	1605	1472	91.71	133	8.29	39	2.43
19/10/2018	1898	1753	92.36	145	7.64	48	2.53
22/10/2018	1904	1760	92.44	143	7.51	45	2.36
23/10/2018	1798	1653	91.94	105	5.84	40	2.22
24/10/2018	1873	1729	92.31	104	5.55	41	2.19
25/10/2018	1721	1576	91.57	95	5.52	46	2.67
26/10/2018	1796	1652	91.98	96	5.35	50	2.78

This study was based determining the impact rainfall has on the performance of signalised intersections. Data was therefore collected during the dry and rainfall conditions. Table 4.30 gives traffic volume and discharge headway data for dry weather days and for the different rainfall intensities. The volume is given in 5-minute period intervals and discharge headway based on vehicle position. From the table it can be noted that on average the traffic volume reduces with increase in rainfall intensity whereas the discharge headway increases with increase in rainfall intensity.

Table 4. 30: Typical traffic volume and headway for varying rainfall conditions for Site 004

Site 004: Dry and rainy								
Period/ (Vehicle position for h)	Dry		Light rain		Moderate rain		Heavy rain	
	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)	Volume (veh)	Headway (s)
1	178	2.59	145	2.46	132	2.81	120	2.9
2	123	2.21	141	2.07	150	2.2	135	2.33
3	176	2.02	148	2.14	140	2.26	120	2.38
4	125	1.91	136	1.85	164	2.04	130	2.05
5	182	1.63	146	1.65	160	1.94	150	1.93
6	131	1.64	145	1.7	150	1.81	120	1.93
7	175	1.58	147	1.63	119	1.87	140	1.93
8	130	1.56	144	1.6	138	1.66	130	1.74
9	176	1.57	142	1.61	119	1.76	150	1.78
10	131	1.5	144	1.58	130	1.69	150	1.78
11	184	1.56	149	1.54	117	1.69	125	1.74
12	125	1.49	150	1.57	118	1.83	150	1.8
Total	*1836 veh/hr*		*1739 veh/hr*		*1637 veh/hr*		*1620 veh/hr*	

4.3 Summary

The empirical results from surveyed sites have now been presented showing travel time, speed, distance, red-light running, volume, and vehicle composition under different weather conditions. The frequency of red-light running was quite low, below 2% of traffic volume. For vehicles involved in the red-light violation, they were moving above the posted speed limit on average and those that stopped were moving well below the speed limit. The highest recorded volume of vehicles under dry weather conditions was at site 001with 1963 vehicles during the one-hour duration count while site 002 recorded the lowest at 1693 vehicles. The highest recorded difference in the volume of 216 vehicles was at site 004 with 1836 vehicles during dry weather and 1620 vehicles during heavy rainfall conditions, which is a 12% reduction. The lowest recorded difference in volumes of 17 vehicles was at site 002 with 1693 vehicles during dry weather and 1676 vehicles during light rainfall, a 1% reduction. For saturation headway, on average site 003 recorded the highest under dry weather conditions at 1.77 seconds, while site 002 recorded the lowest at 1.67 seconds. Passenger vehicles accounted for more than 90% of total vehicles for all four study sites. Sites 001 and 002 had a higher percentage of heavy goods vehicles at 4.05%, while site 003 had 0.9% and site 004 2.51%. Site 004 recorded the lowest passenger car proportion at 91.79% and the highest medium goods vehicles at 6.43%. Rainfall data was also presented from the rain gauge stations that were within a given distance from the study sites. The number of rainfall days varied from site to site, but care was taken to ensure that a minimum of five days was considered under each case and rainfall intensity. The rainfall intensity was classified according to the rain intensity as the light rain (L) with intensity (i) $<$ 2.5mm/h. moderate rain (M) with intensity $>$ 2.5mm/h but $\leq$10mm/h, heavy rain (H) with intensity $>$ 10mm/mm/h but $\leq$ 50mm/h. The empirical data presented here is used in the next chapter (chapter 6) to develop a new quality of service criteria. The new criteria and the rest of the data were then used to evaluate the impact of rainfall on signalised intersection quality of service.

CHAPTER 5

IMPACT OF RAINFALL ON RED-LIGHT RUNNING

5.1 Overview

Chapter 4 presented the empirical data that was used in this study and key among them was travel time, speed, distance, and whether a vehicle stopped or violated a red light. Those three were the main parameters used in determining the probability or red-light violations at signalised intersections. As rainfall was a key determinant for this study, data were presented for both dry and rainy weather conditions. This chapter utilizes that data to model the probability of red-light running and what influence rainfall has on that.

As the lights turn from green to yellow, drivers are aware that it means that their right of way is coming to an end and as such a driver has to choose whether to stop or attempt to cross the stop-line before the red lights come on. The choice of driver action determines whether the driver accelerates, maintains their speed, or decelerates. Those who choose to cross the stop-line before red, will either accelerate or maintain their speed if high while those who choose to stop will in most cases slow down to achieve that. All the parameters mentioned; speed, acceleration, and deceleration are affected by rainfall. This chapter investigates the impact rainfall has on these parameters and what implication this has on yellow and red-light running.

All study sites had a constant yellow time of 3 seconds and a posted speed limit of 16.67m/s, which approximately equals to 50m as the distance covered during the yellow duration (yellow distance). Measurement for speed was thus done at the 50m mark. Time was also considered an important parameter, the time it took to cover the 5om distance was referred to as the travel time, the time from the onset of the yellow time that a vehicle passed the designated point was noted. The impact of rainfall on this time aspects and what implication this had on yellow/red-light running was also investigated in this chapter.

The rest of the chapter is structured as follows: Section 5.2 deals with the definition of red-light running as laid out in the SARTSM. Section 5.3 discusses the impact of rainfall on red-light running, where each site is evaluated separately. Section 5.4 looks at rainfall impact on driver behaviour during yellow and all-red intervals especially on those parameters that contribute to RLR. The chapter is summarised in section 5.4.

5.2 Red-light running as per the South Africa Road traffic signs manual (SARTSM)

The(SARTSM, 2012a) gives a formula for the determination of yellow time for dry conditions. For the wet weather conditions, the manual provides a formula to calculate the all-red time with the knowledge that the extra all-red acts as an extension of the yellow time. The two formulae are as shown in equations 5.1 and 6.2 below.

$$\boldsymbol{Y = t_y + 1/2.\frac{V/_{3\cdot 6}}{A_y + g.^{G}/_{100}}} \qquad 5.1$$

Where: Y is the yellow time, t_y is the reaction time (0.75 s), A_y is the deceleration rate ($3.7m/s^2$), V is the speed limit, g is the acceleration due to gravity ($9.8m/s^2$) and G is the gradient on approach to signal (%). *** The value calculated above is subject to a minimum of 3s for speed limit of 60km/h or less*.

For the all-red interval, the manual provides a table that gives the value for dry weather conditions, the value is determined from the speed limit, the gradient of approach, and the provided yellow interval. In cases of wet weather conditions, equation 5.2 below is used to calculate the all-red time.

$$\boldsymbol{All - red = t_r + \frac{1}{2}\frac{V/_{3.6}}{A_r + g.^{G}/_{100}} + \frac{W}{V/3.6} - Y} \qquad 5.2$$

Where: Y is the yellow time, t_r is the reaction time (1.0 s), A_r is the deceleration rate ($3.0m/s^2$), V is the speed limit, g is the acceleration due to gravity ($9.8m/s^2$), G is the gradient on approach to signal (%) and W is the clearance width (m)

The manual recommends that for purposes of law enforcement, red-light violation should only commence during the last second of the all-red time. Secondly, the extended all-red is considered an extension of the yellow time. Considering all these and the fact that the sites studied had all-red clearances above 3 seconds, vehicles entering the intersection during the first 2 seconds of the all-red were not considered to have legally run the red-light and therefore for this study that action was named Yellow Light Overrun(YLO); entering the intersection during the first 2 seconds of the all-red time. The red-light running violation was considered only for vehicles that entered the intersection during the last second of the all-red duration.

5.3 Impact of rainfall on the probability of Red-light Running

This section details the influence of rainfall on the probability of red-light running given driver approach speed from a marked point on the intersection approach, travel time, and sight distance. On all study sites, three-speed regimes (free flow, transition, and posted speed limit) were in operation as shown below in figure 5.1, each site was divided into three zones (free flow-FVZ, transition-TVZ, and posted speed-PVZ), and posted speed zone sub-divided into dilemma-DS and braking-BS section. The free flow speed represented the average speed at the free flow zone, and the desired drivers' speed in the absence of traffic control devices. At the transition speed zone, drivers may reduce or increase speed in this zone at the onset of yellow light. The posted speed limit is influenced by drivers' decision and road safety measures in place (often speed cameras). Free flow speeds were collected during the green light phase and used to determine sight distance. The approach speeds were collected at the yellow time interval and used to estimate travel time. Note travel time equals free flow time where the lead vehicle speed in traffic flow is unimpeded, and traffic flow was subjected to an inconsequential degree of saturation. The generalised 85th percentile free-flow speed was 80km/h (22.24m/s), assuming an average reaction time of 2s; the ensuing FVZ sight distance was approximately 112m from the stop line. In contrast, the 50m distance from PVZ to the stop line was a function of posted speed (60km/h) multiply by yellow time(3s) interval. The dilemma section within PVZ was then marked at 20m, and the braking section at 30m. This demarcation was done for all sites and speed measurements were done under dry and varying rainfall intensities.

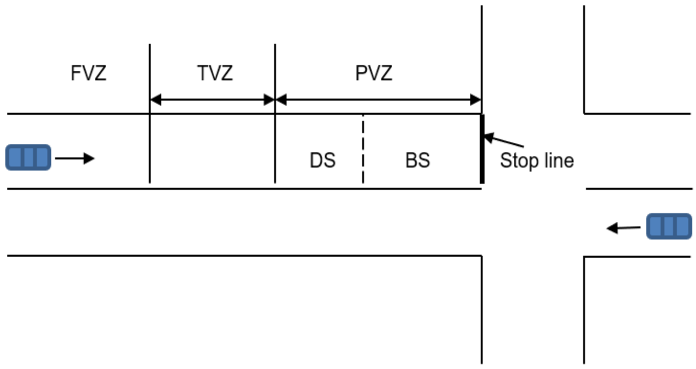

Figure 5. 1: Typical speed zone demarcations

Three main parameters were derived from this and used in regression modelling. Travel time, approach speed, and sight distance. Approach speed was measured at the 50m distance during yellow interval, travel time was then determined from this speed and the distance to the intersection. Speed of approach during the green interval was then used to calculate the sight distance. To determine the probability of driver stopping or RLR, a binary logistic regression model was used where the regression model took the form given in equation 5.3.

$$Logit\ (P) = ln\ (P/1 - P) \quad 5.3$$

Which can also be rewritten as:

$$L_p = ln\left(\frac{P}{1-P}\right) = \beta_o + \beta_1\tau + \beta_2 v + \beta_3 d \text{ for } 0 < \text{P} < 1 \quad 5.4$$

From which the probability of a driver's choice during the yellow interval can be calculated by:

The probability of RLR $$P(Y = 1) = \frac{e^{\beta_o+\beta_1\tau+\beta_2 v+\beta_3 d_3}}{1+ e^{\beta_o+\beta_1\tau+\beta_2 v+\beta_3 d_3}} \quad 5.5$$

The probability of stopping $$P(Y = 0) = \frac{1}{1+ e^{\beta_o+\beta_1\tau+\beta_2 v+\beta_3 d_3}} \quad 5.6$$

Where:

P represents the probability of a driver's choice, d is stopping distance

β_0 represents the constant of the model and β_1, β_2, β_3 are model coefficients

τ is travel time to the stop line, $t_f\left(1 + \partial\left[\frac{T_v}{Q}\right]^{\mu}\right)$, T_v denotes traffic volume

Q is signalised intersection capacity

v is the vehicle's approach speed at the onset of a yellow time interval

The free-flow speed (v), travel time to stop line (t), and sight distance (d) were the independent parameters, and driver's choice behaviour was the dependent variable. Free-flow speed samples were collected during the green phase, and sight distance was measured from the free flow speed zone to the stop line. Travel time to the stop line was measured as the time taken to cover 50m length to the stop line (the distance covered in 3 seconds of yellow time at the posted speed of 16.67m/s). At the onset of yellow light, the dependent variable was represented by y, where y = 0 means the driver brings the vehicle to a halt at the stop-line and y = 1 denotes that the driver proceeded beyond the stop-line into the intersection at the onset of red light. The results of the regression analysis are discussed next per site and for dry and rainfall conditions.

5.3.1 Site 001

Analysis was done under dry weather conditions, and this was then replicated for the different rainfall intensities. Analysis of speed showed that under dry conditions, considering the approach speed at the dilemma zone (50m from intersection stop-line), under RLR the average speed was 21.74m/s (78km/h). For the vehicles that stopped safely the average speed was 13m/s (47km/h). This shows that drivers that complied with the posted speed limit of 60km/h during the yellow interval were likely to stop at the stop line in time. Whereas drivers with speed over the legal limit were more likely to violate the red light.

Given travel time(s), speed (m/s), and sight distance (m) regression models were fitted for this site under dry and rainfall weather conditions. The probability of RLR and stopping was estimated in each case and the results for site 001 dry weather conditions are summarised in table 5.1. From the table, the *t'* values all have an absolute value greater than 2.5; therefore, all the variables used in the regression equation were useful in predicting red-light running violation. The R^2 value (0.91) is greater than 0.5, and the F value is greater than 4 suggesting the model equation did not happen by chance. The model equation for dry weather therefore becomes; $y = -18.292 + 1.829t + 1.897v - 0.390d$

Table 5. 1: Site 001 dry weather conditions summary results

Site 001 - Dry weather conditions						
	d	v	t	Constant	Stats Test	
t' values	-6.06776	6.385393	5.762879	-6.34413	t > 2.5	ok
Coefficients	*-0.38963*	*1.896641*	*1.828983*	*-18.2915*		
Std. Error	0.064214	0.297028	0.317373	2.883224		
R^2	0.909696	0.150005	#N/A	#N/A	R2 > 0.5	ok
F df	124.2424	37	#N/A	#N/A	F > 4	ok
Residuals	8.386953	0.832559	#N/A	#N/A		

Assuming a travel time of 3s, based on the posted limit of 16.67m/s and distance of 50m and substituting in the model equation above, and applying equations 5.5 and 5.6 gives the probability of RLR under dry weather conditions as 33.9% and that of stopping as 66.1%. Assuming that the speed limit rule is rigorously enforced at the study and the distance to stop line clearly marked, what is the best yellow time and distance to stop line combination it may be queried. To estimate the best combination, the distance values 60m, 70m, and the yellow time values 2s, 3s, and 4s were used iteratively with results shown below in table 5.2. These results show that increasing yellow light time from 3s to 4s increased red-light running whilst decreasing yellow light time from 3s to 2s decreased red-light running under dry weather conditions. For 60m and 70m distances, in all cases, the probability of RLR was zero and this further shows that enforcing a speed limit, has the potential to reduce unintentional red-light violations, that is red-light running for drivers that are trapped in the dilemma zone.

Table 5. 2: Dry Weather Sensitivity Analysis for site 001

D (m)	*T* (s)	RLR (y =1)	RLR (y = 0)
50	2s	7.60%	92.40%
	3s	33.90%	66.10%
	4s	76.10%	23.90%
60	2s	0.02%	99.98%
	3s	0.01%	99.99%
	4s	0.06%	99.94%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

For light rainfall conditions, analysis of speed showed a reduction compared to dry weather conditions. For vehicles involved in RLR the average speed was 17.21m/s (62km/h), this is slightly above the posted speed limit but lower than the one recorded for dry weather conditions. For vehicles that stopped the average speed was 12.24m/s (44km/h) which is again lower than the one recorded for dry weather conditions showing that on average light rainfall causes a reduction in average speed. Using the data to estimate a binary logistic regression equation for light rainfall intensity yielded the results posted in table 5.3.

Table 5. 3: Site 001 Light Rainfall intensity summary results

Site 001 - Light Rainfall (LR) intensity						
	d	v	t	Constant	Stats Test	
t' values	-2.86372	2.883241	2.513122	-2.73677	t > 2.5	ok
Coefficients	-0.99406	4.342704	3.01225	-33.5911		
Std. Error	0.347121	1.506188	1.198609	12.27399		
R^2	0.630387	0.31529	#N/A	#N/A	$R^2 > 0.5$	ok
F df	21.03494	37	#N/A	#N/A	F > 4	ok
Residuals	6.273123	3.678096	#N/A	#N/A		

The equation for light rainfall intensity becomes y = -33.591 + 3.012t + 4.343v - 0.994d. The statistical tests for R^2, t, and F both gave satisfactory results as can be seen in table 5.3. Assuming travel time of 3s, a distance of 50m, and the posted speed of 60km/h for this case and substituting the model equation values showed the probability of RLR as 13.4% and that of stopping as 86.6%. Comparing this with the dry weather conditions shows that light rainfall causes the probability of RLR to reduce by up to 65%.

The results for sensitivity analysis considering distances 50 to 70 and 2 to 4s time are presented in Table 5.4

Table 5. 4: Light Rainfall Sensitivity Analysis for site 001

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.76%	99.24%
	3s	13.40%	86.60%
	4s	75.90%	24.10%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

The results similar to those of dry weather conditions show that reducing the time from 3 to 2 reduces the probability of RLR while increasing it also increases the RLR probability. For the 60m and 70m distances in all instances, the probability or RLR was because of speed reduction.

