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CALIBRATION AND VALIDATION OF A MICRO-SIMULATION MODEL FOR OPERATIONAL PERFORMANCE EVALUATION OF URBAN CORRIDORS

BY

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Declaration

I, Manneea Alina Montsi, declare that the content of this thesis represents my own unaided work and has not been previously submitted for any degree or examination at this or any other university, except where specifically stated.

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(Signature)

Signed in Cape Town, this ____12____ day of ____March____2020

Abstract

The use of traffic analytical and simulation models has gained more recognition in recent years from the transportation and traffic engineering community as alternatives to conventional methods for traffic congestion alleviation. These models are employed for the design, analysis and management of the transport system. The models offer a cost-effective means of assessing the effect of congestion alleviation alternatives on the transport system performance before implementation. Traffic analytical models such as SIDRA are considered as individual intersection models. However, when SIDRA is employed for the analysis of urban corridors with successive intersections, the impact each intersection has on the performance of the other is not considered and therefore, the actual performance of each intersection on the corridor is not truly reflected. For performance analysis of intersections locally, SIDRA is presently used and performance improvement alternatives are warranted, irrespective of the nature of the road network. This then raises concerns on the performance analysis adequacy of such networks.

To analyse urban corridors with successive intersections, simulation models such as VISSIM (a micro-simulation model) are employed because of their ability to capture and assess movement interactions of intersections within the network. Moreover, the applicability of the models for local traffic conditions must be investigated as none of the models were developed using South Africa's traffic conditions. The objectives of the study were to evaluate the performance improvement variations at two signalised intersections that were upgraded, and then to calibrate both SIDRA and VISSIM for the local urban corridor with successive signalised intersections. The better performing model was then validated using a new dataset different from that used for calibration purposes. The model was further used to perform an overall network performance analysis. The performance improvement variation was established by comparing before and after performance data with model predicted performance data (based on delay and LOS) for morning and afternoon peak hours (07:00 am – 08:00 am and 16:30 pm – 17:30 pm, respectively). An urban corridor with four successive signalised intersections was calibrated against field-measured performance (with respect to delay, LOS and travel times).

At the two upgraded intersections, significant variations between the models' predicted performance improvement and the obtained performance after upgrades were observed. The model (SIDRA) showed overestimation and underestimation behaviour regarding performance improvement predictions. The two models' (SIDRA and VISSIM) calibration results compared to field performance illustrated that the VISSIM model more accurately replicated field performance. The validation results of the calibrated VISSIM model well replicated the field performance showing a strong correlation between the two data sets. The validation results showed that for both delay and travel times the VISSIM model error results were lower than 12% for both MAPE and RMSE. The overall network performance showed that the corridor segment evaluated was operating at an undesirable level of service. The study concluded that isolated performance analysis of intersections might not present the actual performance of the intersection where an urban corridor consisting of successive intersections is concerned. Therefore, the VISSIM model can be additionally used for holistic performance analysis on local urban corridors. It is recommended that further research on the local applicability of the VISSIM model for performance analysis of larger urban networks and on urban corridors with mixed intersections be considered.

Dedication

- To my beloved family
- To Nkomeng Pali and Dumisani Valene
- To Diana Webster and Stephen Turner

Your love, support and motivation got me through this journey. Thank You!
Psalm 3

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Nomenclature

Constants

ax	Average stand still distance
ax_{addit}	Calibration parameter
ax_{mult}	Calibration parameter
bx_{addit}	Additive part of safety distance
bx_{mult}	Multiplicative part of safety distance
c_i	Capacity of lane group i (veh/h)
C	Cycle length (sec)
CF	Acceleration/Deceleration correction factor
d	Safety distance between vehicles
d^2	Square of the differences or deviations in the rankings
$\sum V_{iq}$	Sum of vehicle-in-queue counts (veh)
f_{PA}	Supplemental adjustment factor for platoon arriving on green time
FVS	Fraction of vehicles stopping
g_i	Effective green time for lane group i (sec)
g_i/C	Effective green ratio for lane group i
g/C	Proportion of green time available
G	Gradient (%)
I	Upstream filtering/metering adjustment factor
I_s	Interval between vehicle-in-queue counts(sec)
K	Incremental delay factor dependent on controller settings
L_{n-1}	Vehicle length
N	Sample size
P	Proportion of vehicles arriving on green time
PF	Uniform delay progression adjustment factor
s_i	Saturation flow rate for lane group i (veh/h)
SL	Speed limit (60 or 80 km/h)
T	Duration of analysis period (h)
TL	Number of through lanes (1 or 2)
v	Free flow velocity
V_{tot}	Total number of vehicles arriving during the survey period (veh)
X	v/c ratio or degree of saturation for lane group i
x_i	Sample value
$\bar{x}$	Sample mean
z	Drivers' behaviour ranging from 0 to 1

Greek Letters

P	Spearman’s rank correlation coefficient

Terms and concepts

ANOVA	Analysis of Variance
API	Application Programming Interface
ATCSs	Adaptive Traffic Control Systems
AWSC	All-Way-Stop-Control
Calibration	Procedure utilised in traffic analysis to (statistically) guarantee that the operation and performance of a particular model correlates with field measured data
Car-following	Phrase used to explain the condition of a vehicle with time gap less than a predetermined maximum value
CBD	Central Business District
CoCT	City of Cape Town
COM	Component Object Model
DOS	Degree of Saturation
EMME	Equilibre Multimodal, Multimodal Equilibrium
FHWA	Federal Highway Administration
Gap-acceptance	Term that defines and quantifies the interaction between prioritised and non-prioritised road users
GPS	Global Positioning System
GUI	Graphical User Interface
HCM	Highway Capacity Manual
HOV	Heavy Occupancy Vehicles
ITP	Integrated Transport Plan
ITS	Intelligent Transportation System
LOS	Level of Service
MAPE	Mean Absolute Percentage Error
Macroscopic (macro-) simulation	Simulation at a less detailed level
Mesoscopic simulation	Simulation at an intermediate level of abstraction compared to both microscopic and macroscopic simulation modelling
Microscopic (micro-) simulation	Simulation at a more detailed level
MoE	Measure of Effectiveness
NDoT	National Department of Transport
OD	Origin and Destination
OPAC	Optimised Policies for Adaptive Control strategies
PGWC	Western Cape Provincial Government
RHODES	Real-time Hierarchical Optimised Distributed Effective System

Simulation (traffic)	Real-time traffic system event occurrence operations imitations summarised over time. Traffic simulation is cognate with demonstrating the traffic systems procedures and can be conducted at various levels within the traffic network depending on the objective of the study being conducted
SAPS	South Africa Police Services
SCATS	Sydney Coordinated Adaptive Traffic System
SCOOT	Split Cycle Offset Optimisation Tool
SIDRA	Signalised and (Un-signalised) Intersection Design Research and Aid
Traffic system	A hypothesis used to characterise the traffic system procedure as intricate and vigorous interactions among the essential components at various hierarchical stages. The three mostly acknowledged components are: the roadway infrastructure, the road-user and the vehicle
TDA	Transport and Urban Development Authority
TIS	Traffic Impact Studies
TMC	Traffic Management Centre
TSIS	Traffic Software Integrated System
TWSC	Two -Way Stop-Control
UTC	Urban Traffic Control
UTMS	Urban Transportation Modelling System
Validation	Relative to simulation, validation alludes to the procedure of ensuring that after calibration the output of the developed model conforms (within the statistical allowance) to the field measured values, in this way ensuring the developed model abilities
VISSIM	Verkehr In Städten – SIMulations Model

1 Introduction

In recent times, urban road traffic, especially in the central business districts (CBDs) and metropolitan areas, has increased immensely, leading to issues such as congestion and roads accidents along road networks. Mobility, safety and sustainability are some of the utmost important aspects of the transportation industry because they impact considerably on the country's economic growth and the well-being of commuters (Mishra, 2016).

1.1 Background and motivation

Continuous traffic growth is a phenomenon experienced by most urban cities such as Cape Town which is the most congested city in South Africa (Kriel, 2017). Though congestion cannot be eradicated, control measures such as the construction of new roads and road widening can be put in place to alleviate traffic congestion. In order to limit this issue, caution is practised with regard to efficient transport infrastructure and transport planning management. However, most municipal governments do not have the financial capacity to keep up with the traffic demand by mitigating the situation with the construction of new infrastructure (the conventional way of mitigating traffic congestion). This conventional approach faces limitations from budget restrictions and inevitable increases in the cost of infrastructure construction, particularly in CBDs where there is a high concentration of traffic. Moreover, newly constructed infrastructure means that new vehicle traffic will be attracted, a phenomenon referred to as *induced traffic* (Borsari, 2012).

The introduction of intelligent transport systems (ITS) and the evolution of software engineering have led to the provision of alternatives to conventional methods for mitigating traffic congestion for traffic and transportation professionals. ITSs employ traffic analysis tools to design, assess and manage traffic systems. According to Reza (2013), traffic analysis tools are divided into two categories: analytical and simulation models. Traffic analytical models are applied for the analysis of intersections in isolation (i.e. analysing an intersection without considering adjacent intersections) (Fichera, 2011). Although traffic analytical models are still being employed because of their quick and reliable predictions and noteworthy field validation and implementation, they have several shortcomings.

These shortcomings include a limited approach when managing coordinated traffic impacts from sections adjacent to the other in the road network (such as a corridor, an arteria with successive intersections) in the traffic system. For the analysis of isolated (individual) sections of a network, it is taken as though the road or the intersection is not influenced by adjacent facilities. In an interview conducted with the Somerset West Senior Traffic Engineer Mr Mdlangaso (2018), he confirmed that one of the analytical models, SIDRA, is still being utilised for performance analysis of intersections, even in situations where intersections are successively located (either in an arteria or corridor) and isolation evaluation of intersections is conducted. In addition, pending the performance predictions by the model, upgrades are then implemented in the field. This then raises concerns of the adequacy of the performance improvement implemented in the field as a result of performance improvement alternatives provided by the model.

In order to address these shortcomings, simulation models have been introduced. According to Jobanputra (2013), simulation models provide comprehensive results for the whole study area. Simulation models have the ability to effectively assess the impact at one intersection due to the congestion present at another intersection. In addition, simulation models enable real-time visualisation of the transport system, often the critical primary form of validation, which can be analysed in a practical environment before field implementation. This is appealing in cases where geometric and operational changes would be costly (Jobanputra, 2013).

Most of these models, however, are developed based on different traffic condition and road geometries as in South Africa, bringing into question the applicability of the same default parameter values as those used in other countries. Thus, there is a need for calibration and validation of models for proper utilisation locally for traffic analysis. As Ahmed (2005) states, there is minimal information available for analysts in the application of simulation models, adequate models to employ as well as the precision of these models individually. According to Pretorius *et al.* (2004), there are numerous factors to be taken into consideration while modelling urban intersections, such as the appropriate calibration and validation of models that will impact the operational analysis of the subject intersections or corridor.

Simulation models are categorised into macroscopic, mesoscopic and microscopic models. Macroscopic simulation models (also referred to as macro-simulation models) are primarily applied for the analysis of large geographical areas and considered suitable for transportation planning purposes and for identifying spatial interactions within the transportation system (Moodley, 2016). Mesoscopic simulation models are a combination of both microscopic and macroscopic simulation models (Alexiadis *et al.*, 2004). Microscopic simulation models (also referred to as micro-simulation models) such as VISSIM analyse relatively large road networks due to their ability to assess and simulate individual vehicle movement in detail (from the entry point until it exits the network). Microscopic simulation models are applicable to local area networks (roads and intersections) to develop and assess traffic management strategies (Vanderschuren, 2007).

It is against this background that this study, using a case study in Cape Town, focussed on assessing the potential of a micro-simulation model VISSIM in terms of its applicability as an additional model for the analysis of a local urban corridor with successive signalised intersections in comparison with an analytical model, SIDRA.

1.2 Research problem

Traffic congestion has increasingly become an alarming concern for most countries, particularly in urban areas (Saidallah *et al.*, 2016). To better understand the congestion issues in urban road networks, traffic analysis tools (analytical and simulation models) are employed. The analysis of road networks using analytical models such as (SIDRA) shows considerable deficiencies where there is an interaction of vehicles from neighbouring intersections and other related traffic components in the network. This is because the models are unable to give a reasonably true reflection of the queue propagation from all intersections within the network (Cvitanić *et al.*, 2012).

While the use of SIDRA has been adopted by several researchers worldwide for road networks performance analysis (Fichera, 2011; Butera, 2012; Tianzi *et al.*, 2013; De Beer *et al.*, 2017), the results from those studies show that SIDRA underestimates or overestimates the operational performance of intersections in comparison with other model data or field measured data. According to Fichera (2011), SIDRA can only simulate a few impacts upstream and downstream to the intersection evaluated along the road network. For this reason, SIDRA is, for the most part, considered an independent intersection model. De Beer *et al.* (2017) suggest that any changes which are being introduced or implemented on the corridor are bound to affect the performance of the upstream and downstream intersections. Therefore, these effects might not be taken into consideration when evaluating the intersections in isolation.

From the literature, no documentation was found regarding the use of SIDRA in South Africa, except for Yumlu (1995), who calibrated the model using isolated (individual) signalised intersections from Johannesburg and Durban. However, no calibration was noted with respect to the use of SIDRA on signalised intersections in Cape Town. Considering this, SIDRA is still presently being used for local intersection performance analysis, irrespective of the nature of the network from which performance improvement alternatives provided by the model (SIDRA) are being warranted (Mdlangaso, 2018). This then raises the concern of the applicability of SIDRA for local traffic condition analysis, especially on corridors with successive signalised intersections.

As noted from the literature, a few researchers have attempted the calibration of a micro-simulation Paramics for traffic conditions in Cape Town (Vanderschuren, 2007; Jobanputra & Vanderschuren, 2012) but neither was dealing with urban corridors or arterials with successive signalised intersections. As no documentation was uncovered on the adaptation of VISSIM, the suitability of the model for performance evaluation of local traffic conditions needs to be tested. Nikolic *et al.* (2010) express that the ease of use of each model regarding qualitative (sensitivity to geometry and applicability) and quantitative characteristics (operational performance measures) must be investigated in the interest of finding the most appropriate model. In this regard, calibration and validation are crucial.

1.3 Research question

1. What is the adequacy of isolated analysis of signalised intersections located in between successive signalised intersections on an urban corridor?

2. What are the limitations of an analytical model in comparison to a simulation model on urban corridor analysis consisting of successive signalised intersections with respect to holistic operational performance evaluation of existing traffic conditions?

1.4 Objectives

The primary objectives of this study are as follows: (i) to evaluate the accuracy and capability of traffic engineering software (SIDRA) on the operational performance improvement prediction for local traffic; and (ii) to evaluate the operational performance estimation capabilities of the two models (SIDRA and VISSIM) on an urban signalised corridor segment. The two models were calibrated for local traffic conditions. Thereafter, the better performing model in comparison to field data was validated for the applicability on the local signalised urban corridor. In order to achieve this, the following objectives were met:

- to measure the effect of the implemented geometric upgrades at signalised intersections on the operational performance difference;
- to measure the difference between calibrated models results (SIDRA, VISSIM) and field measured operational performance at the signalised urban corridor;

- to measure the difference between the validated model and field measured operational performance at the signalised urban corridor; and
- to measure using statistical techniques, the difference between modelled and field measured operational performance.

1.5 Significance

The proposed study evaluates the accuracy of the currently used traffic analysis tool (SIDRA) by the City of Cape Town on real-time performance representation. The study is beneficial to the local traffic and transportation engineers regarding the competency of the models on road network performance analysis (especially for urban corridors with successive intersections). The study presents the limitations of using traffic analytical and simulation models for performance evaluation of signalised urban corridors.

1.6 Delineation

The study was restricted to evaluating the effect of geometric upgrades on performance improvement of two signalised intersections and to measuring the competency of two models – SIDRA and VISSIM –on the performance analysis of a local signalised urban corridor in Somerset West, Cape Town, South Africa. Other types of intersections (such as at-grade priority intersections, roundabouts), safety performance measures and modelling software were not covered. The degree of saturation (DOS) nor the volume/capacity (v/c) ratio for each movement was considered in the evaluation because their computation in VISSIM is time-consuming. Along the chosen study corridor segment, the intersection in between Gordon and Fagan intersections was disregarded during the analysis and no data was collected. This was because the intersection serves two commercial trip-generating places (a filling station and grocery store); therefore, the assumption was that the incoming and outgoing traffic volume was equivalent. The effect of signal spacing between the intersections was also not considered in this study. In addition, the effect of pedestrians on the performance of an urban corridor with signalised intersections was not investigated as this was not within the scope of the current study.

1.7 Methodology

This work was executed in two stages. Stage one investigated the effects of geometric upgrades at two signalised intersections based on performance improvement. This was accomplished by comparing the operational performance results before the geometric upgrades, modelled results and post upgrades performance (such as delay and Level of Service [LOS]) of the 'before' and 'after' geometric upgrade results with the modelled results. For stage two, both SIDRA and VISSIM models were calibrated for the operational performance analysis of local urban corridors with four consecutive signalised intersections using field data. Thereafter, the better performing model was validated using a new independent data set. For the calibration and validation procedures, delay, LOS and travel time were selected as the operational performance measures to be compared to field measure data.

1.8 Thesis organisation

Chapter 1 introduces the motivation and background to the research, the research problem, together with the research question, aims and objectives. It then details the research significance, research scope and delineation as well as the methodology.

Chapter 2 presents an overview of congestion impact on the flow of traffic in metropolitan cities, different aspects of traffic control and the elements of traffic operations. Thereafter, the fundamental topology of traffic engineering models is presented, specifically focusing on the review of microscopic simulation models and analytical models. Various examples of both microscopic simulation models and analytical models are discussed, particularly with providing details on SIDRA (an analytical model) and VISSIM (a micro-simulation model). The models' calibrating techniques are discussed, along with the importance of models' validation for local traffic condition applicability. The significance of investigating the adequacy of traffic models on performance prediction is also discussed. Previous research comparing microscopic simulation and analytical models, as well as 'before' and 'after' studies in relation to model performance improvement prediction, are presented.

Chapter 3 details the procedure adopted for conducting the research, specifically research design, research methodology, calibration and validation procedures, description of the study location, field data collection and reduction techniques, as well as the operational performance measures to be evaluated. This chapter also details the statistical measures used to evaluate reliability between the calibration and validation data and field measured data.

Chapter 4 presents the results obtained from the case study used to undertake several evaluations: firstly, geometric upgrade effect on the performance improvement of two signalised intersections; secondly, the competency of two models (SIDRA and VISSIM) on the operational performance analysis of a signalised urban corridor, based on delay, LOS and travel time; and finally, the statistical reliability. The performed overall network analysis results are also presented in this chapter.

Chapter 5 discusses the results obtained, with a detailed analysis of the effect of the implemented geometric upgrades at the two signalised intersections in relation to performance improvement. This was deduced from the comparison of the 'before' and 'after' upgrade implementation results. The analysis in relation to establishing the relationship between the results of the two models (SIDRA and VISSIM) on operational performance evaluation of a local signalised urban after calibration is also presented, leading to the validation of the better performing model. The statistical reliability of the calibration and validation data is discussed accordingly. Then the analysis of the general network performance conducted is presented. An analysis of the current study results in comparison to previously conducted research work is detailed.

Chapter 6 presents the conclusions drawn from this study, summarising the research findings with regard to the research question. The chapter also highlights contributions to the field, together with suggestions for further research areas in this field.

2 Literature review and theory

2.1 Introduction

Traffic congestion has, in recent years, escalated in severity, posing a significant problem in transportation networks in cities especially during peak hours. The issue has become even worse in large metropolitan areas, predominantly within the central business district (CBD). The consequence of traffic congestion, especially for daily commuters and motorists, is prolonged travel times and delays, more emissions and higher fuel consumption (Fatima, 2015). For Cape Town, like many other cities, traffic congestion results from many factors, not the least of which is the substantial number of people commuting regularly to and from work or tertiary institutions by public or private vehicles during the same time intervals. Another contributing factor is the increase in population within and around urban areas and the number of private car owners. With the increasing traffic congestion in metropolitan areas, traffic behaviour on urban road networks needs to be properly understood for appropriate transportation management and control strategies to be introduced. As a result, simulation modelling is gaining recognition in the Transport and Traffic Engineering field because it is considered an effective and cost-effective tool for the analysis of transportation system problems (Reza, 2013).

To effectively diagnose issues in transportation and traffic engineering and to select the appropriate mitigation plans for the road network, the capability to measure, assess and predict traffic operations is one fundamental element. Traffic analysis tools can be utilised at high levels to establish the performance of a facility, and at refined levels, such as for the development of traffic signal timing plans. Traffic operation analysis demonstrates the functional utility of how effectively intersections can accommodate all user group demands. Developments within computer technology have paved the way for an environment where simulation models have become a useful tool for traffic and transportation engineers in relation to road network design and management. Microscopic simulations can be introduced when assessing alternative geometric changes and traffic signal timing plans prior to field implementation of the design. They are able to estimate quantities that are challenging to observe in the field such as air quality, consumption rates for fuel and accident risk factors. These models can likewise be used for evaluating the capability of emerging technologies such as ITS (Park & Schneeberger, 2003).

In microscopic simulation, as the model monitors vehicle movements individually, these can be represented both in lateral and longitudinal form. This type of simulation, for the most part, utilises car-following, gap-acceptance and lane-changing models to signify vehicular movements individually. Nonetheless, due to various driving behaviours in real-time traffic, various obligatory parameters need to be integrated into simulation models; thus, model calibration for local traffic conditions is mandatory (Reza *et al.*, 2016).

2.2 South African context

Currently, Cape Town faces the challenge of congestion within the metropolitan area due to the increase in private vehicle ownership (Kriel, 2017). Simulation of traffic scenarios to motivate solutions by integrating different forms of movement within the traffic system is important. The Integrated Transport Plan (ITP) admits that congestion results in additional travel time incurred by commuters due to the existence of, and interaction with, other vehicles and people. Estimating, quantifying and finding solutions to congestion are not easy tasks with direct outcomes. The conventional assessment paradigm depends on traffic stream speeds, travel time or LOS. Traffic volumes during peak periods have been growing consistently in recent decades and numerous components of the road network have since come to or reached their capacity during the peak hour period (Kriel, 2017). South African traffic congestion, as is the case in Cape Town, is increasing as per information obtained from the TomTom Traffic Index 2019, which shows the average annual level of congestion for Cape Town at 32%, ranked 101 out of 416 cities worldwide.

The Traffic Management Centre (TMC) in Goodwood, Cape Town, was established to monitor the local traffic network in Cape Town, respond to traffic incidents and enhance traffic flow efficiencies (Kriel, 2017). This was accomplished by implementing various forms of infrastructure. This strategy was endorsed by multiple stakeholders such as City of Cape Town (CoCT), now known as TDA, Western Cape Provincial Government (PGWC), South African National Roads Agency Ltd (Sanral), National Department of Transport (NDoT), as well as the South African Police Service (SAPS) (Kriel, 2017).

Kriel (2017) further contends that the management of the traffic network was primarily achieved by providing traffic system awareness by way of navigation devices (GPS) which preliminarily indicated shortcomings regarding their accuracy and reliability. Transport services addressed by TMC include Transport Network Operations and Information Centre, Integrated Rapid Transit (IRT), Metropolitan Police and the Traffic Services. Traffic flow alleviation within the Western Cape will improve commuting time, enabling commuters to make informed decisions in planning their trips and enhancing incident responses.

According to Vanderschuren (2007), the South African government, together with public and private organisations, are on the verge of evaluating the potential advantages of applying (ITS) in the country. Therefore, microscopic simulation models can be considered for carrying out such investigations. With most of these models evolving in developed countries, their appropriateness to evaluate the prevailing traffic conditions in developing countries needs to be investigated, as design parameters in models can vary depending on local conditions. Vanderschuren (2007) further highlights that not enough research in South Africa has been conducted regarding driving behaviour for microscopic simulation model evaluation.

2.3 Congestion

Traffic congestion is a critical issue as it has deleterious effects on the environment: traffic flow movement, fuel waste and higher possibilities of accidents. Initially, freeways were constructed to allow free mobility to road users. However, the progressive escalation of car ownership has consistently increased traffic congestion, especially within and around the metropolitan areas. In numerous places around the world, the capacity of existing transport networks is incapable of meeting this demand, inevitably resulting in traffic congestion in urban areas and metropolitan regions in particular. It also negatively affects the key transit roads, potentially overfilling some public transport links as well as protracted queues at some airports. Over-capacitating of road networks results in prolonged journey times and declining travel reliability.

With the tremendous impact of traffic congestion on the mobility of commuters within metropolitan areas, solutions are neither straightforward nor unique. In trying to address this issue, the conventional techniques such as the construction of new roads are neither desirable nor possible. Then, traffic and transportation engineers opt for performance improvement of the already existing system to increase the road network efficiency. In alleviating traffic congestion, traffic control is considered one of the most essential tools, alongside maintenance and optimisation methods of a traffic control system (Stevanovic, 2006).

2.4 Traffic flow

Traffic comprises of a number of vehicles moving in a road network with each travelling at different and continually varying speeds. When saturation occurs on a segment of the road network, a vehicle's travelling speed diminishes as opposed to the speed it would be travelling in the absence of any congestion (Swartz, 2017). On highways, *traffic flow* is generally described as an element of space and time, but when it comes to signalised intersections, traffic flow functions are normally depicted regarding the arrivals and departures of vehicles. These get translated further in terms of arrival, stop and departure sequence. Traffic flow functions are regulated by controlling signalised intersections to create a safe environment for pedestrian and vehicular traffic; this requires establishing the appropriate traffic signal timing to reduce delays anticipated by commuters (Chaudhry, 2013).

Signalised intersection efficiency is evaluated by the ease of accumulation of queues during the red intervals which are dissipated during the green period. Mismanagement of signalised intersections results in prolonged delays for motorists and an increased number of stops encountered. Given that signalised intersections contribute to interrupted flow, efficiency determination is treated differently from that of uninterrupted traffic flow. Delay at intersections experienced by vehicle drivers is the main parameter that represents the quality of services provided by the signalised intersection, and these are recognised due to the traffic control at signalised intersections. The traffic flow upstream is cut off by the traffic signal control; thus queues begin to develop until the green signal appears which enables the movement of the queue (Chaudhry, 2013).

2.5 Traffic control

2.5.1 Intersections

An *intersection* is a space shared by at least two or more roads in which traffic flows are merging, separating and interlacing with each other to reach their desired destination. Traffic intersections are complex and challenging at most locations of the urban and suburban road traffic network. In a road network, an intersection is a determining factor for traffic safety status and operational performance of the transport system (Gurung, 2016). According to Eriskin *et al.* (2017), the smooth flow pattern of vehicular movement through intersections ought to be properly organised and controlled to ensure driver safety and well-being during commutes. The orientation of the traffic flows can be made using signalisation, whereby traffic signals are installed at intersections as a measure of managing and controlling the flow. Signalised intersections constitute an important component in the urban transportation network. It conveys substantial movement of pedestrians, motorised and non-motorised vehicles and these movements generate conflict at crossings in their turning and merging manoeuvres.

According to Ranjitkar *et al.* (2014), the design of intersections is a complex procedure where aspects such as operation efficiency, safety features and geometrical imperatives are taken into consideration.

Inadequately designed intersections may contribute to the ever-increasing dilemma of traffic congestion. The operational efficiency of the intersection relies on the predominant conditions of traffic control and road infrastructure. In order for intersections to control different types of traffic movements, they are distinctly designed contingent on various geometric parameters.

The design of intersections influences corridor capacity and safe movement of traffic flow in various directions. The way traffic moves at intersections and approaching traffic volume during peak hours determine lane width, auxiliary lane requirement and traffic flow channelisation as deemed necessary. Additionally, island shapes and arrangement of intersections differ depending on the type of intersection design (Villegas *et al.*, 2017). The urban road network is ultimately controlled by the functionality of major intersections. However, most intersections fail in accommodating the escalating number of vehicles during peak hours in Maribor, Republic of Slovenia (Dobovšek & Sever, 2005).

Intersections play a critical role in any road network, especially where there is an interaction of traffic flow from different directions in the system (Belay, 2015). Due to unsettling influences from mixed traffic, pedestrians and green time loss, intersections experience much lower capacities compared to approaching links. To maximize traffic flow operational efficiency and safety, traffic signals, intersections, roundabouts, stop and yield controls are employed as measures for dispersing conflicting movements on time. Thus, aspects such as capacity, degree of saturation, delay, length of queue and average speed are used to assess the intersection performance. The estimation of these performance measures is vital for planning, design and operational purposes. According to Ranjitkar *et al.* (2014), intersections are classified into three categories – un-signalised intersections, signalised intersections and roundabouts – which allow for appropriate solutions based on the relevant type of intersection. At each intersection category, movement control is carried out in various ways such as stop signs, traffic signals and roundabouts.

Signalised intersections

Villega *et al.* (2017) emphasise that, in urban area road networks, *signalised intersections* are where traffic congestion is most evident. This is due primarily to the fact that they collect traffic volumes from diverse tactics, which are then given the right of way by traffic signals to avoid conflict of movement. In the case of an inefficient and inadequately signalised intersection, the intersection will experience congestion compiled by safety-related issues. The control of turning traffic movements at signalised intersections is a critical encounter, especially when there is high turning demand.

Signalised intersections play an important role on highways, freeways and arterial roads. Therefore, there are various measures which have been incorporated for intersection analysis and simulation, which in general are able to quantify drivers' traversing experiences at the signalised intersection. The most popular measures include delay, average queue length and number of stops experienced by the vehicles on the road network. Liu *et al.* (2008) explain that when dealing with an intersection, some of the performance measures can be collected straight from the field or calculated from the existing traffic systems. For instance, vehicular measures such as volume and occupancy can be calculated from collected detector actuations, while traffic signal measures which encompass green-time, red-time, yellow-time and cycle-time can be obtained from traffic controllers.

Un-signalised intersections

Prasetijo and Ahmad (2012) describe *un-signalised intersections* as when two or more roads cross one another, and the movement is not controlled by traffic signals. Therefore, every intersection requires a form of control to ease mobility. In most cases, un-signalised intersections are the most common type of intersection, dealing with minimal volume traffic flow between major and minor streets. For un-signalised intersections, two-way stop-controlled (TWSC) and all-way stop-controlled (AWSC) are the typical operation options. Un-signalised intersections regulate low volumes of traffic flow.

Roundabouts

According to Eidmar and Hultman (2014), *roundabouts* are spaces in the traffic stream where the traffic circulating is given the right of way over incoming traffic. The principal aim of a roundabout design is to serve as a speed control measure of incoming vehicles without interfering with their movement. Vehicle entry speed at a roundabout is higher than that of four-way stop intersection entry speed because of the geometric design of the roundabout approach angle. Contrary to that, the roundabout entry capacity might be higher compared to right turn flow (equivalent to left turn flow in South Africa) at a four-way stop controlled intersection, while with regard to delay, roundabouts present minimal delay over a four-way stop controlled intersection (Eidmar & Hultman, 2014).

2.5.2 Traffic signal control system

According to Stevanovic (2006), the evolution of traffic control systems, together with the development of electronic technologies, has changed the manner of communication and control of traffic signals over the last few decades. These developments have enabled the optimisation of off-line traffic signals. Traffic signal control systems are classified into pre-timed, actuated and adaptive systems.

In pre-timed signalisation, pre-defined techniques re-occurring at a fixed interval define the way space is shared in conflicting flows. The fixed interval is therefore alluded to as cycle length, equivalent to the sum of the green, red and amber times at the intersection for a particular stream. The cycle length at signalised intersections encompasses movement allocation within a particular stream of the network, where green time denotes allowable movement for that stream. Red time indicates movement restriction of that particular stream at the intersection. During the transition from the green signal to the red signal, an amber signal comes up to warn the driver of the approaching red signal. Pre-timed signals are more or less the same as actuated signals since they are both time limit constrained. However, actuated signals are capable of changing phases prior to reaching their time limit, provided there is low demand at the signalised intersection. Actuated signals are subject to skip phases if such a phase is not in demand. Hence, these types of signals are more favourable at night or in rural areas, as these are low-demand settings (Jain, 2016).

Fixed-time signal control

According to Einhorn (2012), there are various techniques that can be utilised to determine the functionality of the traffic signal under fixed time control, such as allocating more green time to the major roads and minimum time to minor roads. In some cases, the British method is adopted, one which stipulates that green time be distributed in a manner that the ratios of volume to saturation flow is balanced on all approaches. Fixed timed signal control also attempts to minimise delay by proportionally distributing green time, considering the overall volume from all approaches.

Pre-timed signal control

Pre-timed traffic signal control systems use fixed sequence and often reiterate the traffic signal indication sequence. For this type of signal control, continual cycle length time, split length and sequence are mandatory to the system controllers. The optimisation of the traffic signal timing schedule is carried out based on traffic flow archived data. However, the pre-timed traffic signal system is capable of considering traffic flow variations during peak and off-peak hours. Once the stage of traffic signal timing is successfully completed, the remaining task for the traffic engineer will then be to fine-tune the traffic signal schedule as per traffic flow demand. This, on the other side, presents a disadvantage because pre-timed signal control is dependent on the availability of resources. In the case where the necessary resources are available for the pre-timed traffic signal control, then in comparison to the actuated traffic signal control, they are affordable regarding purchase, installation and maintenance. Pre-timed signals are most appropriate for signalised intersections with possible steady traffic flow and are beneficial in situations where progression is essential, like in consecutively located intersections (Stevanovic, 2006).

Actuated signal control

As mentioned by Stevanovic (2006) and Alemayehu (2015), actuated traffic signal control is different from the pre-timed signal control in that their signal timing indication is dependent on the traffic flow variation, while pre-timed operate on fixed signal timing durations. The installation procedure of the actuated signal control system is categorised into four major constituents: the unit controller, detectors, traffic lights (traffic signal heads) and the communication system. Stevanovic (2006) further indicates that actuated traffic signal control systems are most applicable where there is a need to abate traffic flow interruptions from various sides of the networks experiencing highest traffic demand. They are classified into two main groups: semi-actuated and fully actuated control systems. In semi-actuated control systems, the green time indication is given to minor streams only if there is a vehicle detected; thus they are more suitable for minor streams with light traffic volumes. At fully actuated traffic control systems, the traffic signal can sense vehicles approaching at the network, thereby accommodating and serving traffic signal phases relevant to the approach demand. These two main groups can either be coordinated or uncoordinated, whereby the traffic signal control system is capable, from the already existing library, of identifying the relevant or appropriate traffic signal plan based on recent traffic flow conditions observed.

