ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Assessing the Effect of Light Rail Transit in Travel Corridor Congestion

(A Case Study on Addis Ababa, Ethiopia North-South Corridor)

By

Elshaday Alemayehu Tekele

A Thesis Submitted to the School of Graduate Studies in Partial Fulfillment of The Requirements for the Degree of

Master of Science

In

Road and Transport Engineering Research

Advisor: Bikila Teklu Wodajo (Ph.D.)

October 2019

Addis Ababa, Ethiopia

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Approved by Board of Examiners:

____________________ Advisor	____________________ Signature	____________ Date
____________________ Internal Examiner	____________________ Signature	____________ Date
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____________________ Chairperson	____________________ Signature	____________ Date

UNDERTAKING

I certify that this research work titled "***Assessing the Effect of Light Rail Transit in Travel Corridor Congestion: A Case Study on Addis Ababa,Ethiopia North-South Corridor***" is my original work performed under the supervision of my research advisor **Dr. Bikila Teklu**. The work has not been presented elsewhere for assessment and a degree in any other university. Where material has been used from other sources has been properly acknowledged.

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ABSTRACT

Transportation plays a vital role in almost all cities around the world because it provides access for people to employment, education, entertainment, health care, and other services. As developing capital city, the urbanization rate in Addis Ababa is increasing rapidly, which in turn increase's daily commuters, which leads to the necessity of alternative mode of transport, so currently light rail transit has been implemented and found to be effective. Indeed, Light Rail Transit is an effective solution to deal with this problem. Light Rail Systems can be found in a variety of land use context and can operate under a different type of right of way. However, there is an exceptional possibility for vehicle congestion which is traveling along the corridor. The vehicles in this corridor are experiencing additional delays and travel time where there is at grade intersections and pedestrian crossings.

The primary objective of this thesis is to assess the effect of light rail transit on travel corridor congestion using a microscopic simulation model called VISSIM. Microscopic traffic simulation models have been playing an essential role in the evaluation of alternatives. To achieve high fidelity and credibility for the simulation model, calibration and validation of the model are the most important. This research presents a calibration and validation of the model by adjusting the car following behavior. The first car following behavior input parameters adjusted is average standstill distance, additive part of standstill distance, and multiplicative part of standstill distance. The validity of the calibration model was also checked through error checking and statistical comparison, it was found that the performance measures obtained by the simulation model were representative of the field condition. The result between the modeled values showed no discrepancy between the model simulation and field measure of performances.

The overall result indicates that the reduction of heavy vehicles, on-street parking restriction in the study area has a significant effect on the performance-related measures against traffic (delay, capacity, queue length, and the number of stops) and emission (CO, NOx, VOC) and fuel use. According to Dunnett's multiple comparison test introductions, an active transit signal priority (ATSP) controller is found to be effective in improving performance measures.

Key Words: ***Light Rail Transit, Calibration, Validation, Simulation Model and Active Transit Signal Priority, capacity, delay, queue length, number of stops.***

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LIST OF ABBREVIATION AND ACRONYMS

ax – standstill distance

AM – Ante Meridiem

Bxadd – additive part of standstill distance

Bxmult – multiplicative part of standstill distance

CI – Confidence Interval

CO – Carbon Monoxide

D – Desired Safety Distance

DHV – Directional Hourly Volume

GEH – Geoffrey E. Havers

H0 – Null Hypothesis

H1 – Alternative Hypothesis

LRT – Light Rail Transit

LRV – Light Rail Vehicle

LT – Left Turn

ME – Mean Error

MPE – Mean Percent Error

MUTCD – Manual on Urban Traffic Control Devices

NOx – Nitrogen Oxide

PM – Post Meridiem

R – Pearson correlation coefficient

RMSPE – Root Mean Square Percent Error

RNS – Random Number of Seed

RT – Right Turn

TM- Through Movement

TSP – Transit Signal Priority

U – Thiel's Inequality Coefficient

Uc – Variance Proportion

Um – Bias Proportion

Us – Covariance Proportions

UT – U Turn

VISVAP – Visual Vehicle Actuated Program

VOC – Volatile Organic Compound

CHAPTER ONE: INTRODUCTION

1.1 Background

Transportation plays an important role in almost all cities around the world because it provides access for people to employment, education, entertainment, health care, and other services. However, with the rapid increase in private cars in recent years, traffic congestion has become a major issue in many big cities. The level of congestion in these areas is also increasing because of the rise in population, economy, housing, jobs, and urbanization. Light Rail Transit has been implemented to improve the traffic congestion in Addis Ababa. Indeed, Light Rail Transit is an effective solution to deal with this problem. Light Rail System can be found in a variety of land use context and can operate under a different type of right of way. With the flexibility of light rail systems in congested cities, they can attract a significant share of urban trips and reduce congestion in road networks.

Congestion is most often associated with road transport and occurs when the volume of traffic approaches the available capacity (Anrew & Tony, 2015). This leads to queuing and an increase in travel time. Congestion varies according to the time of day. It is especially high in the morning and late afternoon because of commuting to and from work and school.

Addis Ababa has two Light rail Transit corridors one running South-North and the other East-West. LRT corridor has been segregated along in the middle of traffic. The LRT has three modes of running system at grade, elevated and underground.

This study will evaluate the effect of Light Rail Transit on travel corridor by identifying average travel time and delay caused by the service using VISSIM traffic simulation software to model the effect of LRT on the travel corridor. Microsimulations are widely used in traffic operations and management analysis because simulation is safer, less expensive and quicker than field implementation (Park & Schneeberer, 2002). VISSIM which is used in this study is a microscopic time step and behavior-based simulation model. It can model traffic flow, pedestrian, and public transport making it a useful tool for the evaluation of different alternatives solutions to minimize travel time and decrease congestions. Any model created using VISSIM needs to be calibrated to sufficiently represent field conditions and validated until the acceptable error tolerance is achieved. Then different scenarios can be evaluated and tested.

1.2 Statement of The Problem

Light rail has been the most effective way to meet the transportation demand of a city throughout the world. However, light rail transit has the inherent weakness of producing congestion at grade crossings or level crossings, creating several conflict points for road traffic and pedestrians (Chad & Lester, 2004). These crossings force both road traffic and the tram to lower their speed, which in turn increases travel time and decreases the overall efficiency of the network.

The light rail transit system is an effective and environmentally friendly means of transportation of goods and services. Addis Ababa is currently employed in two LRT lines running EW and SN. This mode of transport has contributed to the mobilization of around 150,000 riders daily (Anon., 2019)and has a high contribution in reducing accidents and increasing safety. However, having this merit does not make it absolute; there is a downside to it. Due to access management issues along the corridor, because of the median running light rail transit the semi-exclusive nature of the alignment integration vehicles are forced to turn (Left Turn and U-Turn) only and on the provided intersection. So, during the peak hour of the day which are from 7:00 to 9:00(morning) and 4:00 to 6:00 (evening) where there is congestion with maximum queue length 500m and sometimes more. The problem is not due to vehicle management, and there is also a high involvement of pedestrian crossing at the intersection. The inclusion of pedestrians can lead to vehicle delays where vehicles are required to wait for the pedestrian phase and for pedestrians to clear the intersection. This research mainly focuses on the assessment of the LRT effect on the travel corridor.

1.3 Research Question

1.3.1 Research Question

- How to calibrate and validate the simulation model to car following behavior?
- Does Light rail trait frequency have an effect on congestion?
- Is their any measure of performance change in scenarios?
- What are the factors contributing to an increase in congestion?

1.4 Objective of The Study

The objectives of the study are presented in general and specific.

1.4.1 General Objective

The study aims to assess the effect of light rail transit in congestion along the travel corridor which makes delay increase for the vehicles traveling that corridor.

1.4.2 Specific Objective

The specific objectives include: -

- To calibrate and validate the simulation model for car-following behavior,
- To evaluate the effect of the Light Rail Transit frequency on congestion,
- To determine the change in the measure of performances of intersection across scenarios,
- To recommend for congestion reduction solution,

1.5 Scope, Significance and Limitation

1.5.1 Scope of The Study

The scope of the study covers the vehicular traffic movement along the Light Rail Transit at grade crossings (at Adey Ababa Grade crossing) and pedestrian crossings (Adey Ababa and Saris).

1.5.2 Significance of The Study

Now a day's road traffic congestion has become a major problem in cities throughout the world. The outcome of this research very significant. One intended outcome of the study, the researcher will have a better understanding of the impact of light rail construction in the median of the highway which increases congestion and become familiar with state-of-the-art microsimulation software. The second intended outcome is to identify different approaches to congestion relief measures which will reduce travel time and delay the research can also be used as a foundation to make better traffic congestion analysis.

The theoretical significance of this thesis:

- Introduces a modern way of congestion assessment through accurate visualization of the existing problem in the field through simulation.
- Changes our understanding in performance evaluation and present a new method of simulation model calibration and validation procedure by combining different tests.

Specifically, it is expected that the results of this study will have the following importance;

- Calibrated and validated model can help further extend to develop city-wise assessment,
- Helps to understand congestion in a more realistic cost-effective way,
- Helps to find the main effects parameter changes in the at grade crossing,
- The practical significance of this research will help experts and stakeholder to implement alternative among the tested traffic management strategies,
- It will show the effect of heavy vehicles, on-street parking, and transit signal priority.

1.5.3 Limitation of The Study

The study will not prove to tell interruptions due to pavement distress, interchange, and effect of surface roughness contribution to vehicle delay.

1.6 Challenges

The following challenges were faced during the thesis period;

1. During the data collection period, it was hard acquiring permission from authorities.
2. Due to the data-intensive nature of the research, it was very hard to collect all the required data
3. Manual data extraction was very time-consuming since there is no software package to extract data.
4. Licensed VISSIM was only available at Transport Program Management Office (TPMO) so the researcher was forced to wait for turns and availability of the license dongle.

1.7 Organization of the Thesis

This study is organized in five chapters. ***Chapter one*** describes the general background of the study, statement of the problem, objectives, research questions, challenges faced during the study, scope significance and limitation of the study. ***Chapter two*** deals with literature reviews including congestion of at grade crossings, calibration, and validation of VISSIM and assessment of similar problems in the world. ***Chapter three*** describes research methods and materials including the description of the study area, data analysis methodology, sample size determination, study design, data collection methodology, data extraction methodology and study procedure***. Chapter four*** deals with results and discussion, a detail of data analysis, and the results are presented in this chapter as well. At last***, chapter five*** consists of the conclusion and recommendation based on the findings of the study and future studies are described.

CHAPTER TWO: LITERATURE REVIEW

2.1 Chapter Introduction

This chapter intends to look at several kinds of literature written in the areas that the scope of this research is concerned about.

2.2 Light Rail Transit

Light rail transit (LRT), according to the Texas MUTCD defined as, "a mode of metropolitan transportation that employs LRT vehicles (commonly known as light rail vehicles, streetcars, or trolleys) that operate on rails in streets in mixed traffic, and LRT traffic that operates in semi-exclusive rights-of-way, or in exclusive rights-of-way. Grade crossings with LRT can occur at intersections or at mid-block locations, including public and private driveways" (Texas MUTCD Part-8, 2011)**.**

LRT alignments can be grouped into one of the following three types: (TCRP Report 17, 1994)

A. Exclusive: An LRT right-of-way that is grade-separated or protected by a fence or traffic barrier. Exclusive alignments use full grade separation of both motor vehicle and pedestrian crossing facilities, thereby eliminating grade crossings and operating conflicts and maximizing safety and operating speeds.

B . Semi-exclusive: An LRT alignment that is in a separate right-of-way or along a street or railroad right-of-way where motor vehicles, pedestrians, and bicycles have limited access and cross at designated locations only. On segments of this type of alignment where the right-of-way is fenced, operating speeds are maximized; however, these higher speeds are typically maintained for shorter distances, often on segments between grade crossings.

C. Mixed-use: An alignment where LRT operates in mixed traffic with all types of road users. This includes streets, transit malls, and pedestrian malls where the right-of-way is shared. (TCRP Report 17, 1994)**.**

Vehicle Congestion

(Takyi, et al., 2013) states that congestion occurs when the vehicular volume of traffic is more than its design capacity known as its saturation. They also opine that congestion can be perceived as the inevitable consequence of the following cases; scares road, parking area or due to inefficiency of traffic management. (Feifei, et al., 2016) argues that traffic congestion has become a major problem in many cities in order to mitigate improvements in traffic facilities advanced control and management strategies are required.

(Reid , et al., 2014) describes that congestion reduction using highway expansion and implementation through light rail transit would not decrease congestion indeed it has an opposite effect on its intended purpose. Light rail transit affects vehicle congestion along its corridor. A study conducted in Denver about light rail transit effect on travel corridor shows that traffic within the zone of influence of the light rail system increased by 31% when compared to 41% outside the zone (Sutapa & Andrew, 2012).

(Reid , et al., 2014) founded that after the introduction of Light rail transit effect on traffic, energy consumption and air pollution. According to their finding traffic with the LRT decrease due to high-quality transit service leading to transit ridership increase and LRT saves about 362,000 and 500,000 gallons of gasoline and prevents about 7 million and 10 million pounds of CO_2 from being emitted to the atmosphere each year respectively.

(LRT Service Guidelines, 2007) suggested that the type of right of way of the light rail system has a significant effect on operation. LRT operating on the semi-exclusive right of way must improve operating speed and enhance safety while crossing intersections, LRT travel time must remain comparative to automobiles traveling in the same corridor.

Based on the result founded by (Chad & Lester, 2004) after testing different scenarios it appeared that the light rail crossing frequency and the number of vehicles at the crossings have a great effect on the average increase delays in vehicles. The total delay keeps increasing with increasing volumes and crossing frequencies when the road was oversaturated.

When LRT operates within the median of the street, the delays experienced depends on the turning movement and its relationship to the crossing location that conflict with the LRT tends to have higher delays

The above table explains the summary of the additional delay that was found for each of the variables.

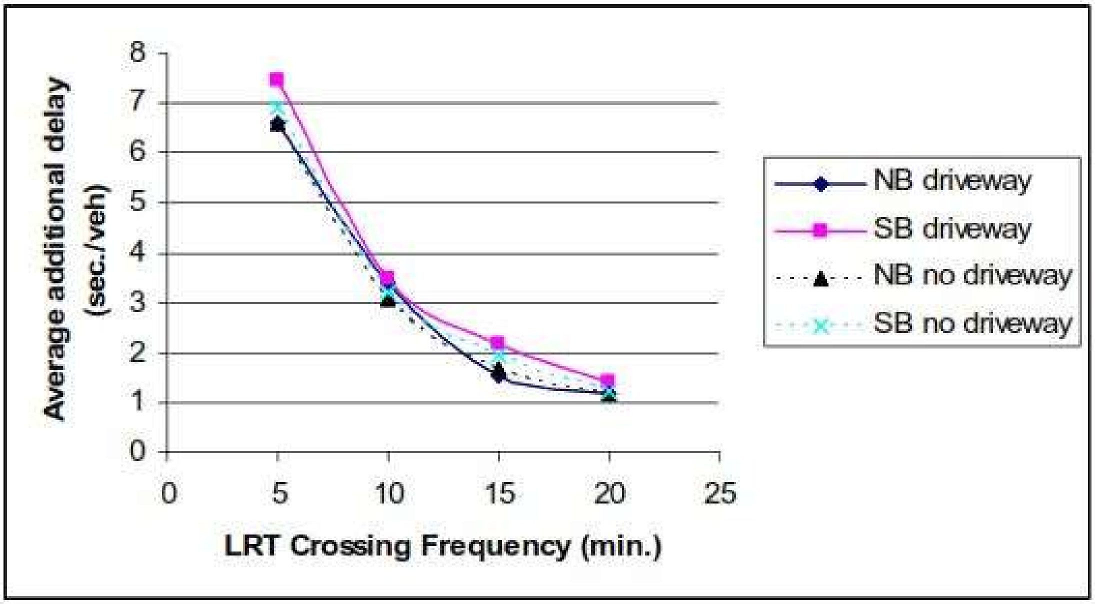

Figure 1: Average additional delays for driveway test scenarios

The above figure shows the average delays experienced by vehicles within a driveway upstream of LRT crossing in each direction. The preliminary results based on five simulation runs show that additional delays for this scenario were within 0.5 seconds /vehicle seconds/vehicle of the corresponding additional delays *without* driveways. This indicates that the presence of driveways near LRT crossings does not create a condition that would substantially affect the additional delays experienced by vehicles (Chad & Lester, 2004).

2.3 Microscopic Simulation Model

Microsimulation is the modeling of individual vehicle movements on a second or sub-second basis for the purpose of assessing the traffic performance of highway and street systems, transits, and pedestrians. (Dowling, et al., 2004) it is widely used in transportation operations and management analysis because they are safer, less expensive, and faster than field implementation and testing. it is a useful tool to effectively analyze and evaluate the proposed alternatives. For instance, an intersection can be simulated for different signal timing plans, changing geometries of the junction, different peak hour volumes and its effects are discovered before implementing it. Simulation models are widely used to analyze the traffic networks in different modes of transportation and various general or specialized simulation packages have been employed (Osman , et al., 2016)**.** A microscopic model of traffic flow attempts to analyze the flow of traffic by modeling driver to driver and driver to road intersections within a traffic stream which respectively analyzes the interaction between a driver and another driver on road of a single driver on the different features of a road (Mathew, 2014)**.**

2.3.1 Traffic Simulation Models

Traffic model Simulation is an increasingly popular and effective tool for analyzing a wide variety of dynamic problems associated with complex processes that cannot readily be described in analytical terms. Usually, these processes are characterized by the interaction of many system components or entities whose interactions are complex in nature. Specifically, simulation models are mathematical/logical representations of real-world systems, which take the form of software executed on a digital computer in an experimental fashion. The most important advantage is that these models are by no means exhaustive (Mathew, 2014)**.**

At the most detailed level, a microsimulation takes a vehicle from its network entry point to its final exit. Interactions between vehicles at intersections may be represented, and boarding and alighting at public transport stations or stops may be modeled in detail. At a less detailed level, a simulation may refer to platoons, user groups, lanes, competition among operators or parts of the transport network. At the microsimulation level, the purpose of modeling every vehicle in the network is to reproduce traffic conditions as close to the real conditions as possible. (Taplin, 2014) microscopic traffic flow characteristics are based on the trajectories on individual vehicles and the time history of speed and acceleration. A distinction is made between the maneuvers “following”, including “overtaking” and “passing”. In the passing maneuver it is important to know whether oncoming vehicles must be allowed for; if no, it is better to regard lane changing as a separate maneuver. The following model describes how a driver-vehicle element follows a vehicle ahead; a part is played by perception, decision, and vehicle control aspects. (Botma, 1981) many models have been drawn upon car-following and lane changing.

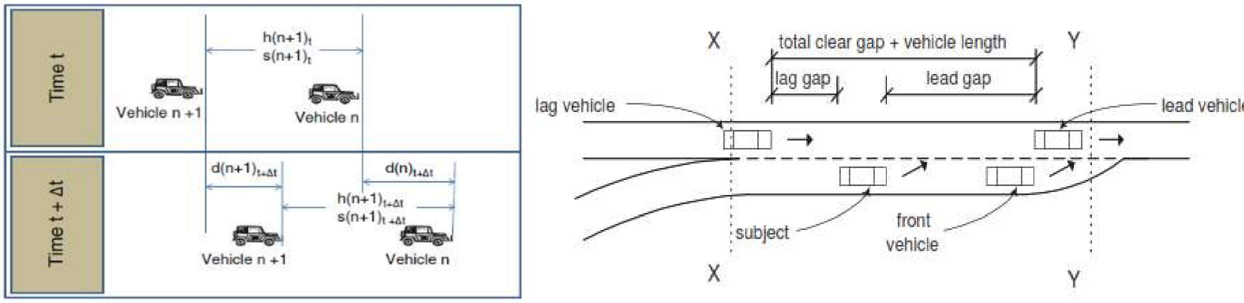

Figure 2: A conceptual model for car-following

The car following process is modeled using car-following algorithms. (Elefteriadou, 2014) explain that these algorithms determine the movement of the following vehicle at time t + dt, as a function of the relationship to a leading vehicle at time t. in the figure above a conceptual model for the relationship between the following vehicle (Vehicle n+1) and leading vehicle (Vehicle n) is illustrated. (Elefteriadou, 2014) further describes that at time t, vehicle n+1 is following vehicle n with time headway $h(n+1)_t$ and a space headway $s(n+1)_t$.

2.4 VISSIM

PTV VISSIM is the leading microscopic simulation program for modeling multimodal transport operations and belongs to the Vision traffic suit software. It is realistic and accurate in every detail, VISSIM creates the best conditions for you to test different scenarios. VISSIM is now being used worldwide by the public sector, consultants, and university. In addition to the simulation of vehicles by default, you can also simulate pedestrians based on the Wiedemann model (PTV VISSIM user manual 11, 2018).

Table 1: Simulation tools comparison

Simulation tool	Models bus routes	Left side bus stops	Accurate Bus Interaction	Bus schedule flexibility	Model short link	Import arterial and AutoCAD	3D Simulation
Corsim	Yes	No	Limited	Limited	No	No	No
Vissim	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Paramics	Yes	No	Yes	Limited	Limited	Yes	Limited
Simtraffic	No	No	No	No	Limited	No	No

Based on the above criteria, (Eddie, et al., 2001) it was determined that VISSIM was the most appropriate simulation software. VISSIM offered excellent modeling and complicated Public Transport (bus, tram) routes.

VISSIM is a microscopic, discrete traffic simulation system modeling motorway traffic as well as urban traffic operations. Based on several mathematical models. The system can be used to investigate private and public transport as well as pedestrian movements. Traffic engineers and transport planners assemble applications by selecting appropriate objects from a variety of primary building blocks. In order to simulate multi-modal traffic flows, technical features of pedestrians, bicyclists, motorcycles, cars, trucks, buses, trams, light (LRT), and heavy rail are provided with options of customization (Fellendorf & Vortisch, 2010).