Table 5. 5: Site 001 Moderate Rainfall intensity summary results

Site 001 - Moderate Rainfall (MR) intensity						
	d	v	t	Constant	Stats Test	
t' values	-2.45357	2.638055	3.108888	-3.00696	t > 2.5	ok
Coefficients	-1.15737	5.202717	4.218201	-44.0325		
Std. Error	0.471708	1.972179	1.35682	14.64352		
R^2	0.842206	0.202712	#N/A	#N/A	$R^2 > 0.5$	ok
F df	64.04848	36	#N/A	#N/A	F > 4	ok
Residuals	7.895681	1.479319	#N/A	#N/A		

Under moderate rainfall, the results for the binary logistic regression equation are presented in table 5.5. All the statistical tests done were satisfactory and the equation was y = -44.033 + 4.218*t* + 5.203*v* - 1.157*d*. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under moderate rainfall intensity of 7.5% and that of stopping as 92.5% which is a further reduction on the results recorded for light rainfall as well dry weather conditions. The results for sensitivity analysis are presented in table 5.6. Like the results for dry and light rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60mand 70m were all zero for the probability of RLR.

Table 5. 6: Moderate Rainfall Sensitivity Analysis for site 001

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.10%	99.90%
	3s	7.50%	92.50%
	4s	84.60%	15.40%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

For heavy rainfall intensity, the results for the binary logistic regression equation are presented in table 5.7. All the statistical tests done were satisfactory and the resultant equation; y = -74.655 + 6.848t + 9.727v -2.267d. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under heavy rainfall intensity of 0.5% and that of stopping as 99.5% which is a further reduction on the results recorded for dry, light, and moderate rainfall. The results for sensitivity analysis are presented in table 5.8. Like the results for dry, light, and moderate rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60mand 70m were all zero for the probability of RLR.

Table 5. 7: Site 001 Heavy Rainfall intensity summary results

Site 001 - Heavy rainfall (HR) intensity						
	d	v	t	Constant	Stats Test	
t' values	-2.7367	2.821315	2.935161	-2.946956	t > 2.5	ok
Coefficients	-2.2673	9.726766	6.848214	-74.65527		
Std. Error	0.828479	3.4476	2.333165	25.33301		
R^2	0.643223	0.304812	#N/A	#N/A	$R^2 > 0.5$	ok
F df	21.63448	36	#N/A	#N/A	F > 4	ok
Residuals	6.030218	3.344782	#N/A	#N/A		

Table 5. 8: Heavy Rainfall Sensitivity Analysis for site 001

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.00%	100.00%
	3s	0.50%	99.50%
	4s	82.00%	18.00%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

A summary of all the regression equations for site 001 are presented in Table 5.9. from the table light rainfall caused a reduction in the probability of RLR of 60%, moderate 78%, and heavy 98.5%. It can therefore be concluded that rainfall reduces the probability of RLR, and the magnitude of the reduction is dependent on the rainfall intensity.

Table 5. 9: Summary of Probability RLR for site 001 all weather conditions

Weather	**Model Equations**	**R^2**	**RLR, Y = 1**	**RLR, Y= 0**
Dry	y = -18.29 + 1.829t + 1.8975v - 0.390d	0.91	33.90%	66.10%
LR	y = -33.591 + 3.012t + 4.343v - 0.994d	0.63	13.40%	86.6%
MR	y = -18.29 + 1.829t + 1.8975v - 0.390d	0.84	7.50%	92.50%
HR	y = -74.655 + 6.848t + 9.727v -2.267d	0.64	0.50%	99.50%

The differences for site 001 under the different weather conditions are further illustrated in figure 5.2 below. This is based on an assumption of 16.67m/s as the speed, distance of 50m, and travel time of 3s and shows that traveling below the posted speed limit will reduce chances of RLR under all conditions.

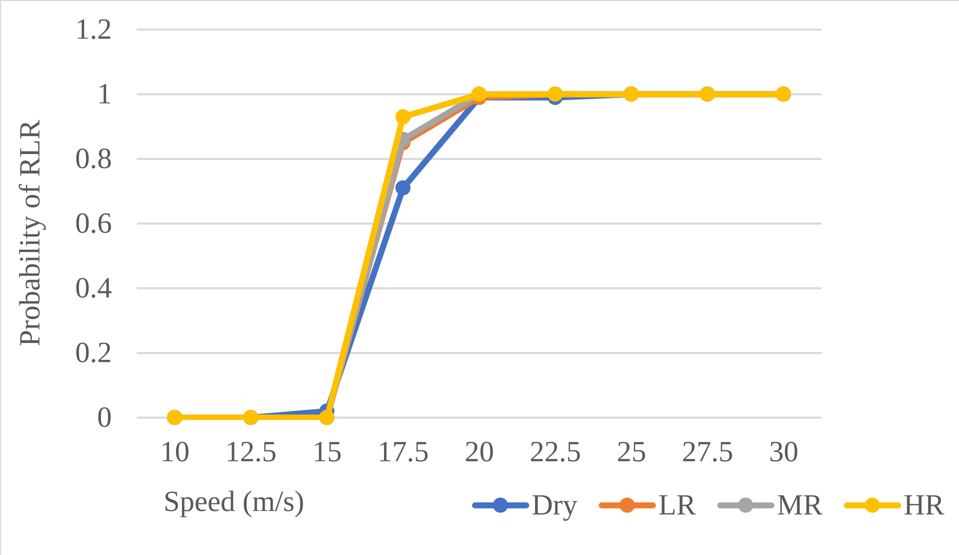

Figure 5. 2: Probability of RLR for Site 001 under different weather conditions

5.3.2 Site 002

Analysis of speed showed that under dry conditions, considering the approach speed at the dilemma zone (50m from intersection stop-line), under RLR the average speed was 19.80/s (71km/h). For the vehicles that stopped safely the average speed was 12.2m/s (44km/h). This shows that drivers that complied with the posted speed limit of 60km/h (16.67m/s) were likely to stop at the traffic light line. Whereas drivers with speed over the legal limit were more likely to violate the red lights at the signalised intersection.

Given travel time(s), speed (m/s), and sight distance (m) regression models were fitted for this site under dry and rainfall weather conditions. The probability of RLR and stopping was estimated in each case and the results for site 002 dry weather conditions are summarised in table 5.10. From the table, the *t'* values all have an absolute value greater than 2.5; therefore, all the variables used in the regression equation were useful in predicting red-light running violation. The R^2 value (0.77) is greater than 0.5, and the F value is greater than 4 suggesting the model equation did not happen by chance. The model equation for dry weather therefore is $y = -11.068 + 0.947t + 1.3408v - 0.287d$

Table 5. 10: Site 002 dry weather conditions summary results

Site 002- Dry weather conditions						
	d	v	t	Constant	Stats Test	
t' values	-3.56678	3.821195	3.256769	-3.767406	t > 2.5	ok
Coefficients	-0.28686	1.340812	0.947036	-11.06804		
Std. Error	0.080426	0.350888	0.29079	2.93784		
R^2	0.765124	0.248861	#N/A	#N/A	$R^2 > 0.5$	ok
F df	40.17664	37	#N/A	#N/A	F > 4	ok
Residuals	7.464624	2.291473	#N/A	#N/A		

Assuming a travel time of 3s, based on the posted limit of 16.67m/s and distance of 50m and substituting in the model equation above, and applying equations 5.5 and 5.6 gives the probability of RLR under dry conditions as 44.5% and that of stopping as 55.5%. Assuming that the speed limit rule is rigorously enforced at the study site and the distance to stop line clearly marked, what is the best yellow time and distance to stop line combination. To estimate the best combination, the distance values 60m, 70m, and the yellow time values 2s, 3s, and 4s were used iteratively with results shown below in table 5.11. These results show that increasing yellow light time from 3s to 4s increases red-light running whilst decreasing yellow light time from 3s to 2s decreases red-light running under dry weather conditions. For 60m distance, the probability of RLR is below 50% but is more than zero. It is highest when the time is increased to 4 seconds. For 70m distance, in all cases, the probability of RLR was less than 1%. This shows that with an enforced speed limit and a properly marked speed boundary (marked at 60-70m for these study sites) it may be possible to reduce red-light violations and especially the unintentional ones caused by drivers caught in the dilemma zone.

Table 5. 11: Dry Weather Sensitivity Analysis for site 002

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	23.80%	76.20%
	3s	44.50%	55.50%
	4s	67.40%	32.60%
60	2s	1.70%	98.30%
	3s	4.40%	95.60%
	4s	10.50%	89.50%
70	2s	0.10%	99.90%
	3s	0.30%	99.70%
	4s	0.70%	99.30%

For light rainfall conditions, analysis of speed showed a reduction compared to dry weather conditions. For vehicles involved in RLR the average speed was 17.17m/s (62km/h), this is slightly above the posted speed limit but lower than the one recorded for dry weather conditions showing that on average light rainfall caused a reduction in average speed for RLR. For vehicles that stopped the average speed was 12.59m/s (45km/h) which is similar to the one recorded for dry weather conditions. Using the data to estimate a binary logistic regression equation for light rainfall intensity yielded the results posted in table 5.12.

Table 5. 12: Site 001 Light Rainfall intensity summary results

Site 002- Light Rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-11.1206	11.06734	8.8654	-10.034087	t > 3	ok
Coefficients	-0.57322	2.486835	1.788982	-19.521543		
Std. Error	0.051546	0.2247	0.201794	1.9455226		
R^2	0.797445	0.15503	#N/A	#N/A	$R^2 > 0.5$	ok
F df	48.55555	37	#N/A	#N/A	F > 4	ok
Residuals	3.500979	0.889265	#N/A	#N/A		

The equation for light rainfall intensity; $y = -19.522 + 1.789t + 2.487v - 0.573d$. The statistical tests for R^2, t, and F both gave satisfactory results as can be seen in table 51.2. Assuming travel time of 3s, a distance of 50m, and the posted speed of 60km/h for this case and substituting the model equation values showed the probability of RLR as 20.40% and that of stopping as 79.60%. Comparing this with the dry weather conditions shows that light rainfall causes the probability of RLR to reduce by up to 54%. The results for sensitivity analysis considering distances 50 to 70 and 2 to 4s time are presented in Table 5.13 .

The results in table 51.3 like those of dry weather conditions show that reducing time from 3 to 2 reduces the probability of RLR while increasing it also increases the RLR probability. For the 60m and 70m distances in all instances the probability or RLR was below 1% and this can be attributed to the reduced speeds and the assumption that the speed limit is enforceable and as such from these distances' chances of RLR are very minimal.

Table 5. 13: Light Rainfall Sensitivity Analysis for site 002

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	4.10%	95.90%
	3s	20.40%	79.60%
	4s	60.60%	39.40%
60	2s	0.01%	99.99%
	3s	0.08%	99.92%
	4s	0.50%	99.50%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

Under moderate rainfall, the results for the binary logistic regression equation are presented in table 5.14. The R^2 value was 0.77 which is above 0.5, all the other statistical tests done were satisfactory and the resultant binary equation; y = -45.266 + 4.093*t* + 5.813*v* -1.334*d*. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under moderate rainfall intensity of 5.80% and that of stopping as 94.20% which is a further reduction on the results recorded for light rainfall as well dry weather conditions. The results for sensitivity analysis are presented in table 5.15. Like the results for dry and light rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60mand 70m were all zero for the probability of RLR.

Table 5. 14: Site 002 Moderate Rainfall intensity summary results

Site 002- Moderate Rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-4.64922	4.703339	4.239188	-4.550869872	t > 2.5	ok
Coefficients	-1.33415	5.813151	4.093373	-45.26633928		
Std.Error	0.286963	1.235963	0.965603	9.946744373		
R^2	0.774465	0.248093	#N/A	#N/A	$R^2 > 0.5$	ok
F df	42.35154	37	#N/A	#N/A	F > 4	ok
Residuals	7.820211	2.27735	#N/A	#N/A		

Table 5. 15: Moderate Rainfall Sensitivity Analysis for site 002

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.10%	99.90%
	3s	5.80%	92.50%
	4s	78.70%	15.40%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

For heavy rainfall intensity, the results for the binary logistic regression equation are presented in table 5.16. All the statistical tests done were satisfactory, and the equation; y = 55.242+ 4.953t + 7.299v - 1.702d. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under heavy rainfall intensity of 2.2% and that of stopping as 97.80% which is a further reduction on the results recorded for dry, light, and moderate rainfall. The results for sensitivity analysis are presented in table 5.17. Like the results for dry, light, and moderate rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60m and 70m were all zero for the probability of RLR.

Table 5. 16: Site 002 Heavy Rainfall intensity summary results

Site 002- Heavy Rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-2.81038	2.95681	3.2516254	-3.226338153	t > 2.5	ok
Coefficients	-1.70196	7.299221	4.9524984	-55.24178802		
Std. Error	0.6056	2.468614	1.5230839	17.12213209		
R^2	0.810808	0.225574	#N/A	#N/A	$R^2 > 0.5$	ok
F df	52.85627	37	#N/A	#N/A	F > 4	ok
Residuals	8.068531	1.882688	#N/A	#N/A		

Table 5. 17: Heavy Rainfall Sensitivity Analysis for site 002

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.02%	99.98%
	3s	2.20%	97.80%
	4s	75.90%	24.10%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

A summary of all the regression equations for site 002 is presented in table 5.18. from the table, light rainfall caused a reduction in the probability of RLR of 54%, moderate 87%, and heavy 95%. It can therefore be concluded that rainfall reduces the probability of RLR, and the magnitude of the reduction is dependent on the rainfall intensity.

Table 5. 18: Summary of Probability RLR for site 002 all weather conditions

Weather	**Model Equations**	**R^2**	**RLR, y = 1**	**RLR, y = 0**
Dry	y = -11.068+ 0.947t + 1.3415v - 0.287d	0.77	44.50%	55.50%
LR	y = -19.522 + 1.789t + 2.487v - 0.573d	0.80	20.40%	79.60%
MR	y = -45.266 + 4.093t + 5.813v - 1.334d	0.77	5.80%	94.20%
HR	y = -55.242 + 4.952t + 7.299v -1.702d	0.81	2.20%	97.80%

The differences for site 002 under the different weather conditions are further illustrated in figure 5.3 below. This is based on an assumption of 16.67m/s as the speed, distance of 50m, and travel time of 3s and shows that traveling below the posted speed limit will reduce chances of RLR under all conditions.

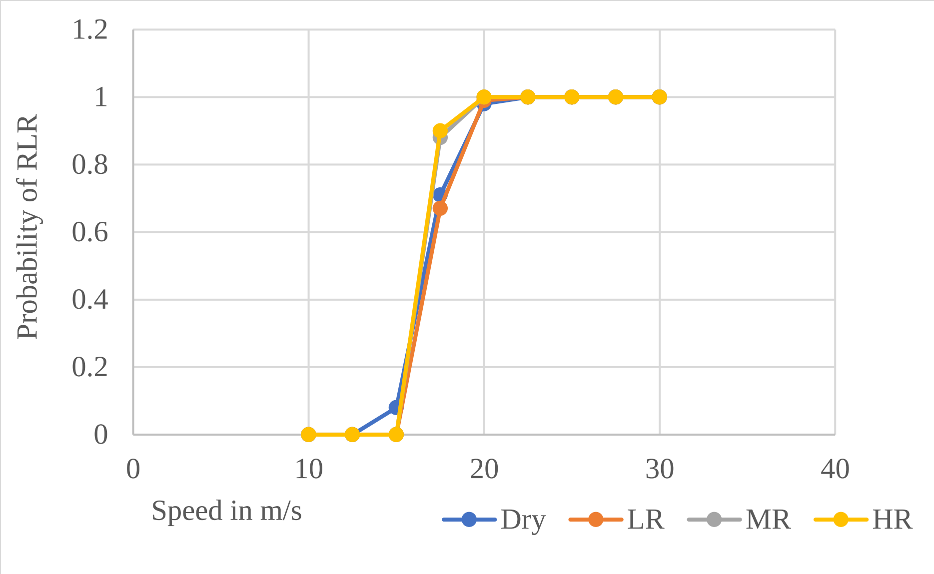

Figure 5. 3: Probability of RLR for Site 002 under different weather conditions

5.3.3 Site 003

Analysis of speed showed that under dry conditions, considering the approach speed at the dilemma zone (50m from intersection stop-line), under RLR the average speed was 21.43m/s (77km/h). for the vehicles that stopped safely, the average speed was 13.76m/s (50km/h). Given travel time(s), speed (m/s), and sight distance (m) regression models were fitted for this site under dry and rainfall weather conditions. The probability of RLR and stopping was estimated in each case and the results for site 003 dry weather conditions are summarised in table 5.19. From the table the *t'* values all have an absolute value greater than 2.5; therefore, all the variables used in the regression equation were useful in predicting red-light running violation. The R^2 value (0.74) is greater than 0.5, and the F value is greater than 4 suggesting the model equation did not happen by chance. The model equation; $y = -14.394 + 1.435t + 1.445v - 0.289d$

Table 5. 19: Site 003 dry weather conditions summary results

Site 003- Dry weather conditions						
	d	v	t	Constant	Stats Test	
t' values	-4.47896	4.917198	5.033191	-5.3189615	t > 2.5	ok
Coefficients	-0.28925	1.444533	1.434636	-14.393895		
Std. Error	0.064579	0.293772	0.285035	2.70614768		
R^2	0.743672	0.230002	#N/A	#N/A	$R^2 > 0.5$	ok
F df	75.43245	78	#N/A	#N/A	F > 4	ok
Residuals	11.9713	4.126259	#N/A	#N/A		

Assuming a travel time of 3s, based on posted limit of 16.67m/s and distance of 50m and substituting in the model equation above, and applying equations 5.5 and 5.6 gives the probability of RLR under dry conditions as 38.4% and that of stopping as 61.6%. To estimate the best combination for distance and time, the distance values of 60m, 70m, and the yellow time values of 2s, 3s, and 4s were used iteratively with results shown below in table 5.20. These results show that increasing yellow light time from 3s to 4s increases red-light running whilst decreasing yellow light time from 3s to 2s decreases red-light running under dry weather conditions. For 60m distance, the probability of RLR is below 20% but is more than zero. It is highest when the time is increases to 4 seconds. For 70m distance, in all cases the probability of RLR was less than 1% which are similar to the results posted for sites 001 and 002.