Adaptive signal control

In Adaptive Traffic Control Systems (ATCS), traffic signal timing efficiency is improved online, where the traffic flow deviation influences the way the signal timings are adjusted. The real-time traffic flow data, such as traffic volume and occupancy in ATCS, is measured with the aid of the stop line or upstream detectors. Depending on predicted or measured data, the optimisation of traffic signal timing in ATCS assists with the traffic congestion alleviation and the reduction of car emissions (Jiao *et al.*, 2015). There are several widely deployed ATCS, such as the Sydney Coordinated Adaptive Traffic System (SCATS), which was developed in Australia and is used mostly in North America and the Split Cycle Offset Optimisation Tool (SCOOT) established in the United Kingdom (UK) by the Transport Research Lab. Then there is the Real-time Hierarchical Optimised Distributed and Effective System (RHODES) whose prototype was developed in the United States (US) at the University of Arizona, as well as the Optimised Policies for

Adaptive Control Strategy (OPAC) which was established jointly by the University of Massachusetts at Lowell in the US and Parsons Brinkerhoff Farradyne Inc (Stevanovic, 2006).

2.5.3 Traffic volume characteristics

For intersection management, traffic attributes employed in the evaluation of the respective intersections are pivotal, as any miscalculation or inappropriate assessment might lead to constraints of funds and result in unnecessary construction. On the other hand, inadequate consideration of conditions such as daily peak hours and peak recreational seasons can lead to ineffectual facilities (Rodegerdts *et al.*, 2004).

To establish a pertinent traffic profile, the traffic engineer should be able to differentiate traffic volume and traffic demand. In relation to an intersection, traffic volume typically demonstrates vehicle departure rates, while vehicle arrival patterns signify traffic demand at that intersection. The traffic demand approach is more appropriate when dealing with over capacitated or saturated conditions, as it is capable of replicating the true demand at the intersection contrary to the traffic volume. The vehicle arrival and departure difference, which identifies the number of vehicles not being served by the traffic signal, is the volume that needs to be considered during traffic operation evaluations. Nonetheless, traffic volume in some instances might be much less than traffic demand as a result of an oversaturated situation at the upstream or downstream traffic signal. In such an event, micro-simulation analysis tools are more appropriate to employ for the evaluation process of the respective intersection (Rodegerdts *et al.*, 2004; Alemayehu, 2015).

2.5.4 Intersection geometry

According to the HCM (2000), the intersection geometry is demonstrated graphically, a graphical representation which should encompass all necessary information such as intersection approach grades, number of lanes, lane width and the parking situation. In the case where the intersection has an exclusive right turn or left turn lanes, such lanes ought to be noted together with their storage lengths. As expressed by Rodegerdts *et al.* (2004), the traffic demand or volume that an individual intersection can handle is dependent on geometric features of the intersection. When establishing the intersection supply, the saturation flow rate is a critical aspect to consider. This is also comparable to capacity in that it expresses the number of vehicles passing at a particular point per hour. Nonetheless, saturation flow is computed based upon an assumption that the traffic signal gets a green phase for a straight hour. Then from the computed saturation flow rate, the capacity can also be computed by the product of saturation flow times and the green time-to-cycle length ratio.

2.5.5 Signal timing

Traffic signal timing affects the intersection and how the intersection performs under prevailing traffic conditions. Vital factors of signal timing include the following.

Effective green time

Generally, the effective green time phase is computed based on the signal phase volume percentage of the critical lane relative to the intersection total critical volume. It denotes the amount of available time to serve the movement of the vehicles at the intersection during the cycle phase and is equivalent to the green time shown, less the start-up loss time, and adding the end gain time. In the case where excessive green time is offered, the traffic signal cycle will not be completely put to use, resulting in driver frustration; therefore, the traffic signal will not effectively be utilised. In contrast, if minimum green time is given, queuing vehicles at the intersection might not all be cleared, thereby resulting in cycle failure (Alemayehu, 2015).

Clearance interval

The clearance interval, illustrating the required time for vehicles to clear the intersection safely, encompasses the clearance interim of the amber (yellow) phase together with the red phases (Rodegerdts *et al.*, 2004).

Loss – time

Loss time signifies the section of the signal phase that is not being used by vehicles at the intersection. In a period of one signal phase, the loss time happens twice, that is, the time the vehicles accelerate after being given the green time and the time a vehicle, in anticipation of the red phase, starts to decelerate. The capacity reduction is due to prolonged loss time, which is also a result of a decrease in inadequate green time at the signalised intersection. The geometric design of the intersection also influences the amount of loss time; intersections with skewed approaches or those that are wide in design are susceptible to higher loss time as compared to traditional intersections (Rodegerdts *et al.*, 2004; Alemayehu, 2015).

2.5.6 Cycle length

Cycle length is usually adjudicated in a time frame of one hour and on how often a respective movement is catered for. The length of the cycle affects the efficiency of the green time at an intersection; hence short cycle lengths create cycle failures while increasing delays and queue lengths. In a situation where the traffic signal system is coordinated or pre-timed, the cycle length is fixed, meaning it is not controlled by the demand at the intersections (Rodegerdts *et al.*, 2004).

2.5.7 Progression

Progression is defined as the movement of a group of vehicles (platoon) from one intersection to another. A well-coordinated traffic signal system is one which allows vehicles from the upstream intersection to move in a platoon format to reach the intersection downstream during the green time. At the point when this happens, fewer vehicles reach the intersection during the red signal phase, which then results in queue length and delays being reduced. Nonetheless, the opposite is evident: in a case of an inadequately coordinated traffic signal system, platoons arrive at the downstream intersection during the red phase, increasing delay and queues, more than when vehicles arrive at the intersection randomly (Alemayehu, 2015).

2.5.8 Performance analysis

Capacity, queue lengths, level of services and average delays are some of the elements of performance analysis used to determine the general performance of the signalised intersection. The performance results will demonstrate how well or poorly the intersection can control the traffic flow within the network. Performance measurement is an effective tool for traffic engineers to assist in monitoring road network traffic operations from single movements at a signalised intersection up to the whole traffic network. These methods can assess the effectiveness of operational strategies which are implemented as well as present indicators which assist with the evaluation of the system performance. Performance measures can establish traffic network problems to provide decision makers with vital information for improving transportation services (Lui *et al.,* 2008). When carrying out intersection analysis, the most employed measures of effectiveness are the average control delay and the 95th percentile queue. Average control delay is used to evaluate the road network level of service, while the 95th percentile queue length represents the storage capacity of the network as well as spill-over, provided there are other intersections in close proximity to the subject intersection (Ahiamadi, 2013).

Zheng *et al.* (2013) explain that the quality of the road network movement is associated with the performance of the signalised intersection. In this regard, there are different traffic parameters that can evaluate signalised intersection performance. In relation to the intersection performance measurement, the HCM (2000) uses the average control delay, depicted as a fraction of overall delay, which sums up signal operation at intersections. The average control delay also defines the performance of intersections with reference to volume-to-capacity and level of service.

According to Lui *et al.* (2008), performance measures on the transportation system constitute a significant source of information with regard to decisions associated with infrastructure resource allocation, project evaluation and monitoring of investment plans. The introduction of ITS within the transport system has enhanced the importance of generating accurate and timely performance measures which can either be used for traffic management strategy optimisation or provide information to commuters with respect to travel paths. Additionally, the challenge for transportation systems has been side-tracked from basic infrastructure development to the management of the current transportation resources and providing motorists on the road better services under different conditions.

Capacity

Cvitanič *et al.* (2007) express that the selection of cross-section elements and the design of the highway depends on the capacity analysis of each road segment, including highway sections and intersections. They further explain that the purpose of the evaluation of capacity is to see to it that the planned highway network can handle the current and future traffic flows adequately in terms of the LOS. The degree of saturation, referred to as the volume-to-capacity (v/c) ratio, signifies the adequacy of an intersection to handle the vehicular demand. Volume-to-capacity under 0.85 demonstrates that the road network adequately manages the capacity and there are no anticipated delays and long queues to frustrate motorists. As the volume-to-capacity ratio slowly approaches 1.0, the movement in the network slows down, queuing develops and delay come. At the point where demand is greater than capacity (that is a volume-to-capacity more than 1.0), traffic flow becomes unstable.

According to Chaudhry and Ranjitkar (2009) and Akçelik (1981), for capacity calculations, saturation flow rate is a critical aspect which is usually adjusted for the current traffic conditions such as lane width, grades, right and left turns, heavy vehicles occupancy, parking arrangements, right turn and public transport blockage. Therefore, the saturation flow rate is expressed as the highest steady traffic flow rate of the queue during the green phase period at the signalised intersection. Lost time is also an integral part of the concept of saturation flow.

Saturation flow and saturation flow rate are the key influences for the calculation of capacity at signalised intersections (Hussein, 2016). Bennett *et al.* (2009) define *saturation flow* rate as, "the maximum flow rate that can pass through a given traffic movement or intersection approach under the prevailing roadway and traffic conditions, expressed in vehicles per unit time, normally vehicles per hour" (pp 74). The elementary capacity model for a typical traffic signal, presented in Figure 2.1, shows that as the traffic signal switches to green light, the traffic flow increases gradually as the vehicles pass through the stop line until it matches the saturation flow. Unless the queue capacity is reduced or the green time duration expires, the saturation is at a constant flow rate.

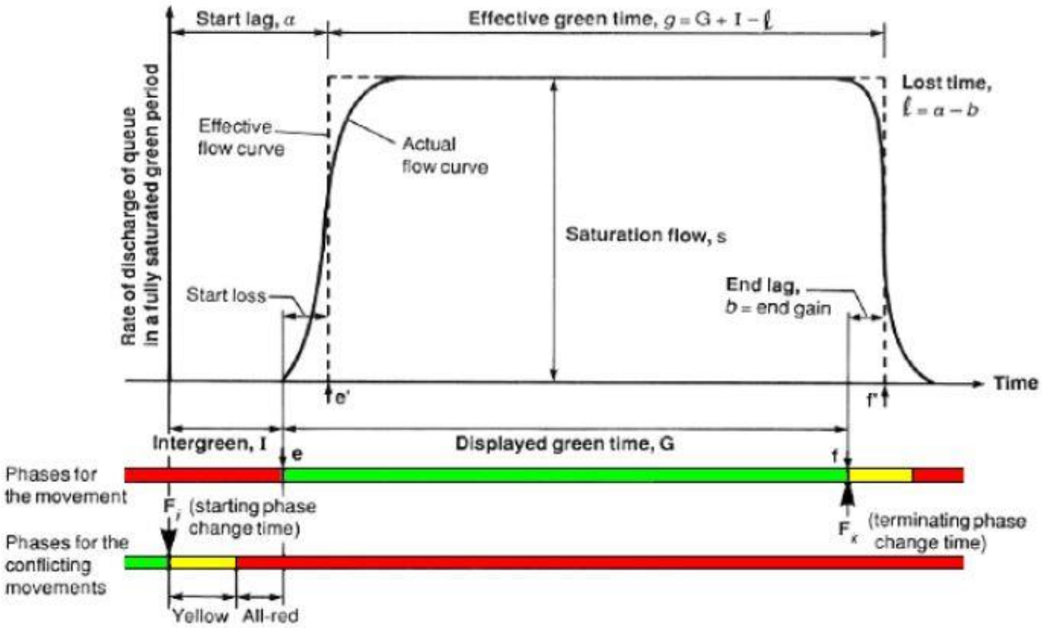


Figure 2.1 Basic traffic signal capacity model (Akçelik, 1981)

At signalised intersections, capacity is conceptualised from both saturation flow and saturation flow rate. For a particular lane group, the projected or actual demand flow rate for that particular lane group (v_i) and saturation flow rate (s_i) ratio defines the lane group's flow ratio. Therefore, the flow rate is denoted as $(v/s)_i$. Capacity is then given by Equation 2.1 (HCM, 2000).

$$c_i = s_i \frac{g_i}{C} \tag{2.1}$$

Where;

c_i	=	capacity of lane group I (veh/h)
s_i	=	saturation flow rate for lane group i (veh/h)
g_i/C	=	effective green ratio for lane group i
g_i	=	effective green time for lane group or approach i
C	=	cycle length in seconds

Delay

At signalised intersections, delay is deemed a critical parameter when evaluating the level of service (LOS) of the facility and for the optimisation of traffic signal timing. Delay estimation at signalised intersections contributes significantly to the analysis of the various transport modes' travel time and performance. Furthermore, real-time delay computation contributes extensively to signalised intersection design and evaluation applications. For instance, to measure the traffic signal operation parameters at isolated and coordinated intersections, the optimisation of delay is crucial as it is used as the primary optimisation criteria. Additionally, in the Highway Capacity Manual (HCM), average control delay is utilised to establish the LOS of signalised intersections (Ghasemlou *et al.*, 2015).

Ghasemlou *et al.* (2015) further state that delay at intersections can be computed in numerous ways, either by site observation or using analytical models. Based on entry or exiting properties of traffic flows, analytical models define delay time as time-dependent, deterministic and steady state. Regarding delay computation at signalised intersections under different traffic flow conditions (oversaturated and under-saturated), deterministic and steady-state models are typically used. However, when the saturated degree is equivalent to 1, deterministic models are capable of calculating absolute delay, whereas in steady-state models, the absolute delay is calculated when the saturated degree is 0. Figure 2.2 illustrates the typical definition of delay at the signalised intersection.

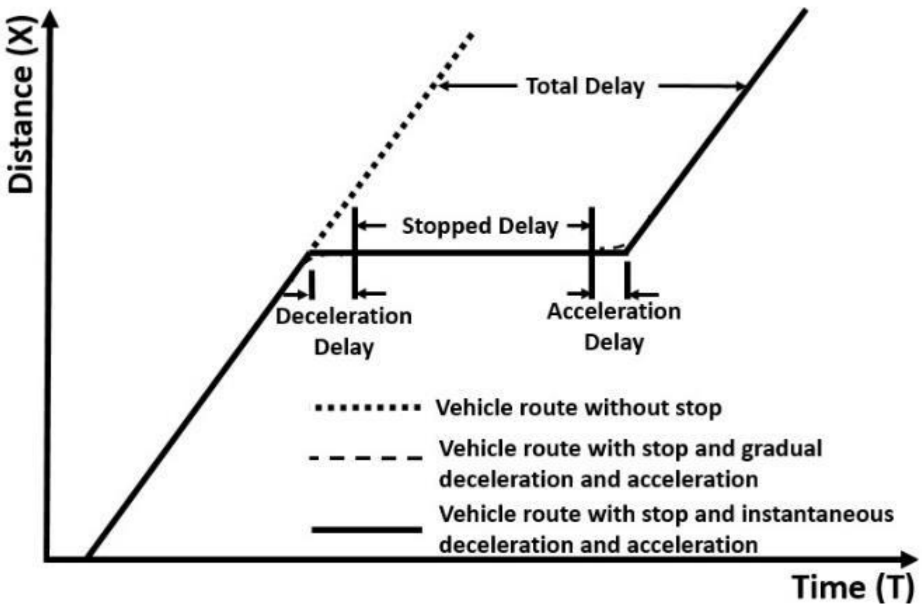


Figure 2.2 Definition of total, stopped, deceleration and acceleration delays (Ghasemlou et al., 2015)

The vehicular delay as demonstrated in Figure 2.2 may be classified into three groups: deceleration, stopped and acceleration. *Deceleration* is defined as time passed for a vehicle to slow down from its running speed in order to stop at the intersection. *Acceleration* is defined as the time elapsed until the vehicle is in motion. *Stopped delay* can be interpreted as the time during which the speed the vehicle is moving is lower than that of the pedestrian average speed (1.2 m/s).

Level of service

There are numerous procedures for computing LOS at signalised intersections, including Highway Capacity Manual method, Webster's method and Normann method, for example. The HCM (2000) contains the computational procedure for capacity and LOS analysis for signalised intersections. The analysis takes into consideration a wide range of existing conditions, including traffic composition, allocation of traffic movements, geometric characteristics and details of the intersection signalisation. The LOS is therefore evaluated in reference to control delay per vehicle (in seconds/vehicle). Control delay is attributed to traffic signal operation for signalised intersections (Kumar & Ranjitha, 2013).

The LOS for intersections ranges from A, which demonstrates the free flow of traffic or excellent traffic conditions, to F, which indicates highly congested conditions with tremendously long delays. Urban arterials are characterised by platoon flows or traffic flow operations where vehicles have a tendency to be clustered together. The operational quality is measured primarily by the efficiency of the traffic signal coordination and how the individual intersections along the arterial operate (HCM, 2000). A categorisation shown in Table 2.1. Hussein (2016) states that the operational performance of the traffic flow is generally signified by the concept of level of service, through factors such as travel time, speed, traffic interruptions, ease of movement and comfort.

Table 2.1 HCM (2000) categorisation of level of service (LOS)

LOS	Average Delay per Vehicle
A	Very low control delay <10 seconds per vehicle; progression is very favourable; most vehicles arrive during the green signal; most vehicle do not stop. Short cycle lengths might also contribute to minimum delay
B	Control delay 10< 20 seconds per vehicle; progression is good and/or cycle lengths are a bit shorter. More vehicles stop as compared to LOS A, causing higher levels of average delay.
C	Control delay 20<35 seconds per vehicle; progression is fair and/or cycle lengths are a bit longer. Individual cycle failure may begin to appear at this level. The number of vehicles stopping is significant, although many vehicles still pass through without stopping.
D	Control delay 35<55 seconds per vehicle; progression is unfavourable, cycle lengths are long, or has a high flow rate to capacity ratio. Many vehicles stop, and proportion of vehicles not stopping diminishes. Individual cycle failures are visible.
E	Control delay 55< 80 seconds per vehicle; progression is poor, cycle lengths are long, high flow rate to capacity ration is experienced. Individual cycle failures are frequently occurring.
F	Delay 80< seconds per vehicle; progression is very poor, cycle lengths are long. Many individual cycle failures. Arrival flow rates exceed intersection capacity. This level is considered unacceptable for most drivers.

Unlike the computation of individual signalised intersections, urban streets' LOS computation is quite different, based on the average travel speed of through-vehicles for a particular segment or the holistic urban street under evaluation. According to the HCM (2000), urban streets' LOS is influenced by two aspects – intersection control delay and the number of traffic signals present per kilometre. For urban streets, average travel speed, the critical aspect in determining the operational performance, is calculated from the average control delay of the through movements at the signalised intersections and average travel times (running time) in the segment or the entire urban street. The criteria adopted by the HCM (2000) for the classification of urban street LOS is detailed in Table 2.2, based on the urban street average travel speed and class.

Table 2.2 Urban street level of service (HCM, 2000)

Urban Street Class	I	II	III	IV
Range of free-flow speed (FFS)	90 to 70 km/h	70 to 55 km/h	55 to 50 km/h	50 to 40 km/h
Typical FFS	80 km/h	65 km/h	55 km/h	45km/h
LOS	Average Travel Speed (km/h)			
A	> 72	> 59	> 50	> 41
B	56-72	46-59	39-50	32-41
C	40-56	33-46	28-39	23-32
D	32-40	26-33	22-28	18-23
E	26-32	21-26	17-22	14-18
F	≤ 26	≤ 21	≤ 17	≤ 14

2.6 Topology of transport models

- *Static transport models* – For this type of transport model, the process representation is momentary, as these models do not integrate the time concept. Events are defined according to how often they occur, thus enabling the model system to show possible stationary condition assumptions in a specific time. The acquisition of changes in time-of-day are obtained through independent models for a given time period in the network (for example, different peak hours represented by one model individually as well as off-peak hours model) (Flügel *et al.*, 2014).

- *Dynamic transport models* – These models consider the concept of time and time-related processes with the process in the model. The dynamic transport models also make allowance for time-dependency of vehicles embarking on their Origin and Destination (OD) pairs on respective routes, as well as the modelling of time-dependent development due to congestion and delay (Flügel *et al.*, 2014).

2.7 Classification of traffic models

2.7.1 Application orientation

Based on application orientation, Reza (2013) claims that simulation models are categorised as transportation planning and design, traffic operation or transportation safety models. Transportation planning models allow town planners to investigate different patterns of urban development and to gather and provide data of the land use, population and employment hierarchy which is then used in travel and transportation demand assessments. In transportation planning, the essential interest is the demand estimation. The following are a few examples of those models: TRANPLN, TRANSCAD and TRANSIMS. There are various applications of traffic operation models as well:

- Isolated intersections: SOAP, SIDRA, SIGNAL;
- Urban Street Networks: SYNCHRO, TRANSYT-7F, PASSER IV;
- Arterials and Highways: PASSER II, PASSER III;
- Integrated Networks: VISSIM, DYNEMO, CORSIM; and
- Freeway and Freeways Corridors: INTEGRATION.

2.7.2 Uncertainty content

Uncertainty of content, a commonly used method of categorizing simulation models, demonstrates the stochastic or deterministic nature of the simulation models as well as time representation of the simulation model's static or dynamic characteristics. When none of the model components is conditional to uncertainty, then such model is defined as a deterministic model. When the opposite happens, the model is defined as stochastic (Reza, 2013).

2.7.3 System update

In order for the model to be considered continuous, the traffic system dates must correlate with the network travel times. The model will be denoted as discrete, provided an update of the traffic system is not running at fixed time intervals. Discrete models are categorised into two groups: discrete time models and discrete event models. In discrete time models, the recalculation of the traffic system condition and traffic system elements is dependent on a fixed time interval, whereas in discrete event models, only significant events contributing to the traffic operation will necessitate that the traffic system be updated. For instance, every time the traffic signal phase changes at signalised intersections, the traffic condition will then be updated (Reza, 2013).

2.7.4 Level of aggregation

According to Reza (2013), traffic simulation models in relation to the level of aggregation can be categorised three different ways: low fidelity (microscopic simulation models), mixed fidelity (mesoscopic simulation models) and high fidelity (macroscopic simulation models). Traffic flow in macroscopic models is modelled using the continuity equation. The equation demonstrates speed, density and flow-rate relationship. The microscopic simulation model, to show detailed traffic operation and driver or vehicle behaviour, utilises the car following and the lane-changing theories. These models include different analytical methods such as queuing analysis and shock wave analysis. Mesoscopic simulation models, when compared to microscopic simulation models, showcase traffic flow in great detail, but the same cannot be said for the description of the traffic flow interactions and activities, as the level of detailing in those is at a very low level. Thus, only a few simulation models fall into the mesoscopic simulation models category. From the traffic demand perspective, simulation models can be further categorised into flow-based traffic simulation models (SimTraffic, CORSIM) or as network or corridor traffic simulation models (VISSIM, PARAMICS, AIMSUN).

For flow-based models, traffic simulation under this category is fundamentally structured to replicate link performance. Also, for traffic input data, flow-based models use vehicle turning percentages and vehicle entry volumes. Therefore, as per the permitted turning probabilities, vehicles are led to the link downstream the moment they enter the traffic network. In contrast, lane-based models focus on replicating network trip generation behaviour. Here, input traffic flow demand is demonstrated through Origin and Destination (OD) matrices. Network or corridor models also produce traffic assignment by employing particular algorithms for routing as part of decreasing total travelling cost, or a certain discrepancy relatable to total travelling cost (Reza, 2013).

2.8 Traffic analysis tools

Traffic analysis tools aid traffic engineers and transportation planners to analyse transportation networks for both present and future traffic conditions. They contribute to the decision-making process that leads to transportation solutions. Traffic analysis tools alone however do not single handedly contribute in the decision-making process, but they assist transportation and traffic engineers to understand and evaluate alternatives. Due to the possibility of the increase in the complexity of improvement concepts, the importance on the choice of the suitable traffic analysis tool for each traffic condition cannot be over emphasised (Alexiadis *et al.*, 2004).

2.8.1 Travel demand forecast models

According to Flügel *et al.* (2014), the road network mobility demand of individual commuters is estimated through travel demand models. The basic strategic congestion-sensitive travel demand model estimates the movement of commuters mainly stratified by socio-economic groups, utilising different modes of transport such as private vehicles, public transport and trains within a specific study area, taking into consideration imminent inter-zonal movements such as travel time and cost. These models interpret the travelling decisions concluded by commuters, their destinations and the transport modal choice, which results in the OD matrices.

To predict future travel demand regarding transportation modal choice, trip destination and route choice, the travel demand models assess the existing number of commuters using different modes of transport together with employment and population prediction. Travel demand models are not capable of evaluating or analysing operational changes or travel management strategies. However, the principal intent of travel demand models is to evaluate the impact of major highway projects on the transportation system. The Urban Transportation Modelling System (UTMS), also referred to as the four-step planning model, is a technique used in conventional travel demand models. This technique necessitates the prediction of the number of trips originating and imminent to a particular zone (called trip generation). The origin-destination matrices linking origins and destinations (trip distribution) are then developed establishing the percentage of commuters which will be facilitated by the available mode of transport (modal split), and allocating every trip generated to its respective route (trip assignment). Despite the use of the four-step planning model technique worldwide, because of its inadequacies in relation to the travel behaviour coherent theory, travel demand models have resulted in modelling inconsistencies (Turley, 2007).

2.8.2 Traffic analytical models

As stated by Alexiadis *et al.* (2004), analytical traffic models are generally used at intersections for the estimation of capacity and for performance measures such as delay and queue length. This is due to confinements of these models towards network analysis proficiency as they are more appropriate for minor parts of the road network such as intersections (both signalised and un-signalised), roundabouts or even road segments. Ekman (2013) contends that to analyse the condition of a network system, analytical traffic models employ mathematical computations. These models also incorporate minor random elements, although in the case of a deterministic model, the same input data will inevitably produce the same output data. A substantial number of these analytical models depend on queue-method models and gap time models, whereby these models classify roads into major or minor based on the traffic rules and regulations of the respective intersections under study. Incoming vehicles approaching the intersection from a minor road are not regarded as part of the actual traffic flow but interpreted as queuing traffic instead (Eidmar & Hultman, 2014).

Highway Capacity Manual

The Highway Capacity Manual (HCM) was established by the Transportation Research Board in the United States (US). Five versions of the tool have been published, with HCM (2010) the most recent. The manual is a widely used document for reference and as a benchmark for developing specific manuals about local transportation purposes. The HCM entails guidelines and systematic procedures for calculating different MoE in various aspects of the transportation system such as highways, freeways, roundabouts, signalised and un-signalised intersections and corridors. The impact of public and private transportation, pedestrians and non-motorised vehicles on the performance of the road network is also addressed in this manual.

Capcal

Ekman (2013) states that Capcal, software developed in Sweden, easily and instantly computes capacity and performance of signalised and un-signalised intersections as well as roundabouts using the critical elements of time gap and saturation. Ekman (2013) explains that Capcal software is not drastically different from SIDRA in terms of its operation. The required input data and output data are straightforward for the first-time user to easily comprehend.

Transport Research Laboratory (TRL)

According to Ekman (2013), the Transport Research Laboratory (TRL) in the United Kingdom (UK) developed an empirical model (meaning the model is non-traffic theory-based) that utilises a linear regression model for capacity estimation. This model neglects to consider driver behaviour while estimating the capacity of the roundabout, but only factors in the geometric values of the road network. In comparison to other more complex models, the TRL model poses some restrictions, as capacity underestimation on road networks with less traffic flow is not easily avoided with the best fit and linearity technique used by the model. The additional disadvantage of the model is the difficult in identifying saturated and oversaturated traffic conditions.

Signalised and (Un-signalised) Intersection Design and Research Aid (SIDRA)

Ranjitkar *et al.* (2014) describe SIDRA (Signalised and Un-signalised Intersection Design and Research Aid) as micro-analytical software, commonly utilised for lane analysis of various intersection types in traffic engineering. The software uses other traffic models combined with a repetitive approximation process for providing MoE estimates such as the intersection capacity, total delay, length of queues and car emission levels. The intersection operational efficiency is conducted using this software as it has demonstrated competence and yielded feasible results which are practical for implementation. According to Yumlu (1995), the software allows the user to identify priority movements at signalised intersections and gap acceptance parameters such as headway and critical gap while running the software.

Fatima (2015) insists that SIDRA is based on an advanced micro-analytical tool for evaluation and design of independent intersections and network of intersections, together with separate class movement of lane-modelling such as heavy and light vehicles, bicycles, buses and light rail. The modelling software also supports capacity and an extensive range of measures of performance estimates such as delay, queue length, vehicular and pedestrian stops, fuel consumption, emission of pollutants, operating costs and LOS. In the study conducted by Yumlu (1995), SIDRA was calibrated for South African traffic conditions by field data from Johannesburg and Durban. For the validation process, the calibrated model results were compared with real-time intersection data in Pretoria. The observed results indicated that the saturation flow adjustment factor sensitivity, delay parameters and turning movement parameters are high. It was, therefore, suggested that the SIDRA recalibration of sensitive parameters is an important issue for the future the model to accurately represent the traffic conditions in South Africa.

As stated by Alemayehu (2015), because the SIDRA intersection software was established based on Australian traffic conditions as the default settings of the software, for its applicability in other countries with varying traffic conditions, the calibration of the software is mandatory. This SIDRA calibration can be carried out in various ways; the process is conducted by means of changing capacity-influencing parameter values, achieved in several ways such as directly altering follow-up headway and critical gap values or by modifying calibration parameters – environmental influences and entry or circulating flow – depending on the type of intersection being analysed. With respect to the influence of the environment, this encompasses the surrounding environment at the intersection, for example, intersection design, visibility, intersection grade, speed, driver aggressiveness and time of reaction, heavy vehicles count, pedestrian movement and parking bays near the intersection.

In the SIDRA Intersection User Guide, according to Ekman (2013), the software is depicted as an improved micro-analytical model. SIDRA consists of a lane-by-lane evaluation technique together with a vehicle drive-cycle model utilised in capacity and performance measure estimation by means of iterations. The model's adaptability allows for analysis of both merging and uninterrupted flow conditions. Moreover, SIDRA is capable of fuel consumption and operating cost estimations of the evaluated network or traffic system. Variables dependent on all traffic in a lane group in SIDRA, as explained by Yumlu (1995), are specified as saturation flow adjustment factors of that lane group, with adjustment factors such as lane width, lane gradient, parking space and bus factors. Contrary to saturation flow adjustment factors, individual vehicle class related factors, such as heavy vehicles and turning vehicles, are termed as car equivalents that are in conformity with specific lane group arrangements and therefore changed to traffic composition factors.

2.8.3 Simulation models

Simulation processes help in the representation of complex real-time systems and from that, numerous alternatives can be compared through different system designs (Lee, 2008). Tianzi *et al.* (2013) state that for proper control and evaluation of signalised intersections, simulation software is an effective tool, though this at times becomes a challenging and time-consuming process, as all software has its own merits. Due to the ability of simulation models to present statistical measures of effectiveness and the fact that they have the capacity to evaluate complex transportation systems which demand a large amount of computation power, they are gaining greater recognition, especially in the transportation and traffic engineering industry. In the interest of conducting vital analytical tasks, such as transportation planning and monitoring, surveillance of the traffic flow, incident detection, environmental impact, energy consumption and vehicular guidance systems, traffic flow behaviour mathematical modelling is mandatory (Reza, 2013). Traffic simulation models can be used in various ways:

- testing newly developed designs through the evaluation of the impact of vast geometric designs on the road network before any construction can be implemented;
- training traffic management centre operators in a real-time laboratory context;
- analysing traffic calming measures and incident impact (generally on an area-wide basis);
- conducting traffic impact assessment studies;
- investigating transit priority schedule and the impact of the transit on travel delay;
- evaluating road safety strategies before implementation; and
- measuring energy preserving techniques and emission modelling.

Consequently, traffic simulation models can be opted for in situations where:

- conventional analytical models may not be deemed applicable;
- a road network congestion situation continues over a given period of time; and
- visual demonstration is needed to provide a good understanding of the present system performance with the objective of justifying observed statistical results.

According to Mishra (2016), simulation can contribute immensely in ascertaining the suitability and efficiency of any proposed strategy. Simulation is described as an imitation of the real-time system for evaluating the system's progression over time. Simulation has, throughout the years, become an important tool for transportation and traffic engineers for modelling road networks, policies and traffic flow, as well as for the analysis of numerous new developments introduced by these models. The effectiveness of developed strategies can be demonstrated in a simulation platform first, for observing the applicability of strategies before real-life implementation. This platform also allows for different designs to be compared with one another to determine the best fit solution for the task at hand.

The incorporation of simulation models for the assessment of traffic operations and traffic systems is gaining recognition amongst traffic engineers and transportation planners, possible attributable to its better documentation. They also have become more intuitive and easier to use because of rapid developments in computing power and programming skills. Simulation models offer several advantages over conventional traffic analysis tools simply because they can generate a complete set of results for an entire subject study area. These models enable valuable visualisation of the real-time performance of a facility, thereby acting as a preliminary form of validation, verifiable in a virtual environment prior to field application. This is advantageous where operational and geometric changes would be costly (Jobanputra, 2013; Chaudhry & Ranjitkar, 2009).

Shah *et al.* (2016) state that traffic simulation can be explained as mathematical modelling of transportation systems through computer software to assist in the planning, design and operation of the entire transportation system. Simulation is essential as it intervenes in situations where analytical or numerical models fail to adequately assess road networks. Simulation models can provide a detailed study of the road network or facility being investigated, which might not be presented when using analytical or numerical models. They also have an advantage over analytical or numerical models as they can visually demonstrate the present and future scenarios. As explained by Jobanputra and Vanderschuren (2012) and Milam and Choa (2000), the technique of studying traffic operations and system impact with the aid of traffic simulation models has gained recognition in recent years from the issue of urban in-migration and densification of cities. The recommendation is that simulation models present distinctive advantages over conventional traffic analysis tools in that they are capable of producing inclusive results of the study location along with online visualisation which is a significant preliminary method of visual validation.

Traffic simulation models are well-recognised tools for the evaluation of planned control measures, either being related to the improvement of the infrastructure system or new and advanced equipment for motorists such as intelligent cruise control. Based on the extent of the investigation, distinctive aspects are significant with regard to infrastructure and vehicular modelling. When dealing with large networks, macroscopic simulation models are a viable choice, while for evaluating and analysis of smaller networks in great detail, microscopic simulation models are best. When dealing with the evaluation of intelligent transport systems that impact individual driver behaviour, microscopic simulation models have gained popularity due to their advanced computing capabilities (Fellendorf & Vortisch, 2010). According to Otković *et al.* (2013), simulation outcomes rely upon the model choice and proper calibration procedures. For traffic networks, a competent model is an important tool for structuring control measures of the network under evaluation as the model is then used to simulate the traffic behaviour of the network, with a view to assess impact on traffic flow (Pinna, 2007). According to Yang *et al.* (2016), there are three categories of traffic simulation models: microscopic, macroscopic, mesoscopic and nano simulation models.

