ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIROMENTAL ENGINEERING

INVESTIGATING THE START-UP LOST TIME FOR SELECTED SIGNALIZED INTERSECTIONS OF ADDIS ABABA

By:

Dessalegn Alemu Bekele

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

Advisor: Bikila Teklu Wodajo (PhD)

November, 2019

Addis Ababa, Ethiopia

ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

INVESTIGATING THE START-UP LOST TIME FOR SELECTED SIGNALIZED INTERSECTIONS OF ADDIS ABABA

By:

Dessalegn Alemu Bekele

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

Approved by Board of Examiners:

Dr. Bikila Teklu Advisor	____________ Signature	____________ Date
Dr. Getu Segni Internal Examiner	____________ Signature	____________ Date
Mr. Yohannes Legesse External Examiner	____________ Signature	____________ Date
____________ Chair Person	____________ Signature	____________ Date

UNDERTAKING

I, the undersigned, certify that this research work titled "**Investigating the Start-up Lost Time for Selected Signalized Intersections of Addis Ababa**" is my original work performed under the supervision of my research advisor Dr. Bikila Teklu and has not been presented elsewhere for assessment and for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

Signature: _______________

Undertaker's Full Name: Dessalegn Alemu Bekele,

Place of Undertaking: Addis Ababa, Ethiopia.

Date: November, 2019

ABSTRACT

In Transportation, Signalized Intersections are an important nodes of traffic movement and this node are designed and their performance evaluated based on the adopted standards. In this respect, most of the parameters that are used for design and capacity evaluation of Signalized Intersection are taken from Highway Capacity Manual (HCM) which was developed by Transportation Research Board in Washington, D.C. Accordingly, the default values will not represent the actual condition of the traffic for a specific study area. To this effect, calibration of the values should need to be the major task for researchers and traffic agencies. Lost time is one of the key parameters used to estimate an optimum cycle time. It is a very sensitive parameter and small variation of lost time may cause large changes of cycle time.

Different studies show a great deal of variations in start up lost time and saturation flow. This indicates a lack of stability and this is acknowledged in the HCM. Due to these instabilities, HCM recommends that local data collection be performed to produce more accurate estimates of local start-up lost time and saturation flow.

In this Study, experimental research is carried out for estimating the start-up lost time and the contributing factors affecting its value. For this purpose, observations were carried out on four selected signalized intersections in Addis Ababa City. Data was collected by a video recording of traffic flow at four selected signalized intersections of Addis Ababa city. Recorded data were analyzed for start-up lost time by extracting the information of time headway of the vehicles. In this research a more accurate way of estimating Start Up Lost Time is proposed by developing applicable and replicable models.

The analysis showed that the overall estimated mean of start-up lost time is 2.513 Sec for all observed sites with a different mix of traffic and Geometric condition as well as from the analyzed intersections the ideal intersections were selected as per HCM recommendation and it shows that the mean start up lost time for an ideal intersections is 2.383 Sec and t test showed that the mean start up lost time is significantly different with the HCM recommendation of 2sec per phase. Start Up Lost Time data for eleven vehicular compositions were fitted to a probability distribution. Lognormal, Weibull and Johnson Transformed were found to be best fitted probability distribution for the Start Up Lost Time. Regression Analysis was conducted by Multiple Linear Regression technique to predict Start Up Lost Time. From regression analysis Mix of Traffic (Vehicular Composition), Approach Grade, Lane Width, Approach Width, Green Time, Type of Movement, Lane Position for Maneuvering and Saturation Headway were the factors that affect start up lost time and found statistically significant to predict Start Up Lost Time.

Key Words: *Start Up Lost Time, Saturation Headway, Signal Timing, Model and Signalized Intersection*

ACKNOWLEDGEMENTS

First of all, I would like to thank Almighty God for his endless mercy, giving me excellence and efficiency in time of difficulty and a moment of weakness. Second in line, it is my great pleasure to forward my deepest gratitude to my research advisor Dr. Bikila Teklu for his genuine guidance, valuable comments and help me achieve my goal in this research. Thirdly I would like to thank my Co- Advisor W/rt. Helen Zewdie for her valuable support, comment and guidance in the course of this research and become a good mentor.

I have a heart full appreciation to Abel Wube, Tesfaye Abdisa, Neway Woldeyes and Ayenew Mekonen for helping me during the data extraction and collection of the data and also it is my pleasure to express my gratitude to my family for their unreserved support and courage.

The last but not the least, I would like to thank the Ethiopian Roads Authority (ERA) for funding my graduate study of Road and Transport Engineering stream at the Addis Ababa University.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER ONE: INTRODUCTION

1.1. General Background

Signalized Intersections play a vital role in the management of traffic movement which are widely used in different roads in many areas around the world for making efficient and effective maneuvering of traffic flow on the Junctions **(Mathew, 2017)**. In general, these traffic light signals are seen as feasible and efficient tools to guarantee the efficiency and safety of intersections, especially on city roads and arterial roads. Maintaining reasonable signal timing is necessary, but difficult. After the implementation of this signal control, several studies are conducted to improve their efficiency as well as to improve the safety problems **(Zhao, Zhang, Lu, Zhang, & Ma, 2015)**.

In urban highway transportation systems, signalization is mostly utilized in the management of intersections. A well designed signalization system can provide many advantages such as both the ensuring of pedestrian and vehicle safety and the enhancement of intersection capacity **(Çalişkanelli & Tanyel, 2016)**. Researches have shown that the proper installation and operation of traffic signals can reduce the severity of crashes and congestions. However, unnecessary or inappropriately designed signals can adversely affect traffic safety and mobility **(FHWA, 2013)**.

In the design of signals, it is needed to consider the behavior of drivers, road way condition and the environment of the actual area **(Koonce, 2008)**, whereas due to the lack of all the required data from the actual condition and due to inefficiency of transportation agencies to customize the required inputs for the design of this signals they have considered the default values which are estimated in the other areas. Owing to this fact, the performance of signalized intersections is under question.

As cognizant that, the efficiency of signalized intersection is measured based on the amount of delay from the free flow condition and also measured based on the reduction of crashes **(HCM, 2010).** In relation to this, recently in Ethiopia especially in Addis Ababa city traffic congestion is the main problem for the city and the delay due to the traffic problem highly affected the day to day life of the people and the same it highly affects the economy of the country **(Temesgen, 2015)**.

As merely understood, this signalized intersection improves the occurrence of accidents as well as congestion problems, whereas recently a long queues are observed in the Addis Ababa city signalized intersections. The cause of this congestion is due to poor performance of the signals due to inappropriate optimum cycle lengths which makes the question on the design of signal timings. Signal timing is an allocated time for one intersection to efficiently maneuver the demand of traffic flow. Due to the variability in the traffic volume it is difficult to accurately determine the required time for that signalized intersection by considering this different signal algorithm equation recommend to use the critical lane volume.

Beyond to the variability of traffic flow at signalized intersections, the parameters which are adopted from different recommendations highly affected the design of signalized intersections. Signal algorithm considered drivers behavior as an input parameter which lost time as key in the computation of the optimum Cycle Length **(Md Hadiuzzaman, 2008)**. Lost time is defined as the unused time in the effective green period due to the different factors of the drivers.

In signal operation, lost time occurred during the start of green and clearance at the end of green time and named as Start Up lost time and Clearance lost time respectively. Most of the studies revealed that fewer drivers during the intention of green time will lose few seconds due to their perception reaction on the signal light and the reaction of preceding vehicles **(HCM, 2010)**. As mentioned above, Start-up lost time is a part of lost time which basically used in the design of signal timings as well as performance studies of the Signalized intersection.

In our country, this parameter considered in the signal designs by adopting or customizing it from HCM recommendation which is 2.0 Sec while different researches are estimated this value and revealed that it is variable from city to city of one country **(Mohamed Shawky, 2016)**. Further, the studies are conducted on the factors that affect its value and revealed that there are different arguments on the Geometric and traffic parameters effect on the subject matter.

Hence, based on the above motivations, the researcher conducts the study on the investigation of start-up lost time and the factors that affect its values by conducting direct observation of the sites with accurate measurement techniques.

1.2. Statement of Problem

Even though least number of vehicles operate on the road networks and intersections of Addis Ababa, compared with developed nations, traffic Congestions at intersections is becoming increasing and extensive delays at intersections of Addis Ababa is observed and this affects the day to day life of people **(Kidus, 2016)**. As it is known, these delays occurred due to the intersections perform over their capacity and this over capacity flow is not managed with appropriate signal designs and other measures it will create delays in the overall traffic system. To manage traffic movement of the signalized intersections, appropriate cycle length design is mandatory and consideration of human factors is also necessary.

On top of this, signal design of intersections considered lost time as a key parameter in the determination of the desired cycle length. The considered value for lost time in our country is adopted from HCM manuals while the driving behavior and maneuvering characteristics of the actual condition is different from the adopted values. Thus, accurate estimation of lost time is crucial for determining the optimal cycle length because a small variation of lost time may cause a large change of cycle length. To this effect, quantifying and Standardizing start up lost time for Addis Ababa intersections is mandatory in order to enable design signals according to the Drivers behavior and to determine the parameters that will affect the start- up lost time.

1.3. Hypothesis and Research Questions

1.3.1. Hypothesis

By Statistical test we can arrive at a viewpoint enable to determine the standard value of Start-up lost time for the respective the study area condition and also enable to determine the parameters that affect Start up lost time. The null hypothesis for this study is that the new start-up lost time equal to 2sec as HCM 2010 default value and on the other hand the alternative hypothesis will be the new start up lost time different from 2sec.

Ho: Start-up lost time, $\mu = 2$ sec

H1: Start-up lost time, $\mu \neq 2$ sec

Further, Start-up delay takes place at the first four vehicles as per HCM recommendation, for this the null hypothesis ($n_0 = 4$) while the alternative hypothesis is ($n_1 \neq 4$).

1.3.2. Research Question

- ✓ How much is the Start -up lost time on the selected intersections of Addis Ababa?
- ✓ What are the factors that affect start -up lost time on selected signalized intersection of Addis Ababa?
- ✓ Which is an appropriate probability distribution for Start-up lost time on the selected intersections of Addis Ababa?
- ✓ What does the regression model entail on the effect of each factors on start-up lost time for different combination of vehicles?
- ✓ Does the Start up lost time vary among different combination of vehicles?

1.4. Objective of the Study

1.4.1. General Objectives

The general objective of this study is to customize the default HCM recommended value for the Start-up lost time that used for the Design and Capacity evaluation of Signalized Intersection which contains several flow conditions and to find factors that will affect Start-up lost time significantly.

1.4.2. Specific Objectives

The Specific Objectives of this research are: -

- ✓ To determine an appropriate start-up lost time based on the actual condition of Addis Ababa
- ✓ To determine factors which affect Start up Lost time at Signalized Intersections of Addis Ababa City
- ✓ To determine the fitted probability distribution of Start-up lost time
- ✓ To develop mathematical model for Start-up lost time using multiple linear regression
- ✓ To determine the variability of start-up lost time by different vehicle combinations

1.5. Scope and Limitations of the Study

1.5.1. Scope of the Study and Limitations

The scope of this research covers determination of Start-up lost time for the local condition and modeling of Start-up lost time (regression and distribution model) for selected Signalized Intersections of Addis Ababa city. In this study, the effect of weather, incident, the model of vehicle and age deviation of drivers is not considered.

1.5.2. Further Researches that can be made

The researcher can encourage others to work on the stated limitation with a wider study to arrive at efficient models.

1.5.3. Significance of the study

The study will play a vital role in the design, performance analysis, capacity evaluation and congestion study for signalized intersections of Addis Ababa and improve the design errors in using of default values which are customized from HCM Manual. As the fact known that, small deviation of lost time causes large variation in the cycle length due to it is the sensitive parameter as shown in the equation below. Hence, quantifying and standardizing start up lost time is crucial for the determination of optimal cycle length as far as for evaluation of signalized intersections.

i.e. Desirable Cycle Length, Cd= $\frac{L}{1-[\frac{Vc}{1615*PHF*\frac{v}{c}}]}$

where, L = total lost time s/cycle, sensitive parameter in the design of Cycle Length

PHF = Peak Hour Factor

Vc = Critical lane Volume

v/c = target v/c ratio for critical movements

1.6. Organization of the Study

This research is basically divided into Five main chapters. Chapter one intends to introduce the underlying background science to the topic and the intended purposes in doing the research. The second chapter extends the effort to look on background science in detail by reviewing vast previous studies in the area of the topic. Chapter three describes research methods and materials including description of the study area, data analysis methodology, sample size determination, study design, data collection methodology, and data extraction methodology. Chapter four deals with analysis and discussion, detailed data analysis and discussion of results is presented in this chapter. The last chapter of the study is conclusion and recommendation main findings of the study along with recommendation for future studies are presented in this chapter.

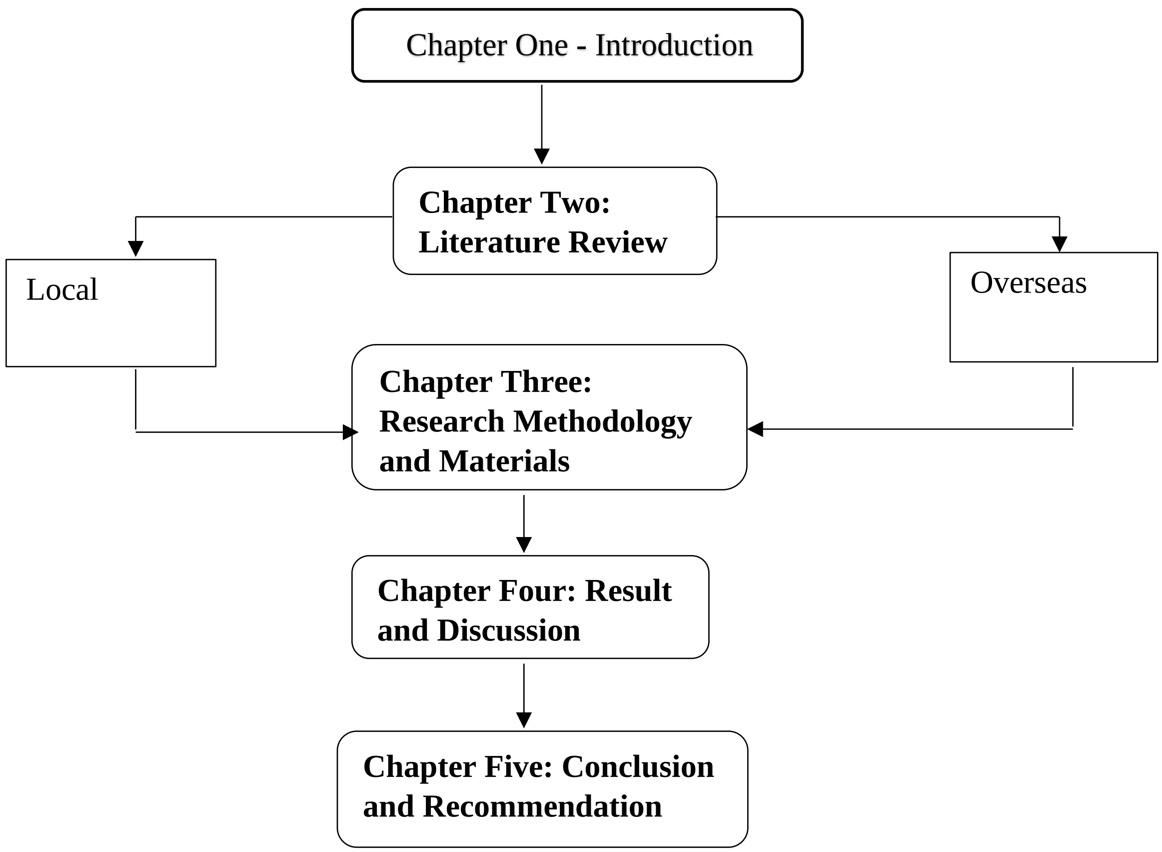


Figure 1.1: Thesis Organization and Flow Chart

CHAPTER TWO: LITREATURE REVIEW

2.1. Introduction

This Chapter provides an overview of the current Transportation Engineering Literature on the subject topic and discusses the Concepts of Signalized Intersection, Start-up lost time and the factors that will affect the start-up lost time. It intends to look on several literatures written in the areas of this research and Section 2.2 will be discussing about Signalized Intersections and their development. Section 2.3 Traffic flow Modeling for Homogenous and Heterogeneous traffic are discussed. In section 2.4 the characteristics of traffic flow at Signalized Intersection and Vehicle Discharge Characteristics are discussed. Section 2.5 Signal Timing and the parameters that considered in the signal timing are presented. In section 2.6 the general definition of Start-up lost time, importance of startup lost time, observation on the estimated value of start-up lost time on different countries, different methods start up lost time modeling along with goodness of fit test of the models, and factors affecting start up lost time are discussed. In last section of this chapter, section 2.7, Summary of literature review is discussed.

2.2. Signalized Intersections

The signalized intersection is a facility to safely allocate right of ways for conflicting traffic movements. These intersections ensure safety of vehicular and pedestrian traffic; however, they are also considered as bottlenecks in the urban street network **(Q. Yang, 2012).** The efficient system of urban streets is highly dependent on how signalized intersections are operating and to operate efficiently it will require to design accurately by customizing all the values in the local condition.

The more centralized, urbanized and motorized society depends on the knowledge obtained from traffic signal controls at urban intersections to inform contemporary traffic management theories. Correspondingly, the stop line is setup as a traffic road marking to indicate the need for vehicles to stop in the red signal phase **(Shuai Yang, 2013)**.

2.2.1. Signalized Intersection Development

On 10th December 2018, the railway engineer J. P. Knight erected the first manual switch traffic light in London (Philpot, 2005). People used this red and green device to open up an entirely new way of perceiving and understanding traffic management in urban areas. After the traffic light was invented, adding a yellow light to the red and green traffic signal was firstly used in Great Britain in 1918 (Sobey, 2006). As early traffic lights only included red and green lights, the red, green and yellow light was considered to be the first definitive version of modern traffic lights. The traffic control signal system had reached a significant milestone. Today, the three-colored traffic lights system is still the most favored means of traffic control at urban junctions. In urban highway transportation systems, signalization is mostly utilized in the management of intersections.

Interestingly, after the introduction of traffic signals, traffic management research increased. Traffic management covers aspects, such as research of traffic signal systems and traffic stream characteristics. In the 1930s, Greenshield conducted pioneering research into traffic dynamic performance at urban junctions at the Yale Bureau of Highway Traffic. The Greenshield's traffic flow study was the first to apply probability theory to the description of urban road traffic dynamic performance (Lieu, 1999). In recent years, with the help of computers, intelligent transportation light systems (ITS) are drawing more attention in the traffic management research field. In addition to progress made in traffic light control technology, research on driver behaviors has also opened a new frontier for traffic management in recent years.

2.3. Traffic Flow Modeling

A lot of researches are conducted in Traffic Engineering in order to avoid congestion problems and severe accident in the transportation system and those studies are conducted by modeling the actual traffic condition. Traffic Modeling used in planning, design and operation of transportation facilities. (Scott, 2011) identified that Traffic modeling will be classified and studied in to two ways either in Macroscopic and Microscopic ways.

2.3.1. Microscopic Traffic Model

With the increasing complexity of traffic network and traffic management systems, microscopic traffic simulation has become one of the major tools to evaluate and optimize various traffic management and control systems.

Microscopic traffic models usually contain various parameters describing traffic flow characteristics and driver behaviors, and those parameters need to be properly calibrated to replicate realistic traffic **(Lu, 2016)**.

Here is the discussion on two major microscopic parameters that are commonly used in traffic operations: space headway and time headway.

2.3.1.1. Space Headway

The physical gap between two vehicles is defined as spacing. Space headway is determined by measuring the distance, either from front to front or bumper to bumper, between the two vehicles as they pass a common observation point and expressed in feet or meters **(Bhuiyan, 2018)**.

2.3.1.2. Time Headway

The elapsed time between the arrival of pairs of vehicles at a common observation point is defined as the time headway and is expressed in seconds **(Bhuiyan, 2018)**.

2.3.2. Macroscopic Traffic Model

The macroscopic traffic model establishes the relationship between flow, speed and density at a larger scale, rather than looking at individual vehicles **(Bhuiyan, 2018)**. Macroscopic Traffic flow modeling deals with the general or average aspect of traffic flow. The first traffic flow model to be developed by researchers was a macroscopic model in the 1950's and the model was based on the average behavior rather than individual vehicles. The advantages of macroscopic traffic flow modeling are simplicity of its calculations and have to deal with lesser parameters than microscopic models. Although macroscopic modeling has the above advantages, it cannot incorporate specific details in the traffic stream that can be modeled with microscopic model (Scott, 2011)**.**

Macroscopic Traffic Parameters are;

- ✓ Flow
- ✓ Speed
- ✓ Density

2.3.3. Homogenous and Heterogeneous Traffic Modeling

The composition of traffic in developing countries is mixed, with a variety of vehicles, motorized and non-motorized, using the same right of way. The motorized or fast moving vehicles include passenger cars, buses, trucks, motorcycles and Heavy Vehicles.

Since 1950s, considerable research has been done to develop traffic flow models for roadways with mainly homogeneous traffic, representing the composition of traffic primary in developed countries (Khan, 1999).

Heterogeneous or mixed traffic systems operate very differently, compared to homogeneous traffic systems. The traffic in mixed flow is comprised of fast moving and slow moving vehicles or motorized and non-motorized vehicles. The vehicles also vary in size, maneuverability, control, and static and dynamic characteristics. Traffic is not segregated by vehicle type and therefore, all vehicles travel in the same right of way. Smaller size vehicles often squeeze through any available gap between large size vehicles and move in a haphazard manner **(Dey, Nandal, & Kalyan, 2013)**. While Under homogeneous traffic flow condition there is limited interaction between vehicles, also less intrusion from other road users which makes it less difficult to determine the traffic parameter. In most developing countries the prevailing traffic flow condition indicates the existence of a heterogeneous or mixed traffic flow where different motorized and non-motorized vehicles use non isolated infrastructure **(Liu, Chen, & Gao, 2011)**. It has been noted that heterogeneous traffic flow characteristics is affected by geometric features of the roadway, and proportion and properties of type of vehicle in the traffic stream. Driver behavior in heterogeneous traffic flow condition is also significantly different from homogeneous traffic flow condition.

Small cars following a truck trailer makes an unusual and unique decision to overpass than in homogeneous traffic (Khan, 1999). Very limited studies have been done to develop an understanding of traffic for heterogeneous or mixed traffic condition. This research tries to consider a heterogeneous mix of traffic to adequately represent the actual condition of traffic.

2.4. Characteristics of Traffic Flow at Signalized Intersection

Traffic Flow at Signalized Intersection usually described in terms of arrivals and departures. These functions are further translated in terms of a sequence of arrivals, stop and departures. The control at signalized intersections regulates the arrivals, stop and departures to provide a safe and smooth environment for vehicular occur on signalized intersections and pedestrian traffic. The ultimate objective is to determine the signal settings that minimize the expected delay to the Vehicle (SHAHZAD, 2013).

Signalized intersection is a shared space where two or more conflicting traffic streams are separated in terms of time with alternatively allowed movements to use the facility. During this process the control at signalized intersections interrupts traffic flow in an orderly and deterministic manner. At signalized intersections, the behaviors of driver and traffic are concerned mostly at transitions of signal phases, e.g. the onset of yellow or green. Decision makings and traffic flow variations are complex at the transitions of signal phases. The concern of safety and efficiency issues also focused on these periods of time in a signal cycle. For this reason, Traffic Engineers implement some devices like red-light camera and countdown timers to improve the operational performance and safety at signalized intersections **(Q. Yang, 2012)**.

Vehicles are stopped during the red interval and the interrupted stream starts building up as a queue. One of the first objectives is to clear that queue when the interrupted stream is given a green signal. The accumulated queue is then allowed to depart by providing green time. To establish a stable traffic flow, the green time must be long enough to clear the queue accumulated during the red interval. Once the queue is cleared, the departure flow in the remaining green time equals the arrival flow (SHAHZAD, 2013).

The efficiency of a signalized intersection is evaluated by how smoothly these queues accumulated during red interval are dissipated during green period. Any inconsistency in dissipation results in longer delays and more stops (**Allsop, 1971)**.

2.4.1. Queue Discharge Characteristics at Signalized Intersection

During Departure process, the first waiting driver will usually take some time to react to the red-to-green change before releasing the brake and starting acceleration. Subsequent drivers will usually also incur some reaction time, which will be shorter with every subsequent driver. In the process of departing, when the drivers that efficiently utilize the time without losing their green time will improve the efficiency of signalized intersection. A number of studies have indicated that drivers maintain their following distance based on the time headway that is independent of vehicle speed (Winsum, 1996)**,** (Van Winsum, 1999) and (Chang, 2001). In the departure process, the capacity of the traffic stream is a basic parameter. The capacity parameter is determined based on the saturation discharge headway **(Akçelik, 2008)**.

The main traffic parameters at a signalized intersection include discharge headway, saturation flow, and capacity. Among these, discharge headway is an important one since it is used to determine other parameters such as saturation headway and start-up lost time at intersections. These two parameters have in turn been used in determining optimal signal timings. Inaccuracies in Start-Up lost time and Saturation headway values would lead to non-optimal signal operations **(Radhakrishnan & Ramadurai, 2015**). Several studies have been carried out on this values and the factors that affecting discharge headway of vehicles.

Discharge headway at signalized intersection can be defined as the time interval between two successive vehicles on a lane crossing the stop line at an intersection during the green time. (Greenshields B. S., 1947) was one of the first studies on discharge headway in which reported average headways for the first five vehicles of the queue.

(Carstens, 1971) reported the average starting delay for vehicles in queue as 0.75 Sec and average headway spacing for straight moving cars as 2.29 Sec per vehicle. (Moussavi, 1990) conducted studies on departure headways at signalized intersections in Nebraska and concluded that departure headways show high variability for different intersections possibly because of the different traffic and geometric conditions prevailing there. They also came up with a set of values for departure headway of first seven queue positions. (Bonneson, 1992) developed a model for discharge headway at signalized intersections based on driver reaction time, driver acceleration, and vehicle speed. His model showed that a minimum discharge (saturation) headway is reached only after eighth or ninth queue position. (Al-Ghamdi, 1999) conducted a study on the discharge headway at intersections in Riyadh, Saudi Arabia and observed that it is not reliable to use discharge headway values from other countries in Saudi Arabia due to changes in factors such as driver behavior and intersection geometry, and came up with average headway values for different queue positions.

Several other studies have come up with distributions for discharge headways. (Jin, 2009) studied the departure headways at signalized intersections and found that distributions of departure headways at each position in queue follow a log-normal distribution except the first one. A car-following model was also proposed to explain this behavior which can be used for intersection capacity analysis and traffic control. **(Liu, 2011)** introduced a hazard based model to analyze the first discharge headway of queuing vehicles. The model has been developed on the basis of data collected from Beijing and it was found that the first discharge headway is dependent on the vehicle type and complexity of intersection and any other disturbance to the vehicle movement will further increase the discharge headway**.** (Wu, 2010) studied departure headway distributions and the study revealed that for modeling headway data, log-laplace distribution model is suitable at free flow conditions and log-logistic model during peak hours. (Yin, 2009) also arrived at similar conclusion about fitting headway distributions to headway data for free-flow state and congested state. Their findings include that headway data follows log-normal distribution when traffic is in free-flow state and log-logistic distribution when traffic is congested.