Common applications include the following:

- Corridor studies on heavily utilized motorways to identify system performance, bottlenecks, and potentials of improvement.
- Advanced motorway studies including control issues like contra-flow systems, variable speed limits, ramp metering, and route guidance.
- Development and analysis of management strategies on motorways including mainline operation and operational impacts during phases of construction.
- Corridor studies on arterials with signalized and non-signalized intersections.
- Analysis of alternative actuated and adaptive signal control strategies in sub-area networks.
- Signal priority schemes for public transport within multi-modal studies. Traffic circulation, public transport operations, pedestrian crossings, and bicycle facilities are modeled for various layouts of the street network and different options of vehicle detection.
- Alignment of public transport lines with various types of vehicles such as Light Rail Transit (LRT), trams, and buses with refinements in design and operational strategy. This also includes operation and capacity analysis of tram and bus terminals.
- Investigations on traffic calming schemes including detailed studies on speeds during maneuvers with limited visibility.
- Presentation of alternative options of traffic operations on motorways and urban environments for public hearings.

2.4.1 Fundamental Core Models

VISSIM is based on a **traffic flow model** and **light signal control**. These exchange detector readings and signaling status. You can run the traffic flow simulation of vehicles or pedestrians as animation in VISSIM. You can clearly display many important vehicular parameters in windows or you can output them in files or databases, for example, travel time distributions and delay distributions differentiated by user groups. The **traffic flow model** is based on a car-following model (for the modeling of driving in a stream on a single lane) and on a lane-changing model. External programs for **light signal control** model the traffic-dependent control logic units. The control logic units query detector readings in time steps of one to 1/10 second. You can define the time steps for that reason and they depend on the signal control type. Using detector readings, e.g. occupancy and time gap data, the control logic units determine the signaling status of all signals for the next time step and deliver them back to the traffic flow simulation (PTV VISSIM user manual 11, 2018)**.**

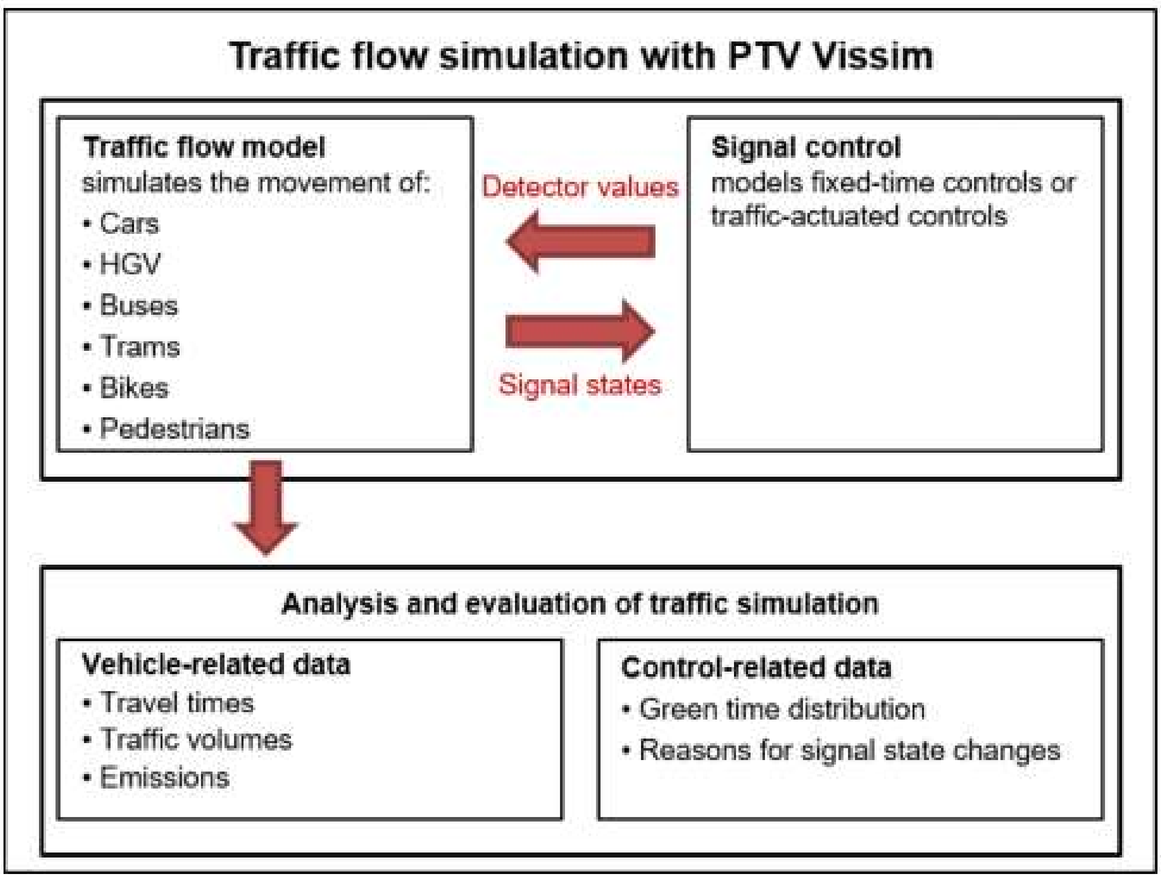

Figure 3: Traffic flow simulation with VISSIM

2.4.2 Operating principles of the car following model

Vehicles are moving in the network using a traffic flow model. The quality of the traffic flow model is essential for the quality of the simulation. In contrast to simpler models in which a largely constant speed and a deterministic car following logic are provided, VISSIM uses the psychophysical perception model developed by (Wiedemann, 1974). The basic concept of this model is that the driver of a faster-moving vehicle starts to decelerate as he reaches his individual perception threshold to a slower moving vehicle. (Ahmed & Kazi, 1999) since he cannot exactly determine the speed of that vehicle, his speed will fall below that vehicle's speed until he starts to slightly accelerate again after reaching another perception threshold. There are slight and steady acceleration and deceleration. The different driver behavior is taken into consideration with distribution functions of the speed and distance behavior (PTV VISSIM user manual 11, 2018).

VISSIM is the stochastic traffic simulator that uses the psycho-physical driver behavior model developed by R. Wiedemann (Wiedemann, 1974). VISSIM combines a perceptual model of the driver with a vehicle model. Every driver with his or her specific behavior characteristics is assigned to a specific vehicle. As a result, driver behavior corresponds to the technical capabilities of his vehicle. The behavior model for the driver involves a classification of reactions in response to the perceived relative speed and distance with respect to the preceding vehicle. Drivers can make the decision to change lanes that can either be forced by a routing requirement or made by the driver to access a faster-moving lane. Four driving modes are defined: free driving, approaching, following, and braking. In each mode, the driver behaves differently, reacting either to his following distance, or trying to match a prescribed target speed (VISSIM Calibration and Validation Technical Report, 2006).

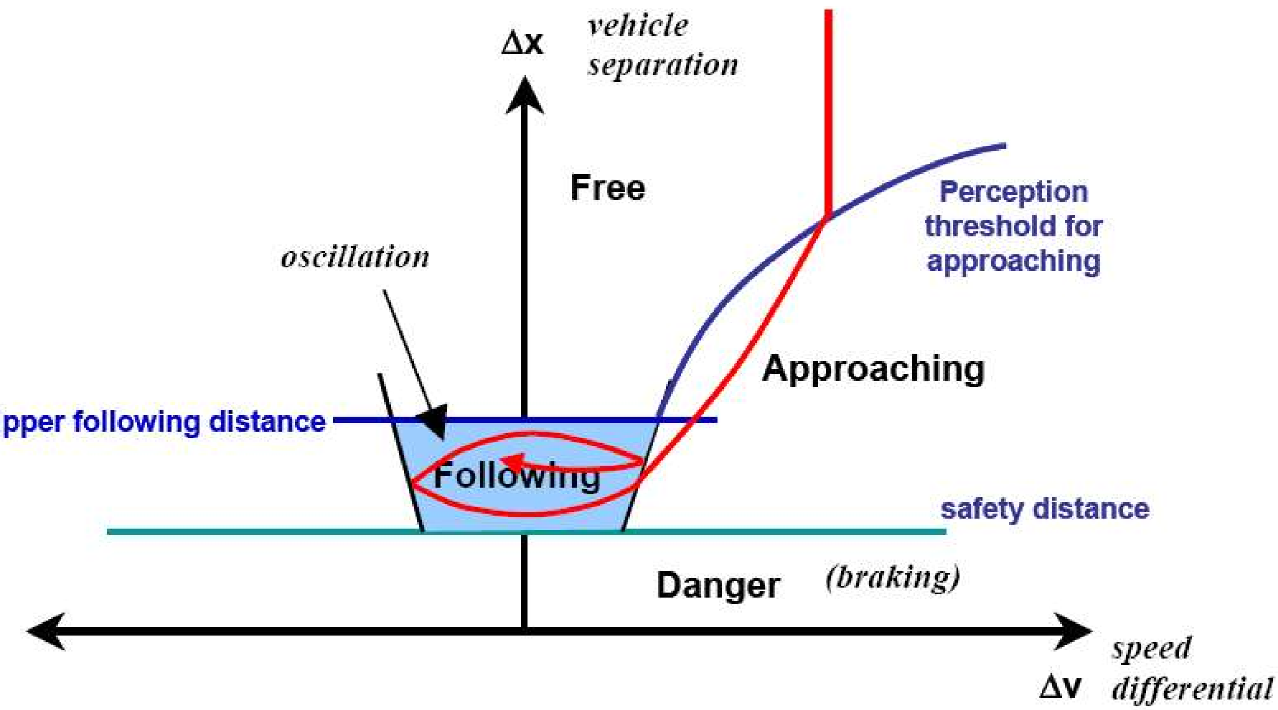

Figure 4: Wiedemann's car following theory

Wiedemann's traffic flow model assumes that there are basically four different driving states for a driver (Fellendorf & Vortisch, 2010);

- **Free driving**: No influence of preceding vehicles can be observed. In this state, the driver seeks to reach and maintain his desired speed. The speed in free driving will vary due to imperfect throttle control. It will always oscillate around the desired speed.
- **Approaching**: Process of the driver adapting his speed to the slower speed of a preceding vehicle. While approaching, the driver decelerates, so that there is no difference in speed once he reaches the desired safety distance.
- **Following**: The driver follows the preceding car without consciously decelerating or accelerating. He keeps the safety distance more or less constant. However, again due to imperfect throttle control, the difference in speed oscillates around zero.
- **Braking**: Driver applies medium to high deceleration rates if the distance to the preceding vehicle falls below the desired safety distance. This can happen if the driver of the preceding vehicle abruptly changes his speed or the driver of a third vehicle changes lanes to squeeze in between two vehicles (PTV VISSIM user manual 11, 2018).

2.4.3 Lateral Movement

Lateral movement in VISSIM can be structured in lane selection and lane-changing (Fellendorf & Vortisch, 2010)**.**

2.4.3.1 Lane Selection

If a driver is not aware of any necessary lane change because he is far away from the next relevant intersection, he chooses the lane with the best interaction situation. Three tests are performed: First, the driver decides if he wants to leave the current lane (Ahmed & Kazi, 1999)**.** This is the case whenever the interaction state (i.e., the driving mode in Wiedemann's car-following model) is different from free. Then he checks the neighboring lanes if there is a better interaction situation, i.e., either free or a higher time-to-collision. If one of the neighboring lanes provides a better situation downstream, the last check is if a lane change is possible considering the vehicles upstream, what is modeled as gap acceptance is described below in more detail.

However, lane selection is often governed by mandatory lane changes for desired turns at junctions downstream. In VISSIM's network coding, each connector has two distances attached: the lane changes the distance and the emergency stop distance. The lane change distance describes when a driver becomes aware of the upcoming connector; typical values range between 100 and 500 m. (PTV VISSIM user manual 11, 2018) from that point on he will consider the connector in his lane selection. The emergency distance is the distance to the connector where a driver will stop when he was not able to reach the necessary lanes to change to the connector. A connector leaving a link is typically attached to only some of the link's lanes, e.g., a single-lane right turn on a three-lane main road link will be connected only to the rightmost lane of the link. This means a car must be on the rightmost lane to change to the connector (PTV VISSIM user manual 11, 2018).

A driver who follows a defined route knows which connectors he has to take to follow this route. All these connectors have lane change distances, and there might be parts of the route where the driver is in the lane change distance of several connectors at the same time (when the distance between the connectors is less than the lane change distances). The driver considers all connectors that he is aware of when selecting a lane. The desired lane is the one that allows him to follow his route through the upcoming connectors with the least number of mandatory lane changes.

2.4.3.2 Lane Changing

The actual lane-changing logic in VISSIM is used to decide if it is possible to change to the desired neighbor lane or not. The desired lane is a result of the lane selection process for either free or mandatory lane changes based on gap acceptance: A driver is willing to accept that he forces a lag vehicle on the desired lane to decelerate. (Ahmed & Kazi, 1999) the value of this accepted deceleration is a matter of calibration, and for mandatory lane changes it is as well a function of the distance to the emergency stop position of the next connector where the lane change has to be completed, i.e., the driver becomes more aggressive closer to the point of an emergency stop. In a similar way, the driver is willing to accept to decelerate himself in case of a mandatory lane change. (PTV VISSIM user manual 11, 2018) the accepted deceleration values for lag vehicles upstream and for the vehicle itself are parameters of the behavior model and can be defined selectively for pairs of link and vehicle types.

2.5 VISSIM Calibration and Validation

Calibration is the process in which the various parameters of the simulation model are adjusted until the model accurately represents field conditions. The parameters of VISSIM, which affect the behavior of network created in it, are adjusted during calibration so that the model replicates field conditions. the procedure which provides an acceptable level of confidence in the model results (Siddharth, et al., 2013). During VISSIM calibration, model outputs were compared against field data to determine if the output was within acceptable levels (VISSIM Calibration and Validation Technical Report, 2006).

The numerous calibration parameters that can be modified are categorized based on their characteristics. They are (Siddharth, et al., 2013)

- Desired speed distributions
- Acceleration/Deceleration distribution
- Driving behavior parameter

2.5.1 Defining the Wiedemann 74 model parameters

This model is an improved version of Wiedemann 1974's car-following model. the following parameters are available. Average standstill distance (w74ax) defines the average desired distance between two cars. The tolerance lies from-1 to +1m which is normally distributed at around 0.0 m, with a standard deviation of 0.3 m. default value 2.0.

Additive part of safety distance (w74bxAdd) value used for the computation of the desired safety distance, d. Allows adjusting the time requirement values. Default 2.0 Multiplicative part of safety distance (w74bxMult) value used for the computation of the desired safety distance, d. Allows adjusting the time requirement values. Greater value = greater distribution (standard deviation) of safety distance. Default 3.0 (PTV VISSIM user manual 11, 2018),

The desired safety distance is calculated from:

$$\mathbf{d = ax + bx} \quad (1)$$

where ax is standstill distance

$$\mathbf{bx} = (\mathbf{bxadd} + \mathbf{bxmult}(z))\sqrt{\mathbf{V}}$$

V: vehicle speed (m/s)

Z is a value with range {0,1}, which is normally distributed around 0.5 with a standard deviation of 0.15

Validation is the process whereby modeled outputs are compared against independent collected observed data. (Preater, 2013) model Validation tests the accuracy of the model by comparing traffic flow data generated by the model with that collected from the field. Validation is directly related to the calibration process because adjustments in calibration are necessary to improve the model's ability to replicate field measured traffic conditions (Park & Schneeberer, 2002)**.**

2.5.2 VISSIM Calibration

Microscopic traffic simulation software tools have become increasingly more popular in recent years to analyze traffic operation. Traffic simulation models use a stochastic process to model traffic conditions given a set of geometry, traffic demand, vehicle routing, and driving behavior inputs. The calibration and validation stage of a simulation is the key components of the modeling process.

Calibration is defined as the adjustment of computer simulation model parameters to accurately reflect prevailing conditions of a road network. Example of adjustable model parameters

- Lane changing aggressiveness
- Car following behavior
- Lane change gap acceptance
- Route choice
- Vehicle speed distribution

- Vehicle acceleration distribution

According to (Park & Schneeberer, 2002) based on their study once a model parameter is calibrated, an evaluation is conducted to assess the credibility of the model. At first simulation, animation outputs are checked against real-world traffic to see if there exists a significant difference. Then quantitative indicators are selected and compared with the calibrated model and field observations. Furthermore, checking the model variability using random seed numbers. Multiple runs are necessary in order to acquire a credible simulation result. Based on the significant level, acceptable error and result variability depend on the required number of runs.

Validation is defined as the process of comparing simulated model results with field measurements to determine the accuracy of the simulated model. After identifying the validated parameter settings then it can be used as the baseline setting that reflects overall driving behavior. When analyzing future scenarios baseline parameter settings have not been modified. Once the model is validated then it can be used with confidence to analyze future scenarios some modifications to trip distribution, travel demand and change in geometry.

2.5.2.1 Method for Calibration

Calibration of the microsimulation model is a key component to the success of the simulation There are three different methods of calibration

1. Base model development
2. Planning of calibration approach
3. Model calibration and validation

A. **Base Model Development**

This is the first stage in the process of calibration. It provides input to the calibration planning stage. During the base model development study area size, data collection requirement and selection of time period should be made.

B. **Planning calibration approach**

The second stage in the calibration process is to plan the calibration approach. The major tasks in this stage include; selection of Measure of performance (MOP), determination of validation target, data collection for calibration and validation.

C. **Selection of MOP**

In the validation stage, it has been compared to simulated values for chosen MOP to field values of the same MOP. The validation process is used to determine how closely the simulated model replicates real-world field conditions. Validation measurement can be quantitative or qualitative.

D. **Quantitative MOP**

It is easily measured in the field. Examples are travel time, vehicle speed, vehicle acceleration rate, congestion maps, queue length, etc.

E. **Quantitative MOP**

They are not easily quantifiable and requires sound judgment or field observation like queuing and congestion done through video or field inspection. MOP qualitative includes; visual inspection of queuing, off-ramp lane change distance, car following characteristics, lane changing acceleration rate.

F. **Statistical validation**

There are a large number of statistical validations of the simulation model. These approaches include the goodness of fit measures, confidence interval and statistical tests (Tomer & Harris, 2004).

The choice of the appropriate methods and their application depends on the validation of the traffic simulation models depends on the nature of the output data. The following cases are considered;

- Single valued MOP (average delay, total throughput)
- Multivariate MOP (Time-dependent flow, travel time on the different section)

G. **Goodness of fit**

A number of the goodness of fit measures can be used to evaluate the overall performance of simulation models (Tomer & Harris, 2004) (John , et al., 2003). Popular among them are;

- GEH Statistic

  The GEH statistic is a formula to compare two sets of traffic volumes and gets its name from Geoffrey E. Havers, who invented in 1970. A formula in a form of chi-squared statistic that is designed to be tolerant of larger errors in low flows. (Traffic Modelling Guidlines, 2013) (Dowling, et al., 2004)

- Root Mean Square Error (RMSE)
- Root Mean Square Percent Error (RMSPE)
- Mean Error (ME)
- Mean Percent Error (MPE)

  Indicates the existence of systematic under or over predictions in the simulated measurements
- The correlation coefficient (r)

  Is another popular measure used to show the linear associations between the simulated and observed traffic measurement (John , et al., 2003)
- Theil's Inequality Coefficients (U)

  A measure that provides information on the relative error bounded between 0 and 1 implying perfect and worst fit possibilities respectively. It is decomposed into three proportions of inequalities, Bias (Um), Variance (Us) and covariance (Uc) proportions. (Tomer & Harris, 2004)

GEH & RMSE are a useful measure of goodness of fit between modeled and observed (Traffic Modelling Guidlines, 2013). Simulation models provide a wealth of information with regards to performance measures, including delay, queue length, and emission.

2.5.3 Operational Measures of Effectiveness

The following are the most commonly used measures of performance used to evaluate compare the fidelity of the simulation model.

2.5.3.1 Delay

Vehicle delay is considered as one of the most important measures of performance (MOP) in intersection traffic operations because it allows traffic engineers to evaluate the overall performance of a traffic system. Currently, vehicle delay is used as a principal performance measure to determine the intersection level of service (LOS), estimate average speed, and calculate fuel consumption and emissions. Therefore, it is essential to develop a reliable method to accurately measure the delay in real-time. (Shatanawi, et al., 2018).

According to the Transportation Research Board's Highway Capacity Manual (HCM , 2010) control delay is defined as the additional travel time experienced by a vehicle affected by intersection control. Vehicle delay is one of the most important measures of performance (MOP), as it allows traffic engineers to evaluate the performance of a signal system. (Shatanawi, et al., 2018). (Chaudhry & Ranjitkar, 2009) presents that the delay calculated with the analytical approach is comparable to that of predicted through the simulation model.

2.5.3.2 Capacity

The Highway Capacity Manual defines the capacity as the maximum hourly rate at which persons or vehicles can be reasonably expected to traverse a point or a uniform segment of a lane or roadway during a given time period, under a prevailing roadway, traffic, and control conditions (HCM , 2010). It is the maximum rate at which vehicles can pass through a given point in an hour under prevailing conditions; it is often estimated based on assumed values for saturation flow.

Capacity accounts for roadway conditions such as the number and width of lanes, grades, and lane use allocations, as well as signalization conditions. (Chaudhry & Ranjitkar, 2009) discussed in their study in the comparison between microsimulation and the analytical model shows that a correlation exists in volume to capacity ratio calculated through a simulation model and analytical approaches. It was also observed that the output is almost identical to that of the HCM 2000. They also stated that "This study revealed that a micro-simulation approach can address some of the limitations of the analytical approaches in the capacity analysis of signalized intersection."