The suitability and effectiveness of a simulation model in analysing various traffic flow situations depends on its aptitude to represent driver, road network and infrastructure specifics in real-time. To achieve this, the simulation model must be calibrated; the process of calibration acts on the variance between default model setting assumptions and real-time traffic conditions. Calibration procedures regulate the extent of model default parameters, designating the fundamental mechanics of the model to be fine-tuned or modified by the user so that the model can closely represent real-time traffic conditions (Maheshwary *et al.*, 2016). Traffic stream behaviour complexity and challenges in performing real-world traffic experiments elevate the simulation model to an essential traffic engineering analysis tool. The physical propagation of traffic flows can be distinctly depicted using traffic flow models, allowing the traffic engineer or the analyst to model large scale real-world conditions in detail (Tettamanti, 2015). Simulation models, also helpful for identifying the duration of the observed conditions, are capable of elucidating capacity and delay impacts in the network system (Borsari, 2012).

For road networks, traffic simulation can be executed on a microscopic and macroscopic level. At a microscopic level, traffic flow description at a high-resolution is demonstrated through vigorous individual vehicle behaviour, while at the macroscopic level, the model parameters (such as flow, density and average traffic travel times) perceive traffic as continuous. Macroscopic simulation models are often used for transient or large spatial domains due to their computational demand advantage over microscopic simulation models. This is because, at the microscopic level, they deal with the behaviour of traffic at small road networks with few intersections. For both simulation models, a thorough understanding of the primary parameters (networks traffic origin and destination data and traffic demands) is important for a valid traffic simulation design, since in most cases, the availability of such data is minimal; therefore, intelligent calibration is advised (Tettamanti, 2015).

Macroscopic simulation models

In macroscopic simulation models, the relationship between flow and density control, the vehicular movement and the simulation process takes a different approach as simulations are executed in sections rather than through tracking of individual vehicles (Owen *et al.*, 2000). According to Reza (2013), numerous macroscopic traffic simulation software packages have been developed by software companies and research groups. Most of these have been developed solely for research while others are meant for solving real-time traffic engineering problems. Reza (2013) further states that macroscopic simulation models gained more recognition after the publication of “The SMARTEST Project”. Its main goal was the evaluation of the existing micro-simulation models, to differentiate their benefits and limitations, in order to elevate the competence of the models. Commercial simulation models compared to research models are unique with regard to product development which signifies their demand in the market. The rapid evolution of these models has resulted in efficient tools capable of alleviating and mitigating vast and significant transportation issues.

Macroscopic simulation models do not have the analysis capability for transportation upgrades in a more detailed manner as compared to microscopic simulation models (Aljamal, 2017). One prominent macroscopic simulation model, Equilibre Multimodal Multimodal Equilibrium (EMME), is used by traffic engineers and transport planners in more than 85 countries, including South Africa, Australia, Canada, Central and South America, across Asia and in most European countries (Hildebrand & Hörtin, 2014). Moodley (2016) points out that macroscopic simulation models such as EMME attempt to replicate the interaction between land use and transport. Macroscopic simulation models, used for transportation planning of metropolitans and human activity locations such as living, education, working areas, determine spatial interactions or trip generation in the transport system. EMME is briefly explained in the next section as one of the models categorised as a macroscopic simulation model:

- **EMME** was developed in the 1970s by the Centre of Research on Transportation (CRT) at the University of Montréal, in Canada. The first commercial EMME 2 software was made available in the 1980s by the CTR which was founded by Professor Michael Florian amongst others. The software was further developed and EMME 3 emerged, enabling the model to incorporate graphic interfaces with a variety of tools for the simulation process and for general analysis of the road network. EMME 4 introduced more advanced developments such as modelling of crowds, vehicle discomfort, capacity limits and waiting time increment through a congestion assignment tool. The software is continuously being developed with regard to graphic interfaces, analysis approached, and virtual and zone-level travel demand model applications (Hildebrand & Hörtin, 2014).

Mesoscopic simulation models

The development of mesoscopic simulation models came about to conciliate between microscopic simulation models and macroscopic simulation models. The availability of mesoscopic simulation models is declining due to the concomitant availability of computational requirements of microscopic modelling. In mesoscopic simulation models, movement is controlled by the link’s average speed or the entire network’s average speed (Alexiadis *et al.*, 2004). Generally, mesoscopic simulation models integrate both macroscopic and microscopic simulation models aspects, enabling individual vehicles to be considered in the network, although mesoscopic simulation primarily deals with a vehicle's dynamics and not so much with the details of vehicle following and lane changing behaviour. Due to the advantage of incorporating aspects of both the simulations models (macroscopic and microscopic), mesoscopic simulation models are less demanding when it comes to data input and computational requirements as well as expertise. There are two types of mesoscopic simulations models, those that consider vehicles in a group or in a platoon form moving through the network, and those that use individual vehicles' basic dynamics to establish traffic flow dynamics (Barceló, 2010).

Microscopic simulation models

Microscopic simulation models, as explained by Yin (2014), were developed in the 1950s and were constituted by car-following models. Accordingly, since the establishment of simulation models, numerous studies have been conducted concerning their competence and field applications. As the models are technologically advanced as compared to analytical models, they are typically employed for a detailed evaluation of local network operations such as congestion at intersections, freeway, lane changing and merging sections, weaving points, urban corridor and arterial operations. The evolution of microscopic simulation models dates as far back as the early stages of digital computers when the basic principles were set up, through the seminal work of Robert Hermann with the General Motors Group. Nonetheless, computer-related requirements affected their development, but this all changed as hardware and software improvements made them affordable (Venter *et al.*, 2001).

The ability of microscopic simulation models to predict different factors within the traffic flow sphere – such as traffic signal timings, desired speed and gap acceptance – together with the ability to analyse road network traffic conditions, demonstrated potential for exploration. The models can monitor individual vehicle movements which enable the analyst to investigate road network configurations and operational situations that are far beyond the limits of typical analytical tools (Chaudhry & Ranjitkar, 2009). Microscopic simulation models are effective in situations where the intersections are located within the influence of adjacent intersections and are affected by the operations upstream or downstream. The graphical simulation provided by the microscopic model outputs is desired for the field observation verification of traffic operations. In microscopic simulations, the behaviour of vehicles and drivers in a network is defined by properties such as awareness, aggregation, minimum headway between vehicles and gap acceptance. These properties are allocated to the population after the statistical perturbation and distribution. With simulation models, numerous outputs are observed: queue length information, traffic flow, vast movements and fuel emission (Borsari, 2012).

Microscopic simulation models have become of great assistance to traffic and transportation engineers as they allow for a network's traffic conditions to be modelled in real-time. This limits disruptions to the daily operation of the road network, a cost-effective technique for evaluation and analysis of the possible measurement of improving road network performance. Microscopic simulation models are useful to traffic and transportation engineers and their adequacy to modelling real-time traffic conditions is crucial. The models' adequacy in modelling traffic condition in real-time will assist transportation professionals in selecting the best simulation model applicable to the saturated or oversaturated traffic conditions since these traffic simulation models were designed on the basis of free flow or unsaturated traffic conditions (Tedla, 2009). Microscopic simulation models are most suitable for geometric designs, traffic control, and the evaluation and analysis of numerous traffic management measures, such as congestion and incident management, ramp metering and road works. Microscopic simulation models operate on the concept of traffic flow reproduction, where they simulate the behaviour of individual vehicles. These microscopic simulation models are able to capture all the dynamics of the traffic phenomena subject to time as well as cater for driver reaction (Venter *et al.*, 2001).

The majority of presently available microscopic simulation models are those pertaining to car following, lane changing and gap acceptance for simulating vehicular behaviour. Car following models are a group of static models, where the driver's (follower) reaction is the response to the immediate vehicle preceding (leader) motion in the traffic flow. Car following models have, however, common disadvantages in that the built-in parameters are universal, meaning that they are the same for the whole network under evaluation, although each driver's behaviour is known to be affected by traffic conditions. In that case, the selected microscopic simulation model should be capable to account for car following modelling based on local driver behaviour (Venter *et al.*, 2001). According to Vanderschuren (2007) and Miller *et al.* (2004), microscopic and mesoscopic simulation models are more practically applicable in the transportation and traffic engineering industry where most operational software packages are available (but are mainly commercially supplied). They are capable of modelling detailed operations of individual vehicles, showcasing the urban transportation system. A selection of microscopic simulation models is detailed below:

- **CORSIM -** (CORridor SIMulation) is a microscopic stochastic model established and still maintained by the United States (US) Federal Highway Administration (FHWA). This simulation model integrates both the NETSIM model (used mainly for urban street simulation) and the FRESIM model (used for the simulation of freeways). The aim of this simulation model is to evaluate different types of networks, freeways, corridors and urban streets. The model also has the capability of simulating various intersection controls. Additionally, the model is able to manage various road geometries such as number of lanes and turning in the network as well as a considerable number of the road network traffic conditions. At entry points, intersections or the road network at large, the roadway segments are represented by the links, while the nodes denote change. This simulation model also allows for the integration in the windows-based environment and interface delivery (Ahiamadi, 2013). According to Reza (2013), operational features include the reaction of the driver towards upcoming network geometric changes, numerous driver habits, diverse types of vehicles, heavy vehicle movements, responsive traffic ramp metering and clock time.

- **Paramics** – (PARAllel MICroscopic Simulation), a micro stochastic model introduced by Quadstone Limited, is subdivided into five components: modeller, processor, analyser, programmer and monitor. This simulation model is capable of simulating the movement of individual vehicles based on car following and lane changing aspects on roundabouts, advanced signal controls, arterials, freeway networks, incidents and high occupancy vehicle (HOV) lanes. Through simulated road networks, three-dimensional car movement in animation is enabled by the Graphical User Interface (GUI). Additionally, in Paramics, simulation of the individual vehicle can be tailored through the Application Programming Interface (API), allowing the simulated results to replicate real-time traffic conditions. The API can also be used in other aspects of the traffic flow alleviation measures such as traffic signal optimisation, versatile ramp metering and incident detection. For the simulation model, the input parameters are divided into four categories: general configuration, network attributes, traffic assignment and demand data, while travel time, traffic flow, speed, density, delay and queue length are the expected model output parameters (Reza, 2013).

- **Aimsun** – Xiao *et al.* (2005) report that Advanced Interactive Microscopic Simulation for Urban and Non-Urban Networks (AIMSUN) was developed by the Poletecnica de Catalunya in Barcelona, Spain. According to Reza (2013), the simulation model proficiencies, amongst others, include a computer-based platform and representation of real-time traffic conditions. Throughout the simulation period, the gap acceptance, lane changing, and car following models present individual driver behaviour inside the AIMSUN simulation model. The simulation model also entails an Application Programming Interface (API) which allows a few user-defined application communications. One of the advantages of AIMSUN is that it has the competency of detailed traffic network modelling and thus can yield several measures of effectiveness.

- **VISSIM** – VISSIM, an acronym in the German language for “traffic in towns- simulation”, is software that is a stochastic type of a microscopic simulation model established by the Planung Transport Verkher (PTV), a company based in Germany. The software was developed in the early 1970s at the University of Karlsruhe. In 1993, it was commercially distributed by the PTV group in Germany. The software, developed for the analysis of urban traffic and public transport operations, is comprised of two principal components: traffic simulation and a signal state generator. Traffic simulation deals with vehicular movement while the signal state generator deals with the communication of status between detector information and the traffic simulator (Bloomberg & Dale, 2000). According to Reza (2013), Moen *et al.* (2000) also explain that VISSIM is capable of delivering MoE commonly used in the traffic engineering domain. Moreover, it can also model numerous types of vehicles in both arterials and freeways under various composite traffic situations.

According to Fellendorfand and Vortisch (2010), regarding road network performance, the evaluation of VISSIM is carried out by taking into consideration, the rational model of the driver and the individual vehicle behaviour model of the perspective traffic mode. Reaction and response time are encompassed in the driver behavioural model. Tettamanti (2015) further explains that the VISSIM simulation model, extensively employed by traffic and transportation engineers as well as researchers in different traffic-related problems, is used for the development and designed traffic congestion alleviation measures within different road networks. The VISSIM simulator uses a Wiedemann originally developed model called psycho-physical driver behaviour, allowing the engineer or other practitioner to design any road network geometry and establish simpler simulations through a convenient graphical user interface (GUI). However, in cases where practitioners must access and contrive some of the objects in VISSIM during the dynamic simulation, the GUI is unsatisfactory (Tettamanti, 2015). In such cases, another interface is introduced in conformity with Component Object Model (COM), a technology that facilitates software inter-process communication. With the VISSIM COM, the practitioner can control the model’s internal objects characteristics dynamically.

The software is time-step-based and behaviour-based, generally developed for modelling flow of both urban traffic and public transit. There are two inbuilt components that use the interface communication channel: the traffic simulator (microscopic traffic model) which simulates vehicular movement and generates equivalent output results, and the signal state generator which updates the traffic signal status in relation to the subsequent simulation stages. The traffic signal status determination is through detector data acquired from traffic simulators distinctly based on time-step (the variation ranges from 1-10 stages per second) and converts back the status to the testing system (Xiao *et al.*, 2005).

PTV, as claimed by Ahmed (2005), is a well-recognised simulation model for transportation planning, design and evaluation of existing as well as developed operations. The software offers a detailed level of integration particularly among the strategic planning, traffic engineering and transportation operations procedures within the transportation planning industry. PTV VISSIM is a microscopic simulation model that caters for the simulation of traffic and transit manoeuvres. According to Xaio *et al.* (2005), in VISSIM, the input data incorporates features such as lane assignment, lane geometries, traffic demands computed from traffic flow rates and various vehicle turning movements, OD matrices, vehicle speed distributions, traffic signal timing plans as well as deceleration and acceleration. The traffic signal at an intersection may be actuated, fixed or adaptively controlled using VAP. VISSIM is also equipped to generate MOEs such as total delay, stopped-time delays and queue lengths, consumption of fuel and emissions. The model has previously been employed in vast transportation projects such as improvement and analysis of signal priority logic of transit traffic flow, analysis and maximising network operations of the combination of actuated and coordinated signal timings, as well as the investigation of network locations where traffic movement is slow, together with traffic from neighbouring facilities (Xaio *et al.*, 2005).

Nano simulation models

Nano simulation models, a new addition to the simulation field, are at times is referred to as the traffic safety modelling tool that adjudicates the driver's steering behaviour modelling together with time-notion response detailed components with the purpose of integrating traffic safety. Nano simulation models are often classified as a micro-simulation sub-category. Traditionally, in microscopic simulation models, the programming procedure is as such to avert vehicles collisions. Therefore, the Helsinki University of Technology (HUT) established the Helsinki University of Technology Simulation (HUTSIM) model, aimed at finding a way to incorporate the principles of the nanoscopic model like time reaction lapse and response errors into driver behaviour models in the HUTSIM (Turley, 2007). Microscopic simulation models used for the analysis of the urban traffic condition could benefit fundamentally from newly developed validation techniques, as this will minimise the need to continuously calibrate and validate models before application when evaluating the city infrastructure or even implementing policies (Jenelius *et al.*, 2017).

2.9 Model calibration

According to Sykes (2010), a definition of *calibration* is, "the method of changing parameters used in a model to guarantee that input data is reflected accurately". The resulting validation process is undertaken to run an independent verification on the calibrated model. For the model development process, two sets of data are prerequisite, whereby one set is utilised for the calibration of the model by altering parameters in the model to guarantee that the results output is in agreement with field measured data. Then the second set of data is to confirm that performance characteristics of the calibrated model match the set of observed field data.

Dong *et al.* (2015) and Hourdakis *et al.* (2013), Park and Schneeberger (2003) were among the first to publish research proposing a detailed microscopic simulation models calibration methodology. Hourdakis *et al.* (2013) proposed a generic three-stage calibration technique where the simulation model can easily be fine-tuned; stage one being a volume-based calibration method, followed by the speed-based calibration method and lastly, an optional objective-based method. This model, implemented in a case study in Minnesota, was proven effective with regard to advancing the model's performance relative to real-time traffic conditions. Park and Schneeberger (2003) describe a systematic calibration technique as identification of measures of effectiveness to be investigated, data collection, determining model calibration parameters, and experimental design (as a way of limiting the number parameter combinations). This also includes iterative simulations for every parameter set, relating model parameters to measures of effectiveness, establishing a model's parameter sets, evaluation of the established parameter sets, and lastly, validation of the developed model using new data. This technique was evaluated with a case study by Park and Schneeberger (2003), with results demonstrating the benefits of the calibrated model in comparison with the un-calibrated model. Different though these two calibration procedures may seem, they are in essence similar in that, by adjusting the simulation model parameters, they both confirm MoE in simulation results to real-time field data (Dong *et al.*, 2015).

According to Jobanputra and Vanderschuren (2012) and Dowling *et al.* (2004), model calibration is necessary. This is because it is impractical for a single model to have all the required parameter values that influence the real-time traffic conditions or even replicate local traffic conditions of the entire network of the road facility. All the models must be adjusted to analyse the local conditions effectively. Generally, calibration model definitions might involve the procedure of comparing parameters of the model with real-time data for ensuring that the realistic representation of the traffic condition and environment is maintained. Praticò *et al.* (2012) also emphasise that with the calibration process, the goal is to limit the inconsistency between model output and real-time measurements. Qi (2006) explains that microanalytical models have various independent parameters to portray traffic control operations, characteristics of the traffic flow and driver behaviour. These types of models consist of built-in default parameters for every variable, while still enabling the input of values depending on the evaluated traffic condition. The model parameter changes should be based on real-time conditions during the calibration procedure and be justifiable by the software user.

2.9.1 Methods of calibration

Manual calibration

The manual calibration method is still used frequently in private consulting practices due to its advantages. Calibration of simulation models manually is usually not the ideal procedure because micro-simulation software uses a prodigious number of combinations. However, some of the advantages of this method of calibration are that it requires minimal computational demand and is reasonably easy to put into practice. It is also in compliance with measures of effectiveness, the length of a bottleneck, time and location, and driver behaviour in general. The biggest disadvantage of manual calibration is that the results obtained tend to become less optimal in comparison to the results obtained from an automated calibration method (Dong *et al.*, 2015).

Automatic calibration

As stated by Dong *et al.* (2015), although some researchers previously conducted calibration of microscopic simulation models using a manual method of calibration, most researchers use automated calibration techniques since the automated technique can reach a closely optimal result. Additionally, as a result of the computing power evolution and availability of resources, an increasing number of researchers are choosing the automated calibration technique as these techniques can accommodate the requirements of the calibration procedure.

2.9.2 VISSIM calibration technique

- System calibration

According to Rrecaj and Bombol (2015), this is the high order calibration level where the objective is to confirm the entire operation of the model with reference to system assumptions. Therefore, the calibration process satisfies the validity of input assumptions related to the model. The system calibration level parameters incorporate multiple assumptions such as vehicle route choice, inputs of the traffic demand, nature of the traffic in the study area, study area boundaries, transient demand and routing distribution and seeding period.

- Modelling infrastructure supply

The primary contrast between micro-simulation models and other traffic and transportation engineering models is their ability to provide a detailed delineation of road networks. Micro-simulation models include individual road network features such as the number of lanes, lane restrictions, design of the intersection (i.e. roundabouts, signalised intersections, un-signalised intersections and ramps) and public transport lanes. Moreover, nodes and links are the essential components of the road network in micro-simulation models: the nodes indicate any road network layout changes such as slope changes, curves and number of lanes, while the links are simply the connectors between the nodes (Borsari, 2012).

- Intersections

Various intersections available in the road network, either being signalised, un-signalised, and the roundabout or even ramps can be illustrated. Ramps are specific to different network system requirements. Nonetheless, they are fundamentally, with respect to intersections and the way they are modelled, similar to priority junctions. The advantage of having ramps in a network is that they can model priority and joining traffic stream behaviour. There is a universal requirement for a road network to identify the vehicle stopping line for the driver to evaluate, prior to making an entry, if there is sufficient time or space to complete the movement. The position, then identified as a stop line, is an essential vehicle kinematics point of reference (Borsari, 2012).

Borsari (2012) further highlights that the location at which vehicles stop during the red phase at the traffic signal, in the case of signalised intersections, is demonstrated by the stop lines. The way the intersection is modelled is impacted by the presence or the absence of nodes. Working on the link single lanes is practical if there are not any nodes in the network. That is, a traffic signal is linked to individual lanes and the behaviour of the intersection will then be determined by all the traffic signal cycle lengths of that particular intersection, regarding the movement formation at the intersection. However, the connections at the intersection determine the movement allowance.

In situations where nodes are present, working on single lanes is therefore not possible as the intersection is presented by nodes and further, the number of lanes contributes to the input data from the links associated with those nodes. As a result, there are three stages of modelling the traffic signal in these circumstances: firstly, the permitted movements, and their relative lanes, need to be specified (which lanes permit which movement in the intersection); secondly, there should be stage definition (a stage is described as a platoon of vehicles that moves during the same signal phase). Lastly, the input of all the phase signal times (red, yellow and green), along with all essential signals, offsets within the same traffic signal phase movements. An *offset* can be defined as the period in which vehicular movement stops for a few seconds prior to a signal phase end to permit left turn movement.

- Links

Micro-simulation models introduce a large control over a considerable number of characteristics of links, enabling the user to model the network layout in detail. Firstly, a link has to be part of a specific road classification; the classification affects the link's highest permitted speed and the driver's behaviour. For this situation, the width of the road takes priority over the number of lanes. However, the number of lanes still needs to be modelled so as to give a true picture of the facility for the simulation to be more realistic, along with vehicle space occupational specifications. The closure of links in the road network can permit lane restriction for particular vehicles, based on their characteristics (vehicle type, weight, height or width). An example would be a lane designated only for buses. Stop lines at each link symbolise compulsory locations (points) where vehicles are expected to pass as well as the link's starting point relative to the intersection surroundings from which vehicle movements can be initiated on the links (either changing of lanes or overtaking other vehicles) (Borsari, 2012). Figure 2.3 shows a typical signalised intersection coded in VISSIM with links and connectors.

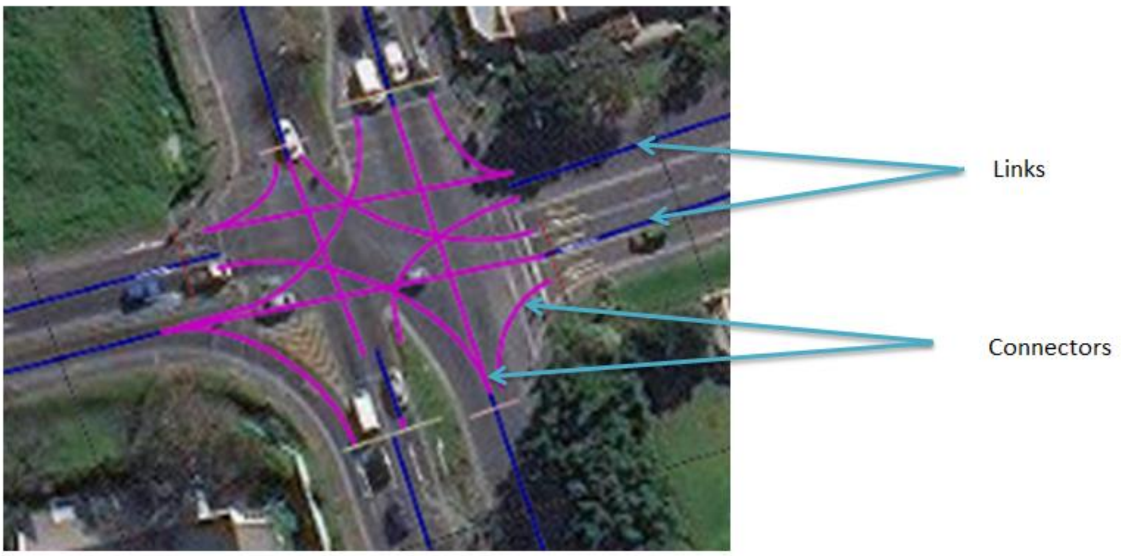


Figure 2.3 Typical coded signalised intersection in VISSIM

- Operation calibration

This procedure, addressing the adjustment of parameters of the model, influences the overall traffic operations of the respective study area. This section of calibration deals with adjusting intricate parameters of driver behaviour that influence the transportation network's entire capacity, driver aggressiveness and lane changing behaviour. The operational calibration stage is also important for freeway congestion modelling as well as local driving behaviour that can generally influence a particular study area's traffic flow, speed, density and congestion (Rrecaj & Bombol, 2015).

- Driver behaviour model

Rao and Rao (2015) explain that the driver behaviour in VISSIM constitutes two behavioural models: lane changing and car following models. However, both models necessitate extensive mathematical representation to replicate the real-time traffic flow situations. The model uses the Wiedemann 99 model (applicable for the analysis of freeway traffic) and the Wiedemann 74 model (applicable for the evaluation and analysis of arterial/urban traffic).

- Car following theory model

The Wiedemann model in VISSIM integrates psychophysical car following models regarding longitudinal vehicular movement and adjacent vehicle movement for the rule-based algorithm. The Wiedemann model assumes that the driver can occupy one of the four driving positions while travelling on the highway: free driving, following, approaching and decelerating position. Figure 2.4 graphically illustrates the car following model according to the Wiedemann concept (Fellendorf & Vortisch, 2010).

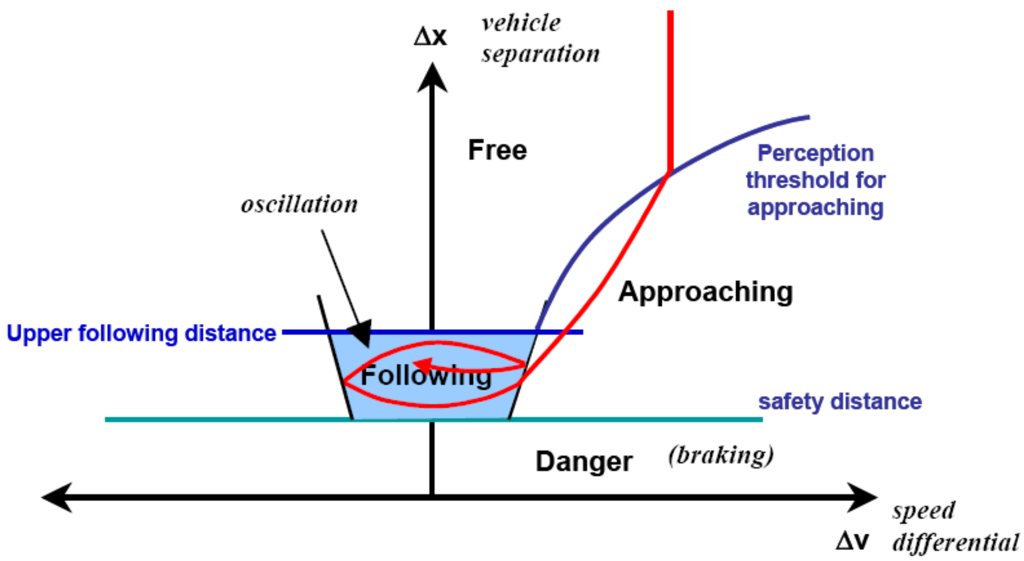


Figure 2.4 Wiedemann car following theory (Fellendorf & Vortisch, 2010)

Free driving: In this driving mode, the driver tries to reach and maintain their specific desired speed. The speed in this driving mode cannot be kept constant, but fluctuates around the desired speed as a result of restrained accelerator control.

Approaching: This refers to the stage where the driver reduces their own speed due to the speed of the preceding vehicle. While approaching, the driver decelerates in order to minimise the speed difference of the two vehicles to zero by the time the driver reaches their ideal safety distance.

Following: At this stage, the driver follows the preceding vehicle with no cognisant acceleration or deceleration. The driver maintains the safety distance pretty much consitantly.

Braking: At this stage, the driver applies medium to high deceleration rates provided the distance with the preceding vehicles falls below the desired safety distance. This can occur if the preceding vehicle unexpectedly changes its speed or if the third vehicle decides to switch lanes in front of the preceding vehicle.

As explained by Espejel-Gracia *et al.* (2017) and Fellendorf and Vortisch (2010), the Wiedemann 74 model used in VISSIM, based on driver behaviour, is computed in accordance to Equation 2.2 below.

$$\mathrm{d} = \mathrm{ax} + ((\mathrm{bx}_{\mathrm{addit}}) + (\mathrm{bx}_{\mathrm{mult}}) \times (\mathrm{z})) \times \sqrt{\mathrm{v}} \qquad (2.2)$$

Where:

d	=	safety distance between vehicles
ax	=	the average standstill distance
bx_{addit}	=	additive part of safety distance
bx_{mult}	=	multiplicative part of safety distance
z	=	range value between 0 and 1, that refers to driver's behaviour
v	=	free flow velocity

The term ax in Equation 2.2 is expressed in Equation 2.3 as,

$$\mathrm{ax} = \mathrm{L}_{\mathrm{n}-1} + \mathrm{ax}_{\mathrm{addit}} + \mathrm{RND1}_{\mathrm{n}} \times \mathrm{ax}_{\mathrm{multip}} \qquad (2.3)$$

Where:

L_{n-1}	=	vehicle length
ax_{addit} and ax_{mult}	=	calibration parameters
$RND1_n$	=	normal distribution of random vehicles' numbers

2.9.3 Calibration criteria

As indicated by Jobanputra and Vanderschuren (2012), calibration of a simulation model can be carried out with the evaluation of capacity and operational performance measures like delays, queue lengths, speed and travel time. The calibration procedure, depending on the number of parameters evaluated in a study and their interactions, can be a complex and time-consuming activity, more so in cases of more than one criterion application.

According to Jobanputra and Vanderschuren (2012), numerous studies propose limits to a single criteria approach when calibrating a model. The single criteria approach focuses only on one attribute such as speed or travel time and does not consider the accuracy of other attributes such as headway, gap acceptance, delay and acceleration. In a single criterion, the procedure takes the form of a sequence; one set of parameters is fixed for the calibration of the next set of parameters and so on. Hence, the limitation of the single criteria method of calibration has led to the development of a multi-criteria approach in solving model calibration issues, as shown in Table 2.3.

Table 2.3 Simulation calibration using multiple criteria (Jobanputra & Vanderschuren, 2012)

Study	Type of Optimization	Model	Network	Measures of Performance	Results	Note
Toledo et. al. (2004)	Iterative Averaging	MITSIMLab	Freeway	Speed & Density	4.6 % (MAE for speed)	Only speed data shown; does not apply multi-criteria framework
Balakrishna et. al. (2007)	Simultaneous Perturbation Stochastic Approximation (SPSA)	MITSIMLab	Freeway	Volume (counts)	22 to 65 % (RMSPE)	Introduces a multi-criteria framework but does not apply it.
Ma et. al. (2007)	SPSA	PARAMICS	Freeway	Link Capacity & critical occupancy	0.70 % (Sum of GEH)	Two-criteria calibration
Ciuffo et. al. (2008)	OptQuest/Multi start Heuristic) OQMS	AIMSUM	Freeway	Network travel time	11 % (RMSPE speed); 17% (RMSPE Volume)	Mean absolute error ratio
Duong et. al. (2010)	genetic algorithm	VISSIM	Freeway	Volume & Speed	1.9 % (RMSPE Speed); 10.5 % (RMSPE Volume)	Introduces the concept of Pareto optimality (non-dominance) to the traffic calibration problem
Huang and Sun (2009)	NGSA II	VISSIM	Freeway	Volume & Speed	1.0 (Volume Fitness) and 0.97 (Speed Fitness)	Applies the NSGA II without looking at the resultant non dominant set

2.9.4 Calibration targets

Dowling *et al.* (2004) explain that calibration targets are where the numerous parameters of the simulation model are fine-tuned in efforts to obtain acceptable results or those that closely represent field data results. The adjustment of one parameter in the model has a major effect on other parameters, either being in the evaluation of network or corridor. Therefore, the traffic engineer or traffic analyst must be aware that the calibration process of a simulation model requires an iterative and multi-dimensional. Jobanputra and Vanderschuren (2012) add that although calibration aims to confirm simulated output results to that observed in the field or in real-time, there is a limit to the number of iterations performed for improving the model's accuracy. As such, calibration targets are normally set for travel time, traffic flow rates, speed, delay and queue lengths, because these are designated as the limitations of calibration targets regarding vehicles.

2.10 Model validation

Model validation only commences after the calibration process, whereby calibration can no longer occur further to improve the output of the model (Madi, 2016). Pinna (2007) explains that in designing congestion control measures, planning road works in an area or network, or the modification of a particular traffic network structure, the use of a good traffic flow model is crucial. Simulation model validation is at times a challenging and laborious procedure to even render an accurate definition. During validation, the model is analysed for rational, model structure accuracy and behaviour as opposed to the referent system. Additionally, in model validation, the objective is to impart confidence in the simulation technique. In principle, validation of a model in the context of traffic and transportation simulation represents a correlation between simulated model results and real-time data (Ahmed, 2005).

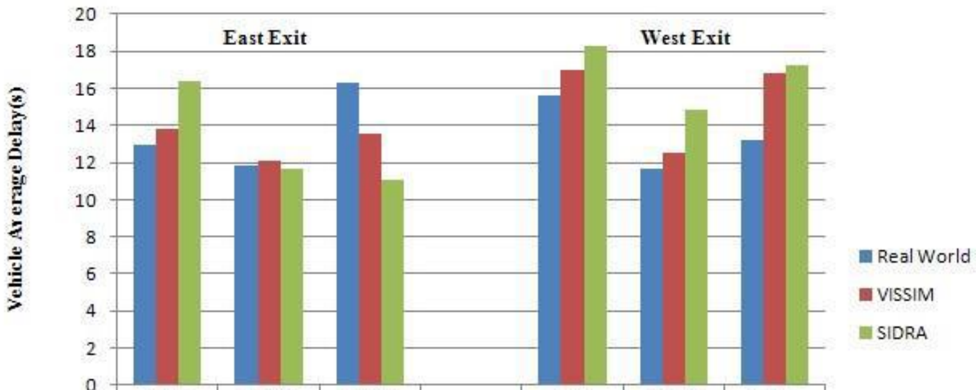
El Esawey and Sayed (2011) indicate that the intention of model validation is the comparison of the calibrated model results in newly collected real-time data apart from the data used in the process of calibration. Data from different entities such as a different corridor or network or data collected from the same entity under evaluation but with different traffic conditions or travelling periods to those used during the calibration process, as well as applying other measures of effectiveness, can be used for model validation (Park & Scheeberger, 2003; Hollander & Liu, 2008). The simulation model is acknowledged as valid only in the case where MoE identified from a new real-time data collection set closely represent the simulated model results. If not, then the calibration of the driver behaviour parameter set requires recalibration (El Esawey & Sayed, 2011).