2.4.2. Discharge Headway on Heterogeneous Traffic

Heterogeneous or mixed traffic systems operate very differently, compared to homogeneous traffic systems. The traffic in mixed flow is comprised of fast moving and slow moving vehicles or motorized and non-motorized vehicles. The vehicles also vary in size, maneuverability, control, and static and dynamic characteristics. Traffic is not segregated by vehicle type and therefore, all vehicles travel in the same right of way. Smaller size vehicles often squeeze through any available gap between large size vehicles and move in a haphazard manner (**Dey et al., 2013**).Very limited studies have been done to develop an understanding of traffic for heterogeneous or mixed traffic condition.

(Maini, 2000) conducted a study on discharge characteristics of heterogeneous traffic at signalized intersections in two Indian cities - Baroda and New Delhi and determined clearing speed of vehicles from intersection and concluded that clearing speed does not vary significantly with vehicle type and vehicles move as a single platoon at intersection and those with higher performance are affected by those with lower performance. (Arasan V. K., 2005) suggested a method for modeling heterogeneous traffic flow by simulation with vehicles of wide ranging characteristics. The model was validated and found to replicate traffic without lane discipline and could be used for further studies on heterogeneous traffic. (Arasan V. V., 2006) applied a simulation model to estimate saturation flow to study the effect of road width on saturation headway under heterogeneous traffic conditions. It was found that there is a significant increase in saturation headway with an increase in road width. (Radhakrishnan, 2011) proposed a methodology to develop saturation headway model based on dynamic PCUs. PCU values are determined by minimizing the difference between ideal and observed flows using their coefficient as the objective function and then saturation flow model is developed by regressing saturation flow in vehicles against the percentage of each class of vehicle.

The impact of traffic and geometric parameters on the Discharge headway value for the vehicles after the fourth vehicle in queue position are investigated by (Habtamu, 2018). The results found that the departure headway variability due to vehicle size and left turn movement will result large headway than through movements. In addition, it is indicated that, the lane width has negative relation with departure headway whereas the Approach grade has positive relation.

The impact of the geometric parameters was also investigated by (Bonneson, 1992). The results found that the left-turn radii affect the headway of the queued vehicles. The larger radii of the left-turn paths resulted lower headways. In addition, it is indicated that queue length per cycle and lane volume has a negative effect on the headway of the first twelve vehicles.

(Radhakrishnan and Ramadurai, 2015) Heterogeneous traffic Discharge headway values were having variation and were different from homogeneous traffic scenario where the headway tends to follow a constant value after initial four or five vehicles. Vehicle type, lateral position of vehicle in road section, and elapsed green time were identified as the factors affecting discharge headway. Models for computing discharge headway were developed using linear regression and linear mixed regression. Linear model had bias which was eliminated using linear mixed model. From the review of existing literature, it is evident that discharge headway varies from place to place. It depends on factors such as vehicle type, intersection geometry, and traffic characteristics.

2.5. Signal Timing

Traffic Signals are electrically operated traffic control devices that provide an indication for roadway users to advance their travels by assigning right of way to each approach and movement (FHWA, 2013). Installation of Traffic Signals attempting to obtain any of the following:

- Optimization of Travel delay
- Reduction of crash frequency and/or Severity
- Prioritization of Specific roadway user type or movement
- Accommodation of new or increase in traffic volumes (such as addition of an approach at new development) (FHWA, 2013)

Types of Traffic signal operation include pre-timed, semi-actuated, fully-actuated, hybrid, adaptive or traffic responsive (FHWA, 2013)**.** Pre-timed signal gives right of way to movements based on the predetermined allocation of time. Semi-actuated signals use various detection methods to identify road users on the minor approaches, while fully-actuated signals recognize users on all approaches. The pre-timed signal control system is still widely used in the world and our country the only Signal Controller; even the most recent research has focused on updating the systems to adaptive signal control systems. However, due to the high costs of implementing these new systems, they need to perform significantly better than the present pre-timed signal systems (Smith, 2006).

In 1995, Akcelik published his transport research report, “Traffic signals: capacity and timing analysis.” This report introduced a basic theory of traffic flow at signalized intersections and detailed the signal timing setting process. Akcelik explained the process for the calculation of the signal timings as follows. The first step is calculation of the cycle time by using approximate optimum or practical methods. The algorithms are expressed as follows:

$$C_o = \frac{(1.4+K)L+6}{1-Y}$$ **Eqn-2.1**

Whereas, Co is approximate optimum cycle time; L is intersection lost time in seconds; Y is intersection flow ratio and k is equal to K/100, the stop penalty parameter. Meanwhile, the typical stop penalty values K is set as k = 0.4 for minimum fuel consumption, k = 0.2 for minimum cost and k = 0 for minimum delay.

Further to Akcelik signal timing equation, Webster's equation is widely used in design of signal times as shown herein below.

$$\text{Desirable Cycle Length , Cd} = \frac{L}{1-\left[\frac{Vc}{1615*PHF*\frac{v}{c}}\right]}$$ **Eqn-2.2**

where, L = total lost time s/cycle, sensitive parameter in the design of Cycle Length

PHF = Peak Hour Factor

Vc = Critical lane Volume

v/c = target v/c ratio for critical movements

Considering the above equations and practical perspectives, if the intersection lost time is small, it will significantly decrease the approximate optimum cycle time and small deviation of lost time causes large variation in the cycle length due to this it is the sensitive parameter as shown in the above equations. Thus, quantifying and standardizing start up lost time is vital for the determination of optimal cycle length.

2.6. Start Up Lost Time

2.6.1. Definition of Start Up Lost Time

Different researchers defined the start-up lost time and defining as the unused time by the first few vehicles in the initiation of green time.

Start-up lost time is defined as the additional time, in seconds consumed by the first few vehicles in a queue at a signalized intersection above and beyond the saturation headway, because of the need to react to the initiation of the green phase and to accelerate (HCM, 2010).

Start-up lost time is defined as the excess time that is needed for a number of vehicles to pass through the signalized intersection compared with that would be needed if the signal did not exist **(Matsoukis & Efstathiadis, 2010)**.

The first several vehicles in the queue experience start-up time losses that result in their movement at less than the saturation flow rate. This time loss is referred to as the start-up lost time, which is made up of the perception and reaction time (response time) to the change in signal indication along with the vehicle acceleration time to free-flow speed. Start-up lost time is important in evaluating capacity and driver reaction to the traffic signal indication. Reaction time was considered to include the perception time in addition to the reaction time to the onset of the green arrow indication (Brehmer C.L., 2003).

(Minh and Sano, 2003) defined start-up lost time is the time lost due to driver reactions and vehicle acceleration. The start-up lost time is estimated by the sum of the difference between the observed headway of each vehicle and saturated headway. Correspondingly, (Li & Prevedouros, 2000) defined that start up lost time is the underutilized time due to vehicle accelerating process. Graphical representation start- up lost time as shown in the figure 2.1:

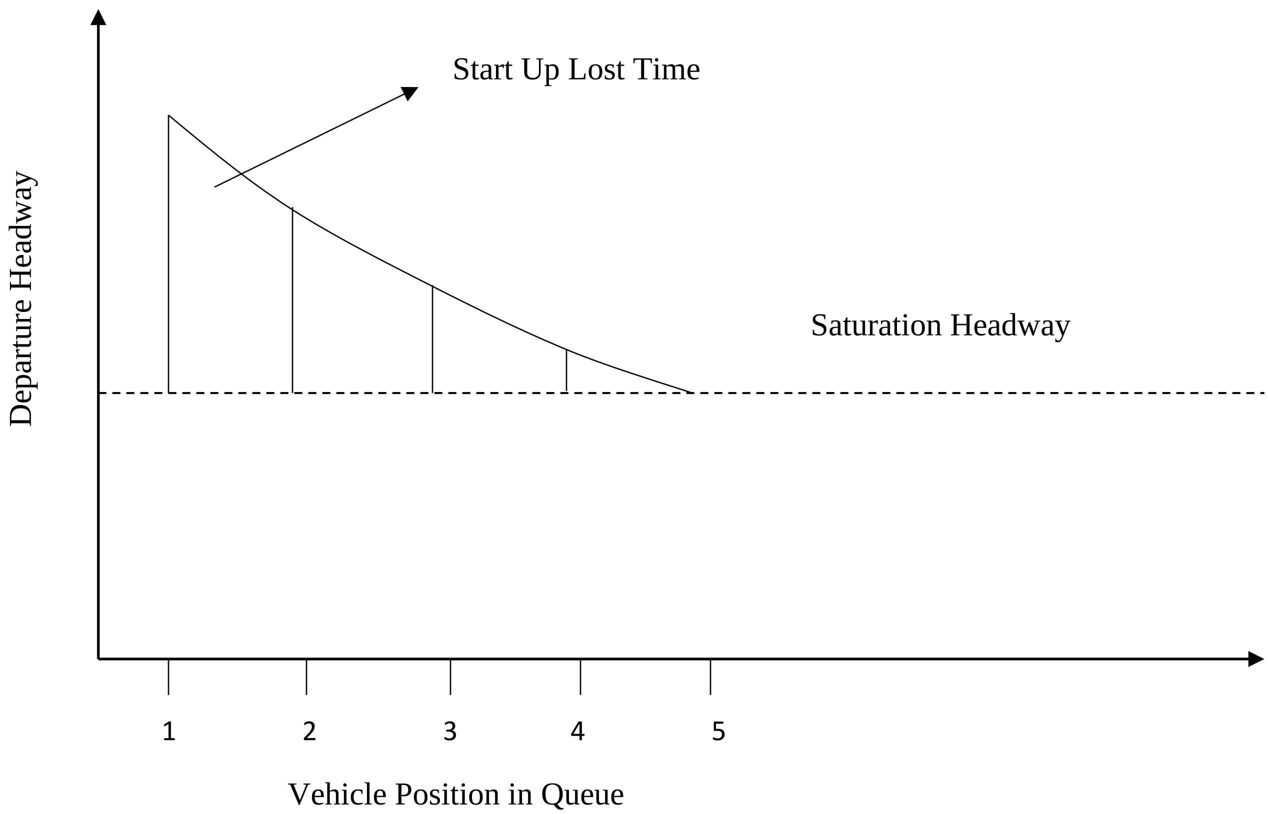


Figure 2.1: Queue Discharge Headway of Vehicles

(HCM, 2010) recommends that the minimum discharge headway be calculated as the average of the headways for the fifth through last queued vehicles. This approach implies that the first four vehicles affect all of the start-up lost time (i.e., N= 4) and it indicates that start-up lost time is generally about 2.0 Sec/phase.

In the computation of the startup lost time determination of discharge headway of the first few vehicles is mandatory. The discharge headway also defined that the difference in the passage time of the front or rear bumper successive vehicles from the stop line. If the front bumper is used, the headways mainly depend on the length and acceleration characteristics of the leading vehicle while if using rear bumper, the headway mainly depends on the length, diving behavior and acceleration characteristics of the following vehicle **(Tong, H Y & Hung, W.T, 2002).** In consideration of this, for this study discharge headway determined by the difference in the passage time of the rear bumper of successive vehicles.

For the first queued vehicle, the definition of headway has to be modified as there is no preceding vehicle. The definition can have a number of variations. Variations are not only due to the selection between front and rear bumper of the vehicle but also the use of the green or amber period for the start counting of the headway. To be consistent with the vehicle discharge headway definition discussed in the above, the rear bumper of the first vehicle is counted **(Tong, H Y & Hung, W.T, 2002).** Some drivers start moving their car before the signal turns green and those vehicles are discarded in this study due to their illegal driving.

Hence, the first vehicle headway is different from the others due to the fact that its response merely depends on the initiation of green time and the first headway is defined as the time elapse from the initiation of green time to cross the stop line.

The start-up lost time is used in the calculation of intersection capacity and level of service. It is assumed that vehicles can move through the intersection at the saturation headway for the duration of the effective green time. The latter is equal to the real green time minus the lost times at the beginning and end of a green phase. A typical value for the total start up lost time of an approach is two seconds, but can range anywhere between 0 and 8 seconds (**Bester & Varndell, 2002**).

2.6.2. Importance of Start Up Lost Time

Startup lost time is part of the total lost time which occur at the beginning of departure process and a lot of studies conducted to merely represent this condition in the traffic movement at signalized intersections **(LUIS F, 2006)**. In part of the total lost time, it is used as a key parameter in the determination of optimum cycle length in Websters Equation and as per this equation, this parameter is a very sensitive parameter which affects the determination of the optimum cycle length. Further, in the lack of local condition data different manuals recommend to use the default value**.** To this effect, different scholars and research institute conduct study on the start-up lost time and revealed that poor traffic signal timing accounts for 5-10% of all traffic delay, or about 300 million vehicle- hours of delay, on major roadways **(Coallition, 2005).** From this, using an actual lost time and providing an optimum signal timing will reduce,

- ✓ Delay by 15 to 40%
- ✓ Travel time up to 25%
- ✓ Emission up to 22%
- ✓ Fuel consumption up to 10%

2.6.3. Queue position for Saturation headway

Saturation headway of vehicles is the constant average headway between vehicles after the first few vehicles until the vehicles in queue at the beginning of green clears the intersection and from the stooped vehicles the discharge headway will attain constant after two or three vehicles **(Matsoukis & Efstathiadis, 2013**).

During the discharge of the Queue, it is understood that the departure headway will be constant after the 4th or 5th vehicle and this constant headway will become the saturation headway as well as it depend on the reaction of drivers, vehicle following behavior and lane changing habits **(Çalişkanelli & Tanyel, 2016**).

Based on the **(HCM, 2010),** the headway after fourth vehicle not show significant variability in its departure and considered in the saturation headway estimation and also (Greenshields, 1935) found that the average discharge headways become constant after the first three or four vehicles. Further this, (**Radhakrishnan & Ramadurai, 2015**) found that the headway tends to follow a constant value after initial four or five vehicles. Albeit, the average discharge headways become stable from position 6th (sixth) under mixed traffic conditions and the saturation headway is 1.57s **(Dey et al., 2013**).

2.6.4. Determination of Start-up Lost Time

The driver of the first vehicle in the queue must observe the signal change to green and react to the change by releasing the brake and accelerating through the intersection. As a result, the first headway will be comparatively long. The second vehicle in the queue follows a similar process, except that the reaction and acceleration period can occur while the first vehicle is beginning to move. The second vehicle will be moving faster than the first as it crosses the stop line, because it has a greater distance over which to accelerate. Its headway will generally be less than that of the first vehicle. The third and fourth vehicles follow a similar procedure, each achieving a slightly lower headway than the preceding vehicle. After four vehicles, the effect of the start-up reaction and acceleration has typically dissipated. Successive vehicles then move past the stop line at a more constant headway until the last vehicle in the original queue has passed the stop line **(HCM, 2010)**.

A study on the leading vehicle discharge headway on the basis of data collected in Beijing reported that the average headway of the first vehicle found to be much longer than the one observed in the other developed countries. Vehicle type and intersection complexity were the major factors which affects the first discharge headway, apart from that disturbances at the beginning of the green would dramatically increase the average headway of the first vehicle. Generally, vehicle type and phase number were found to have a positive effect on first discharge headway, lane assignment and lateral position of vehicles also have a significant effect on first discharge headway **(Liu et al., 2011).**

The first queued vehicles response time to the signal change and the response time of successive vehicles is different and should needed to considered separately. Accordingly, the average starting response time varied from 0.63 to 2.86 seconds and the response time of successive vehicles ranges from 1 second to 1.75 second and also 4% to 30% of the drivers are involved for the response to the signal change from the standing queue (Greenshields B. S., 1947)**.**

The startup delay at signalized intersections has been investigated in a significant number of prior studies. The typical observed value of the startup delay ranges from 1.0 sec to 2.0 sec and **Highway Capacity Manualy 2010** recommend 2.0 Sec/phase. Unlikely to this, several researches in different countries revealed that the start -up lost time will variable due to the driving characteristics, maneuvering characteristics, Geometric condition and signal condition of the study area. Table 2.1 summarizes the findings of the estimated values of the start-up delay from prior studies.

Table 2.1: Observed Start Up Lost Time in Prior Studies

Source	**Date**	**Country (location)**	**Queued Vehicle Number (n_o)**	**Average Start-up Lost Time (sec)**
(Leong H. J., 1964)	1964	Australia (Sydney)	4	1.12
(Gerlough, 1967),	1967	USA (Los Angeles)	5	2.05
(Carstens, 1971)	1971	USA (Iowa)	4	0.75
(Agent, 1983)	1983	USA (Lexington, Kentucky)	4	1.40
(Lee, 1986)	1986	USA (Kansas)	5	3.04
(Roess R. P., 1989)	1989	USA (Texas)	4	1.31
(Efstathiadis S., 1995)	1995	USA (Texas)	4	1.34
(Jacobs, 1998)	1998	South Africa (Stellenbosch)	5	1.43
(Al-Ghamdi, 1999)	1999	Saudi Arabia (Riyadh)	4	2.99

Source	Date	Country (location)	Queued Vehicle Number (n_o)	Average Start-up Lost Time (sec)
(Li & Prevedouros, 2000)	2002	USA (Honolulu)	4	1.76
(David S. H., 2013)	2013	David, et al., [14]	5	2.16
(Shawky & Al-ghafli, 2016)	2016	United Arab Emirates (Abu Dhabi)	4	2.201

2.6.5. The Factors which affect Start Up Lost Time

Several factors that affect the value and distribution of the start-up loss time were also investigated in the prior studies. These factors include the turning movements (trough and left turn), queue length, intersection geometry, time of the day, weather condition, visibility of traffic light etc.

Regarding the turning movements, it was found that the startup loss of the through movement is larger than that of the protected left-turn movement (Roess R. P., 1989) **.** In addition, high standard deviation values were observed for both movements and reflect a big variation of the start-up loss among drivers. Further, Direction of movement had significant effect on the startup lost time and vehicle departure headway **(Matsoukis & Efstathiadis, 2013)**.

Likewise, **(Çalışkanelli, Coşkun Atasever, & Tanyel, 2017)**, start-up lost time value increases rapidly when queue length exceeds 17 veh/ln and percentage of buses reaches 20~25%. The results also indicate that start-up lost time increases as cycle time increases and lower start-up lost time values can be observed in left or right turning lanes. Further **(Çalışkanelli et al., 2017)** stated that Vehicle type and gradient, Pedestrians in the intersection, Perception/reaction time which varies from driver to driver and Psychological factors are some of the factors that affect start up lost time.

However, other studies on the effect of maneuvering characteristics revealed that there is no significant difference in startup lost time for through and left turn movement and mainly the most attainable causes for the startup lost time are driver inattention, vehicle failure and other causes (Earl George T. and Heroy, 1966).

Similarly, (Leong H. J., 1964) conducted study for in the start-up lost time and headway distribution for through movement only by excluding the effect of heavy vehicles and found that after fourth vehicle the headway becomes constant. In this Study, confirmed that the start-up lost time will not have significant difference in the position of lane and lane width.

The lane width and the lane position (for the ranges that were studied) did not have a significant effect on the startup lost time and the vehicle headways **(Matsoukis & Efstathiadis, 2013)** . However,**(Çalışkanelli et al., 2017)**, Lane width and bus percentage are other two parameters which are related to start-up lost time. Start-up lost time decreases as the lane width increases. This is because the drivers feel safer and move faster as the queue discharges. On the other hand, the start-up lost time value increases rapidly when queue length exceeds 17 veh/ln and percentage of buses reaches 20~25%. The results also indicate that start-up lost time increases as cycle time increases and lower start-up lost time values can be observed in left or right turning lanes.

Also, no significant differences between peak and off-peak hours in terms of the start-up loss. Several researchers have found that headway values decrease as the queue length increases. In one of the earlier studies, (Carstens, 1971) examined the start-up delay and vehicle headway values for different types of vehicles and stated that there was a decrease in the headway values depending on the queuing position.

(Lin & Thomas, 2005) determined that the values of headway between the vehicles continued to decrease depending on the queuing position and showed that the headway value continued to decrease even after the fifteenth vehicle in the queue. Another study by (Lieu, 1999) showed that the headway values for small vehicles were smaller and the size of the vehicle in the first line of the queue had a significant impact on the headways of the other vehicles in the queue.

In summary, the headway time or driver response time tends to decrease from the first to the fourth vehicle and then remains constant when the delayed start-up reaction has dissipated. Therefore, saturation headway is achieved and maintained after approximately 10 seconds from green onset or after the fourth vehicle in the queue crosses the stop-line.

As illustrated above and according to **(HCM, 2010)**, Startup lost time is the additional time consumed by the first few vehicles in a queue at a signalized intersection above or beyond the saturation headway because of the need to react to the initiation of the green phase and to accelerate.

In relation to this, different researchers have different argument on the second vehicle delay phenomena, (Greenshields B. S., 1947) stated that the second vehicle can start as soon as the first if the second vehicle queue position is not too close the first vehicle. (Greenshields B. S., 1947) explained that to achieve the (impractical) zero response time, the second driver would need to react to the green signal rather than to the movement of the first vehicle.

(Jan, 2009) identified that the three following characteristics of vehicle departure. Firstly, the second queuing vehicle needs more time to cross because its driver could only react following the movement of the first vehicle. Secondly, successive vehicles in the row keep equal intervals after the fifth vehicle. Thirdly, during the green signal phase, vehicles cross the intersection randomly after the queuing vehicles clear. These characteristics are classified as start-up lost time and saturation headway

David S. H., Kevin P. H., Steven D. S conducted research on the Implications of Distracted Driving on Start-Up Lost Time for Dual Left-Turn Lanes, the study analyzes the start -up lost time for dual left turn by carrying out observations of six field locations in three states. The researcher concludes that the start-up lost time is faced by the first five vehicles with the mean amount of 2.16 Sec.

(Earl George T. and Heroy, 1966) investigated that no significant difference in through lane and left turn movement for the start-up lost time. In connection this, the average starting response time of successive vehicles at peak hour and off-peak hour shows no significant difference. As per their study, found that the first four vehicles will experience start up delay with an average of 1.118 Sec.

Signal types and the time of the day have no significant impact on entering headway at signalized intersections as well as the inside lane of highway has lower headway than the outside lane of the highway (Lee, 1986) . Further, Lee investigated that influence of posted speed limit at signalized intersection and declared that the posted speed limit 20mph have significant higher headway compared with the signal intersection with 30mph and when the queue length increases the headway between the vehicles will decrease.

There are different factors which affect the value of the start-up lost time. A lot of researchers undertaken investigation of this factors, from the parameters Queue length have a negative correlation with the start-up lost time (Li & Prevedouros, 2000).

Start - up lost time and start response time of the drivers are direct related parameters and if start response time is large then the start-up lost time becomes increased and a lot of researches revealed that may parameters like geometry of approach (Approach grade, Lane width and Approach width), Cycle length, Maneuvering Type etc. may have significant effect on the start- up lost time as well as start response time of the drivers. **Highway Capacity Manual 2010** declared that narrow lanes have negative impact on the saturation flow rate and wide lanes allow an increase in the traffic flow. This statement shows that in the narrow lane the drivers waist an extra time and becomes the start -up lost time to be large.

2.6.6. Start Up Lost Time Probability Distribution Models

As explained earlier, start-up lost time, effective departure flow rate and departure headway are frequently used to measure intersection capacity and delays (Jiyuan Tan, 2013). Due to its importance, lots of investigation had been carried out in the last four decades to study the probability distribution of these parameters for traffic simulation, signal timing planning, traffic management purpose and traffic flow forecasting. Distribution models contain richer information and thus help us make more accurate estimation of traffic efficiency (Jiyuan Tan, 2013).

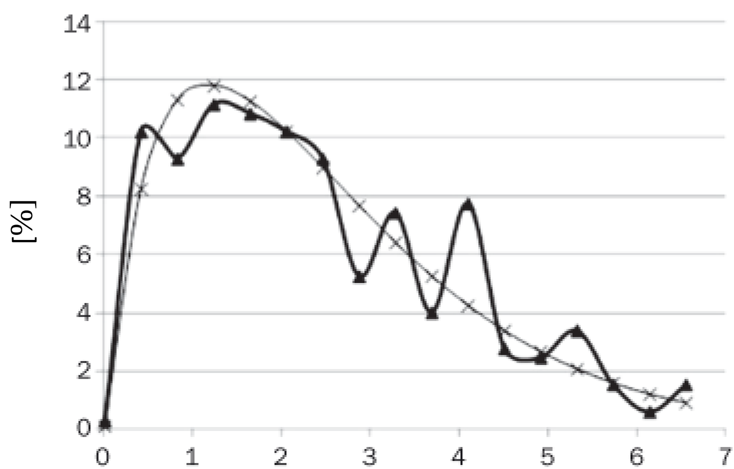
Due to different traffic flow condition, driver behavior, and geometric characteristics, departure headway distributions vary from place to place and the same start up lost time also shows variability from the site to the site.

In literature, only a limited number of studies were carried out for the distribution of start-up lost time values. One of the latest studies (Jiyuan Tan, 2013) belongs to where they suggested a distribution function for start-up lost time values. The start-up lost time follows a shift lognormal distribution, whose parameters can be approximately calculated as Eqn. 2.3.

$$T4 = \sum_{i=1}^{4} hi \sim LN(\mu_{T4}, \delta_{T4})$$

With $\delta^2{}_{T4} = \mathrm{Ln}\left(1+\frac{e^{\delta^2}-1}{4}\right), \mu_{T4} = \frac{1}{2}\delta^2 + \ln\left(\frac{\sum_{i=1}^{4} e^{ui}}{\sqrt{1+\frac{e^{\delta^2}-1}{4}}}\right)$ .... **Eqn -2.3**

(Çalışkanelli et al., 2017) conducted study on the modeling of Start-up lost time and proven that, various statistical Distributions were examined to define the appropriate statistical distribution model. The analysis showed that Weibull Distribution is the statistical model which can be used to represent start-up lost times at signalized intersection approaches with Anderson Darling value is 0.628 and P value is 0.102 for Weibull distribution. The probability distribution curves of observed values *f*(*ts*) and fitted Weibull distribution *g*(*t*) are presented in *Figure 2.2*.



Start-up Lost Time (s)

Figure 2.2: Cumulative distribution curves of observed values $f(t_g)$ and Weibull prediction $g(t_g)$

Source: **Çalışkanelli et al., 2017**

The Weibull distribution is generally used as a lifetime distribution in reliability applications; it can be said that start-up lost time can also be assumed as one of the measures of traffic breakdown at signalized intersections.
The probability density of Weibull distribution as per **(Çalışkanelli et al., 2017)** developed model is:

$$g(t_s) = \left(\frac{\beta ts^{\beta-1}}{\eta^{\beta}}\right) e^{\left[-\left(\frac{ts}{\eta}\right)^{\beta}\right]} \quad \text{.................................... } \textbf{Eqn-2.4}$$

2.6.6.1. Probability distribution Goodness of fit tests

Goodness of fit test in distribution fitting is a way of determining whether a data comes from a specified distribution or not (Anderson, 2011). There are two distribution goodness of fit tests which are frequently used to compare continuous variables, The Anderson-Darling, and The Kolmogorov-Smirnoff tests.