Table 5. 20: Dry Weather Sensitivity Analysis for site 003

D (m)	*T* (s)	RLR (y =1)	RLR (y = 0)
50	2s	12.94%	87.06%
	3s	38.42%	61.58%
	4s	72.37%	27.63%
60	2s	0.82%	99.18%
	3s	3.34%	96.66%
	4s	12.68%	87.32%
70	2s	0.05%	99.95%
	3s	0.20%	99.80%
	4s	0.80%	99.20%

For light rainfall conditions, analysis of speed showed a reduction compared to dry weather conditions. For vehicles involved in RLR the average speed was 17.86m/s (64km/h) which is above the posted speed limit but lower than the one recorded for dry weather conditions, for vehicles that stopped the average speed was 13.3m/s (48km/h showing that on average light rainfall caused a reduction in average speed. Using the data to estimate a binary logistic regression equation for light rainfall intensity yielded the results posted in table 5.21. The statistical tests for R^2, t, and F both gave satisfactory results as can be seen in table 5.21. The equation for light rainfall intensity; $y = -26.208 + 2.447t + 3.110v - 0.685d$. Assuming travel time of 3s, a distance of 50m, and the posted speed of 60km/h for this case and substituting the model equation values showed the probability of RLR as 21.61% and that of stopping as 79.59%. Comparing this with the dry weather conditions shows that light rainfall causes the probability of RLR to reduce by up to 44%. The results for sensitivity analysis considering distances 50 to 70 and 2 to 4s time are presented in Table 5.22

Table 5. 21: Site 003 Light Rainfall intensity summary results

Site 003-Light Rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-2.81935	2.971992	2.791169	-3.0298099	t > 3	ok
Coefficients	-0.68522	3.109781	2.446787	-26.208094		
Std. Error	0.243042	1.046362	0.876617	8.6500789		
R^2	0.653778	0.303638	#N/A	#N/A	$R^2 > 0.5$	ok
F df	23.9187	38	#N/A	#N/A	F > 4	ok
Residuals	6.615608	3.503439	#N/A	#N/A		

The results similar to those of dry weather conditions show that reducing the time from 3 to 2 reduces the probability of RLR while increasing it also increases the RLR probability. For the 60m and 70m distances in all instances, the probability or RLR was below 0.5% and this can be attributed to the reduced speeds and the assumption that the speed limit is enforceable and as such from these distances' chances of RLR are very minimal.

Table 5. 22: Light Rainfall Sensitivity Analysis for site 003

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	2.33%	97.67%
	3s	21.61%	78.39%
	4s	76.10%	23.90%
60	2s	0.00%	100.00%
	3s	0.03%	99.97%
	4s	0.34%	99.66%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

Under moderate rainfall intensity, the results for the binary logistic regression equation are presented in table 5.23. The R^2 value was 0.53 which is above 0.5, all the other statistical tests done were satisfactory and the resultant regression equation; $y = -40.651 + 3.612t + 5.386v - 1.252d$. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under moderate rainfall intensity of 6.73% and that of stopping as 93.27% which is a further reduction on the results recorded for light rainfall as well dry weather conditions. The results for sensitivity analysis are presented in table 5.23. Like the results for dry and light rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60mand 70m were all zero for the probability of RLR.

Table 5. 23: Site 003 Moderate Rainfall intensity summary results

Site 003-Moderate Rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-3.82257	3.820202	3.296501	-3.595633869	t > 3	ok
Coefficients	-1.25201	5.386118	3.612343	-40.6514393		
Std. Error	0.32753	1.409904	1.095811	11.30577828		
R^2	0.530615	0.350291	#N/A	#N/A	$R^2 > 0.5$	ok
F df	29.3916	78	#N/A	#N/A	F > 4	ok
Residuals	10.81936	9.570881	#N/A	#N/A		

Table 5. 24: Moderate Rainfall Sensitivity Analysis for site 003

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.19%	99.90%
	3s	6.73%	92.50%
	4s	72.80%	15.40%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

For heavy rainfall intensity the results for binary logistic regression equation are presented in table 5.25. All the statistical tests done were satisfactory, R^2 value was 0.64 which is above 0.5 and the equation was y = -43.259+ 3.776t + 5.814v -1.355d. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under heavy rainfall intensity of 5.88% and that of stopping as 94.12% which is a further reduction on the results recorded for dry, light, and moderate rainfall. The results for sensitivity analysis are presented in table 5.26. Like the results for dry, light, and moderate rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60m and 70m were all zero for the probability of RLR.

Table 5. 25: Site 003 Heavy Rainfall intensity summary results

Site 003- Heavy Rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-3.92958	4.035006	3.919537	-4.08144949	t > 3	ok
Coefficients	-1.35534	5.814364	3.77578	-43.2592274		
Std. Error	0.344906	1.44098	0.963323	10.5989864		
R^2	0.64108	0.305962	#N/A	#N/A	$R^2 > 0.5$	ok
F df	28.57812	48	#N/A	#N/A	F > 4	ok
Residuals	8.025823	4.493408	#N/A	#N/A		

Table 5. 26: Heavy Rainfall Sensitivity Analysis for site 003

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.14%	99.86%
	3s	5.88%	94.12%
	4s	73.16%	26.84%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

A summary of all the regression equations for site 003 is presented in table 5.27. from the table, light rainfall caused a reduction in the probability of RLR of 44%, moderate 82%, and heavy 85%.

Table 5. 27: Summary of Probability RLR for site 001 all weather conditions

Weather	**Model Equations**	**R2**	**RLR, y = 1**	**RLR, y = 0**
Dry	y = -14.394+ 1.435t + 1.445v - 0.289d	0.74	38.40%	61.60%
LR	y = -26.208+ 2.447t + 3.110v - 0.685d	0.65	21.61%	78.39%
MR	y = -40.651+ 3.612t + 5.386v - 1.252d	0.53	6.73%	93.27%
HR	y = -43.259 + 3.776t + 5.814v -1.355d	0.64	5.88%	94.12%

The differences for site 003 under the different weather conditions are further illustrated in figure 5.4 below. This is based on an assumption of 16.67m/s as the speed, distance of 50m, and travel time of 3s and shows that traveling below the posted speed limit will reduce chances of RLR under all conditions.

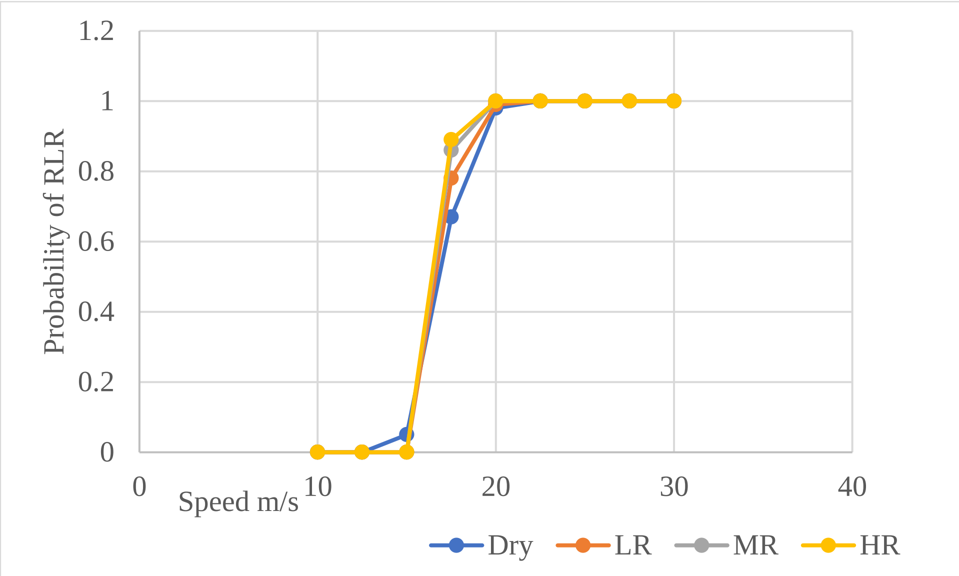

Figure 5. 4: Probability of RLR for Site 003 under different weather conditions

5.3.4 Site 004

Analysis for this site was done like for the previous three sites under dry and different rainfall intensities. Analysis of speed showed that under dry conditions, considering the approach speed at the dilemma zone (50m from intersection stop-line), under RLR the average speed was 22.5m/s (81km/h). for the vehicles that stopped safely, the average speed was 13.43m/s (48km/h). This shows that drivers that complied with the posted speed limit of 60km/h were likely to stop at the intersection stop-line. Whereas drivers with speed over the legal limit were more likely to violate the red lights at the signalised intersection.

Given travel time(s), speed (m/s) and sight distance (m) regression models were fitted for this site under dry and rainfall weather conditions. The probability of RLR and stopping was estimated in each case and the results for site 004 dry weather conditions are summarised in table 5.28. From the table the *t'* values all have an absolute value greater than 2.5; therefore, all the variables used in the regression equation were useful in predicting red-light running violation. The R^2 value (0.78) is greater than 0.5, and the F value is greater than 4 suggesting the model equation did not happen by chance. The model equation for dry weather therefore was; $y = -7.036 + 0.661t + 0.680v - 0.126d$

Table 5. 28: Site 004 dry weather conditions summary results

Site 004- Dry weather conditions						
	d	v	t	Constant	Stats Test	
t' values	-2.78641	3.283509	3.041819	-3.5453376	t > 3	ok
Coefficients	-0.12594	0.679949	0.661336	-7.036014466		
Std. Error	0.045198	0.20708	0.217415	1.984582361		
R^2	0.780792	0.23661	#N/A	#N/A	$R^2 > 0.5$	ok
F df	67.6757	57	#N/A	#N/A	F > 4	ok
Residuals	11.36628	3.191092	#N/A	#N/A		

Assuming a travel time of 3s, based on the posted limit of 16.67m/s and distance of 50m and substituting in the model equation above, and applying equations 5.5 and 5.6 gives the probability of RLR under dry conditions as 49.6% and that of stopping as 50.4%. Assuming that the speed limit rule is rigorously enforced at the study and the distance to stop line clearly marked, what is the best yellow time and distance to stop line combination it may be queried. To estimate the best combination, the distance values 60m, 70m, and the yellow time values 2s, 3s, and 4s were used iteratively with results shown below in table 5.29. These results show that increasing yellow light time from 3s to 4s increases red-light running whilst decreasing yellow light time from 3s to 2s decreases red-light running under dry weather conditions. For 60m distance, the probability of RLR is below 50% but is more than zero. It is highest when the time is increased to 4 seconds. For 70m distance, in all cases, the probability of RLR was less than 15%, which is the highest recorded for the 70m distance and could be attributed to the high average speeds recorded for this site under dry conditions. It should also be noted that sites 003 and 004 had a higher share of public transport vans that are generally aggressive in traffic.

Table 5. 29: Dry Weather Sensitivity Analysis for site 004

D (m)	*T* (s)	RLR (y =1)	RLR (y = 0)
50	2s	33.72%	66.28%
	3s	49.64%	50.36%
	4s	65.64%	34.36%
60	2s	12.62%	87.38%
	3s	21.86%	78.14%
	4s	35.15%	64.85%
70	2s	3.94%	96.06%
	3s	7.36%	92.64%
	4s	13.33%	86.67%

For light rainfall conditions, analysis of speed showed a reduction compared to dry weather conditions. For vehicles involved in RLR, the average speed was 18.8m/s (68km/h) which is above the posted speed limit but lower than the one recorded for dry weather conditions, for vehicles that stopped the average speed was 14.49m/s (52km/h) showing that on average light rainfall caused a reduction in average speed. Using the data to estimate a binary logistic regression equation for light rainfall intensity yielded the results posted in table 5.30. The statistical tests for R^2, t, and F both gave satisfactory results as can be seen in table 5.30. The equation for light rainfall intensity becomes $y = y = -34.123 + 3.347t + 3.863v - 0.844d$. Assuming travel time of 3s, a distance of 50m, and the posted speed of 60km/h for this case and substituting the model equation values showed the probability of RLR as 13.13% and that of stopping as 86.87%. Comparing this with the dry weather conditions shows that light rainfall causes the probability of RLR to reduce by up to 74%. The results for sensitivity analysis considering distances 50 to 70 and 2 to 4s time are presented in Table 5.31

Table 5. 30: Site 004 Light Rainfall intensity summary results

Site 004- Light Rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-4.35291	4.562085	4.932854	-4.885200421	t > 3	ok
Coefficients	-0.84404	3.862881	3.347068	-34.12282695		
Std. Error	0.193902	0.846736	0.678526	6.984939001		
R^2	0.535695	0.341629	#N/A	#N/A	$R^2 > 0.5$	ok
F df	21.9214	57	#N/A	#N/A	F > 4	ok
Residuals	7.675371	6.652498	#N/A	#N/A		

Table 5. 31: Light Rainfall Sensitivity Analysis for site 004

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.52%	99.48%
	3s	13.13%	86.87%
	4s	81.12%	18.88%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.09%	99.91%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

The results like those of dry weather conditions show that reducing time from 3 to 2 reduces the probability of RLR while increasing it also increases the RLR probability. For the 60m and 70m distances in all instances the probability or RLR was below 0.5% and this can be attributed to the reduced speeds and the assumption that the speed limit is enforceable and as such from these distances' chances of RLR are very minimal.

Under moderate rainfall intensity the results for binary logistic regression equation are presented in table 5.32. The R^2 value was 0.5 which is equal to 0.5, all the other statistical tests done were satisfactory and the resultant binary equation was y = -45.024+ 4.702t + 4.814v - 1.038d. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under moderate rainfall intensity of 7.18% and that of stopping as 92.82% which is a further reduction on the results recorded for light rainfall as well dry weather conditions. The results for sensitivity analysis are presented in table 5.33. Like the results for dry and light rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60mand 70m were all zero for the probability of RLR.

Table 5. 32: Site 004 Moderate Rainfall intensity summary results

Site 004- Moderate rainfall intensity						
	d	v	t	Constant	Stats Test	
t' values	-3.78774	3.834738	3.590879	-3.774243357	t > 3	ok
Coefficients	-1.03792	4.814397	4.701727	-45.02440568		
Std. Error	0.274021	1.25547	1.309353	11.92938595		
R^2	0.500302	0.347449	#N/A	#N/A	$R^2 > 0.5$	ok
F df	19.02298	57	#N/A	#N/A	F > 4	ok
Residuals	6.889407	6.881085	#N/A	#N/A		

Table 5. 33: Moderate Rainfall Sensitivity Analysis for site 004

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.07%	99.93%
	3s	7.18%	92.82%
	4s	89.50%	10.50%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.03%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

For heavy rainfall intensity the results for binary logistic regression equation are presented in table 5.34. All the statistical tests done were satisfactory, R^2 value was 0.54 which is above 0.5 and the equation was y = -67.961 + 6.780t + 7.830v -1.740d. With this and assuming a time of 3s and distance of 50m at the posted speed limit of 16.67m/s gives the probability of RLR under heavy rainfall intensity of 1.60% and that of stopping as 98.40% which is a further reduction on the results recorded for dry, light, and moderate rainfall. The results for sensitivity analysis are presented in table 5.35. Like the results for dry, light, and moderate rainfall, increasing time from 3 to 4 increases RLR probability. The results for 60m and 70m were all zero for the probability of RLR.

Table 5. 34: Site 004 Heavy Rainfall intensity summary results

Site 004 - Heavy Rainfall						
	d	v	t	Constant	Stats Test	
t' values	-3.54966	3.450549	2.814685	-3.1033323	t > 3	ok
Coefficients	-1.73997	7.830412	6.779377	-67.960886		
Std. Error	0.490179	2.269324	2.408574	21.8993257		
R^2	0.53627	0.339927	#N/A	#N/A	$R^2 > 0.5$	ok
F df	14.2626	37	#N/A	#N/A	F > 4	ok
Residuals	4.944147	4.275365	#N/A	#N/A		

Table 5. 35: Heavy Rainfall Sensitivity Analysis for site 004

D (m)	*T* (s)	RLR (*y* =1)	RLR (*y* = 0)
50	2s	0.00%	100.00%
	3s	1.60%	98.40%
	4s	93.65%	6.35%
60	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%
70	2s	0.00%	100.00%
	3s	0.00%	100.00%
	4s	0.00%	100.00%

A summary of all the regression equations for site 004 is presented in Table 5.36. from the table light rainfall caused a reduction in the probability of RLR of 74%, moderate 86%, and heavy 97%. It can therefore be concluded that rainfall reduces the probability of RLR, and the magnitude of the reduction is dependent on the rainfall intensity.

Table 5. 36: Summary of Probability RLR for site 004 all weather conditions

Weather	**Model Equations**	**R^2**	**RLR, y = 1**	**RLR, y = 0**
Dry	$y = -7.036 + 0.661t + 0.680v - 0.126d$	0.78	49.60%	50.40%
LR	$y = -34.123 + 3.347t + 3.863v - 0.844d$	0.54	13.13%	86.87%
MR	$y = -45.024 + 4.702t + 4.814v - 1.038d$	0.50	7.18%	92.82%
HR	$y = -67.961 + 6.780t + 7.830v - 1.740d$	0.53	1.60%	98.40%

The differences for site 004 under the different weather conditions are further illustrated in figure 5.5 below. This is based on an assumption of 16.67m/s as the speed, distance of 50m, and travel time of 3s and shows that traveling below the posted speed limit will reduce chances of RLR under all conditions.