2.5.3.3 Queue Length

A queue is important to measure of effectiveness that should be evaluated as a part of the analysis. In simulation model vehicle queues are needed to determine the amount of storage needed for turn lanes and weather a spill occurs at upstream facilities.

Approaches that experience an extensive queue are a representation of rear-end collision likelihood. The main objective is to optimize the operation of the existing traffic system and solve the traffic problem at the intersection.

2.5.3.4 Emission

Traditionally intersection and segment evaluation has focused mainly on efficiency and safety; but in recent years, the increasing importance of air pollution produced by vehicular traffic has suggested that environmental considerations should be added to the above aspects as a criterion for intersection evaluation.

One of the benefits of using a microsimulation model is that it provides you with Carbon mono Oxide, Nitrous Oxide, Volatile Organic Compounds, and fuel consumption. Intersections are critical elements of road networks in terms of air quality impact, and their control type and geometric configuration can affect significantly vehicular emissions. At intersections vehicles usually slow down and often stop, thus interrupting traffic flow in varying patterns (Claudio , et al., 2017).

2.6 Literature Summary

Regardless of the research done in congestion assessment through analytical process and using sidra intersection software (Mohammed, 2017) (Kassahun, 2007) (Tadele, 2017). There is a limited scientific research identifying the most significant parameters and their contribution in congestion along the light rail including at grade crossing and station crossing. Zone of influence in most of the researches done is very small. The utilization of microscopic simulation models through accurate representation of field condition via calibration and validation (Siddharth, et al., 2013) (Park & Schneeberer, 2002).

Having a properly calibrated model, changes can be made to assess the effect of light rail transit corridor through a simulation. By assessing the car following parameter and having filed acquired measures of performance the effect of on street parking, heavy vehicle and signal introduction can be obtained at a microsimulation level.

CHAPTER THREE: RESEARCH METHODOLOGY

In this section of the chapter the research methods, materials and procedure will be used are explained.

3.1 Description of The Study Area

The study was conducted in Addis Ababa, Ethiopia along the Light Rail Transit. Which contains two lines running with a total length of 31.6Km and 39 stations. The research will be conducted at segments where LRT is running at grade and at an intersection with the highway (grade crossing) of Line 2 called South-North Corridor. Saris – Adey Ababa – Nifas silk 1; there is one at grade intersection LRT and Highway.

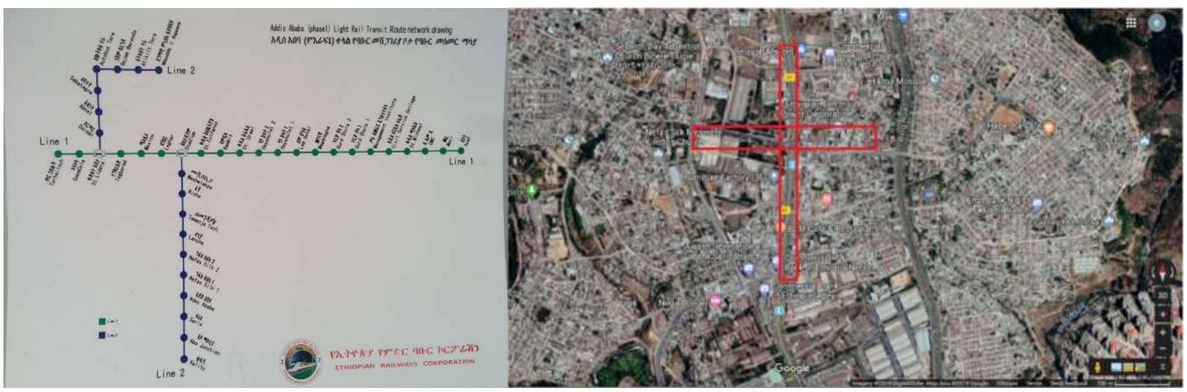

Figure 5: Study area

3.2 Description of The Study Corridors

Generally, Problems associated with grade crossing

- High pedestrian volume most of the time and extremely high at the peak,
- High traffic volume which leads to extreme congestion,
- A high number of turning vehicles due to access management,
- Sight distance problem,
- Even if there is signal violation is too much,
- High potential for a pedestrian surge,
- On-street parking,

From the Above corridor

- Severe peak-hour congestion at Adey Ababa intersection has been observed,
- The very extreme pedestrian movement has been observed at the following; Adey Ababa and Saris,
- Severe peak-hour congestion at Adey Ababa,
- Extremely high pedestrian volume each day,
- Passenger car dominate the traffic composition,

Driving Condition

- Small Car and minibus taxi are major commuters;
- Medium frictional interruption present;
- Very high friction between turning and through movements of Adey Ababa intersection

3.3 Research Approach

The research approach in this thesis involves using PTV VISSIM for base model development, model calibration, validation and scenario of at grade intersection to determine intersection delay, queue length, and emission. To do this task primary data collected through video recording and manual extraction was executed. A base model that represents the field conditions was constructed and calibration of the base model to best mimic the field condition through the system and the operational calibration technique was done. Calibrated model stability tested using a different random number of seeds to account for day to day variability of vehicular inflow. The model was then validated with a new set of data and then different scenarios were tested against the existing condition.

3.4 Research Design

This section will provide the overall effort of the research. Great attention was given on which type of data to focus on. That is the study design, and as a result, in the following topic, several types of research were reviewed in raw to see what parameters there are that shall be captured. And narrowed down to the following data.

In order to see what parameters, affect the traffic many factors have been tried to capture simultaneously and categorized as traffic-related, pedestrian-related, transit-related, driver behavior- related. These factors are needed in any of the following forms; counts, statistically determined by sample size, through observation, and analysis on a desk.

3.5 Data Requirement

Data required for the study are categorized below; contains traffic-related, pedestrian-related, transit-related, and geometric related.

- Geometric data: Number of lanes, lane width
- Vehicle data: *Volume: 15-min volume, Composition: Car, Minibus, Bus, Truck, tram: Movement type: through, right-left, and U-turns*
- Desired speed: approach speed
- Reduced speed area: reduced speed area and reduced speed
- Travel time: 1km travel time
- Pedestrian data: *Pedestrian volume: 15- min volume: Crossing type: location, crossing behavior*
- Priority
- Tram Headway
- Parking data: *Occupancy: vehicle parking occupancy from all bays: Turnover:*
- Bus loading unloading points
- Traffic sign, marking & other inventories
- Traffic movement behavior

On the following chart, the data used with their respective categories is assigned to explain what type of data used.

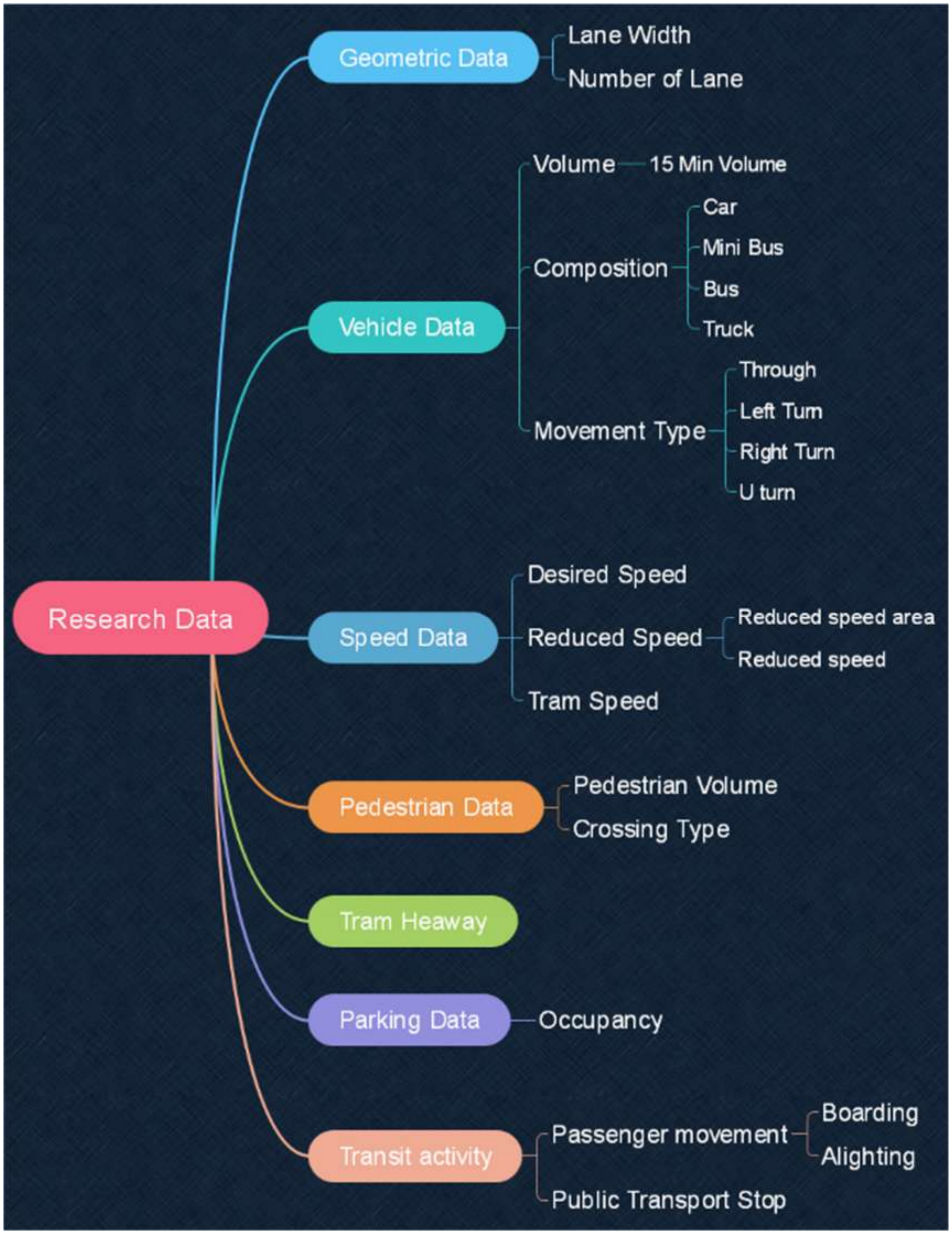

Figure 6: Research data

3.6 Data Sample size

1. Traffic volume

According to the (Traffic engineering manual, 2002), a 2-hours of morning and evening count with 15 minutes of the interval were done for three consecutive mid-day of Tuesday, Wednesday and Thursday.

2. Vehicle composition

As per the (PTV VISSIM user manual 11, 2018) vehicles for traffic study shall be classified as car, minibus, bus, and truck based on VISSIM pre-existing vehicle composition availability in the software.

3. Travel Time

Travel time data is one of the most important data types in the VISSIM model. According to **Travel time collection handbook** by (Shawn, et al., 1998) for license plate matching with an average coefficient of variation 0.25 for congestion of traffic 1 to 2 hours with a 95% confidence interval 96 match as a minimum is required.

The Sample size for travel time $n = (\frac{z * c.v}{e})^2$ **(2)**

Where z= z-statistics based on a confidence interval

E= relative permitted error (%)

c.v= coefficient of variation

4. Vehicular speed

For assessing the vehicular reduced speed, space means value will be used for an error of E=/x-μ/=0.5 standard deviation of 3 and 95% confidence interval sample size. The collection of reduced speed data will help in VISSIM modeling to produce a queue and congestion condition.

$$n = \left(\frac{z\sigma}{E}\right)^2 = \left(1.96 * \frac{3}{0.5}\right)^2 = 138\ Vehicles$$

5. Parking studies

The parking studies were made for the entire analysis period of 2 hours.

6. Geometric characteristics

Functional classification, lane (width, number), median (type, width), walking (walking, utilization), length of road segment, transit line data, transit activity location

7. Pedestrian data

Crossing and parallel movement volume of the pedestrian in peak condition of 15-minute interval.

8. Light rail transit headway

The headway of light rail transit headway was collected for the entire analysis period of 2 hours.

9. Transit activity

The transit data collected in the current transit includes, Alighting and boarding passenger, transit crossing a location of boarding and alighting passenger in transit vehicle data was gathered for the entire analysis period of 2 hours.

10. Number of model repetition

Based on (Dowling, et al., 2004) with a minimum difference in means of 2 and 90% confidence interval is 6.

$$CI_{(1-a)} = 2 * t_{(1-a/2),N-1} * \frac{s}{\sqrt{N}} \quad \textbf{(3)}$$

3.7 Research methods

3.7.1 Data collection

To produce a quality VISSIM model, an intensive amount of data was collected to build the base model. in addition, the level of calibration and validation requires additional primary and secondary data,

3.7.1.1 Primary data collection

A video camera was used to record the vehicular and pedestrian movement at Adey Ababa at grade intersection and Saris station crossing for two hours in the morning and evening peak hours in midweek of Tuesday, Wednesday and Thursday except for Monday and Friday since there exist exaggerated traffic movement on these days. A total of a 12-hour video was recorded for analysis. The morning peak was captured from 7:00-9:00 AM and the evening peak was from 4:00-6:00 PM at the study locations Adey Ababa and Saris from 14 February to 16 February 2019. The video was captured so it can capture all the legs of the intersection including the road and light rail movement.

Figure 7: Adey Ababa Intersection

Figure 8: Saris station crossing

3.7.1.2 Secondary data collection

The secondary data used for the research were; Geometric data and areal image from google maps for VISSIM background.

3.7.1.3 Data Extraction

Vehicle volume, routing decision, pedestrian volume, light rail headway, transit activity was extracted from video using video playback technique. Using a stopwatch for speed. Plate matching method for travel time and vehicle count method for parking studies.

3.8 Method of data analysis

The reliability of the simulation model is determined by its result of how close to the reality since a trial and error method of the calibration was adopted to calibrate the model. Results in high accuracy error between the simulated and field conditions; has to be checked using error-checking mechanisms and statistical analysis methods. After obtaining the simulation results data were analyzed in order to check for variability in field and model result. GEH statistic, Root mean square error, Theil's Inequality coefficients and other error checking methods were utilized during the calibration and validation of the model.

1.GEH - The GEH statistic is a formula to compare two sets of traffic volumes and gets its name from Geoffrey E. Havers, who invented in 1970. A formula is a form of chi-squared statistic that is designed to be tolerant of larger errors in low flows.

Table 2: Calibration guideline

Criteria and measures	**Calibration acceptance target**
Hourly Flows, Model Versus Observed	
Individual Link Flow	
Within 15%, for 700 veh/h <Flow<2700	> 85% of the cases
Within 100 veh/h, for flow< 700 veh/h	> 85% of the cases
Within 400 veh/h, for Flow >2700veh/h	> 85% of the cases
Sum of All Link Flows	within 5% of the sum of all links count
GEH Statistic < 5 for individual Link Flow*	> 85% of the cases
GEH Statistic for the sum of All Link Flows	GEH < 4 for the sum of all the Link Counts

Travel time Model Versus Observed	
Journey times, Network	
Within 15%	> 85% of the cases
Visual Audit	
Individual Link Speeds	
Visually Acceptable Speed Flow Relationship	To analyst satisfaction
Bottlenecks	
Visually Acceptable Queuing	To analyst satisfaction

$$GEH = \sqrt{\frac{(Yi-Xi)^2}{\frac{Yi+Xi}{2}}}$$, where Xi is simulated value and Yi is observed value (4)

2. Root Mean Square Error (RMSE) - is a frequently used measure of the difference between values (simulated) predicted by VISSIM and the value observed. It is the quadratic mean of the difference between simulated and observed values. A lower value is advisable.

$$RMSE = \sqrt{\sum_{i=1}^{n} \frac{(Yi-Xi)^2}{n}}, \quad (5)$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals.

3. Mean Absolute Percent Error (MAPE) - Is the absolute percent difference between the simulated and observed value? Cannot be used if there are zero values and ca not exceed 100 %

$$MAPE = \left(\frac{100\%}{n}\right) \sum_{i=1}^{n} \left| \frac{(Yi-Xi)}{Yi} \right| \quad \textbf{(6)}$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals.

4. Theil's inequality coefficients - A measure that provides information on the relative error bounded between 0 and 1 implying perfect and worst fit possibilities respectively. It is decomposed into three proportions of inequalities, Bias (Um), Variance (Us) and covariance (Uc) proportions.

$$U = \frac{\sqrt{\frac{1}{n}*(\sum(Yi-Xi)^2)}}{(\sqrt{\frac{1}{n}*\sum Yi^2}) + \sqrt{\frac{1}{n}*\sum Xi^2})}, \quad (7)$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals. U is bounded between 0 and 1 implying perfect and worst fit respectively

$$Um = \sqrt{\frac{(n*(Y-X)^2)}{(\sum(Yi-Xi)^2}}, \quad (8)$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals. Bias proportion Um which measures the symmetric error over count or undercount caused by excess or loss of vehicle

$$Us = \sqrt{\frac{(n*(\sigma y-\sigma x)^2)}{(\sum(Yi-Xi)^2}}, \quad (9)$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals.

σy and σx are standard deviations of simulated and observed

Variance proportion Us

$$Us = \sqrt{\frac{(2n*(1-r)\sigma y \sigma x)}{\sum(Yi-Xi)^2}} \quad (10)$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals. Covariance proportion Uc, the measure of unsymmetrical error and r is correlation coefficient

The following table describes the satisfactory limit value for Theil's inequality

Table 3: Theil's inequality coefficients boundary

Parameter	Desired Values
U	[0,1]
Uc	>0.9
Um	<0.1
Us	<0.1

5. correlation coefficient - Is another popular measure used to show the linear associations between the simulated and observed traffic measurement.

$$r = \frac{1}{n-1} * \sum_{i=1}^{n} \frac{(Xi-X)(Yi-Y)}{\sigma y \sigma x}, \quad (11)$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals.

r is the correlation coefficient, $\sigma y,$ and σx are standard deviations of simulated and observed,

6. ME and MPE - Indicates the existence of systematic under or over predictions in the simulated measurements.

$$ME = \frac{1}{n} * \sum_{i=1}^{n} (Yi - Xi) \quad (12)$$

$$MPE = \frac{1}{n} * \sum_{i=1}^{n} \frac{Yi - Xi}{Yi} \quad (13)$$

Where Xi is simulated value, Yi is observed value and n is the number of intervals.

The above tests are performed to quantify the total percentage error and indicate the existence of under or over prediction of the simulator. In addition to that according to the recommendation from (Park & Schneeberer, 2002), a statistical test was performed to check whether simulated vs. modeled results and model stability using different seed numbers significantly different with 0.05 significance level.

1. Student T-test

In comparing mean differences between the existing condition and simulated model

2. Dunnett's Multiple comparisons

We are not always interested in the comparison between the two groups. Sometimes, we may have determined whether differences exist among the means of three or more groups. Dunnett's multiple comparisons used to test if differences exist among groups with a control variable.

With Null and alternative hypothesis of;

Ho: All means are equal

H1: All means are not equal

3.9 Research Procedure

In this part, the detailed research procedure will be explained including base model development, model calibration, model validation, and the VISSIM coding process. The research has gone through many stages in the coming, the first task

3.9.1 Base model Development

Traffic congestion assessment is a major task in the city's transport assessment. Congestion assessment is a *very expensive, data-intensive and time-consuming* task to perform using traditionally accepted methods. Nowadays, the development of traffic simulation software has contributed to making this kind of assessment easy to assess and implement. VISSIM is one of the greatest multimodal microscopic simulation software developed by the German PTV AG Groups. The software passed a 40 years journey in the making, development and refinement process.

In this research, it has been tried to assess the effect of Light Rail Transit aka LRT, effect travel corridor congestion. It mainly focuses on at grade intersection located in Addis Ababa, Ethiopia known as Adey Ababa at a grade crossing. At first assessment of existing intersection through a thorough data collection for base data development has been conducted.

Data has been collected for morning and evening peak times. Morning peak and evening peak for three consecutive mid-week working days (Tuesday, Wednesday and Thursday). For the model, the maximum of this has been selected. The VISSIM simulation modeling started on May 16, 2019. Starting from this day forward development of the base model, calibration, validation and scenario management have taken about two months. The purpose of simulating this model is to find out and alleviate the existing congestion problem in the research area through a multimodal microscopic traffic simulation model using VISSIM. Different researches have been done before targeting congestion mitigations but due to many limitations, it was incapable of reducing congestions. But by using VISSIM which has the capability of modeling transit activity alongside road transport due to its multimodal nature of the software it was able to simulate many scenarios. VISSIM 9.00-13 was used for simulation and modeling. All parameters used in this model are kept in default unless stated otherwise in this research.

a. **Model Extent**

The network object in the model is based upon the existing corridor and around a 1km corridor, length has been considered including ***at grade intersection*** and ***two station crossings***. During the base model preparation stage of the research as shown in the figure below.

Model duration and evaluation periods - The model developed has been made to mimic the following time of day field traffic movement;

1. Morning peak of ***7:00-9:00 AM***
2. Evening peak of ***4:00-6:00 PM***

The following figure represents the sum of the observed traffic flow in the intersection approaches during the peak hour for the morning and evening traffic. The profile describes that peak hour flow in the intersection is from ***7:00-9:00 AM*** for morning and ***4:00-6:00 PM*** for the evening flow.