- ✓ The Anderson-Darling Test

The Anderson-Darling Test was developed in 1952 by Theodore Anderson and Donald Darling. Its main purpose was to detect sample depart from normality and was commonly used in engineering fields (Engmann, 2011). The Anderson-Darling test is a refined form of the Kolmogorov- Smirnoff Statistics (Vose, 2010).
The Anderson-Darling Test Statistics is

$$AD = -n-\frac{1}{n}\sum_{i=1}^{n}(2i-1)[lnF(Xi) + \ln(1 - F(x_{n-i+1}))] \quad \text{..............} \textbf{Eqn- 2.5}$$

Where:
n = the sample size,
F(x) = CDF for the specified distribution,
i = the i^{th} sample, calculated when the data is sorted in ascending order

- ✓ The Kolmogorov-Smirnoff Test

The Kolmogorov-Smirnoff test was first developed and introduced by **Kolmogorov (1933, 1941) and Smirnoff (1939).** It compares deviation or vertical distance of empirical distributions from proposed theoretical distributions (Engmann, 2011). The only criteria of the K-S test is the maximum distance between the cumulative distribution function of the proposed distribution and the observed cumulative distribution of the data (Vose, 2010).
Kolmogorov-Smirnoff (K-S) test Statistic is

$$Dn = Max[|Fn(x) - F(x)|] \quad \text{..} \textbf{ Eqn-2.6}$$

Where: -

Dn is known as K-S distance
n is total number of data points
F(x) is Distribution function of fitted distribution
Fn (x) is i/n
i is the cumulative rank of the data point**.**

According to (Vose, 2010)**,** the Anderson-Darling test is more dominant over the Kolmogorov-Smirnoff test due to the following reasons.

- "Ψ(x) compensates for the increased variance of the vertical distances between distributions."
- "f(x) weights the observed distance by the probability that a value will be generated at that x-value"
- "The vertical distances are integrated over all values of x to make maximum use of the observed data (the K-S statistic only looks at the maximum vertical distance)."

An overall indication that the Anderson-Darling test is a better goodness of fit measure than the Kolmogorov-Smirnoff test (Vose, 2010), the result of this study is also supported by (Razali, 2011) concluded that the Anderson-Darling test outperform than the Kolmogorov-Smirnoff test.

2.6.6.2. Start Up Lost Time Regression Modeling

There are not sufficient studies using regression analysis to mode start-up lost time with the predictor variables. A study by using regression modeling is conducted by **(Çalışkanelli et al., 2017).** Data was collected at eight selected intersections in Turkey and the data was collected with video cameras on the intersections of flat grade without the impedance of Pedestrians and Cyclists. According to **(Çalışkanelli et al.,)** study, start up lost time was modeled by multiple linear regression by using the predictors Cycle time, green time, lane width, traffic composition (percentage of minibus, bus and trucks), left or right turning rate of vehicles and queue lengths. From the results it was apparent that lane width (WL), bus percentage in the queue (%bus) and queue length (Lq) affect the start- up lost time. From this, it is understood that a different mix of traffic (heterogeneous traffic) has significant effect on the start- up lost time a rather than homogenous traffic.

(Li & Prevedouros, 2000) developed a predictive model for start-up lost time by using queue length as predictor and the study confirmed that start up lost time is affected by queue length and recommended to develop an efficient model for start-up lost time by using Geometric, Pavement and Traffic parameters.

2.6.6.3. Regression Parameter Estimation Methods

Maximum Likelihood and Ordinary Least Squares which are the best linear unbiased estimators of the underlying population parameter (Washington, 2010).

- ✓ Maximum Likelihood Estimation

Maximum Likelihood Estimation (MLE) is a method of estimating the parameter by maximizing a likelihood function, so that the assumed statistical model the observed data is most probable and Maximum likelihood estimation is one of the most popular and important regression parameter estimation technique. Maximum likelihood technique results in the maximum likelihood estimate (Washington, 2010).

- ✓ Least Square Estimation

The method of least square is a standard approach in a regression analysis to approximate the solution of overdetermined systems. It presents a method for estimating regression model parameters using the sample data. Least square estimation finds the least or minimum solution of the squared disturbances (Washington, 2010)**.**

2.6.6.4. Study Site Selection Criteria's

(Tong, H Y, Hung, W.T, 2002) recommends the following study site selection criteria;

- Different geometrical characteristics;
- Significant proportion of heavy vehicles;
- Significant proportion of turning vehicles for mixed lanes;
- No parking; and
- Insignificant disturbances from bus stops

2.7. Summary of Literature Review

Based on the literature reviewed, start up lost time is variable from city to city of one country due to the prevailing traffic condition and driver behavior. Reviewed literatures confirmed that use of default value for startup lost time which adopted from different areas in design of signal timing will lead to inaccurate signal timing which makes disturbance in traffic operation. An optimum signal timing will reduce Delay by 15 up to 40%, Travel time up to 25%, Emission up to 22% and Fuel consumption up to 10%. Hence, consideration of signal timing design parameters with local condition is mandatory.

The factors which affect start up lost time are investigated by different researchers and the studies revealed that there is different argument on the effect of the factors due to the variability of drivers behavior.

Only limited number of studies are conducted on the probability distribution of start-up lost time. The studies shown different probability distribution for start-up lost time due to the mix of traffic and behavior of drivers. Owing to this, it is necessary to conduct study on the probability distribution of start-up lost time based on Addis Ababa City mix of traffic.

In addition to probability distribution of start-up lost time, there is no sufficient studies using regression analysis to model start-up lost time with the predictor variables. The researchers conduct regression model of start-up lost time recommend to develop an efficient model for start-up lost time by using Geometric, Pavement and Traffic parameters.

CHAPTER THREE: RESEARCH METHODOLOGY

3.1. Description of the Study Area

In order to conduct the study, the researcher selected the study areas which are compatible for accurate determination of the scope of the research. In doing so, four sites are selected based on the following selection criteria and the minimum sample size of the study value determined based on statistical method.

3.1.1. Selection Criteria

3.1.1.1 Traffic Flow Condition

As it is cognizant that the study is needed to represent the real condition of traffic behavior and to represent this the sites shall have a heterogeneous mix of traffic even if it is challenging in the analysis or determination of the results. To this effect, the selected sites/signalized intersections have heterogeneous traffic characteristics in order to represent the actual condition of traffic flow.

3.1.1.2. Geometric Conditions

Geometric condition is the one parameter that affects the scope of the research due to the impact of accelerating and decelerating situation of the vehicles. Owing into consideration of this situation, it is necessary to simulate the scope of this research at different geometric grade (Approach grade), Lane width, Approach width and type of lane. Hence, into merely consideration of this impact, the study areas with different geometric condition are selected for this study.

3.1.1.3. Approach Layout and Number

As it is known, most of the signalized Intersection of the city of Addis Ababa has three and four legged approaches and also the layout of this approaches are right angle, offset, oblique and skewed T arrangement. To this effect, the selected intersections in this study needed to represent the above mentioned conditions to develop an efficient model. Hence, for this research, intersections with three and four number of approaches as well as the intersections with an approach layout of right angle, offset, oblique and skewed T are selected to significantly represent the geometric layout of the city intersections.

3.1.1.4. Signal Timing Conditions

Signal Timing is an allocated time for the movement of vehicles at Signal controlled intersections. As per the demand of traffic volume an allocated time will vary. To this effect, in the city of Addis Ababa the signal timing is varied due to the predicted saturated flow condition. Hence, to sufficiently represent this effect in the study, this study considered the sites that have different signal cycle length and green time.

3.1.1.5. Coordination Effect

The Coordinated signals have an effect on the nearby intersection on the performance and overall traffic maneuvering system. To this effect, the researcher tried to select Isolated Intersections for this research.

3.1.1.6. Other Factors Considered on Selection of the Study Area

The selection of Signalized Intersection considered to minimize the factors that out lie the data of Start -up lost time and also the Intersections selected with minimum disturbance in terms of Bus stoppage and Parking in their approaches.

3.1.1.7. Location of Study Area

The study area is located in the city of Addis Ababa, and the traffic flow, geometric characteristics of the intersection represent the city of Addis Ababa.

3.1.2. Selected Locations

The researcher tries to assess all signalized intersection of the city through field survey and come up with a concrete selection based on the above listed factors. Finally, for this study the researcher selected three four legged and one three legged intersections with different approach layout and grades as well as the selected intersections represent the traffic condition of the city and the brief detail of Intersections with the above parameters indicated on the Appendices **(Appendix A)** and the following table shows a detail description of the study area.

Table 3.1: Detail Description of the Study Area

Lebu Merathaile Signalized Intersection							
Item No	Approach Name	No of Lane	Approach Grade(%)	Approach Width(m)	Green Period	Pavement Roughness Condition (IRI)	Remark
1	German-Lebu	3	2.48	13.535	73 TH, 40 LT	9.485	Selected
2	Kality-Lebu	4	-2.27	12.535	73 TH, 40 LT	6.644	
3	Laphto-lebu	3	1.28	10	50		
4	Sebeta-Lebu	3	-0.5	10	30	6.789	

Jemo Michael Signalized Intersection

Item No	Approach Name	No of Lane	Approach Grade(%)	Approach Width(m)	Green Period	Pavement Roughness Index (IRI)	Remark
1	Jemo-Michael	3	2.72	10.7	30	7.464	Selected
2	Ayertena-Michael	4	-1.26	12.347	40	5.617	
3	Mexico-Michael	3	-0.81	10	46	8.132	
4	German-Michael	4	1.26	12.347	40	6.997	

Gerji Mebrathaile Signalized Intersection

Item No	Approach Name	No of Lane	Approach Grade(%)	Approach Width(m)	Green Period	Pavement Roughness Index (IRI)	Remark
1	Jackross-Gergi m/hail	3	-4.02	9	30	4.428	Selected
2	Megenagna-Gerji m/hail	3	-2.49	10	25		
3	Goro-gerji m/hail	3	4.16	9	40	5.0119	

Saris Abo Signalized Intersection

Item No	Approach Name	No of Lane	Approach Grade(%)	Approach Width(m)	Green Period	Pavement Roughness Index (IRI)	Remark
1	Bole – Saris Abo	4	4.24	11.58	45	7.171	Selected
2	Kaliti – Saris Abo	4	1.56	11.94	45	6.892	
3	Bulebula- Saris	2			25		
4	Saris - Bulbula	2			20		

3.2. Research Data

As indicated in different Studies, Startup lost time affected by different Traffic and Geometric factors. In this study, Eight factors are considered as a Geometric, Traffic, Pavement Condition Parameters for this study. These factors are, Approach Width, Lane Width, Approach Grade, Lane Position, Type of Movement (Through or Left Turn), Vehicular Composition, Green Time and

Pavement Condition. This Parameters Classified as Geometric, Traffic and Pavement Condition Parameters. The Geometric Parameters are Approach Width, Lane Width, Approach Grade and Lane Position where as the Traffic Parameters are Vehicular Composition (Mix of Traffic), Type of Movement and Lane position in the movement. The Pavement Condition is selected due to the fact that acceleration and deceleration of Vehicular Movement related to the pavement Condition which is represented by the Roughness condition of the Vehicles and in this study International Roughness Condition of all Intersections Studied.

3.2.1 Data Requirement

In this Research the Required data classified into four Class this are Geometric Data, Traffic Data, Signal Data and Pavement Condition Data.

Traffic Related Data

- ✓ Movement Type
- ✓ Vehicle Type
- ✓ Vehicular Composition
- ✓ Saturation Headway
- ✓ Start Up Lost Time
- ✓ Lane position of the Vehicles

Geometric Related Data

- ✓ Approach Grade
- ✓ Lane Width
- ✓ Approach Width

Signal Related Data

- ✓ Green Time
- ✓ Cycle Length

Pavement Condition Related Data

- ✓ International Roughness (IRI) data for each Approach of the Signalized Intersection

3.2.2. Vehicle Types for the Study

This study aimed to analyze the effect of different types of vehicle combination on the study value and classified the vehicle types into seven divisions as Car (includes small cars, taxi and pickups), Small Bus, Large Bus, Articulated Bus, Small Truck, Large Truck and Truck Trailers.

According to different studies, Start Up Lost Time is the combination effect of the first few vehicles and to understand this, in this study, 11 Combination of the above Seven Vehicle Types are represented.

Vehicle Types used in this study and their axle number, Class and Description according to Ethiopian Road Authority 2013 Manual classification is as follows.

Table 3.2: Classification of Vehicles used in this study according to ERA 2013 Pavement Design Manual (Table 2.2, Page 2-3)

Type of Vehicle	Class	Axles	Description
Car	1	2	Passenger Car and Taxi
Small Bus	3	2	≤27 Seats
Large Bus	4	2	>27 Seats
Small Truck	5	2	≤3.5 tones
Large Truck	7	2	>7.5 tones
Truck Trailer	8	3	>7.5 tones
Articulated Bus	*	>3	>27 Seats

In accordance with table 3.2, Vehicular Classification is conducted and in this study one new type of vehicle classification is induced due to the fact that the introduction of new large Articulated Bus in the City and to merely represent this type of Vehicle the study treated this type of the Vehicle independently in the computation of Departure Headway.

On the Top of this, as we know the vehicle types has an impact on the study value while the study value as mentioned above is a combination effect of this type of vehicles especially the first few vehicles in the Queue. Hence, to significantly address the impact of Vehicle Type on the study value, it is mandatory to combine the above type of Vehicles based on the frequently observed Arrival and Departure in Signalized Intersection of Addis Ababa.

3.2.3. Vehicular Composition for the Data Requirement

As stated above, the study value is a combination effect of different vehicular composition. In order to assess the effect of different vehicle types on the study value, it is mandatory to conduct a combination of different vehicle types. Accordingly, the combination of different vehicle types is conducted through the observation of frequently occurred real combinations in signalized intersections of Addis Ababa. In this study, the following vehicular combinations are identified based on observation of mix of traffic that frequently occur on the study sites and also this combination prevails the two extreme conditions of Vehicular combination which is Combination A and E.

- ✓ Vehicular Composition A = if all the first four arrived Vehicles in the Queue are Car
- ✓ Vehicular Composition B1 =If three of the Vehicles in the first Queue are Car and the remaining are either Small Bus or Small Truck
- ✓ Vehicular Composition B2 = If three of the Vehicles in the first Queue are Car and the remaining are either Large Bus or Large Truck
- ✓ Vehicular Composition B3 = If three of the Vehicles in the first Queue are Car and the remaining are either Truck Trailer or Articulated Bus
- ✓ Vehicular Composition C1 = If two of the Vehicles in the first Queue are Car and the remaining are either Small Bus or Small Truck
- ✓ Vehicular Composition C2 = If two of the Vehicles in the first Queue are Car and the remaining are either Large Bus or Large Truck
- ✓ Vehicular Composition C3 = If two of the Vehicle in the first Queue are Car and the remaining are either Large Bus or Large Truck
- ✓ Vehicular Composition D1 = If one of the Vehicle in the first Queue is Car and the remaining are either Small Bus or Small Truck
- ✓ Vehicular Composition D2 = If one of the Vehicle in the first Queue is Car and the remaining are either Large Bus or Large Truck
- ✓ Vehicular Composition D3 = If one of the Vehicle in the first Queue is Car and the remaining are either Truck Trailer or Articulated Bus
- ✓ Vehicular Composition E = If all the Vehicles are Large Bus and Large Truck or Articulated Bus and Truck Trailer

3.2.4. Sample Size

To come up with an efficient model, determination of Sample Size is a preliminary task of the researcher and for this study the minimum sample size is determined based on the historical sample means and standard deviation of different studies. As detect that, Sample size is based on the significant level of the study and the minimum number of Start-up Lost Time data is determined as follows;

$$n = \left[\frac{Zc\delta}{E}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{Eqn-3.1}$$

Where, E is Margin of Error, δ is Standard Deviation, and Zc is the Z score

Selection of standard deviation value is based on the previous studies on the Start Up Lost Time. According to the previous studies, the standard deviation value of Start-up lost time will deviate from 0.5 to 0.8 and in this study the Standard deviation value used is 0.5 and for 95% Confidence

Interval the Margin of Error is 0.05 and the minimum number of Start Up Lost Time required will be:

n = $\left[\frac{Zc\delta}{E}\right]$ = (1.96*0.5/0.05) = 385 Start-up lost time……………………………..**Eqn 3.2**

According to (Vanvoorhis, 2007)**,** the general rule of thumb for determining sample size for regression analysis is 50+8*m where m is number of independent variables but the minimum number of samples should be 50.

For this study m=8 N = 50 + 8*8 = 116

A total of 1133 Start Up lot time data collected for this study and the following table presents the sample size for Start Up lost time data.

Table 3.3: Sample Size for Start Up Lost Time Data

Item No.	Vehicular Composition	Sample Size
1	A	630
2	B1	148
3	B2	146
4	B3	17
5	C1	60
6	C2	72
7	C3	9
8	D1	12
9	D2	24
10	D3	8
11	E	7
	Total	**1133**

As shown in table 3.3, the sample sizes for vehicular composition of C3, D3 and E are small compared to other vehicular composition this deviation on the sample sizes occurred due to the least occurrence of large vehicles on the day time traffic maneuvering at Signalized Intersections of Addis Ababa.

3.2.5. Sample Eligibility Criteria

In this Study, Samples with Pedestrian impedance or an accident on the maneuvering of vehicles and those vehicles crossed the stop line during red lightening and the vehicles not queued in the red light are rejected in the study.

3.3. Data Collection

3.3.1. Primary Data Collection

The Primary Data for this study is Geometric related data's by directly measuring of Approach Width and Lane width with Rollo Meter and Approach Grade by Leveling.

3.3.1.2. Secondary Data Collection

Traffic video data were taken from the previous works which is undertaken for the partial fulfillment of Master of Thesis in Road and Transport Engineering in Addis Ababa University by Michael A and Habtamu C. The Secondary Data was taken from researchers due to the following reasons;

- ✓ The data was collected by the unique way with phantom 4 drones with the latest technology for two and half hours from 6:30 AM to 9:00 AM in the morning for each intersection. The Quality of data is high according to the study and it is useful to accurately determine the traffic parameters rather than using normal video cameras.

- ✓ The data was collected in the end of May, 2018 which is near to the start of this study and the exact date of collection of the data for Four Signalized Intersection as follows

    - At Gerji Mebrathail on Tuesday May, 29, 2018.
    - At Saris Abo on Wednesday May, 30, 2018.
    - At Lebu Mebrathail on Thursday, May, 31, 2018.
    - At Jemo Michael on Tuesday, June, 5, 2018.

- ✓ Into considering the quality of data and limitation of time as well as budget the researcher planned to take the data which was collected by the above researchers.

The pavement Condition of the study area is indicated by the International Roughness Index on each Approaches of Intersecting Roads. In this study, Pavement Condition data were collected from Addis Ababa City Road Authority Central Data Base Center while during taking the data the major challenge is the date of collection of this data. To this effect, the study tries to assess time series data of the pavement condition for two years of period 2018 and 2019 G.C. As understood from the time series data the International Roughness Index value is not majorly deviate and there is no any maintenance during this period for all Selected Signalized Intersections.

3.3.2. Data Extraction

The major task in executing this study was extraction of Traffic Video Data. The Traffic Video Data is collected for all Intersections from 6:30 AM to 9:00 AM in the Morning due to the fact most of the selected intersections are Conveying loops for the Commercial Areas i.e. Mexico, Megenagna, Kaliti, Jemo, Bole and German. To this effect, most of the commuters use the morning time to arrive their business area as well as in the morning time all drivers are active compared to the evening time and the occurrence of distracted drivers will minimized during the morning time. Hence, the collected data will effectively address the objective of the study.

After examination of the data, the next step is extraction of video data. As stated above, Start Up Lost Time is defined as an extra time wasted by few vehicles from the saturated condition. To execute this study, it requires Departure Headway of the Vehicles and Saturation Headway per each phase.

Accordingly, the first task is extraction of Departure Headway data for all Vehicles for the Collected data with Queue Position of the Vehicles and this is the major challenge of the study due to the lack of traffic Video extraction software and it is executed manually by video playing software. In order to minimize the error in extraction of data the researcher tries to conduct different video playing software and the one that will give the data with microseconds is used in this study and the maximum error in the data extraction was 0.033 seconds.

In this study, for all selected signalized intersections a total of 17,900 departure headway of Vehicles extracted from the traffic video data based on the above vehicle classification which is very time consuming and challenging. From 17, 900 departure headways 12,861 is Through Movement and 5,039 is Left Turn Movement Departure Headway. The figure herein below shows the camera view of all the Signalized Intersections.

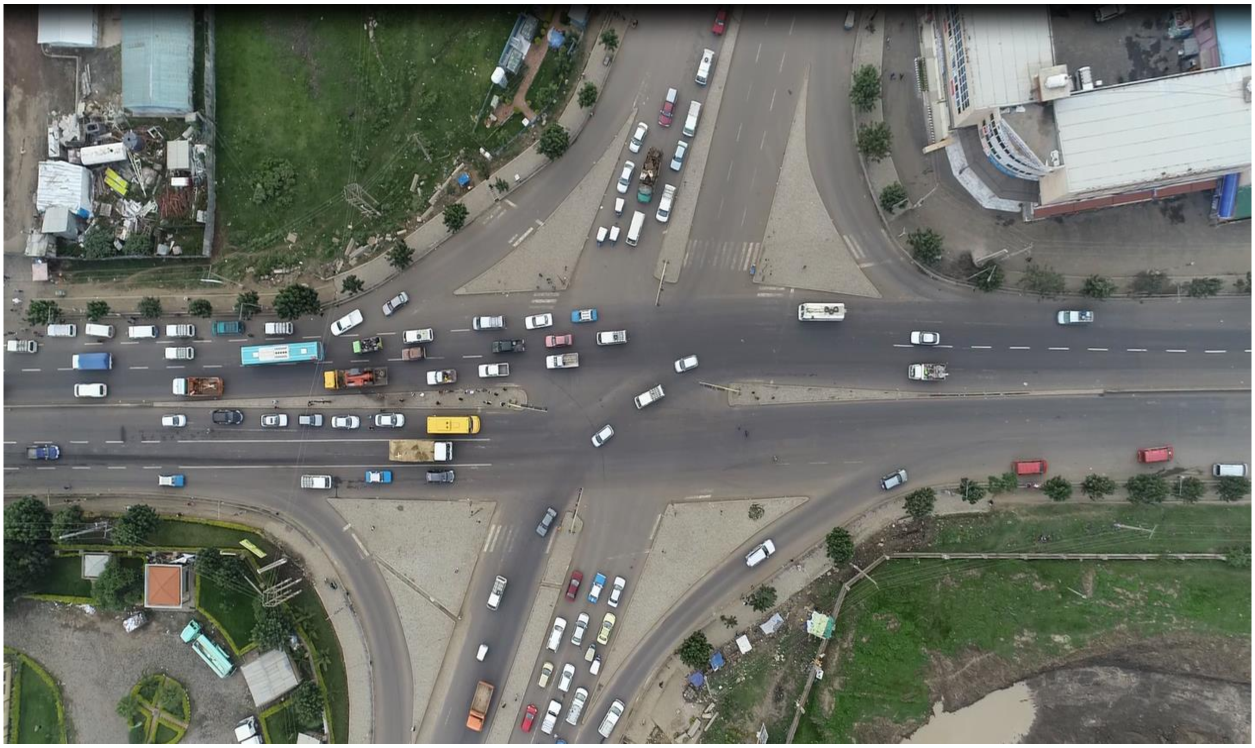


Figure 3.1: Lebu Mebrathaile Signalized Intersection

Figure 3.2: Jemo Michael Signalized Intersection

Figure 3.3: Saris Abo Signalized Intersection

Figure 3.4: Gerji Mebrathaile Signalized Intersection

3.4. Data Analysis

The manipulated data was used to analyze the start -up lost time for each signalized intersection and how will its value deviate with different approach grade, approach width, lane width, cycle length, maneuvering type and vehicle composition will be conducted. Statistical analysis used a basic tool in analysis of the data.

- ✓ With the minimum of ten (10) queues are selected for the sampling
- ✓ Headway for each lane of each approach is determined when the rear bumper of the vehicle crossing the stop line and for the leading vehicle the headway is an elapsed time from initiation of green time to the rear bumper of the vehicle crossing the stop line.
- ✓ A minimum of 25cycles headway determined for each lane of each approach
- ✓ Headway vs Vehicles in the Queue is plotted

3.4.2. Estimation of Passenger Car Unit (PCU)

Urban roads characterized by a different mix of traffic, resulting complex interaction between various kinds of vehicles. To Cater this, different studies revealed that transforming of this heterogeneous mix of traffic to homogenous traffic by converting all the vehicles to the same unit. The unit generally employed is a Passenger Car Unit (PCU), each vehicle type is converted to equivalent PCU.

According to (Umama, 2010) several approaches to estimate PCU values have been used and the most commonly applied approaches is the headway approach.

(Hounsell, 1989) and (Leong L. , 2004) found that the average headway method is the best for predicting Saturation Headway, lost times and Passenger Car units (PCU) factors.

In this study, considering the widely application of average headway methods the Passenger Car Unit is estimated based on the Average Headway of each vehicle type for each approach.

(Greenshields B. S., 1947) estimated PCU value by the following equation. This method is known as a basic headway method.

$PCU_i = H_i/H_c$

Where PCU_i= passenger car unit of vehicle type i, H_i = average headway of vehicle type i, H_c = average headway of passenger car.

3.4.2. Estimation of Queue position for Saturation

Subsequent to determining the PCU value for each vehicle, estimation of queue position for saturation and start up lost time is mandatory to estimate the study value. According to (HCM, 2010), the headway after fourth vehicle not show significant variability in its departure and considered for saturation headway estimation. In order to estimate the saturation headway and start up lost time of vehicles it is necessary to determine the location of queue position for saturation beginning. Hence, queue position for saturation was determined based on testing with t- test and confirming with the headway curve.

3.4.3. Start Up Lost Time Estimation and Modeling

According to (HCM, 2010), the start-up lost time is defined as the total time difference between the saturation headway and the first four headways.

$$\text{SULT} = \sum_{i=1}^{4}(hi - hs) \qquad \textbf{Eqn 3.3}$$

Whereas, SULT is Start up Lost time, i is the position of vehicle in the queue, hi is the discharge headway of the i^{th} vehicle in sec and hs is the saturation headway (sec).

In connection this [**Minh and Sano, 2003],** start-up lost time is the time lost due to driver reactions and vehicle acceleration. The start-up lost time is estimated by the sum of the difference between the observed headway of each vehicle and saturated headway.

In this study, estimation of saturation headway was executed after converting all the vehicles into the same unit of passenger car and after determining the queue position for saturation.

$$\text{Saturation Headway hs} = \text{Avearge} \sum_{i=5}^{N} hi \qquad \textbf{Eqn 3.4}$$

Whereas, hi= Departure headway of the vehicles after 4^{th} Queued Vehicle

Following this, Start Up Lost Time per each phase is determined by the sum of the difference between the observed headway of each vehicle and Saturation Headway.

Start Up Lost Time SULT = $\sum_{I=1}^{n}(hi - hs)$ where hs, saturated headway, hi is queue discharge headway of the i-th vehicle where n is the location of saturated headway (vehicle position for saturated headway).

In this study, start-up lost time was determined based on different vehicle composition of the first vehicles which face lost time from the saturation condition.

3.4.4. Probability Distribution Modeling of Start Up Lost Time

The estimated data of start-up lost time was analyzed and its statistical distribution of the data was tested with Minitab 18 statistical software package.