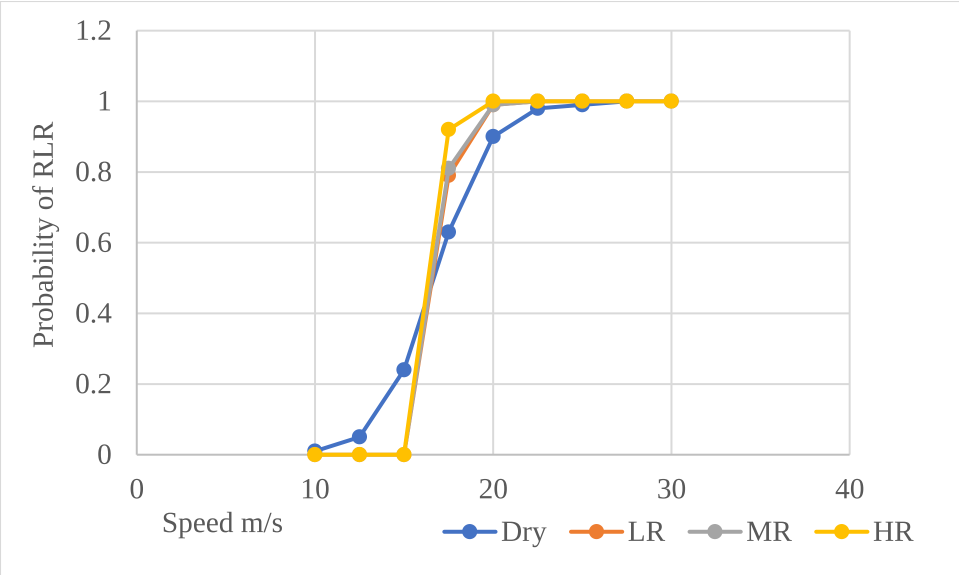

Figure 5. 5: Probability of RLR for Site 004 under different weather conditions

5.4 Rainfall impact on driver behaviour during yellow and all-red intervals

This section looked at the factors that contribute to red-light running and how they are affected by varying intensities of rainfall. Considering the speed of 60km/h(16.67m/s) and a 3 second yellow time, the distance traveled during yellow (16.67x3 ~50m) was marked. From the marked point at the onset of yellow and during the entire yellow time, vehicle speed as well the time from onset of yellow when a vehicle passed the marked point were extracted. At the intersection, stop-line speeds of vehicles crossed during the all-red interval were extracted and for those that stopped the time when they stopped was recorded. The time from onset of yellow when they passed the 50m mark was recorded, time of crossing stop-line/ stopping was also recorded and used to estimate vehicle acceleration/ deceleration. As the distance was kept constant, speed and acceleration/deceleration parameters were evaluated. The extracted data was then used to estimate binary logit models using equations 5.3 to 5.6 above.

The record for driver action to either stop at the end of the yellow interval or proceed was noted as; 0 for stop and 1 for RLR. The model equations for all study sites are presented in Table 5.38. From the table, the first subscript represents the site number (001-004) while the second represents the weather condition (D- dry, L- light rainfall intensity, M- moderate rainfall intensity, H- heavy rainfall intensity). All the R^2 values for the model equations were above 0.5 showing the fit of the parameters used in the estimation**.** Generally, from table 5.37, an increase in both speed and acceleration levels caused an increase in the probability of RLR. The magnitude of that increase grew with increase in rainfall intensity.

Table 5. 37: Model equations for all the sites and different weather conditions

Site	**Model Equation**	R^2
001	$Z_{1D} = -15.373 + 0.965v + 0.294\alpha$	0.74
	$Z_{1L} = -21.854 + 1.371v + 1.060\alpha$	0.83
	$Z_{1M} = -37.022 + 3.375v + 11.778\alpha$	0.74
	$Z_{1H} = -166.068 + 13.803v + 33.530\alpha$	0.75
002	$Z_{2D} = -36.71 + 2.732v + 10.610\alpha$	0.58
	$Z_{2L} = -41.065 + 3.778v + 13.126\alpha$	0.73
	$Z_{2M} = -89.795 + 7.734v + 17.100\alpha$	0.75
	$Z_{2H} = -137.700 + 14.055v + 46.553\alpha$	0.65
003	$Z_{3D} = -2.142 + 0.162v + 1.543\alpha$	0.67
	$Z_{3L} = -4.461 + 0.357v + 2.540\alpha$	0.73
	$Z_{3M} = -6.899 + 0.769v + 3.870\alpha$	0.74
	$Z_{3H} = -7.632 + 0.943v + 5.450\alpha$	0.66
004	$Z_{4D} = -40.460 + 3.157v + 23.540\alpha$	0.74
	$Z_{4L} = -62.900 + 7.070v + 30.870\alpha$	0.81
	$Z_{4M} = -66.320 + 9.243v + 50.451\alpha$	0.73
	$Z_{4H} = -75.090 + 13.444v + 64.260\alpha$	0.66

**** v= Speed in m/s and α = acceleration in m/s^2**

5.4.1 Impact of Acceleration/ deceleration

From the extracted speed data (at 50m mark and intersection stop-line) and the recorded timestamps, acceleration and deceleration values were calculated for vehicles that entered the intersection during the all-red and red intervals as well as those that stopped. In general, because all the vehicles recorded passed the 50m mark during the yellow time, those that passed either during all-red or red either maintained their speed or recorded different values of acceleration while those that stopped decelerated. These values were calculated for all the sites and under the different weather conditions and the results are summarized in Table5.38. The table shows the percentage reduction in acceleration and deceleration that was caused by the varying rainfall intensities.

Table 5. 38: Reduction in acceleration/ deceleration under different weather conditions

Site	**Weather condition**	**Acceleration in m/s^2**			
		YLO/RLR	% Reduction	Stop	% Reduction
001	Dry	2.13		-3.40	
	Light	1.51	29.11	-2.51	26.18
	Moderate	0.95	55.40	-2.45	27.94
	Heavy	0.85	60.09	-2.33	31.47
002	Dry	2.30		-3.38	
	Light	2.25	2.17	-2.50	26.04
	Moderate	1.55	32.61	-2.11	37.57
	Heavy	0.95	58.70	-1.62	52.07
003	Dry	2.05		-2.37	
	Light	1.60	21.95	-2.16	8.86
	Moderate	1.20	41.46	-2.08	12.24
	Heavy	0.95	53.66	-1.87	21.10
004	Dry	2.24		-2.53	
	Light	1.56	30.36	-2.28	9.88
	Moderate	0.99	55.80	-1.87	26.09
	Heavy	0.84	62.50	-1.97	22.13

For all the observations, on average, the vehicles involved in RLR accelerated while those that stopped decelerated. The average acceleration /deceleration values were found to be within the acceptable range (compared to values used in SARTSM for the determination of yellow and al-red intervals). For acceleration, there was a 20.95% reduction under light rainfall intensity, 46.32% under medium, and 58.74% reduction under heavy rainfall intensity. For deceleration, light rainfall caused a reduction of 17.74%, medium 25.96%, and heavy rainfall of 31.69%. The rate of reduction in acceleration is higher than deceleration, which could be an indication of how cautious drivers get under rainfall conditions.

To determine the effect of acceleration, speed was kept at a constant 16.67m/s which is the posted speed limit and the acceleration/deceleration varied between -4m/s^2 and 4m/s^2. The results were plotted and are presented in figures 5.6 and 5.7 below.

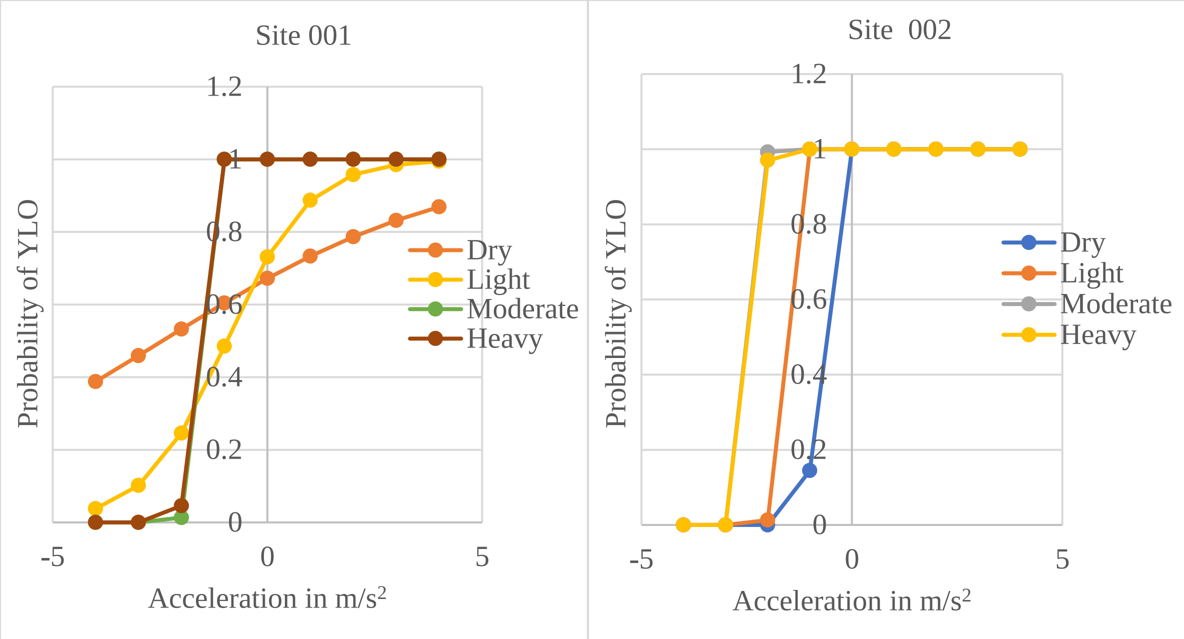

Figure 5. 6: The impact of acceleration/ deceleration on the probability of RLR

The figure shows there is a difference between sites 001 and 002. For site 001, under dry conditions and moving at the speed limit of 16.67m/s the probability of RLR is just above 60%. The probability increased to almost 100% under heavy rainfall conditions. For site 002, the probability of RLR is almost 100% for all cases. The difference between the two could be attributed to drivers in site 001 showing slightly more aggressive behaviour in their choice to cross the stop-line during the all-red interval. These differences can also be noted in figure 5.6.

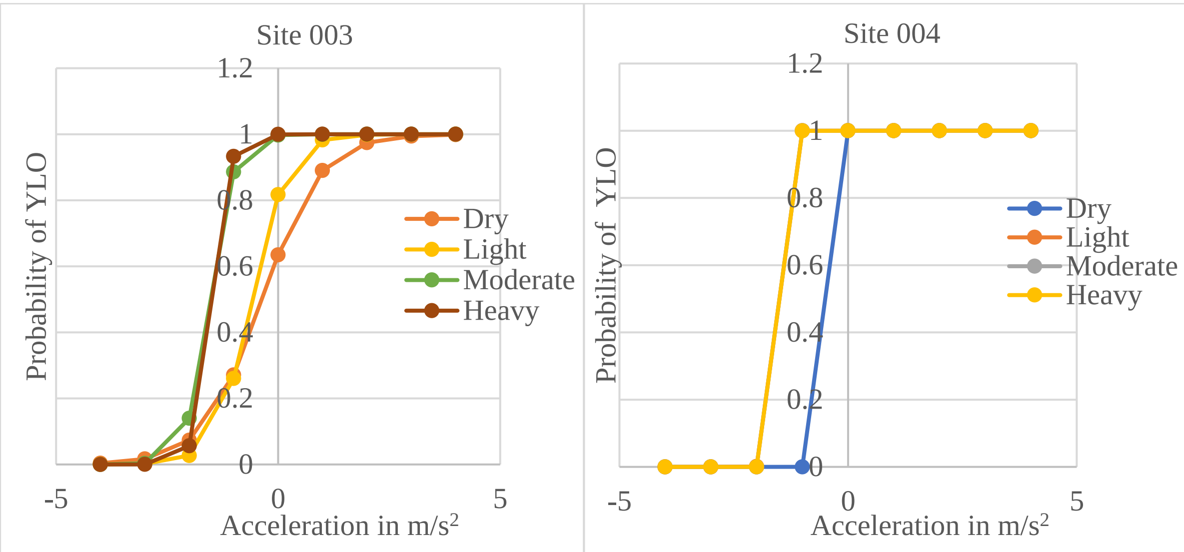

Figure 5. 7: The impact of acceleration /deceleration on the probability of RLR

From the figures above, at the posted speed limit and with zero acceleration the probability of RLR is above 0.5 for all cases. This is because these are vehicles that pass the 50m mark in the 2nd and 3rd second of the yellow time and as such chances of crossing the stop-line before the end of the yellow light are low. Under the moderate and heavy rainfall intensities even for values of acceleration below 0, the probabilities of RLR are still high and this could be because of the generally lower speeds experienced during wet weather conditions and therefore requiring longer time to get to the stop-line and cross.

5.4.2 impact of time

The time parameter was considered in this section in terms of how long after the start of the yellow time, a particular vehicle passed the 50m mark. The yellow time for all the study sites was 3 seconds and as such the time difference values ranged from 0 (those passing at the exact onset of yellow time) to 3 seconds (Those passing at the end of the yellow interval). An exception was made for vehicles that may have crossed the marked point during the all-red duration and went on to run the red light. Considering this definition of time, a comparison was made for all study sites under dry and different rainfall intensities.

Under dry conditions, the average times were 2.74s, 2.96s, 2.80s, and 3.17s for sites 001 to 004 respectively. This means that for vehicles that were involved in RLR, they were passing the 50m mark at the end of the yellow interval and could have stopped if they wanted or were forced to (like with speed enforcement). Under light rainfall conditions, this time reduced by 6.42% to 2.73s, by 16% to 2.44s under moderate, and by 21% to 2.3s under heavy rainfall conditions. This means that as rainfall intensity increased, the time difference from the onset of the yellow time for vehicles caught in RLR decreased and this can be attributed to lower speeds exhibited during rainfall conditions.

For those vehicles that stopped at the stop-line in time, under dry conditions they passed the 50m mark about 2 seconds from the onset of yellow time. The difference between those that stopped and those involved in RLR being the speed. These times also reduced with an increase in rainfall intensity, on average the light rainfall intensity time was less by 8.49%, that of moderate by 12.06%, and heavy by 23.94%. These reductions imply that as rainfall intensity increased drivers reacted to the yellow time by slowing down and stopping than attempting to cross the stop-line. Full detailed results for all the sites are presented in table 5.39.

Table 5. 39: Reduction in time under different weather conditions.

Site	Weather condition	Time from onset of yellow (s)			
		RLR	% Reduction	Stop	% Reduction
001	Dry	2.74		2.74	
	Light	2.73	0.36	2.31	15.69
	Medium	2.52	8.03	2.34	14.60
	Heavy	2.40	12.41	1.82	33.58
002	Dry	2.97		2.20	
	Light	2.83	4.71	2.10	4.55
	Medium	2.60	12.46	2.02	8.18
	Heavy	2.55	14.14	1.47	33.18
003	Dry	2.80		1.87	
	Light	2.40	14.29	1.70	9.09
	Medium	2.23	20.36	1.61	13.90
	Heavy	2.02	27.86	1.63	12.83
004	Dry	3.17		1.73	
	Light	2.97	6.31	1.65	4.62
	Medium	2.43	23.34	1.53	11.56
	Heavy	2.23	29.65	1.45	16.18

From table 5.40 all vehicles caught in RLR violations pass the 50m mark during the last second of the yellow time and as such were well able to see the red light coming on before the intersection stop-line. Under dry weather conditions because of higher speeds, some of these went to violate the red light, these violations were fewer under rainy conditions because of reduced speeds and acceleration. The reduction of that time as the rainfall intensity increased goes to further illustrate what has been mentioned earlier that it requires more time to get to the stop-line under wet weather conditions.

5.5 Summary

From the foregoing discussion, the following is the summary of the findings in this chapter.

In terms of the probability of red-light running, given driver approach speed, travel time to the intersection, and applicable sight distance, rainfall caused a significant reduction. The probability reduces from about 42% on average under dry conditions to 17% under light, 7% under moderate, and 2.5% for heavy rainfall intensity. This implies that under rainfall conditions due to speed reduction and hence increase in travel time it becomes near impossible to violate a red light. Rainfall can therefore be said to be a mitigating factor for red-light violations. Therefore, looking for measures to replicate driver behaviour during rainy conditions (speed and acceleration control) can help reduce red-light violations during normal dry weather conditions.

Most of the vehicles entering the intersection after the elapse of the yellow interval do so during the all-red interval. The frequency of RLR was quite low, most being below 1% of traffic volume. This could be attributed to the fact that on all study sites for the phases considered, the flow conditions were all unsaturated. The unsaturated conditions meant that at the end of each phase, the number of vehicles approaching the intersection was quite low and there were no vehicles left uncleared at the end of the phase. The presence of an all-red interval and bearing in mind the SARTSM definition of red-light running (vehicles entering during the last second of the all-red interval) also contributed to the low frequency.

Speed and acceleration impact RLR. An increase in both parameters increased the probability of RLR. An increase in rainfall intensity was also found to impact the parameters causing the probability of RLR to decrease. Rainfall was found to reduce speed as well as acceleration levels.

Speed and time from the onsite of yellow were found to impact the probability of RLR as well. An increase in both speed and time led to an increase in the probability of RLR. The longer after the onset of yellow that a vehicle passed the 50m mark, the higher the chances of RLR. This can be taken to mean that providing clearer information to drivers during the yellow interval could assist regulate violations. The information could be in form of speed control for example because in all cases of violation drivers were moving above the posted speed limit. The cases where speed was reduced because of rainfall showed lower cases of violations, showing how key speed control could become effective in reducing cases of RLR and especially unintentional cases involving drivers trapped in the dilemma zone.