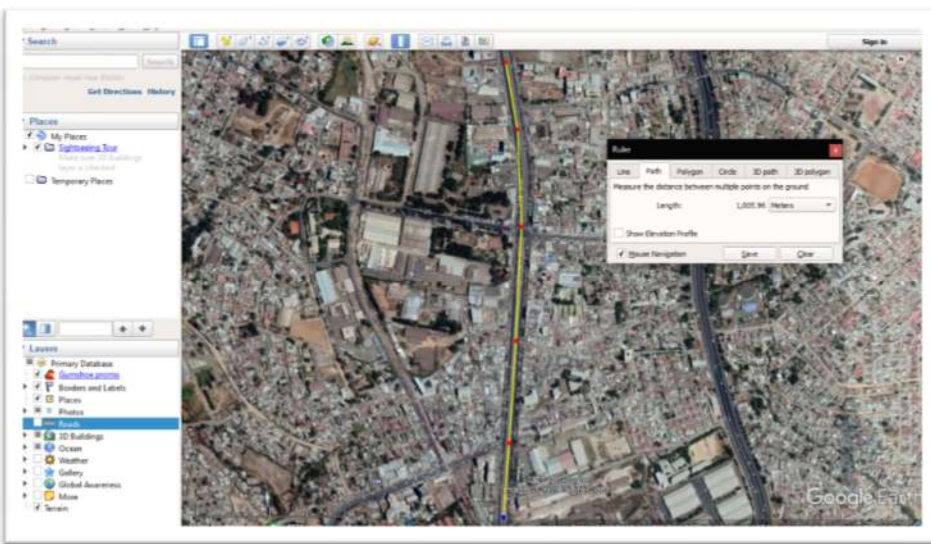

Figure 9: 1-km research area Adey Ababa- Saris

b. **Simulation resolutions**

The model as simulated by ***10-time steps per second*** of resolution. Meaning the driver's decisions are assessed 10 times every second and in order to be consistently maintained throughout the base model development, calibration, validation and scenario development stage of the research. Since there is no preferred value different kinds of literature recommend a ***minimum of 5-time step per second***.

c. **Traffic Input Data**

Traffic count and travel time have been collected for morning and evening peak times. Morning peak from 7:00-9:00 AM and evening peak from 4:00-6:00 PM for three consecutive mid-week working days (Tuesday, Wednesday and Thursday). For the model, the maximum of this has been selected.

d. **Traffic volume and composition**

Vehicle routing count data at Adey Ababa intersection was collected at a ***15-min interval*** and classified into the following vehicle compositions***; Car, Minibus, Bus, and Truck***. This information was required to develop the traffic demand matrix for the model.

e. **Travel time**

Travel time data was obtained by ***manual plate matching technique*** setting up collectors at the different end of the two approaches of the intersection (Saris and Yosef leg). The data has been ***collected for peak hours*** only. A ***1km*** traverse distance was taken for travel time collection since the travel time handbook recommends a ***minimum of 800m*** (Shawn, et al., 1998)**.**

f. **Pedestrian data**

The pedestrian data was undertaken alongside the vehicular data ***at the 15-min interval*** at both Adey *Ababa intersection and Saris station crossing* for both morning and evening period.

g. **Transit Data**

Transit data about *alighting and boarding passenger* and *headway* for the analysis period.

h. **Base Model Development**

A base model that represents the field condition was created by using the network object elements in VISSIM. A network of 1 Km section was created containing one at grade intersection at Adey Ababa and two stations at Saris and Adey Ababa.

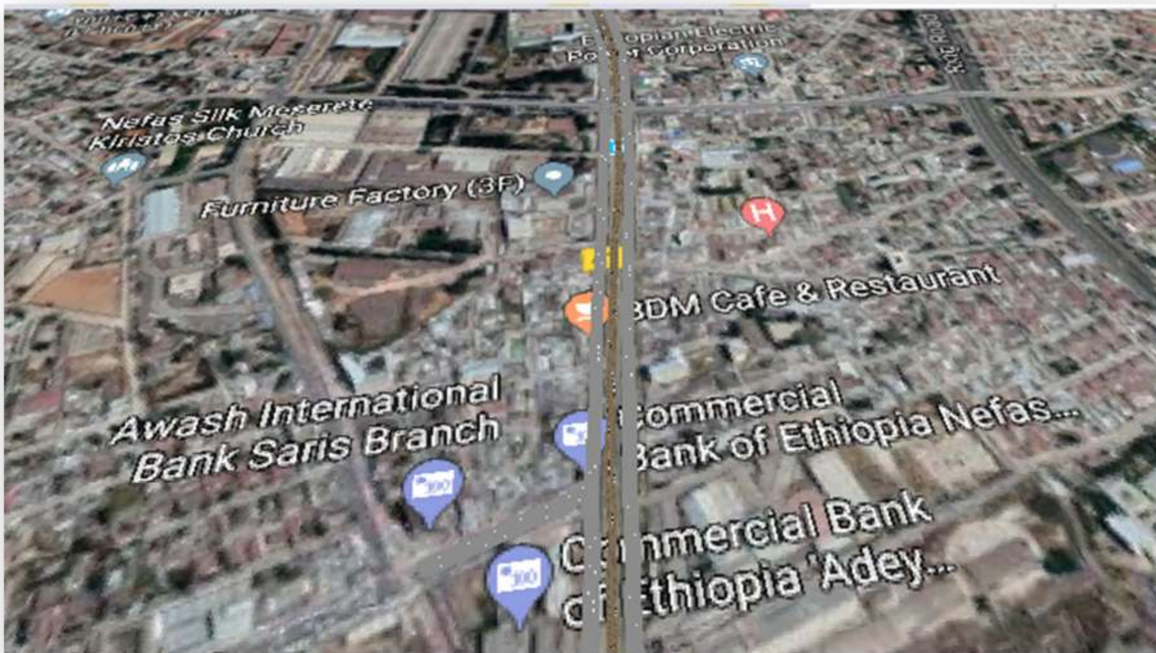

Figure 10: Snapshot VISSIM base model

The network elements used to create the base model were; ***Links, connectors, reduced speed area, desired speed, conflict area, priority rule, public transport***…

A background image from google maps with a scale of ***1:200*** meters was used and the model was created by *superimposing* over the background to mimic the existing layover. Links from Saris, Yosef, Addis Sefer and Biheretsigie based on the actual number of lane and width have been prepared; connectors as per vehicles routing in the field as shown in the figure above.

Table 4: Descriptive statistics of speed data

approach	Mean	Standard Error	Median	Standard Deviation	Kurtosis	Skewness	Range	Minimum	Maximum
Yosef	18	1	18	6	3	2	25	11	36
Saris	16	1	15	5	0	1	19	8	27

After the road network was created ***reduced speed*** areas were inserted as per the field study. After speed study, ***the reduced speeds of 16 km/hr. from Saris and 18 km/hr. from Yosef*** were found.

Reduced speed was added as per its distribution as shown in the following figures.

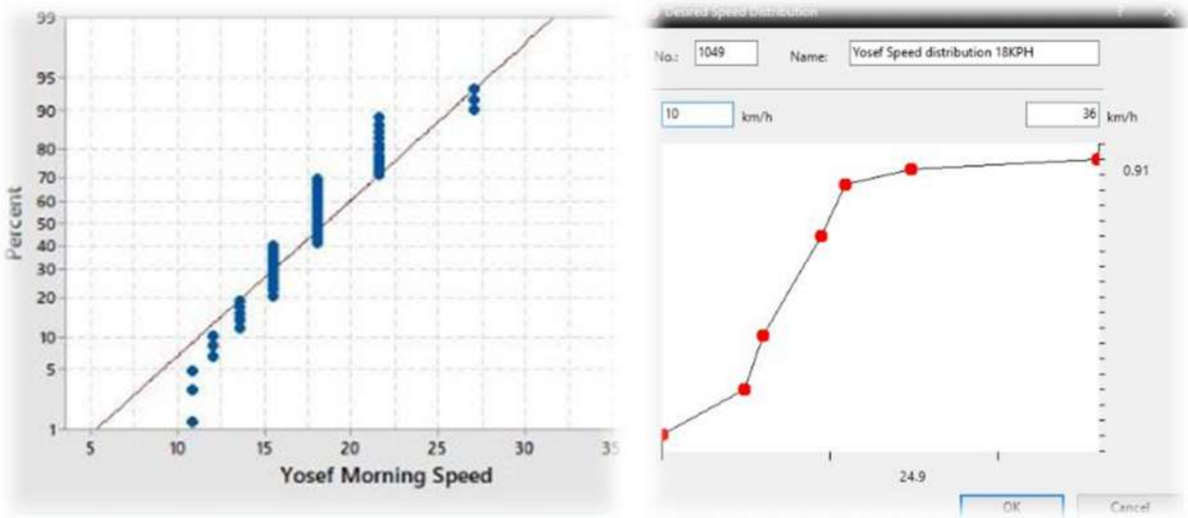

Figure 11: Speed distribution curve

The reduced speed distribution of each vehicle was given as input for the simulation model in VISSIM. The maximum and minimum values (10,36 km/hr.) of the speeds and distribution between these were defined in the model. the reduced speed distribution profile for the vehicles as shown in the figure below. The speed distribution for the vehicle is generally an “S” shaped curve as shown in the figure. Adequate care was taken to ensure that the speed distribution defined in VISSIM represented the values observed in the field. The values have been given based on the observed data to accurately represent field conditions through the simulation model and thereby an attempt has been made to estimate the realistic output. Vehicles approach ***desired speed*** is also found to be ***40 km/hr.*** at the intersection.

A priority rule for light rail transit was given in the model since the transit movement should not be interrupted by both vehicular and pedestrian movement. Then a vehicle and light rail transit route decision was given with their corresponding relative flow as shown in the table below,

Table 5: Vehicle relative flow

	Approach-from Saris	Approach-from Bihertsigie	Approach-from Addis Sefer	Approach-from Yosef
RT	3%	7%	31%	5%
LT	9%	84%	31%	9%
TM	71%	10%	38%	60%
U-T	17%	0%	0%	27%

Table 6 above describes the relative flows emerging from each leg of the at grade intersection. Vehicle composition and vehicle flow based on the field observations are given as an input to the simulation model for the given time interval. In the model, a vehicle type of Car, Minibus, Bus, Truck, and Tram used with their corresponding composition for each leg as shown in the table below. The vehicle model deals with defining each vehicle type that is plying on the field and hence considered for the simulation.

Table 6: Vehicle composition

	Composition			
	Car	**Minibus**	**Bus**	**Truck**
saris	48%	39%	6%	7%
Biheretsigie	79%	14%	1%	6%
Addis safer	69%	23%	1%	7%
Yosef	61%	31%	2%	6%

The above table contains the percentage share of vehicle types in each leg. There is a higher share of private cars and very low public transport usage. Flow is given as input based on the observed data encompassing different time intervals and different directions collected from the traffic volume count survey. Vehicles have been randomly generated following exponential distribution as per the observed volume and compositions are given in the model input. The vehicle input in each leg which is the hourly vehicular demand of the directional hourly volume of the peak hour was given as an input to the software with its vehicle composition.

Table 7: directional hourly volume

Morning	Approach	**saris**	**Biheretsigie**	**Addis Sefer**	**Yosef**
	DHV	976	653	392	1166

As shown in the above table 8 there is the vehicle demand from major lane Yosef and Saris are very high.

3.9.2 Base Model Calibration

After building the base model the next stage to accomplish is to calibrate the base model to best mimic the field conditions for further assessment and evaluation or scenario development. The model was calibrated in both *operational and system-level* using morning traffic data. The system calibration stage was performed by the suppling of system element data through detail field studies such as ***desired speed decision, reduced speed area, conflict, priority, and parking***.

The desired speed of 40km/hr. was assigned in the model as a vehicular approach speed. Then a reduced speed of 18km/hr. and 16km/hr. with respect to their speed distribution as shown in figure 11 above by making the probability plot of speed was also given for the model.

The model was able to produce 112 conflict points; which needs giveaway assignment and all conflict points were assigned to replicate the field condition.

Parking occupancy and duration for both on-street parking and minibus loading/ unloading areas were field studies are done (***see appendix***) it was found that almost all vehicles were parked for the *2-hour analysis period* and an average loading and unloading of the minibus was able to be found *2 minutes with 100 percent attraction rate*.

The operational calibration stage was done by changing VISSIM's driving behavior. The driving behavior in VISSIM comprises car following, lane changing and lateral movement. But in the study, only the car following behavior parameter was used for model calibration and other parameters remain default unless described. VISSIM car following behavior was based on Professor R. Wiedemann's model. Since Wiedemann 99 model is for freeways (PTV VISSIM user manual 11, 2018) Wiedemann 74 car following was used for model calibration.

After ***hundreds of simulations run*** it was able to calibrate the model. The methodology in the literature was used for calibration. During calibration planning the first thing was to determine the

measure of performance where we can compare simulated and field values; ***travel time and vehicle throughput (Intersection capacity)*** were selected as a measure of performance for calibration and validation of the model.

The calibration parameters used were ***average standstill distance, additive part of standstill distance and multiplicative part of standstill distance*** with default value in VISSIM ***2, 2 and 3*** respectively. ***A trial and error method of the operational calibration was utilized***, after many trials and error checking ***0.5, 0.15, and 0.15*** was found the best among the many trial outputs analyzed and the field study conducted on-site through a detail observation on the average standstill distance and safety distance between consecutive vehicles. After running the simulation for the 7200 seconds of analysis period result of travel time and vehicle throughput with a target of ***RMSE within 15%*** boundary and ***GEH statistics of less than 5*** were found.

VISSIM uses a random seed number to compensate for day to day vehicle arrival. This is performed in VISSIM by changing the random number of seed (RNS) in the simulation parameter settings. The first analysis was performed using a random seed number of 42. But in order to check for ***model stability,*** a minimum of five repetitions with different seed numbers was advisable but, in this research, model stability was checked for seven different random numbers of seed.

3.9.3 Validation of Model

The validation stage executed in order to check whether the calibrated model was able to validate using another set of data which is; the evening traffic field result. Here, without changing the calibrated model only by giving the evening result such as volume input, composition, relative flow for routing decision, pedestrian input, etc.… was able to check the validity of the calibrated model. Using the validated model, it was able to obtain a model output with seven different random seed numbers.

3.9.4 Condition Assessment and scenario development

In this stage of the research, it was conducted that what significant factors have caused the bottlenecks and also other possible congestion alleviating scenarios were introduced in order to reduce the travel time and delay in section. In addition, queue length, number of stops, emission and capacity were used as comparative variables across the scenarios developed.

Scenarios were compared using ***traffic parameters*** such as; *capacity, delay, travel time, queue length, and the number of stops* and ***environmental factors*** such as; *CO emission, NO emission, volatile Organic Compound (VOC) and fuel consumption is* ***recorded only to see the percentage difference***. Here it has to be noted that the above parameters used to show the variability in the existing condition which is created using the VISSIM, since it has been calibrated and validated with different number of seed in order to check the fidelity of the simulation model reproducing the existing field condition as much as possible, meaning we can utilize the other parameters provided by the tool to our advantage to show and check the difference in scenarios developed.

The assessment scenarios developed are described as follows,

Scenario 1: Double Light Rail Transit Frequency

Scenario 2: Double Light Rail Transit Frequency and Avoid on-street parking

Scenario 3: Avoid on-street paring

Scenario 4: Reduce Heavy Vehicle

Scenario 5: Signal control introduction

Scenario 6: Signal control introduction plus double light rail transit frequency

According to the field study and observation plus detail literature review on similar mitigations strategies done during the research period the researcher has come to develop scenarios, which are believed to be effective in reducing the congestion. The first scenario is developed on the concept of the designed frequency of the light rail transit is not currently being implemented and this case assesses the effect of the current LTR frequency with smaller headway. While the second scenario looks into the effect of the first scenario plus on-street parking restrictions. The third scenario provides the effect of on-street parking during the peak hour of the day. The fourth scenario presents the heavy vehicle reduction since it cannot be completely restricted a two percent heavy vehicle rate is utilized in this model. scenario five is all about the introduction of the signal in the at grade crossing within the mind to give a priority to the transit vehicle. Indeed, active transit signal priority was implemented in VISSIM to evaluate the effects. The last scenario presents scenario five-plus double the light rail frequency.

The following flow chart shows the research procedure step by step.

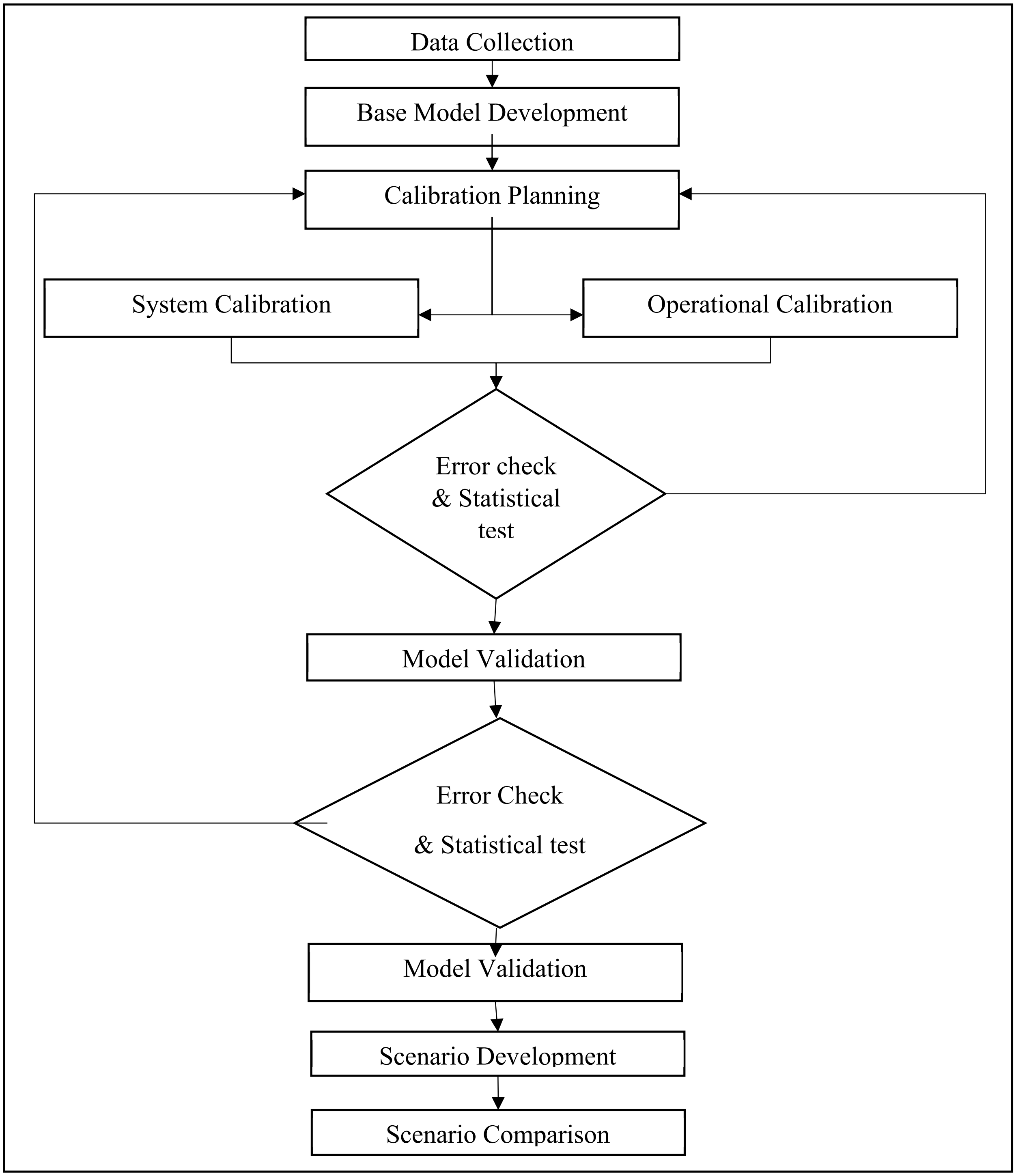

Figure 12: Research procedure

CHAPTER FOUR: RESULTS AND DISCUSSION

In this chapter, research finding is presented and discussed analytically, statistically, graphically and in tabular forms. The chapter is subdivided into six parts; Assessing existing conditions, development a base model using VISSIM, Calibration of VISSIM, validation, assessing congestion contributing factors, Scenario comparison.

4.1 Background Traffic Information

In this part of the research traffic data, travel time and transit analysis of the existing condition result and discussion will be presented.

The first part of the traffic analysis discussion will be about the traffic demand and composition. Figure 13 shows a chart that describes briefly the volume of hourly demand for the Morning and evening traffic. According to the chart, there is a high influx of vehicle demand from Yosef and saris approach.

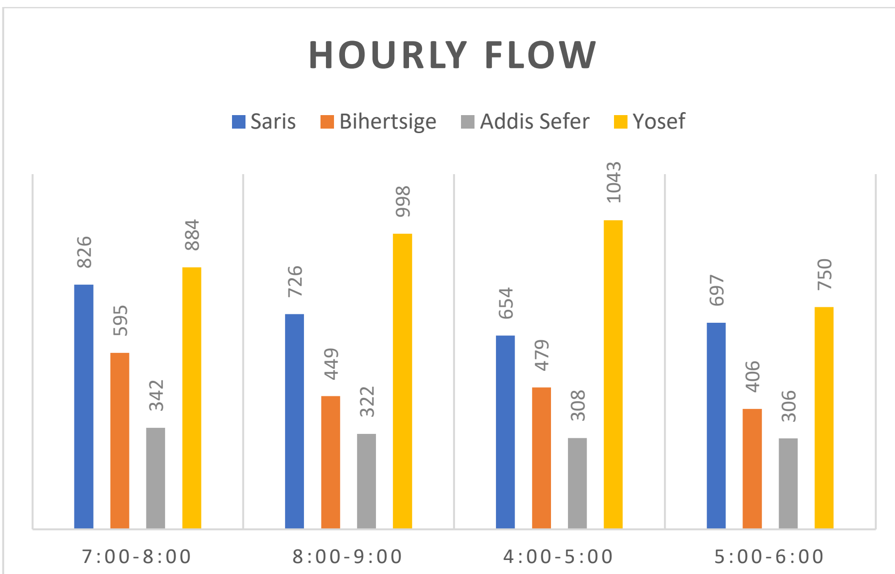

Figure 13: Hourly vehicular flow

The traffic count in the above figure 13 specifies the vehicle hourly flows obtained in the analysis period. The peak hours were found to be 7:00-8:00 AM and 4:00-5:00 PM. The vehicular distribution in the 15-minutes interval was found is in figure 14 below. It shows the minimum and maximum numbers of vehicles in the 15- minutes interval in the intersection.