The probability distribution is fitted to Start Up Lost Time for each vehicle composition. Prior to fitting any probability distribution of the data is tested for outliers using Grubb's test and removed from the analysis. Then the data tested for normality using Anderson-Darling normality test and for non-normal data other probability distribution are fitted.

Probability plots and Anderson-Darling test are considered to test the existing distribution models. The P-P plot compares an empirical cumulative distribution function of a variable with a specific theoretical cumulative distribution function. If the selected variable matches the test distribution, the points cluster around a straight line.

The following are some common distributions that are usually used for fitting Start Up Lost Time Value. Lognormal, Weibull and Johnson Transformation function.

Grubbs' test with the following Null and Alternate Hypothesis is used to determine outliers.
Ho: - All the data values come from the same normal population
H1:- Smallest or largest data value is an outlier

Anderson-Darling Test with the following Null and Alternate Hypothesis is used to
test for normality.

Ho: - Data values do come from the specified probability distribution
H1:- Data values do not come from the specified probability distribution.

- ✓ Parameter estimation

Parameter estimation for probability distribution is based on the maximum likelihood (ML) method. Maximum Likelihood Estimation (MLE) is a method of estimating the parameters by maximizing a likelihood function, so that under the assumed statistical model the observed data is most probable. The point in the parameter space that maximize the likelihood function is called a maximum likelihood estimate.

The likelihood of a set of data is the probability of obtaining that particular set of data given the chosen probability model. It provides a consistent approach to parameter estimation problems. This means that maximum likelihood estimates can be developed for a large variety of estimation situations. The method provides us with a minimum variance unbiased estimators as the sample size increases.

- ✓ Goodness of fit test

Goodness of fit testing is unique type of hypothesis test that can be used to determine if a sample data set has been drawn from specific population distribution.

The most commonly used Goodness of fit tests are,

- The chi-square
- Kolmogorov-Smirnov
- Anderson-Darling
- Shipiro-Wilk

The chi-square test used for a discrete distribution while, for continuous distribution the Anderson-Darling test and Kolmogorov-Smirnov test are used. To determine the goodness of fit test first calculate the test statistics Anderson- Darling or Kolmogorov-Smirnov and then compute the approximate p-value associated with the test Statistics. In this study, the Anderson-Darling goodness of fit test is used.

- ✓ Anderson Darling Goodness Fit test

The Anderson-Darling Goodness of Fit Test (AD-Test) is a measure of ***how well the data fits a specified distribution***. The hypotheses for the Anderson-Darling test are:

- H_0: The data follow a specified distribution
- H_1: The data do not follow a specified distribution

The Formula for Anderson Darling goodness of fit test is,

$$AD = -n-\frac{1}{n}\sum_{i=1}^{n}(2i-1)[lnF(Xi)+\ln(1-F(x_{n-i+1}))] \quad \textbf{Eqn-3.5}$$

Where:
n = the sample size,
F(x) = CDF for the specified distribution,
i = the *i*th sample, calculated when the data is sorted in ascending order

After calculating the Anderson Darling value and P-value, Use the corresponding p-value (when available) to test if the data come from the chosen distribution. If the p-value is less than a chosen alpha 0.05, then reject the null hypothesis that the data come from that distribution. While sometimes p-value is greater than alpha for more than one probability distribution and when this happens, it is necessary to compare the probability distribution by using the Anderson Darling value and P value and in order to conclude one distribution is best, its Anderson-Darling value substantially lower than others or P – value Substantially larger than the others.

3.4.5. Start Up Lost Time Regression Modeling

Multiple linear regression is used to model Start up Lost Time for different Vehicle Composition and Minitab 18 statistical analysis software is used as a major data analysis tool. Start Up Lost Time of different vehicle composition modeled against below listed variables.

Dependent Variable

- ✓ Start-up Lost Time for different vehicle composition

Independent Variables

- ✓ Approach Grade
- ✓ Approach Width
- ✓ Lane Width
- ✓ Green Time
- ✓ Vehicle Composition of the first four vehicles which faced for start-up lost time which is listed below;
    - o Vehicular Composition A = if all the first four arrived Vehicles in the Queue are Car
    - o Vehicular Composition B1 =If three of the Vehicles in the first Queue are Car and the remaining are either Small Bus or Small Truck
    - o Vehicular Composition B2 = If three of the Vehicles in the first Queue are Car and the remaining are either Large Bus or Large Truck
    - o Vehicular Composition B3 = If three of the Vehicles in the first Queue are Car and the remaining are either Truck Trailer or Articulated Bus
    - o Vehicular Composition C1 = If two of the Vehicles in the first Queue are Car and the remaining are either Small Bus or Small Truck
    - o Vehicular Composition C2 = If two of the Vehicles in the first Queue are Car and the remaining are either Large Bus or Large Truck
    - o Vehicular Composition C3 = If two of the Vehicles in the first Queue are Car and the remaining are either Large Bus or Large Truck
    - o Vehicular Composition D1 = If one of the Vehicle in the first Queue is Car and the remaining are either Small Bus or Small Truck
    - o Vehicular Composition D2 = If one of the Vehicle in the first Queue is Car and the remaining are either Large Bus or Large Truck
    - o Vehicular Composition D3 = If one of the Vehicle in the first Queue is Car and the remaining are either Truck Trailer or Articulated Bus
    - o Vehicular Composition E = If all the Vehicles are Large Bus and Large Truck or Articulated Bus and Truck Trailer (no car, small bus & small truck)
- ✓ Type of Movement (Through and Left Turn)

- ✓ Pavement Condition (International Roughness Index)
- ✓ Lane position of the vehicles (Near Midian (inner lane), middle lane and Near curb (outside lane))

Vehicle composition, Type of Movement and lane position are the categorical variables for the model. Base Condition of this model is Vehicular Composition A, Inner lane and Through Movement

The general mathematical formulation of the regression model is written as follows: -

$$SULT = \beta 0 + \beta 1 * AG + \beta 2 * GT + \beta 3 * LW + \beta 4 * AW + \beta 5 * ML + \beta 6 * OL + \beta 7 * LT + \beta 8 * B1 + \beta 9 * B2 + \beta 10 * B3 + \beta 11 * C1 + \beta 12 * C2 + \beta 13 * C3 + \beta 14 * D1 + \beta 15 * D2 + \beta 16 * D3 + \beta 17 * E \quad \ldots \textbf{Eqn - 3.6}$$

Where: -

SULT is Start Up Lost Time

$B0, \beta 1 \ldots, \beta 17,$ are estimated coefficients from the regression analysis

AG is approach Grade

GT is Green Time

LW is Lane Width

AW is Approach Width

ML is Middle lane position

OL is Outside lane position

LT is Left Turn

B1,B2,B3,C1,C2,C3,D1,D2,D3 & E are the vehicular composition

3.4.5.1. Multiple Linear Regression Assumptions

Before computing multiple linear regression Analysis, the following Assumptions should need to be fulfilled;

1. The relationship between the independent and dependent variables to be linear. The linearity assumption can best be tested with scatterplots.

2. The errors between observed and predicted values (i.e., the residuals of the regression) should be normally distributed. This assumption may be checked by looking at a histogram or a Q-Q-Plot.

3. No multicollinearity in the data and Multicollinearity occurs when the independent variables are with too highly correlated each other. Multicollinearity may be checked by;

    - ✓ Correlation matrix and when computing a matrix of Pearson's bivariate correlations among all independent variables, the magnitude of the correlation coefficients should be less than .80.

- ✓ Variance Inflation Factor (VIF) – The VIFs of the linear regression indicate the degree that the variances in the regression estimates are increased due to multicollinearity. VIF values higher than 10 indicate that multicollinearity is a problem.

4. Homoscedasticity (residuals have a constant variance along the fitted value). A scatterplot of residuals versus predicted values is a good way to check homoscedasticity and Breusch-Pagan /Cook-and Weisberg test also used to check this assumption. If the data show homoscedasticity, the data points will be fairly randomly distributed with a fairly even spread of residuals at all predicted values.

   Breusch-Pagan / Cook-Weisberg test for heteroscedasticity
   Ho: Constant variance

- ✓ Goodness of fit test

The developed model is tested with statistical goodness of fit tests in this case the adjusted R^2 test is conducted. The closer the Adjusted R^2 value to 1 the better the model. Adjusted R^2 test is preferable because SSE / − (n-p) is the error or residual mean square and SST / − (n-p) is a constant, R^2 adj will only be increased when a variable added to the model which reduces the error mean square.

$$R^2Adj = 1 - \frac{SSE(n-p)}{\mathrm{SST(n-1)}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{Eqn -3.7}$$

Where; -SSE is sum squared regression error
SST is sum squared total error
n is sample size
p is number of predictors

Stepwise multiple linear regression used for the modeling of the variable and the significant predictors for the response are determined based on their P-value.

CHAPTER FOUR: RESULTS AND DISCUSSIONS

4.1. Estimation of Queue Position for Saturation Headway and Start Up Lost Time

Prior to conducting queue position for saturation headway, passenger car unit is estimated for each approaches of the intersection by the headway ratio method and the data values of passenger car unit for each approach and vehicle type is shown in the table below and please see **Appendix B** for the detail data value of PCU values. A total of 17,900 discharge headway of seven vehicle types extracted from the video data and to convert this to the same unit headway ratio method is used and the same to all approaches PCU values are computed.

Table 4.1: PCU values for Goro to Megengna Approach

Gerji Mebrathaile Signalized Intersection		
Goro to Megengna		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.064	1.000
Small Bus	2.788	1.350
Small Truck	2.548	1.234
Large Bus	4.375	2.119
Large Truck	4.291	2.0787
Articulated Bus	6.000	2.906

After converting all the vehicles to the same unit of passenger car unit by calculating it with the headway ratio method, the headway curves are plotted with the queue position of the vehicle. Accordingly, the position of saturation beginning or the vehicles which face start up lost time was determined. The tabular and graphic data herein below presents the headway curve and mean headway with the queue position of the vehicle.

Table 4.2: Goro to Megengna Discharge Headway vs Queue Position

Discharge Headway							
Queue Position	**N**	**Mean**	**St.Dev**	**Minimum**	**Median**	**Maximum**	**Skewness**
1	113	3.1242	0.5938	2.4455	2.9772	6.3276	2.42
2	113	2.7997	0.6542	1.7858	2.6494	4.6434	0.98
3	113	2.5401	0.8504	1.0731	2.5121	5.3556	0.52
4	113	2.4052	0.97	1.0744	2.2128	5.6383	0.86

5	113	2.2628	0.7821	0.5	2.167	5.084	1.09
6	112	2.2197	0.715	0.916	2.125	4.667	0.91
7	107	2.1766	0.7325	1	2.166	5.75	1.44
8	106	2.1163	0.5945	0.6782	2.0543	4.1696	0.96
9	106	2.0632	0.6228	0	2.0415	4.75	0.67
10	101	2.0193	0.518	0.917	2	3.167	0.26
11	97	2.1322	0.6695	1.1481	2.002	4.833	1.29
12	94	2.0509	0.6179	0.833	2	4.083	0.57
13	91	2.2311	0.9384	1.166	2.159	8	3.13
14	86	2.172	0.938	0.922	1.917	7	2.54
15	84	2.1276	0.883	1	2	8	3.92
16	78	2.0908	0.5295	1.25	2	4.333	1.51
17	75	2.0171	0.6906	0.916	1.917	5.083	2.13

According to the mean data of this table, Analysis of Variance was conducted using t- test. Before conducting t-test it is necessary to check the normal distribution of time headway of the vehicles after the 4th vehicle and the distribution shows that the time headway the vehicles after the fourth vehicle shows normal distribution with the following Anderson – Darling Test Result.

Distribution	**Anderson – Darling Test Result**	
	AD	P
Normal	0.198	0.855

As we shown in table 4.2, the queue position of the vehicles becomes the independent variable and the data shows independence based on the queue position of the vehicles. Into consideration of this points, t- test was conducted by using the following null and alternative hypothesis.

Null Hypothesis Ho = the mean discharge headway after the fourth queued vehicle are equal
Alternative Hypothesis H1 = the mean discharge headway after the fourth queued vehicle are not equal

To conduct this hypothesis, the mean discharge headway of the 5th queued vehicle is compared with the mean discharge headway of all the vehicles after the 4th Vehicle. Accordingly, the mean discharge headway of the vehicle after the 4th vehicle is 2.1292 Sec while the discharge headway of the 5th vehicle is 2.2628 Sec and the analysis of means based on t- test is shown herein below.

T statistics and Descriptive Statistics for 5th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
113	2.2628	0.7821	0.0736	(2.1170, 2.4086)

μ: mean of Sample

Test

Null hypothesis	H_0: μ = 2.1292
Alternative hypothesis	H_1: μ ≠ 2.1292

T-Value	P-Value
1.82	0.072

The p value this statistic is 0.072 > 0.05 which entails that there is no ground to reject the null hypothesis which means the mean discharge headway of the vehicles after the 4th vehicle are the same with the mean discharge headway of the vehicle on 5th queued vehicle. This implies that the 5th queued vehicle is under the saturation condition.

Further, the discharge headway of 4th vehicle mean discharge headway was compared with the mean discharge of all the vehicles after the 4th vehicle which is 2.1292 Sec and t- test shown herein below.

T statistics and Descriptive Statistics for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
113	2.4052	0.9700	0.0912	(2.2244, 2.5860)

μ: mean of Sample

Test

Null hypothesis	H_0: μ = 2.1292
Alternative hypothesis	H_1: μ ≠ 2.1292

T-Value	P-Value
3.02	0.003

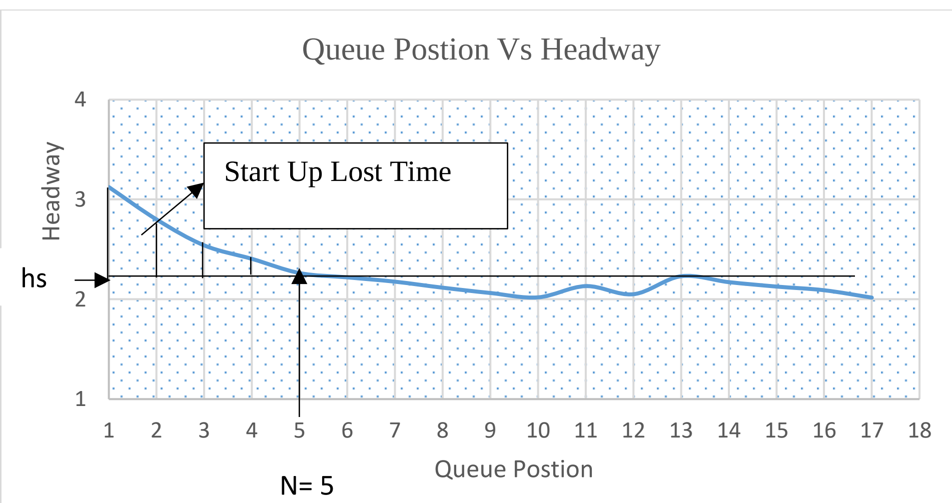


Figure 4.1: Mean Discharge Headway Vs Queue position for Goro to Megengna Left Turn

As shown in the above t – statistics, the P- value for testing 4th queued vehicle is 0.003 which is less than 0.05 means that the 4th queued vehicle mean discharge headway is not the same with the mean discharge headway of all the vehicles after the 4th vehicle. Hence, saturation headway of the vehicle begins at the 5th queued vehicle and the first four vehicles faced start- up lost time as shown in the above statistics and figure 4.1.

Table 4.3: Descriptive Statistics for Queue Position Vs Mean Headway Jacros to Megengna Through Movement

Discharge Headway							
Queue Position	**Total**	**Mean**	**StDev**	**Minimum**	**Median**	**Maximum**	**Skewness**
1	131	3.263	0.447	2.347	3.259	5.017	0.67
2	131	2.553	0.661	1.299	2.373	4.643	0.94
3	130	2.471	0.876	0.917	2.315	5.473	0.7
4	131	2.343	0.803	0.916	2.167	4.431	0.78
5	128	2.241	0.907	0.750	2.083	5	0.89
6	127	2.258	1.010	1	2	6.084	1.33
7	122	2.180	1.113	0.750	1.834	8.166	2.73
8	119	2.241	0.945	0.833	2.083	6.084	1.3
9	114	2.203	0.936	0.833	2.041	6.334	1.21
10	106	2.347	0.98	0.75	2.167	6.916	2.24
11	103	2.165	0.996	0.715	2	7.084	1.85
12	94	2.127	0.885	0.834	1.917	5.084	1.04
13	87	2.087	0.854	0.666	1.833	5.334	0.93
14	68	1.967	0.825	0.916	1.75	4.75	1.19
15	60	1.985	0.842	0.833	1.833	5.334	1.56
16	47	1.814	0.534	1	1.75	3.166	0.77

The same to the above t- Statistics was conducted for determination of queue position for saturation. Before conducting t-test it is necessary to check the normal distribution of time headway of the vehicles after the 4th vehicle and the distribution shows that the time headway the vehicles after the fourth vehicle shows normal distribution with the following Anderson – Darling Test Result.

Distribution	**Anderson – Darling Test Result**	
	AD	P
Normal	0.371	0.363

As we shown in table 4.3, the queue position of the vehicles becomes the independent variable and the data shows independence based on the queue position of the vehicles. Into consideration of this points, t- test was conducted by using the following null and alternative hypothesis.

Null Hypothesis Ho = the mean discharge headway after the fourth queued vehicle are equal
Alternative Hypothesis H1 = the mean discharge headway after the fourth queued vehicle are not equal.

T – Statistics and Descriptive Statistics for the 5^{th} queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
128	2.2407	0.9076	0.0802	(2.0820, 2.3994)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1346$
Alternative hypothesis	H_1: $\mu \neq 2.1346$

T-Value	P-Value
1.32	0.188

The P value of the test is > 0.05 which means that the 5^{th} queued vehicle mean discharge headway is
the same to the mean discharge headway of all the vehicles after the 4^{th} vehicle.

T – Statistics and Descriptive Statistics for 4^{th} queued vehicle

N	Mean	St.Dev	SE Mean	95% CI for μ
131	2.3429	0.8035	0.0702	(2.2040, 2.4818)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1346$
Alternative hypothesis	H_1: $\mu \neq 2.1346$

T-Value	P-Value
2.97	0.004

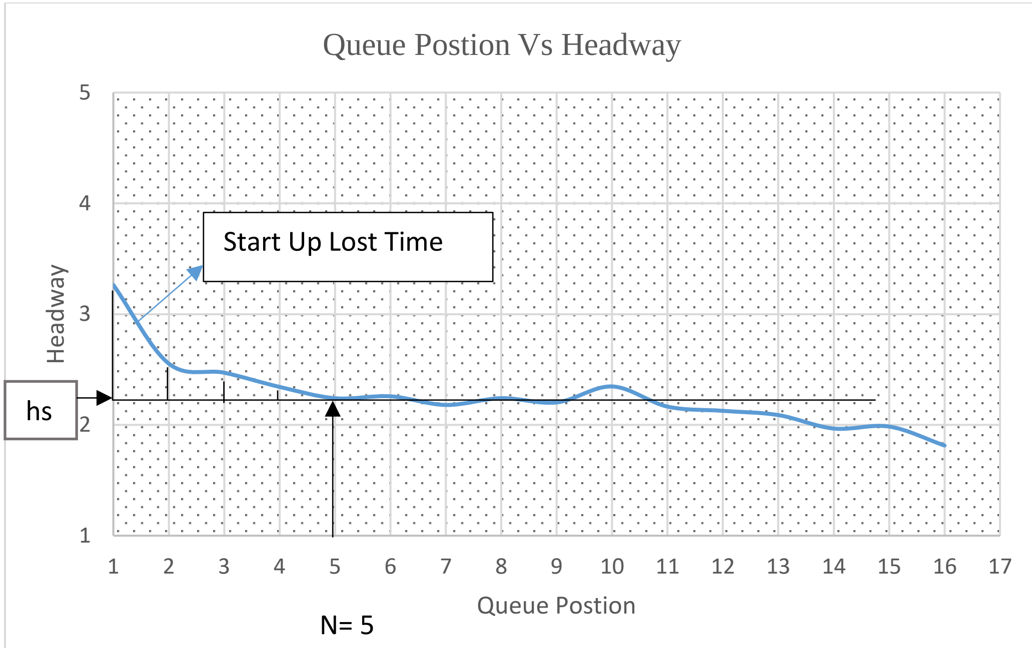


Figure 4.2: Queue Position Vs Mean Headway Jacros to Megengna Through Movement

The P value of the test is < 0.05 which means that the 4^{th} queued vehicle mean discharge headway is not the same with the mean discharge headway of all the vehicles after the 4^{th} vehicle which entails that the 4^{th} queued vehicles is not in the saturation condition and concluding that the first four vehicles face start up lost time and the vehicles after the 4^{th} queued are in the saturation condition as shown in the above t- statistics and also the figure 4.2 shows that saturation headway begins at 5^{th} queued vehicle and the first four vehicles face start up lost time.

The same of the above statistics was done for all the approaches and the queue position for the estimation of saturation headway and start-up lost time is determined. And the result showed that all the vehicles after the 4^{th} vehicles are under the saturation condition and the first four vehicles faced start up lost time. See **Appendix C** for detail analysis of estimation of Queue position for saturation Headway and start up lost time.

4.2. Start Up Lost Time Data

1133 start-up lost time data computed by deducting the first four vehicles discharge headway from the saturation headway of the vehicles. According to this Equation;

$$\text{SULT} = \sum_{i=1}^{4} hi - hs \text{ -- } \textbf{Eqn-4.1}$$

Whereas, SULT is the Start Up Lost Time, hi is the discharge headway of the i^{th} vehicle, hs is the saturation headway of vehicles and i is the position of vehicle. The tabular and graphic descriptive Statistics of the computed start up lost time data is presented herein below.

Table 4.4: Overall investigated sites start-up lost time data

Variable	Total	Mean	StDev	Minimum	Median	Maximum
Start Up Lost Time	1133	2.5128	0.6137	1.1440	2.4863	5.2071

This indicates that start up lost time of overall observed sites mean value is 2.5128 sec per phase and the same it is necessary to check this value variability with HCM recommendation is necessary. To conduct, Analysis of variance with HCM 2010 recommendation t test was conducted and the result shown herein below.

T – test for checking the observed result variability with HCM 2010 recommendation

N	Mean	StDev	SE Mean	95% CI for μ
1133	2.5128	0.6137	0.0182	(2.4770, 2.5486)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2$
Alternative hypothesis	H_1: $\mu \neq 2$

T-Value	P-Value
28.13	0.000

As indicated in the t –test, the observed result in this study is significantly different with HCM recommendation due to P-value of t – test is less than 0.05. However, the HCM recommendation 2 Sec per phase is recommended for the ideal condition of intersections while the selected intersection in this study are multi-complex to represent the real scenario and to test the variability of start- up lost time it is necessary to select the intersections that seems to the ideal condition and full fill the following condition,

- ✓ No heavy vehicle (Homogenous traffic)
- ✓ Approach grade is flat
- ✓ No parking in the near to the intersection

In order to fulfill the above ideal condition criteria's, the intersections with vehicular composition A (all the vehicles in the queue are car) and the intersection approach grades less than 1% grade and greater than -1% grade are selected for comparing study value with HCM 2010 recommendation. For an ideal intersection Sebeta – Lebu approach at Lebu Signalized Intersection with -0.5% grade and Mexico – Michael approach at Jemo Micheal signalized intersection with -0.8% grade are selected. The following table shows the analysis of the result.

Table 4.5: Ideal Intersection Start Up Lost Time

Variable	Vehicular Composition	Approach Grade	N	Mean	St.Dev	Minimum	Median	Maximum
Start Up Lost Time	A (all the vehicles are car)	-0.8% & -0.5%	97	2.37305	0.2378	1.7507	2.4006	2.7676

t- test was conducted the above result variability with HCM 2010 recommendation

N	Mean	StDev	SE Mean	95% CI for μ
97	2.3730	0.2378	0.0241	(2.3251, 2.4209)

μ: mean of Sample

Test

Null hypothesis H_0: $\mu = 2$ sec as per HCM recommendation

Alternative hypothesis H_1: $\mu \neq 2$

T-Value	P-Value
15.45	0.000

The t-test shows that the P-value which is less than 0.05 indicates that the null hypothesis will be rejected and the same it shows that the recommended value of start-up lost time as per HCM 2 Sec per phase is significantly different from the study value for an ideal intersections. Hence, it is necessary to calibrate and customizing the default values HCM recommendation prior to use in the design of signals and performance evaluation of signalized intersections.

Table 4.6: Start Up Lost Time data for each vehicular Composition

Start Up Lost Time						
Vehicular Composition	**Total**	**Mean**	**StDev**	**Minimum**	**Median**	**Maximum**
A	634	2.1542	0.4096	1.1440	2.1629	2.9632
B1	145	2.5689	0.3296	1.8970	2.5573	3.3101
B2	149	3.1087	0.3339	2.3267	3.0970	4.0609
B3	17	3.6340	0.3641	3.1538	3.5938	4.5095
C1	57	2.5569	0.3473	1.8326	2.5330	3.5152
C2	76	3.1972	0.2757	2.5560	3.2116	3.8560
C3	9	4.187	0.324	3.567	4.267	4.582

D1	12	2.8976	0.2102	2.5670	2.9588	3.2083
D2	24	3.2839	0.3269	2.6090	3.2362	4.0451
D3	8	4.186	0.393	3.727	4.077	4.792
E	2	5.010	0.279	4.812	5.010	5.207

As shown from table 4.6, it indicates that when the composition of heavy and/or large vehicles in the first four vehicles increases or the number of heavy vehicles increases start up lost time will become increased significantly. From the table Vehicle Composition E has a large Start up lost time and Vehicle Composition A has lowest Start Up lost time compared to the others which implies when all the first four vehicles are car start up lost time will become low while when all the first four vehicles are either Large Bus and Large Truck or Truck Trailer or Articulated Bus Start Up lost time will become increases which means that start up lost time is depend on the size of the vehicles. Figure 4.3 shows a box plot of start-up lost time with different vehicle composition and movement.