As indicated by Madi (2016), micro-simulation model calibration and validation are vital procedures as they enable the model to simulate vehicular activities for closely representing real-time vehicle dynamics. Depending on the variables chosen for analysis of the network, for both the calibration and validation procedure of the simulation model, the statistical analysis procedures such as time series analysis and paired or multiple comparisons can be used. As determined by previous research, most guidelines recommend the sensitivity testing method for identifying the most significant model parameter to be adjusted, based on the type of network or corridor under evaluation. This is because the calibration process starts with the model's universal network parameters (default parameters), and then follows with fine-tuning of the local network-specific parameters so they can closely represent the real-time network conditions.

Hussein *et al.* (2017) conducted a calibration and validation study using both the Wiedemann 74 and Wiedemann 99 car following models for the evaluation of lane changing behaviour for individual signalised intersections in Karachi, India, under heterogeneous traffic conditions. One intersection was used for the calibration of the model and to validate the developed model in PTV VISSIM and then the model was then tested on another signalised intersection. To evaluate the correlation between field observed intersection performance and simulated performance, Geoffrey E. Havers (GEH) statistics were employed. The results for average standstill distance with regard to the car following model and lateral distance with regard to lane changing behaviour models from GEH showed an excellent correlation between field observation and simulated results.

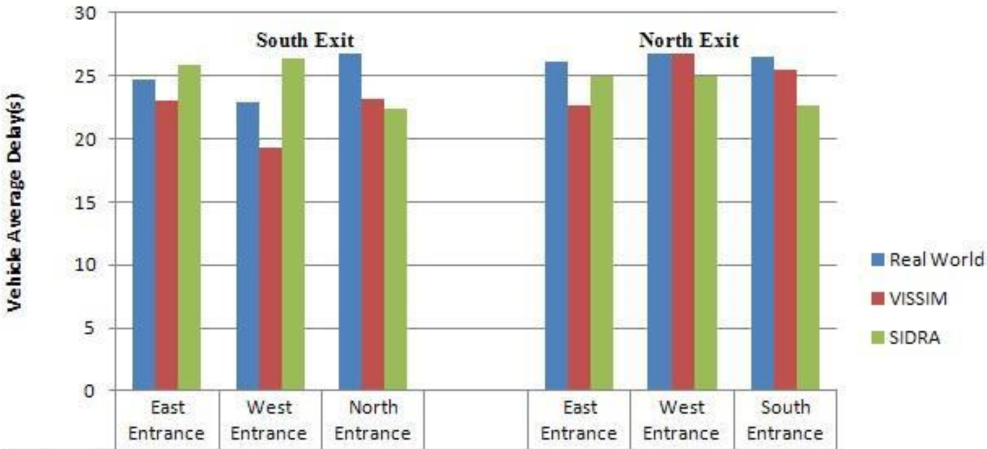
Comparison of calibrated model output results to field data has been investigated in previous studies, with study conclusions highlighting the significance of model calibration to enhance model certainty. An example is a study by Bared and Edara (2005) where the microscopic simulation model VISSIM was compared to analytical models SIDRA and RODEL on roundabouts, using gap acceptance as a calibrating parameter. The capacity results indicated that the calibrated model in VISSIM was close to representing field observed capacity in comparison to the two analytical models. Gagnon *et al.* (2008) evaluated three microscopic simulation models (PARAMICS, SimTraffic and VISSIM) with two analytical models (RODEL and SIDRA) against field measurements on modern roundabouts based on delay estimations, with models calibrated using critical headway and speed distribution. Results demonstrated that VISSIM performed with the most accuracy in comparison with the empirical delay measured.

Tianzi *et al.* (2013) conducted a study at two individual intersections comparing the operation simplicity and output error of two traffic analysis tools, VISSIM and SIDRA. A few conclusions were drawn from the study, firstly based on simplicity: SIDRA was more operable as compared to VISSIM with regard to the construction of the road network, traffic signal phasing setting and the speed of output results. Both models were calibrated individually, and the average delay was then used to compare the output results of the models against field observations, as depicted in Figure 2.5 and 2.6. VISSIM, in comparison to SIDRA, better represented the field average delay, with mean errors of 10.18% and 14.78%, respectively. The study further pointed out that in the case of signalised intersection simulation, the VISSIM model is the desirable choice when it comes to average delay accuracy prediction.



	West Entrance	South Entrance	North Entrance		East Entrance	South Entrance	North Entrance
Real World	12.92	11.8	16.3		15.61	11.7	13.22
VISSIM	13.83	12.07	13.55		16.96	12.51	16.8
SIDRA	16.4	11.7	11.1		18.3	14.8	17.2

Figure 2.5 Signalised intersection, east and west average delay (Tianzi *et al.*, 2013)



	East Entrance	West Entrance	North Entrance		East Entrance	West Entrance	South Entrance
Real World	24.67	22.87	26.75		26.16	26.8	26.54
VISSIM	23.02	19.23	23.16		22.67	26.72	25.43
SIDRA	25.9	26.4	22.4		24.9	24.9	22.6

Figure 2.6 Signalised intersection average delay, south and north exit (Tianzi *et al.*, 2013)

Tawfeek *et al.* (2018) calibrated and validated the VISSIM model using the vehicle class (passenger cars, microbuses, heavy vehicles and buses) on a 12 km long urban corridor. The study reported a similar performance improvement trend between uncalibrated and calibrated vehicle classes. The improvement ranged from 58% to 91% for the respective vehicle classes. Similarly, El Esawey and Sayed (2011) conducted a study on the calibration and validation of the VISSIM model for medium-size networks using an urban condensed grid network of more than 100 signalised intersections. The results showed an improvement between the default and calibrated travel time estimates. Mean Absolute Percentage Error (MAPE) values of 44% and 14% were observed using default and calibrated parameters, respectively. The study further determined that the VISSIM average travel times were between one standard deviation of the field measured travel times, indicating a reasonable proximity between VISSIM and field measured travel times. Fatima (2015) evaluated modal congestion management strategies and their influence on operating characteristics of an urban corridor consisting of four intersections. After model calibration, travel speed, average delay and queue distance predictions by the model were observed. The difference between model results and the field observations at all intersections were below 2%, a result considered an acceptable error difference. This then qualified the model to be used for further analysis.

2.11 Statistical analysis

Chalermwongphan and Upala (2018) claim that there are numerous statistical methods used to evaluate the goodness of fit between model predicted values and field measured values such as Root Mean Square Error (RMSE), Root Mean Square Percentage Error (RMSPE%), Mean Error (ME) and Mean Absolute Percentage Error (MAPE%). Table 2.4 shows the acceptable indicators of these statistical methods. For these methods, obtaining the values or percentage values close or equal to 0 implies a perfect fit between the evaluated data sets. These methods are the most popular as they were applied by numerous researchers to evaluate the validity of model estimates under uncalibrated and calibrated conditions (Chaudhury & Ranjitkar, 2009; Preethi *et al.*, 2016; Wang *et al.*, 2018). According to Shaaban and Radwan (2005), another statistical method adopted by many researchers is the Relative Error (RE) which calculates the variation between field and modelled data as a percentage. Xiao *et al.* (2005) state that when numerous simulation models are compared, it is imperative to keep the percentage output error as minimal as possible by spending more time during calibration. This will improve the accuracy of the models on replicating field measure conditions. Walpole *et al.* (2011) express that analysis of variance (ANOVA) is a common statistical tool for evaluating the difference between population means.

Table 2.4 Typical statistical measurements used to quantify the model's accuracy (Chalermwongphan & Upala, 2018)

No.	Factor	Formula	Acceptable Indicators
1	Root Mean Square Error (RMSE)	$RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(Y_i - X_i)^2}$	RMSE = 0 implies perfect fit.
2	Root Mean Square Percentage Error (RMSPE%)	$RMSPE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\frac{\left(\frac{Y_i - X_i}{Y_i}\right)^2}{n}}$	RMSPE% = 0 implies perfect fit. Note: The percentage error cannot exceed 100%
3	Mean Absolute Deviation (MAD)	$MAD = \frac{1}{n}\sum_{i=1}^{n}\lvert Y_i - X_i\rvert$	MAD = 0 implies perfect fit.
4	Mean Bias Error (MBE)	$MBE = \frac{1}{n}\sum_{i=1}^{n}(Y_i - X_i)$	MBE < 0 = modelled count data is less than observed traffic volume, and MBE > 0 = modelled count data is more than observed traffic volume.
5	Mean Percentage Error (MPE%)	$MPE\% = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{Y_i - X_i}{Y_i}\right) \times 100}$	-10% < MPW < 10% = acceptability ranges.
6	Mean Absolute Percentage Error (MAPE%)	$MAPE\% = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left\lvert\left(\frac{Y_i - X_i}{Y_i}\right) \times 100\right\rvert}$	MAPE% = 0 implies perfect fit. Note: The percentage error cannot exceed 100%

Bluman (2000) explains that correlation coefficients calculated from sample data measure the strength and direction of a relationship between two variables. As various correlation evaluation techniques can be employed, sample size plays a vital role in choosing the appropriate correlation evaluation method. The Spearman's rank correlation coefficient, used to investigate the correlation between two data sets, is mainly adequate for a smaller sample size (less than ten). The Spearman's rank correlation coefficient (p) can take values from -1 to +1. This technique uses a ranking system where the variables are ranked depending on the sample size from 1 upwards. A ranking of 1 represents the least variable within the data set while the highest-ranking value represents the highest variable within the data set, depending on the sample size evaluated. The closer the p - value to 1 is obtained, the stronger the correlation between the data set it represents, irrespective of the sign (negative or positive) (Rupi *et al.*, 2015).

Nyantakyi *et al.* (2014) conducted a study evaluating the performance indication measures at an intersection in Amakom, Ghana, using micro-simulation models in Synchro/SimTraffic. Geometric, traffic and signal control data, including important parameters which impacted greatly on the calibration process of the models, were collected from real-time field data. From the results observed, at a 5% significance level, t-test and Chi-square test, revealed a strong correlation of headway with saturation flow in comparison with speed for both simulated and field conditions. It was concluded that change in phase plan with geometric development would improve the LOS of the intersection under evaluation.

2.12 Differentiation between micro-simulation models and analytical models

Micro-simulation is an alternative model for the analysis of traffic. In comparison to static methods applied in the analytical models, simulation models use a dynamic method of modelling. The benefit of micro-simulation models is that they are capable of modelling traffic individually, where both pedestrians and vehicles get independent behaviour classification. The stochastic approach of the traffic in micro-simulation models is due to the random variables used. Furthermore, given the fact that micro-simulations are mainly static and dynamic in nature, the network aspects such as the road, lanes, yield lines or stop lines that are unchanged in the road network are classified under static aspects of the network. Alternatively, vehicles and pedestrians are classified as the dynamic aspect of the network because they can change any time during the analysis period (Ekman, 2013). Studies that also compared analytical and micro-simulation models from different researchers present varying outcomes, where in some cases, comparable results were observed between micro-simulation models and analytical models, and in other cases, significant differences were observed.

2.13 Reviewed literature

A summary of the reviewed literature is presented in Table 2.5 in which various researchers evaluated the capability and compatibility of different analytical software such as SIDRA and micro-simulation models such as VISSIM in comparison to one another and to real-time performance of different facilities, and to assess which models best represent the real-time performance based on various measures of effectiveness.

Table 2.5 Summary of reviewed literature

Author(s)	Topic	Traffic network	Measure	Comparison	Findings	Traffic conditions
Roundabouts						
Gagnon *et al*. 2008	Calibration potential of two analytical models (aaSIDRA and RODEL) and three micro-simulation tools(Paramics, SimTraffic and VISSIM)	Modern Roundabouts	Delay	Two analytical models (aaSIDRA and RODEL) and Three micro-simulation tools (PARAMICS,	VISSIM was the most versatile compared to all the softwares.	Not specified
Chen & Lee 2016	A case study on Multi-lane roundabouts under cogestion: Comparing software capacity and delay with field data	Multi-lane roundabouts	Capacity and delay estiamtion	Uncalibrated modela results (RODEL , SIDRA and VISSIM) to field data	SIDRA and RODEL over estimated the capacity as compared to the field data. VISSIM was found to be the best out of the three	Saturated traffic conditions
Bared & Edara 2005	Simulated capacity of roundabouts and impact of roundabout within a progressed signalised road	Roundabouts	Capacity	VISSIM , RODEL and SIDRA to field data	VISSIM results were comparable with the U.S. field data compared to RODEL and SIDRA	Not specified
Mills 2011	Entry- lane capacity analysis of roundabouts in Texas using VISSIM, SIDRA and the Highway capacity manual	Roundabouts	Capacity	SIDRA and VISSIM to HCM	SIDRA was found to be compatable to HCM than the VISSIM results to HCM	Not specified
Signalised intersections						
Tianzi *et a* l. 2013	Comparitive study of VISSIM and SIDRA signalised intersections	Signalised intersections	Average delay	SIDRA and VISSIM to field data	SIDRA was found to be easily operable but VISSIM was to be the accurate to real time representation	Not specified
Al-Omari & Ta'amneh 2007	Validating HCS and SIDRA software for estimating delay at signalised intersections in Jordan	Signalised intersections (5)	Control delay	HCS 2000 and SIDRA	SIDRA was found to be the better software as compared to the HCS 2000	Not specified
Jameel 2011	Estimating delay time at Palestine street intersection in Baghdad city using HCM and SIDRA models	Signalised intersections	Delay	HCM and SIDRA to field data	Although the two models results were found to be in agreement with the field data, the conclusion was that SIDRA needs significant improvements in order to suite the traffic conditions in Baghdad	Under-saturated, saturated and over saturated
Liamsiriwattana 1994	Comparison of signalised intersection analysis models for Australia and Thai intersections	Signalised intersections (4 @ each location)	Delay, stops and queue lengths	INTANAL, SCATES, SIDRA and SIMSET to field data	The software were found to be suitable for traffic conditions in Adelaide (Australia) but showed need for adjustment for the use with Bangkok (Thailand) traffic conditions	Not specified
Mixed intersections						
Fichera 2011	A practical comparison of VISSIM and SIDRA for the assessment of development impacts	(2) signalised intersections, (5) roundabouts and (8) priority controlled intersections	Delay and queue length	SIDRA and VISSIM	SIDRA calculated higher average delay than VISSIM at intersections with low traffic demand and where there is geometric negotiations needed	Low traffic demand
Arterials/corridor and urban networks						
Elesawey & Sayed 2011	Calibration and validation of micro simulation models for medium sized networks	Urban network (100 signalised intersections)	Travel time	VISSIM to real time data	The observed travel times were in reasonable match to simulated travel times from the calibrated model	Condensed network
Ahmed 2005	Calibration of VISSIM to the traffic conditions of Khobar and Dammam, Saudi Arabia	Signalised arterial(3 interections for calibration and 3 intersections for validation)	Travel time, speed and queue length	VISSIM to real time data	The results obtained from the validation showed that simuated results from VISSIM compared to field observation were within acceptable range	Saturated traffic conditions
Reza 2013	Calibration and validation of Paramics microscopic simulation model for local traffic conditions in Saudi Arabia	Urban arterial(3 signalised intersection for calibration and 3 signalised intersections for valaidation)	Travel time and queue legnth	Paramics, SimTraffic and TRANSYT- 7F to real time data	The results obtained showed Paramics to be a better representation to real time results. The model developed in Paramics was then used to evaluated the optimised signal timing plans to assess the eefctiveness of the optimesd plans.	Not specified
Park *et al*. 2006	Application of microsimulation modle calibration and validation procedure: A case study of Coordinated Actuated Signal system	Isolated intersection and and urban arterial network with 12 coordinated actuated signalised intersections	Travel time and queue legnth	VISSIM and CORSIM	The results showed that the calibrated and validated models were able to adequately replicate field conditions.Also the calibration and validation procedure used for isolated intersection proved to be applicable for an arterial under both VISSIM and CORSIM	Not specified

2.14 Conclusion

Substantial literature presents evidence of the benefits of using traffic analysis tools such as analytical and simulation models for different design alternatives on performance improvement of road networks. These have been explored for traffic congestion alleviation in urban areas from which appropriated alternatives are implemented. After the implementation of alternatives provided by the models, it is imperative to investigate the effectiveness of the implemented changes, as this will shed light on the model's accuracy on performance improvement predictions. Comprehensive research in relation to the calibration and validation of analytical and simulation models such as SIDRA and VISSIM has been carried out concerning their applicability under various road network compositions. However, there are a few areas that have not received proper attention from the transportation engineering community. The work on these models by most of the researchers has been primarily on the performance analysis of isolated signalised and un-signalised intersections.

Only a few researchers have attempted to compare the applicability of SIDRA and VISSIM on performance analysis of intersections. No literature could be obtained on the applicability limitations of SIDRA and VISSIM on the operational performance analysis of an urban corridor. Moreover, no literature was found regarding the calibration of VISSIM for South Africa's traffic condition analysis. Additionally, the local applicability of the models needs to be investigated as none are developed in South Africa for local conditions. Therefore, the driver behaviour built-in parameters of these models are bound to differ. Little information was found for the calibration of SIDRA for the South African condition; and in particular, no calibration was considered in Cape Town. This then raises concerns on the model prediction adequacy and the analysis suitability for these particular local traffic conditions.

From these identified gaps, the following objectives were formulated for this study: to assess the effect of implemented geometric upgrades at two recently upgraded signalised intersections on the operational performance difference. And secondly, to calibrate both SIDRA and VISSIM and compare their results with field measured operational performance at a local signalised urban corridor with successive intersections. The better performing model was subsequently validated for the operational performance analysis of a signalised urban corridor. These objectives will be obtained through the methodology detailed in Chapter 3.

3 Research methodology

3.1 Introduction

Traffic engineering modelling and analysis software such as SIDRA and VISSIM are widely used as assisting tools as they are able to run different traffic flow improvement alternatives to assist traffic and transportation engineers alleviate congestion. Based on the modelled-out results, alternatives can be implemented to alleviate practical traffic congestion. However, the accuracy of the traffic modelling tool needs to be evaluated to ascertain that the model provides results that represent the real-time performance of the investigated facility. This section describes the methodology adopted to carry out the study objectives.

3.2 Research design

The methodology employed for this study was split into two stages. Stage one evaluated how well SIDRA predicted the operational performance improvement of two upgraded signalised intersections. This was done by comparing the operational performance results before the implementation of the geometric upgrades and model-predicted results as well as post upgrade operational performance results and model-predicted results of the two intersections. The required data for this section of the project was collected as follows: the before upgrade and model-predicted data was retrieved from the Somerset West Traffic and Transportation Office, while the post-upgrade data was collected in the field following the completion of the upgrades through an appointed service provider.

Then followed stage two, the calibration and validation of the two traffic analysis tools used in this study (SIDRA and VISSIM) on a selected local urban corridor constituting four consecutive signalised intersections. Input data as per the requirements of the two models were collected as well as calibration data which enabled the comparison of simulated results and field observed data. Calibration was carried out to adapt the models to local conditions as these models, as shown from the literature review, are not locally developed; thus calibration is essential. After the calibration process of the two models (SIDRA and VISSIM), the next stage was then to validate the better performing model. This showed the model which best represented field measured operational performance between the two. Two separate data sets (morning peak hour and afternoon peak hour) were collected on the same corridor so that one set could be used for the calibration and the other for the validation process. Figure 3.1 shows the structure of the methodology employed for the two sections of the study.

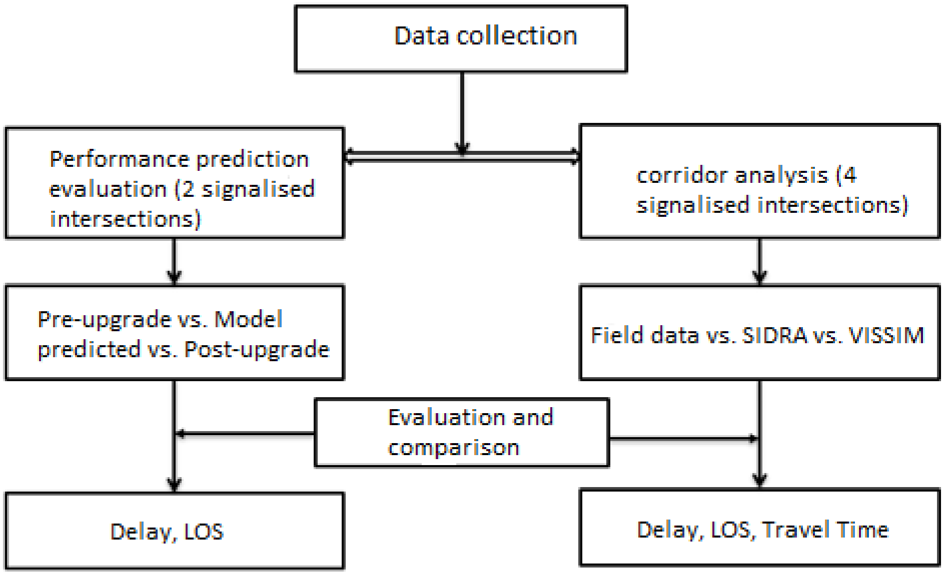


Figure 3.1 Research methodology flow design

3.3 Site description

The study area was selected based on particular criteria – ease of data collection and where recent developments were implemented based on an analytical tool (SIDRA) predictions. A corridor segment was chosen in Somerset West, located on the East Side of the main city Cape Town. Figure 3.2 and Figure 3.3 show the two recently upgraded signalised intersections which form part of the corridor segment that was evaluated.

Figure 3.2 Gordon intersection-left-before upgrade and right-post upgrade (Google Maps)

Figure 3.3 Andries Pretorius intersection: left-before upgrade and right-post upgrade (Google Maps)

The corridor segment is about 0.8 km and consists of a network of four signalised intersections (inclusive of the two upgraded ones) all connecting to a minor arterial (M9) as depicted in Figure 3.4. The study area is in a commercial area and operates on moderately high traffic volumes for most parts of the corridor segment, especially during morning and afternoon peak hours: the first intersection being Fagan Street and Ridgil Lane, then Gordon Road intersection, Andries Pretorius and Bizweni intersection and lastly, Hospital intersection (identified as intersections 1, 2, 3 and 4, respectively).

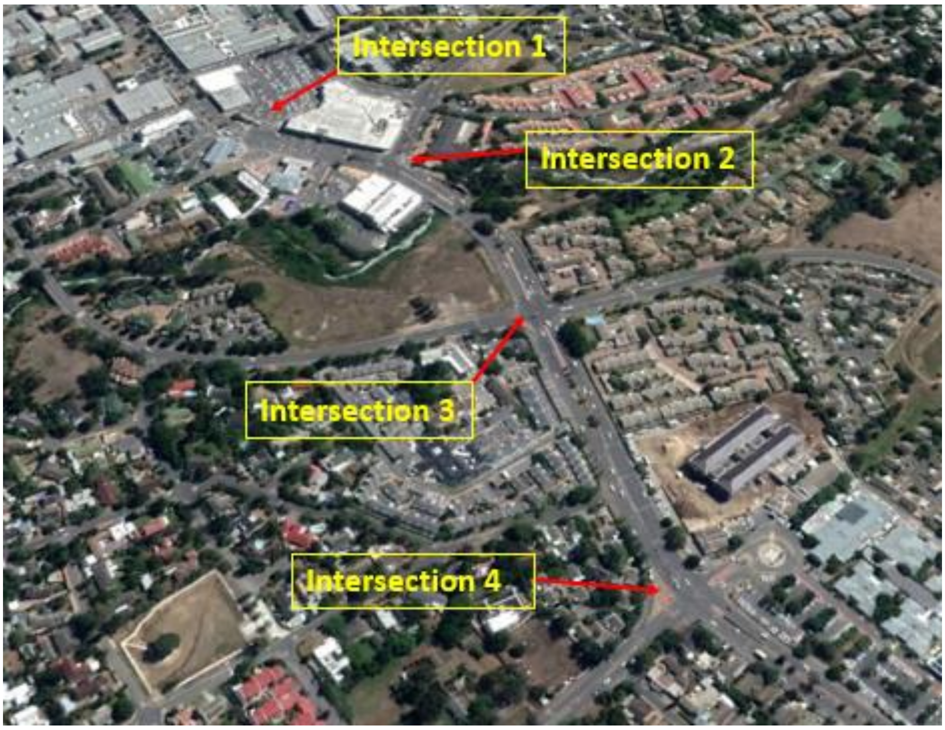


Figure 3.4 Study corridor in Somerset West showing intersections (Google Maps)

The road class is determined based on three primary criteria namely: the significance and size of the trip generator, travel distance and travel stage. The two main road classes and their trajectories are detailed in Table 3.1, showing the functionality of each criterion with respect to the relevant road class. The hierarchy of road functional classification is presented in Table 3.2.

Table 3.1 Classification of roads (TRH 26)

Primary class	Trip generator	Reach of connectivity	Travel stage
Mobility roads	Large or strategic generators	Longer travel	Through, destination not reached
Access streets	Individual properties	Short connections	Local, stop at destinations

Table 3.2 Road function classification (TRH 26)

Number	Function	Description
Class 1	Mobility	Principal arterial
Class 2		Major arterial
Class 3		Minor arterial
Class 4	Access/Activity	Collector street
Class 5		Local street
Class 6		Walkway

In conformity with TRH 26, the intersections constituting the corridor segments that were evaluated are classified as in Table 3.3.

Table 3.3 Classification of the intersections in the corridor segment

Intersection ID	Approach	Road Classification	Description
1	Main Road M9	Class 3	Minor Arterial
	Fagan Street	Class 4	Local Distributor
	Ridgil Line	Class 4	Local Distributor
2	Main Road M9	Class 3	Minor Arterial
	Gordon Road	Class 4	Local Distributor
3	Andries Pretorius Street	Class 3	Minor Arterial
	Main Road M9	Class 3	Minor Arterial
	Bizweni Avenue	Class 4	Local Distributor
4	Hospital Road	Class 5	Local Street
	Main Road M9	Class 3	Minor Arterial
	Sir Lowry's Pass	Class 3	Minor Arterial

3.4 Data collection

Collection of data can take a substantial amount of time, but the most critical aspect is to make sure that the correct data is collected for a given study. The comparison data (performance measures evaluated) to be used between field observation and model-simulated data should be the same in order for the effective correlation to be achieved; the same form of data needs to be collected. Data can be collected in numerous ways in the field, either by using equipment such as drones or by carrying out traffic counts manually at intersections at specific times on specific days. In the case of manual data collection, the observer counts all vehicles for the given evaluation period. A time interval of 15 minutes, according to the HCM 2000, is to be adopted when opting for this method. For this study, as manual data collection was adopted, this exercise was carried out on an average weekday as per the HCM 2000. Data collection was conducted during morning and afternoon peak hours because these represent the corridor operation at full capacity and to maintain accuracy of data collection.

Datasets that can easily be collected from the field and those that can be generated by the simulation model need to be properly identified for the comparison to have significance. The MoEs can be used for both driver behaviour parameter calibration and model validation; therefore, the field-collected dataset can be divided into two sets depending on the collection intervals used. Field measured MoEs of the facility under-study might be of the same day but different peak periods (e.g. morning peak and afternoon peak dataset), different counting days altogether, or data collected from different facilities (e.g. several intersections or corridors) within the same location. This enables the one dataset to be dedicated to the calibration process while the other set can be used to validate the model (El Esawey & Sayed, 2011).

3.4.1 Geometric data

According to the user guide of both models in this study, SIDRA Intersection Version 8 Plus and VISSIM 11, the required geometric data for all intersections were collected to be certain that the modelled intersections are as per the field conditions. Information such as the design of each intersection, the number of approaching and exiting lanes and lane configuration of each intersection is required. The summary of existing geometry data of each signalised intersection of the study network (corridor segment) is detailed in Table 3.4 below.

Table 3.4 Network geometric data

Intersection ID	Intersection Name & Type	Approach Leg	Entry Lane(s)	Exiting Lane(s)
1	Fagan Intersection – 4-Legged intersection	Main Road Northbound(M9)	3	3
		Ridgil Lane	2	1
		Main Road Southbound(M9)	3	2
		Fagan Street	2	1
2	Gordon Intersection –T intersection	Main Road Northbound(M9)	3	2
		Gordon Road	2	2
		Main Road Southbound(M9)	4	3
3	Andries Pretorius Intersection – 4-Legged intersection	Main Road Northbound(M9)	4	2
		Bizweni Avenue	3	2
		Main Road Southbound(M9)	3	2
		Andries Pretorius Street	3	1
4	Hospital Intersection – 4-Legged intersection	Main Road Northbound(M9)	4	3
		Hospital	2	1
		Sir Lowry 's Pass	3	2
		Main Road (M9)	3	2

3.4.2 Traffic volume study

The traffic flow behaviour assessment at road networks, one of the common practices in road traffic engineering, is measured in vehicles per hour (veh/h). The procedure incorporates a recording of different vehicle types moving within a network and the distribution of the demand (through and turning movements). Traffic volume, classified as the fundamental parameter, is used by transportation professionals during planning, design and control of facilities as well as operational analysis and management of existing facilities.

In this regard, care needs to be taken during traffic volume counts as inaccurate records will affect the effective use of the data. In efforts to assure the accuracy of the traffic volume data, a service provider (Easy Surveys Traffic Count Company) was appointed. The traffic counts were carried out on a normal weekday for the duration of three hours for both morning (AM) and afternoon (PM) peak period, then the data was reduced to one peak hour for both the AM and PM peak hours where highest volumes were observed at each intersection (Table 3.5). Traffic volumes are presented in Table 3.6 for the respective peak hours while Figure 3.5 shows the traffic movements key plans.

Table 3.5 Traffic counts reduction to the relevant peak hours

Corridor	Time of day	Counting period	Peak hour
Somerset West (Main Road M9)	Morning (AM)	06:00 - 09:00	07:00 - 08:00
	Afternoon (PM)	15:00 - 18:00	16:30 - 17:30

Table 3.6 Traffic volumes

Intersection ID	Time	Traffic movements											
		1	2	3	4	5	6	7	8	9	10	11	12
1	AM	12	399	7	18	7	22	52	718	120	39	6	12
	PM	43	606	16	47	23	72	51	431	105	210	31	68
2	AM	53	386	N/A	N/A	N/A	N/A	N/A	699	824	597	N/A	51
	PM	119	751	N/A	N/A	N/A	N/A	N/A	503	493	771	N/A	96
3	AM	108	977	30	404	91	24	908	1024	35	330	249	54
	PM	258	1224	30	17	108	559	418	758	41	40	82	143
4	AM	99	662	614	833	92	138	213	1077	58	17	39	40
	PM	42	1015	741	583	38	251	194	520	24	57	78	115

***N/A = No movement on the approach**

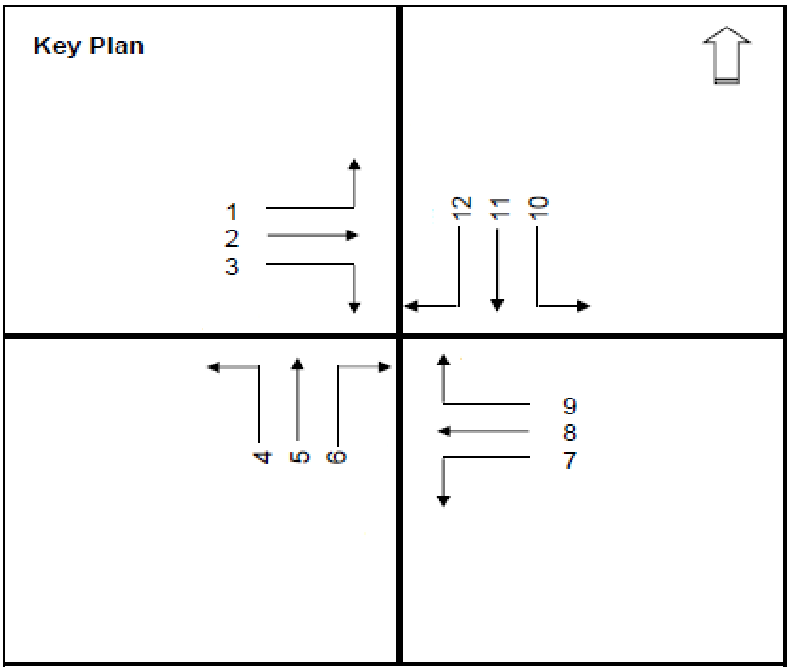


Figure 3.5 Intersections traffic movement key plan

3.4.3 Travel time

Travel time can be described as the overall time that a vehicle takes to complete its trip cycle from a certain origin to its particular destination in a road network. This type of study can be conducted in various ways – using the interview method, direct observation at a particular intersection or network, capturing of license plates of vehicles passing through the area of study, the average vehicle and the moving vehicle method. The choice of the method is conditional to the study purpose, type and size of the segment being investigated in conjunction with the availability of equipment and resources. The average vehicle and the moving vehicle technique are the most commonly used due to satisfactory correlation with the actual travel times (Ahmed, 2005).

The current study adopted the average vehicle technique which then measures travel time together with distance travelled, whereby a test car travelled through the study areas to collect the data. This method required a test vehicle and a driver, two observers, two stopwatches and datasheet. The distance of the intersections along the segment was obtained from the test vehicle odometer and travel times were recorded as the vehicle passed each point in the segment. According to the HCM 2000 requirements, the procedure was repeated six times for the test to be considered valid for the peak hour of the study. Table 3.7 and 3.8 detail the average travel time values for six experiments observed in the field and constitute part the data used for the calibration process of the models.

Table 3.7 Corridor observed travel times during the PM peak hour (collected after upgrades implementation)

S. No.	Link	Distance (m)	Field travel time (s)
1	Fagan Street Intersection to Gordon Road Intersection	126	25
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	47
3	Andries Pretorius Street Intersection to Hospital Intersection	256	30
4	Hospital Intersection to Andries Pretorius Street Intersection	297	43
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	29
6	Gordon Road Intersection to Fagan Street Intersection	94	13

Table 3.8 Corridor travel times observed during the AM peak hour (collected after upgrades implementation)

S. No.	Link	Distance (m)	Field travel time (s)
1	Fagan Street Intersection to Gordon Road Intersection	126	31
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	50
3	Andries Pretorius Street Intersection to Hospital Intersection	256	32
4	Hospital Intersection to Andries Pretorius Street Intersection	297	72*
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	30
6	Gordon Road Intersection to Fagan Street Intersection	94	14

*The high travel time value was due to high vehicle volume observed at this link compared to the other links.