In terms of distance, drivers at the yellow distance (50m mark) at the exact onset of yellow time were found to safely clear the intersection, for those in the dilemma zone, it was dependent on driver action. Those that accelerated (beyond the posted speed limit) passed without a RLR violation while those that decelerate or maintained the posted speed limit safely stopped. The decision to either stop or clear the intersection was found to be made in the distance between the 70m and 50m distances (dilemma zone). Therefore, it can be argued achieving uniformity in driver decision/action for drivers in the dilemma zone through speed enforcement, physical demarcation, or other safety measures could mitigate Red-light violation related incidences.

CHAPTER 6

IMPLICATIONS OF RAINFALL ON SERVICE QUALITY

6.1 Overview

Chapter 4 presented all the empirical data that was used in this study. This chapter used that data to estimate what varying intensities of rainfall have on signalised intersection saturation flow, capacity, and delay. The data was also used to develop a new criteria table that could be applied to evaluate signalised intersection quality of service. The developed criteria table was then used to quantify the impact rainfall has on signalised intersection service quality. Figure 6.1 below gives a brief breakdown of the steps followed to carry out this section of the study. In this study service quality was used to replace the commonly used level of service (HCM, 2016). Service quality definition was also expanded to incorporate the degree of saturation as an additional assessment parameter. The remainder of the chapter is divided into sections as follows: section 6.2 deals with the impact of rainfall on saturation flow for all study sites. Section 6.3 details the impact of rainfall on capacity for all the study sites. Section 6.4 discusses the development of the new quality of service criteria table. Section 6.5 applies the newly developed criteria table to analyse the impact rainfall has on quality of service and section 6.6 gives the chapter summary.

Collect data on; traffic signal information, volume, turning movements, headway, and rainfall

Check passenger car equivalent values using $PCE = \frac{\bar{h}_i}{\bar{h}_c}$

(Headway method) and modify PCE values accordingly

Compute saturation headway and calculate saturation flow by

$S = \frac{3600}{h_s}$ under dry and rainfall conditions

Calculate effective green time using $g = G + Y + AR - t_L$ then compute capacity using $c = s * ({}^{g}/_{c_o})$ under dry and rainy weather conditions

Develop new service quality criteria based on:

Degree of saturation $X = \frac{v}{c}$

Delay $d = d_1(PF) + d_2 + d_3$

$$d_1 = \frac{0.5C\,(1 - {}^{g}/_{C})^2}{1 - [min(1, X)\,{}^{g}/_{C}]}$$

$$d_2 = 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{8klX}{cT}}\right]$$

Evaluate the impact of rainfall on capacity and service quality

Figure 6. 1: Determination of rainfall implication on signalised intersection service quality

6.2 Determination of Rainfall Impact on Saturation Flow at Signalised Intersections

Rainfall data was a key input because the study was centred around determining the influence rainfall has on the performance of signalised intersections (red-light running and service quality in particular). Rain gauge stations were marked out, and the rainfall data was obtained from the eThekwini website and counterchecked with one maintained by the university's Electrical Engineering department. The rainfall data obtained was in terms of the amount and duration, this was then converted to rainfall intensity (mm/h) and then classified depending on the quantity. Rainfall intensity was obtained by dividing the amount recorded by the time duration. The rainfall intensity data obtained was then classified into light rainfall (L), with rain intensity < 2.5mm/h; moderate rainfall (M), with intensity 2.5 – 10mm/h; and heavy rainfall (H), with the intensity of 10 – 50mm/h, per the World Meteorological Organization (WMO). Very heavy rainfall with intensity > 50mm/h was not considered in this research work because the duration and the specific times studied this rainfall intensity was not recorded.

The procedure to determine the impact of rainfall on signalised intersection quality of service involved several steps and determination of rainfall impact on parameters that are used to measure the quality of service. The first one to be determined was the rainfall impact on saturation flow. To determine saturation flow, discharge headways were measured by manual observation of the videos and recording of time headways using a stopwatch. Vehicles were recorded as they crossed the stop-line into the intersection, from the 1^{st} up to the 14^{th} or last in the queue. To determine the vehicle number to be used in the estimation of discharge headway, first, a plot of discharge headway versus the vehicle position was done. The results showed the values stabilize after the 4^{th} vehicle; this was then subsequently used to determine the saturation headway.

The recorded discharge headways were then subjected to descriptive statistical analysis in terms of the mean, median, standard error, and skewness. The results for all site()s and through traffic are presented in Table 6.1 below

Table 6. 1:Descriptive statistic results for discharge headway values

Site	Sample size	Mean	Median	Standard error	Skewness	Kurtosis
001						
Dry	1500	1.42	1.41	0.01	0.01	-0.56
Light	400	1.56	1.54	0.02	0.36	-0.16
Moderate	450	1.56	1.53	0.02	0.34	-0.63
Heavy	400	1.68	1.63	0.02	0.35	-0.33
002						
Dry	1300	1.53	1.49	0.02	0.50	0.16
Light	150	1.60	1.58	0.03	0.44	-0.16
Moderate	200	1.62	1.58	0.03	0.66	0.34
Heavy	490	1.73	1.69	0.02	0.64	0.29
003						
Dry	1000	1.52	1.51	0.01	0.31	0.09
Light	280	1.68	1.60	0.02	0.61	0.25
Moderate	200	1.75	1.70	0.03	0.56	0.23
Heavy	470	1.92	1.92	0.02	0.66	1.28
004						
Dry	1400	1.56	1.54	0.01	0.39	0.23
Light	430	1.61	1.55	0.02	0.83	0.76
Moderate	400	1.76	1.72	0.02	0.74	0.38
Heavy	500	1.81	1.76	0.02	0.81	0.83

From the table; there are differences between the mean and median values, the difference ranges from 0.00s to 0.08s which was consistent with past findings (Shao and Liu, 2012), (Rahman et al., 2005) and (Wang et al., 2011). In terms of data distribution, the skewness was positive in all cases pointing to slight asymmetry of the headway values. The standard error for all cases was quite small and could be due to the large sample size in this study. The tabled results indicate that for the most part, the average values of the discharge headway are greater than median values which implies a slight underestimation of the saturation flow rates when using average values. (Darren and Paul, 2010)in their book consider values of kurtosis and skewness of between -2 and 2 to be acceptable to prove normal univariate distribution of data. This, therefore, meant that even with the slight asymmetry shown by the outcome of the descriptive statistics, it is reasonable to assume that the data falls within the range of normal distribution. As such using normal statistical results of mean and median to determine the discharge headway could proceed.

From the analysis, it was determined that the traffic composition at the selected study sites was not homogenous. The traffic had light, medium, and even heavy goods vehicles in the traffic during the considered times. Determination of passenger car equivalent (PCE) values was thus deemed necessary to convert traffic volume to flow and to cater for all vehicle types considered. To determine the PCE values, the average headway method was used. The average headway between two cars following each other was determined as was the average between two medium vehicles and two heavy vehicles, expression 5.1 was then used to determine the PCE value for each vehicle type. The determination was done for both dry and light rainfall conditions since it is based on headway values which are affected by weather conditions.

$$PCE = \frac{\bar{h}_i}{\bar{h}_c} \qquad 6.1$$

Where: h_i is the average headway of vehicle under consideration, h_c is average headway for passenger car

The results are summarised in Table 6.2 below. The table also gives the PCE values in the South African Geometric Design manual (SANRAL, 2011) that apply to the country.

Table 6. 2: PCE values

Vehicle type	Weather condition				SANRAL	Chi-Square results
	Dry	Light	Moderate	Heavy		
PC	1	1	1	1	1	0.04
MV	1.52	1.58	1.6	1.64	1.8	0.03
HV	1.99	2.01	2.02	2.06	2.3	0.04

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

A test was done to check if the difference between the PCE values obtained in this study were different from those in the manual. The test was carried out at the 95% level of confidence, where the null hypothesis stated that the difference was not statistically significant showed that the null hypothesis was accepted. The PCE values provided in the SANRAL design manual were therefore adopted to be used in this study. The discharge values as recorded in table 5.1 considered passenger cars only, additional discharge values were obtained considering all vehicle types. It was done for all sites for both through and turning traffic and taking into consideration all weather conditions. Further with the determined discharge headway values, the saturation flow rates were obtained using equation 6.2 below. The results of the saturation flow rates are presented in table 6.3.

$$S = \frac{3600}{h_s} \qquad 6.2$$

Where, S= saturation flow rate in pcu/hr/ln and h_s= average discharge headway

Table 6. 3: Saturation flow rate values for all sites and weather conditions

Site	Weather Condition	Through Traffic		Right turning traffic	
		Saturation headway h_s (s)	**Saturation flow(pcu/h)**	**Saturation headway h_s (s)**	**Saturation flow (pcu/h)**
001	Dry	1.62	2222	1.75	2057
	Light	1.65	2182	1.87	1925
	Moderate	1.75	2057	2.17	1659
	Heavy	1.80	2000	2.35	1532
002	Dry	1.62	2222	1.80	2000
	Light	1.67	2156	1.89	1905
	Moderate	1.73	2081	1.93	1865
	Heavy	1.80	2000	2.08	1731
003	Dry	1.63	2209	1.84	1957
	Light	1.68	2143	2.14	1682
	Moderate	1.74	2069	2.20	1636
	Heavy	1.78	2022	2.28	1579
004	Dry	1.64	2195	1.90	1895
	Light	1.78	2022	1.96	1837
	Moderate	1.92	1875	2.14	1682
	Heavy	1.93	1865	2.19	1644

From table 5.3, it can be noted that discharge headway values increase with an increase in rainfall intensity consequently the saturation flow rate values decrease. For through traffic, light rainfall intensity causes a reduction of 3.91%, 8.68% for moderate, and 10.88% for heavy. For right-turning traffic, light rainfall intensity causes a reduction of 7.07%. moderate 13.44% and heavy 17.88%. The general reduction figures show that the percentage reduction is higher for right-turning traffic as compared to through traffic. This could be attributed to the slight complexity of turning manoeuvre and the fact that drivers get more cautious when the road surface becomes wet and slippery when it rains.

6.3 Determination of Rainfall Impact on Capacity

The next parameter to be determined was capacity and specifically what impact different rainfall intensities have on the capacity. Capacity at signalised intersections is determined based on several parameters; one is saturation flow as detailed in section 5.2, then lost time and effective green time must be calculated too. Lost time is made up of start-up lost time and clearance lost time. Start-up lost time (SULT) which is the time lost at the beginning of the phase was determined according to the HCM, 2016 guidelines. While recording the discharge headway, the difference between the saturation headway and the first four discharge headways was summed up as shown in equation 5.3 below and averaged per and weather condition to give the SULT for all sites.

$$SULT = \sum_{1}^{4} \Delta n \qquad 6.3$$

Where Δn = headway difference between n^{th} vehicle and saturation headway

A summary of the SULT results is presented in table 6.4 below. Generally, the SULT values for dry conditions were the lowest and the values increased as the rainfall intensity increased. For most of the cases under moderate and heavy rainfall intensities, the recorded SULT values were more than 2 seconds, which is the value recommended for use under the guidelines like (SARTSM, 2012b) and (HCM, 2010). This means that with moderate and heavy rainfall intensities the SULT values generally used may be inadequate. On average, light rainfall intensity caused the SULT to increase by 7.5%, moderate by 19.7%, and heavy by 25%. T-test results showed that for moderate and heavy rainfall intensities, the differences between the two and dry conditions are statistically significant. (Perrin et al., 2001)and (Sun et al., 2013b) recorded similar results in their studies.

Table 6. 4: Start-up lost time under different rainfall intensities

Site	Weather Condition	Through Traffic	Right turning traffic	Cycle time (s)
		SULT (S)	SULT (s)	
001	Dry	1.83	1.74	120
	Light	1.84	1.85	
	Moderate	2.26	2.13	
	Heavy	2.44	2.18	
002	Dry	1.93	1.94	120
	Light	2.30	2.10	
	Moderate	2.37	2.22	
	Heavy	2.40	2.37	
003	Dry	1.86	1.37	100
	Light	1.90	1.41	
	Moderate	1.96	1.73	
	Heavy	2.06	1.82	
004	Dry	1.90	1.00	100
	Light	2.23	1.03	
	Moderate	2.26	1.24	
	Heavy	2.42	1.26	

Effective green and capacity were the next parameters to be determined. To do this equation 6.4 based on the HCM guideline below was used to determine effective green. The effective green time for a given movement is the sum of displayed green, yellow, and all-red clearance intervals less the total lost time. The total lost time is given as the sum of the SULT and the clearance lost time (2s was used for all sites).

$$g = G + Y + AR - t_L \qquad 6.4$$

Where: g= effective green time, G= displayed green time, Y= displayed yellow time, AR= all red time, and t_L= total lost time

To determine the capacity equation 6.5 was then used.

$$c = s * \left(g / c_o \right) \qquad 6.5$$

Where: c = Capacity in veh/h, g = effective green time in seconds, C_o = cycle time (s)
,
s= saturation flow rate

For all cases and under all weather conditions, the effective green time and capacity were determined and the summary of the results is presented in Table 6.5 below. From the table, the impact of rainfall on capacity can be summarised as; for through traffic, light rainfall causes a capacity loss of 4.25%, moderate 9.18%, and heavy 11.50%. The impact on the capacity loss for right-turning traffic was 7.38% for light rainfall intensity, 14.50% for moderate, and 19.15% for heavy. In general, the capacity loss is higher for right-turning traffic than through traffic. This could be attributed to right-turning traffic operating at near capacity and as such, any interference to the traffic due to rainfall had a slightly higher impact. Similar results on differences in the impact of rainfall between through and right-turning traffic were observed for saturation flow rate. The results of previous studies did not classify rainfall as was done in this case, nevertheless, similar capacity loss reductions were recorded by (Edward et al., 2014)

Table 6. 5: Effect of rainfall on capacity

Through Traffic						**Right-turning traffic**			
Site	**Weather Condition**	**g (s)**	**C_o (s)**	**S (pc/hr/ln)**	**C (pc/hr)**	**g (s)**	**C_o (s)**	**S (pc/hr/ln)**	**C (pc/hr)**
001	Dry	74.17	120	2222	1374	30.26	120	2057	519
	Light	74.16	120	2182	1348	30.15	120	1925	484
	Moderate	73.74	120	2057	1264	29.87	120	1659	413
	Heavy	73.56	120	2000	1226	29.82	120	1532	381
002	Dry	67.07	120	2222	1242	25.06	120	2000	418
	Light	66.70	120	2156	1198	24.90	120	1905	395
	Moderate	66.63	120	2081	1155	24.78	120	1865	385
	Heavy	66.60	120	2000	1110	24.63	120	1731	355
003	Dry	53.14	100	2209	1174	30.63	100	1957	599
	Light	53.10	100	2143	1138	30.59	100	1682	515
	Moderate	53.04	100	2069	1097	30.27	100	1636	495
	Heavy	52.94	100	2022	1071	30.18	100	1579	477
004	Dry	48.10	100	2195	1056	16.00	100	1895	303
	Light	47.77	100	2022	966	15.97	100	1837	293
	Moderate	47.74	100	1875	895	15.76	100	1682	265
	Heavy	47.58	100	1865	888	15.74	100	1644	259

** g= effective green time, C_o = cycle time, s= saturation flow rate, C= capacity

6.4 Development of service quality criteria

In this section, the steps taken to develop a new quality of service criteria are discussed. The proposed criteria table has expanded parameters incorporating degree of saturation and average back of queue (queue length) to the commonly used control delay parameter. The section is divided into two, where sites 001 and 002 are considered together and sites 003 and 004 also grouped.

6.4.1 Development of criteria for service quality assessment – Sites 001& 002

To determine the effect of rainfall on the quality of service at signalised intersections, a new quality of service criteria table was developed. The new criteria table incorporated degree of saturation and queue length in addition to the delay used in the current HCM manual. A stepwise approach was employed to establish the parameters required and develop the proposed criteria table.

The first step was to calculate the degree of saturation (X), which is simply a ratio of the volume to the capacity of the lane group/ traffic lane being considered. This means that for sites 001 and 002 and for through and right-turning traffic the prevailing traffic volumes had to be obtained. Volume was obtained directly from the traffic videos by manually counting the vehicles as they passed the intersection stop-line. The volume was measured for one hour in each case and recorded in vehicles per hour. Using the measured volume values and the capacity values from table 5.5 and applying equation 6.6 below the degree of saturation values were obtained. The summary of the results is presented in table 6.6 below.

$$X = \frac{v}{c} \qquad 6.6$$

Where: v= traffic volume (veh/h), c= capacity (veh/h)

Table 6. 6: Volume and degree of saturation results

Site	Weather Condition	Through Traffic			Right-turning traffic		
		Volume (Veh/hr/ln)	Capacity	v/c (X)	Volume (Veh/hr/ln)	Capacity	v/c (X)
001	Dry	644	1373	0.47	384	519	0.74
	Light	630	1348	0.47	361	484	0.75
	Moderate	599	1264	0.47	350	413	0.85
	Heavy	578	1226	0.47	332	381	0.87
002	Dry	564	1242	0.45	324	418	0.78
	Light	559	1198	0.47	315	395	0.80
	Moderate	543	1155	0.47	300	385	0.79
	Heavy	530	1110	0.48	290	355	0.82

From table 6.6, rainfall has an impact on traffic volume. An increase in rainfall intensity causes a decrease in prevailing traffic volume. The impact is higher on right-turning as compared to through traffic. On average, for through traffic, light rainfall intensity leads to a 1.6% reduction in volume, 5.4% under moderate, and 8.2% under heavy rainfall intensity. For the right-turning traffic, on the other hand, light rainfall intensity causes a 4.33% reduction in volume, moderate 8.07% reduction, and heavy 11.96%. The difference in volume reduction between through and right-turning traffic is like that observed for saturation flow and capacity. The degree of saturation (X) for both through traffic and right turning increased with an increase in rainfall intensity (in some cases remained constant), the low change in X could be attributed to the corresponding reduction in volume and the generally low values of X which indicate these lane groups were operating well below their capacity.