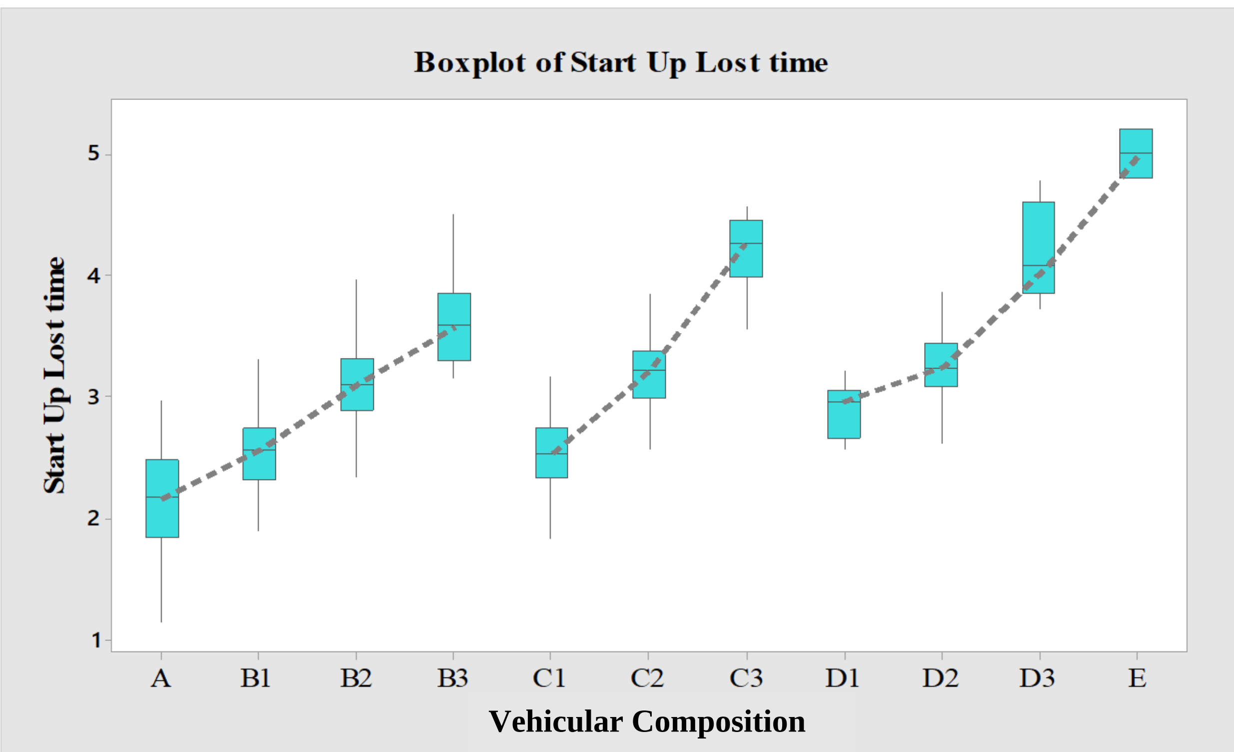


Figure 4.3: Box Plot of Start Up Lost Time Data

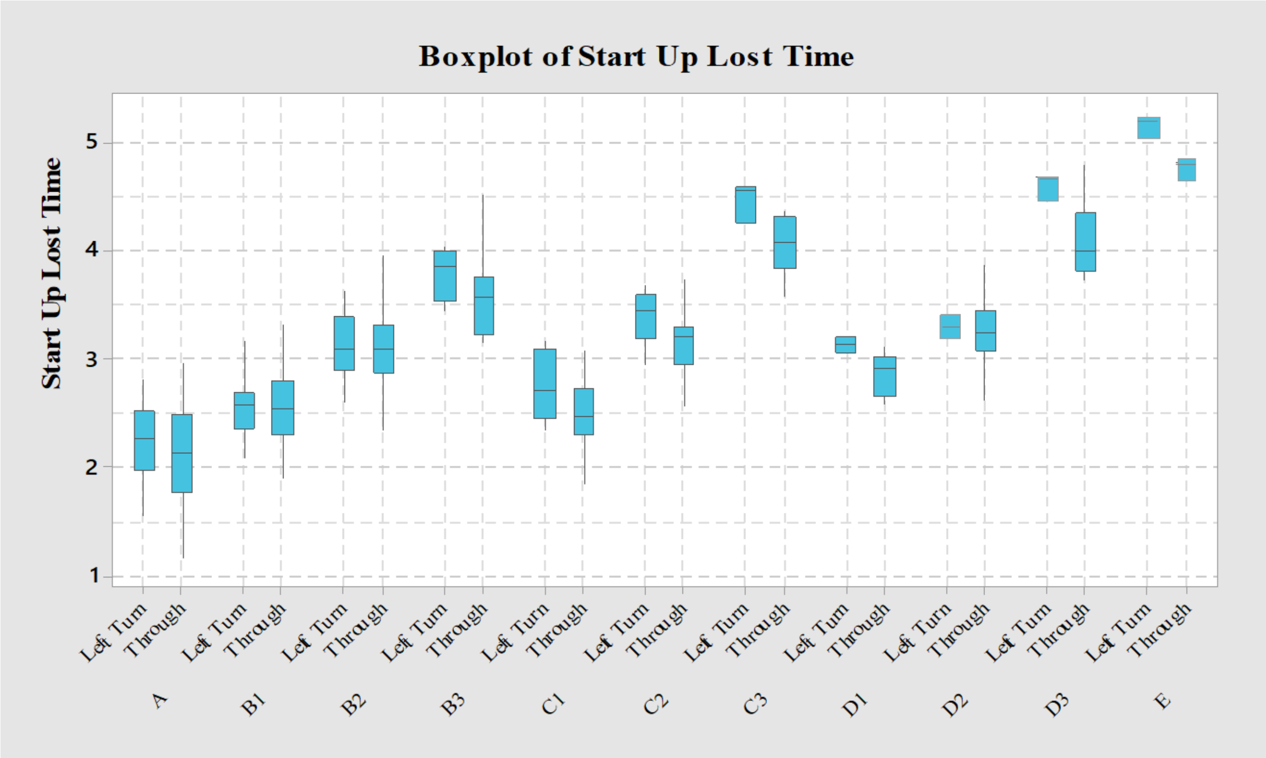


Figure 4.4: Box Plot of Start Up Lost Time Data with Vehicle Composition and Movement

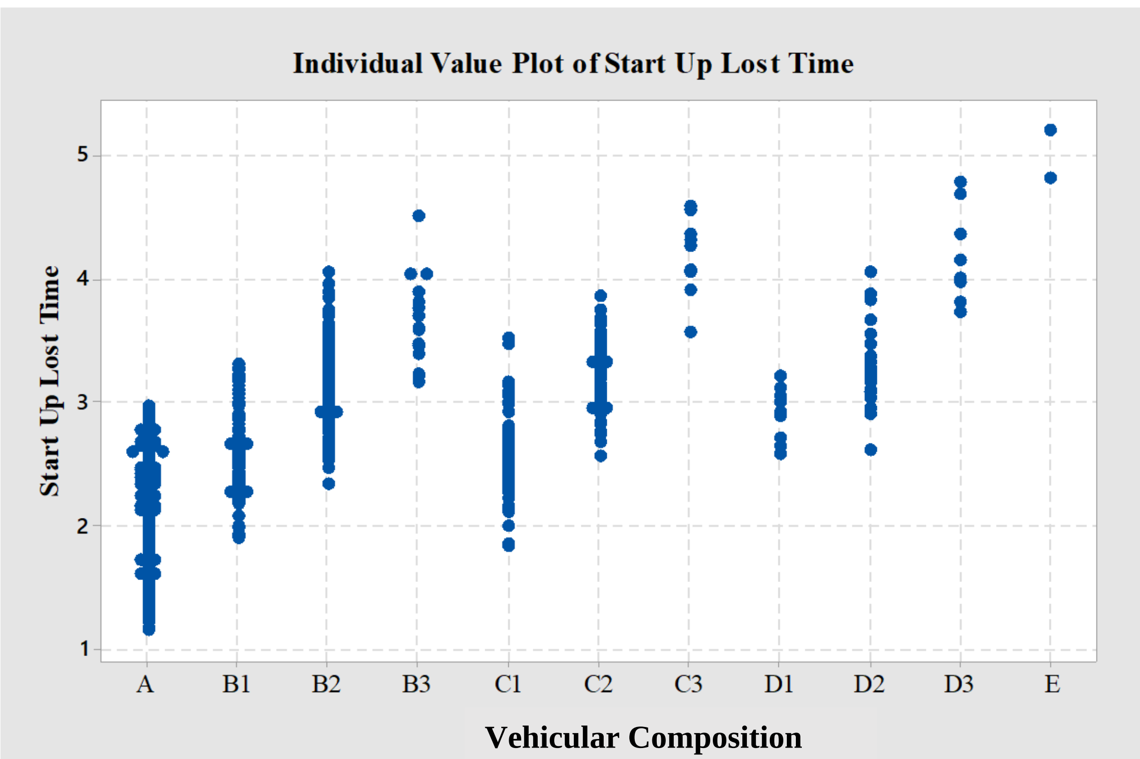


Figure 4.5: Individual Value Plot of Start Up Lost Time

Figure 4.3 indicates that, the increase in percentage of heavy vehicle composition will have a large start up lost time value. Hence, it directs that when the percentage of large vehicles increase or the percentage of Large Bus, Large Truck, Truck Trailer and Articulated Bus increases then the start-up lost time will become increased.

The same to the above, figure 4.4 indicates that, start up lost time value will deviate with the movement type of different vehicle composition and the same left turn movement has larger start up lost time value than through movement of each vehicle composition which tells us that maneuvering in left turn is challenging in acceleration of the vehicles and the same the start-up lost time value is increased if the mix of heavy vehicles increased.

Figure 4.5 entails that, the individual value plot provides an increase of start-up lost time value with the composition of large vehicles percentage increases as the same to Figure 4.3 and Figure 4.4. The individual value plot of start-up lost time value shows high deviation in the Vehicle Composition A, B2, B3 and C1 compared to the others vehicle composition which is the same to the standard deviation of start-up lost time indicated in table 4.6.

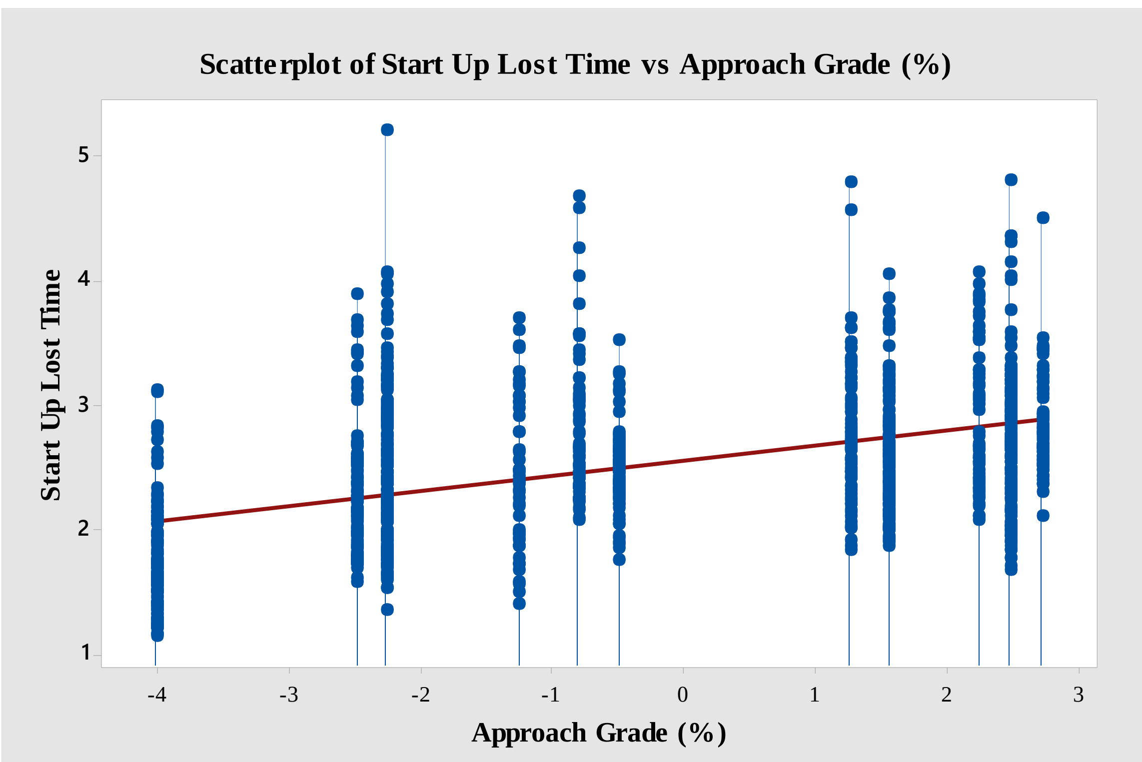


Figure 4.6: Scatter Plot of Approach Grade Vs Start Up Lost Time

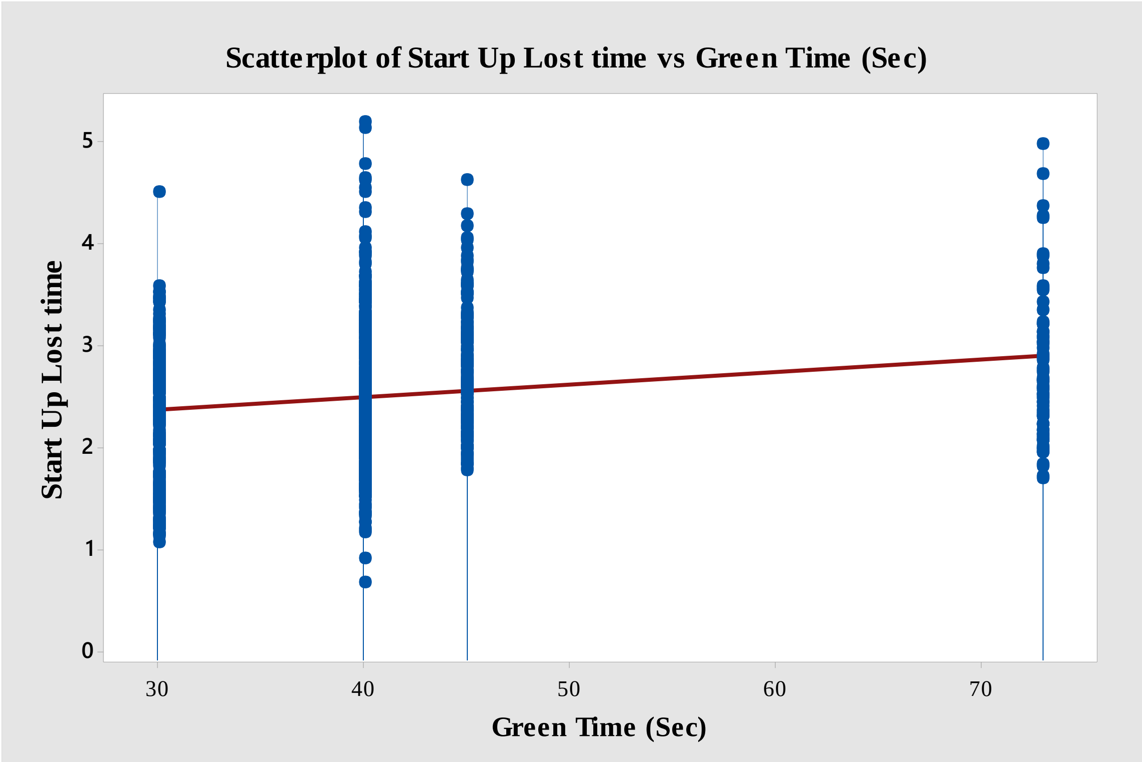


Figure 4.7: Scatter Plot of Start Up Lost Time Vs Green Time

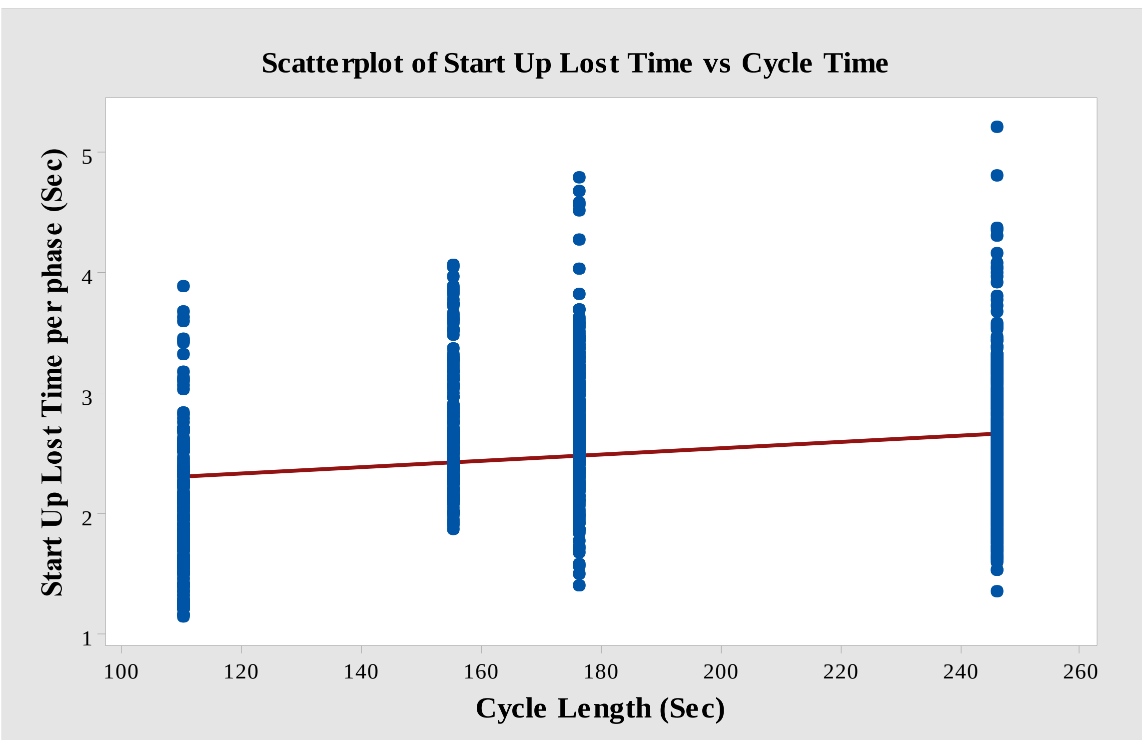


Figure 4.8: Scatter Plot of Start Up Lost Time Vs Cycle Length

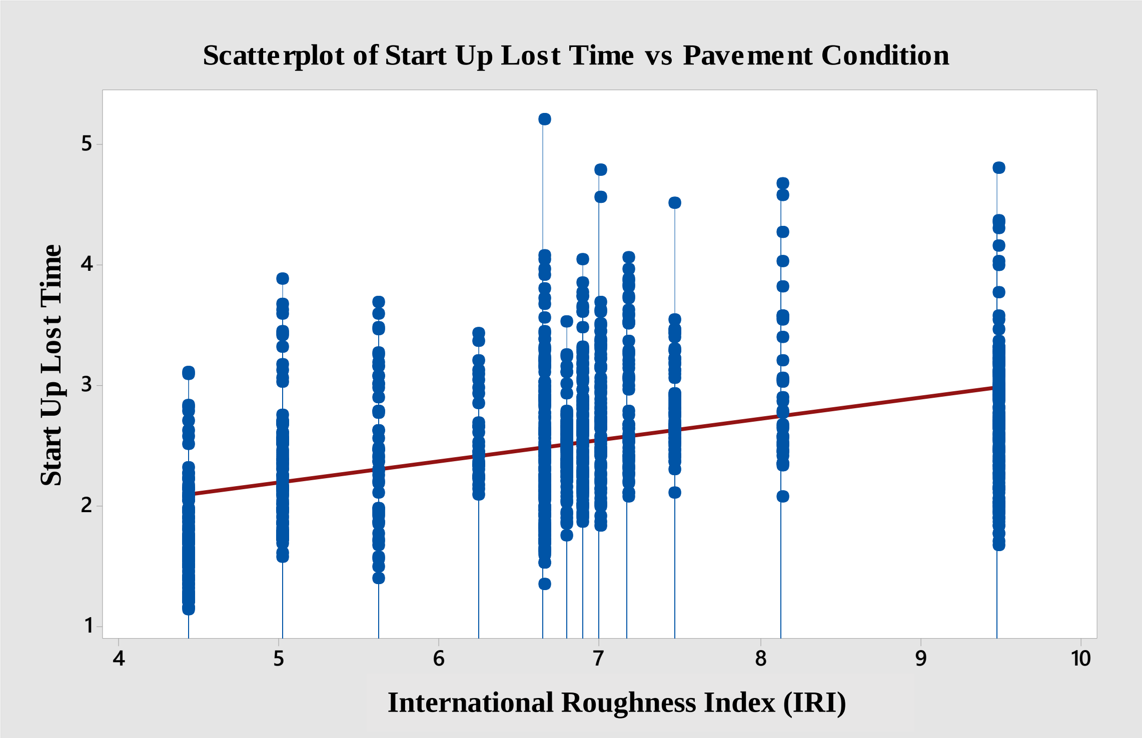


Figure 4.9: Scatter Plot of Start Up Lost Time Vs Pavement Condition

On figure 4.6, when Approach Grade increases the Vehicles Start Up lost time will increase and in downgrades the startup lost time value decreases significantly which means that the vehicles accelerate when approaching a downgrade intersection.

As shown in figure 4.7 and 4.8, start-up lost time value will increase for longer cycle time and green time. This indicates that the drivers feel depressed during long cycle length and their response for the indication of green time during long cycle time will become larger due to this start up lost time is increased for longer cycle time and green time.

As indicated in figure 4.9, Start-up Lost Time value becomes increased when the international roughness index increases. As cognizant that, International Roughness Index is a measure of the roughness condition of the pavement when IRI value is high the pavement is under poor condition and it has poor riding quality while when IRI value is low the pavement is under good riding quality. Based on this interpretation, when the intersecting approach road of the pavement is under poor surface and poor riding quality then the start-up lost time value is increased due to the fact that the vehicles cannot accelerate and the same when IRI value is low the start-up lost time value is decreased which means that the vehicles accelerate due to good pavement condition.

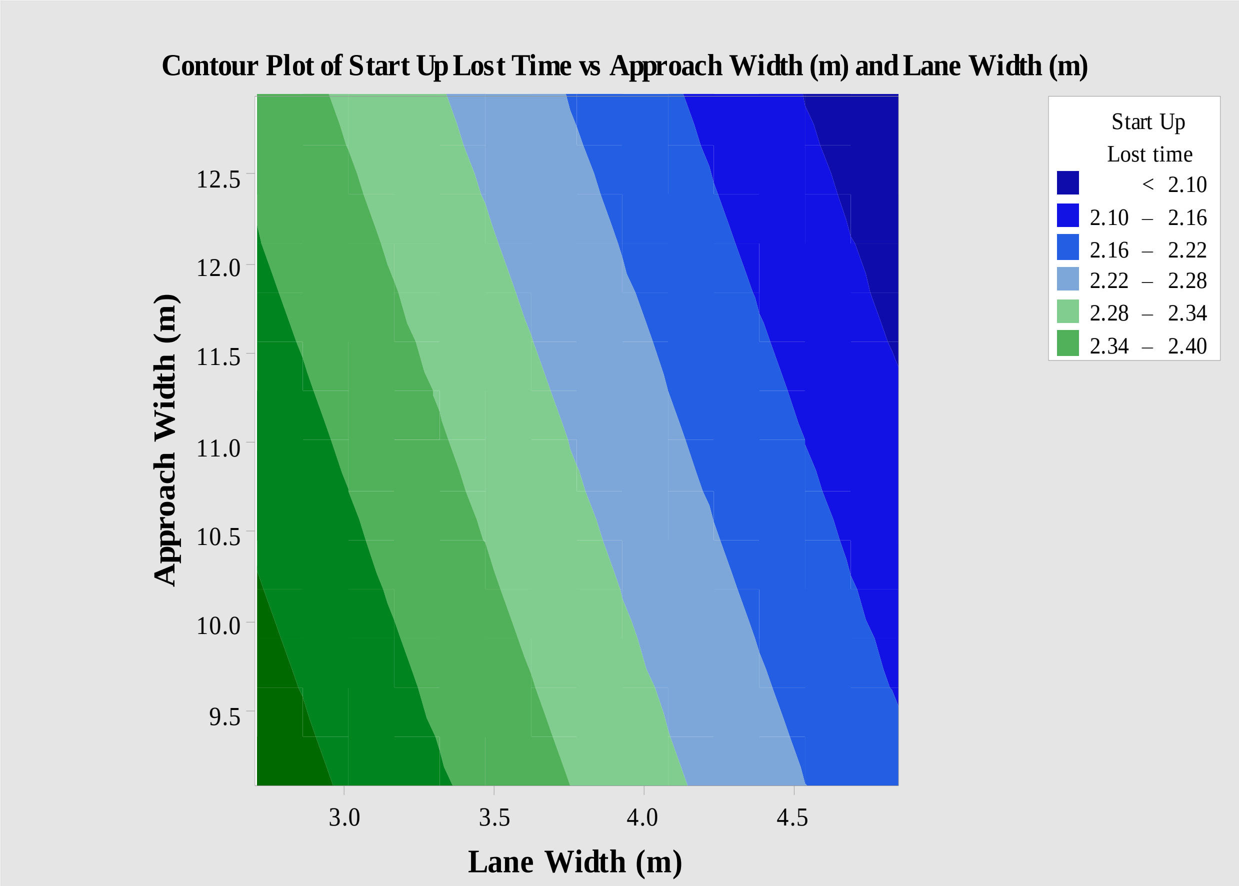


Figure 4.10: Contour Plot of Start Up Lost Time Vs Lane Width and Approach Width

As illustrated in figure 4.10, as approach width and lane width of the lane increases then start up lost time will become lower and when approach width and lane width decrease then start up lost time value will increase. From this, it is understood that when the lane width and approach width is wider the drivers feel safer and they drive as they want while if lane width and approach width becomes narrower the drivers will be guided by the narrower lane and start up lost time becomes larger. The indicated lane width on the graph is taken from actual measurement of the sites. The deviation of the existing sites lane width from standard lane width is managed by dividing the existing lane width with lane marking in order to avoid uneconomical use of such excessive lane widths.

4.3. Start Up Lost Time Distribution Modeling

Distribution modeling of start-up lost time data is conducted with minitab18 statistical software. Due to a heterogeneous mix of traffic, it is understandable that one distribution model for start-up lost time will not fit. To this effect, distribution modeling is conducted for each vehicle composition by testing the goodness of fit test through Anderson Darling Test. First conducting Normality test of the data through Anderson Darling Test with the following Hypothesis.