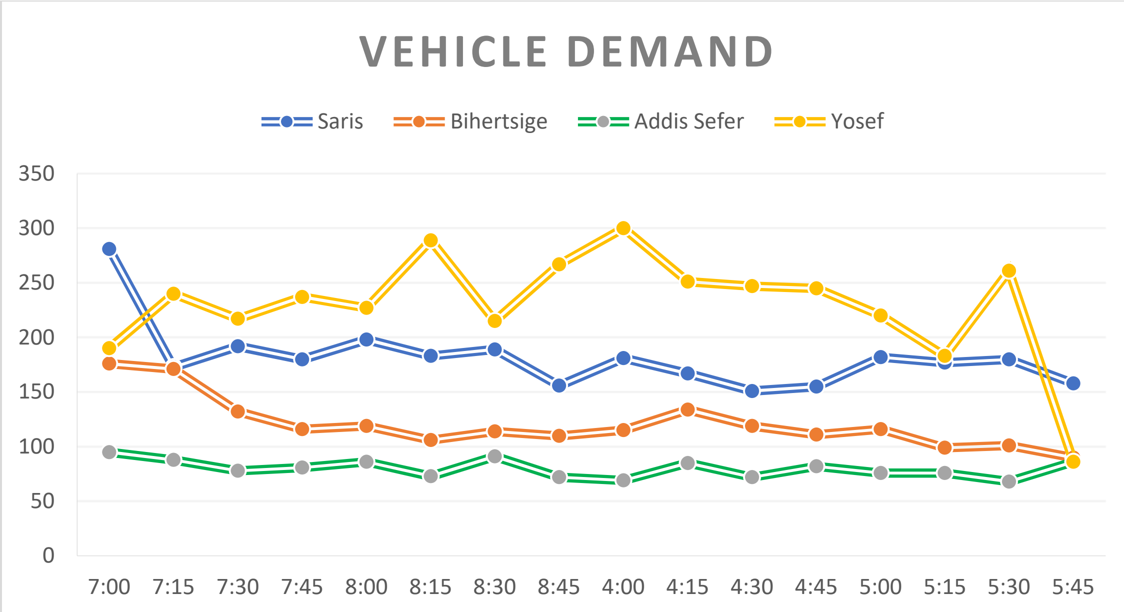

Figure 14: 15-min vehicular distribution

Figure 15 below expresses the vehicle composition for both morning and evening periods. A significant number of car and minibus share is observed from all legs. It can be seen that all legs have high vehicular flow. Yosef leg has a great proportion of traffic compared to the others.

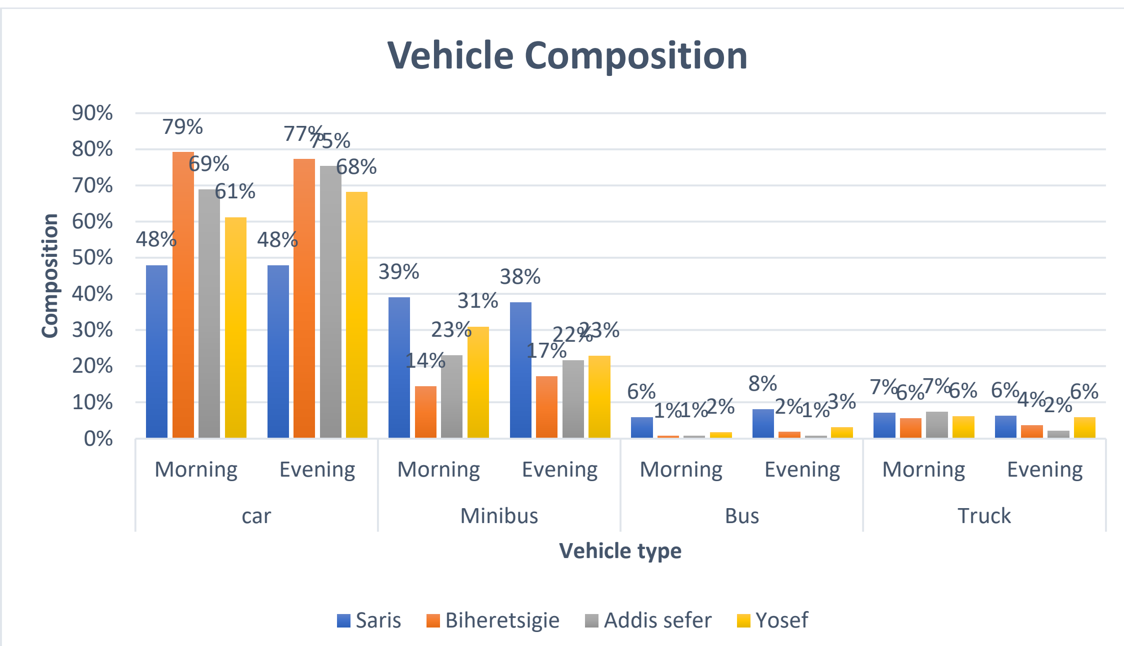

Figure 15: Vehicle Composition

The relative vehicle routing is plotted in Table 9 below. There is 71% and 60% through movement from Yosef and Saris approaches respectively. 84% left turning a vehicle from Biheretsigie and an even distribution of relative flow from Addis Sefer leg is observed. It can be noted that there exists a very great proportion of car and minibus over the other vehicle types.

Table 8: Vehicle relative flow

	Approach-from Saris	Approach-from Bihertsige	Approach-from Addis Sefer	Approach-from Yosef
RT	3%	7%	31%	5%
LT	9%	84%	31%	9%
TM	71%	10%	38%	60%
U-T	17%	0%	0%	27%

The above result shows that there exists a moderate amount of U-turn movement in the major street and this was due to the access management problem because of the median running LRT line, turning movement is allowed in the at grade crossing only.

According to the vehicular composition study done the percentage of heavy vehicle was found to be as shown below. The percentage of the heavy vehicle does not exceed 10% as shown in table 10 below. Even they have a considerable effect on the intersection performance measures.

Table 9: Percent Truck

PERCENT TRUCK		
	Morning	Evening
saris	7%	6%
Biheretsigie	6%	4%
Addis safer	7%	2%
Yosef	6%	6%

The following table 11 contains the hourly volume which is given or the simulation model, as can be observed from the result Approach Yosef and Saris has 1166 veh / hr. and 976 veh /hr. during the morning peak hour of the day respectively. While, 653 veh /hr. and 392 veh /hr. flow from Biheretsige and Addis Sefer was observed.

Table 10: Directional Hourly Volume

DHV (Directional Hourly Volume)				
	from saris	**from Biheretsige**	**from Addis Sefer**	**from Yosef**
morning	976	653	392	1166
evening	864	548	343	1247

The peak hour factor is the total volume ration to the maximum 15-minute rate of flow within the hour. Table 12 presents the PHF of the intersection.

Table 11: Peak hour factor

	Peak hour factor	
APPROACH	**Morning**	**Evening**
saris	0.73	0.97
biheretsigie	0.88	0.96
Addis safer	0.94	0.88
Yosef	0.89	0.87

The intersection has a high amount of pedestrian movement in all legs, table 13 presents the pedestrian data collected from the intersection.

Table 12: Pedestrian crossing

Hour	**Minute**	**Morning**	**Evening**
7:00am -8:00am	**0-15**	534	589
	15-30	625	703
	30-45	692	769
	45-60	802	882
8:00am - 9:00am	**0-15**	854	975
	15-30	700	957
	30-45	589	931
	45-60	490	862

4.1.2 Transit Activity

In this section of the research, it will cover the transit activity will be discussed in details. As one of the main parameters in the simulation modeling stage, the following table 14 shows the light rail vehicle headway.

Table 13: LRT Headway

Movement	Headway in min.
LRT Headway, NS	12
LRT Headway, SN	15

According to the above LRV headway result, it can be seen that the South north headway is 15 minutes which is larger than the North-south headway of 12 minutes. The frequency of the light rail vehicles in North southbound is greater than the South northbound.

The transit activity in stations is one of the major parameters to be used in the model since the research is comprised of two station the data collection result for the alighting boarding and crossing passenger is summarized in the following section of the research As can be seen from the result in the table there is a significant amount of transit-oriented activity in saris compared to that of Adey Ababa station, this is due to the fact that in saris the business district activity is very high. The pedestrian crossing in saris is also high at the station but in Adey Ababa, pedestrians use the at grade intersection for crossing.

The table 15 below presents the number of alighting passengers in the light rail transit service during the analysis period at Adey Ababa. The average number of alighting passengers indicate 62 passengers per hour during the morning period and 102 passengers per hour during the afternoon period.

Table 14: Number of Alighting Passengers at Adey Ababa

Number of alighting Passengers		
Minute	**Morning**	**Afternoon**
0-15	8	10
15-30	10	31
30-45	25	12
45-60	11	-
0-15	18	59
15-30	31	33
30-45	13	71
45-60	7	41

Table 16 below describes the number of pedestrians crossing at the permitted pedestrian crossing location at Saris station. The result indicates that there are 4284 pedestrians per hour crossing the station and the corridor during the morning period while 5202 pedestrians per hour crossings

Table 15: Saris Station crossing pedestrians

	Number of pedestrian crossing	
	Morning	**Evening**
0-15	1003	990
15-30	1111	990
30-45	1183	1046
45-60	1291	1336
0-15	1182	1316
15-30	1042	1538
30-45	920	2171
45-60	836	1016

Table 17 below contains the number of alighting passengers at Saris during the morning and evening periods. The result shows that the average 236 passengers alighting from the transit vehicle while there are only 336 passengers alighting from the transit vehicle during the evening period. This show that there is a high number of alighting passengers from the transit service at Saris during the evening period.

Table 16: Saris station alighting passengers

Alighting Passengers		
	Morning	**Evening**
0-15	60	77
15-30	36	92
30-45	61	91
45-60	54	-
0-15	39	119
15-30	57	135
30-45	68	101
45-60	97	57

4.1.3 Travel time results

The travel time result is summarized in the following table. The travel time data is used for model calibration and validation stage of the research. The models' ability to accurately replicate the travel time within the permissible boundary is important for the microsimulation model.

Table 17: Travel time

Travel Time in minutes	
From Saris to Yosef	**From Yosef to Saris**
6.5	12

The above result is obtained from averaging the recorded travel times in North-south and South north directions during the research period.

4.2 Model Calibration Result

The objective of the calibration process is to obtain the best possible match between model performance estimates and the field measurements of performance. It may be noted that there are no universally accepted procedures for conducting the calibration and validation of a transportation network. The responsibility lies with the modeler to implement a suitable procedure that provides an acceptable level of confidence in the model results. In this particular research a two-stage error checking, meaning with volume throughput and travel time and the statistical test was conducted in order to see whether the field result has been replicated in the simulation modeling.

4.2.1 Volume throughput

The model was first run with a random seed number of 42 by default, by which changing the random seed number changes the vehicular arrival type. Since VISSIM is a stochastic microsimulation model, it uses Monte Carlo distribution for vehicle inserting. The model was calibrated with Wiedemann 74 driving behavior parameters. With ***0.5 m, 0.15 and 0.15*** average standstill distance, additive part of standstill distance and multiplicative part of standstill distance respectively. The model result is summarized in the **appendix-A,** and table 19 contains the different error-checking results for volume throughput made between the field data and the model result.

Table 18: Base Model output for vehicle throughput

42 random seed number			
Morning	***Result***	***Remark***	***Condition***
GEH	0.056	Pass	<4
RMSPE	7%	Pass	<15%
MAPE	9%	Pass	<15%
ME	-0.5	Pass	low
MPE	-1%	Pass	plus, or minus 10%
U	0.066	Pass	close to 0
Um	3.411E-05	Pass	<0.1
Us	0.003	Pass	<0.1
Uc	1.14	Pass	>0.9

The above table 19 shows the error checking result for the default random seed number and as per the result, the GEH value of 0.05 for all link was found to be less than requirement making the simulation and the field vehicle throughput model replication perfect match. The GEH value has been adopted as the main indicator of the extent to which modeled traffic flows match the corresponding field data. From the error checking assessment result shown above it can be seen that the base model was calibrated successfully. Now the model needs to be checked for stability for vehicle variability.

After the calibration of the base model six other models similar to the calibrated model in everything except the random seed numbers in order to check for model fidelity to account day to day vehicle arrival difference. Model stability was checked with the use of different random seed numbers in the model parameter of VISSIM interface and the following results were found for six different random seeds for vehicle throughput.

Table 19: Model stability result in-vehicle throughput

Morning	***Random seed number***					
	35	***34***	***33***	***31***	***37***	***41***
GEH	0.056	2.571	1.553	1.346	0.084	0.834
RMSPE	14%	15%	14%	16%	10%	14%
r	-0.683	-0.486	-0.792	-0.907	-0.717	-0.723
MAPE	12%	13%	11%	13%	8%	3%
ME	0.5	23.25	14	12.125	-0.75	7.5
MPE	-0.87%	-4.35%	-3.04%	-2.88%	-0.53%	-2.00%
U	0.071	0.073	0.076	0.083	0.056	0.076
Um	0.00003	0.059	0.020	0.013	0.0001	0.006
Us	0.024	0.056	0.033	0.052	0.017	0.043
Uc	1.119	1.020	1.087	1.076	1.126	1.093

Table 20 shows that seven random seed numbers are used in the simulation for testing model stability. Random seed numbers used are 35, 34, 33, 31, 37 and 41. Using the two-hour flow in the model corresponding error or difference was able to be obtained. According to the above assessment, the vehicle throughput presented in the above section provides confidence that the modeled corridor field and simulation performance results to match.

4.2.2 Travel Time

The acceptability range set in this research for travel times should be within 15% of the corresponding field values (or within 1 minute) in at least 85% of the cases. The observe mean travel times of the corridor has been used to compare against simulated values. According to figure 16, travel time variability between the existing field value simulated value with different random seed numbers.

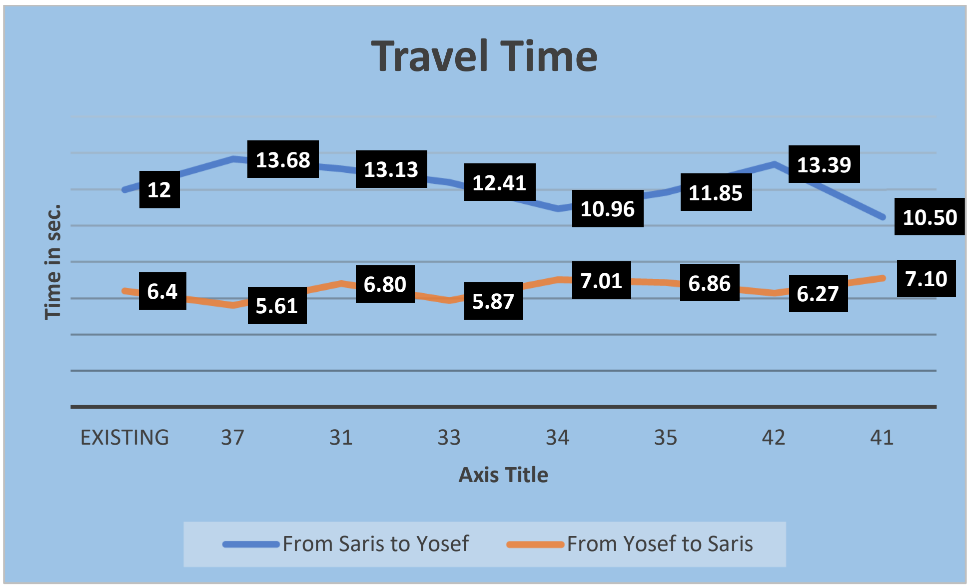

Figure 16: Travel time

Table 21 compares the mean-field and simulated travel times from saris to Yosef and Yosef to saris during the AM peak hour of the day. As can be seen from the result below the travel time outputs have to meet the criterion. The above table shows travel time measure is within the required boundary of 15%. The difference between the field and simulated was less than the target value.

Table 20: Travel time result for base model and stability

		37	31	33	34	35	42	41
Travel Time	From Saris to Yosef	13.68	13.13	12.41	10.96	11.85	13.39	10.5
	From Yosef to Saris	5.61	6.8	5.87	7.01	6.86	6.27	7.1
RMSPE	From Saris to Yosef	12%	9%	3%	-10%	-1%	10%	-14%
	From Yosef to Saris	-14%	6%	-9%	9%	7%	-2%	10%

The result is a good indication of modeled simulation is accurately replicating the field conditions. But a further statistical investigation can have to be made.

4.2.3 Statistical evaluation of calibration model

To show that there is no statistically significant difference between field and simulated values with different random seed numbers student t-test for the level of significance 0.05 was checked for travel time, flow and intersection capacity. The detail step wise analysis is in **appendix-B**.

Flow in field and simulation with different random seed number results were tested. The result of the equal variance test **(Brown-Forsythe)** indicates the likelihood that the two groups are sampled from populations with equal variances but does not guarantee the equality or inequality of the two variances. Use the results of **Welch's test**, where equal variances are not assumed if the equality of the population variances of the two groups is in doubt.

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups with their p values higher than the 0.05 significance.

Table 21: Calibration result in a statistical result

Test Subjects	**Description**	**Vehicle throughput**		**Travel Time**	
Statistical test summary, Existing Vs RNS 34	t-test	P-Value	Status	P-Value	Status
	Normality test	0.374	Passed	0.277	Passed
	Equal variance test (Brown-Forsythe	0.159	Passed	p<0.05	Failed
	Equal variance Assumed (student t-test)	0.44	Accept	0.956	Accept
	Equal Variance not Assumed (Welch's test)	0.442	Accept	0.956	Accept
Statistical test summary, Existing Vs RNS 42					
	Normality test	0.251	Passed	0.354	Passes
	Equal variance test (Brown-Forsythe	0.906	Passed	p<0.05	Failed
	Equal variance Assumed (student t-test)	0.984	Accept	0.971	Accept
	Equal Variance not Assumed (Welch's test)	0.984	Accept	0.971	Accept
Statistical test summary,					
	Normality test	0.395	Passed	0.069	Passed

Existing Vs RNS 35	Equal variance test (Brown-Forsythe	0.532	Passed	p<0.05	Failed
	Equal variance Assumed (student t-test)	0.9845	Accept	0.971	Accept
	Equal Variance not Assumed (Welch's test)	0.985	Accept	0.971	Accept
Statistical test summary, Existing Vs RNS 33					
	Normality test	0.172	Passed	0.092	Passed
	Equal variance test (Brown-Forsythe	0.683	Passed	p<0.05	Failed
	Equal variance Assumed (student t-test)	0.623	Accept	0.989	Accept
	Equal Variance not Assumed (Welch's test)	0.624	Accept	0.989	Accept
Statistical test summary, Existing Vs RNS 31					
	Normality test	0.405	Passed	0.073	Passed
	Equal variance test (Brown-Forsythe	0.549	Passed	p<0.05	Failed
	Equal variance Assumed (student t-test)	0.691	Accept	0.873	Accept
	Equal Variance not Assumed (Welch's test)	0.692	Accept	0.873	Accept
Statistical test summary, Existing Vs RNS 37					
	Normality test	0.397	Passed	0.297	Passed
	Equal variance test (Brown-Forsythe	0.625	Passed	p<0.05	Failed
	Equal variance Assumed (student t-test)	0.972	Accept	0.936	Accept
	Equal Variance not Assumed (Welch's test)	0.972	Accept	0.937	Accept
Statistical test summary, Existing Vs RNS 41					
	Normality test	0.101	Passed	0.472	Passed
	Equal variance test (Brown-Forsythe	0.679	Passed	p<0.05	Failed
	Equal variance Assumed (student t-test)	0.797	Accept	0.916	Accept
	Equal Variance not Assumed (Welch's test)	0.798	Accept	0.916	Accept

The above table 22 shows that all the student t-test for vehicle throughput and travel time result between existing and simulated results have p-value greater than the significance 0.05. meaning the difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups. The P-values of the above table is clear that the performance of the simulation model with the different random number of seeds is statistically similar. With the statistical comparison, and the p-value is greater than its significance 0.05, the difference between these multiple cases can be considered as the test fails to say statistical different.

4.3 Validation of Model

The validation stage executed in order to check whether the calibrated model was able to validate using the evening traffic field result. Here, without changing the calibrated model only by giving the evening result such as volume input, composition, relative flow for routing decision, pedestrian input, etc. was able to check the validity of the calibrated model. Using the calibrated model, it was able to obtain a model output with seven different random seed numbers. All results obtained are within the required boundary giving us a validated model to proceed.

Table 22 Model validation Result

Evening	**Random seed number**						
	42	35	34	33	31	37	41
GEH	3.195	1.706	0.671	1.281	2.044	0.983	2.663
RMSPE	12%	10%	9%	10%	12%	12%	11%
MAPE	9%	9%	7%	7%	10%	10%	10%
ME	28.25	15	5.875	11.25	18	8.625	2350%
MPE	-4.90%	-2.71%	-1.11%	-2.05%	-3.20%	-1.65%	-4.05%
U	0.058	0.052	0.045	0.048	0.061	0.059	0.057
Um	0.152	0.055	0.011	0.035	0.057	0.014	0.108
Us	0.224	0.212	0.273	0.228	0.335	0.356	0.306
Uc	0.748	0.877	1.158	0.875	0.743	0.771	0.713
LOS(ALL)	F	F	F	F	F	F	F

4.4 Scenarios comparison with traffic-related parameters

4.4.1 Intersection capacity

The capacity of an intersection is one of the vital parameters we can use in order to analyze the differences in scenarios.