The next step involved the determination of delay. The delay, in this case, was to be used to develop the new quality of service criteria table, for this reason only dry weather conditions were used. To calculate the delay, the degree of saturation values ranging from 0 to 1 were used and the HCM equations 5.7 to 5.9 below were used.

$$d = d_1(PF) + d_2 + d_3 \qquad 6.7$$

d_3 =0 because there was no overflow delay, PF=1 and X varying from 0.0 to 1.0

$$d_1 = \frac{0.5C\left(1 - {}^{g}/_{C}\right)^2}{1 - \left[min(1,X)^{g}/_{C}\right]} \qquad 6.8$$

Where: d_1 = control delay, C= cyclce time (120 s for sites 001 and 002), g= effective green (74.17s and 67.07s for sites 001 and 002 respecively), X= varied from 0 to 1

$$d_2 = 900T\left[(x-1)+\sqrt{(x-1)^2+\frac{8klX}{cT}}\right] \qquad 6.9$$

Where: d2= incremental delay, T= duration of analysis (1 hour), c= capacity (1374 for 01 and 1242 for 02), k and l factors obtained from the manual (0.5 and 1 respectively for both cases).

Applying the above equations and varying X from 0 to 1 as already mentioned, the delay values were obtained and are presented in table 6.7 below.

Table 6. 7: Delay values for Sites 001 and 002

	Site 001	**Site 002**
X	**Delay (s)**	**Delay (s)**
0	8.75	11.67
0.1	9.48	12.52
0.2	10.31	13.50
0.3	11.30	14.67
0.4	12.49	16.00
0.5	13.97	17.65
0.6	15.87	19.73
0.7	18.46	22.53
0.8	22.48	26.82
0.9	30.83	35.69
1	71.50	77.50

To illustrate the relationship between delay and degree of saturation, the values in table 6.7 were plotted and the results are presented in figure 6.2 below.

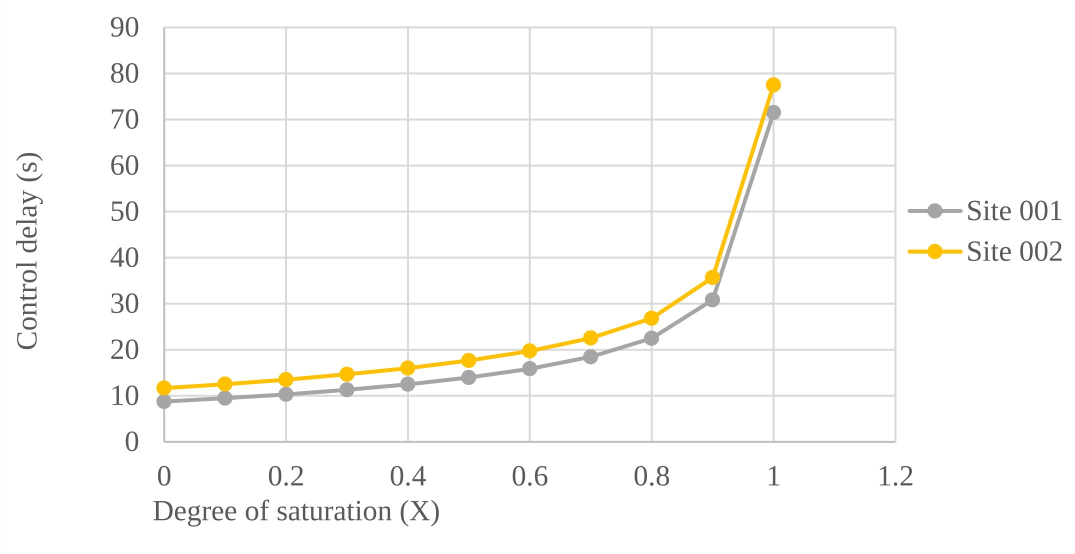

Figure 6. 2: Relationship between control delay and degree of saturation

The results combining the degree of saturation (X) and delay formed the first iteration in the development of a new criteria table and are summarised in table 6.8 below. If the criteria table was to be divided as per the divisions of the X values, there would be overlaps in the values of delay.

Table 6. 8: First iteration for sites 001 and 002

X	Site 001 Delay (s)	Site 002 Delay (s)
0	8.75	11.67
0.1	9.48	12.52
0.2	10.31	13.50
0.3	11.30	14.67
0.4	12.49	16.00
0.5	13.97	17.65
0.6	15.87	19.73
0.7	18.46	22.53
0.8	22.48	26.82
0.9	30.83	35.69
1	71.50	77.50

To come up with divisions where the parameters do not overlap, a back-and-forth process was employed and standard deviation values were used to determine the lower and upper boundaries. Dividing into five divisions was found to give reasonable results. With the overlap in divisions 0 to 0.3, they were put into one class and the rest divided into divisions of 0.2 and by this, the Level of Service (LOS) classes in HCM as well as the Canadian capacity guide, were maintained. To determine the lower and upper boundaries of each class, standard deviation values for each considered parameter were calculated and used. For Site 001, the proposed new criteria table is as shown in table 6.9. The table also shows the HCM and Canadian Capacity guide divisions for delay and X respectively.

Table 6. 9: Proposed criteria table for site 001

LOS/SQ	**X**	**Delay(s/veh)**	**delay (s/veh)-HCM**	**X - Canada capacity guide**
A	0.0< X <0.3	0< D <11	*≤ 10 ≥*	*0.0- 0.59*
B	0.3< X <0.5	11< D <15	*>10 and ≤ 20*	*0.60- 0.69*
C	0.5< X <0.7	15< D <20	*>20 and ≤ 35*	*0.70- 0.79*
D	0.7< X <0.9	20< D <33	*>35 and ≤ 55*	*0.80- 0.89*
E	0.9< X <1.0	33< D <72	*>55 and ≤ 80*	*0.90- 0.99*
F	>1.0	>72	*> 80*	*1.0 or >*

The HCM LOS are for this study renamed the service quality (SQ) class. For delay values, comparing HCM values, and those proposed for this new criteria table, the differences are low for class A, just a one-second difference. The highest-class F has a higher difference of 8 seconds, with the new criteria table having a lower value. For X values comparing the new criteria and the Canadian capacity guide, the highest classes D to F are quite similar. For classes A to C, the new criteria have lower divisions. In general, it should also be noted that the newly developed criteria were based on limited data for this study and as such a more comprehensive study would need a wider-ranging study and data collection.

For site 002, the same process was repeated, where divisions based on X values was done and the standard deviation was calculated to establish the lower and upper limit. The final proposed table results are presented in Table 6.10. Comparison with both HCM and the Canadian capacity guide was done as well. The degree of saturation classes for this case were like those of site 001, which means similarity in comparison with the Canadian capacity guide. For delay cases, site 002 showed some slight differences with the HCM manual as well. For the highest-class F, the difference was 2 seconds, and it was 5 seconds for class A. in general also, the HCM classes have a higher range in delay values for each of the classes, and this can be attributed to the fact the manual classes come from an amalgamation of different cases and scenarios as compared to just this one site case. Expanding the study to more cases and locations would probably then expand the ranges for the new criteria.

Table 6. 10: Proposed criteria table for site 002

LOS	(X)	Delay(s/veh)	delay (s/veh)- HCM	(X)- Canada capacity guide
A	0.0< X <0.3	0.0< D <15	≤ 10 ≥	0.0- 0.59
B	0.3< X <0.5	15< D <18	>10 and ≤ 20	0.60- 0.69
C	0.5< X <0.7	18< D <24	>20 and ≤ 35	0.70- 0.79
D	0.7< X <0.9	24< D <38	>35 and ≤ 55	0.80- 0.89
E	0.9< X <1.0	38< D <78	>55 and ≤ 80	0.90- 0.99
F	>1.0	>78	> 80	1.0 or >

6.4.2 Development of criteria for quality-of-service assessment – Sites 003 & 004

As was mentioned in section 6.4.1 above a stepwise approach was employed to establish the parameters (degree of saturation, delay, and queue length) required and develop the proposed criteria table. The first step was to calculate the degree of saturation (X), which is simply a ratio of the volume to the capacity of the lane group/ traffic lane being considered. This means that for sites 003 and 004 and for through and right-turning traffic the prevailing traffic volumes had to be obtained. Volume was obtained directly from the traffic videos by manually counting the vehicles as they passed the intersection stop-line. The volume was measured for one hour in each case and recorded in vehicles per hour. Using the measured volume values and the capacity values from table 6.5 and applying equation 6.6 the degree of saturation values were obtained. The summary of the results is presented in table 6.11.

Table 6. 11: Volume and degree of saturation results for sites 003 and 004

Site	**Weather Condition**	**Through Traffic**			**Right turning traffic**		
		Volume (veh/hr/ln)	Capacity	X	Volume (veh/hr/ln)	Capacity	X
003	Dry	634	1174	0.54	497	599	0.83
	Light	627	1138	0.55	433	515	0.84
	Moderate	615	1097	0.56	431	495	0.87
	Heavy	610	1070	0,57	420	477	0.88
004	Dry	612	1056	0.58	306	303	1.01
	Light	579	966	0.60	298	293	1.02
	Moderate	546	895	0.61	289	265	1.09
	Heavy	540	887	0.61	285	259	1.10

From table 6.11, rainfall causes a reduction in volume for both through and right-turning traffic. For the through traffic, light rainfall intensity causes a reduction of 3.2%, moderate 6.9%, and heavy 7.8%. The impact on the right-turning traffic; light rainfall intensity causes a reduction of 9%, moderate 14%, and heavy 16.28%. Overall, the reduction is higher for the through traffic than it is for the right-turning, the same was observed for sites 001 and 002. Rainfall also caused the degree of saturation to increase. The increase was more for the right-turning traffic compared to through traffic. This could be explained by the fact that right-turning traffic lanes were operating at capacity and any slight change in capacity affected the degree of saturation comparatively more than the through traffic lanes that were unsaturated.

The next step involved the determination of delay. The delay was to be used to develop the new quality of service criteria table, for this reason only dry weather conditions were used. To calculate the delay, the degree of saturation values ranging from 0 to 1 was used and the HCM equations 5.7 to 5.9 were used. Applying the above equations and varying X from 0 to 1, the delay values were obtained and are presented in table 6.12.

Table 6. 12: Delay values for varying degrees of saturation

X	Site 003 Delay (s)	Site 004 Delay (s)
0	11.00	13.47
0.1	12.17	14.33
0.2	12.70	15.33
0.3	13.75	16.47
0.4	15.00	16.88
0.5	16.51	19.44
0.6	18.44	21.48
0.7	21.08	24.26
0.8	25.16	28.60
0.9	34.00	38.00
1	76.00	78.50

The results obtained and recorded in table 6.12, were used in a plot of control delay versus degree of saturation as presented in figure 6.3

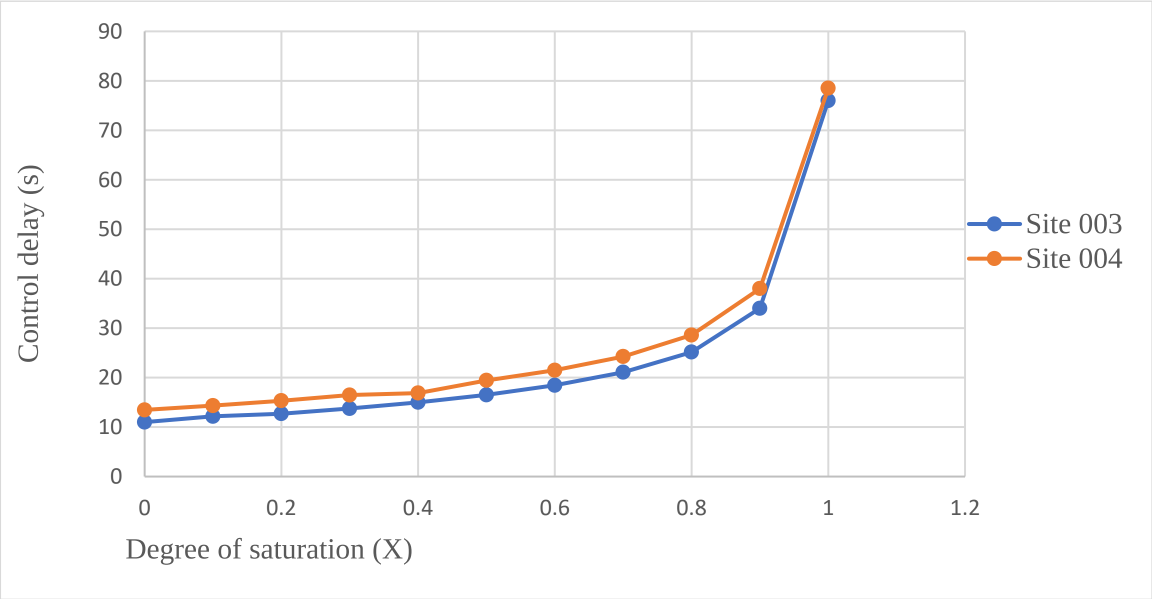

Figure 6. 3: Relationship between control delay and degree of saturation- Sites 003 and 004

Combining the results for control delay and X considering both sites 003 and 004, the results are shown in Table 6.13

Table 6. 13: First iteration for sites 003 and 004

X	Site 003 Delay (s)	Site 004 Delay (s)
0	11.00	13.47
0.1	12.17	14.33
0.2	12.70	15.33
0.3	13.75	16.47
0.4	15.00	16.88
0.5	16.51	19.44
0.6	18.44	21.48
0.7	21.08	24.26
0.8	25.16	28.60
0.9	34.00	38.00
1	76.00	78.50

Table 6.13 shows X values in 10 divisions, looking at these and the boundaries for each case, there would be overlaps were it to be used as-is for criteria development. To come up with divisions where the parameters do not overlap, a back-and-forth process was once again employed, and standard deviation values were used to determine the lower and upper boundaries. Dividing into five divisions was found to give reasonable results as well as being consistent with sites 001 and 002. With the overlap in divisions 0 to 0.3, they were put into one class and the rest divided into divisions of 0.2 and by this, the Level of Service (LOS) classes in HCM as well as the Canadian capacity guide, were maintained. To determine the lower and upper boundaries of each class, standard deviation values for each considered parameter were calculated and used. For site 003, the proposed new criteria table is as shown in Table 6.14 below. The table also shows the HCM and Canadian Capacity guide divisions for delay and X respectively. Table 6.15 gives the results for site 004.

Table 6. 14: Proposed new criteria table for site 003

LOS/SQ	X	Delay(s/veh)	delay (s/veh)- HCM	(X)- Canada capacity guide
A	0.0< X <0.3	0< D <14	≤ 10 ≥	0.0- 0.59
B	0.3< X <0.5	14< D <17	>10 and ≤ 20	0.60- 0.69
C	0.5< X <0.7	17< D <22	>20 and ≤ 35	0.70- 0.79
D	0.7< X <0.9	22< D <36	>35 and ≤ 55	0.80- 0.89
E	0.9< X <1.0	36< D <76	>55 and ≤ 80	0.90- 0.99
F	>1.0	>76	> 80	1.0 or >

Comparing the results for site 003 and both HCM and the Canadian capacity guide as shown in table 6.14, for delay values, the highest-class F has a difference of 4 seconds. And as previously recorded for sites 001 and 002, the division of delay values in HCM has a higher range than for this case. The division for X is like that of sites 001 and 002 as, like the two, levels D to F are similar. Differences can be noticed in levels A to C, where the Canadian capacity guide has level A with a wide range (0.0 to 0.59), the range was lower for this study.

For site 004 as shown in table 6.15, comparing delay values, class F has a similar value for this new criteria table as HCM. Differences can, however, be noted in the other classes, where class A, for example, has a 4-second difference.

Table 6. 15: Proposed new criteria table for site 004

LOS/SQ	v/c (X)	Delay(s/veh)	delay (s/veh)- HCM	X- Canada capacity guide
A	0.0< X <0.3	0< D <16	≤ 10 ≥	0.0- 0.59
B	0.3< X <0.5	16< D <20	>10 and ≤ 20	0.60- 0.69
C	0.5< X <0.7	20< D <25	>20 and ≤ 35	0.70- 0.79
D	0.7< X <0.9	25< D <40	>35 and ≤ 55	0.80- 0.89
E	0.9< X <1.0	40< D <80	>55 and ≤ 80	0.90- 0.99
F	>1.0	>80	> 80	1.0 or >

Combing all the tables for sites 001 to 004, where the level of service (LOS) is now referred to as service quality (SQ), the proposed criteria table could be used to evaluate the quality of service at signalised intersections is presented in table 6.16. As was already mentioned the variation in X values for all the cases was similar for all four sites. For delay values, there is some slight difference in the values for the different sites, up to 5 seconds in some cases.