Ho: data values come from specified probability distribution

H1: data values do not come from specified probability distribution with Significance level 0.05

4.3.1. Distribution Fitting Start Up lost time with vehicle composition

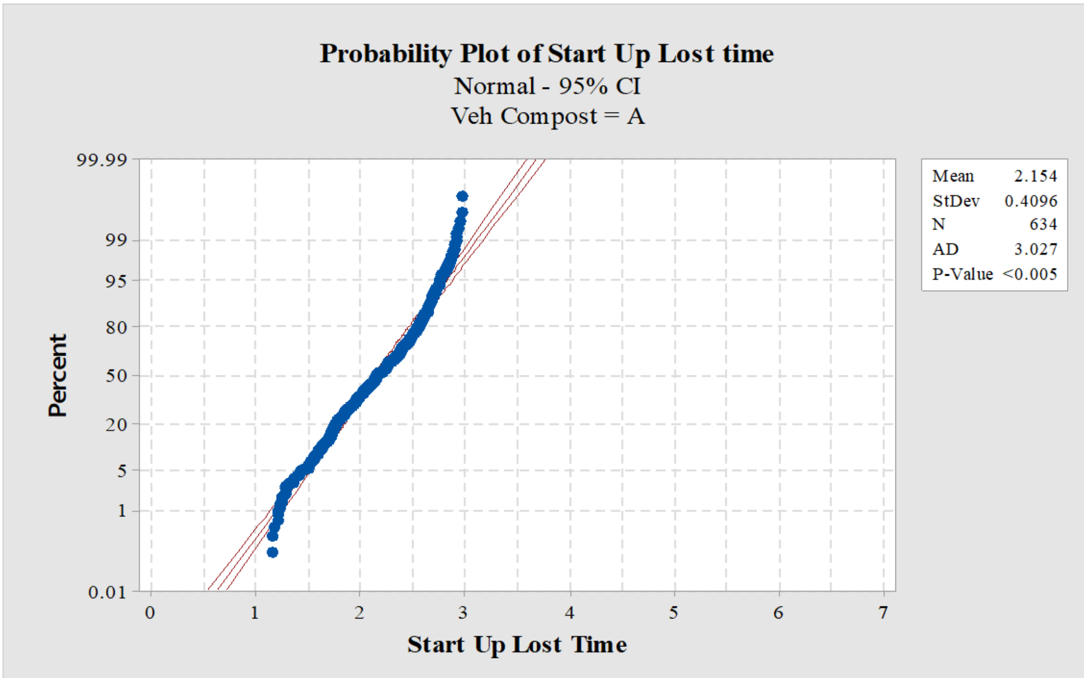


Figure 4.11: Normal Probability plot of Vehicular Composition A (all the 1st four vehicles car)

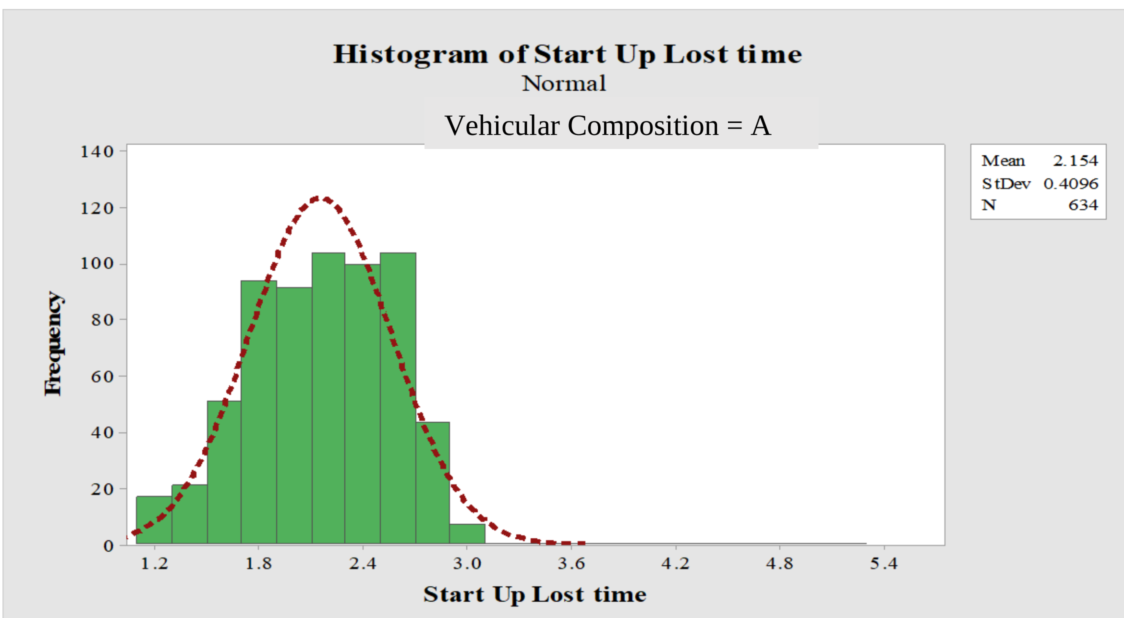


Figure 4.12: Histogram of Vehicular Composition A (all first four vehicles are car) with Normal Curve

As seen from figure 4.11 and 4.12, Start-Up lost time for vehicular compositions A (all the first four vehicles are car) Probability distribution have large data values depart from the normality line and also the fit curve of the histogram is right skewed as shown in Figure 4.11. while, in order to make sure the data significantly departs from normality line, it is necessary to check the P- Value and Anderson- Darling Value. The P-Value from Anderson Darling Test is <0.005 which leads to accept the alternative hypothesis which states that the data doesn't come from normal distribution.

Subsequently, the data does not come from normal distribution, it is mandatory to find the probability distribution that fits the data. To conduct this, it is necessary to perform either Non normal Probability distribution or Normal Transformed distribution by certain function. All possible distribution are tested for the start-up lost time data for Vehicular Composition A (all the first four vehicles are car). Johnson Transformation with the equation below fits the Probability distribution of Vehicular Composition A (all the first four vehicles are car). To determine the best fit the distribution the higher P-value and the smaller AD Value are selected. All non-normal distributions are tested and none of them not fit the distribution. If certain distribution best fits the data, the data points need to closer the straight line than other probability distributions.

To this effect, Johnson Transformation with the equation below shows the best fit for Vehicular Composition A.

Johnson Transformation Function to fit the Non-Normal Data is shown herein below

Transformation Function = $-0.376 + 1.066 \times \text{Ln}((X - 0.946) / (3.057 - X))$ ……….**Eqn 4.2**

Where: X is original start up lost time data

Probability Plot of Start Up Lost Time of Vehicular Composition A
Normal – 95% CI, AD: 0.451, P: 0.273

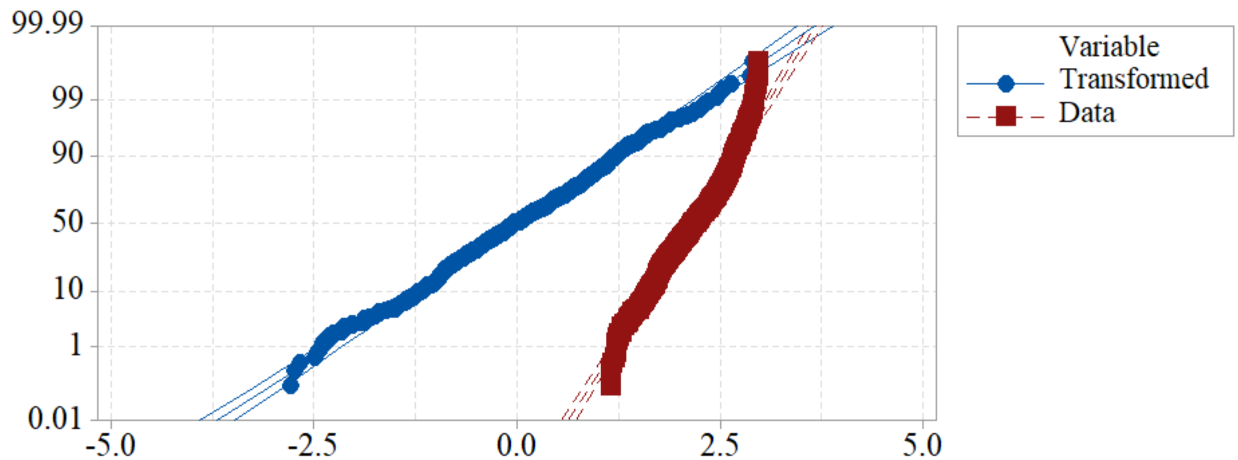


Figure 4.13: Johnson Transformation Probability Plot Start up Lost Time of Vehicular Composition A

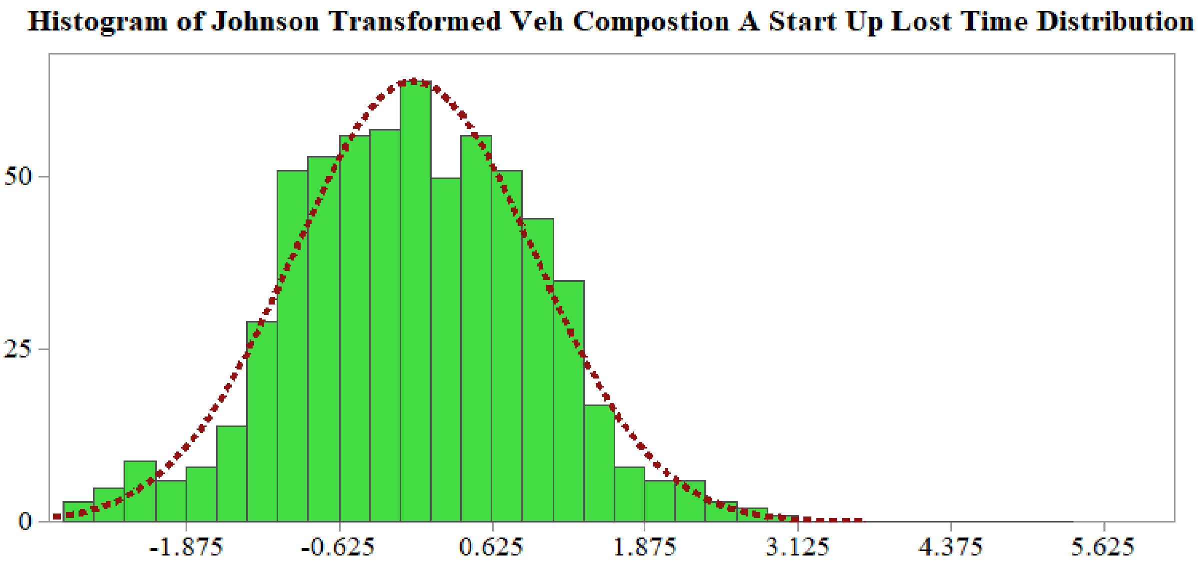


Figure 4.14: Histogram of Start Up Lost Time Distribution of Vehicular Composition A after Johnson Transformation

As shown in figure 4.13 and 4.14, for vehicular composition A all the possible probability distribution is tested and Johnson Transformed function through the above equation fits the probability distribution with P value of 0.273 and Anderson Darling Value of 0.451.

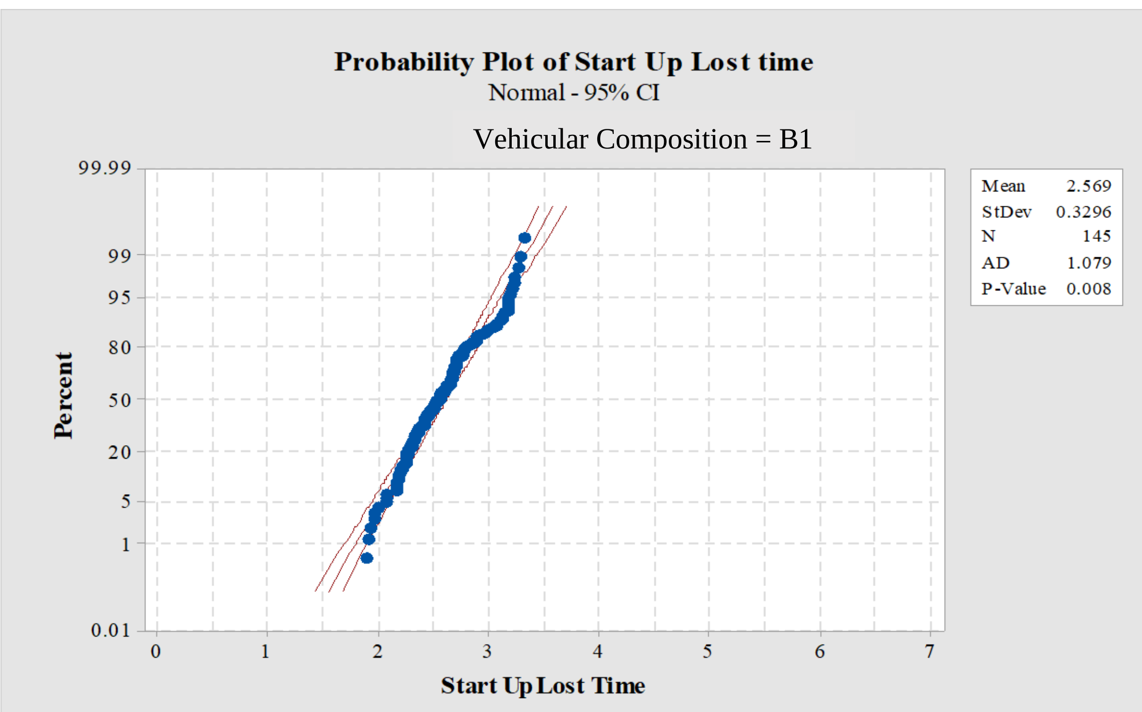


Figure 4.15: Probability Distribution Plot of Start Up Lost Time for Vehicular Composition B1 (Three of the first vehicles are car and the other one is either Small Bus or Small Truck)

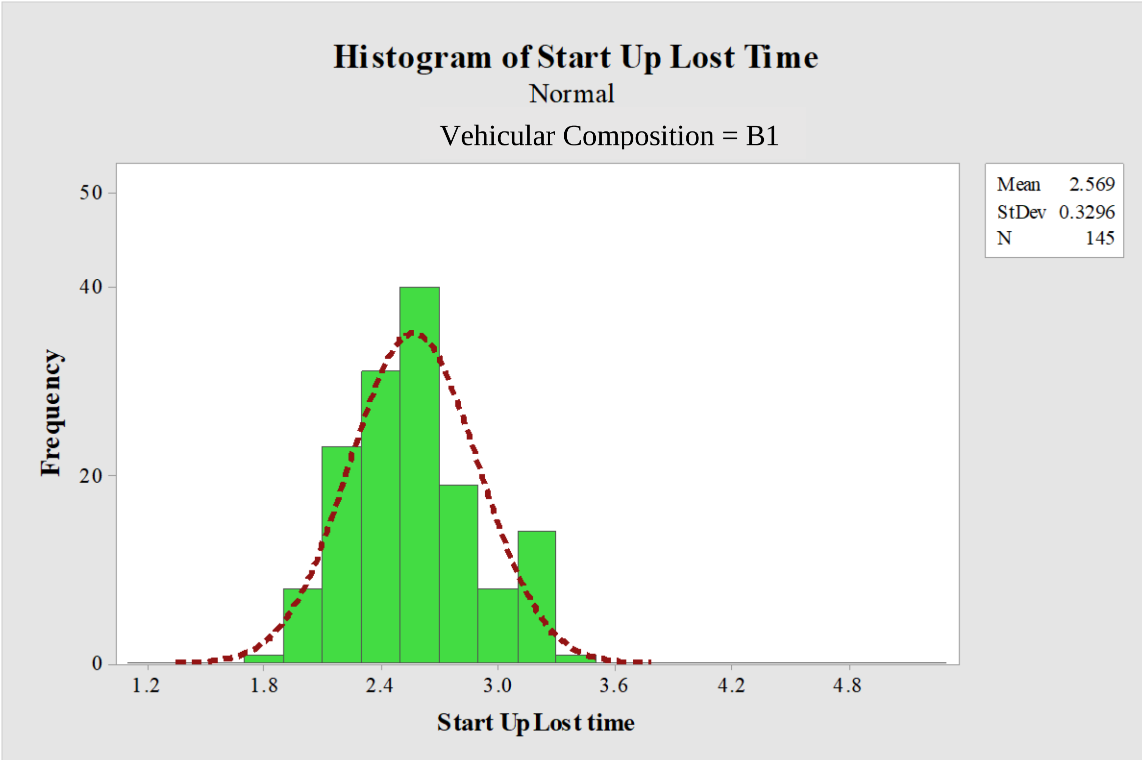


Figure 4.16: Histogram of Start Up Lost Time for Vehicular Composition B1

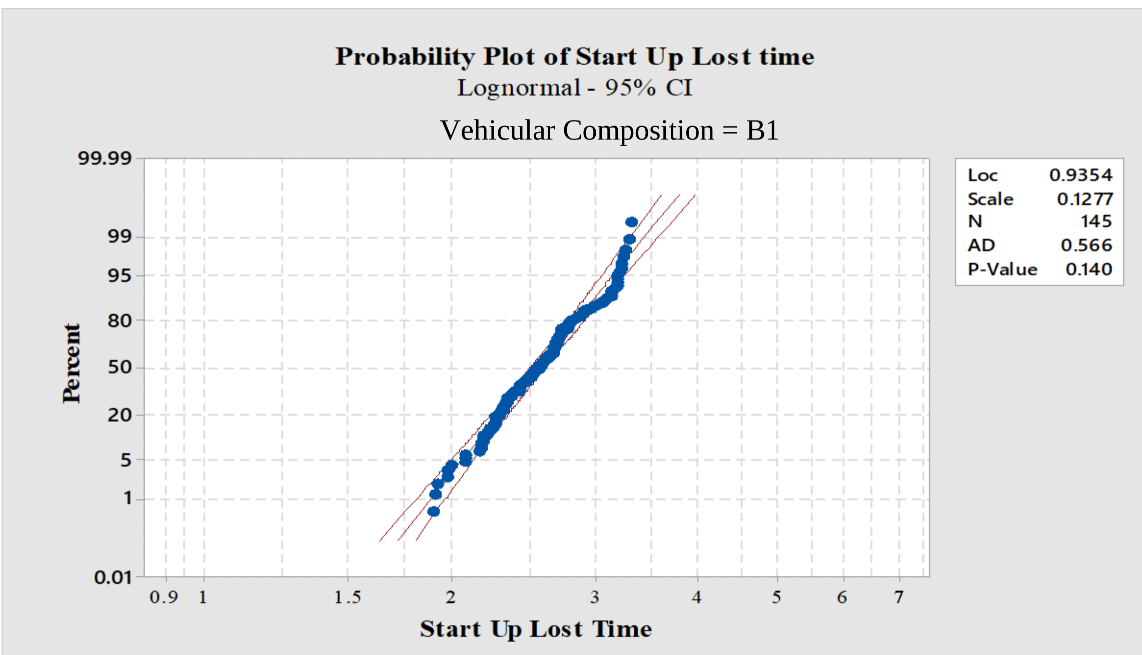


Figure 4.17: Lognormal Probability Distribution Plot of Start Up Lost Time for Vehicular Composition B1

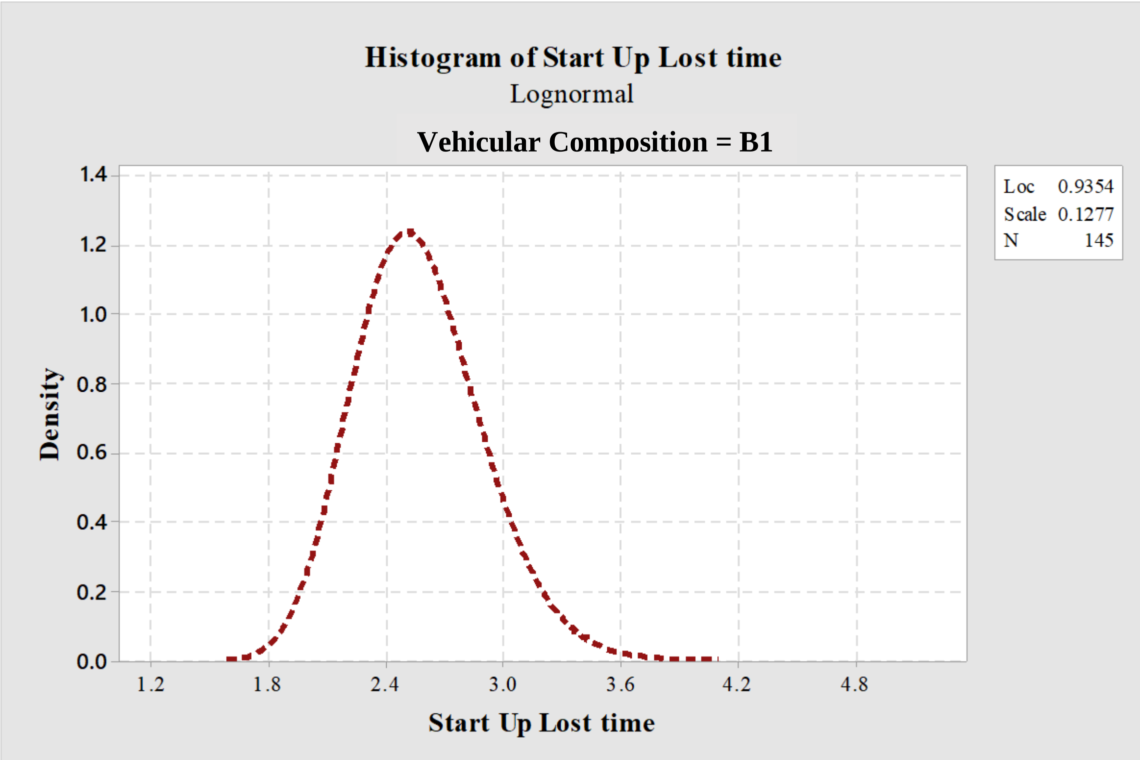


Figure 4.18: Lognormal Histogram of Start Up Lost Time for Vehicular Composition B1

As shown in figure 4.15 and 4.16, Vehicular Composition B1 also shows deviation from the normal distribution as the same to Vehicular composition A. As illustrated in figure 4.15, P- value for normal distribution of Vehicular Composition B1 is 0.008 which is less than 0.05 and it implies that the data not come from normal distribution and the same the Histogram of the data values is also left skewed from the normal distribution plot. Hence, it is necessary to find the best fit probability distribution by conducting non normal probability distribution of the data for vehicular Composition B1 (three of the 1st four vehicles are car and the other one is either Small Bus or Small Truck). As indicated in figure 4.17 and 4.18, Lognormal Probability distribution with the estimated parameters below fits for vehicular Composition B1.

Table 4.7: Estimated Parameters and AD Value for Vehicular Composition B1

Distribution	Estimated Parameters		Anderson – Darling Test Result	
	Location	Scale	AD	P
Lognormal	0.9354	0.1277	0.566	0.140

The table below demonstrates fitted probability distribution for all composition of Vehicles along with the estimated parameters and AD value. For detailed analysis of fitting distributions for nine composition of Vehicular types see Appendix- E.

Table 4.8: Estimated Parameters and AD Value for all Vehicular Compositions

Vehicular Composition	Distribution	Estimated Parameters			Anderson – Darling Test Result	
		Location	Shape	Scale	AD	P
A	Johnson Transformed	-	-	-	0.451	0.273
B1	Lognormal	0.9354		0.1277	0.566	0.140
B2	Lognormal	1.129		0.1069	0.188	0.902
B3	Lognormal	1.286		0.09819	0.246	0.714
C1	Lognormal	0.9299		0.1346	0.246	0.748
C2	Lognormal	1.159		0.0868	0.229	0.802
C3	Weibull	-	17.00	4.323	0.212	>0.250
D1	Weibull	-	17.36	2.989	0.350	>0.250
D2	Lognormal	1.185		0.0985	0.425	0.291
D3	Lognormal	1.428		0.0924	0.266	0.580
E	Lognormal	1.611		0.0557	0.250	0.227

4.4. Start Up Lost Time Regression Modeling

Multiple Linear Regression Modeling is used to develop a mathematical model for the Start Up Lost Time data of different vehicular compositions of vehicles. Accordingly, Multiple Linear Regression Model is developed for each Categorical Predictors through Minitab 18 Statistical software after testing all the assumptions of Multiple Linear Regression Model as presented herein Below and a total of 1133 start-up lost time data are used for regression modeling.

4.4.1. Examining Multiple Linear Regression Assumptions

To conduct Multiple linear regression, it is mandatory examining of the major assumption before conducting any regression analysis to avoid misleading of the result due to the violation of one or more assumption.

- ✓ Assumption 1: - Linear Relationship between dependent variable and each of independent variable

There should be linear relationship between dependent and independent variable. Scatter Plot of dependent variable versus independent variable expresses linear relationship between variables. According to Figure 4.6,4.7,4.8 and 4.9 there is no significant evidence for the existence of non-linear relationship between the dependent and independent variables.

- ✓ Assumption 2: - Multicollinearity (the predictor variables should not highly correlated with each other)

If the independent variables are highly correlated it gives misleading result even if R^2 adjusted is significant. Pearson's bivariate correlations is conducted among all independent variables; the magnitude of the correlation coefficients should be less than 0.8. According to this, all the predictor variables have less than 0.8 and Pearson correlation among them as shown in the table 4.9. Further, Variance inflation factor basically tells us if there is inflation or an increase in standard error. If the VIF is greater than 10 then there is Multicollinearity. As we can see in the table 4.10 all the Variance inflation factors corresponding to each explanatory variables is less than 10 so it can be concluded that there is no a Multicollinearity problem. (See Table 4.9 and 4.10).

Table 4.9: Correlation Matrix between Independent Variables

Variables	**Green Time (Sec)**	**Approach Width (m)**	**Lane Width (m)**	**Approach Grade (%)**	**Saturation Headway (sec)**
Green Time	1				
Approach Width (m)	0.419	1			
Lane Width (m)	0.024	0.228	1		
Approach Grade (%)	0.307	0.440	0.475	1	
Saturation Headway (sec)	-0.125	-0.092	0.302	0.119	1

Table 4.10: Predictor Variables Variance Inflation Factor

Variable	**Green Time (Sec)**	**Approach Width (m)**	**Lane Width (m)**	**Approach Grade (%)**	**IRI**	**Through Movement**	**Vehicular Composition**	**Saturation Headway (sec)**
VIF	1.79	2.46	1.56	3.44	4.47	1.24	1.064	1.18

- ✓ Assumption 3: - Homoscedasticity

The data is Homoscedasticity, when the variance along the line of best fit remains the same along the best fit line. If our data shows homoscedasticity, the residuals exhibit an overall fairly random display. The scatter plot of fitted value versus residuals exhibit an overall fairly random display as shown in the Figure 4.19. Hence, it can conclude that there is no heteroscedasticity and the model shows Homoscedasticity.

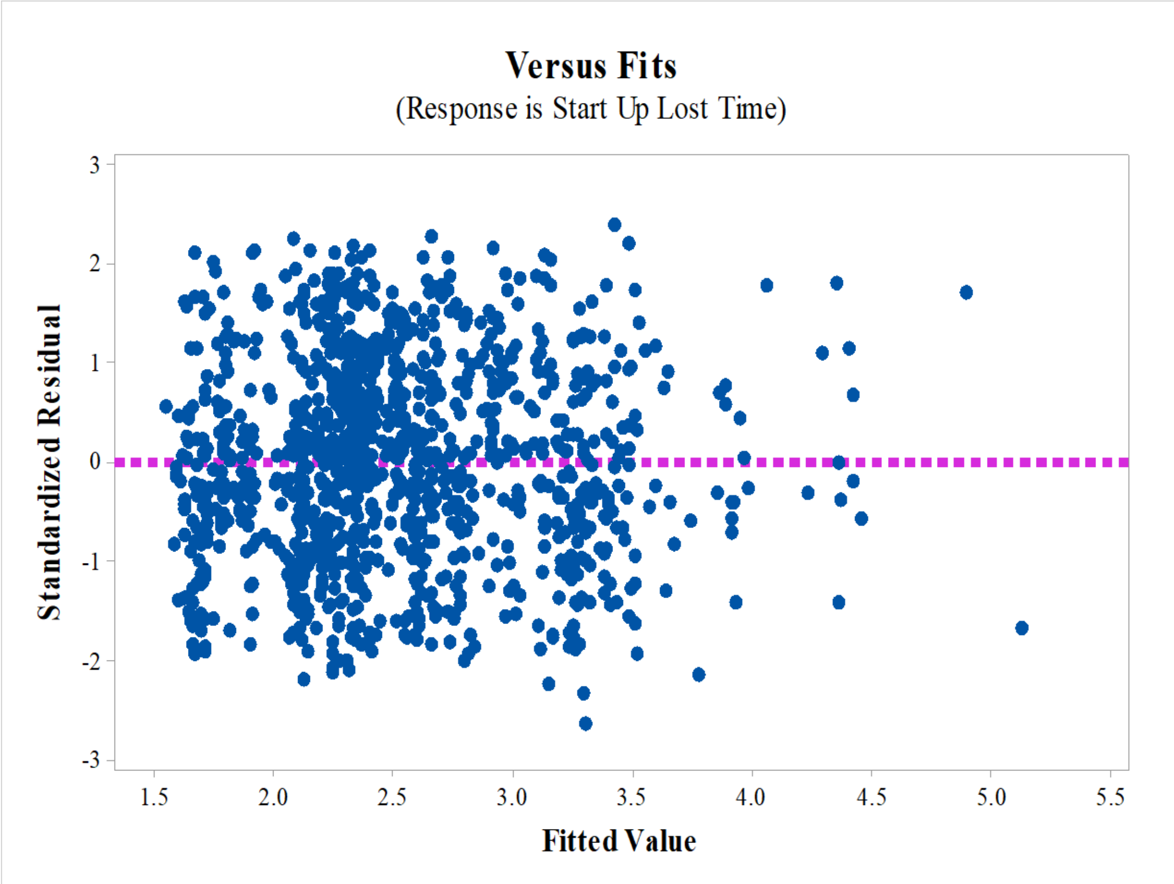


Figure 4.19: Scatter Plot of Standardized Residual Versus Fitted Values

Assumption 4:- no significant outlier, high leverage point or highly influential points, which represent observation in the data set.