Table 23: VISSIM Capacity output of the simulation model

Capacity (veh/hr.)		
Scenarios	**Capacity**	**Difference in %**
Existing	2569	
double LRT frequency	2530.5	-1.52%
Avoid on-street parking and double LRT	2584.5	0.60%
set 2% heavy vehicle	2651.5	3.11%
Avoid on-street parking	2729.5	5.88%
Signal passive LRT, 136 sec	2497.5	-2.86%
Signal with LRT doubled	2509	-2.39%

According to the assessment scenario 1, 5 and 6 shows a decrease in intersection capacity were compared to the existing case. Where in scenario 1 a reduction in ***1.52%*** has been observed so, doubling the current light rail vehicle headway there will be a small decrease in capacity because due to the fact that light rail vehicle movement will disrupt the vehicular movement specially the number of crossing movement. There is a 2.86% reduction of intersection capacity due to the introduction of signals. Here it is good to notice that the volume of the intersection increases doesn't mean the performance will reduce, the only scenario where the volume increases are when avoiding on-street parking and reducing the heavy vehicle because while reducing the heavy vehicle, since one heavy vehicle can be replaced by two or more light vehicle due to its time and space occupation. Accordingly, the capacity will increase. The same analogy works for avoiding on street parking except parked vehicle occupied space will be replaced by moving ones. This shows that the roads were used underutilized. It can be seen that the above test shows there is very little difference in performance measures occurs in the scenarios.

Statistical Testing

In order to check that if any statistical difference exists among the intersection capacity results between the scenarios and the existing conditions Dunnett's multiple comparisons between samples with a control group (existing condition) were checked with 95% confidence level n 0.05 significant.

Ho: All means are equal

H1: Not all means are equal

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	6	20941	3490	1.30	0.277
Error	49	131980	2693		
Total	55	152922			

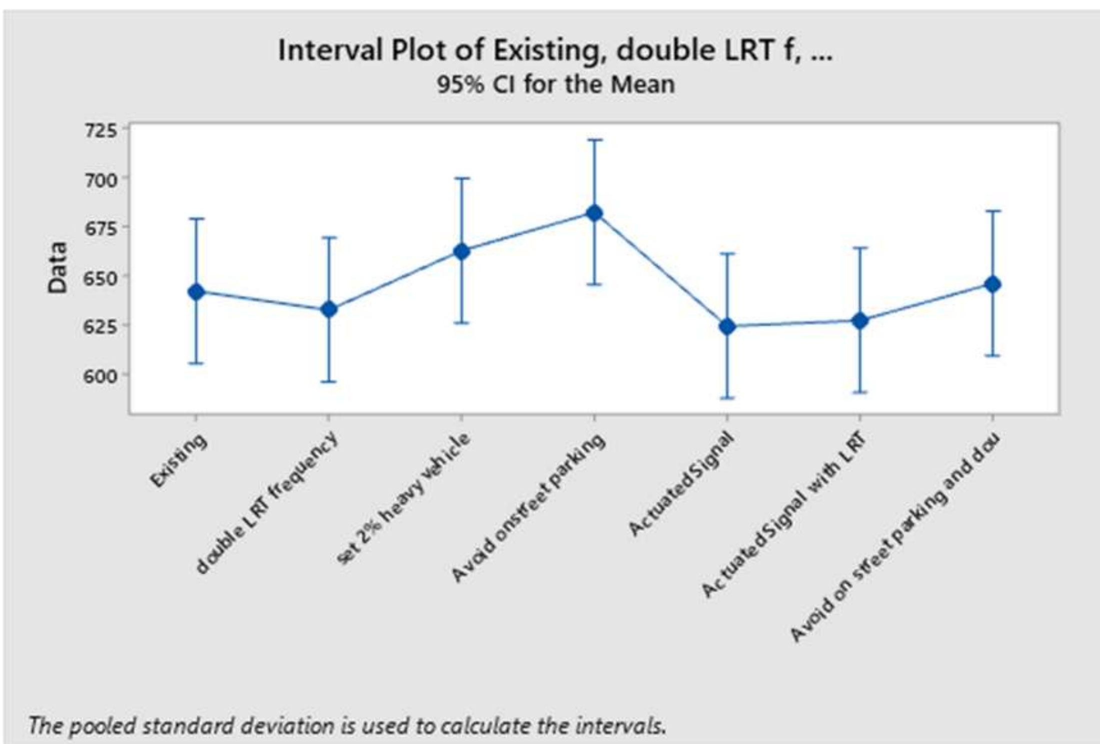

Figure 17: Dunnett's Interval Plot for scenario comparison based on capacity

From the P-value 0.277 and the above interval plot, it is clear that intersection capacity among the scenarios was found to be significantly similar, with the P-value greater than 0.05, the difference between these multiple cases failed to found any statistical difference. From the interval plot, it supports the null hypothesis all means are equal since all the scenarios intersect each other, the statistical assessment fails to reject the null hypothesis.

4.4.2 Control Delay

In this research, the control vehicular delay is one of the major measures of effectiveness in the comparison of scenarios. It was tried to observe that how the different changes made have made an effect on the control delay in the intersection. From the observed result below it has been observed that scenario 3 & 4 shows that a 9.46% & 8.63% decrease in vehicle delay when compared to the existing condition respectively. This is due to the fact-heavy vehicle and on-street parking has an effect on intersection delay.

The control delay was one of the main measures of effectiveness in this research. For each of the scenarios, the delay was recorded at the intersection the summarized VISSIM delay data is presented as follows.

Table 24: VISSIM Delay Output of simulation model

Delay (sec/veh)		
Scenarios	**Delay**	**Difference in %**
Existing	0.479	
double LRT frequency	0.489	1.94%
Avoid on-street parking and double LRT	0.559	14.22%
set 2% heavy vehicle	0.424	-12.97%
Avoid on-street parking	0.415	-15.42%
Signal passive LRT, 136 sec	0.232	-106.35%
Signal with LRT doubled	0.230	-108.46%

A comparison was made between individual scenarios against the existing condition based on the percentage of changes obtained. Particularly, it can be observed in the result that scenario 4 gives more than a 100% decrease in vehicle delay due to the introduction of the signal controller in the intersection. This excessive reduction in the overall delay is due to the presence of high conflict between vehicle to vehicle, vehicle to LRT, vehicle to pedestrian, and pedestrian to LRT was removed due to the introduction of signal light which reduces conflict points making the vehicle

delay and travel time also reduce. Conversely, doubling the LRT frequency would not have any significant effect on the intersection delay.

This result was expected because the intersection with signal controller especially with active transit signal priority has a tendency of reducing the number of stops and go action in the intersection while compared to the currently existing condition.

Statistical test

In order to see if any statistical difference exists between the intersection delay results between the scenarios and the existing conditions Dunnett's multiple comparisons between samples with a control group (existing condition) were checked with 95% confidence level n 0.05 significant.

The Control delay was compared with a control group (existing condition) with different scenarios. These comparisons were carried out with 0.05 tolerance (Test included in the appendix). The test was done for 95% confidence interval based on the hypothesis;

Ho: All means are equal

H1: Not all means are equal

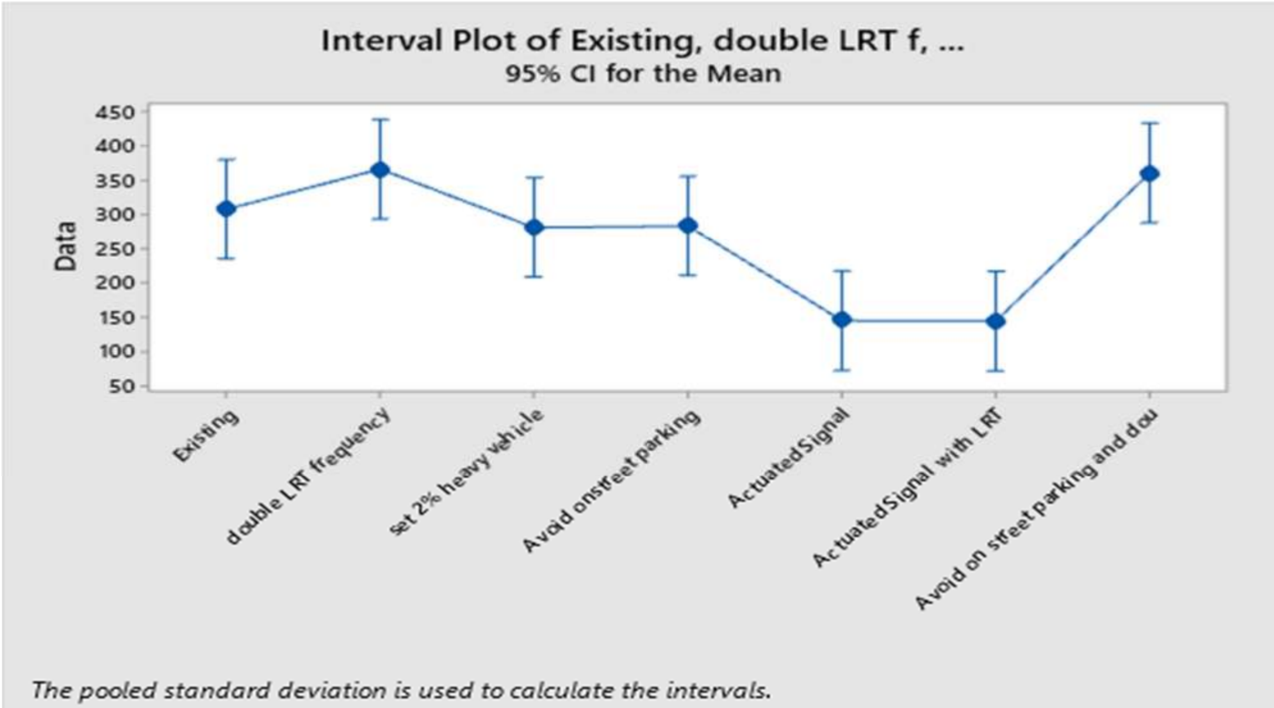

Figure 18: Dunnett's Interval Plot or scenario comparison based on delay

Furthermore, the above figure illustrates that the confidence interval delay comparison between existing conditions and scenarios. According to the statistical analysis, only two of the scenarios has produced a statistically significant difference from the other cases. It has shown significant difference values with the p-value less than 0.05, a significant different scenario is found among the multiple comparison with existing condition of control group. The difference in the means is great enough to accept the possibility that the significant different scenario is the introduction of signal controller (See **appendix-D and E**). This may have been due to the introduction of signal (ATSP) that can help reduce delay and travel time, which interns will improve the intersection performance plus will have a significantly high effect on the environmental factors by reducing pollution.

4.4.3 Queue Length

The maximum and the average queue length was collected in the node evaluation for all the scenarios and the existing condition. Vehicle queue length has a variability as follows;

Table 25: VISSIM Queue Length Output of simulation model

Queue Length(m/30sec)		
Scenarios	**Queue Length**	**Difference in %**
Existing	6.49	
double LRT frequency	7.25	10.50%
Avoid on-street parking and double LRT	6.03	-7.39%
set 2% heavy vehicle	6.04	-7.33%
Avoid on-street parking	7.07	8.32%
Signal passive LRT, 136 sec	3.35	-93.19%
Signal with LRT doubled	3.27	-97.83%

From the above table, we can observe that the vehicle queue length doubling the LRT frequency and Avoiding on-street parking has increased with 10.5% and 8.32% respectively. On the contrary, the other scenario has shown in a significant reduction in queue length among this the introduction of signal light in the intersection has resulted in reducing the queue length by 93.2% almost by half. This is due to the decrease in conflict point which is one of the advantages of signal head introduction. According to the signal design, it was decided to use 136 seconds of cycle length during the signal design stage meaning at least the queue will be dissipated every 136 seconds.

On the other hand, the controller in the simulation tends to operate differently. Specifically, during the LRT movement, the controller is not operating to minimize the overall delay, this is because two or more phases cannot be completely served due to conflicting nature with the LRT movements. Instead of serving each approach to minimize the delay. The controller serves the approaches that are not in conflict with the passage of the LRT. During this time, the volume of the vehicles being served is lower. Lower volumes and fewer phases to serve could mean that there are fewer stops and therefore less stops in the non-conflicting movements with the LRT. It seems that during the passage of LRV, the non-conflicting movement with the LRT has the priority to go along with consequently, will have considerably lower queue length. This also would be the case for emission results to be lower (**will be discussed in section 4.5**).

Statistical test

Analysis of variance specifically Dunnett's multiple comparisons with a control group (existing condition) was done to check if any statistical difference exists between the existing condition and the scenarios with 0.05 tolerance and 95% confidence level with the hypothesis of;

Ho: All means are equal

H1: Not all means are equal

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	6	118536	19756	5.10	0.000
Error	49	189881	3875		
Total	55	308417			

Table 26: Means Result with the confidence interval of the simulation model

Factor	N	Mean	St.Dev.	95% CI
Existing	8	194.6	60.0	(150.3, 238.8)
double LRT frequency	8	217.4	87.0	(173.2, 261.6)
set 2% heavy vehicle	8	181.3	67.0	(137.1, 225.5)
Avoid on street parking	8	212.2	89.0	(168.0, 256.4)
Actuated Signal	8	100.72	16.36	(56.49, 144.95)
Actuated Signal with LRT	8	98.36	21.66	(54.13, 142.58)
Avoid on street parking and double LRT	8	181.2	52.8	(137.0, 225.4)

Pooled StDev = 62.2506

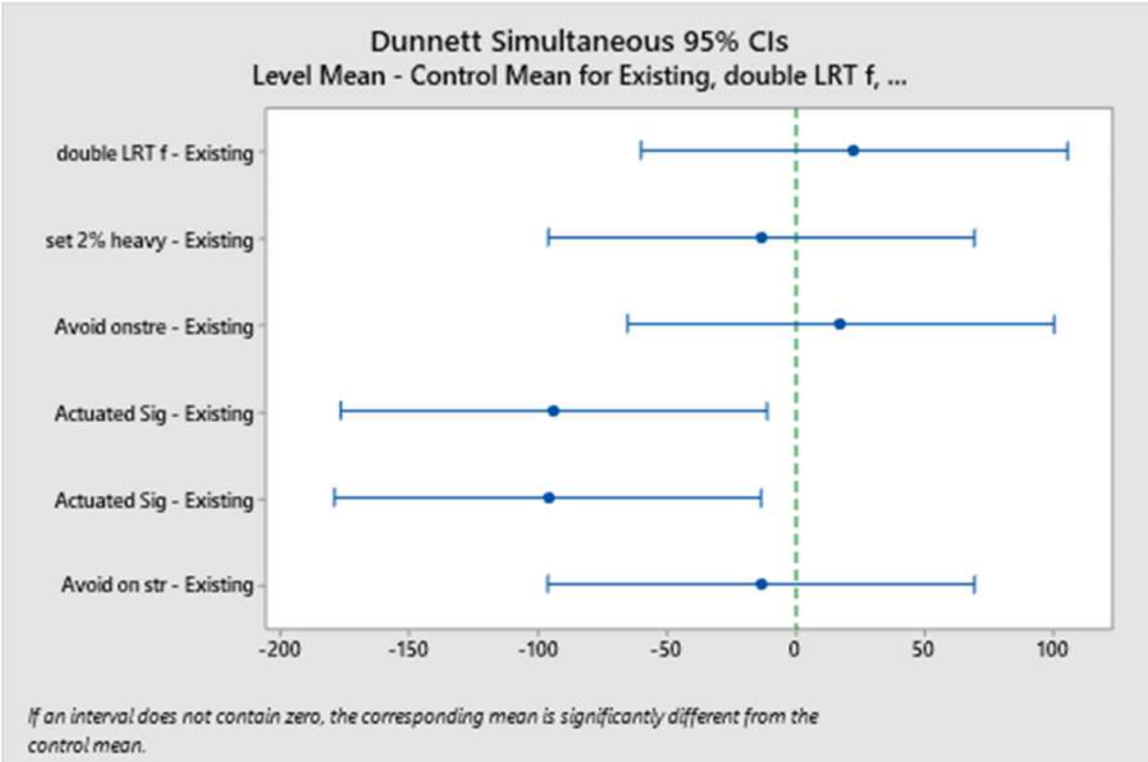

Figure 19: Dunnett Simultaneous 95% Cis based on Queue Length

According to the statistical test done using Dunnett's Pairwise comparison with the control group of the existing condition, the result suggests them exists a significant difference. Specifically, in the scenario where a signal is introduced in the intersection with the p-value being less than that of its corresponding significance level of 0.05. For detail see **appendix-D part-2**

4.4.4 Number of stops and Level of Service (LOS)

The number of stops vehicles made while traveling in the corridor is obtained as follows;

Table 27: Number of stops of the simulation model

Number of stops per vehicles							
	Existing	double LRT frequency	Avoid on-street parking and double LRT	set 2% heavy vehicle	Avoid on-street parking	Signal passive LRT, 136 sec	Signal with LRT doubled
Number of stops	0.034	0.038	0.042	0.0299	0.0298	0.007	0.0069
Difference in %		10.59%	17.98%	-14.31%	-14.79%	-389.15%	-391.40%

Table 28 presents simulation assessment result based on the number of stops vehicles made while traversing the corridor, the result shows that there is an increasing number of stops per vehicle in the scenarios doubling the frequency of LRT and avoiding on-street parking with double LRT 10.59% and 17.98%, this is due to vehicles stop to make way for LRV which in turns increase the number of stops. While it shows a decrease in the number of stops in heavy vehicle reduction and avoiding on-street parking with 14.31% and 14.79% and the introduction of the signal controller will reduce the number of stops by almost our times the existing condition.

Level of service is commonly used performance measure, in this research LOS is computed as per (HCM , 2010). As per the result obtained from VISSIM the level of service for all of the legs has been found to be “F” for the existing condition as well as the different scenarios that have been stated. LOS “F” represents the condition where the average vehicle delay is excessive. This is due to an excessive delay in the intersection and higher volume to capacity ratio. This does not mean there is no change in the intersection performance measures but rather the measure has not changed the LOS in the intersection. This indicates that the traffic condition moves with significant delay.

4.5 Scenarios with environmental related parameters

This part of the analysis involves finding the total network vehicle emission and fuel consumption using VISSIM. This includes the total vehicular emission and fuel consumption for the corridor being analyzed. Specifically, the output includes Carbon Monoxide (CO), Nitrogen oxide (NOx), Volatile Organic Compound (VOC), and fuel consumption.

Using the node evaluation feature in VISSIM will allow us to use many parameters for scenario comparisons stage in the research. Node evaluation is used specially to determine specific data from the intersection and vehicle trajectory record. The output from the software, emissions and fuel consumption for all of the vehicles modeled in VISSIM. It also determines emissions below the total emission and fuel consumption output is presented as follows. *Note that this analysis is done based on VISSIM vehicle emission output only to see the percentage difference within scenario, results do not implicate Addis Ababa pollution level. The percentage change shows the effectiveness of the scenarios.*

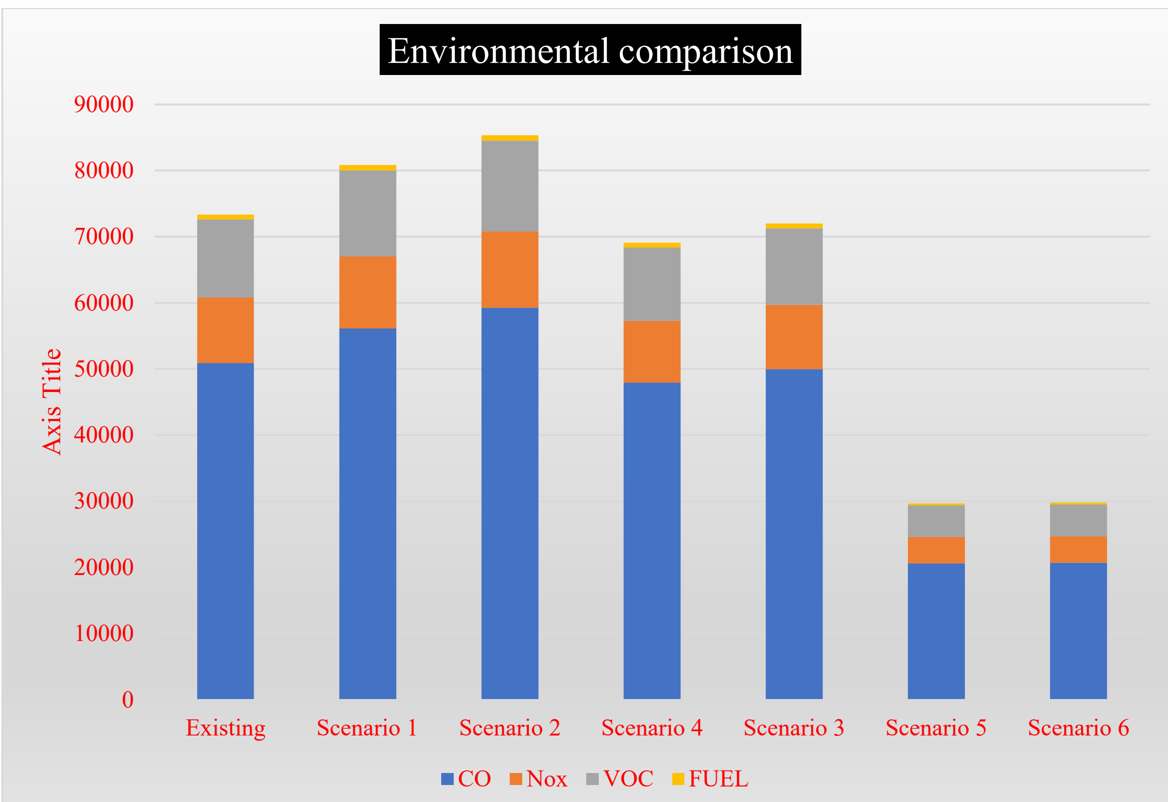

Figure 20: Environmental parameter comparison

Carbon monoxide emission is one of the greenhouse gases being emitted from the transport sector. In this research, it was determined from the simulation model that the CO, NOx and VOC emission are the critical factor to be considered in the scenario management stage of the research, based on VISSIM emission calculator integrated within the tool. The research use emission and fuel consumption their percentage difference to compare scenarios

In VISSIM, the fuel consumption of an individual vehicle can be obtained from the vehicle record, in which two types of fuel consumption for the current simulation step are defined: in mg/s and L/100km. in the node evaluation, fuel consumption means the total fuel consumed by one certain vehicle type or all vehicle types in gallons within the selected area. In the network performance equation, the fuel consumption reflects the fuel consumed by one certain vehicle types in kg for the entire network. Also, the fuel consumption can be obtained from the link evaluation, in which it is defined as the fuel consumption during the current interval (mg/m/s).