The standard deviation for each links' travel time was calculated using Equation 3.1. The calculations are detailed in Appendix B 1.

$$s = \sqrt{\frac{1}{N-1}\sum_{i=1}^{N}(x_i - \bar{x})^2} \tag{3.1}$$

Where:

s	=	standard deviation
N	=	number of observations
x_i	=	observed values of a sample
$\bar{x}$	=	mean value of the observations

3.4.4 Field delay

Field delay studies are typically conducted to measure the operational performance of the intersection and several methods can be used for field measurement of delay as well as the number of stops at the identified signalised intersection. Delay is a MoE which directly relates the driver's experience and expresses the extra time expended in traversing the signalised intersection. Another aspect taken into consideration as a contributor to the calculation or estimation of delay is the queue length as this determines when the intersection will begin to impede discharge from a nearby intersection upstream.

Koganti (2012) acknowledges that it is normally agreed that given the normal traffic conditions, the prime objective of an intersection operation is minimising delay to improve the level of service. However, when traffic gets extremely congested, where the demand is continual and delay is intense, the preference of the system operation ought to be set to maximise the capacity of the intersection in order for many vehicles to clear the intersection: this minimises blockage. Table 3.9 shows the before upgrade field delay values and LOS for the upgraded signalised intersections for both morning and afternoon peak hours. This data was retrieved from the Somerset West traffic engineering department.

Table 3.9 Average field delay(s) retrieved (before implementation of upgrades)

Gordon Intersection			Andries Pretorius Intersection		
Peak Hour	Field Delay(s)	LOS	Peak Hour	Field Delay(s)	LOS
AM	173	F	AM	65	E
PM	60	E	PM	55	E

Field delay data was collected as per HCM 2000 technique for both sections of the study. The technique required four observes (one at each intersection), stop watches and the data sheets. The observer kept track of the number of vehicles in the queue during the red signal until they leave the intersection as well as keeping count of the total vehicles arriving during the survey period. This included vehicles which arrived at the intersection during the green signal but stopped due to the queue in front of the vehicle. For the through movement, the vehicle is qualifying to have exited the intersection when the rear axle of the vehicle crosses the stop line. For the turning movement, the vehicle is qualifying to have exited the intersection only when it has cleared the opposing traffic and starts accelerating back to the free-flow speed. In order to maintain the HCM 2000 field delay procedure, a 15 minutes observation period was adopted for this study as per the HCM. To determine the number of cycles required for each intersection, the observation period (15 minutes) was divided by the cycle length obtained at each intersection. To determine the count intervals at each intersection, the cycle length at each intersection was divided by the number of cycles. Equation 3.2 was used to compute the average time-in queue per vehicle arriving during the observation period while Equation 3.3 was used to compute acceleration/deceleration correction delay.

$$\text{Time in} - \text{queue per vehicle} = \left(I_s \frac{\sum V_{iq}}{V_{tot}}\right) \times 0.9 \qquad (3.2)$$

Where:

I_s	=	interval between vehicle-in-queue counts (s)
$\sum V_{iq}$	=	sum of vehicle-in-queue counts (veh)
V_{tot}	=	total number of vehicles arriving during the survey period (veh)
0.9	=	empirical adjustment factor (accounts for possible errors that may occur)

$$\text{Accel/Decel correction delay} = \text{FVS} \times \text{CF} \qquad (3.3)$$

Where:

FVS	=	Fraction of vehicles stopping
CF	=	Accel/Decel correction factor

Tables 3.10 and 3.11 shows the average field delay values at the intersections along the study corridor for the duration of the 15 minutes observation period, which was accomplished by adding Equation 3.2 and 3.3 at each intersection. This data was collected post the upgrade of the two intersections (Gordon and Andries Pretorius intersections).

Table 3.10 Corridor average field delay(s) and LOS during AM peak hour (collected after upgrades implementation)

Intersection ID	Intersection Name	Field average delay (s)	LOS
1	Fagan Street Intersection	21	B
2	Gordon Road Intersection	32	C
3	Andries Pretorius Street Intersection	118	F
4	Hospital Intersection	91	F

Table 3.11 Corridor average field delay(s) and LOS during PM peak hour (collected after upgrades implementation)

Intersection ID	Intersection Name	Field average delay(s)	LOS
1	Fagan Street Intersection	36	D
2	Gordon Road Intersection	46	D
3	Andries Pretorius Street Intersection	32	C
4	Hospital Intersection	29	C

3.4.5 Saturation flow

Saturation flow is a main input for optimal signal timing. Even a small variation in saturation flow value potentially affects changes in cycle length, thereby influencing the competence and operations of an urban system (Naghawi & Idewu, 2014). Figure 3.6 overviews saturation flow and all relevant parameters influencing the computation procedure of a signalised intersection saturation flow rate.

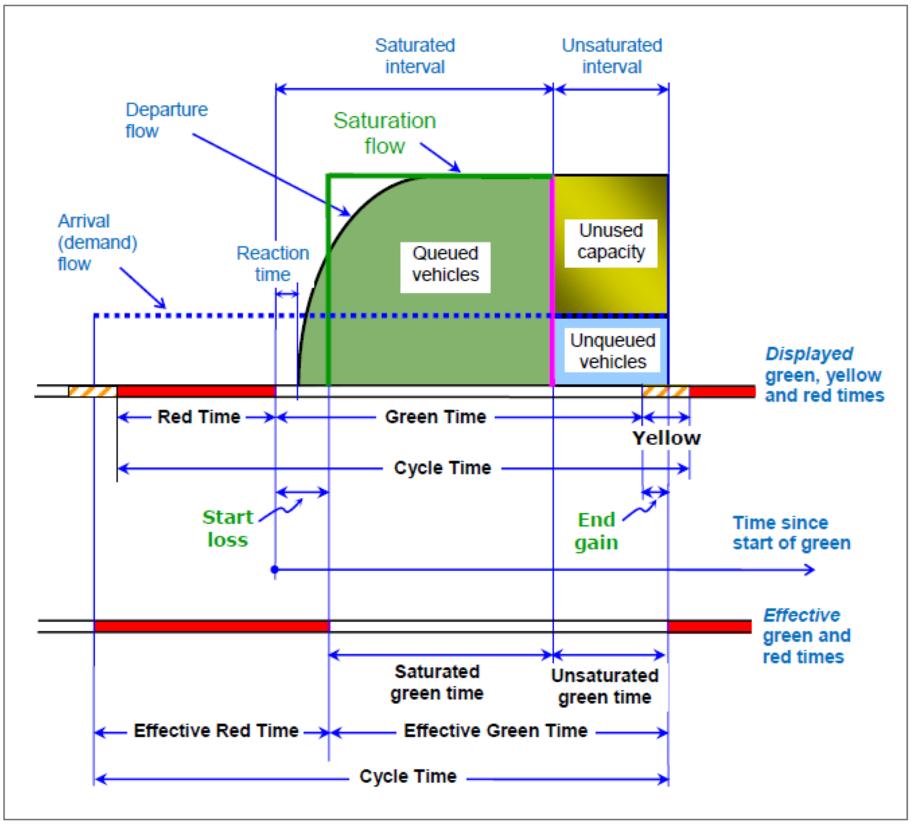


Figure 3.6 Typical saturation flow at a signalised intersection (SIDRA Intersection 8 User Guide)

As stated by Bester and Meyer (2007), saturation flow, a vital measure of performance in relation to road traffic because it illustrates traffic flow at its peak, is widely used for the design and control of signalised intersections. At an intersection, saturation flow depicts passenger car units (PCU) in any given traffic condition, especially in saturated conditions. This means that for a specific intersection, if the traffic signal would give the light indication for an entire hour at any approach of the intersection and the traffic condition would reach the capacity of the intersection approach, the saturation flow at that intersection will then be the number of PCUs that go through an hour.

$$S = 990 + 288TL + 8{,}5SL - 26{,}8G \qquad (3.4)$$

Where:

S	=	saturation flow rate (veh/h)
TL	=	number of through lanes (1 or 2)
SL	=	speed limit (60 or 80 km/h)
G	=	gradient (%)

For this study, Equation 3.4 was used for the computation of saturation flow at major approaches of the intersection, resulting in an average saturation flow of 2076 (veh/h).

3.4.6 Signal control data

Traffic signal control data is comprised of cycle lengths, signal phases, offsets and cycle splits. The cycle length is described as the time taken to shift from the green light, amber, red and back again to the green light, referred to as one complete cycle measured in seconds (s). A signal phase defines a cycle length which is dedicated to one or more movements in the network given the right of way at different intervals. An offset is described as a difference in time between the green time start at one intersection for a certain movement (e.g. right turn movement) and the beginning of the green time at the next intersection for the same movement direction. The traffic signal data for the current study, presented in Table 3.12, was not collected directly from the field, but from the Traffic Management Centre (TMC) through the assistance of Innovative Transportation Solutions (ITS). This data was required for the network coding in both models.

Table 3.12 Traffic signal timing data information

Corridor	Intersection ID	Cycle time (s) AM	Cycle time (s) PM
Somerset West	Fagan Street/Main Road/Ridgil Lane	88	96
	Gordon Street/Main Road	88	96
	Andries Pretorius/Main Road/Bizweni	120	114
	Hospital Road/Main Road	90	90

3.5 Model development

The base model in each software package was done as per the provided user guide to ensure that all relevant aspects of base model building were carried out accordingly. Before proceeding with the calibration and validation of a model, the coded network requires verification which entails the thorough evaluation of the developed network to be certain that the network is aligned with the actual conditions. The primary task in the calibration and validation of a model is to create the study network and to ascertain that the network is modelled to scale.

To achieve this, the user guide from both SIDRA and VISSIM provided directions for carrying out each step of base model development. Preparing any analysis tool for the design, evaluation and improvement of alternatives constructed generally starts with developing the model of existing conditions, verifying the model functionality in comparison with the actual facility and finally, calibrating the model to replicate existing traffic conditions. Lastly is the stage of validation, where the calibrated model parameters are implemented on the network extension, using new data which was not used during calibration such as a different counting period or different performance measures.

3.5.1 SIDRA

- Network coding

For this study, SIDRA Intersections 8 Plus version was used and according to the user guide (Figure 3.7) a flow chart in coding a typical site in SIDRA shows all required input variables. In SIDRA, each intersection must be modelled individually and then processed (run the intersection analysis) to check for errors or warnings. After modelling all the study intersections in isolation, the intersections were subsequently added in the network section for analysis. It should be noted that this version only allowed the addition of two intersections in the network section; therefore, it was decided to split the study corridor segment into three networks (i.e. the first network includes intersections 1 & 2, second network includes intersection 2 & 3 and the third intersection includes intersections 3 & 4). Figures 3.8 and 3.9 show two of the study intersections coded in SIDRA.

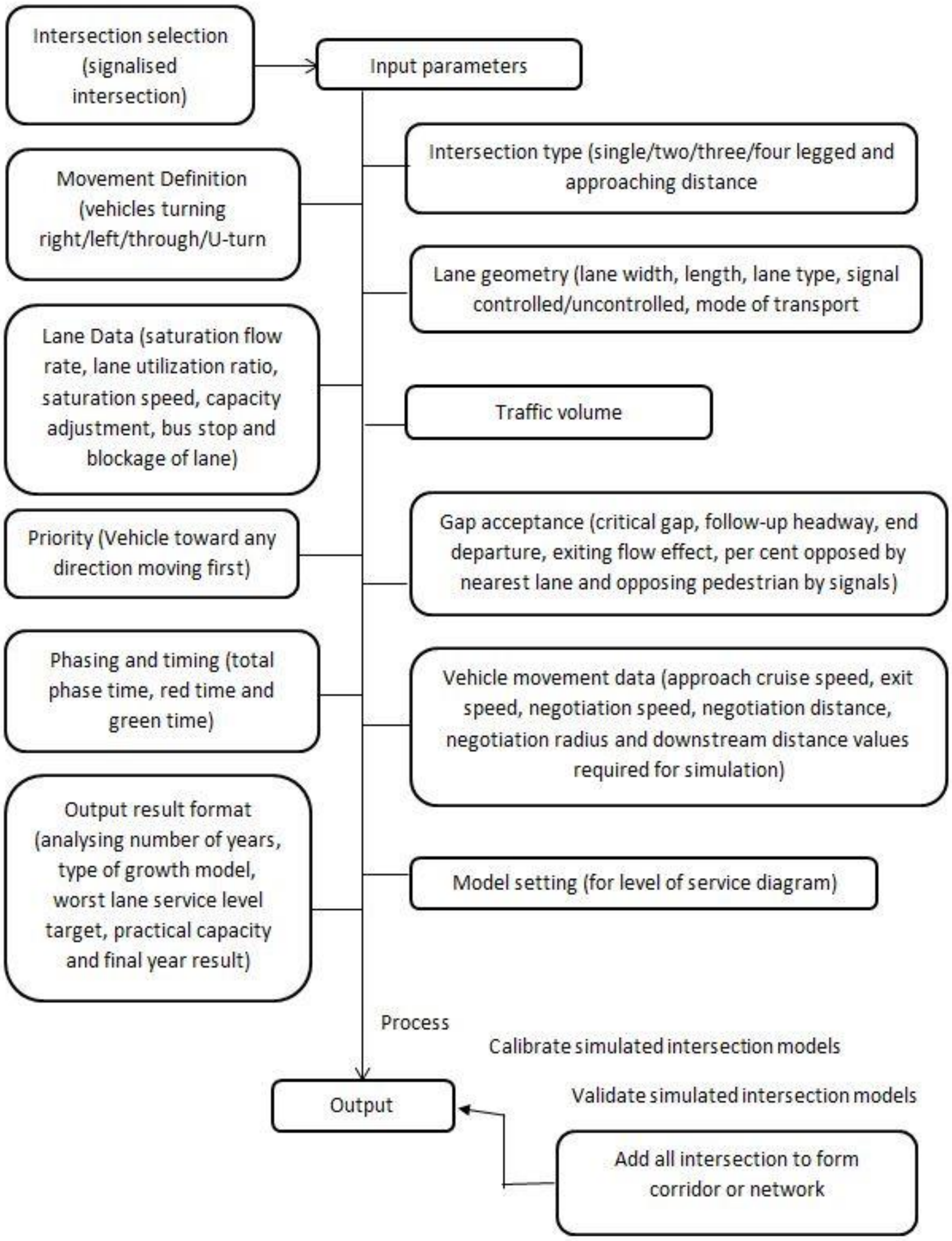


Figure 3.7 Typical network coding in SIDRA (SIDRA Intersection 8 User Guide)

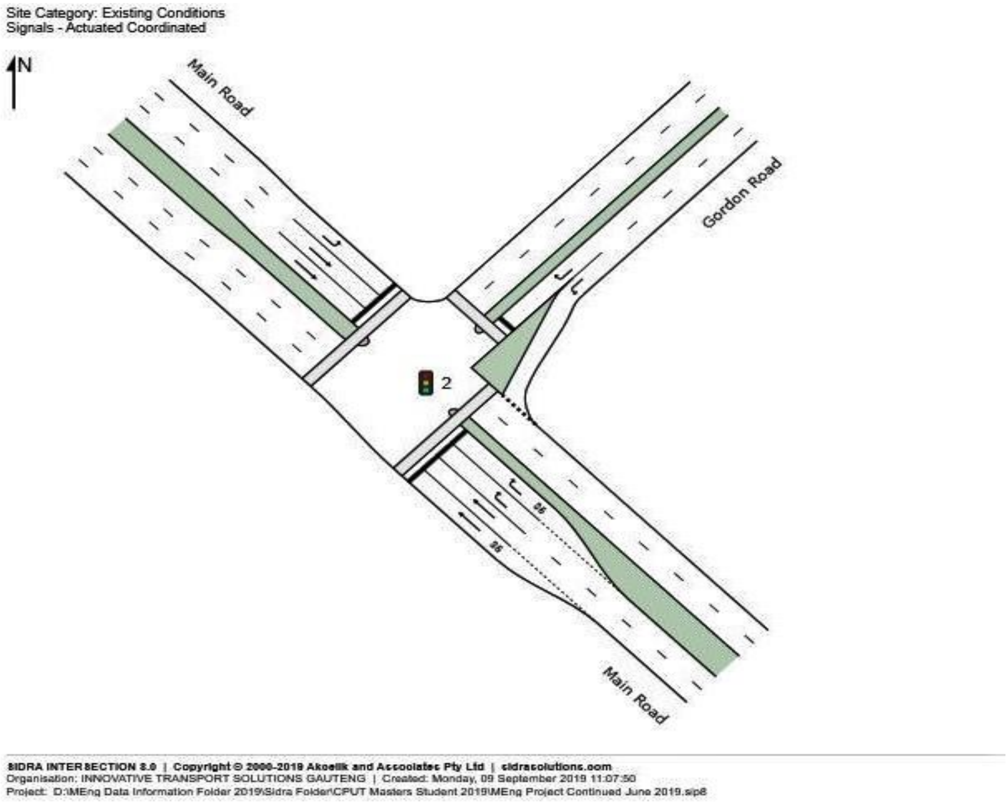


Figure 3.8 Gordon intersection (intersection 2) modelled in SIDRA

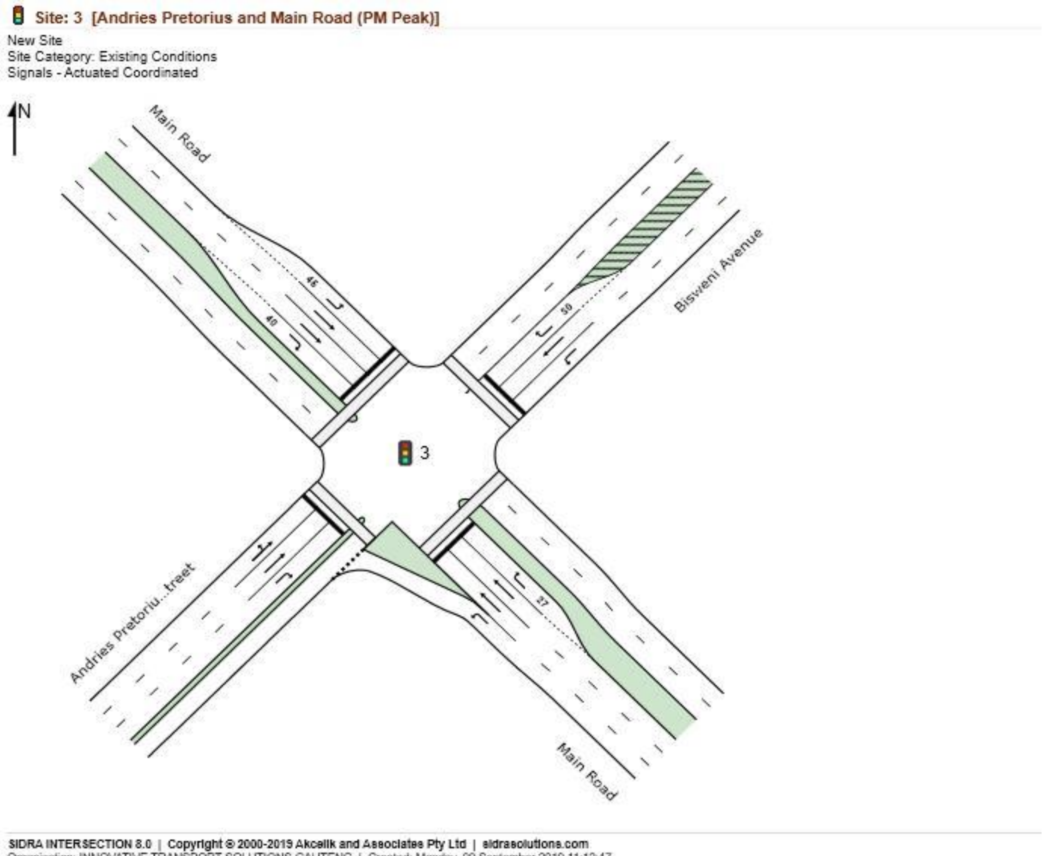


Figure 3.9 Andries Pretorius intersection (intersection 3) modelled in SIDRA

- Calibration

For this study, manual calibration was used, whereby parameters analysed for a major impact on driver behaviour were adjusted as proposed in the reviewed literature and in line with the field-collected data, where applicable. As a result, only parameters relevant to delay and travel time were calibrated; specifically, saturation flow rate and lane widths at each intersection. All other variables were maintained at default settings due to data collection resource restrictions.

3.5.2 VISSIM

- Network coding

Unlike in SIDRA, network coding in VISSIM was easier as all intersections were coded simultaneous, though the process was time consuming. VISSIM provides a background map for the network site, making it easy to model the intersection, network or corridor as per-existing conditions. Table 3.13 provides detailed information regarding the VISSIM model development data which was considered for this study, while Figures 3.10 and 3.11 show two study intersections modelled in VISSIM using links and connectors. For VISSIM, the background map allowed for a precise trace of the network for this study. A series of links and connectors created the network, whereby links follow the design of the road (straight or curvature) and the connectors connect the links, modelling the necessary lane extensions and contractions as well as turning areas in the network.

In VISSIM, several simulations run assessed the model's functionality. The verification stage revealed coding errors such as vehicle inputs at a few links not being completed, vehicles arriving and being unable to find the next link. It should be noted that coding errors picked up by the model represent abnormal vehicular movements which need to be rectified accordingly. For the VISSIM model, one simulation run was set to 1 hour and 30 minutes and took an average of 20 to 30 minutes to complete. The initial 15 minutes of the simulation run was to warm up the network, and the last 15 minutes was an allowance to confirm with certainty that there were vehicles in the network for the 1-hour evaluation period on which results were based.

Table 3.13 VISSIM model development data

Major Category	Data Type
Network data	Links with starts and endpoints Link lengths Number of lanes Lane storage length for turning movements Connectors between links to model turning movements Position of signal heads
Traffic volume data	Through movements and turning volume counts Vehicle composition Vehicle length
Signal timing control	Cycle length Splits Phase sequence
Measurement data used to compare with simulated results	Travel time Delay Link average speed

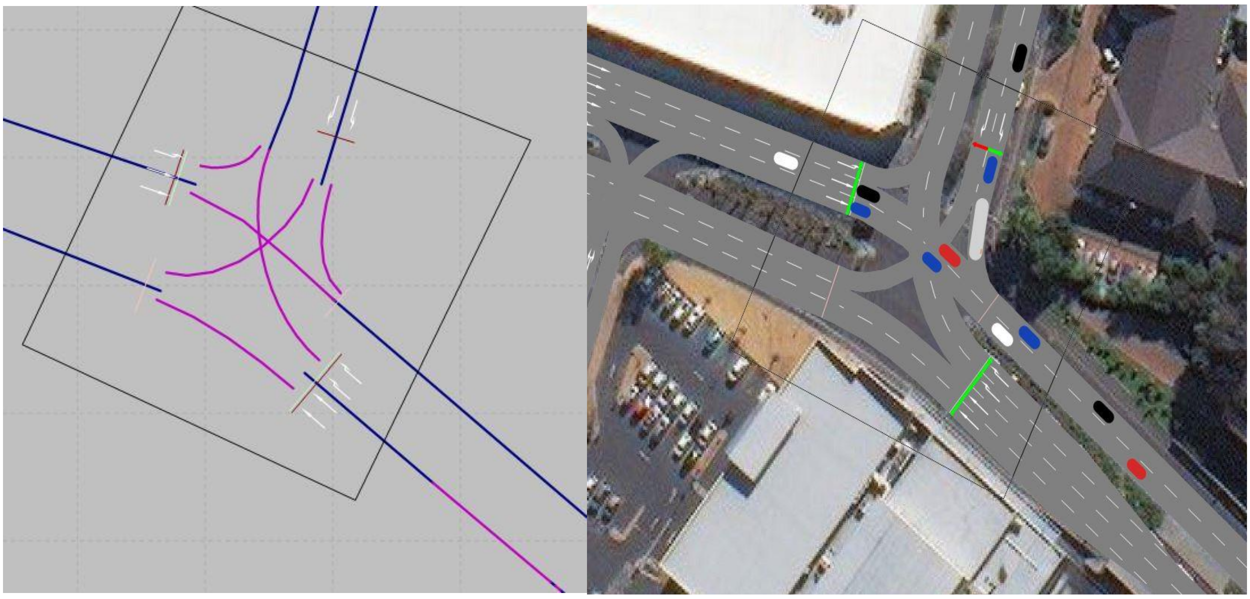


Figure 3.10 Gordon intersection (intersection 2) modelled in VISSIM

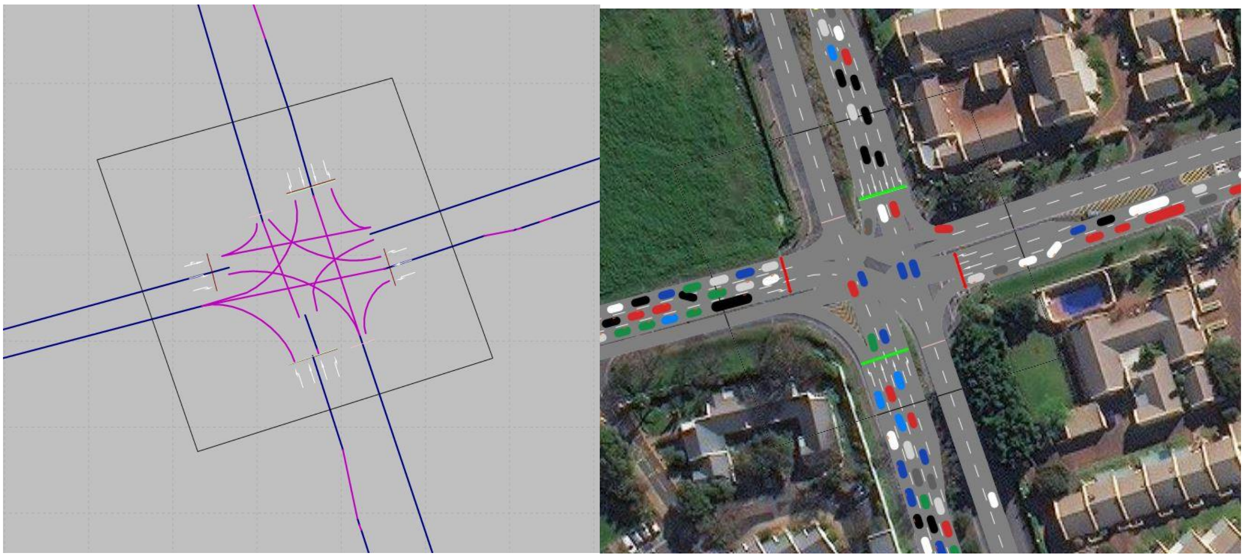
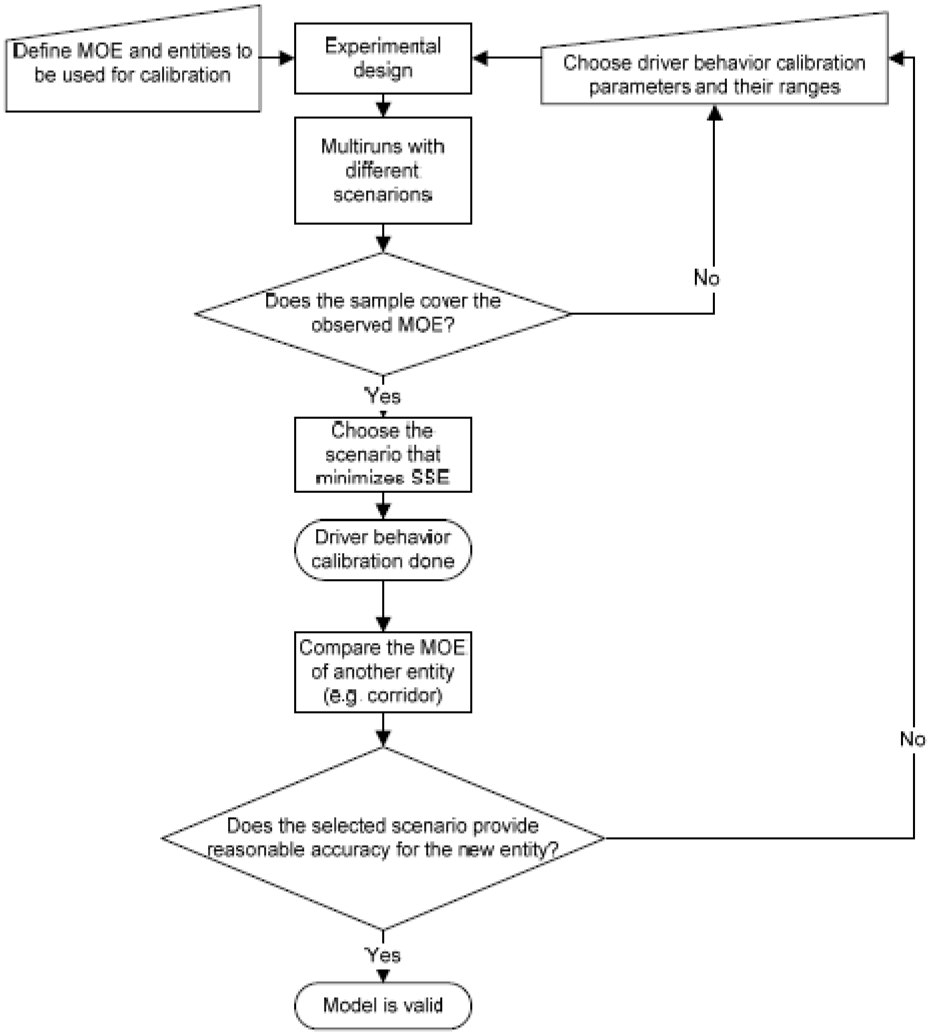


Figure 3.11 Andries Pretorius intersection (intersection 3) modelled in VISSIM

- Calibration

With respect to the calibration process, unlike traffic analytical models, traffic simulation models have various factors for characterising and reproducing prevailing traffic conditions, such as traffic flow characteristics and driver behaviour. VISSIM contains default values for all parameters (based on the traffic conditions of the model's country of origin), as shown in Table 3.14. However, only driver behaviour default parameters are shown with reference to this study. Unfortunately, the VISSIM simulation model user manual does not provide enough information suitability of the default parameters (traffic conditions and operations on which the default parameters are based), leaving the user with the obligation to confirming that adequate modifications are made not based entirely on engineering discretion, but based on field measured data. Calibration of simulation model individual parameters, under normal situations, is bound to improve the model's capability to produce estimates that closely represent field observed conditions, with an allowable error margin. Figure 3.12 shows the calibration and validation procedure adopted for the corridor segment analysis.



*Sum of the Squared Errors (SSE)

Figure 3.12 Proposed calibration and validation procedure

Table 3.14 Default parameters of Wiedemann 74 used in VISSIM

<table>
<tr><td rowspan="4">Car following model</td><td colspan="2">Look ahead distance</td><td>0,00 m - 250,00 m</td></tr>
<tr><td colspan="2">Average standstill distance</td><td>2,00 m</td></tr>
<tr><td colspan="2">Additive part of a desired safety distance</td><td>2,00 m</td></tr>
<tr><td colspan="2">Multiple parts of the desired safety distance</td><td>3,00 m</td></tr>
<tr><td rowspan="3">Lane change</td><td></td><td>Own</td><td>Trailing Vehicle</td></tr>
<tr><td>Max Deceleration</td><td>-4,00 m/s</td><td>-3,00 m/s</td></tr>
<tr><td>Accepted Deceleration</td><td>-1,00 m/s</td><td>-1,00 m/s</td></tr>
<tr><td>General behaviour</td><td colspan="3">Free lane selection</td></tr>
<tr><td>Lateral behaviour</td><td colspan="3">Desired position at free flow: middle of the lane</td></tr>
</table>

The study focused on the parameters that deal with the driver behaviour as detailed in Table 3.15. Other parameters such as acceleration and deceleration were not considered during calibration because the vehicle type, vehicle model and vehicle class in the VISSIM model were found to be similar to those used locally.

Table 3.15 Parameters considered for calibration

Parameters	VISSIM Default Values
Number of observation of vehicles	2
Additive part of desired safety distance	2
Multiplicative part of desired safety	3
Amber signal decision model	Continuous
Lane change distance	200 m

Observed vehicles: This parameter is associated with the urban car-following model. The number of the observed vehicles influence how well the vehicles in the network can anticipate other vehicles movement, thus react appropriately.

Additive part of desired safety distance and multiplicative part of desired safety distance: This parameter influence the calculation of the vehicle safety distance. These are considered the primary parameters that impact the capacity flow.

Amber signal decision model: This model describes the driver's response to the amber signal. In VISSIM, there are two options of this model, the continuous check and the one decision.

- Continuous check: For this option, vehicles assume that the amber light remains amber for 2 seconds and the vehicle continuously decides whether to progress at each time step from there on until passing the signal head.

- One Decision: For this option, VISSIM computes the chance of the driver stopping at the amber light. This option is assumed to produce more accurate outcomes, provided the number of the Observed vehicles is increased appropriately. This is because a signal head in VISSIM internally is modelled

Lane changing distance: This parameter is often utilised together with the emergency stopping distance parameter in order to model the driver's behaviour in maintaining their desired routes. The lane change distance describes the distance from which the vehicles will start to attempt changing lanes. The VISSIM default lane change distance value is 200 m, however, acceptable lane change values range from 150 m to 300 m. The selection of this values was based on ensuring that a vehicle is allowed reasonable distance to make lane change before reaching the intersection.