Table 6. 16: Summary of proposed criteria table

Site 001			**Site 002**		
SQ	(X)	Delay(s/veh)	SQ	(X)	Delay(s/veh)
A	0.0< X <0.3	0< D <11	A	0.0< X <0.3	0.0< D <15
B	0.3< X <0.5	11< D <15	B	0.3< X <0.5	15< D <18
C	0.5< X <0.7	15< D <20	C	0.5< X <0.7	18< D <24
D	0.7< X <0.9	20< D <33	D	0.7< X <0.9	24< D <38
E	0.9< X <1.0	33< D <72	E	0.9< X <1.0	38< D <78
F	>1.0	>72	F	>1.0	>78
Site 003			**Site 004**		
SQ	(X)	Delay(s/veh)	SQ	(X)	Delay(s/veh)
A	0.0< X <0.3	0< D <14	A	0.0< X <0.3	0< D <16
B	0.3< X <0.5	14< D <17	B	0.3< X <0.5	16< D <20
C	0.5< X <0.7	17< D <22	C	0.5< X <0.7	20< D <25
D	0.7< X <0.9	22< D <36	D	0.7< X <0.9	25< D <40
E	0.9< X <1.0	36< D <76	E	0.9< X <1.0	40< D <80
F	>1.0	>76	F	>1.0	>80

6.5 Evaluation of the impact of rainfall on quality of service

The newly developed service criteria tables were used to evaluate the effect of rainfall on the quality of service for all the study sites. To do this, for each site the relevant parameters required to calculate, degree of saturation, and delay were recorded and the equations 5.7 to 5.9 for delay were used to calculate the respective delay values. This was done for each site, for through and right-turning traffic and considering the rainfall intensity.

For site 001, the results are presented in Table 6.17. From the results, the X values increased with increase in rainfall intensity. The increase was however small in magnitude and did not result in a substantial change in delay and the SQ level. This is an inconsistency because it is expected that rainfall would cause the delay and SQ level to increase. This anomaly could be attributed to the reduction in traffic volume that was noted earlier and the fact that these lanes are in general operating well below capacity and that any loss in capacity due to rainfall was quite low and as such is canceled out by the decrease in volume.

For the right-turning traffic, the results were more consistent with expectations. The degree of saturation increased with an increase in rainfall intensity, light rainfall led to a 1.4% increase in X, moderate 14.9%, and heavy 17.6%. The impact on X led to the delay values also increasing for these traffic lanes. The increase in X, delay, however, did not cause the service quality to change.

Table 6. 17: Impact of rainfall on quality of service – site 001

Site 001						
	Through			Right turning		
Weather condition	X	Delay (s)	SQ	X	Delay(s)	SQ
Dry	0.47	14	B	0.74	51.0	D
Light	0.47	14	B	0.75(1.4)	52.3	E
Medium	0.47	14	B	0.85(14.9)	64.5	E
Heavy	0.47	15	B	0.87(17.6)	71.5	E

** The value in brackets is the percentage increase

The impact of rainfall on the quality of service for site 002 is presented in table 6.18 below. For the through traffic lanes, the results are like those of site 001. The increase in the degree of saturation and delay were quite low and thus the service quality level remained constant. For the right-turning traffic lanes, the degree of saturation increased with an increase in rainfall. Light rainfall caused a 2.6% increase, moderate 1.3%, and heavy 5.1%. The increase in X values meant an increase in delay. The overall impact on the quality of service, however, was such that the level did not change.

Table 6. 18: Impact of rainfall on quality of service -Site 002

Site 002						
	Through			Right turning		
Weather condition	X	Delay (s)	SQ	X	Delay(s)	SQ
Dry	0.45	17	B	0.78	59.8	D
Light	0.47	17	B	0.80(2.6)	62.6	E
Medium	0.47	17	B	0.79(1.3)	60.2	E
Heavy	0.48	18	C	0.82(5.1)	67.2	E

** The value in brackets is the percentage increase

For site 003, the results are summarised in Table 6.19 below. For the through traffic lanes, the results obtained are like those of sites 001 and 002. The increase in the degree of saturation and delay were quite low and thus the service quality level remained constant. For the right-turning lanes, rainfall caused the degree of saturation to increase by 1.2% for light rainfall intensity, 4.8% for moderate, and

6.0% for heavy. The delay and average back of queue values also increased but as with the previously discussed cases, the quality-of-service class remains constant.

Table 6. 19: Impact of rainfall on quality of service – Site 003

Site 003						
	Through			Right turning		
Weather condition	X	Delay (s)	SQ	X	Delay(s)	SQ
Dry	0.54	17	C	0.83	53	D
Light	0.55	18	C	0.84(1.2	53	E
Medium	0.56	19	C	0.87(4.8)	54	E
Heavy	0.57	20	C	0.88(6.0)	55	E

** The value in brackets is the percentage increase

The impact of rainfall on the quality of service for site 004 is summarised in table 6.20 below. For the through traffic lanes, the quality of service remained constant while the degree of saturation and delay increased slightly. For the right-turning traffic, the degree of saturation for all the weather conditions was above 1, this means F quality of service for all. The degree of saturation increased by 1% under light rainfall intensity, 7.9% for moderate, and 8.9% for heavy. The quality of service remained F.

Table 6. 20: Impact of rainfall on quality of service – Site 004

Site 004						
	Through			Right turning		
Weather condition	X	Delay (s)	SQ	X	Delay(s)*	SQ
Dry	0.58	21	C	1.01		F
Light	0.60	21	C	1.02(1)		F
Medium	0.61	22	C	1.09(7.9)		F
Heavy	0.61	22	C	1.1(8.9)		F

** The value in brackets is percentage increase

*According to HCM and new SQ criteria for all X values above 1 the SQ level is F

Overall, for all cases, the through traffic for all the sites have degree of saturation less than 0.70 indicating that they are unsaturated. With an increase in rainfall intensity, the X values and delay increased slightly. This reduction in volume and the slight increase in both X and delay did not significantly affect the service quality. For the right-turning traffic, the results obtained showed X values of 0.74 and above. For these, rainfall caused the degree of saturation to increase. Delay values increased as well but the quality-of-service class remained constant. The high delay values and the equally high values for degree of saturation, in this case, indicate a capacity problem for the right-turning traffic. Comparing that, with the SQ for through traffic, it could also be an indication of a problem with signal time allocation among the phases

6.6 Summary

The chapter looked at the impact rainfall has on saturation flow, start-up lost time, capacity, delay, and quality of service. Data obtained was used to develop a criterion for the quality-of-service assessment at signalised intersections. The data used for this development were peak period and dry weather conditions. The newly developed criteria maintained the LOS classes used in HCM but added the degree of saturation (X).

Rainfall was found to cause the saturation flow rates to decrease with an increase in rainfall intensity. Capacity was also found to decrease with an increase in rainfall intensity. For the degree of saturation (X) it was also found to increase with increase in rainfall intensity, the increase was however higher in magnitude for right-turning traffic compared to through traffic. This was attributed to the fact that the right-turning traffic lanes were operating at almost capacity even for dry conditions and that the decrease in volume due to rainfall was not significant enough to cause the X values to decrease.

The developed service quality criteria were then used to evaluate the impact varying intensities of rainfall have. What can therefore be concluded in terms of the implication of rainfall on service quality at signalised intersections is that; rainfall causes a capacity loss dependent on intensity as well the lane group. For through traffic, with light rainfall there was a loss of 4.3%, moderate 9.2%, and heavy of 11.5%. For right-turning traffic the loss recorded was 7.3% for light rainfall intensity, 14.5% for moderate, and 19.2% for heavy. This capacity loss led to an increase in both the degree of saturation and delay. For service quality, for through traffic lanes, in one case, heavy rainfall caused the level to change from B to C. For right-turning lanes that were operating at capacity, there were some recorded instances of SQ level increase. This, therefore, means that the impact of rainfall on SQ also depends on the degree of saturation, where for lane groups that are saturated the impact of rainfall is more substantial than for unsaturated lanes.

CHAPTER 7

CONCLUSIONS

7.1 Overview

This study hypothesises that the extent of red-light running reduction resulting from daylight rainy conditions and service quality deterioration at the signalised intersection is significant. This exercise aimed to establish whether red-light running violation and service quality at signalised intersections are sustainable during rainfall and the relationship between the variables.

Within the purview of the study objectives, dry and rainy weather conditions used and the empirical results investigated in the light of evidence obtained from the examination of survey data. Four signalised intersections studied under dry, rainy weather and daylight conditions in Durban, South Africa. Rainfall classified as light with intensity (i) $\leq$ 2.5mm/h, moderate rain ($2.5 < i \leq 10$ mm/h) and heavy rain ($10 < i \leq 50$ mm/h) were used in conjunction with red-light running data to develop models. The analytical findings for both dry and rainy weather conditions compared. The empirical results from surveyed sites show that probability of red-light running decreased from 17% under dry weather to 6% light rainfall, to 1% moderate and 0% under heavy rainfall. Service quality delivery deteriorated due to rainfall. Rainfall accounted for an average capacity loss of 7.38% light, 14.5% moderate and 19.15% heavy rain. It is reasonable to suggest that speed reduction caused by rainy condition may affect the extent of red-light running reduction. Rainfall caused an increase in the degree of saturation and delay.

Based on the synthesis of evidence obtained from the stochastic relationship between red-light running and rainy conditions it is correct to conclude that rainfall significantly impact red-light running by way of speed reduction and, by extension, capacity loss. It is a welcomed development to trade capacity loss for road safety at a signalised intersection. It is correct to state that rainfall irrespective of intensity would reduce the probability of red-light violations at signalised intersections. In sum, the study showed that:

1. Red-light running significant reduction would result from rainfall;
2. A correlation exists between service quality delivery and rainfall; and
3. Delay and degree of saturation are important to service quality indicators

Section 7.1 summarises the Findings on the extent of red-light running at signalised intersections under dry and rainy conditions. while section 7.2 summarises the Findings on the relationship between red-light running and rainfall intensity. Section 7.3 focuses on the probability of red-light running under dry and rainy weather conditions, while section 7.4 presents the newly developed service quality criteria table with delay and degree of saturation as additional parameters.. Section 7.5 gives the findings on the impact of rainfall on the quality of service at signalised intersections and lastly section 7.6 presents the way forward.

7.1 Findings on the extent of red-light running at signalised intersections under dry and rainy conditions.

On average under dry weather conditions the frequency of red-light running was 0.8% of the total volume. This reduced to 0.4% and 0.3% and 0.05% Under light, moderate, and heavy rainfall conditions respectively. Rainfall therefore causes a reduction in the frequency of red-light running. In looking at the factors that contributed to red-light running, the decision by drivers at surveyed sites to either stop or proceed made between the 70 and 50m (dilemma zone) from the intersection stop-line. For, stopping the decision meant slowing down, and for those not stopping, on average, acceleration occurred. The lack of uniformity in driver action at this critical point was identified as among the contributors to unsafe situations. In terms of the impact of distance to intersection stop-line, it depended on driver action. Those that accelerate (beyond the posted speed limit) passed without an RLR violation, while those that decelerate or maintained the posted speed limit safely stopped without violating a red-light.

7.2 Findings on the relationship between red-light running and rainfall intensity

In analysing red-light running at signalised intersection, the time needed to safely cross the intersection at onset of yellow time was one of the main factors considered. In this study this time was found to increase from 3s during dry weather to 3.6s for light rain, 3.9s for moderate rain, and 4.5s for heavy rain. Therefore, it can be summarised from this that rainfall has a mitigating effect on red light violations especially when the intensity is heavy, under these circumstances it is near impossible to run a red light. Getting drivers under normal dry weather conditions to behave similarly, to how they do when it rains could be a way to improve red light compliance.

7.3 Findings on the probability of red-light running under dry and rainy weather conditions.

In terms of modelling the probability of red-light running, given driver approach speed, travel time to the intersection and applicable sight distance, rainfall caused a significant reduction. The probability reduced from about 42% on average under dry conditions to 17% under light, 7% under moderate and 2.5% for heavy rainfall intensity. It implies that it becomes nearly impossible to violate a red light under rainfall conditions due to speed reduction and hence increase in travel time. The empirical results from surveyed sites showed that the probability of red-light running decreased from 17% under dry weather to 6% light rainfall, to 1% moderate and 0% under heavy rainfall.

7.4 Development of a service quality criteria table with delay and degree of saturation parameters.

South Africa uses the Highway capacity manual for quality-of-service assessment for signalised intersections. HCM uses delay as the sole parameter for the determination of the quality of service. The study found that level of service based on the only delay was unable to reflect user perception of intersection LOS because beyond delay, drivers considered other factors like signal efficiency, left turn safety, clarity of pavement markings, and pavement quality an impact. Drivers were willing to wait a long time at the intersection in exchange for a protected left-turn movement for example. Therefore, it is important to consider additional parameters that influence user perception of quality of service; in this case, degree of saturation because it contains an element of capacity and capacity utilisation that could offer more useful information, especially for traffic management. Given the addition of a new criterion the level of service replaced by the term service quality (SQ) in the study and a new criteria table was developed.

7.5 Findings on the impact of rainfall on the quality of service at signalised intersections

The impact of rainfall on the quality of service at signalised intersections assessed with the newly developed service quality table presented significant outcomes. Some of the results were inconsistent with expectations; differences noted between through traffic and right-turning vehicles. For through-traffic lanes, it found that under the influence of rainfall, the degree of saturation decreased, there was a capacity loss, which led to increases in both degree of saturation and delay. The overall impact on service quality mixed, with few instances where heavy rainfall caused a deterioration in SQ. For right-turning traffic lanes, the results were more consistent with expectations. The degree of saturation increased with an increase in rainfall intensity, with heavy rainfall intensity recording an average increase of 10%. There was also an increase in both degrees of saturation and delay. Overall, the SQ level deteriorated especially under heavy rainfall conditions. The SQ deterioration under rainfall was higher in the right-turning lanes than through traffic ones and this was attributed to the degree of saturation, where the right-turning traffic lanes were more saturated than through traffic.

7.6 The way forward

This study highlighted how yellow interval contributes to red-light violations at signalised intersections especially the communication gap between the drivers and traffic signal control mechanism. Whilst the traffic signal timing does not pose significant problems to drivers, the absence of clearly visible dilemma zone demarcation on approach to the intersection stop line is a challenge that partly contributes to the issue of red-light running. The study showed how the red-light running decision is made in the zone traditionally designated as dilemma zone. By implication providing adequate information at this zone to guide driver behaviour could contribute to reduction of red-light running cases. There is therefore, need to take this further with a field study to provide clearer guideline on the location and type of information to be provided. From this study speed information was identified as key, other types of information could also be tested like signal timing.

In terms of the influence of rainfall on red-light running, rainfall was shown to have a mitigating effect where, in cases of medium to heavy rainfall there is no chance of red-light running. This was shown to be because of reduced speeds, drivers being more cautious and therefore longer headways and reduced cases of acceleration. Because rainfall cannot be generated at will to mitigate red-light running, Traffic managers can come up with measures to change driver behaviour at signalised intersections in terms of speed and car following. This can also be tested out using driver simulators and field work to guide on actual implementation. Information obtained from this can also be used in Intelligent Transportation system applications where drivers could get real time information on state of traffic lights which can aid in decision making.

The study findings also pointed out the need for more robust criteria to evaluate service quality at signalised intersections by incorporating other parameters in addition to the traditional delay used in the USA Highway Capacity Manual. The study also showed the uniqueness of each intersection when it comes to evaluating the performance of road traffic networks especially signalised intersections. The study recommends that highway capacity manual developed for South Africa take cognisance of findings in this study.

The study was limited to selected to limited to selected intersections in one city; expanding this to reach out to all intersection types and incorporate more locations could yield results that are more general and inclusive that can then be incorporated into a manual that can be used for South Africa and even the entire Southern Africa region.

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APPENDIX

Appendix A 1- Publication

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RESEARCH ARTICLE

The Extent of Capacity Loss Caused by Rainfall at Signalised Intersections

J. Oyaro[1] and J. Ben-Edigbe[1,*]

[1]*Department of Civil Engineering, University of KwaZulu-Natal, Durban, South Africa*

Abstract:

Background:

Even though their physical characteristics exert a constant influence on capacity and saturation flows, signalized intersections are fixed facilities not affected by rainfall. Whilst traffic conditions with varying effects can be regulated, rainfall conditions cannot be regulated but compensated for by warning drivers to reduce speed. Speed reduction has an impact on signalised intersection capacity, whilst signalised intersection capacity is a function of saturation flow, effective green, and cycle time. In this paper, a capacity loss is the differential percentage between 'with and without' rainfall scenario.

Aim:

The paper investigated the extent of capacity loss caused by rainfall at signalised intersections.

Methods:

In Durban, South Africa, rainfall data were collected, collated, and correlated with traffic data in a 'with and without' rainfall intensity study. Rainfall intensity was classified according to the rate of precipitation as follows; rainfall intensity(i): light rain (i <2.5mm/h); Moderate rain (2.5mm/h ≤ i < 10mm/h), and heavy rain (10 ≤ i ≤ 50mm/h) as prescribed by the World Meteorological Society.

Results:

Empirical results show that rainfall intensity has an effect on road capacity at a signalised intersection. Generally, for the vehicles going straight, light rain caused a 4.25% capacity loss; moderate rain 9.18% while heavy rain caused an 11.53% capacity reduction. With right-turning vehicles, light rain caused 7.38% capacity loss; moderate rain caused 14.3%, while heavy rain accounted for 19.15% capacity reduction.

Conclusion:

The paper concluded that rainfall at signalised intersections would cause an anomalous capacity reduction. Since the database for the study is small, the paper advocates for further studies based on a broader database to include yellow interval time.

Keywords: Capacity, Start-up lost time, Signalised intersection, Rainfall intensity, Cycle time, World meteorological society.