According to this assumption, the data points with high leverage points, highly influential points and the data points with significant outlier are removed from the analysis and the same the data points with unusual observation also removed.

Assumption 5:- Residuals are normally distributed

The residuals (Errors) should be approximately normally distributed. Histogram plot of the residuals with super imposed normal curve has a bell shaped structure which shows that the residuals are normally distributed.

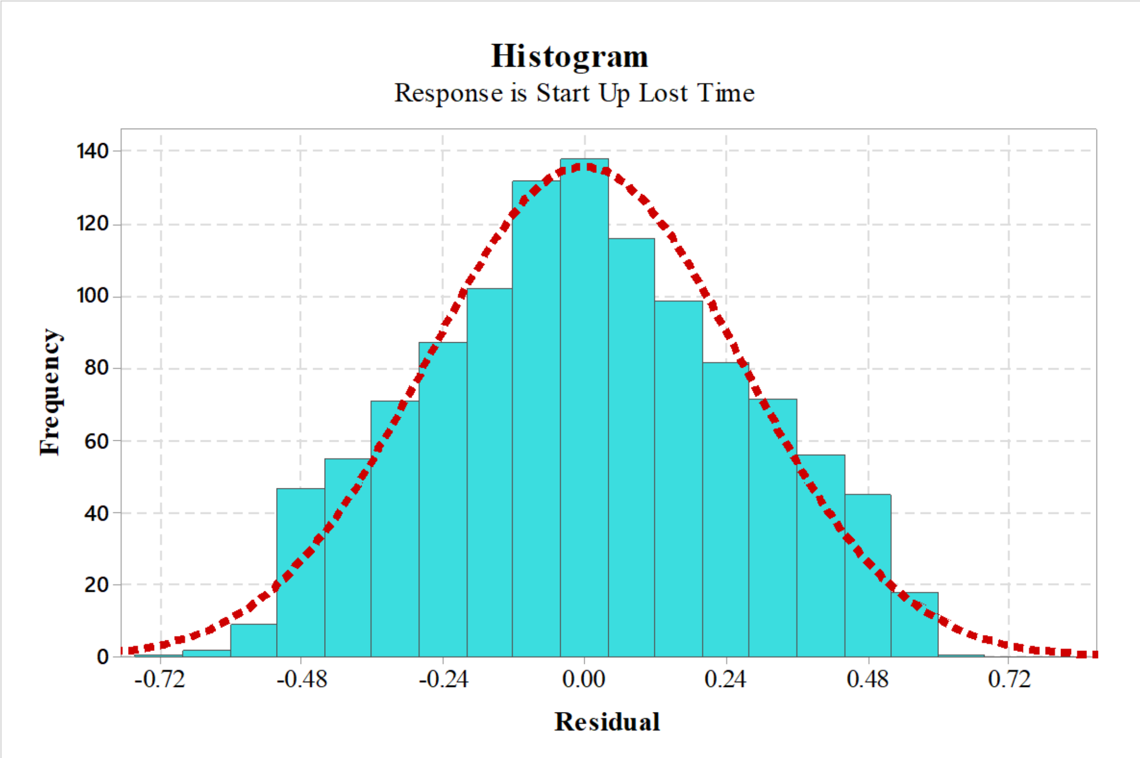


Figure 4.20: Histogram of Residuals with a Normal Curve

4.4.2 Regression Analysis

A stepwise multiple linear regression was performed to model the Start Up Lost Time value with the following independent variables. The independent variables consist continuous and categorical variables. Approach Grade, Lane Width, Approach Width, Saturation Headway, Green Time and International Roughness Index (IRI) are Continuous predictors. Vehicular Composition with eleven categories as indicated in section 3.2.3, Type of Movement has Two Categories (Through and Left Turn) and position of lane are the Categorical Predictors.

Dependent Variable

- ✓ Start Up Lost Time

Predictor Variables

- ✓ Approach Grade in %
- ✓ Approach Width in meter
- ✓ Lane Width in meter
- ✓ Green Time in seconds
- ✓ Saturation Headway in seconds
- ✓ Pavement Condition in International Roughness Index (IRI)
- ✓ Lane position (Inner, Middle and Outside)

- ✓ Movement (Left Turn and Through)
- ✓ Vehicular Composition (A, B1, B2, B3, C1, C2, C3, D1, D2, D2, D3 and E as indicated in table 4.11

Vehicular Composition, Lane Position and Type of Movement are categorical predictors and dummy coded in the regression analysis with the following base case.

Vehicle Composition: - Base Category is Vehicular Composition A (all the first four vehicles are car)

Lane position for the movement: - Inner (Near Median)

Type of Movement: - Through Movement

Stepwise Multiple linear regression analysis is conducted for determination of the parameters through stepwise way. Only one variable is added at each step of the regression. As shown in the table below p-values for each added variables is less than the significance level which confirms that all predictor variables significantly predict the startup lost time and there is an increment in the adjusted r squared value at each step of the regression.

Table 4.11: Stepwise Regression

	Step 1		Step 2		Step 3		Step 4		Step 5		Step 6		Step 7		Step 8	
	Coef	*P*	*Coef*	*P*	*Coef*	*P*	*Coef*	*P*	*Coef*	*P*	*Coef*	*P*	*Coef*	*P*	*Coef*	*P*
Constant	*2.154*		*2.213*		*2.141*		*2.888*		*3.238*		*3.445*		*3.314*		*3.314*	
Vehicular Composition	*2.855*	*0.00*	*2.786*	*0.00*	*2.743*	*0.00*	*2.776*	*0.00*	*2.831*	*0.00*	*2.86*	*0.00*	*2.873*	*0.00*	*2.866*	*0.00*
Approach Grade (%)			*0.094*	*0.00*	*0.105*	*0.00*	*0.132*	*0.00*	*0.139*	*0.00*	*0.13944*	*0.00*	*0.137*	*0.00*	*0.139*	*0.00*
Movement					*0.227*	*0.00*	*0.269*	*0.00*	*0.251*	*0.00*	*0.2461*	*0.00*	*0.2345*	*0.00*	*0.2595*	*0.00*
Lane Width (m)							*-0.224*	*0.00*	*-0.221*	*0.00*	*-0.2010*	*0.00*	*-0.188*	*0.00*	*-0.1978*	*0.00*
Approach Width (m)									*-0.0317*	*0.00*	*-0.03725*	*0.00*	*-0.053*	*0.00*	*-0.05036*	*0.00*
Saturation Headway (sec)											*-0.0998*	*0.00*	*-0.0967*	*0.00*	*-0.1007*	*0.00*
Green Time (sec)													*0.0068*	*0.00*	*0.00625*	*0.001*
Lane Position															*0.0537*	*0.016*
R^2	*62.96%*		*74.77%*		*77.37%*		*80.20%*		*80.63%*		*80.88%*		*81.73%*		*81.23%*	
R^2 *Adj*	*62.63%*		*74.53%*		*76.12%*		*79.97%*		*80.38%*		*80.63%*		*81.46%*		*80.93%*	

As shown in table 4.11, all the predictors found statistically significant to predict the start-up lost time with P value < 0.05 and R^2 Adj = 80.93% and the final Regression Analysis was run to predict Start Up Lost Time from Vehicular Composition, Approach Grade, Type of Movement, Lane Width, lane position (inner, middle and outside), Saturation Headway, Approach Width and Green Time.

Table 4.12: Final Regression Results

Term	Coef	SE Coef	T-Value	P-Value	VIF
Constant	3.314	0.112	29.47	0.000	
Saturation Headway	-0.1007	0.0258	-3.90	0.000	1.20
Green Time (Sec)	0.00625	0.00194	3.22	0.001	1.78
Approach Width (m)	-0.05036	0.00792	-6.36	0.000	2.18
Lane Width (m)	-0.1978	0.0187	-10.58	0.000	1.59
Approach Grade (%)	0.13906	0.00459	30.27	0.000	1.74
Vehicular Composition					
B1	0.4184	0.0250	16.74	0.000	1.10
B2	0.9040	0.0249	36.30	0.000	1.12
B3	1.5420	0.0664	23.23	0.000	1.03
C1	0.4462	0.0377	11.83	0.000	1.07
C2	1.0598	0.0337	31.47	0.000	1.12
C3	2.0935	0.0904	23.15	0.000	1.02
D1	0.7291	0.0787	9.26	0.000	1.02
D2	1.0949	0.0569	19.24	0.000	1.06
D3	2.1324	0.0960	22.20	0.000	1.02
E	2.866	0.190	15.05	0.000	1.01
Lane Position					
middle	0.0537	0.0197	2.72	0.007	1.41
outside	0.0468	0.0236	1.98	0.048	1.29
Movement					
Left Turn	0.2595	0.0212	12.24	0.000	1.47

Regression Equation

SULT = 3.314 - 0.1007*hs + 0.00625*GT- 0.05036*AW - 0.1978*LW+ 0.13906*AG + 0.4184 *B1+ 0.9040 *B2+ 1.5420*B3+ 0.4462 *C1 + 1.0598 *C2 + 2.0935 *C3+ 0.7291*D1 + 1.0949*D2 + 2.1324 *D3+ 2.866 *E + 0.2595*LT + 0.0537*ML+0.0468*OL…**Eqn 4.3**

Where: -

- ✓ *SULT is Start Up Lost Time in Seconds*
- ✓ *hs is Saturation Headway in Seconds*
- ✓ *GT is Green Time in Seconds*
- ✓ *AW is Approach Width in Meter*
- ✓ *LW is Lane Width in Meter*
- ✓ *AG is Approach Grade in %*
- ✓ *B1, B2,B3,C1,C2,C3,D1,D2,D3 & E are Vehicular Composition of B1, B2,B3,C1,C2,C3,D1,D2,D3 & E respectively (dummy takes value of 1 or 0)*
- ✓ *LT is Left Turn Movement (dummy takes value of 1 or 0)*
- ✓ *ML is Middle Lane (dummy takes value of 0 or 1)*
- ✓ *OL is Outside Lane (dummy takes value of 0 or 1)*

As shown in equation 4.3, Start Up Lost Time is modeled by Eight predictors and the coefficient of each predictor shows the relationship between start up lost time and the predictors. Start Up Lost Time will increase if Saturation Headway decreases, which tells that if the start-up lost time of the first four vehicles is large then the vehicles behind the 4th vehicle get long preparation time and the saturation headway will become decreased. In connection to this, Start Up Lost Time becomes Increased with an increment of green time due to the case of the drivers are relaxed and first four vehicles are not panic due to large green time and the same to this when Green Time is Large the cycle length becomes long and during the long cycle time the first drivers become depressed and the start-up lost time become bigger compared to Short Cycle time and Green time.

Approach Width and Lane Width are the Geometric Parameters as seen in the model they have an indirect relationship to the start up lost time. In the model, Start Up lost Time will decrease when Approach Width and Lane Width increases, which indicates that, the drivers are not confused during the maneuvering process when the Approach Width and Lane Width is large and the same to this when Approach Width and Lane Width are narrower start up lost time will become increased due to the case of the drivers maneuvering distracted by narrower lane width and Approach Width.

Approach Grade is one of the parameter in Geometric predictors and the model tells that Approach Grade has a direct relationship with Start Up lost Time. In the model, Start up Lost Time increased when the Approach grade is up grade (steep grade) which indicates that, the vehicles maneuvering process decelerate when approaching up grades (steep grades) due to this effect the vehicles waist large time to traverse the signalized intersection. However, when approaching Grade is Down Grade then Start Up Lost Time will become smaller due to the case that vehicles accelerate in downgrade approaches.

Type of Maneuvering

As shown in the regression equation, left turn movement is Categorical Variable with dummy codded 0 or 1 and this variable shows that when the left turn movement occur start up lost time become larger compared to Through Movement. This occurred due to the fact that left turn movement is a complex movement in signalized intersection and difficult movement for drivers in signalized intersection and also takes long time for discharging. The same effect to drivers discharging process, Start up Lost Time becomes larger for left turn movement compared to through movement.

Lane position

The lane position shows that moving near to the middle lane has slightly larger start up lost time compared to the outside lane and the inner lane. This indicates that, the drivers in the middle lane are crammed by the drivers on the both sides of the lane due to this the drivers on the middle lane position stressed by the drivers on the inner and outside lanes and sometimes they give priority for the outside and inner side drivers for narrow approaches of the intersection. To this impact, the drivers on the middle lane position waist an extra time compared to the other lane position drivers.

Vehicular Composition

The regression model shows that an increase in percentage of large vehicles shows an increment of start up lost time and the same when the large vehicles composition decreases start up lost time becomes decreased. This directs that, the vehicle size becomes the predominant factor on the start-up lost time of the vehicles. In relation to this, when the mix of large vehicles induced in first four vehicles, then following vehicle will waist an extra time due to poor visibility for the initiation of green light due to the leading large vehicles. As understood on the model, Startup Lost Time of vehicular composition A<B1<B2<B3, C1<C2<C3, D1<D2<D3<E.

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

This study mainly aims to estimate the value of start-up lost time and to investigate the factors that affect start up lost time at Addis Ababa City Signalized Intersection. In addition, this study confirmed the significant factors that will affect start up lost time through stochastic way as well as understand the study value probability distribution and determine the fitted probability distribution. Finally, the study conquered that the study value can be modeled by Multiple Linear Regression with the significant predictors and the following concluding remarks are investigated in this study;

- ✓ The vehicles after the fourth vehicle are under the saturated condition and the first four vehicles exposed to the start-up lost time.

- ✓ The estimated mean Start Up Lost Time is 2.513 Sec on the overall investigated sites. To compare and contrast this value with HCM recommendation an ideal condition is selected on the investigated sites. From the observed sites, the ideal condition is selected according to HCM recommendation. To this effect, the signal intersection with no heavy vehicle or all the vehicles are car, flat approach grade as well as no impedance of pedestrians is considered for the ideal condition selection and the ideal condition mean start up lost time is 2.373 Sec. Following this, t test was conducted to compare the study value with HCM recommendation and the t test declared that the study value is significantly different from HCM recommendation. Hence, Customizing and calibrating the of the default values to the local condition is mandatory in the design of Signalized Intersections.

- ✓ Lognormal, Weibull and Johnson Transformed Probability Distribution becomes the fitted probability distribution for the start-up lost time.

- ✓ Multiple Linear Regression model was developed for the start-up lost time by using predictors of Vehicular Composition, Approach Grade, Lane Width, Approach Width, Green Time, Lane position for maneuvering and Type of Movement. Accordingly, model specifies that start up lost time has direct relation to Approach Grade, Green Time and Vehicular Composition and also to the same start up lost time have an indirect relationship with Lane Width and Approach width. Left Turn movement and middle lane movement have large start up lost time compared to the others due to the case that left turn movement is a complex movement for the drivers and the drivers waste long time compared to through movement also Middle lane movement requires serious attention for the drivers and they will become highly frustrated and most of the time the drivers feel unsafe in the middle lane position and they will give priority for the other lane during departing from queued position. Further, effect of pavement condition was investigated on the studied value and entails that when the International Roughness Index is large then start up lost time becomes large due to the case that the roughness of the pavement increases the riding quality of the pavement becomes decreased due to this Start Up Lost Time becomes larger.

- ✓ All variables statistically significant to predict start up lost time at 95% Confidence Interval. The calculated R^2 Adj is 80.93%. In the mix of traffic, the size of the vehicle becomes the predominant factor in start-up lost time and as the percentage of large vehicle size increase then start up lost time becomes increased.

5.2. Recommendations

On the top of this research findings, Start Up Lost Time becomes not studied in Ethiopia and which is an essential parameter in signal timing, capacity and performance evaluation as well as to determine the extension of green time for signalized intersections. Hence, it is necessary to study this parameter by including Clearance lost time with bigger number of intersections for the large scale of the country. Since Start Up Lost Time vary from place to place due to the existing traffic condition and driver behavior, the researcher of this paper recommends for practical use of the developed model and the study value only for intersections of Addis Ababa, Ethiopia. Further, this study considered only the effect of Traffic, Geometric and Pavement condition parameters in addition to this the driver's behavior also needed to be investigated. For future research in this area, it is recommended to test the inter green time of signalized intersections by conducting large scale investigations on the startup lost time. Further Research areas related to this study are listed below:

- Effect of Weather/Climate condition on the Start Up Lost Time
- Effect of incident on the Start Up Lost Time
- Effect of Age and Sex deviation of Drivers on the Start Up Lost Time
- Effect of the model of vehicles on the Start Up Lost Time
- Effect of Coordinated Signals on the Start Up Lost Time
- Effect of Star Response Time on the Start Up Lost Time

REFERENCES

Agent, K. a. (1983). *Analysis of Lost Times at Signalized Intersections.* University of Kentucky, Lexington: Kentucky Transportation Research Program.

Akçelik, R. (. (2008). The Relationship between Capacity and Driver Behavior. *Proceedings of TRB National Roundabout Conference.* Kansas City, USA.

Akçelik, R., Besley, M., & Roper. (1999). *Fundamental Relationships for Traffic Flows at Signalised Intersections.* Vermont South, Australia. 194 p.: Research Report ARR 340. ARRB Transport Research Ltd.

Al-Ghamdi, A. (1999). Entering Headway for Through Movements at Urban Signalized Intersections. *Journal of the Transportation Research Board 1678*, 42–47.

Allsop, R. E. (1971). Delay-minimizing settings for fixed-time traffic signals at a single road junction. *IMA Journal of Applied Mathematics, 8*, 164-185.

Anderson, T. 2. (2011). Anderson-Darling tests of goodness of fit. *In International Encyclopedia of Statistical Science*, (pp. 52-54).

Arasan, V. K. (2005). Methodology for Modeling Highly Heterogeneous Traffic Flow. Journal. *Journal of Transportation Engineering 131*, 544–551.

Arasan, V. V. (2006). Estimation of Saturation Flow of Heterogeneous Traffic Using Computer Simulation. *20th European Conference on Modelling and Simulation.* Bonn, Germany.

Bonneson, J. A. (1992). Modeling queued driver behavior at signalized junctions, Transportation Research Record. *Journal of the Transportation Research Board 1365:*, 99-107.

Bester, C. J., & Varndell, P. J. (2002). The effect of a Leading Green Phase on the Start- up Lost Time of opposing vehicles, (July), 15–19.

Bhuiyan, N. F. (2018). Towards Performance Measure Analysis : development of a left turn Saturation Flow Rate model at signalized intersections towards performance measure analysis : development of a left turn saturation flow rate model at by Claremont Graduate University and California State University Long Beach, (May).

Brehmer C.L., K. (2003). *Evaluation of Traffic signal displays for protected/permissive left-turn control.* Washington D.C: National Cooperative Highway Research program NCHRP . Report 493.

Carstens. (1971). *Some Traffic Parameters at Signalized Intersections.* Traffic Engineering 41.

Çalişkanelli, P., & Tanyel, S. (2016). Investigating the driver's response time at signalized intersections. *Transport*, *33*(2), 1–9. https://doi.org/10.3846/16484142.2016.1250106

Çalışkanelli, S. P., Coşkun Atasever, F., & Tanyel, S. (2017). Start-up Lost Time and its Effect on Signalized Intersections in Turkey. *PROMET - Traffic&Transportation*, *29*(3), 321–329. https://doi.org/10.7307/ptt.v29i3.2214

Chang, G. L. (2001). *Empirical Investigation of Intersection Traffic Patterns Under Maryland Driving Populations - Analysis of the Critical-Lane Volume Method at Signalized Intersections.* The University of Maryland at College Park.

Clement, S. J. (2004). *Simple platoon advancement: a model of automated vehicle movement at signalized intersections.* Transportation Research Part C: Emerging Technologies 12(3–4): 293–320.

Coallition, N. T. (2005). National Traffic Signal Report Card, Executive Summary.

David S. H., K. P. (2013). Implications of Distracted Driving on Start-Up Lost Time for Dual Left-Turn Lanes. *Journal of Transportation Engineering vol. 139 No. 9*, p. 923-930.

Dey, P. P., Nandal, S., & Kalyan, R. (2013). Queue Discharge Characteristics at Signalised Intersections Under Mixed Traffic Conditions, European Transport, Transport Europei, 2013. 55(8), 1-21

Dion, F. R. (2004). Comparision of delay estimates at under saturated and over saturated pre-timed signlized intersection. *Transport Research Part B*, 99-122.

Earl George T. and Heroy, F. M. (1966). "Starting Response of Traffic at Signalized Intersections". *Traffic Engineering, Institute of Traffic Engineers, Vol. 36, No. 10*, 39-43.

Efstathiadis S., M. R. (1995). *Variability of departure headways.* Center for transportation research, University of Texas, research report no. 60019-2, (1995.

Efstathiadis, E. M. (2013). An investigation of the variability of start-up . *Intersections Control and Safety*, 1.

Engmann, S. A. (2011). Comparing distributions: the two sample Anderson-Darling test as an alternative to the Kolmogorov-Smirnoff test. *Journal of Applied Quantitative Methods, 6(3)*, pp. 1-17.

FHWA. (2013). *Signalized Intersection Informational Guide Second edition.* U.S.Department of Transportation.

Gerlough, D. L. (1967). *Improved Criteria for Traffic Signals at Individual Intersection. Highway Research Board.* Washington, D. C. pp. 42: National Research Council, NCHRP Report 32:.

Greenshields. (1935). Distance and time required to overtake and pass vehicles. (pp. 15, pp. 332-342). Highway Research Board Proceedings.

Greenshields, B. S. (1947). *Traffic Performance at Urban Street Intersections.* Eno Foundation for Highway Traffic Control.

Greenshields, B. S. (1947). Traffic Performance at Urban Street Intersections. (pp. 23-30). Yale Bureau of Highway Traffic.

Habtamu, C. (2018). Departure Headway Modeling at Signalized Intersection of Addis Ababa. School of Civil and Environmental Engineering Master of Science Thesis, Addis Ababa, Ethiopia.

HCM, (2010). *Highway Capacity Manual.* Transportation Research Board. Washington, D.C

Hounsell, N. (1989). *The design and performance of signal controlled intersection.* doctoral diss., University of Southampton, Britain, .

ITE. (1994). *Determination of Vehicle Signal Change and Clearance Intervals.* Washington, DC. 16 p.: An Informational Report of the Institute of Transportation Engineers (ITE). .

Jacobs, N. (1998). *The determination of star-up lost time and saturation flow rates at signalized intersections (in Afrikaans).* The determination of star-up lost time and saturation flow rates at signalized intersections (in Afrikaans).

Jan, K. I. (2009). Mathematical application for departure model. *Paper presented at the Proceedings of the 2009 Euro American Conference on Telematics and Information Systems: New Opportunities to increase Digital Citizenship.*

Jin, X. Z. (2009). Departure Headways at Signalized Intersections: A Log-Normal Distribution Model Approach. *Transportation research part C: emerging technologies 17*, 318–327.

Jiyuan Tan, L. L. (2013). Distribution Models for Start-Up lost time and effective departure flow rate. *Journal of Transportion*.

Khan, S. I. (1999). Modeling Heterogeneous Traffic Flow. *Transportation Research Record 1678* (pp. 234-241). Washington,D.C., pp. 234-241.: TRB, National Research Council.

Kidus, A. (2016). *Development of Models of Interrupted Traffic Flow Conditions of Addis Ababa from a Study of an Urban Multi-Lane Highway.* Addis Ababa: Partial Fulfillment of the Requirements for the Degree of Master of Science In Road and Transport Engineering at Addis Ababa University.

Koonce, P. (2008). *Traffic Signal Timing Manual.* 6300 Georgetown Pike, HSR 20, Room No. T301 : Federal Highway Administration.

Lee, J. a. (1986). Entering Headway at Signalized Intersections in a Small Metropolitan Area. *Transportation Research Record 1091, TRB, National Research Council,*, pp.117-126.

Leong, H. J. (1964). "Some Aspects of Urban Intersection Capacity". *Proceedings 1964* (pp. pp. 305-338). Australian Road Research Board.

Leong, L. (2004). *A study on saturation flow rates of through vehicles at signalized intersections based on Malaysian road conditions.* doctoral diss., University Sains Malaysia.

Li, H., & Prevedouros. (2000). Detailed observations of saturation headways and start-up lost times. *Journal of the Transportation Research Record 1802*, 44-53.

Lieu, H. (1999). *Traffic Flow Theory*. Washington DC, United States: Transportation Research Board.

Lin, F., & Thomas, D. R. (2005). Headway Compression During Queue Discharge at Signalized Intersections. Transportation Research Record: Journal of the Transportation Research Board, No. 1920, Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 81–85

Liu, M. M. (2011). An Exploratory Hazard-based Analysis of the First Discharge Headway. *Procedia - Social and Behavioral Sciences 16*, 536–547.

Maini, P. K. (2000). Discharge Characteristics of Heterogeneous Traffic at Signalized Intersections. *Transportation Research Circular E-C018: 4th International Symposium on Highway Capacity*, (pp. 258–270).

Mathew, P. T. V. (2014). Signalized Intersection Delay Models Lecture Notes in Transportation Systems Engineering, 1–18.

Matsoukis, E., & Efstathiadis, S. (2010). An investigation of the variability of start-up lost times and departure headways at signalized intersections in urban areas. Journal of WIT Transactions on the Built Environment, 111, 59–68.

Md Hadiuzzaman, M. M. (2008). Saturation Flow Model at Signalized Intersection for Non-lane Based Traffic. *Canadian Journal of Transportation volume 2, Part 1*.

Messer, C. J., & Fambro. (1977). Effects of signal phasing and length of left-turn bay on capacity. *Journal of the Transportation Research Board 644*, 95-101.

Minh, S. (2003). Analysis of Motorcycle Effects to Saturation Rlow rate at Signalized Intersection in developing countries (Vol. 5, pp. 1211–1222).

Mohamed Shawky, A. A.-G. (2016). Start-up delay Estimation at Signalized Intersections: Impact of Left-Turn Phasing Sequences. *International Journal of Engineering and Applied Sciences (IJEAS)*, 6.

Moussavi, M. T. (1990). *Variability of Departure Headways at Signalized Intersections.* ITE 1990 Compendium of Technical Papers , 313–317.

Philpot, F. (2005). Blasted Light. *Proffesional Engineering, 18(21)*, 48-48.

Radhakrishnan, P. M. (2011). *Passenger Car Units and Saturation Flow Models for Highly Heterogeneous Traffic at Urban Signalised Intersections.* Transportmetrica 7.