3.6 Analysis and presentation of results

To evaluate the effect of the implemented geometric upgrades on the performance improvement of the intersections, a comparison between pre-upgrade, modelled and post-upgrade was carried out. The comparison was with respect to obtained delay and LOS at each intersection. For the urban corridor performance evaluation, the two models (SIDRA and VISSIM) were compared with field measured performance. The performance evaluation criteria of the corridor were based on delay, LOS and travel time. To assess the goodness of fit and reliability of the operational performance estimated by both models, the following statistical measures were used – Relative Error (RE) between model predicted and field measured data – and calculated using Equation 3.5. The MAPE (measure of model prediction accuracy) and RMSE values (standard deviation of the model prediction errors) were also calculated using the field and predicted operational performance data obtained from both models by Equations 3.6 and 3.7. The correlation between field and model predicted operational performance was evaluated using the Spearman's rank correlation coefficient (*p*). Equation 3.8 was applied to calculate the Spearman's rank correlation coefficient.

$$\mathrm{RE} = \frac{\mathrm{OBS} - \mathrm{SIM}}{\mathrm{OBS}} \times 100\% \tag{3.5}$$

Where:

RE	=	relative error (%)
OBS	=	observed data (field data)
SIM	=	simulated data

$$\mathrm{MAPE} = \frac{1}{\mathrm{N}} \sum_{\mathrm{i}=1}^{\mathrm{N}} \left| \frac{\mathrm{OBS} - \mathrm{SIM}}{\mathrm{OBS}} \right| \times 100 \tag{3.6}$$

Where:

MAPE	=	mean absolute percentage error (%)
OBS	=	observed data (Field data)
SIM	=	simulated data
N	=	sample size

$$\mathrm{RMSE} = \sqrt{\frac{\sum_{\mathrm{i}-1}^{\mathrm{N}} (\mathrm{SIM} - \mathrm{OBS})^2}{\mathrm{N}}} \tag{3.7}$$

Where:

RMSE	=	root mean square error (%)
OBS	=	observed data (field data)
SIM	=	simulated data
N	=	sample size

$$p = 1 - \frac{6 \sum \mathrm{d}^2}{\mathrm{n}(\mathrm{n}^2 - 1)} \tag{3.8}$$

Where:

P	=	Spearman's rank correlation coefficient
n	=	number of cases
d	=	difference in paired ranks

3.7 Summary

Traffic congestion has been a burgeoning aggravation in most metropolitan areas worldwide due to issues surrounding urbanisation. Instead of relying on conventional solutions like the construction of new roads, traffic analysis tools such as analytical and simulation models can assist in improving the existing road network and vehicle movement within the road network. To assess the appropriate use of the models at different networks and intersections, the effect on performance improvement predicted by the models needs to be evaluated. It is noted that these models' built-in default parameters such as driver behaviour are based on traffic conditions which differ from South Africa's local conditions because none of the models was developed in South Africa. Consequently, calibration and validation of these models for local application are pivotal to ensure that the actual performance of the road network is obtained. This chapter has discussed the flow process of the work and the data collected. The calibration procedures adopted to develop the base models of the software (both SIDRA and VISSIM) are also detailed. The results will be discussed in Chapter 4.

4 Results

This chapter presents the results obtained from the investigations detailed in Section 3.2. The effect of geometric changes on performance improvement (based on delay and LOS) was assessed for both morning and afternoon peak hours at two signalised intersections. Two models – SIDRA (an analytical model) and VISSIM (a micro-simulation model) – were calibrated for their applicability to local traffic conditions on an urban corridor with four successive signalised intersections. The results presented the comparison of the models and field measure data with respect to delay, LOS and travel time. The model that demonstrated better performance amongst the two was then validated. The validated model was later used to perform a holistic performance analysis of the study network.

4.1 Model prediction accuracy

The geometric upgrades of two signalised intersections, Gordon intersection and Andries Pretorius intersection, were implemented based on operational performance improvement alternatives modelled in SIDRA. It is noteworthy to mention that the intersections were evaluated in isolation, although they are both located in a corridor of successive intersections. From the alternatives provided by the model, the following were implemented:

- Gordon Intersection (Intersection 2)

The main road (M9) Northbound dedicated right turn lane storage was increased to 35 meters and the through lane still on the Northbound was converted to a second dedicated right turn lane. On Gordon Road approach, a southbound left-turning slip lane was also introduced.

- Andries Pretorius Intersection (Intersection 3)

On the Eastbound approach of Andries Pretorius, the existing median was removed. Then the approach lane configuration was changed to two right-turning lanes and a shared through-and-left-turning lane. The left turn slip lane radius was improved to properly merge the main road (M9) into Andries Pretorius existing approach. A dedicated right-turning lane of 25 meters long was constructed on both the northbound and southbound approaches along the main road (M9).

4.1.1 Gordon Intersection (Intersection 2)

For both the AM and PM peak hours, the pre-upgrade modelled performance improvement predictions and post-upgrade delay and LOS at the intersection results were obtained, as presented in Table 4.1 and Table 4.2. The comparison between the SIDRA predicted improvement results with both SIDRA pre-upgrade and post-upgrade results are shown in Figures 4.1 and 4.2, respectively.

Table 4.1 Gordon intersection obtained average delays for both peak hours

Condition	AM peak delay (s)	LOS	PM peak delay (s)	LOS
Pre-upgrade	173	F	60	E
Modelled (SIDRA)	12	B	14	B
Post-upgrade	24	C	38	D

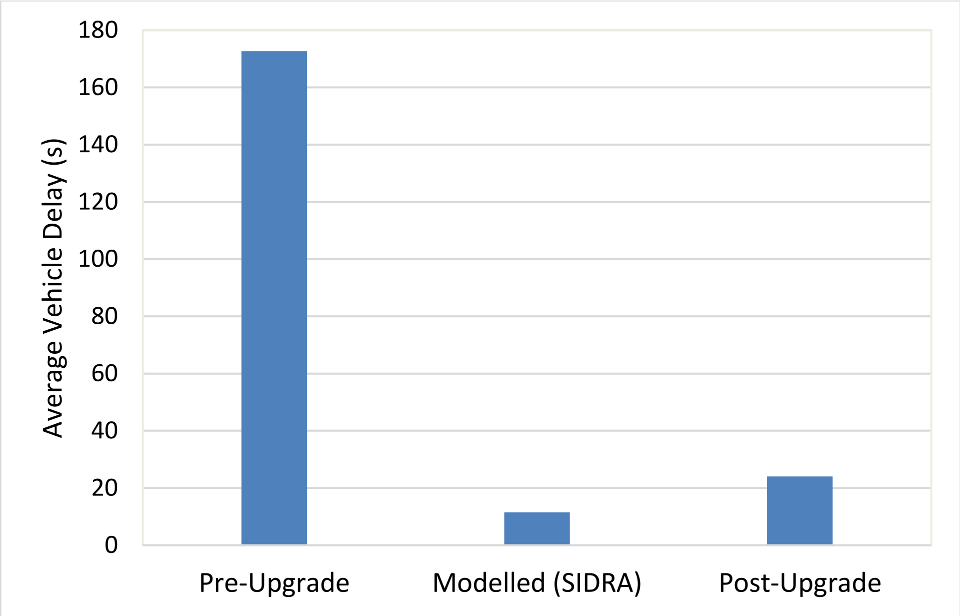


Figure 4.1 Gordon intersections average delays obtained for three conditions of the intersection (AM peak)

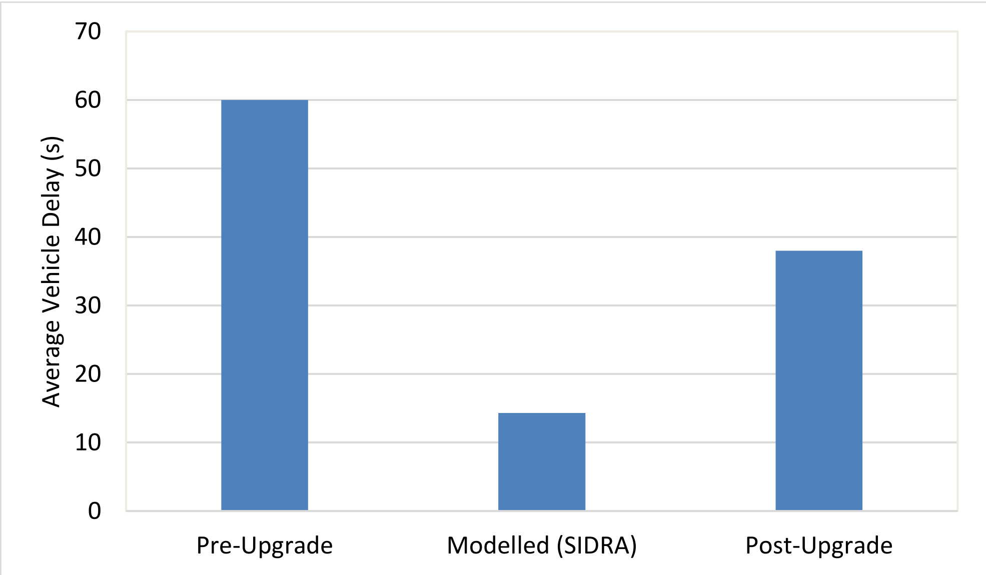


Figure 4.2 Gordon intersection average delays at different conditions of the intersection (PM peak)

4.1.2 Andries Pretorius Intersection (Intersection 3)

The results for both AM and PM peak hours for the pre-upgrade modelled performance improvement predictions and post-upgrade delay and LOS at the second signalised intersection are presented in Table 4.2, while the comparison between model-predicted improvement results with both SIDRA pre-upgrade and post-upgrade results are shown in Figures 4.3 and 4.4, respectively.

Table 4.2 Delay and LOS for both the AM and PM peak hours

Condition	AM Peak delay (s)	LOS	PM Peak delay (s)	LOS
Pre-Upgrade	65	E	55	E
Modelled (SIDRA)	19	B	18	B
Post-Upgrade	96	F	53	D

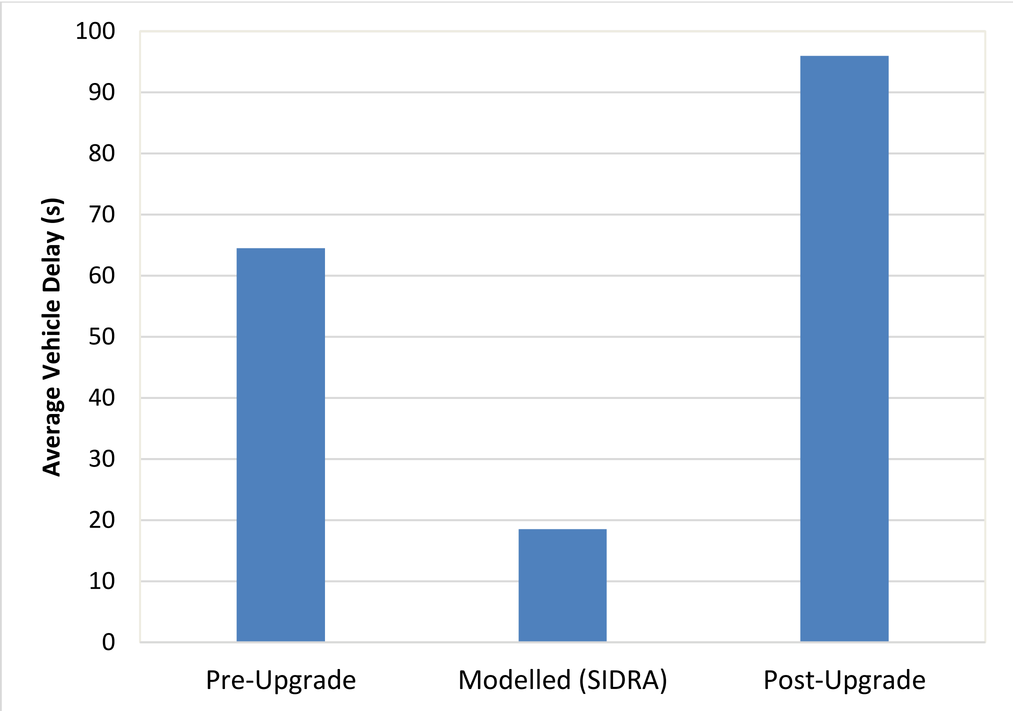


Figure 4.3 Difference in delay for the three conditions of the intersection (AM peak)

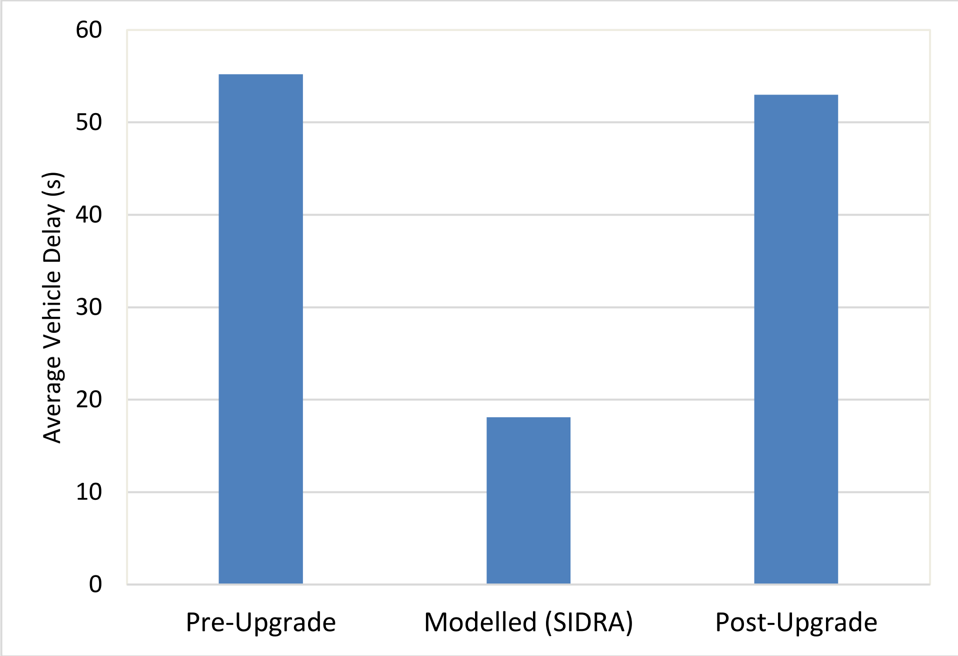


Figure 4.4 Difference in delay obtained for the three conditions of the intersection (PM peak)

4.2 Network analysis-corridor segment

According to Tawfeek *et al.* (2018), the evaluation of corridors with interrupted conditions (consisting of intersections) still needs further exploration to investigate the models' capabilities in handling such conditions to validate their applicability. A local corridor analysis was carried out on a network consisting of four successively located signalised intersections (inclusive of the two upgraded intersections 2 and 3 discussed in Section 4.1) using both models. Both SIDRA and VISSIM were run with default parameters where the estimated outputs were compared to field observed data. Thereafter, the two models were calibrated accordingly. After calibration, the models' results were compared with field data post the upgrade of the two intersections. The final stage was then to validate the adjusted parameters of the model which best represented the field observed performance. This occurred by running the model with a new data set which was not used in the calibration process. The correlation between the model predictions and field-collected data are shown in the sections below. In order to evaluate the accuracy of the model's operational performance prediction, statistical methods (MAPE, RMSE, Spearman's rank correlation coefficient [*p]*) were used. Model overestimation was represented by a positive difference, while model underestimation was represented by a negative difference.

4.2.1 Default parameters

For both models, default parameters identified to impact the driver behaviour were used to evaluate the suitability of applying the model without calibration for local conditions. The estimated measures of performance such as delay, LOS and travel time output results were compared with the field observed data. It should be noted that the two model's travel time computations were quite different; hence, the field data was presented in a way that accommodates each model's computation technique. With respect to delay estimates, SIDRA provides average control delay, while VISSIM provides average vehicle delays. Even though the two models adopt the HCM (2000) method of LOS classification, the notation used was found to be different. SIDRA uses letters (A-F) as outlined in the HCM (2000), while VISSIM uses numbers (1-6).

SIDRA – Delay & LOS

The average delay and LOS results computed by SIDRA, together with the field measured delay times at each intersection for the PM peak hour, are displayed in Table 4.3 and graphically depicted in Figure 4.5. The model showed better performance at intersection 3 where a close delay estimation was observed, showing the least RE = -22% between field data and SIDRA computations. The field and SIDRA LOS for all the intersections in the network range from C to D. These are acceptable according to the HCM (2000) LOS criterion.

Table 4.3 Delay and LOS estimated by SIDRA

Intersection ID	Field Delay (s)	LOS	SIDRA Delay (s)	LOS	Delay difference (s)	RE (%)
1	36	C	21	C	15	42
2	46	D	28	C	18	39
3	32	C	39	D	-7	-22
4	29	C	43	D	-14	-48

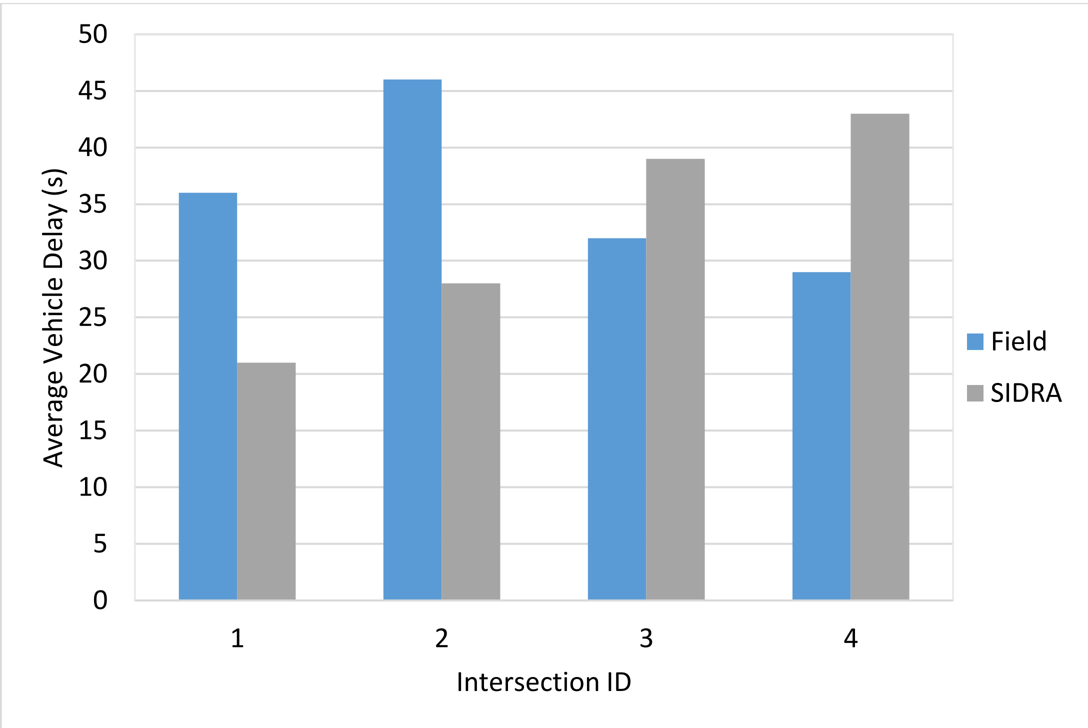


Figure 4.5 Comparison between field and SIDRA estimated delays (using default parameters)

SIDRA – Travel Time

The average travel time results computed by SIDRA, together with the field measured travel times at each intersection for the PM peak hour, are shown in Table 4.4 and graphically presented in Figure 4.6. Intersection 2 provided the least RE percentage value of 36%.

Table 4.4 Travel time estimated by SIDRA

Intersection ID	Intersection Name	Field travel time (s)	SIDRA travel time (s)	Travel time difference (s)	RE (%)
1	Fagan Street Intersection	27	16	11	41
2	Gordon Road Intersection	55	35	20	36
3	Andries Pretorius Street Intersection	28	64	-36	-129
4	Hospital Intersection	24	61	-37	-154

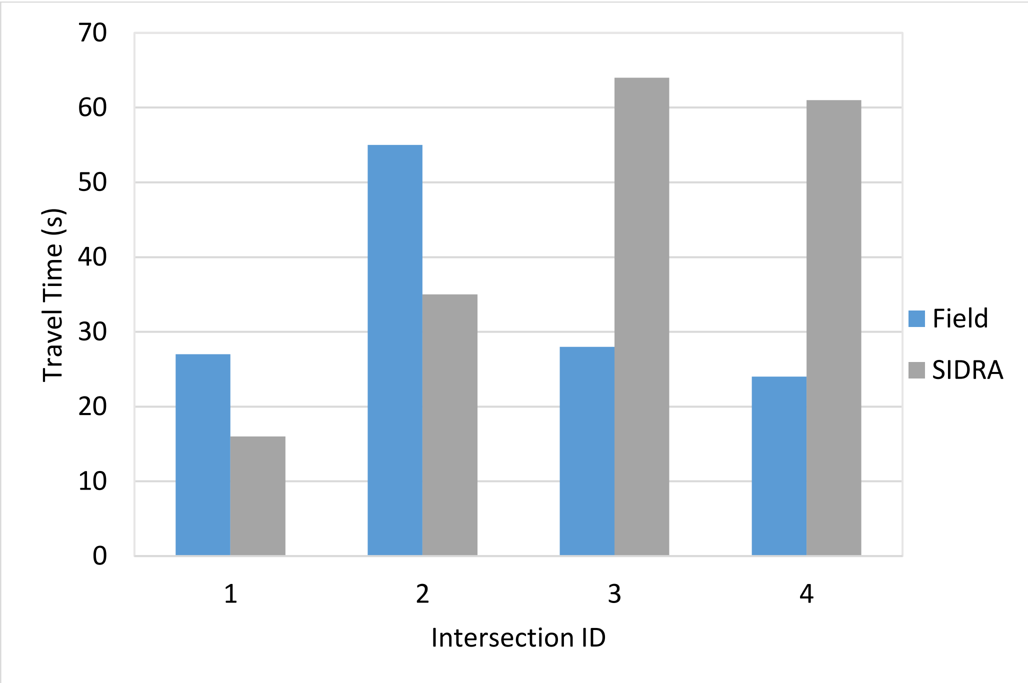


Figure 4.6 Comparison between field and SIDRA estimated travel times (using default parameters)

VISSIM – Delay & LOS

A simulation run was performed for the network with the afternoon peak hour data set using the model driving behaviour default parameters at various seed outputs for all performance measures. The computed average from the five simulation runs for the performance measures were compared to field observed data. Comparison of the results is displayed in Table 4.5 and graphically depicted in Figure 4.7. Intersections 2 and 3 showed the least RE % values of -17 and -19 respectively, demonstrating that the model closely estimated delay in comparison to field measured delay at these two intersections. The LOS as per the delay estimated by VISSIM was observed to be within the range of the field LOS, satisfying the acceptable performance LOS criterion by the HCM (2000).

Table 4.5 VISSIM estimated delays and LOS

Intersection ID	Field delay (s)	LOS	VISSIM delay (s)	LOS	Delay difference (s)	RE (%)
1	36	C	54	4 (D)	-18	-50
2	46	D	54	4 (D)	-8	-17
3	32	C	38	4 (D)	-6	-19
4	29	C	22	3 (C)	7	24

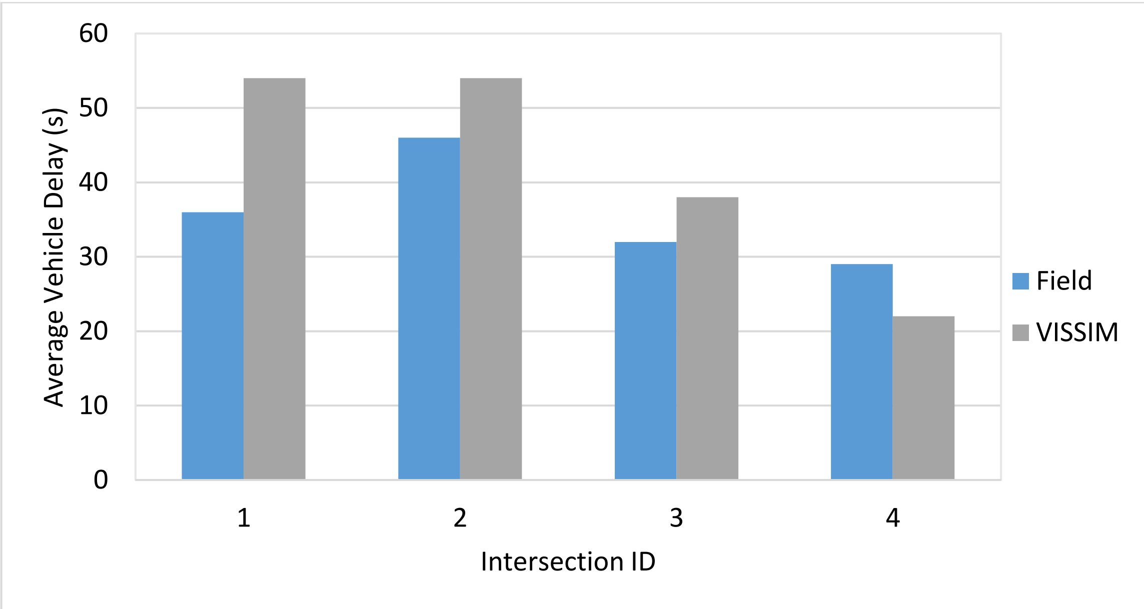


Figure 4.7 Comparison between field and VISSIM estimated delay (using default parameters)

VISSIM – Travel Time

According to Reza (2013), there are no guidelines developed specifically to delineate the acceptable error. This study will, therefore, use the guidelines developed by Fred *et al.* (2002) and cited frequently in the literature which states that the simulated travel time results should be within one standard deviation of the observed performance measures. The model results should be within +/- one deviation from the field recorded travel time for each link.

According to Table 4.6, only site number 6 satisfied this criterion because the VISSIM estimated travel time was within 1 standard deviation of the field measured travel time. Site numbers 1 to 5 did not satisfy the adopted criterion; thus, calibration was initiated to adjust the model to local conditions for all sites to comply with the adopted criterion. Only site number 6 satisfied the adopted criterion and showed the least RE % = -12 between the field and VISSIM travel times. The comparison between the field and VISSIM estimated average travel time is graphically presented in Figure 4.8.

Table 4.6 VISSIM travel times

Site No.	Link	Distance (m)	Field travel time (s)	Standard deviation	VISSIM travel time (s)	Between 1 and -1 standard deviation	Travel time difference (s)	RE (%)
1	Fagan Street Intersection to Gordon Road Intersection	126	25	9,65	38	NO	-13	-51
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	47	5,3	53	NO	-6	-14
3	Andries Pretorius Street Intersection to Hospital Intersection	256	30	7,32	37	NO	-7	-25
4	Hospital Intersection to Andries Pretorius Street Intersection	297	43	8,57	53	NO	-10	-23
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	29	5,52	35	NO	-6	-21
6	Gordon Road Intersection to Fagan Street Intersection	94	13	2,44	15	YES	-2	-12

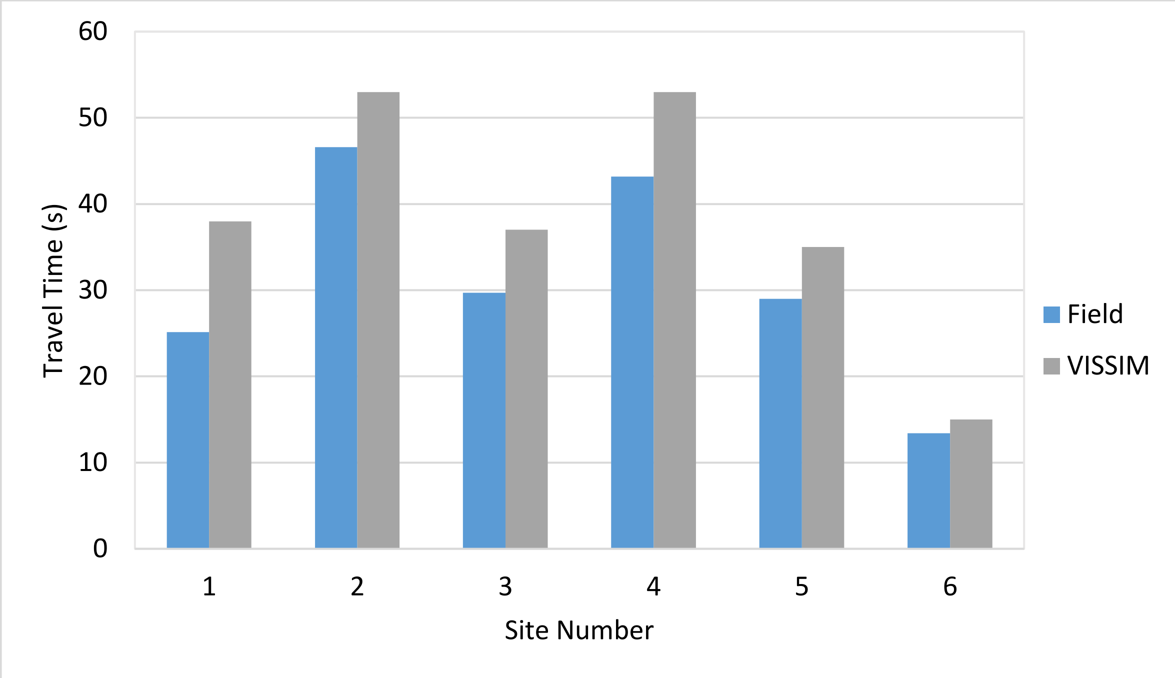


Figure 4.8 Comparison between field and VISSIM travel times (using default parameters)

4.2.2 Model calibration

SIDRA – Delay & LOS

After using the model with default parameters, it was decided to calibrate the model to acquire estimates which closely matched the obtained field measures. The obtained results for delay and LOS are detailed in Table 4.7 and are graphically depicted in Figure 4.9. The model showed the best performance at intersection 3 where close delay estimation was observed compared to other intersections with the least RE % value of -6.

Table 4.7 SIDRA calibrated average delays

Intersection ID	Field delay (s)	LOS	SIDRA delay (s)	LOS	Delay difference (s)	RE (%)
1	36	D	19	B	17	47
2	46	D	24	C	22	48
3	32	C	34	C	-2	-6
4	29	C	42	D	-13	-45

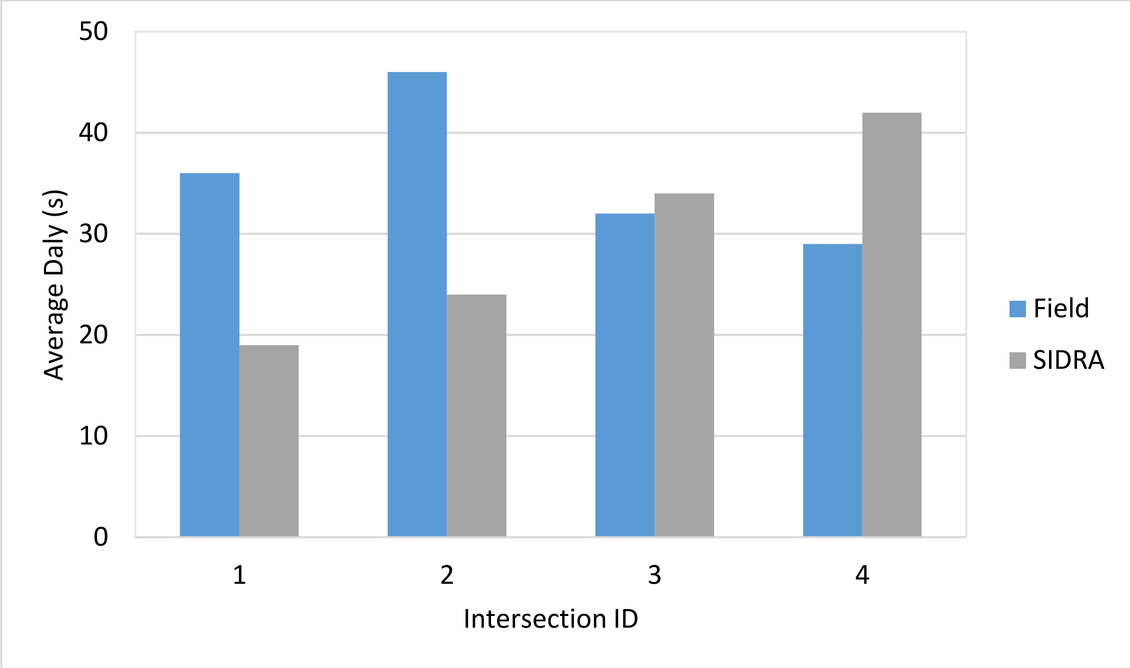


Figure 4.9 Comparison between calibrated field measure and SIDRA average delays (using calibrated parameters)

SIDRA – Travel Time

The calibrated travel time results were compared to field measured travel times at each intersection in the network, as shown in Table 4.8 and graphically depicted in Figure 4.10. The model was observed to have better performed at intersection 2, showing the least RE % value of 42 between the two data sets (field travel time and SIDRA estimated travel time).

Table 4.8 SIDRA calibrated average travel times

Intersection ID	Intersection Name	Field travel time (s)	SIDRA travel time (s)	Travel time difference (s)	RE (%)
1	Fagan Street Intersection	27	14	13	48
2	Gordon Road Intersection	55	32	23	42
3	Andries Pretorius Street Intersection	28	56	-28	-100
4	Hospital Intersection	24	61	-37	-154

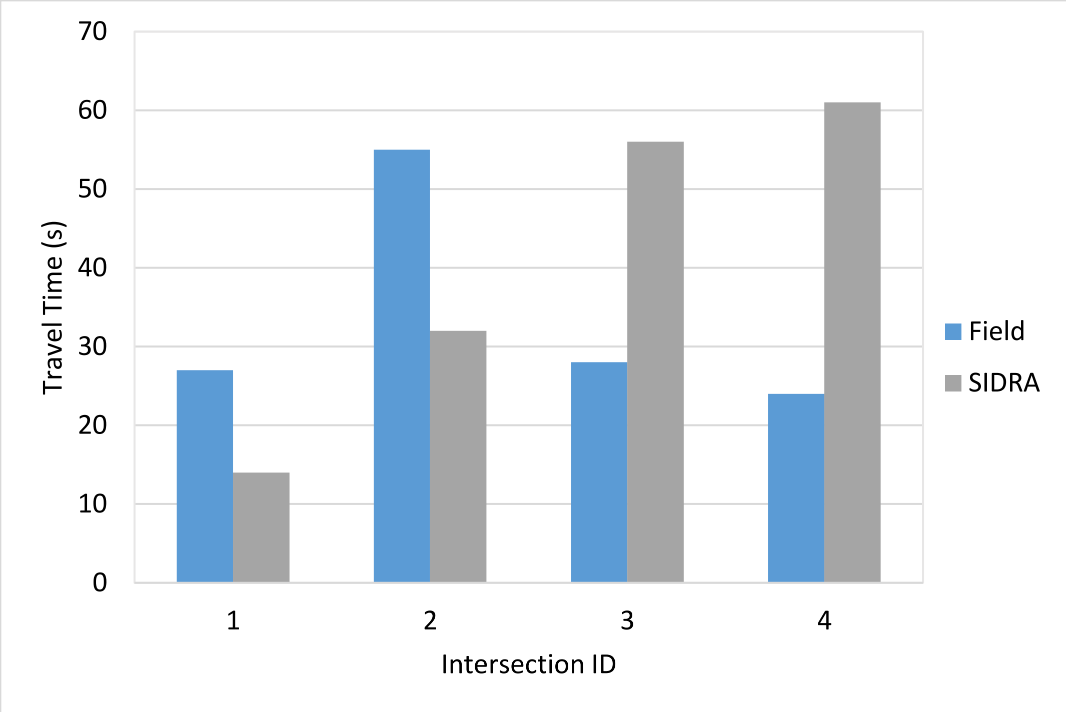


Figure 4.10 Comparison between field and SIDRA travel times (using calibrated parameters)

VISSIM – Delay & LOS

After running the simulation model with default parameters, field observed data and model output results did not match closely; therefore, the calibration of the model was then considered. Multiple runs were initiated to establish the appropriate parameter settings for the local network (refer to Appendix F1). Table 4.9 shows the calibrated average delays at the intersections together with the corresponding LOS, as graphically depicted in Figure 4.11. The least difference between field and VISSIM estimated average delay was observed at intersection 2 with RE % = -4.