Table 29 shows the variability in emission and fuel consumption.

Table 28: VISSIM environment parameter outputs

	CO	Difference in %	NOx	Difference in %	VOC	Difference in %	Fuel	Difference in %
Existing	50901		9903		11797		728	
Scenario 1	56103	9%	10916	9%	13002	9%	803	9%
Scenario 2	59244	14%	11527	14%	13730	14%	848	14%
Scenario 4	47934	-6%	9326	-6%	11109	-6%	686	-6%
Scenario 3	49957	-2%	9720	-2%	11578	-2%	715	-2%
Scenario 5	20595	-147%	4007	-147%	4773	-147%	295	-147%
Scenario 6	20695	-146%	4026	-146%	4796	-146%	296	-146%

The result shows that an increase in emission with scenario 1 and scenario 2, a small decrease in CO emission with scenarios 3 and 4; but there is a significantly high reduction in emission in scenario 5. Specifically, the pollutant emissions generated from Scenario 1 and 2 is 9% and 14%, from scenario 3 and 4 is 2% and 5% reduction in emission and finally scenario 5 and 6 yields in a very high emission reduction of 146% and 147% for CO, NOx, VOC and fuel consumption respectively.

Generally, as power increased, emission production and fuel consumption increase as well. The purpose of this analysis is to show that congestion has a high environmental effect and on top of that scenarios and assessment can be compared based on an environmental basis.

Among the most interesting and surprising result was obtained from the introduction of the active transit signal priority (ATSP). The traffic signal controller in the simulation tends to operate in a manner such that traffic-related parameters will show a significant change from the existing condition. Due to the stop and go traffic exist during the peak hours of the day, it was observed that there is a high number of stops in the existing condition; which in turn will increase the emission produced from the corridor. As can be seen from the above figure, the introduction of the signal controller in the at grade crossing will significantly decrease the stop and go action consequently decreasing the emission in the corridor.

According to the statistics, only one of the scenarios has produced a statistically significant different result from the other cases. It has shown a significantly different result of less than 0.05. this may have been from the introduction of the signal controller which interns help reduce delay and travel time, number of stops and queue length which interns improve the intersection performance plus this will have a significant effect on environmental parameters by reducing pollution such as CO, NOx, VOC, and fuel consumption.

4.6 Summary of Result

Finding of this study were obtained using microsimulation software VISSM, simulation models relies on the quality of calibration and validation to replicate the field condition in the simulation modelling. The research aims to assess the effect of light rail transit corridor in travel congestion through microsimulation methods.

Accordingly, from the error checking process used such as GEH value being less than five and the root mean square error less than 15%, the simulation model has met its target (table 19 and ***appendix-A***). The model was accurately calibrated and validated for Wiedemann's car following parameters of the average standstill distance, additive part of standstill distance, and multiplicative part of standstill distance. Also, model fidelity was checked with seven different random number of seed to account for day to day vehicle variability (Table 20, Table 23 & Figure 16) and resulted in a significantly similar with the field condition (Table 22). According to the student t-test done between each model and field condition with 95% confidence interval it resulted p-value greater than its significant 0.05, supports the null hypothesis of all means of simulated and field values are equal (see ***appendix-B***).

After development of stable model six scenarios were compared, which were developed based on detailed field study, with parameters related to traffic and environment. According to the percentage difference comparison. It shows a 5.88% increase in capacity with scenario where on-street parking is restricted during the peak hour of the day.

The result obtained from doubling the light rail transit frequency does not have an effect on capacity since it only shows 1.5% decrease.

The delay output of the simulation model presents 12.97% and 15.42% reduction in scenarios where heavy vehicles were reduced and on-street parking restricted.

The delay and queue length have reduced by half the field condition (Table 24 and 25) in the scenario where a signal was introduced.

The number of stops was reduced by three times the existing condition. (Table 28)

According to the statistical test done using Dunnett's pairwise multiple comparison with a control group of the existing condition with 95% confidence interval on capacity, delay, queue length and number of stops, it is found that a significant different scenario, which is the introduction of the signal with the p-value less than its significance level (see ***appendix-C,D and E***).

Finally, according to the result found in this chapter the researcher recommends the scenario with the introduction of signal control. Because first it is the only scenario which is significantly different from the other scenarios. Secondly it has shown a great change in traffic performance measure, in reducing delay, queue length and number of stops. As well as showing a reduction in more than half of the existing emission (CO, NOx, and VOC) and fuel use (Table 29).

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATION

5.1 Conclusions

In this study, the researcher aimed to understand the effect of congestion on the travel corridor where LRT is running in the median of the arterial road. In addition, the congestion is evaluated through a simulation model by varying different parameters. In doing so, from the simulation model assessment and statistical comparison of scenarios, results obtained in the previous chapter. The following conclusions and recommendation are drawn;

A microscopic simulation model has the potential of revealing a specific relationship that could affect traffic performance. From the multimodal microscopic simulation model, calibration and validation results show that the simulation tool can reproduce the traffic flow very realistically with reasonable modification to the car following behavior. Therefore, it is possible to use the model to test different real-world scenarios.

According to the research presented in this thesis, the calibration and validation of the VISSIM network are successful. In fact, the model found to be significantly similar in calibration and validation stage of the model's stability test, when tested with a different random number of seeds. It was found that all multiple runs were statistically equal to the field. In addition, the percentile difference was found acceptable to conclude simulation model represents field condition.

Along with the existing base case condition different scenarios were presented and compared. The scenario involving heavy vehicle reduction, avoid on-street parking shows an increase in capacity and number of stops, while the introduction of the signal reduced the capacity and number of stops.

The comparison based on vehicle delay shows that there is an increase in delay in the scenario, double LRT frequency and double LRT with avoiding on-street parking while it shows a reduction in delay when reducing the heavy vehicle, avoiding on-street parking and introducing the signal controller.

A comparison of scenarios on the basis of queue length shows that doubling LRT frequency and avoiding on-street parking increase queue length when heavy vehicle reduction and signal introduction reduces queue length.

The comparison based on the environmental factors shows that emissions (CO, NOx, and VOC) and fuel consumption increases with doubling LRT frequency and avoid no street parking with doubled LRT, while a reduction is obtained in heavy vehicle reduction and signal controller introduction.

According to the statistical analysis made with Dunnett's multiple comparisons between samples with a control group (existing condition) with the result from each Measure of performance it is found that the introduction of the signal controller in the corridor significantly different from the existing condition, meaning the intersection performance will improve significantly if introduce active transit signal priority controller.

According to the assessment the reduction of heavy and avoiding on-street parking during peak hours of the day, the vehicle has shown a moderate amount of changes in intersection performance measures as well as in environmental improvements.

5.2 Recommendation

Depending on the final results of this study the researcher recommends the following points

- The introduction of the signal controller is essential for intersection performance improvement and emission reduction.
- Enforcement laws to be implemented on heavy vehicle operation and on-street parking restriction during peak hours of the day.
- The quality of a simulation model relies on the data collection process. So, the use of advanced equipment, as well as intelligent transportation system technologies, is recommended.

Future research area

- Even though the calibration validation process in this research was a success there is still room for refinement of the concept. Additional work could also be done to explore the usage of different driving behavior parameters to perform calibration.
- Because this study used vehicle throughput and travel time as a measure of performance for model calibration and validation, further research is recommended to include MOPs such as delay and queue length.

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Appendix A
Error Checking Process of Calibrated Model

This part comprises the calibration result obtained and error checking process done for the random number of seed 42. A similar kind of analysis is also done for the remaining six other seed numbers.

Table A-1: GEH value test for calibration model for vehicle throughput result

GEH									
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**	**E=D^2**	**F=A+B**	**G=F/2**	**H=E/F**	**GEH**	**Criteria**
523	742	0-15	-219	47961	1265	632.5	75.827668	8.707908	Pass
669	672	15-30	-3	9	1341	670.5	0.0134228	0.115857	
644	619	30-45	25	625	1263	631.5	0.989707	0.99484	
682	614	45-60	68	4624	1296	648	7.1358025	2.671292	
662	630	60-75	32	1024	1292	646	1.5851393	1.259023	
658	651	75-90	7	49	1309	654.5	0.0748663	0.273617	
670	609	90-105	61	3721	1279	639.5	5.8186083	2.412179	
630	605	105-120	25	625	1235	617.5	1.0121457	1.006055	
5138	5142	0-120	-4	16	10280	5140	0.0031128	0.055793	Pass

Table A-2: Root mean square percent error test for calibration model for vehicle throughput result

RMSPE					
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=A-B**	**E=D/B**	**F=E^2**
560	523	0-15	-37	-0.0707457	0.005005
632	669	15-30	37	0.0553064	0.0030588
679	644	30-45	-35	-0.0543478	0.0029537
744	682	45-60	-62	-0.0909091	0.0082645
680	662	60-75	-18	-0.0271903	0.0007393
568	658	75-90	90	0.1367781	0.0187083
640	670	90-105	30	0.0447761	0.0020049
643	630	105-120	-13	-0.0206349	0.0004258
				total	0.0411602
				n	8
				SQRT	7%

Table A-3: Mean absolute percent error test for calibration model for vehicle throughput result

MAPE					
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**	**E=|B-A|**	**F=E/A**
523	742	0-15	-219	219	0.418738
669	672	15-30	-3	3	0.004484
644	619	30-45	25	25	0.03882
682	614	45-60	68	68	0.099707
662	630	60-75	32	32	0.048338
658	651	75-90	7	7	0.010638
670	609	90-105	61	61	0.091045
630	605	105-120	25	25	0.039683
				MAPE	9%

Table A-4: Mean error test for calibration model for vehicle throughput result

ME			
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**
523	742	0-15	-219
669	672	15-30	-3
644	619	30-45	25
682	614	45-60	68
662	630	60-75	32
658	651	75-90	7
670	609	90-105	61
630	605	105-120	25
		ME	-0.5

Table A-5: Mean percent error test for calibration model for vehicle throughput result

MPE				
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**	**E= D/B**
523	742	0-15	219	0.2951482
669	672	15-30	3	0.0044643
644	619	30-45	-25	-0.0403877

682	614	45-60	-68	-0.1107492
662	630	60-75	-32	-0.0507937
658	651	75-90	-7	-0.0107527
670	609	90-105	-61	-0.1001642
630	605	105-120	-25	-0.0413223
			MPE	-1%

Table A-6: Theil's inequality coefficient test for calibration model for vehicle throughput result

U							
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**	**E=D^2**	**F=B^2**	**G=A^2**	**Criteria met if [0,1]**
523	742	0-15	-219	47961	550564	273529	Pass
669	672	15-30	-3	9	451584	447561	
644	619	30-45	25	625	383161	414736	
682	614	45-60	68	4624	376996	465124	
662	630	60-75	32	1024	396900	438244	
658	651	75-90	7	49	423801	432964	
670	609	90-105	61	3721	370881	448900	
630	605	105-120	25	625	366025	396900	
			Average	7329.75	414989	414744.75	
				U	0.07		

Table A-7: Theil's Bias coefficient test for calibration model for vehicle throughput result

Um					
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**	**E=D^2**	**Criteria met if <0.1**
523	742	0-15	-219	47961	Pass
669	672	15-30	-3	9	
644	619	30-45	25	625	
682	614	45-60	68	4624	
662	630	60-75	32	1024	
658	651	75-90	7	49	
670	609	90-105	61	3721	
630	605	105-120	25	625	
642.25	642.75			58638	
			Um	3.411E-05	

Table A-8: Theil's Variance coefficient test for calibration model for vehicle throughput result

Us					
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**	**E=D^2**	**Criteria met if <0.1**
523	742	0-15	-219	47961	Pass
669	672	15-30	-3	9	
644	619	30-45	25	625	
682	614	45-60	68	4624	
662	630	60-75	32	1024	
658	651	75-90	7	49	
670	609	90-105	61	3721	
630	605	105-120	25	625	
50.8183	46.123282			58638	
			Us	0.003	

Table A-9: Theil's Covariance test for calibration model for vehicle throughput result

Uc					
Simulated value -A	**Actual value-B**	**Number of observation-C**	**D=B-A**	**E=D^2**	**Criteria met if >0.9**
523	742	0-15	-219	47961	Pass
669	672	15-30	-3	9	
644	619	30-45	25	625	
682	614	45-60	68	4624	
662	630	60-75	32	1024	
658	651	75-90	7	49	
670	609	90-105	61	3721	
630	605	105-120	25	625	
50.8183	46.123282			58638	
		r	-0.78218		
		Uc	1.1		

Table A-10: Pearson Correlation coeeficient test for calibration model for vehicle throughput result

r						
Simulated value -A	**Actual value-B**	**Number of observation-C**	**Xi-X**	**Yi-Y**	**D*E**	**F/B16**
523	742	0-15	-119.25	99.25	-11835.6	-5.0495
669	672	15-30	26.75	29.25	782.4375	0.333818
644	619	30-45	1.75	-23.75	-41.5625	-0.01773
682	614	45-60	39.75	-28.75	-1142.81	-0.48757
662	630	60-75	19.75	-12.75	-251.813	-0.10743
658	651	75-90	15.75	8.25	129.9375	0.055436
670	609	90-105	27.75	-33.75	-936.563	-0.39957
630	605	105-120	-12.25	-37.75	462.4375	0.197293
						-5.47526
					r	-0.78218

mean of simulated value	642.25
mean of actual value	642.75
st.dev of simulated value	50.8183
st.dev of actual value	46.12328
	2343.907

Appendix-B

Student t-test for existing condition Vs Different Random Number of Seed

1. **Existing Morning traffic Vs RNS_34**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.374)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.159)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	**N**	**Missing**	**Mean**	**Std Dev**	**SEM**
Existing	8	0	642.750	46.123	16.307
RNS_34	8	0	666.000	68.781	24.318

Difference of means -23.250

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -0.794 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -86.048 to 39.548

Two-tailed P-value = 0.440

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.440).

Equal Variances Not Assumed (Welch's t-test):

t = -0.794 with 12.237 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -86.908 to 40.408

Two-tailed P-value = 0.442

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.442).

2. **Existing Morning traffic Vs RNS_42**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.251)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.906)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
RNS_42	8	0	642.250	50.818	17.967

Difference of means 0.500

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = 0.0206 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -51.541 to 52.541

Two-tailed P-value = 0.984

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.984).

Equal Variances Not Assumed (Welch's t-test):

t = 0.0206 with 13.870 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -51.586 to 52.586

Two-tailed P-value = 0.984

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.984).

3. **Existing Morning traffic Vs RNS_35**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.395)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.532)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
RNS_35	8	0	643.250	60.367	21.343

Difference of means -0.500

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -0.0186 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -58.108 to 57.108

Two-tailed P-value = 0.985

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.985).

Equal Variances Not Assumed (Welch's t-test):

t = -0.0186 with 13.095 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -58.484 to 57.484

Two-tailed P-value = 0.985

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.985).

4. **Existing Morning traffic Vs RNS_33**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.172)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.683)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
RNS_33	8	0	656.750	63.953	22.611

Difference of means -14.000

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -0.502 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -73.792 to 45.792

Two-tailed P-value = 0.623

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.623).

Equal Variances Not Assumed (Welch's t-test):

t = -0.502 with 12.731 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -74.355 to 46.355

Two-tailed P-value = 0.624

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.624).

5. Existing Morning traffic Vs RNS_31

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.405)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.549)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
RNS_31	8	0	654.875	70.649	24.978

Difference of means -12.125

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -0.406 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -76.104 to 51.854

Two-tailed P-value = 0.691

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.691).

Equal Variances Not Assumed (Welch's t-test):

t = -0.406 with 12.050 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -77.089 to 52.839

Two-tailed P-value = 0.692

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.692).

6. **Existing Morning traffic Vs RNS_37**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.397)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.625)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
RNS_37	8	0	642.000	36.700	12.975

Difference of means 0.750

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = 0.0360 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -43.946 to 45.446

Two-tailed P-value = 0.972

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.972).

Equal Variances Not Assumed (Welch's t-test):

t = 0.0360 with 13.327 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -44.158 to 45.658

Two-tailed P-value = 0.972

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.972).

7. **Existing Morning traffic Vs RNS_41**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.101)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.679)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
RNS_41	8	0	650.250	66.633	23.558

Difference of means -7.500

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -0.262 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -68.951 to 53.951

Two-tailed P-value = 0.797

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.797).

Equal Variances Not Assumed (Welch's t-test):

t = -0.262 with 12.456 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -69.674 to 54.674

Two-tailed P-value = 0.798

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.798).

Appendix-C

Student t-test for existing condition Vs Scenarios

1. **Existing Morning traffic Vs Signal introduction**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.208)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.413)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	**N**	**Missing**	**Mean**	**Std Dev**	**SEM**
Existing	8	0	642.750	46.123	16.307
signal	8	0	695.500	27.573	9.749

Difference of means -52.750

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -2.776 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -93.498 to -12.002

Two-tailed P-value = 0.0149

The difference in the mean values of the two groups is greater than would be expected by chance; there is a statistically significant difference between the input groups (P = 0.015).

Equal Variances Not Assumed (Welch's t-test):

t = -2.776 with 11.437 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -94.372 to -11.128

Two-tailed P-value = 0.0174

The difference in the mean values of the two groups is greater than would be expected by chance; there is a statistically significant difference between the input groups (P = 0.017).

2. Existing Morning traffic Vs No Parking

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.837)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.649)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
no Parking	8	0	682.375	55.772	19.718

Difference of means -39.625

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -1.549 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -94.506 to 15.256

Two-tailed P-value = 0.144

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.144).

Equal Variances Not Assumed (Welch's t-test):

t = -1.549 with 13.524 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -94.687 to 15.437

Two-tailed P-value = 0.145

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.145).

3. **Existing Morning traffic Vs Double LRT**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.996)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.526)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
Double LRT	8	0	632.625	61.062	21.589

Difference of means 10.125

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = 0.374 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -47.903 to 68.153

Two-tailed P-value = 0.714

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.714).

Equal Variances Not Assumed (Welch's t-test):

t = 0.374 with 13.026 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -48.313 to 68.563

Two-tailed P-value = 0.714

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.714).

4. **Existing Morning traffic Vs Reduce HV**

t-test

Normality Test (Shapiro-Wilk): Passed (P = 0.218)

Equal Variance Test (Brown-Forsythe): Passed (P = 0.789)

The result of the equal variance test indicates the likelihood that the two groups are sampled from populations with equal variances, but does not guarantee the equality or inequality of the two variances.

Group Name	N	Missing	Mean	Std Dev	SEM
Existing	8	0	642.750	46.123	16.307
Reduce HV	8	0	662.875	62.254	22.010

Difference of means -20.125

Use the results of Welch's test, where equal variances are not assumed, if the equality of the population variances of the two groups is in doubt.

Equal Variances Assumed (Student's t-test):

t = -0.735 with 14 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -78.877 to 38.627

Two-tailed P-value = 0.475

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.475).

Equal Variances Not Assumed (Welch's t-test):

t = -0.735 with 12.905 degrees of freedom.

95 percent two-tailed confidence interval for difference of means: -79.348 to 39.098

Two-tailed P-value = 0.476

The difference in the mean values of the two groups is not great enough to reject the possibility that the difference is due to random sampling variability. There is not a statistically significant difference between the input groups (P = 0.476).

Appendix-D
Dunnett Multiple Comparisons with a Control Grouping Information

1. **One-way ANOVA based on Delay**

Method

Null hypothesis	All means are equal
Alternative hypothesis	Not all means are equal
Significance level	α = 0.05

Equal variances were assumed for the analysis.

Table D-1: Factor Information

Factor	Levels	Values
Factor	7	Existing, double LRT frequency, set 2% heavy vehicle, Avoid on-street parking, Actuated Signal, Actuated Signal with LRT, Avoid on street parking and double LRT

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	6	406553	67759	6.46	0.000
Error	49	514185	10494		
Total	55	920738			

Table D-2: Model Summary

S	R-sq	R-sq(adj)	R-sq(pred)
102.438	44.16%	37.32%	27.06%

Table D-3: Means

Factor	N	Mean	StDev	95% CI
Existing	8	307.9	93.0	(235.2, 380.7)
double LRT frequency	8	366.7	152.3	(293.9, 439.5)
set 2% heavy vehicle	8	281.3	103.1	(208.5, 354.1)
Avoid on-street parking	8	283.5	100.3	(210.7, 356.2)
Actuated Signal	8	145.1	29.9	(72.3, 217.9)
Actuated Signal with LRT	8	144.3	40.1	(71.5, 217.1)
Avoid on street parking and double LRT	8	361.2	135.8	(288.4, 433.9)

Pooled StDev = 102.438

Dunnett Multiple Comparisons with a Control

TableD-4: Grouping Information Using the Dunnett Method and 95% Confidence

Factor	N	Mean	Grouping
Existing (control)	8	307.9	A
double LRT frequency	8	366.7	A
Avoid on street parking and double	8	361.2	A
Avoid on-street parking	8	283.5	A
set 2% heavy vehicle	8	281.3	A
Actuated Signal	8	145.1	
Actuated Signal with LRT	8	144.3	

Means not labeled with the letter A are significantly different from the control level mean.