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1. INTRODUCTION

Road capacity is the maximum hourly flow rate at which road users can reasonably expect to traverse a point or a uniform section of a roadway during a given time under the prevailing roadway, environmental, traffic, and control conditions [1]. The definition suggests that the capacity of a roadway section is a function of numerous variables. At signalised intersections, capacity is defined as the maximum flow rate that would be observed based on the amount of green time that is available. Thus, capacity at signalised intersections is a function of saturation flow, effective green, and cycle time. Signalised intersection capacity loss in this paper is the differential percentage between 'with and without' rainfall scenario. At signalised intersections, traffic lights are used to manage traffic flow conflicts and ensure orderly vehicle movements. When the traffic light is green, vehicles proceed beyond the stop line; when the traffic light turns red, vehicles stop before the stop line. The yellow lights alert the drivers of impending red light or green as the case may be. Signalised intersections, as fixed facilities, are not affected by rainfall. Whilst traffic conditions can be regulated, it is difficult to regulate or predict rainfall. As mentioned in many literatures,

* Address correspondence to this author at Department of Civil Engineering, University of KwaZulu-Natal, Durban, South Africa; Tel: +27 796182384; E-mail: edigbe@yahoo.com.

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Appendix A2 – Conferences

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INFLUENCE OF RAINFALL ON THE PROBABILITY OF RED-LIGHT RUNNING AT SIGNALISED INTERSECTIONS

Oyaro, J[1], Ben-Edigbe, J[2]
[1,2] Sustainable Transportation Research Group, University of KwaZulu-Natal, Durban, South Africa
[1] ben-edigbe@ukzn.ac.za

Abstract: Factors that include road geometric design, traffic flow, road conditions, pedestrian crossings, and adverse weather among others influence drivers' behaviour at signalised intersections. The study looked at influence of rainfall on the probability of red-light running at signalised intersections. Red-light running (RLR) occurs when drivers violate the red light either because they are unable to stop or choose not to stop. Rainfall intensity was divided into three classes; light rain with intensity less than 2.5mm/hr., moderate rain with intensity between 2.5 - 10mm/hr., and heavy rain with intensity, between 10 - 50mm/hr.,. Heavy rainfall was excluded because of excessive drag force and poor visibility induced by torrential rainfall. Based on a 'with and without' rainfall, traffic and rainfall data were collected at three selected sites in Durban, South Africa. At selected sites approach to signalised intersection was divided into three zones (free flow, dilemma, and braking), yellow time fixed at 3s and a posted speed limit of (60km/h)16.67m/s. Based on fixed yellow time and posted speed limit, an estimated 50m distance to the stop line was computed. It was at sites that rainfall caused speed reduction and by extension increased travel time. Results show that the probability of red-light running during dry weather is 0.5, suggesting that RLR decision depends on driver's behaviour at the onset of yellow interval. Red-light running recorded at all sites occurred during dry weather conditions. Given that the road length is not affected by rain, the probability of red-light running diminished with rainfall intensity. Probability of RLR during light rain is 2%, moderate rain is 1% and heavy rainfall is 0%. The times needed to cross the signalised intersection safely at the onset of yellow interval time were increased significantly from t < 3s (dry weather travel time) to 3.73s for light rain, 3.94s for moderate rain and 5.03s for heavy rain. The paper concluded that it is suicidal to attempt red-light running during heavy rainfall, none recorded at all selected sites.

ASCE

Dear Janet Oyaro,

Please refer to the important information regarding submission of your final paper for ASCE International Conference on Transportation & Development (ICTD 2021).

I. Submission Site Access Information

The following is the access information for the paper management site that will be used going forward all the way to the conference. The information that you enter in your speaker/submitter account will be used to create the final program and the conference app

Your Presentation:
ID: 1503616
Title: Complimentary Effect of Chequered Road Markings on the Probability of Yellow Traffic Light Violation at Signalized Intersections

Complimentary Effect of Chequered Road Markings on the Probability of Red-Light Violation at Signalized Intersections

[1]Ben-Edigbe Johnnie PhD MSc BEng (Civil)
[2]Oyaro Janet MSc BEng (Civil)
[3]Bulose Siphesihle MSc BEng (Civil)

[1,2]Sustainable Transportation Research group
Department of Civil Engineering
School of Engineering
University of KwaZulu-Natal, Durban, South Africa

[3]South Africa National Road Agency Ltd.
Port Elizabeth, South Africa

ASCE Conference 2021

Abstract : At the onset of red, some drivers accelerate to clear the intersection, and others decelerate to stop at the stop-line. Red-light violation at signalized intersection occurs when a vehicle enters the intersection during the red phase. Despite attempts by researchers and practitioners to minimize red-light running, safety at signalized intersection is still a challenge in South Africa. The study investigated paper, the complementary effect of chequered road markings (CRM) on the probability of yellow traffic light violation at the signalized intersection. Yellow lights warn drivers that it is permitted to proceed and clear the intersection safety, however in a situation where the vehicle can neither enter nor be in the intersection on

SATC

Subject: Re: SATC 2021 Reviewer Report paper MODELLING DRIVERS ABERRANT RESPONSIVENESS TO YELLOW INTERVAL: CASE STUDY OF DURBAN SIGNALIZED INTERSECTIONS

Dear Prof Ben-Edigbe

We are pleased to inform you that your paper has been finally accepted for presentation at the SATC virtual conference and exhibition which will take place from 5 to 7 July 2021.

Registrations will open on 3 May, and presenters will attend at no cost on the day of presentation. The presenter will be registered by the conference secretariat.
Presentation time will be 15 minutes, and 5 minutes Q and A after the presentation. Clear instructions and guidelines will be sent to you with regards to the virtual presentation.
Please send us a short CV/ Bio of the presenting author (10 lines in Word, written in 3rd person)
Please see the final attached paper. Kindly confirm that this is the correct version to use.

Best regards

Jacqui Oosthuyzen
SATC Secretariat

MODELLING DRIVERS ABERRANT RESPONSIVENESS TO YELLOW INTERVAL: Case Study of Durban Signalized Intersections

Oyaro, Janet Ms.[a], Bulose, Siphesihle Mr.[b], Ben-Edigbe Johnnie Prof.[c]

[a, b, c] Sustainable Transportation Research Group (STR*g*)
Department of Civil Engineering
University of KwaZulu-Natal,
Durban 4041, South Africa
[c]Email: ben-edigbe@ukzn.ac.za

ABSTRACT

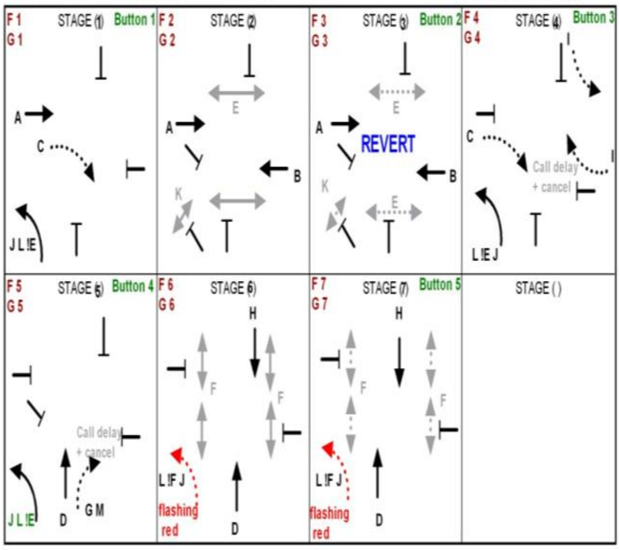
The paper explored the extent of drivers' aberrant responsiveness to yellow intervals at signalized intersections in Durban, South Africa. A signalized intersection primary light pole located at the stop-line has three different kinds of lights; green, yellow and red. It's a given that red means stop, green means go and yellow means when you enter the intersection during the entire yellow interval, it is permitted to proceed and clear the intersection safely, however in a situation where you can neither enter nor be in the intersection on red, you must stop upon receiving the yellow interval. At the onset of yellow, action taken is at driver's discretion; probably explaining why drivers' aberrant responsiveness is prevalent at signalized intersections. In a stochastic study carried out at four selected signalized intersections in Durban, a binary logistic model was used to estimate the probability of red-light running given; speed at a distance from stop line (51m, 70m) and acceleration variables. The results showed that for both the distances; 51m and 70m, the probabilities of red-light running were quite low at low speeds and increased with increase in both speed and acceleration. In the 51m distance, the aberrant behaviour

APPENDIX B1 – SITE 001 SIGNAL INFORMATION

Alpine Road / Umgeni Road / Intersite Avenue

J 12/ 611 Em 15068 Mv 00120

Date of issue:09/07/2018



Checksum F6DC 1 of 65 (CRC C29B) C to Flash (Shane)

Build 174

PHASES	LOCATION - ROAD NAME	DET I/P	PHASES	LOCATION - ROAD NAME	DET I/P
A	Umgeni Rd EBound	2(z2)	I	Umgeni Rd WBound RT	7(z6)
B	Umgeni Rd WBound	3(z1)	I	Intersite Ave LT	
C	Umgeni Rd EBound RT	1(z3)	J(SS)	SS Alpine Rd LT	
D	Alpine Rd	5(z4)	K	Alpine Rd LT Ped	15(pb3)
E	Umgeni Rd Peds	13(pb1)	L(soft)	Alpine Rd LT (J call + extend)	8(z9)
F	Alpine Rd Peds	14(pb2)	M(soft)	Soft Phase G	
G	Alpine Rd RT Right lane	4(z8)	!E(soft)	Soft Phase J green	
G	Alpine Rd RT Centre Lane	9(z10)	!F(soft)	Soft Phase J flashing Red	
H	Intersite Ave	6(z5)			

xChecksum 97B 1 of 64 (CRC 8E6) Build upgrade

xChecksum D57 1 of 63

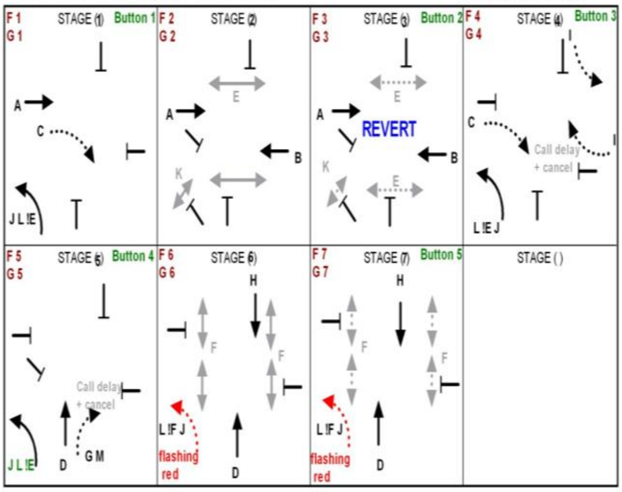
Plan Number	11
Type	Fixed Time Plan
Description	Am Peak
Phase Profile Mapping	!E !F A B C D E F G H I K L M
Fixed Time Plan Data	
Cycle Time	120
Stage Requests	
Definition	Duration
00 / S1 / DX	30
30 / S2 / DX	16
46 / S3 / DX	27
73 / S5 / DX	21
94 / S6 / DX	13
107 / S7 / DX	13

APPENDIX B2 – SITE 002 SIGNAL INFORMATION

Alpine Road / Umgeni Road / Intersite Avenue

J 12/ 611 Em 15068 Mv 00120

Date of issue:09/07/2018



Checksum F6DC 1 of 65 (CRC C29B) C to Flash (Shane)

Build 174

PHASES	LOCATION - ROAD NAME	DET I/P
A	Umgeni Rd EBound	2(z2)
B	Umgeni Rd WBound	3(z1)
C	Umgeni Rd EBound RT	1(z3)
D	Alpine Rd	5(z4)
E	Umgeni Rd Peds	13(pb1)
F	Alpine Rd Peds	14(pb2)
G	Alpine Rd RT Right lane	4(z8)
G	Alpine Rd RT Centre Lane	9(z10)
H	Intersite Ave	6(z5)

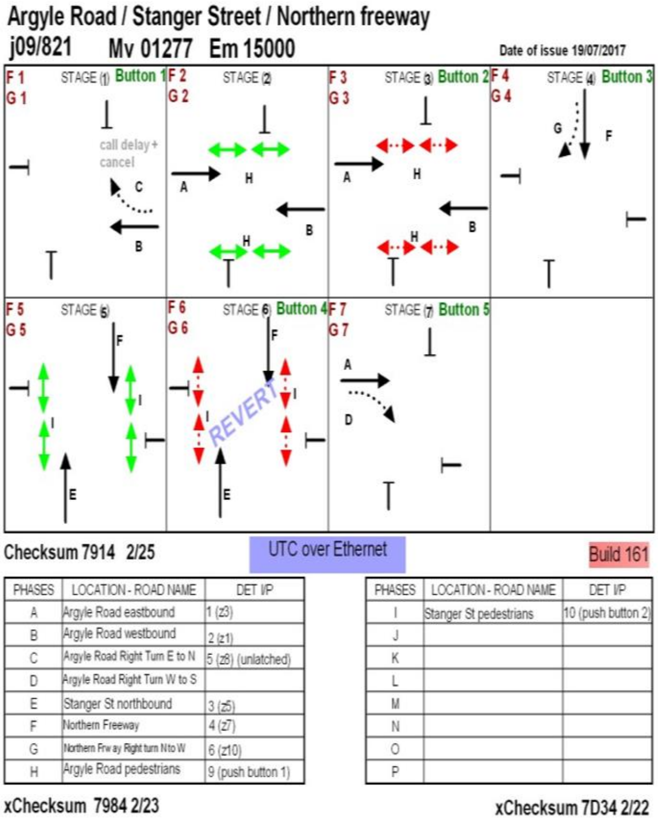
PHASES	LOCATION - ROAD NAME	DET I/P
I	Umgeni Rd WBound RT	7(z6)
I	Intersite Ave LT	
J(SS)	SS Alpine Rd LT	
K	Alpine Rd LT Ped	15(pb3)
L(soft)	Alpine Rd LT (J call + extend)	8(z9)
M(soft)	Soft Phase G	
!E(soft)	Soft Phase J green	
!F(soft)	Soft Phase J flashing Red	

xChecksum 97B 1 of 64 (CRC 8E6) Build upgrade

xChecksum D57 1 of 63

Plan Number	13
Type	Fixed Time Plan
Description	Pm Peak
Phase Profile Mapping	!E !F A B C D E F G H I K L M
Fixed Time Plan Data	
Cycle Time	120
Stage Requests	
Definition	**Duration**
00 / S1 / DX	24
24 / S2 / DX	15
39 / S3 / DX	35
74 / S5 / DX	20
94 / S6 / DX	13
107 / S7 / DX	13

APPENDIX B3 – SITE 003 SIGNAL INFORMATION



PHASES	LOCATION - ROAD NAME	DET I/P
A	Argyle Road eastbound	1 (z3)
B	Argyle Road westbound	2 (z1)
C	Argyle Road Right Turn E to N	5 (z8) (unlatched)
D	Argyle Road Right Turn W to S	
E	Stanger St northbound	3 (z5)
F	Northern Freeway	4 (z7)
G	Northern Frw ay Right turn N to W	6 (z10)
H	Argyle Road pedestrians	9 (push button 1)

xChecksum 7984 2/23

PHASES	LOCATION - ROAD NAME	DET I/P
I	Stanger St pedestrians	10 (push button 2)
J		
K		
L		
M		
N		
O		
P		

xChecksum 7D34 2/22

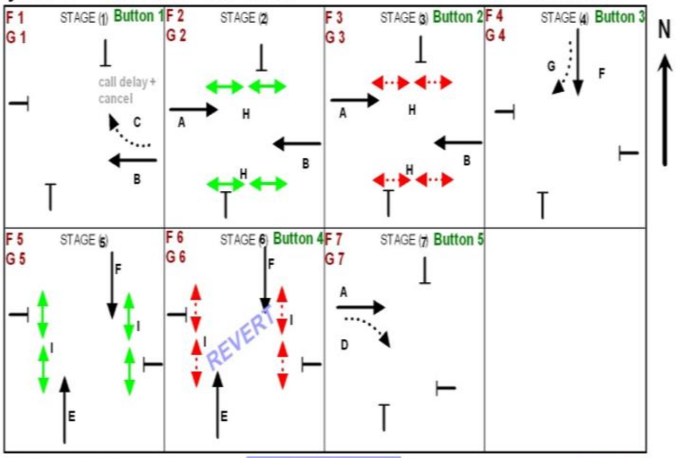
Plan Number	11
Type	Fixed Time Plan
Description	Am Peak
Phase Profile Mapping	A B C D E F G H I
Fixed Time Plan Data	
Cycle Time	100
Stage Requests	
Definition	Duration
00 / S2 / DX	10
10 / S3 / DX	13
23 / S4 / DX	29
52 / S5 / DX	21
73 / S6 / DX	13
86 / S7 / DX	14

APPENDIX B4 – SITE 004 SIGNAL INFORMATION

Argyle Road / Stanger Street / Northern freeway

j09/821 Mv 01277 Em 15000

Date of issue 19/07/2017



Checksum 7914 2/25

UTC over Ethernet

Build 161

PHASES	LOCATION - ROAD NAME	DET I/P
A	Argyle Road eastbound	1 (z3)
B	Argyle Road westbound	2 (z1)
C	Argyle Road Right Turn E to N	5 (z8) (unlatched)
D	Argyle Road Right Turn W to S	
E	Stanger St northbound	3 (z5)
F	Northern Freeway	4 (z7)
G	Northern Frway Right turn N to W	6 (z10)
H	Argyle Road pedestrians	9 (push button 1)

xChecksum 7984 2/23

PHASES	LOCATION - ROAD NAME	DET I/P
I	Stanger St pedestrians	10 (push button 2)
J		
K		
L		
M		
N		
O		
P		

xChecksum 7D34 2/22

Plan Number	13
Type	Fixed Time Plan
Description	Pm Peak
Phase Profile Mapping	A B C D E F G H I
Fixed Time Plan Data	
Cycle Time	100
Stage Requests	
Definition	**Duration**
00 / S1 / DX	16
16 / S2 / DX	10
26 / S3 / DX	13
39 / S4 / DX	14
53 / S5 / DX	32
85 / S6 / DX	15