Table 4.9 VISSIM delays and LOS

Intersection ID	Field delay (s)	LOS	VISSIM delay (s)	LOS	Delay difference (s)	RE (%)
1	36	D	46	4	-10	-28
2	46	D	48	4	-2	-4
3	32	C	37	4	-5	-16
4	29	C	21	3	8	28

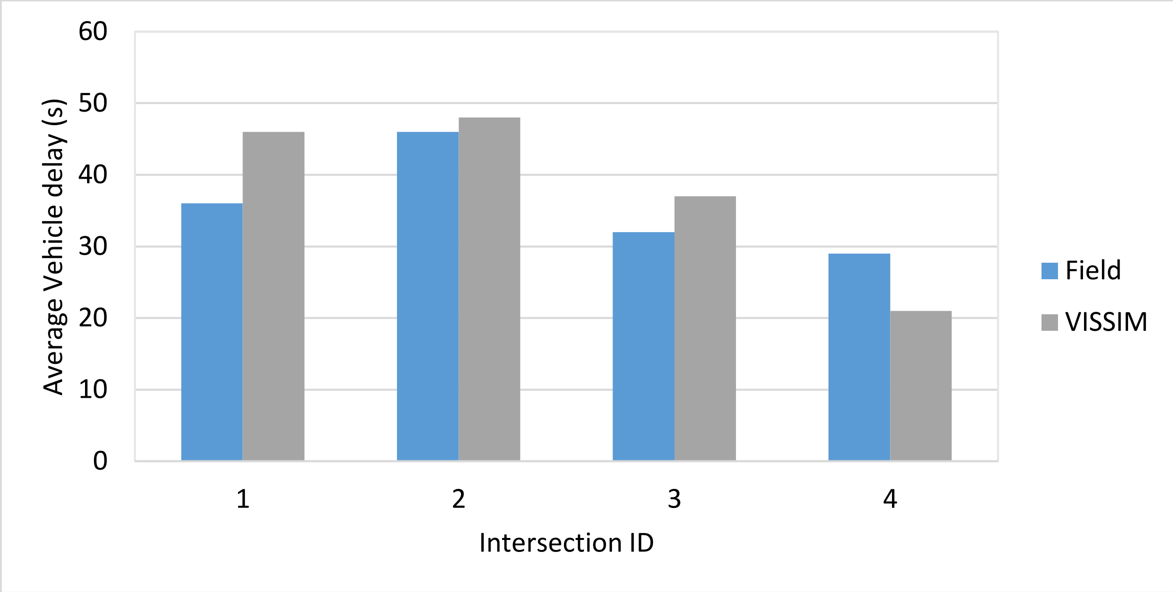


Figure 4.11 Comparison between field and VISSIM delay (using calibrated parameters)

VISSIM – Travel Time

Similar to the delay, the average travel time values estimated by the model were also compared with the field observed travel times shown in Table 4.10 and graphically depicted in Figure 4.12. As the criteria state that the model simulated results should be between one standard deviation of the field observed data (see Appendix D2), this was adopted based on literature (Ahmed, 2005; Reza, 2013) which shows that the VISSIM model computes travel time based on the vehicular movement from one intersection to the other within the network. After calibration, the VISSIM estimated travel times were within one standard deviation of the field travel times. The lowest RE % value of -8 was observed at site number 1.

Table 4.10 VISSIM calibrated average travel times

Site No.	Link	Distance (m)	Field travel time (s)	Standard deviation	VISSIM travel time (s)	Between 1 and -1 standard deviation	Travel time difference (s)	RE (%)
1	Fagan Street Intersection to Gordon Road Intersection	126	25	9,65	23	YES	2	-8
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	47	6,3	53	YES	-6	13
3	Andries Pretorius Street Intersection to Hospital Intersection	256	30	7,32	37	YES	-7	23
4	Hospital Intersection to Andries Pretorius Street Intersection	297	44	8,57	52	YES	-8	18
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	29,00	5,52	34	YES	-5	17
6	Gordon Road Intersection to Fagan Street Intersection	94	13	3,44	16	YES	-3	23

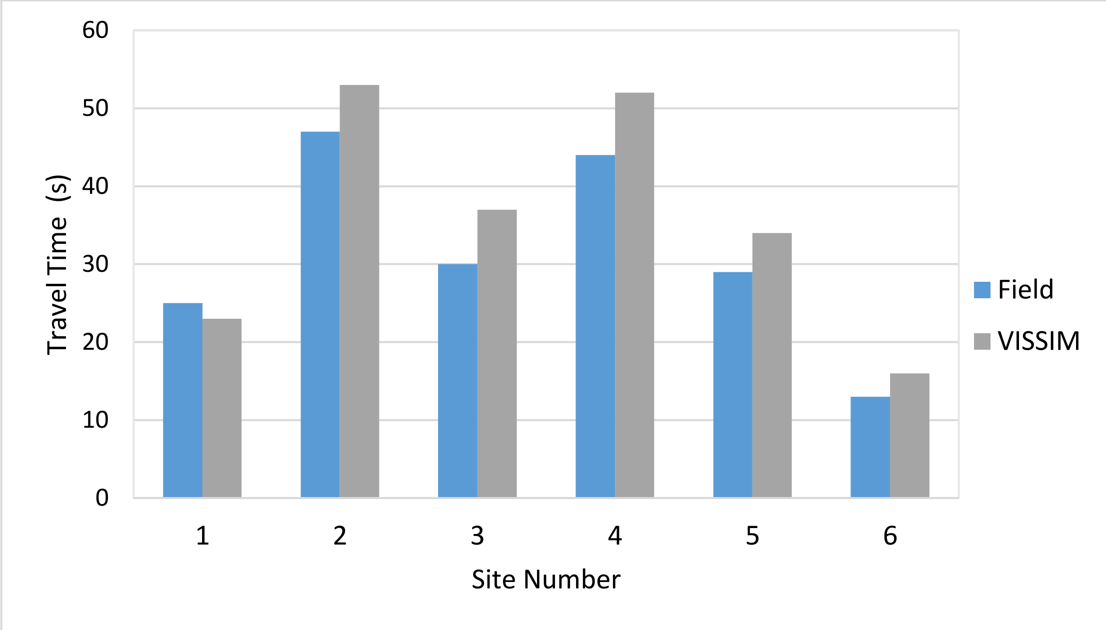


Figure 4.12 Comparison between field and VISSIM travel times (using calibrated parameters)

4.2.3 Model validation

After the calibration phase, the model that best estimated the field measured MoEs was then validated. Model validation was considered the final phase relevant to the investigation of the adequacy of the model in replicating field measured MoEs. Model validation involves testing for model rationality and model behaviour in comparison to the referent network or facility. A new data set (AM peak 07:00 – 08:00 am data) was collected on the same network for the validation of the calibrated parameters. This was to assess if the calibrated model parameters from the first data set (PM peak 16:30 – 17:30 pm data) would be applicable to a different data set. It was unnecessary to build another model as the same network used in calibration was used for the validation. However, the newly collected data was coded in the model for the representation of the AM peak traffic condition. The model was not recalibrated based on driver behaviour, as the calibrated parameter values from the PM peak data were used to run the new data set.

VISSIM – Delay

The developed model in VISSIM was then validated with a new data set. The calibration yielded better results in comparison to the SIDRA model. Table 4.11 presents the average delay and LOS results obtained from the AM peak hour of the corridor segment and Figure 4.13 depicts the graphical comparison of the field measured and VISSIM predicted delay times. The smallest difference between field and VISSIM predicted average delay was observed at intersections 2 and 3 with lowest RE percentage values of -9% and -8%, respectively. The LOS showed intersections 3 and 4 to be operational under worst movement conditions. LOS F was observed at both intersections.

Table 4.11 VISSIM validated delays and LOS

Intersection ID	Field delay (s)	LOS	VISSIM delay (s)	LOS	Delay difference (s)	RE (%)
1	21	B	13	2(B)	8	38
2	32	C	29	3(C)	3	9
3	118	F	128	6(F)	-10	-8
4	91	F	109	6(F)	-18	-20

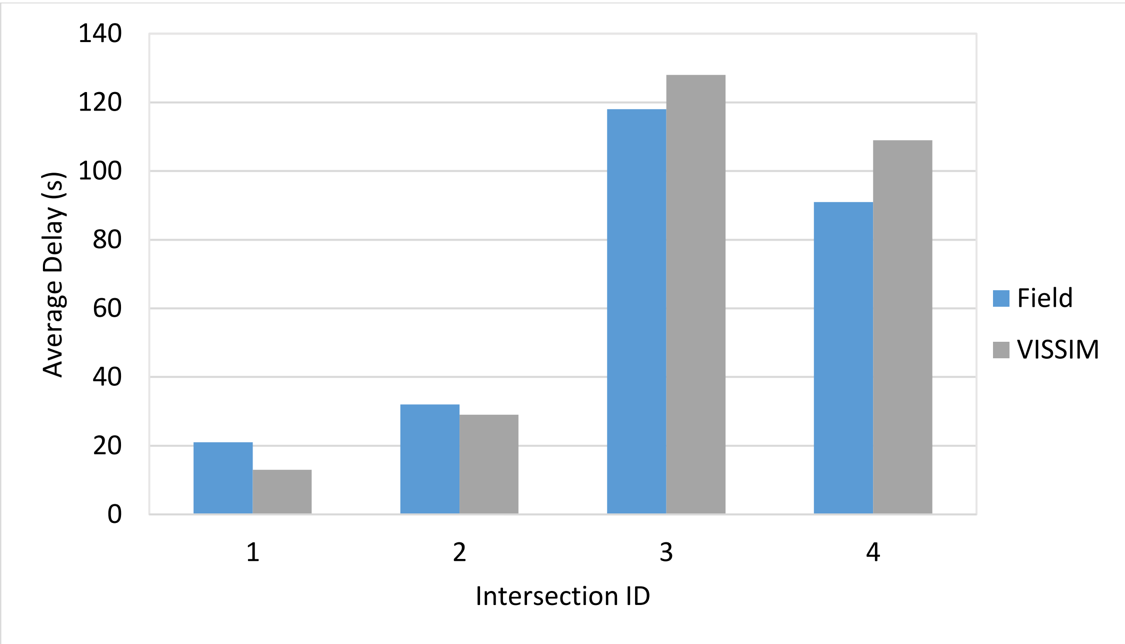


Figure 4.13 Comparison between field and VISSIM average delays (using adjusted parameters)

VISSIM – Travel Time

Table 4.12 shows the travel time results from the VISSIM model, together with the field measure travel times, with the comparison graphically depicted in Figure 4.14. Findings show that the average travel times obtained from the new data set were all within the validation criterion where the simulated results must be between one standard deviation of the field observed data. The lowest RE values of -3% and -4% were observed at site numbers 1 and 2 respectively, showing close travel time prediction by the model to the field measured travel time.

Table 4.12 VISSIM validated average travel times

Site. No.	Link	Distance (m)	Field travel time (s)	Standard deviation	VISSIM travel time (s)	Between 1 and -1 standard deviation	Difference	RE (%)
1	Fagan Street Intersection to Gordon Road Intersection	126	31	4,65	32	YES	-1	-3
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	50	4,30	52	YES	-2	-4
3	Andries Pretorius Street Intersection to Hospital Intersection	256	32	7,32	38	YES	-6	-19
4	Hospital Intersection to Andries Pretorius Street Intersection	297	72	10,03	82	YES	-10	-14
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	30	5,52	35	YES	-5	-17
6	Gordon Road Intersection to Fagan Street Intersection	94	14	2,44	15	YES	-1	-7

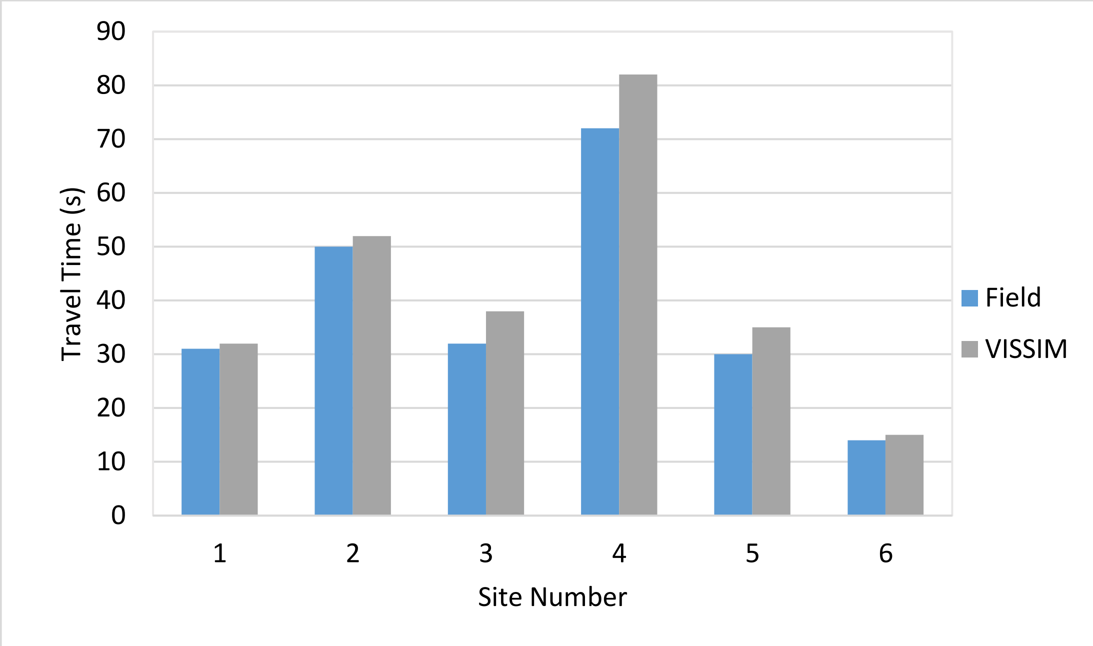


Figure 4.14 Comparison between field and VISSIM estimated travel times (using adjusted parameters)

4.3 Network performance evaluation

After validating the calibrated model, this was then used to analyse the holistic network performance using the same data set collected for the validation process. According to the HCM (2000), the LOS of an urban street is computed from the average travel speed of the through-vehicles for a specific urban street segment or the holistic urban street network being evaluated. Table 4.13 shows the obtained average speed for the entire network during the morning peak hour at 900 second (15 minute) intervals and the average speed value of 16,1 km/h which then yields a LOS (F). The obtained LOS evidences that the network is operating under the worst movement conditions.

Table 4.13 VISSIM network performance results

Time(s)	Speed (km/h)
900	17.8
1800	16.4
2700	15.2
3600	14.9

4.4 Conclusion

It is very important to understand how well traffic analysis tools, either analytical or simulation models, predict performance improvement. This will ensure that the appropriate model is used at the facilities (intersections and corridors) where best suitable. This will also provide relevant information on the limitations of each model. As these models were not developed in South Africa, it is also imperative to calibrate and validate the model to South Africa's local traffic condition data. This will enable traffic and transportation engineers to establish the actual performance of the road facility, and thereby implement proper congestion alleviation measures. From the literature, only a few researchers have investigated the prediction accuracy of traffic analysis tools after performance improvement changes were implemented in the field. In relation to the local context, no studies were identified concerning the limitations evaluation of an analytical model (SIDRA) and simulation model (VISSIM) for a signalised urban corridor analysis. A detailed discussion of the results will be provided in Chapter 5.

5 Discussions

To evaluate the effect of any upgrade implemented in the field based on the performance improvement alternations provided by the model employed, it is important to have before and after studies of road networks as these studies demonstrate how accurate the employed model is when predicting performance improvements. To ensure that the appropriate models are used to accommodate the local traffic conditions, calibration and validation processes must be executed. Therefore, the purpose of this chapter is to discuss the results presented in Chapter 4. The discussion is divided into two sections: first section (5.1) evaluates the effect of the implemented geometric upgrades on predicted performance improvement of the two signalised intersections. The pre-upgrade, modelled and post-upgrade results of the intersections are compared. Second section (5.2) compares the calibrated models (SIDRA and VISSIM) with field measured operational performance. The model that yielded the best performance was then validated. This is to validate the applicability of the model locally for operational performance evaluation. In addition, statistical measures were introduced to evaluate the reliability between calibrated, validated and field measured data.

5.1 Model prediction accuracy

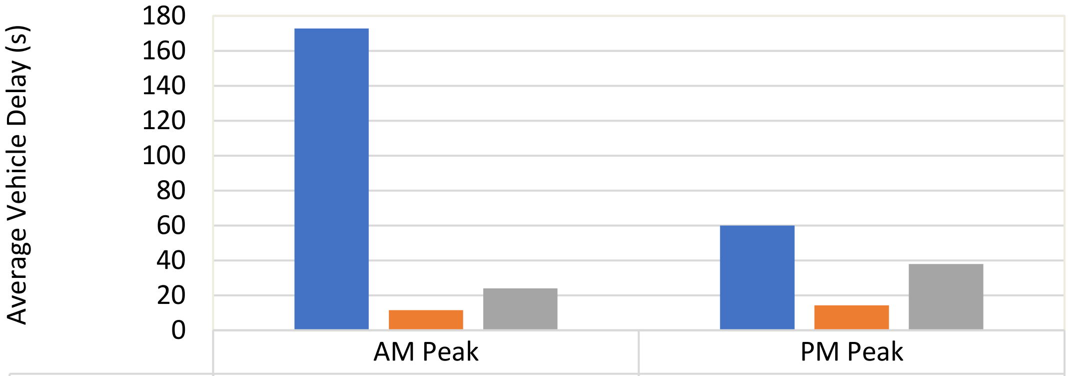
Luttrell *et al.* (2008) emphasise that it is critical to conduct before and after studies on road networks that have been upgraded. These studies assist in evaluating the accuracy of the employed model in predicting the improvements that were implemented. This enables transportation and traffic engineers to assess and validate the adequacy of further utilisation of such a model. These before and after studies also allow for the identification of model limitations regarding the performance analysis of a particular network or facility.

5.1.1 Upgraded intersections

Table 5.1 shows Gordon intersection (Intersection 2) computed performance improvements (i.e., the difference in delay for various conditions observed for both peak hours [AM and PM] as well as the observed variations). Both the AM and PM peak hours predicted performance improvement by SIDRA was not achieved; however, the AM peak hour yielded the better performance improvement of 86% and the lowest RE percentage value of 50%. The comparison between the AM and PM peak hours average delay times at Gordon intersection is presented in Figure 5.1. The model (SIDRA) showed better performance improvement predictions at the AM peak hour. Figure 5.1 indicates that the biggest discrepancy occurred in the peak hour. This was attributed to the increase in traffic volume at movement 9 and 10 (representing Main Road-west approach right turn and Gordon Road-left turn) with 20,7% and 22,8% increase, respectively (Appendix A2).

Table 5.1 Gordon intersection (intersection 2) computed performance improvements for before and after the upgrade

Peak Hour	Pre-upgrade average delay (s)	Model Predicted average delay-SIDRA (s)	Absolute difference (s)	Predicted %improvement
AM	173	12	161	93
PM	60	14	46	77
Peak Hour	Post-upgrade average delay (s)	Model Predicted average delay-SIDRA (s)	Absolute difference (s)	RE (%)
AM	24	12	20	50
PM	38	14	24	63
Peak Hour	Pre-upgrade average delay (s)	Post-Upgrade average delay (s)	Absolute difference (s)	Actual %improvement
AM	173	24	149	86
PM	60	38	22	37



	AM Peak	PM Peak
Pre-Upgrade	173	60
Modelled	12	14
Post-Upgrade	24	38

Figure 5.1 Difference between field delay at both AM and PM peak at different intersection conditions

The computed performance improvement at Andries Pretorius intersection (intersection 3) in Table 5.2 shows that the AM peak hour had a larger performance improvement of 33% and the lowest RE value of 66% in comparison to the PM peak hour. Furthermore, the RE = 80% between post-upgrade and model predicted average delay was observed at the AM peak hour, demonstrating no improvement as the average delay increased, representing deterioration in movement during the AM peak hour. The comparison between the AM and PM peak hours under different conditions is shown in Figure 5.2. Figure 5.2 data provides evidence that SIDRA over-predicted performance improvement for both peaks hours, although the AM peak hour resulted in the worst performance.

Table 5.2 Andries Pretorius intersection (intersection 3) computed performance improvements for before and after the upgrade

Peak Hour	Pre-upgrade average delay (s)	Model Predicted average delay-SIDRA (s)	Absolute difference (s)	Expected %improvement
AM	65	19	46	71
PM	55	18	37	67
Peak Hour	Post-upgrade average delay (s)	Model Predicted average delay-SIDRA (s)	Absolute difference (s)	RE (%)
AM	96	19	77	80
PM	53	18	35	66
Peak Hour	Pre-upgrade average delay (s)	Post-Upgrade average delay (s)	Absolute difference (s)	Actual %improvement
AM	65	96	-32	33
PM	55	53	2	4

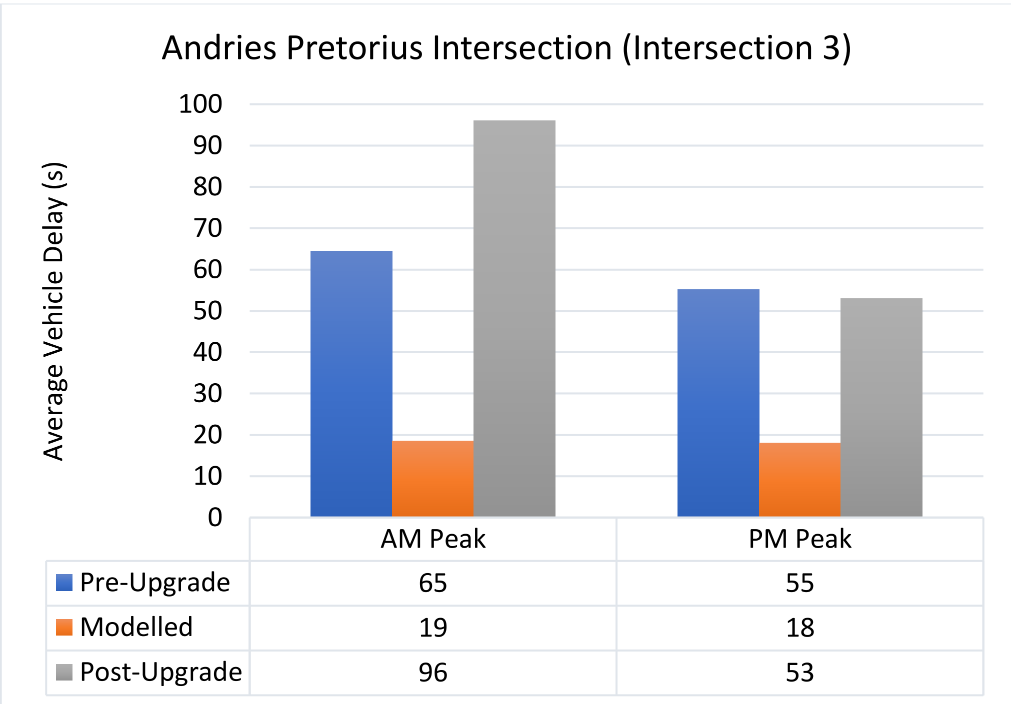


Figure 5.2 Comparison between field delay at both AM and PM peak and different intersection conditions

Observed deviation between the AM and PM peak hours demonstrates high traffic volumes during the AM peak hour, with movement 8 (Main Road-Northbound through lane) (Appendix A3) showing the highest increment of 20%. The difference in delay between modelled condition and post-upgrade condition demonstrates that the model overestimated the operational performance improvement. The increase in delay at Andries Pretorius intersection during the AM peak hour leads to a decrease in LOS (Figure 5.2), causing the vehicle traffic to spill back to the intersection downstream to Andries Pretorius intersection.

The results obtained from the current study were similar to the results obtained by Luttrell *et al.* (2008) who conducted a comparative study on the before and after construction operational analysis between 2000 and 2004 on the SR 826-Palmetto expressway interchange, in Florida, USA. The interchange was evaluated to assess how well the employed model predicted performance improvement. Their study was based upon several case studies with traffic analysis tools for performance improvement implementation. One of the evaluated case studies correlating with the present study was the operational analysis of an expressway interchange for the AM peak hour.

The results from Luttrell *et al.* (2008) showed no significant improvement in post upgrades based on the LOS criterion. In fact, a drop in LOS at some of the interchange approaches was observed, representing an overestimation in performance improvement by the prediction model. This was, however, attributed to the 10% traffic increase observed since the upgrade impacting the congestion at the adjacent intersections and mainline leading to congestion spill back into the interchange ramps. Therefore, if the traffic volume forecast by the model elected is neglected, the model will result in either underestimation or overestimation of the performance measure of interest.

5.2 Network analysis – corridor segment

5.2.1 Default parameters

- Delay

According to Tianzi *et al.* (2013), delay is the most important indicator demonstrating the operational performance for urban intersections. The delay also represents the level of service at the particular urban intersection, indicating the efficiency of the traffic design and signal control as well as providing relevant information regarding drivers' identified frustration and the quality of service (Sun *et al.*, 2013). Thus, for this study, delay was adopted as one of the measures of effectiveness to evaluate the performance of the signalised intersections in the corridor segment. The comparison between field, VISSIM and SIDRA average delays at the four signalised intersections is shown in Figure 5.3. VISSIM shows a better comparison to the field measured delay at intersections 2, 3 and 4 compared to SIDRA, with the lowest percentage difference in delay observed at the respective intersections.

Figure 5.3 Comparison between field and models' delay obtained (using default parameters)

Tables 5.3 and 5.4 show the statistical difference of field performance for results of both VISSIM and SIDRA with respect to delay and travel times. The Spearman's rank correlation coefficient, however, shows a strong relationship between field delay and the models' predicted delays $p = -0{,}8$ for both VISSIM and SIDRA. The travel time data (see Table 5.4) shows that VISSIM model better predicted the field data with $p = -0.77$ compared to SIDRA with $p = 0$.

According to Tawfeek *et al.* (2018), the FHWA Guide from Dowling *et al.* (2004) provides examples of model calibration targets specifically developed for freeways. This can, however, differ with respect to the model's potential utilisation. Some of these targets have been widely used for simulations in urban areas (Oketch & Carrick, 2005; Choi *et al.*, 2008). The FHWA Guide generally suggests that a 15% error margin to the actual data is sufficiently acceptable between field measured data and model results. Tawfeek *et al.* (2018) showed that this error margin is applicable to the error measurements used to assess the reliability of a model and that both the error measurements were to conform to the adopted error margin. The two employed error measurements (MAPE and RMSE) show that both models (VISSIM and SIDRA) for the error measurements for delay and travel times were not within the acceptable error margin (Tawfeek *et al.*, 2018). Calibration of the two models (SIDRA and VISSIM) was therefore performed.

Table 5.3 Delay time statistical analysis obtained for both VISSIM and SIDRA (using default parameters)

Statistics	MAPE (%)	RMSE (%)	*p*
VISSIM	28	11	-0,8
SIDRA	38	14	-0,8

- Travel time

Table 5.4 Travel time statistical analysis for both VISSIM and SIDRA (using default parameters)

Statistics	MAPE (%)	RMSE (%)	*p*
VISSIM	24	8	-0,77
SIDRA	51	28	0

The observed variation in operational performance prediction by both VISSIM and SIDRA can be attributed to the default parameters for both models not being applicable for local traffic condition analysis, thereby motivating the calibration of both models. Most of the reviewed literature (Tianzi *et al.* 2013; Rao & Rao, 2015; Tawfeek *et al.* 2018) does not show the comparison between field and model estimated performance using default parameters. The notion of default parameters not being favourable for application outside the model's country of origin for traffic analysis is supported by most researchers. According to Park *et al.* (2006), most researchers have shown that the simulation models when applied using default parameters, generally do not accurately predict field observed performance. Siddharth and Ramadurai (2013) used VISSIM to evaluate the applicability of the VISSIM model to predict the heterogeneous traffic conditions in India. The VISSIM predicted traffic flow was compared to field observed flow using default parameters: the MAPE value was 28,42% which was considered an inadequate, unacceptable error margin. This was the similar case for the current study whereby model predictions using default parameters yielded MAPE values above the adopted acceptable error margin.

5.2.2 Model calibration

- Delay

After the calibration of the two models to the local traffic condition, the difference between the average delay predictions obtained from the models and the field delay is shown in Figure 5.4. The average delay predictions using VISSIM closely matched the field observations at intersections 2 and 3, while SIDRA was able to reveal a close average delay prediction at intersection 3. Overall, the average delay predictions using VISSIM were better than using SIDRA.

The effect of calibration is in agreement with the study conducted by Tianzi *et al.* (2013) where similar results were observed. Fichera (2011), evaluating various types of intersections individually, obtained results showing SIDRA to have computed higher average delays compared to the average delay computed by VISSIM at low traffic demand. For the current study, the opposite was evident, where at low traffic demand (intersections 1 and 2), SIDRA estimated a lower average delay. This is attributed to the different application techniques for both models in the current study: a network analysis with a series of signalised intersections.

Sun *et al.* (2013), in their study on comparative performance evaluation of two models (CORSIM and VISSIM) for an urban street network, reported that at high congested intersections, VISSIM's predicted delay results are more reliable, closely replicating field measured delay. Similarly, for the current study, VISSIM showed better delay predictions at high congested intersections (intersections 1 and 2), particularly intersection 2 where most delay was observed.

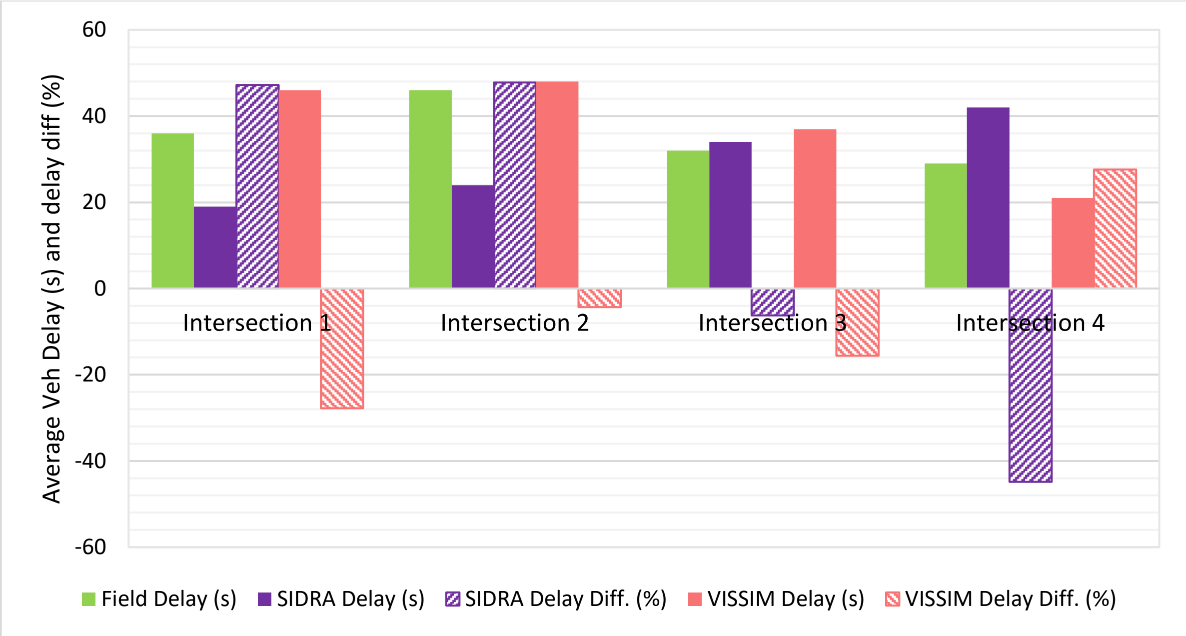


Figure 5.4 Comparison between field delay and those obtained by the models (using calibrated parameters)

Table 5.5 presents a comparison of the applied statistical measures after calibration of both models at the four evaluated intersections. VISSIM showed better delay predictions with the lowest MAPE = 5% and RMSE = 7%, which, according to Tawfeek *et al.* (2018) and Chalermwongphan and Upala (2018), is acceptable (section 2.11). A strong Spearman's ranking correlation (*p* = -1) was observed between VISSIM calibrated and field measured average delay. The VISSIM model statistical results improved more than those for the SIDRA model after calibration, suggesting that the adjusted parameters in VISSIM were better able to accommodate local driver behaviour.

Table 5.5 Delay time statistical analysis obtained for both VISSIM and SIDRA models after calibration

Statistics	MAPE (%)	RMSE (%)	*p*
VISSIM	5	7	-1
SIDRA	37	15	0,8

- Travel time

The statistical measurements used to evaluate the reliability of the travel time estimated by VISSIM and SIDRA is shown in Table 5.6 for the four evaluated intersections. The results for the VISSIM models were statistically more reliable, with the lowest MAPE = 14% and RMSE = 6 %, which according to Tawfeek *et al.* (2018) and Chalermwongphan and Upala (2018), is acceptable, refer to section 2.11 (statistical analysis). The Spearman's rank correlation coefficient (*p* = 1) obtained between VISSIM estimation and field measured travel times, according to Rupi *et al.* (2015), indicates a strong relationship.

Table 5.6 Travel time statistical analysis obtained for both VISSIM and SIDRA after calibration

Statistics	MAPE (%)	RMSE (%)	*p*
VISSIM	14	6	1
SIDRA	41	27	-0,2

The VISSIM and SIDRA models used in this study showed an improvement in error output for both average delay and travel time predictions (see Tables 5.7 and 5.8). From the comparison of the statistical measures applied to both models at default and calibrated parameters, as in Table 5.7 and 5.8, VISSIM compared to SIDRA showed a performance for both average delay and travel times. Similarly, Tawfeek *et al.* (2018) and El Esawey and Sayed (2011), addressing a bigger network in their study, reported performance improvement of the models with respect to the performance measures estimation. Although Fatima (2015) evaluated modal congestion management strategies and their influence on operating characteristics of an urban corridor, a similar performance improvement was noted after the model calibration.