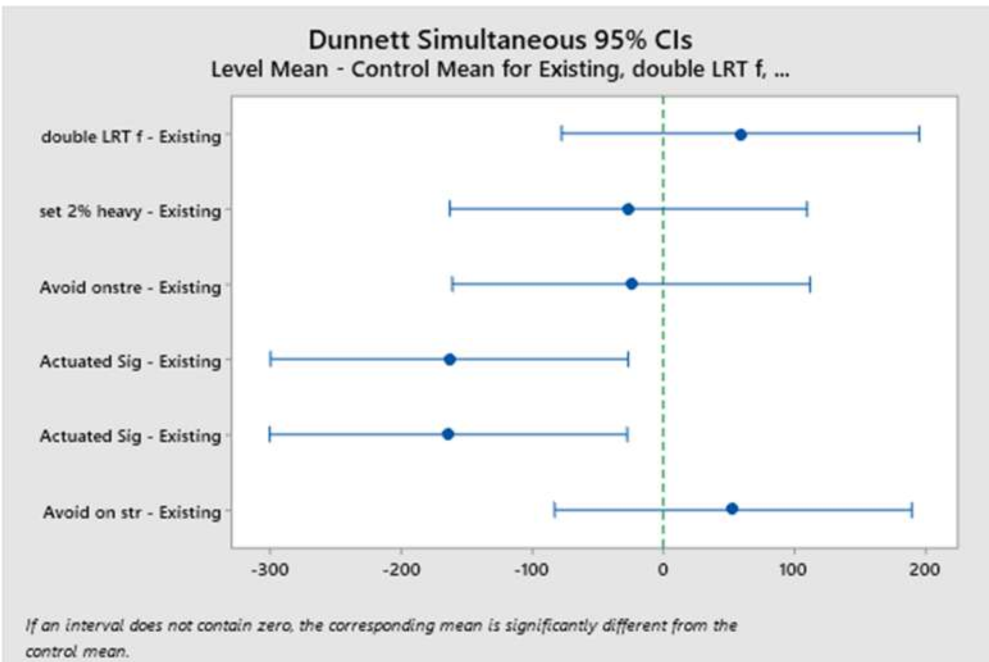

Figure D-1: Dunnett Simultaneous 95% CLs

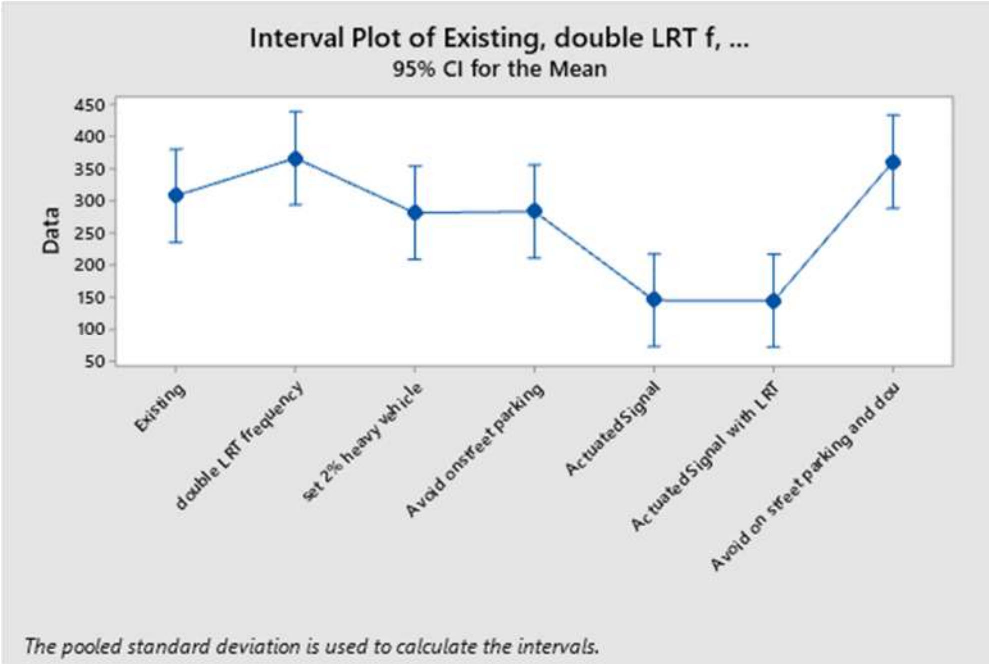

Figure D-2: Interval Plot of scenarios

2. **One-way ANOVA based on Queue length:**

Method

Null hypothesis	All means are equal
Alternative hypothesis	Not all means are equal
Significance level	α = 0.05

Equal variances were assumed for the analysis.

Table D-5: Factor Information

Factor	Levels	Values
Factor	7	Existing, double LRT frequency, set 2% heavy vehicle, Avoid on-street parking, Actuated Signal, Actuated Signal with LRT, Avoid on street parking and double LRT

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	6	118536	19756	5.10	0.000
Error	49	189881	3875		
Total	55	308417			

Table D-6: Model Summary

S	R-sq	R-sq(adj)	R-sq(pred)
62.2506	38.43%	30.89%	19.59%

TableD-7: Means

Factor	N	Mean	StDev	95% CI
Existing	8	194.6	60.0	(150.3, 238.8)
double LRT frequency	8	217.4	87.0	(173.2, 261.6)
set 2% heavy vehicle	8	181.3	67.0	(137.1, 225.5)
Avoid on-street parking	8	212.2	89.0	(168.0, 256.4)
Actuated Signal	8	100.72	16.36	(56.49, 144.95)
Actuated Signal with LRT	8	98.36	21.66	(54.13, 142.58)
Avoid on street parking and double	8	181.2	52.8	(137.0, 225.4)

Pooled StDev = 62.2506

Dunnett Multiple Comparisons with a Control

Table D-8: Grouping Information Using the Dunnett Method and 95% Confidence

Factor	N	Mean	Grouping
Existing (control)	8	194.6	A
double LRT frequency	8	217.4	A
Avoid on-street parking	8	212.2	A
set 2% heavy vehicle	8	181.3	A
Avoid on street parking and double LRT	8	181.2	A
Actuated Signal	8	100.72	
Actuated Signal with LRT	8	98.36	

Means not labeled with the letter A are significantly different from the control level mean.

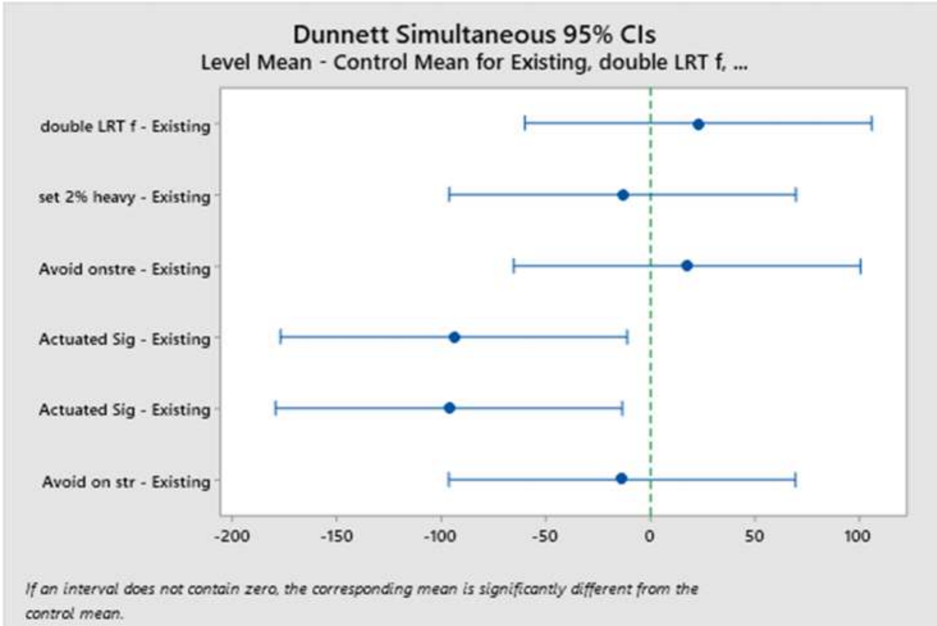

Figure D-3: Dunnett Simultaneous 95% CIs

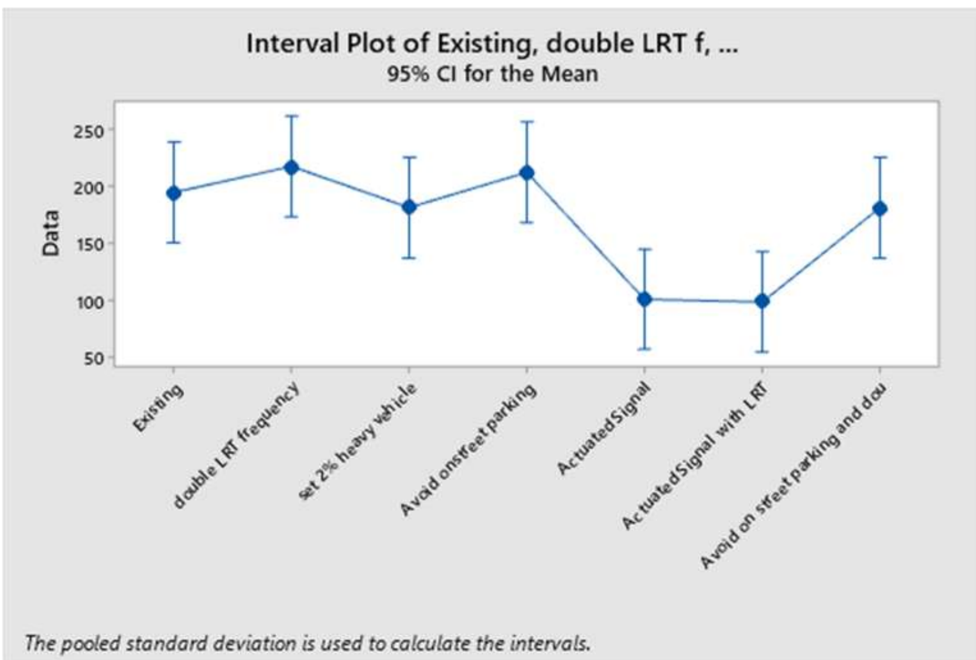

Figure D-4: Interval plot of scenarios

Appendix-E

Scenario ranking Tuckey One Way Analysis of Variance based on delay data

One Way Analysis of Variance based on delay data

Normality Test (Kolmogorov-Smirnov): Failed (P < 0.050)

Test execution ended by user request, ANOVA on Ranks begun

Kruskal-Wallis One Way Analysis of Variance on Ranks

Group	**N**	**Missing**	**Median**	**25%**	**75%**
Existing	8	0	338.145	258.572	369.498
signal	8	0	90.030	84.900	93.520
no Parking	8	0	311.255	215.468	368.810
Double LRT	8	0	386.350	262.175	460.703
Reduce HV	8	0	298.740	227.572	361.155

H = 19.974 with 5 degrees of freedom. (P = 0.001)

The differences in the median values among the treatment groups are greater than would be expected by chance; there is a statistically significant difference (P = 0.001)

To isolate the group or groups that differ from the others use a multiple comparison procedure.

Table E-1: All Pairwise Multiple Comparison Procedures (Tukey Test):

Comparison	**Diff of Ranks**	**q**	**P**	**P<0.050**
Double LRT vs signal	231.000	5.834	<0.001	Yes
Double LRT vs Reduce HV	78.000	1.970	0.732	Do Not Test
Double LRT vs no Parking	69.000	1.743	0.821	Do Not Test
Double LRT vs Existing	36.000	0.909	0.988	Do Not Test
Existing vs signal	195.000	4.924	0.007	Yes
Existing vs Reduce HV	42.000	1.061	0.975	Do Not Test
Existing vs no Parking	33.000	0.833	0.992	Do Not Test
no Parking vs signal	162.000	4.091	0.044	Yes
no Parking vs Reduce HV	9.000	0.227	1.000	Do Not Test
Reduce HV vs signal-70sec	153.000	3.864	0.069	Do Not Test

Note: The multiple comparisons on ranks do not include an adjustment for ties.

A result of "Do Not Test" occurs for a comparison when no significant difference is found between the two rank sums that enclose that comparison. Note that not testing the enclosed rank sums is a procedural rule, and a result of Do Not Test should be treated as if there is no significant difference between the rank sums, even though one may appear to exist.

Appendix-F

Sample Morning & Evening Statistical Equivalency Test

In this section, the research has given more emphasis on the equivalence of Morning and Evening traffic. Since it is going to be bulk to simulate and make the assessment for both conditions but if they have equivalency, we can proceed with just one.

Two-Sample Equivalence Test: Morning, Evening

Test mean = mean of Morning

The reference mean = mean of Evening

Equal variances were not assumed for the analysis.

Table F-1: Descriptive Statistics

Variable	N	Mean	StDev	SE Mean
Morning	32	160.69	65.097	11.508
Evening	32	145.25	67.401	11.915

Difference: Mean (Morning) – Mean (Evening)

Difference	SE	90% Upper Bound	Upper Limit
15.438	16.565	36.898	40

Upper bound is less than 40. Can claim Mean (Morning) – Mean (Evening) < 40.

Test

Null hypothesis:	Mean (Morning) – Mean (Evening) ≥ 40
Alternative hypothesis:	Mean (Morning) – Mean (Evening) < 40
α level:	0.1

DF	T-Value	P-Value
61	-1.4828	0.072

P-Value ≤ 0.1. Can claim Mean (Morning) – Mean (Evening) < 40.

The statistical equivalence test confirmed that the morning and evening traffic was significantly similar to 90%confidence intervals. So, during the analysis after validation of the model only morning traffic was used for assessment and scenario development for the model.

t-test

Normality Test (Shapiro-Wilk): Failed (P < 0.050)

Test execution ended by user request; Rank Sum Test begun

Mann-Whitney Rank Sum Test

Group	N	Missing	Median	25%	75%
Morning	32	0	172.000	97.750	210.750
Evening	32	0	136.500	82.750	190.750

Mann-Whitney U Statistic= 430.000

Yates continuity correction option applied to calculations.

T = 1122.000 n(small)= 32 n(big)= 32 (P = 0.274)

The difference in the median values between the two groups is not great enough to exclude the possibility that the difference is due to random sampling variability; there is not a statistically significant difference (P = 0.274)

One Way Analysis of Variance

Data source: Data 1 in Notebook1

Normality Test (Kolmogorov-Smirnov): Failed (P < 0.050)

Test execution ended by user request; ANOVA on Ranks begun

Kruskal-Wallis One Way Analysis of Variance on Ranks

Group	N	Missing	Median	25%	75%
Evening	32	0	136.500	82.750	190.750
Morning	32	0	172.000	97.750	210.750

H = 1.212 with 1 degree of freedom. (P = 0.271)

The differences in the median values among the treatment groups are not great enough to exclude the possibility that the difference is due to random sampling variability; there is not a statistically significant difference (P = 0.271)

Appendix-G
Sample
Simulated Vs Actual Statistical fit test

The relation between actual and simulated for morning traffic as shown in the histogram their distribution is similar, meaning the simulation has mimic the filed condition with a similar mean and standard deviation.

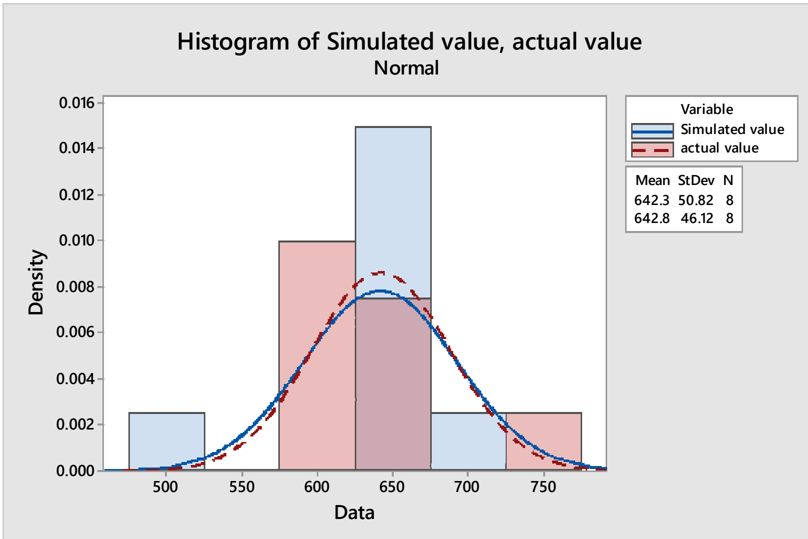

Figure G-1: Histogram of simulated and field vehicle throughput value

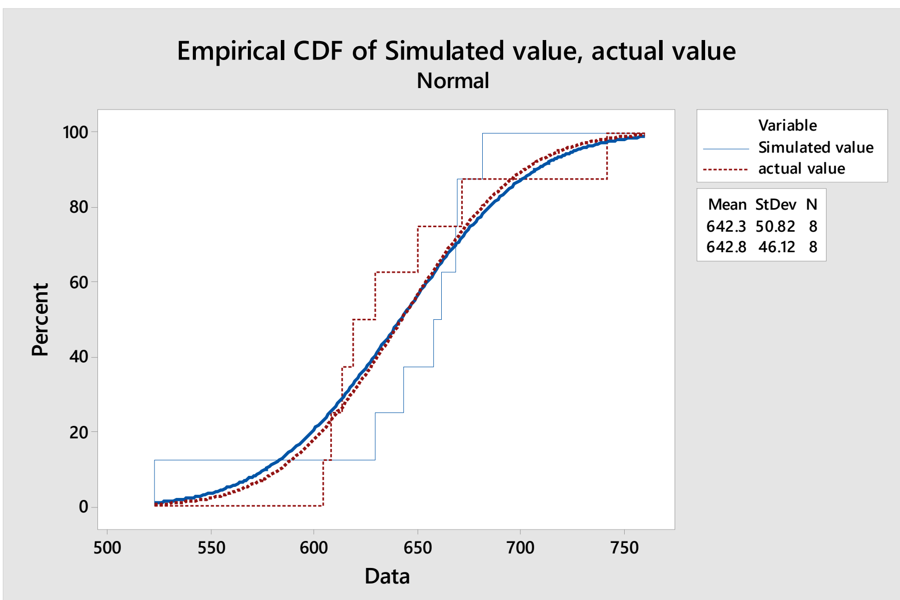

Figure G-2: Empirical CDF of simulated and fields vehicle throughput values

One-way ANOVA: Simulated value, the actual value

Null hypothesis	All means are equal
Alternative hypothesis	Not all means are equal
Significance level	α = 0.05

Equal variances were assumed for the analysis.

Factor Information

Factor	Levels	Values
Factor	2	Simulated value, actual value

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Factor	1	1.0	1.00	0.00	0.984
Error	14	32969.0	2354.93		
Total	15	32970.0			

Means

Factor	N	Mean	StDev	95% CI
Simulated value	8	642.3	50.8	(605.5, 679.0)
actual value	8	642.8	46.1	(606.0, 679.5)

Pooled StDev = 48.5276

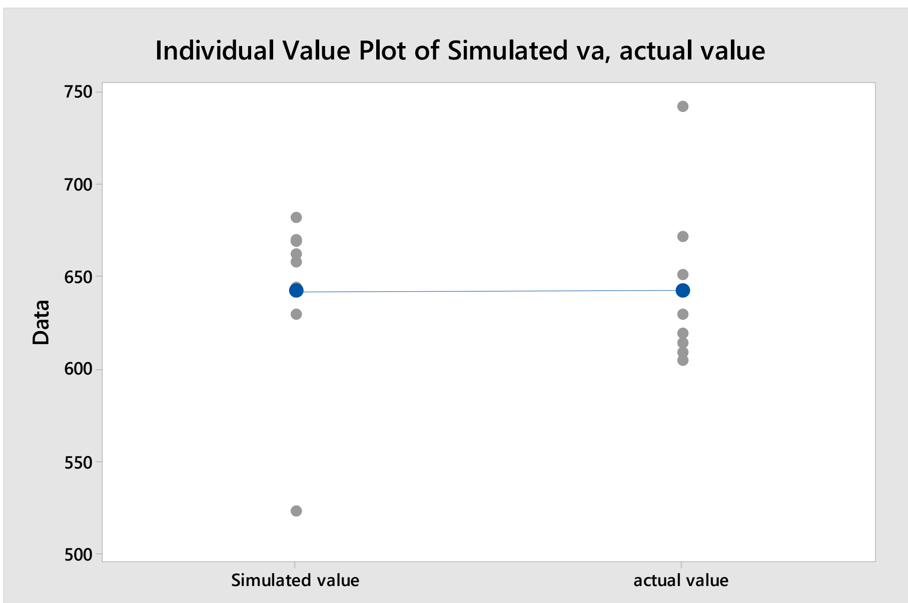

Figure G-3: Individual value plot of simulated and field vehicle throughput value

Based on the above one-way ANOVA test result the null hypothesis was accepted with a p-value of 0.984 the means of simulated and actual is equal with 42 random seed numbers.