Likewise, Dey *et al.* (2018) conducted a calibration and validation study of the VISSIM model using driver behaviour parameters on an isolated intersection, though their study focused on an isolated signalised intersection. Their study, though different peak hours were isolated (09:00-11:00 am and 17:00-19:00 pm) than from the current study, showed that after calibration the VISSIM model traffic volume predictions were closer to the actual measured traffic volumes. In addition, Wang and Gu (2019) investigated a calibration method for a VISSIM model for an urban expressway network. Similar to the current study, they reported that in comparison to the actual measured travel time, the calibrated parameter results were closer than the default parameter results, also indicating a significant performance improvement. This is evidence of the importance of calibration of models and the influence this has on model estimation accuracy.

Table 5.7 Average delay statistical variation at default and calibrated parameters (SIDRA and VISSIM models)

VISSIM Average Delay			
Statistics	MAPE (%)	RMSE (%)	*p*
Default	28	11	-0,8
Calibrated	5	7	-1
Model Improvement (%)	82	37	

SIDRA Average Delay			
Statistics	MAPE (%)	RMSE (%)	*p*
Default	38	14	-0,8
Calibrated	37	15	-0,8
Model Improvement (%)	3,9	-10	

Table 5.8 Travel time statistical variation at default and calibrated parameters (SIDRA and VISSIM)

VISSIM Travel Time			
Statistics	MAPE (%)	RMSE (%)	*p*
Default	24	8	-0,77
Calibrated	14	6	1
Model Improvement (%)	42	21	

SIDRA Travel Time			
Statistics	MAPE (%)	RMSE (%)	*p*
Default	51	28	0
Calibrated	41	27	-0,2
Model Improvement (%)	20	5	

5.2.3 Model validation

Validation of a model is regarded as the final step for evaluating the analysing capabilities of a model to local traffic conditions. A new data set to that which was used during calibration is a must. Therefore, this study opted to use a new data set collected from the same network (AM peak hour 07:00 am – 08:00 am data) for building a validation model. The relationship between field and VISSIM average delay (refer to Figure 4.13) shows a significant improvement in the model prediction using the adjusted parameter values. Nonetheless, a varying trend on average delay estimation was observed, with an underestimation at intersections 1 and 2, and an overestimation at intersections 3 and 4.

Table 5.9 VISSIM validation statistical measurements

	Delay	Travel time
MAPE (%)	4,8	10,6
RMSE (%)	11,1	5,3
p	-1	1

The modified driver behaviour parameters in VISSIM showed better applicability for local traffic conditions due to the decrease in error output for both delay and travel time, as shown in Table 5.9, which according to Tawfeek *et al.* (2018) is within the acceptable error margin. The Spearman’s rank correlation coefficient also shows a close relationship between the field and VISSIM estimated delay and travel times, reinforcing the validity of the adjusted parameter values in VISSIM. Generally, validation results from both average delay and travel times were satisfactory within the acceptable error margin. The developed VISSIM model was then considered valid for application. Validation of a model using a different dataset from the one used for model calibration, according to Ahmed (2005) and Reza (2013), increases the reliability of the model for handling differing traffic conditions. Similar to the current study, although a different error margin was used, Al-Ahmadi *et al.* (2019) evaluated VISSIM to model the driving behaviour on a network of three signalised intersections in Khobar-Dammam, Saudi Arabia. They reported that simulation results using calibrated driving behaviour parameters were comparable to the actual field measured MOEs (link speed, queue lengths and travel times) and that all the MOEs used in the validation of the VISSIM model were within a range of 5-10% to the actual measured values.

5.3 Network performance evaluation

After the model validation, the overall performance of the network was considered with respect to network speed in accordance with HCM (2002). The model estimated a level of service F, which is the worst movement according to the HCM (2000) criterion. The results indicated that the movement within the network during the AM peak decreased as time increased (Table 4.13), which can be attributed to an increase in traffic volume at the network. According to the HCM (2000), the hierarchy of the urban network level of service is such that LOS A represents free flow within the network, while LOS F represents restricted flow during high levels of congestion within the network or critical parts of the network. Therefore, the obtained LOS F for this study regarding the overall performance of the network (corridor segment) clearly showed movement restriction. This can be attributed to high traffic volumes observed at critical intersections within the network, which then presented an over capacitation on certain approaches at each of those intersections.

5.4 Summary

The main purpose of this work was as follows: firstly, to evaluate the effect of the implemented geometric upgrades on the performance of two signalised intersections; secondly, to evaluate the operation performance analysis capabilities of two models (SIDRA and VISSIM) on a local urban signalised corridor, through calibration and validation. It was observed that the model prediction on performance improvement (decrease in delay) at both upgraded signalised intersections showed significant variations to the field measured performance for both AM and PM peak hours. This resulted in the predicted improvement not being achieved. The conclusion was that the model, SIDRA, neglected the influence that adjacent intersections had on the adequate performance analysis of the two intersections, primarily because the upgraded intersections are located in an urban corridor with successive intersections. For this reason, isolated analysis of intersections located in a corridor with a series of intersections does not portray the actual performance.

In relation to the calibration and validation of the two models (SIDRA and VISSIM) for urban corridor analysis, the default parameters had to be tested against field data to warrant the calibration process of the models. It was observed that both models, using default parameters, generated high errors (with respect to MAPE and RMSE) which were not within an acceptable error margin. After calibration, both models (VISSIM and SIDRA) generated much better results regarding delay, LOS and travel time estimations when compared to the models' default parameter results. However, the VISSIM model was determined to be a better performing model compared to SIDRA. The statistical measurements employed to evaluate the reliability of the data showed that the VISSIM model operational performance predictions were within the acceptable error margin. Generally, under both default and calibrated parameters, the VISSIM model performed better than the SIDRA model.

The validation results showed that the adjusted driver behaviour parameters in VISSIM can be applied for the operational performance analysis of local urban corridors. This conclusion was further reinforced by the applied statistical measures showing minimum percentage error (within the acceptable error margin) between model and field data, thereby demonstrating a strong correlation between the two data sets. The overall network performance that was conducted showed that VISSIM is capable of analysing the performance of a network of successive signalised intersections, providing the overall performance results of the network. During the AM peak hour, the network was observed to experience high traffic volumes, resulting in a LOS F which denotes the worst possible performance. This is attributable to the observed decrease in speed observed towards the middle of the peak hour through to the end of the peak hour.

6 Conclusions and recommendations

The use of traffic analysis tools, such as analytical and simulation (micro, macro and meso) models, has evolved in recent years, primarily because they offer a more economical way of evaluating the performance of existing facilities by allowing for the critical evaluation of feasible performance improvement measures prior to field implementation. It is important to ensure that the appropriate traffic analysis tools are employed at specific road facilities (such as intersections and corridors, signalised or un-signalised). This can be improved through calibration and validation of such traffic analysis tools for local traffic conditions. This chapter presents a summary of the work in relation to the outcomes of this study, together with recommendations for future research in this field.

6.1 Summary

The main objectives of this study were as follows: firstly, to measure the effect of geometric upgrades implemented at two signalised intersections on performance improvement (based on delay and LOS); and secondly, to calibrate two models (SIDRA and VISSIM) to local traffic conditions for a comparison of results with field measured operational performance (with respect to delay, LOS and travel time). The results from the better performing model were then validated with an independent data set to test and assess the applicability of the model for performance analysis of a local urban corridor. The calibration process focused mainly on parameters impacting delay and travel time. The case study was a local signalised urban corridor in Somerset West, Cape Town, South Africa. Statistical measures such as MAPE, RMSE and the Spearman's rank correlation coefficient were applied to test and assess the reliability of the data.

6.2 Effect of the implemented geometric upgrades

The performance improvement of the two upgraded signalised intersections was evaluated based on delay and LOS. At the two intersections, post-upgrade performance data was collected for both morning and afternoon peak hours (07:00 – 08:00 am and 16:30 – 17:30 pm, respectively). The data was then compared to the data retrieved from the Somerset West traffic Engineering Department (pre-upgrade and model-predicted data).

The following was concluded for both Gordon and Andries Pretorius intersections (intersections 2 and 3):

- At both intersections, the SIDRA model, using default parameters, predictions obtained on the performance improvement (decrease in delay) did not correlate well with the performance observed post-upgrade of the intersections.

- The variations at these two intersections can be attributed to the model's inability to take into consideration the impact of traffic movement at the adjacent facilities. This is because the intersections are in an urban corridor consisting of a series of intersections in close proximity, and therefore, movement at one influence the other.

Generally, when dealing with performance evaluation of successive signalised intersections, isolated evaluation of either (assessing the intersection performance without considering adjacent intersections) yields a misrepresentation of the actual operational performance of the intersection. Therefore, under these conditions, a corridor analysis is advised. In this regard, critical intersections where most congestion is observed can be identified, allowing for appropriate performance to be implemented along the corridor.

6.3 Calibration and validation

For both SIDRA and VISSIM models prior to the calibration process, the first stage was to run the models with default parameters, which were subsequently compared with field-collected data. The models involved in this current study were calibrated because from the comparative analysis conducted between default and field measured data, there was a high error output based on the statistical measures employed. The results were shown to be within an acceptable range and conforms well to the widely acceptable error for the employed error measurements (MAPE and RMSE). Therefore, the results from the models using default parameters were unacceptable and calibration was initiated. A manual calibration technique was used for both VISSIM and SIDRA.

The following conclusions were drawn:

- The results from both models using default parameters differed significantly from the field measured operational performance (with respect to delay, LOS and travel time), showing high percentage errors. This meant that the built-in default parameters were not applicable for analysis of local urban corridor. Therefore, the models were calibrated for the local traffic conditions.

- Calibration of both SIDRA and VISSIM were carried out successfully based on the field measured data. The models showed improvement after calibration on performance estimation (delay and travel time) in relation to the observed error output between default and calibrated parameters. This demonstrated the importance of calibration for the models.

- After calibration, the comparison of results (based on delay and travel time) from SIDRA and VISSIM with the field measured data was conducted to assess the better performing model. Regarding statistical reliability of the delay results, SIDRA showed MAPE = 37% and RMSE = 15%, while VISSIM showed MAPE = 5% and RMSE = 7%. This shows that the VISSIM model was able to closely replicate the field measured data. The same was concluded for the travel time, where SIDRA compared to VISSIM showed the higher error results with MAPE = 41%, RMSE = 27% and MAPE = 14% and RMSE = 6%, respectively. The VISSIM model, in comparison to the SIDRA model, generated a better Spearman's rank correlation coefficient with both field delay and travel time, where VISSIM yielded p values of -1 and +1 and SIDRA showed p values of -0.8 and -0.2 for delay and travel time, respectively. The VISSIM model, therefore, performed better overall. In addition, VISSIM is found to provide better estimations for the delay and LOS as compared to travel time.

- Both models predicted delay and travel time significantly better after calibration. The adjusted driver behaviour parameters in VISSIM (Appendix E1, Set 4) are applicable for local urban corridor operational performance evaluation.

- The VISSIM model was validated using a new data set different from that used during calibration from the same corridor. Validation of the model was to verify the applicability of the modified driver behaviour parameters in VISSIM for local urban corridor performance analysis. The statistical analysis results showed that the error output of VISSIM compared to the field measured performance was significantly lower and within the acceptable error margin (less than 15%) with a strong correlation observed between the two data sets (p = -1 and p = 1 for delay and travel time, respectively). The validation data statistical results showed MAPE = 4.8%, RMSE = 11.1%, and MAPE = 10.6%, RMSE = 5.3 for delay and travel time, respectively. This suggests a reasonable comparison between the model's estimated and field measured data, and validated that the VISSIM model (as a micro-simulation model) is a promising model for local traffic conditions for signalised urban corridors with successive intersections.

6.4 Network performance evaluation

The validated model was then used on the same network to evaluate the operational performance of the corridor segment during the morning (AM) peak hour. The overall network performance analysis showed that the corridor segment was operating under worst movement conditions. This category of performance by an urban corridor with signalised intersections, according to the HCM (2000), demonstrates limitation in free movement of a vehicle within the network. It was observed that as time increased within the peak hour, the speed decreased. The decrease in speed was attributed to the observed congestion at some of the intersections within the network. The appropriate performance improvement measures can therefore be implemented at critical intersections such as intersection 3 and 4 as they showed higher average delay times, specifically for through movements.

The current study clearly demonstrates that investigating the effect of performance improvement changes implemented in the field, as motivated by traffic analysis tools, is pivotal, emphasising the importance of calibration and validation of these tools for analysis of local traffic conditions. As far as could be ascertained from the literature reviewed, the following novelties contributed: neither of the two models used in this study (SIDRA and VISSIM) have previously been calibrated for local traffic conditions (in Cape Town), nor has comparative analysis on the capability of SIDRA and VISSIM on the performance analysis of an urban corridor with successive signalised intersections been considered. The current study clarifies that calibration can make the VISSIM model prediction deviate under 15% of the actual field data.

6.5 Recommendations for further research

This study only focused on delay, thus providing the LOS criteria and travel time for model calibration and validation. Therefore, other measures of performance could be included. In addition, the current study dealt with a small network; however, if the size of the network is increased, other calibration parameters should be considered.

As far as applicability of traffic analysis tools (either analytical or simulation models) for local traffic condition analysis, calibration and validation are vital. Again, assessment of the effect on performance improvement predicted by these models is critical for ensuring that the appropriate use of these models at the relevant road facility (signalised or un-signalised intersections and corridors) is maintained.

The following are identified as areas that warrant further research:

- The applicability of VISSIM for the analysis of urban corridors with mixed intersections and freeways could be considered for local conditions on a regional scope.

- The developed model in VISSIM from this study should be investigated for applicability on other corridors with the same jurisdiction as the case study used to validate the area-wide suitability of the model.

- The effect of pedestrians on the performance of an urban corridor with signalised intersections should also be investigated as this was not within the scope of the current study. This is because pedestrians have been identified as contributing to road facility performances, especially in urban areas where pedestrians are prolific.

- Lastly, wider research on the before and after studies of projects based on model-predicted upgrades, with the aim of performance improvement of facilities, should also be conducted to assess the effect of the improvements implemented.

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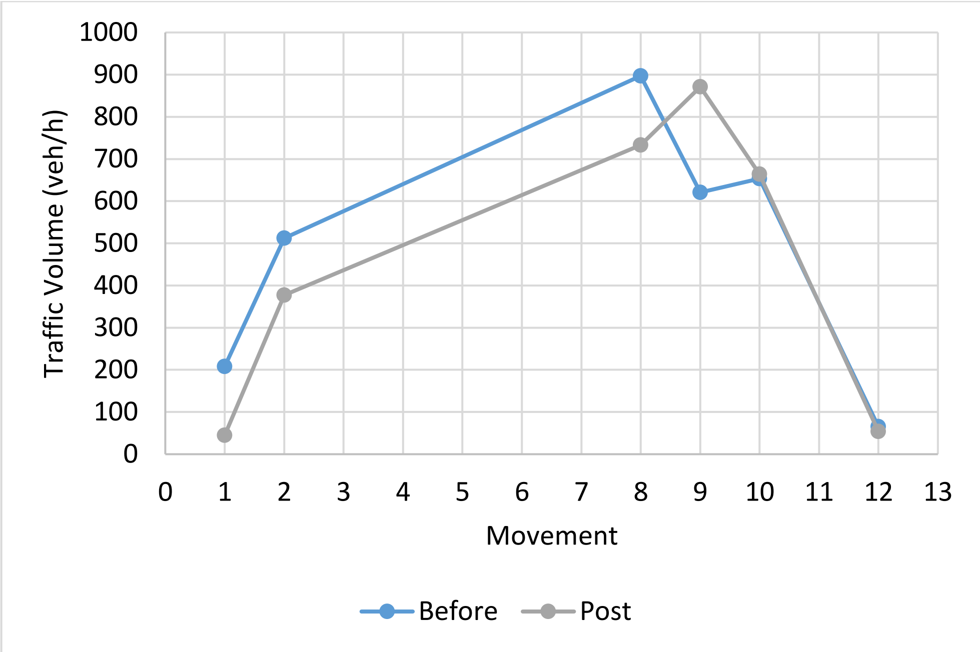
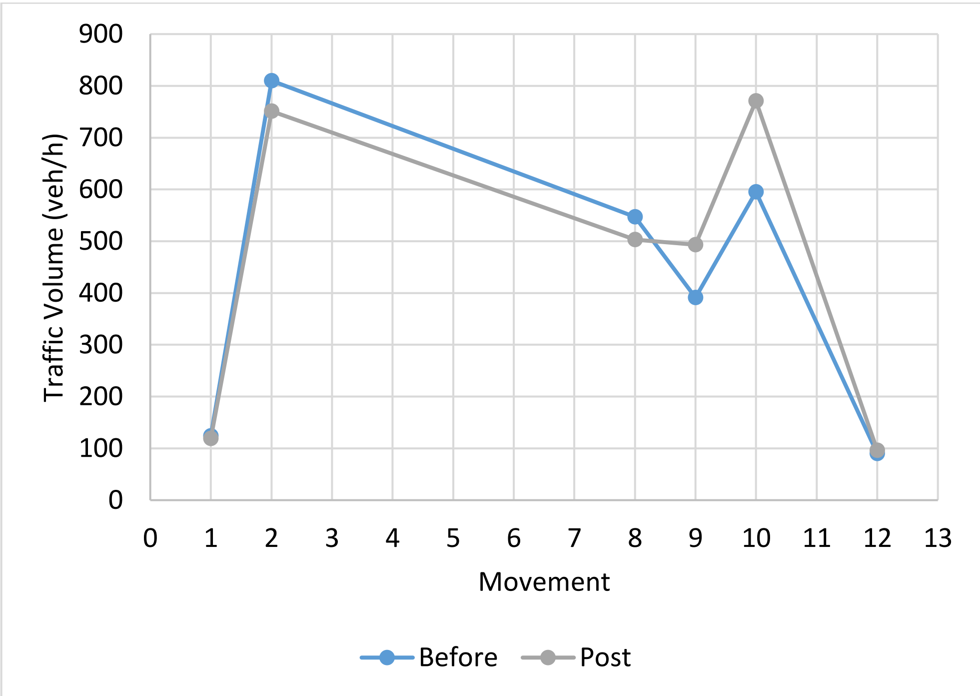
Appendices

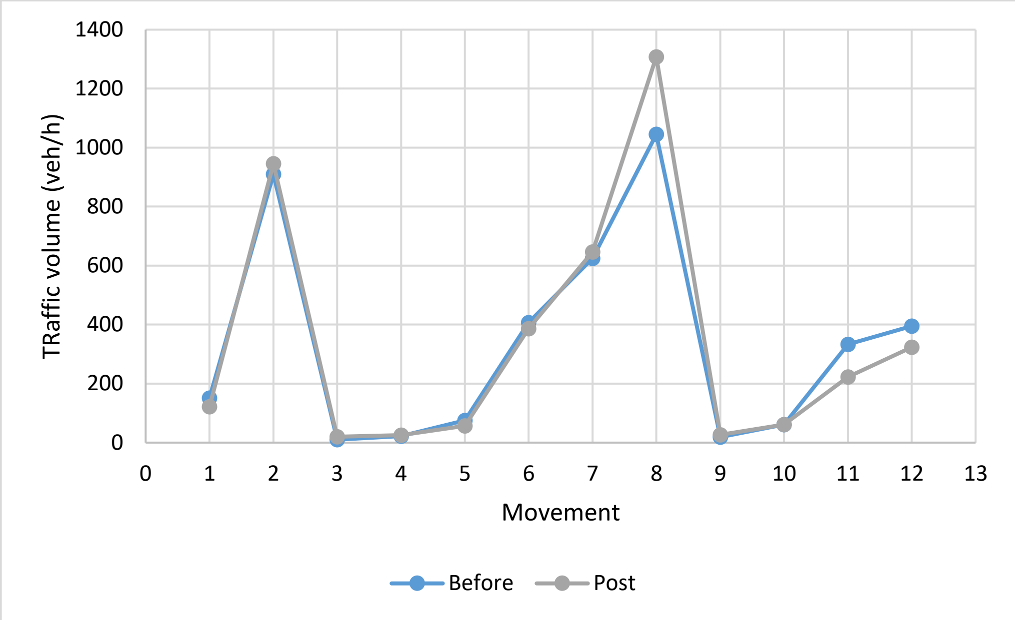
Appendix A. Field traffic volumes



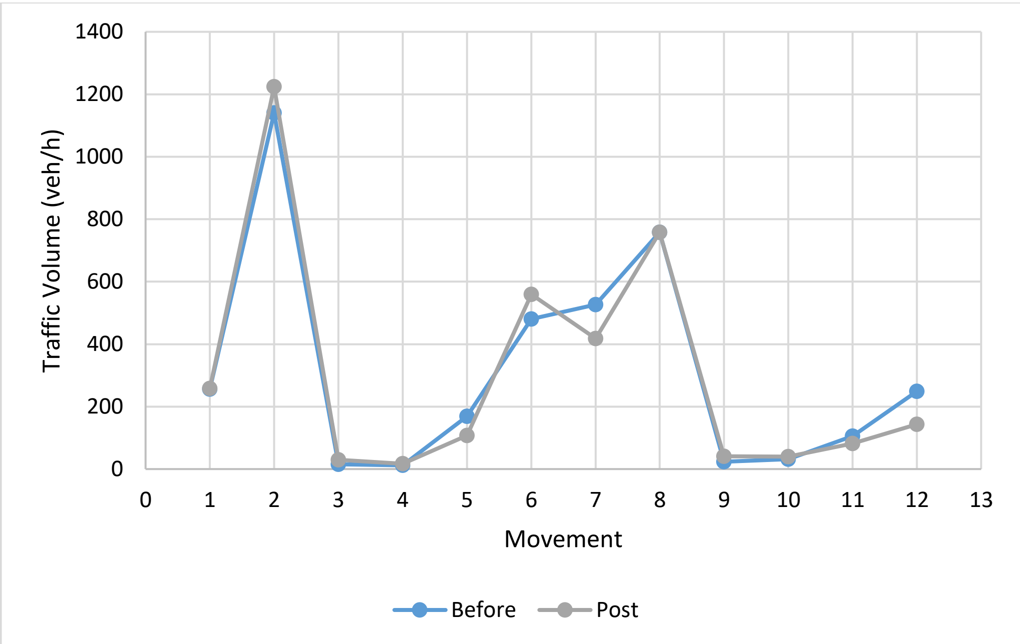
Appendix A 1. Gordon intersection before and post-upgrade traffic volumes (AM peak hour)



Appendix A 2. Gordon intersection before and post-upgrade traffic volumes (PM peak hour)



Appendix A 3. Andries Pretorius intersection before and post-upgrade traffic volumes (AM peak hour)



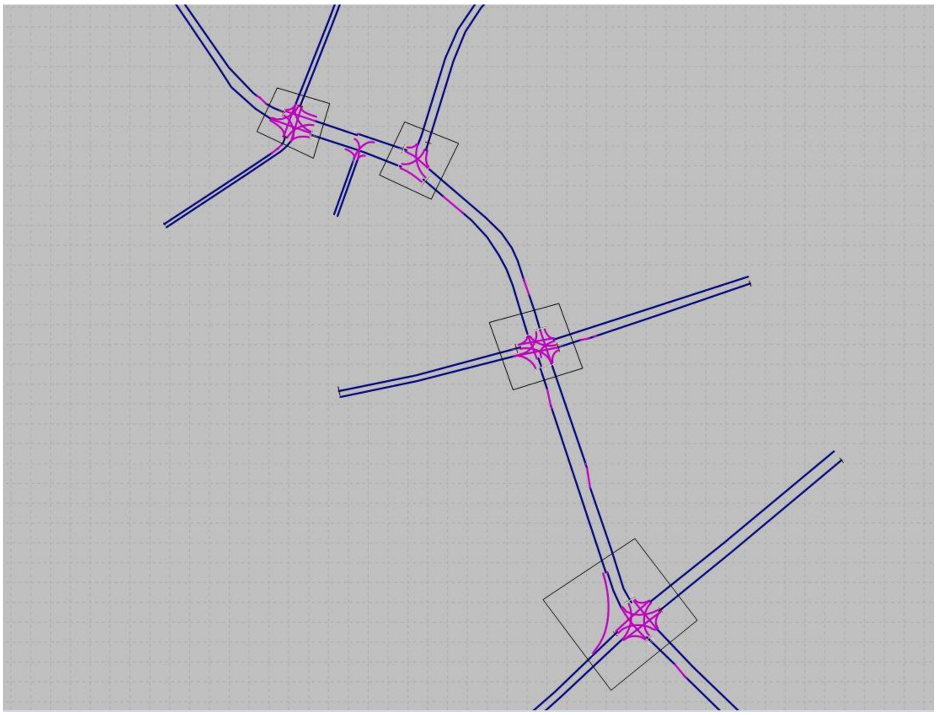
Appendix A 4. Andries Pretorius intersection before and post-upgrade traffic volumes (PM peak hour)

Appendix B. Field travel times data

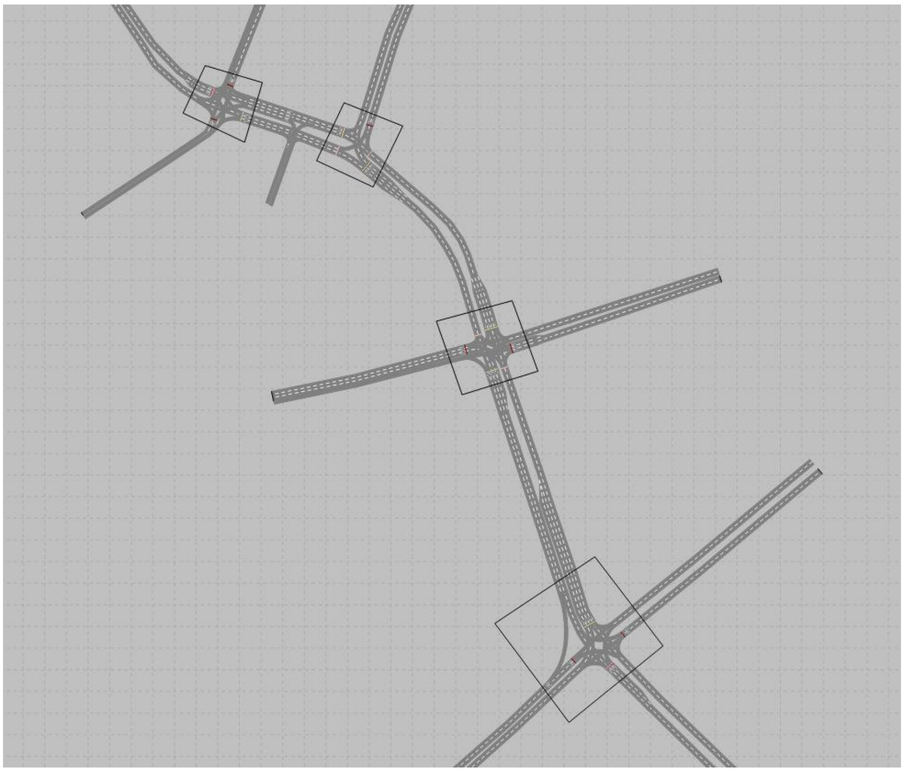
Appendix B 1. Field travel time standard deviation (PM peak hour)

	Travel Time (s)					
	Site No. 1	Site No. 2	Site No. 3	Site No. 4	Site No. 5	Site No. 6
Run 1	25	42	20	34	35	14
Run 2	14	40	22	36	27	10
Run 3	13	53	32	38	33	14
Run 4	34	50	31	48	29	12
Run 5	36	52	35	49	31	13
Run 6	29	43	38	54	19	17
Total	151	280	178	259	174	80
Average	25,1	46,6	29,7	43,2	29,0	13,4
Standard Deviation	9,65	5,30	7,32	8,57	5,52	2,44

Appendix C. Study network coded in VISSIM (corridor segment)



Appendix C 1. Links and connectors coding in VISSIM (network construction)



Appendix C 2. Detailed geometric design of the study network

Appendix D. Travel time statistical analysis

Appendix D 1. Field and VISSIM average travel time statistical analysis (using default parameters)

Site No.	Link	Distance (m)	Field travel time (s)	Standard deviation	VISSIM travel time (s)	Between 1 and -1 standard deviation	Travel time difference (s)	RE (%)
1	Fagan Street Intersection to Gordon Road Intersection	126	25	9,65	38	NO	-13	51
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	47	5,3	53	NO	-6	14
3	Andries Pretorius Street Intersection to Hospital Intersection	256	30	7,32	37	NO	-7	25
4	Hospital Intersection to Andries Pretorius Street Intersection	297	43	8,57	53	NO	-10	23
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	29	5,52	35	NO	-6	21
6	Gordon Road Intersection to Fagan Street Intersection	94	13	3,44	15	YES	-2	12
MAPE								24
RMSE								8
p								0,77

Appendix D 2. Field and VISSIM average travel time statistical analysis (using calibrated parameters)

Site No.	Link	Distance (m)	Field travel time (s)	Standard deviation	VISSIM travel time (s)	Between 1 and -1 standard deviation	Travel time difference (s)	RE (%)
1	Fagan Street Intersection to Gordon Road Intersection	126	25	9,65	23	YES	2	-8
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	47	6,3	53	YES	-6	13
3	Andries Pretorius Street Intersection to Hospital Intersection	256	30	7,32	37	YES	-7	23
4	Hospital Intersection to Andries Pretorius Street Intersection	297	44	8,57	52	YES	-8	18
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	29,00	5,52	34	YES	-5	17
6	Gordon Road Intersection to Fagan Street Intersection	94	13	3,44	16	YES	-3	23
MAPE								14
RMSE								6
p								1

Appendix D 3. Field and VISSIM average travel time statistical analysis (using adjusted parameters)

Site. No.	Link	Distance (m)	Field travel time (s)	Standard deviation	VISSIM travel time (s)	Between 1 and -1 standard deviation	Travel time difference (s)	RE
1	Fagan Street Intersection to Gordon Road Intersection	126	31	4,65	32	YES	-1	3
2	Gordon Road Intersection to Andries Pretorius Street Intersection	203	50	4,30	52	YES	-2	4
3	Andries Pretorius Street Intersection to Hospital Intersection	256	32	7,32	38	YES	-6	19
4	Hospital Intersection to Andries Pretorius Street Intersection	297	72	10,03	82	YES	-10	14
5	Andries Pretorius Street Intersection to Gordon Street Intersection	192	30	5,52	35	YES	-5	17
6	Gordon Road Intersection to Fagan Street Intersection	94	14	2,44	15	YES	-1	7
MAPE								10,6
RMSE								5,3
P								1

Appendix E. Different sets of parameter values during calibration

Appendix E 1. Calibrated parameter values

Parameters	VISSIM Default Values	Calibrated Parameter Values			
		Set 1	Set 2	Set 3	Set 4
Number of observation of vehicles	2	4	4	4	4
Additive part of desired safety distance	2	2,25	2,25	0,7	1,5
Multiplicative part of desired safety	3	3,25	3,25	0,5	1,5
Amber signal decision model	Continuous	One decision	Continuous	Continuous	Continuous
Lane change distance	200 m	300 m	200 m	200 m	200 m

Appendix E 2. Obtained travel times set of parameter values

Site No.	Distance (m)	Field Travel time (s)	Standard deviation	VISSIM Travel time (default)(s)	Between 1 standard deviation	Calibrated Travel time (s)							
						Set 1	Between 1 standard deviation	Set 2	Between 1 standard deviation	Set 3	Between 1 standard deviation	Set 4	Between 1 standard deviation
1	126	25	9,65	38	NO	35,27	NO	34,20	YES	33,26	YES	22,54	YES
2	203	47	5,30	53	NO	52,21	YES	52,25	YES	51,89	YES	51,77	YES
3	256	30	7,32	37	NO	37,59	NO	37,23	YES	37,12	YES	37,01	YES
4	297	43	8,57	53	NO	52,38	NO	52,04	NO	51,63	NO	51,27	YES
5	192	29	5,52	35	NO	34,34	YES	34,69	NO	34,56	NO	34,41	YES
6	94	13	3,44	15	YES	15,79	YES	16,05	YES	15,92	YES	15,81	YES

Appendix F. Spearman's rank correlation calculation

Appendix F 1. SIDRA and VISSIM Spearman's rank results (using default parameters)

Field delay (s)	SIDRA delay (s)	Rank (field)	Rank (SIDRA)	d	d^2
33	21	2	4	2	4
46	28	1	3	2	4
32	39	3	2	1	1
29	43	4	1	3	9
					Total = 18

Field travel time (s)	SIDRA travel time (s)	Rank (field)	Rank (SIDRA)	d	d^2
27	16	2	1	1	1
55	35	4	2	2	4
28	64	3	4	1	1
24	61	1	3	2	4
					Total = 10

Field delay (s)	VISSIM delay (s)	Rank (field)	Rank (VISSIM)	d	d^2
33	54	2	4	2	4
46	54	1	3	2	4
32	38	3	2	1	1
29	22	4	1	3	9
					Total = 18

Field travel time (s)	VISSIM travel time (s)	Rank (field)	Rank (VISSIM)	d	d^2
25	38	2	4	2	4
47	53	6	5	1	1
30	37	4	3	1	1
43	53	5	6	1	1
29	35	3	2	1	1
13	15	1	1	0	0
					Total = 8

Appendix F 2. SIDRA and VISSIM Spearman's rank results (using calibrated parameters)

Field delay (s)	SIDRA delay (s)	Rank (field)	Rank (SIDRA)	d	d^2
36	19	2	1	1	1
46	24	1	2	1	1
32	34	3	3	0	0
29	42	4	4	0	0
					Total = 2

Field travel time (s)	SIDRA travel time (s)	Rank (field)	Rank (SIDRA)	d	d^2
27	14	2	1	1	1
55	32	4	2	2	2
28	56	3	3	0	0
24	61	1	4	3	9
					Total = 12

Field delay (s)	VISSIM delay (s)	Rank (field)	Rank (VISSIM)	d	d^2
36	46	2	3	1	1
46	48	1	4	3	9
32	37	3	2	1	1
29	21	4	1	3	9
					Total = 20

Field travel time (s)	VISSIM travel time (s)	Rank (field)	Rank (VISSIM)	d	d^2
25	23	2	2	0	0
47	53	6	6	0	0
30	37	4	4	0	0
43	52	5	5	0	0
29	34	3	3	0	0
13	16	1	1	0	0
					Total = 0