Radhakrishnan, S., & Ramadurai, G. (2015). Discharge headway model for heterogeneous traffic conditions. *Transportation Research Procedia*, *10*(July), 145–154.

Razali, N. A. (2011). Power of Shapiro-Wilk, Kolmogorov-Smirnov, Lilliefors, and Anderson-Darling tests. *Journal of Statistical modeling and analytics*, pp.21-33.

Roess, P. R. (2010). *Traffic Engineering. 4th edition. Pearson. 744 p.*

Roess, R. P. (1989). *Levels of Service in Shared-Permissive Left-Turn Lane Groups at Signalized Intersections.* Transportation Training and Research Center, Polytechnic University, Brooklyn, N.Y.

Scott, M. (2011). Traffic Modeling. retrieved from http://home2.fvcc.edu/~dhicketh /DiffEqns/Spring11projects /Scott_Miller/Project.pdf (Pp. 1 – 3)

SHAHZAD, C. M. (2013). *Capacity analysis of signalised intersections: effects of green time on saturation flow rate.* Auckland: The University of Auckland.

Shawky, M., & Al-ghafli, A. (2016). Start-up delay Estimation at Signalized Intersections : Impact of Left-Turn Phasing Sequences. *International Journal of Engineering and Applied Sciences*, *3*(11), 7–12.

Smith, B. L. (2006). *Data–Driven Methodology for Signal Timing Plan Development: A Computational Approach.* Computer-Aided Civil and Infrastructure Engineering.

Sobey, E. J. (2006). *A Field Guide to Roadside Technology (Vol. 1).* Chicago : Chicago Review Press.

Temesgen, A. (2015). *Capacity Evaluation of Roundabout and Signalized Junctions.* Addis Ababa: Addis Ababa Institute of Technology, School of Civil and Enviromental Engineering.

Tong, H. Y., & Hung. (2002). Neural network modeling of vehicle discharge headway at signalized intersection. *Transportation Research Part A: Policy and Practice36(1)*, 17-40.

TSS. (2010). *Aimsun 6.1 Users' Manual.* Transport Simulation Systems (TSS) 2005–2010.

Umama, A. (2010). *Passenger Car Equivalent Factors for Level Freeway Segments Operating under Moderate and Congested Condition.* MSc thesis: Marquette University.

Van Winsum, W. (. (1999). The human element in car following models. Transportation research part F. *traffic psychology and behaviour*, 207-211.

Vanvoorhis, C. A. (2007). Understanding power and rules of thumb for. (pp. pp. 43-50). Tutorials in Quantitative Methods for Psychology, 3(3).

Vose, D. 2. (2010). *Fitting distributions to data.* Retrieved March, 8, P.2016 Washington,.

Washington, S.O, Karlaftis,M.G.,and Mannering,F., (2010). *Statistical and Econometric Method for transportation data analysis.* Chapman and Hall/CRC.

Winsum, W. V. (1996). Choice of time-headway in car-following and the role of time-to-collision information in braking. *Ergonomics*, *39*, 579-592.

Wu, J. H. (2010). Departure Headway Distribution Modeling for Ordinary Large Cities Based On Video Information. *Computer Sciences and Convergence Information Technology (ICCIT), 2010 5th International Conference on. IEEE*, (pp. 1040–1045).

Yang, S. (2013). Enlarged Stopping Distance to Improve Vehicle Discharge at Urban Signalised

Intersections for the degree of Doctor of Philosophy Civil Engineering and Built Environment School Science and Engineering Faculty, Queensland University of Technology 2013

Yang, Q. (2012). Driver Response Time of Queuing Vehicles at Urban Signalized Intersections 8th International Conference on Traffic and Transportation Studies Changsha, China, August 1–3, 2012

Yin, S. L. (2009). Headway Distribution Modeling with Regard to Traffic Status. *2009 IEEE Intelligent Vehicles Symposium*, (pp. 1057–1062).

Zhao, Y et al.. (2015). Estimation of saturation flow rate and start-up lost time for signal timing based on headway distribution. *Discrete Dynamics in Nature and Society*, *2015*.

APPENDIX A

Signalized Intersection of Addis Ababa

Table A1: Signalized Intersections of Addis Ababa

Item No.	Location Name	Left Turn Treatment		Channelized		No of Approaches							Working Condition	No of Lanes
						Three			Four			Five or More		
		Protected	Permitted	Yes	No	T	Skew T	Y	Right Angle	Offset	Oblique			
1	Around Semen Hotel		√		√				√				Yes	3
2	Bole Mikael		√		√				√				Yes	4
3	Around Sengatera		√		√				√				Yes	3
4	Near Atlas Hotel		√	√							√		Yes	3
5	**Lebu Mebrathayil**	√		√					√				**Yes**	**3**
6	Agona Cinima (Gotera)		√		√		√						Yes	3
7	Legahr	√			√				√				Yes	3
8	Bauna ena Shay		√		√				√				Yes	4
9	Mexico Round About												No	
10	**Gerji Mebrathayil**		√	√			√						**Yes**	**3**
11	Around Imperial Hotel	√			√				√				Yes	3
12	Coca Bridge												No	
13	Kolefe 18 Mazoria												No	
14	Around Parliament		√		√				√				Yes	4
15	Around National Theater	√			√							√	Yes	4
16	Around Post Office		√	√					√				Yes	3
17	Around ETV		√	√							√		Yes	3
18	Megenagna Round About												No	
19	Near Harambe Hotel		√		√						√		Yes	3
20	Near Shola Market		√		√	√							Yes	4
21	Around Goma Kuteba		√	√					√				Yes	4
22	Near Banko Diroma		√	√							√		Yes	3
23	Meskel Square		√	√							√		Yes	5
24	Near St. Joseph School		√		√						√		Yes	3
25	Near Lideta School		√		√			√					Yes	3

Item No.	Location Name	Left Turn Treatment		Channelized		No of Approaches							Working Condition	No of Lanes
						Three			Four			Five or More		
		Protected	Permitted	Yes	No	T	Skew T	Y	Right Angle	Offset	Oblique			
26	**Jomo Ring Road Michel**		√	√							√		**Yes**	**2**
27	Kera to Gofa Turn		√		√				√				Yes	3
28	Near Kera		√		√				√				Yes	3
29	St. Marry		√		√				√				Yes	2
30	Near to Radisson Blue Hotel												No	
31	Saltemihirit to Safari		√		√				√				Yes	3
32	**Saris Abo**		√		√					√			**Yes**	**4**
33	Kadisco		√		√				√				Yes	4

.

APPENDIX B

Estimated PCU values for all Vehicle Types in each approach of Signalized Intersection

Table D1: PCU Values for all vehicles in each approaches of Signalized intersection

Gerji Mebrahaile Signalized Intersection		
Jacros to Megengna Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.089	1
Small Bus	3.406	1.6308
Small Truck	2.833	1.356
Large Bus	4.199	2.010
Large Truck	4.333	2.074
Truck trailer	5.917	2.833
Saris Abo Signalized Intersection		
Kaliti to Megengna Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.189	1
Small Bus	3.329	1.520
Small Truck	2.587	1.181
Large Bus	4.298	1.963
Large Truck	4.384	2.001
Articulated Bus	5.805	2.651
Truck Trailer	5.215	2.381
Megengna to Kaliti Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.045	1
Small Bus	2.774	1.355
Small Truck	2.592	1.267
Large Bus	3.861	1.887
Large Truck	4.416	2.1587
Articulated Bus	5.667	2.769
Truck Trailer	5.111	2.498
Jemo Michael Signalized Intersection		
Ayertena to German Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.042	1

Small Bus	3.136	1.535
Small Truck	2.866	1.403
Large Bus	3.593	1.759
Large Truck	4.056	1.986
Articulated Bus	5.000	2.448
Truck Trailer	5.800	2.840

German to Ayertena Approach

Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.270	1
Small Bus	3.000	1.321
Small Truck	2.778	1.224
Large Bus	4.889	2.153
Large Truck	4.652	2.049
Articulated Bus	6.600	2.906
Truck Trailer	6.452	2.841

German to Jemo Approach

Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.163	1
Small Bus	2.864	1.324
Small Truck	2.749	1.270
Large Bus	4.323	1.998
Large Truck	4.583	2.118
Articulated Bus	6.200	2.865
Truck Trailer	6.000	2.772

Jemo to Mexico Approach

Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.081	1
Small Bus	2.833	1.361
Small Truck	2.458	1.181
Large Bus	4.000	1.922
Large Truck	4.166	2.002

Mexico to German Approach

Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.163	1

Small Bus	3.250	1.502
Small Truck	2.750	1.271
Large Bus	5.667	2.619
Large Truck	5.000	2.311
Truck Trailer	5.792	2.677
Mexico to Jemo Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	1.892	1
Small Bus	2.763	1.459
Small Truck	2.531	1.337
Large Bus	3.715	1.962
Large Truck	4.652	2.457
Lebu Mebrathaile Signalized Intersection		
German to Kaliti Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.364	1
Small Bus	3.418	1.445
Small Truck	2.643	1.117
Large Bus	4.513	1.908
Large Truck	3.647	2.542
Truck Trailer	5.737	2.427
Articulated Bus	8.266	3.496
Kaliti to German Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	1.870	1
Small Bus	2.506	1.339
Small Truck	2.393	1.279
Large Bus	3.587	1.917
Large Truck	3.329	1.779
Truck Trailer	5.189	2.773
Articulated Bus	5.377	2.874
Kaliti to Sebeta Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.046	1
Small Bus	2.734	1.336

Small Truck	2.382	1.164
Large Bus	3.671	1.793
Large Truck	3.546	1.733
Truck Trailer	6.453	3.152

Sebeta to German Approach		
Vehicle Types	Average Headways (sec)	PCU values (in headway ratio method)
Car	2.369	1
Small Bus	3.058	1.291
Small Truck	3.160	1.333
Large Bus	5.138	2.168
Large Truck	4.3217	1.824

APPENDIX C

Estimation of Queue position for Saturation Headway and Start- up Lost Time

Table C-1: Descriptive Statistics for Kaliti to Megengna Queue Position Vs Mean Headway (sec)

Variable	Queue Position	Total Count	Mean	StDev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	124	3.1067	0.4207	2.4646	3.043	4.8623	1.31
	2	124	2.871	0.597	1.2229	2.8098	5.1569	0.57
	3	124	2.7544	0.7286	1.5012	2.7076	5.4145	0.59
	4	124	2.4143	0.6927	1.211	2.4131	6.3798	1.58
	5	124	2.1645	0.5487	1.0188	2.083	3.667	0.45
	6	124	2.1594	0.5917	1.167	2	5.25	1.75
	7	124	2.1962	0.6287	0.667	2.1155	4.417	0.73
	8	124	2.2245	0.7249	0.9156	2.083	5	1.41
	9	124	2.0908	0.7011	0.5249	1.9368	4.75	1.27
	10	123	2.083	0.5096	0.7078	2	3.7884	0.29
	11	122	2.1822	0.7264	1.083	2.083	7	2.79
	12	119	2.072	0.5359	0.9576	2.083	3.75	0.65
	13	119	2.1068	0.6062	1.0188	2	4.083	1.04
	14	116	2.0848	0.5123	1.1461	2.084	3.417	0.23
	15	112	2.0622	0.4916	0.833	2	3.5	0.32
	16	110	2.1374	0.6388	0.679	2.0415	5.833	1.91
	17	107	2.0993	0.5912	0.667	2	4.834	1.27
	18	100	1.9532	0.5062	0.167	2	3.25	-0.02
	19	92	2.1082	0.6527	0.917	2	4.5	1.26
	20	83	2.0467	0.6197	0.834	2	4.75	1.33
	21	76	2.0296	0.6097	0.833	2	3.667	0.6

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.1059 Sec and as shown in the table 5th queued vehicle mean discharge headway is 2.1645 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

N	Mean	StDev	SE Mean	95% CI for μ
124	2.1649	0.5487	0.0493	(2.0674, 2.2624)

μ: mean of Sample

Test

Null hypothesis H_0: $\mu = 2.1059$

Alternative hypothesis H_1: $\mu \neq 2.1059$

T-Value	P-Value
1.20	0.233

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

T test for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
124	2.4143	0.6927	0.0622	(2.2912, 2.5374)

μ: mean of Sample

Test

Null hypothesis H_0: $\mu = 2.1059$

Alternative hypothesis H_1: $\mu \neq 2.1059$

T-Value	P-Value
4.96	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for kaliti to Megengna Approach.

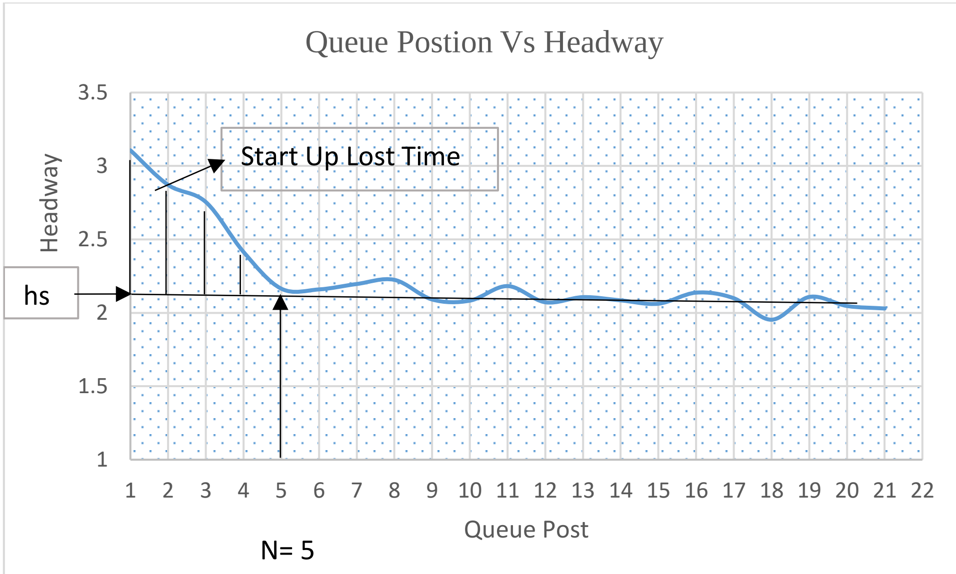


Figure C-1. Queue Position Vs Mean Headway for Kaliti to Megengna Through Movement

Table C-2: Descriptive Statistics Queue Position Vs Mean Headway (sec) for Megengna to Kaliti

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	60	2.889	0.347	2.2913	2.8284	3.8944	0.93
	2	60	2.6843	0.4597	1.7747	2.7059	4.3204	0.6
	3	60	2.556	0.6762	1.3554	2.4897	4.9416	1.23
	4	60	2.1822	0.4577	1.1169	2.0491	3.3	0.44
	5	60	1.8597	0.4125	1	1.833	3.084	0.35
	6	60	1.8153	0.3935	1.0807	1.833	2.917	0.32
	7	56	1.9136	0.4774	1	1.85	3	0.25
	8	56	1.9193	0.4923	0.9219	1.834	3.167	0.64
	9	54	1.8221	0.5207	0.916	1.864	3.667	0.79
	10	51	1.9127	0.4808	0.9264	1.834	3.667	0.85
	11	51	1.8163	0.5158	0.916	1.75	3	0.17
	12	48	1.8766	0.4139	1.083	1.917	2.5814	-0.13
	13	46	1.8492	0.5632	0.667	1.9105	3.416	0.33
	14	46	1.9248	0.5438	1	1.834	4.25	1.67
	15	41	1.846	0.5697	0.917	1.667	3.0026	0.27
	16	38	1.8129	0.4931	1	1.833	3.167	0.91
	17	35	1.8393	0.5675	0.917	1.75	4.25	2.15

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 1.8621 sec and as shown in the table 5th queued vehicle mean discharge headway is 1.8597 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

N	Mean	StDev	SE Mean	95% CI for μ
60	1.8597	0.4125	0.0533	(1.7531, 1.9663)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 1.8621$
Alternative hypothesis	H_1: $\mu \neq 1.8621$

T-Value	P-Value
-0.05	0.964

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

T Statistics for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
60	2.1822	0.4577	0.0591	(2.0640, 2.3004)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 1.8621$
Alternative hypothesis	H_1: $\mu \neq 1.8621$

T-Value	P-Value
5.42	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Megengna to Kaliti Approach.

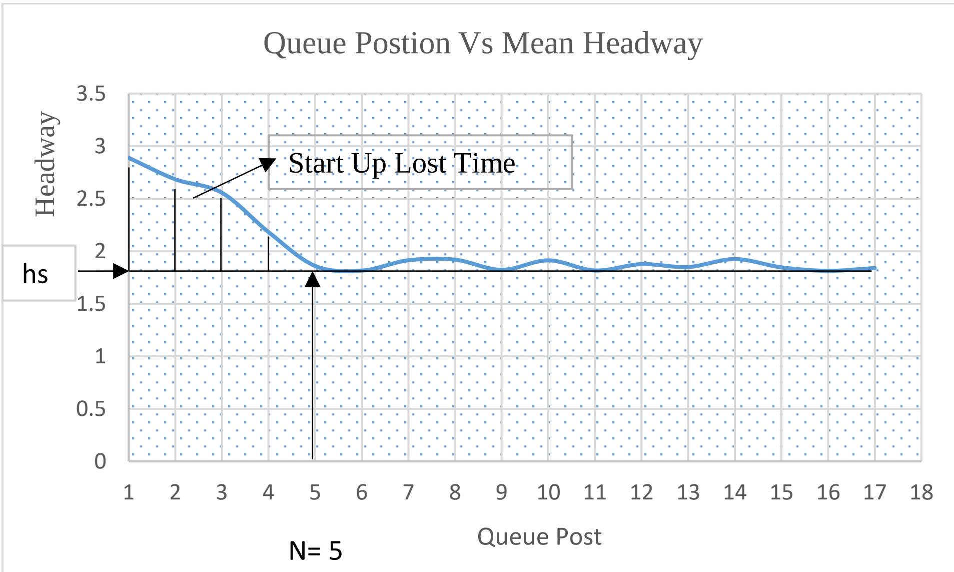


Figure C-2. Queue Position Vs Mean Headway for Megengna to Kaliti

Table C-3: Descriptive Statistics Queue Position Vs Mean Headway(s) for Ayertena to Gereman Through Movement

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	53	3.3327	0.3337	2.6335	3.3445	4.1115	0.25
	2	53	2.854	0.767	1.411	2.694	5.013	1.11
	3	53	2.581	0.852	1.29	2.381	5.473	1.26
	4	53	2.437	1.123	1.198	2.07	7.11	2.6
	5	52	2.0891	0.5064	0.7845	2.1665	3.5	-0.18
	6	52	2.0792	0.5899	1.167	2	3.416	0.77
	7	52	2.0811	0.6947	0.7971	1.917	3.666	0.74
	8	52	1.9603	0.6858	0.833	1.8545	4.5	1.11
	9	51	1.9504	0.6824	1	1.833	4.083	1.06
	10	50	2.0439	0.5447	0.917	2.0818	3.25	0.12
	11	47	1.965	0.726	0.923	1.75	5.5	2.66
	12	42	2.142	0.728	1.167	1.897	3.75	0.8
	13	36	1.968	0.745	1	1.792	3.75	0.92
	14	30	1.989	0.891	0.334	1.708	4.5	1.09
	15	23	1.984	0.683	1	1.803	3.667	1.09

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.022 sec and as shown in the table 5th queued vehicle mean discharge headway is 2.0891 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

T test for 5th Queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
52	2.0891	0.5064	0.0702	(1.9481, 2.2301)

μ: mean of Sample

Null hypothesis	H_0: $\mu = 2.0229$
Alternative hypothesis	H_1: $\mu \neq 2.0229$

T-Value	P-Value
0.94	0.350

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

t Statistics for 4th queued vehicle

N	Mean	St.Dev	SE Mean	95% CI for μ
52	2.437	1.123	0.156	(2.124, 2.750)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.0229$
Alternative hypothesis	H_1: $\mu \neq 2.0229$

T-Value	P-Value
2.66	0.010

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Ayertena to German Approach.

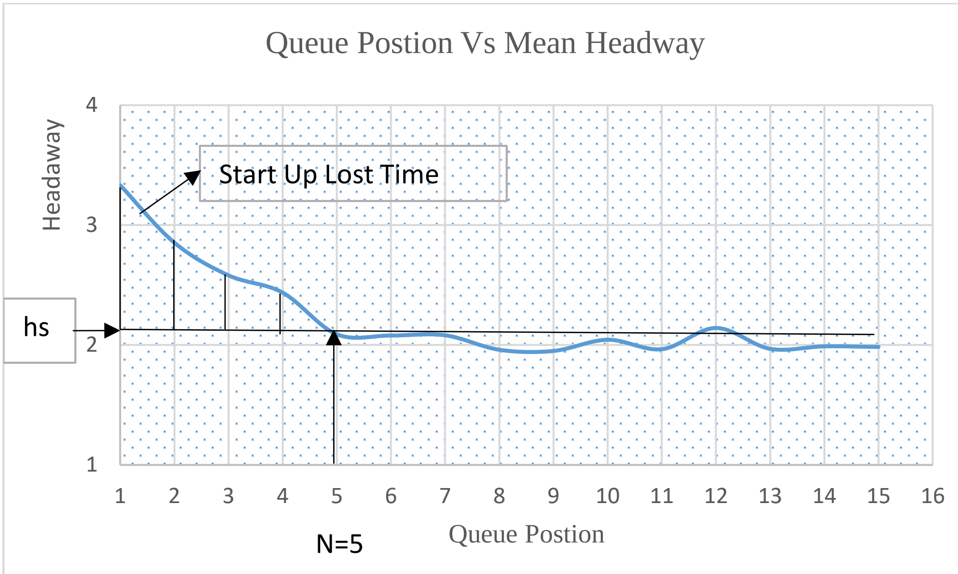


Figure C-3. Queue Position Vs Mean Headway for Ayertena to German Through

Table C-4: Descriptive Statistics Queue Position Vs Mean Headway(s) for German to Ayertena

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	43	3.362	0.5937	2.5729	3.3445	4.7563	0.79
	2	43	2.919	0.664	1.938	2.694	5.133	1.33
	3	43	2.779	0.81	1.361	2.577	5.296	1.35
	4	43	2.65	0.808	1.416	2.628	4.607	0.85
	5	43	2.242	0.703	1.084	2.167	4.903	1.33
	6	43	2.347	0.882	1.25	2.167	5.5	1.78
	7	42	2.306	0.7	0.833	2.327	3.834	0.2
	8	38	2.1016	0.5712	0.917	2	3.5	0.44
	9	37	2.259	0.621	1.135	2.167	3.75	0.35
	10	35	2.165	0.617	1.083	2.17	3.333	0.19
	11	33	2.277	0.732	1.301	2.25	4.667	1.07
	12	30	2.03	0.4157	1.333	2	3.166	0.57
	13	24	1.918	0.632	1.083	1.75	3.334	0.99
	14	18	2.343	0.693	1.464	2.256	3.812	0.58
	15	13	2.118	0.754	1.26	1.916	4	1.29
	16	10	1.795	0.412	0.976	1.685	2.417	-0.4

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.1584 Sec and as shown in the table 5th queued vehicle mean discharge headway is 2.0891 Sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t test for 5th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
43	2.242	0.703	0.107	(2.026, 2.458)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1584$
Alternative hypothesis	H_1: $\mu \neq 2.1584$

T-Value	P-Value
0.78	0.440

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

t test for 4th queued

N	Mean	StDev	SE Mean	95% CI for μ
43	2.650	0.808	0.123	(2.401, 2.899)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1584$
Alternative hypothesis	H_1: $\mu \neq 2.1584$

T-Value	P-Value
3.99	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for German to Ayertena Approach.

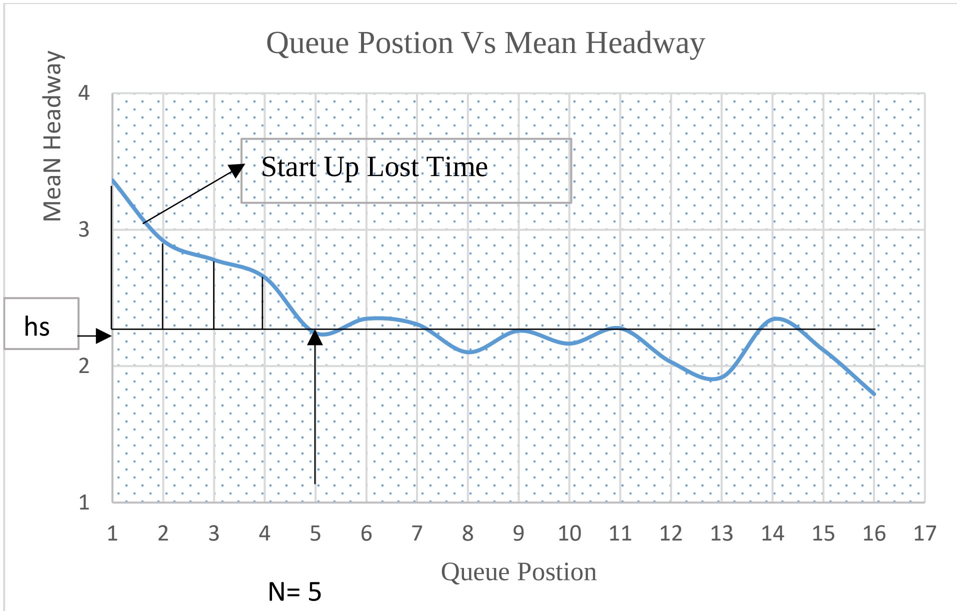


Figure C-4. Queue Position Vs Mean Headway for German to Ayertena Through Movement

Table C-5: Descriptive Statistics German to Jemo Queue Position Vs Mean Headway

Variable	Queue Position	Total	Mean	StDev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	39	3.2114	0.4784	2.3407	3.0484	4.8879	1.32
	2	39	2.8766	0.5345	1.7357	2.885	4.5668	0.46
	3	39	2.7149	0.5982	1.6285	2.6494	4.8679	1.37
	4	39	2.625	0.791	1.467	2.559	4.456	0.69
	5	39	2.2544	0.5747	1.25	2.084	3.75	0.51
	6	38	2.423	0.866	1.334	2.25	5.917	2.52
	7	37	2.267	0.624	1.083	2.366	3.5	-0.31
	8	36	2.1688	0.4743	1.083	2.0835	3.184	0.12
	9	32	2.172	0.68	0.917	2.083	4.166	0.85
	10	30	2.176	0.573	1.083	2.166	3.75	0.79
	11	27	2.021	0.553	0.916	1.917	3.169	0.01
	12	25	1.8863	0.4685	1.083	1.833	3.167	0.98
	13	20	2.084	0.659	1.083	2.189	3.5	0.04
	14	16	2.032	0.553	1.18	1.834	3	0.66
	15	12	1.906	0.529	1.167	1.751	3.083	0.87

	16	11	1.991	0.444	1.333	1.916	3	0.91

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.1151 sec and as shown in the table 5th queued vehicle mean discharge headway is 2.2544 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t test for 5th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
39	2.2544	0.5747	0.0920	(2.0681, 2.4407)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1151$
Alternative hypothesis	H_1: $\mu \neq 2.1151$

T-Value	P-Value
1.51	0.138

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

t test for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
39	2.625	0.791	0.127	(2.369, 2.881)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1151$
Alternative hypothesis	H_1: $\mu \neq 2.1151$

T-Value	P-Value
4.03	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for German to Jemo Approach

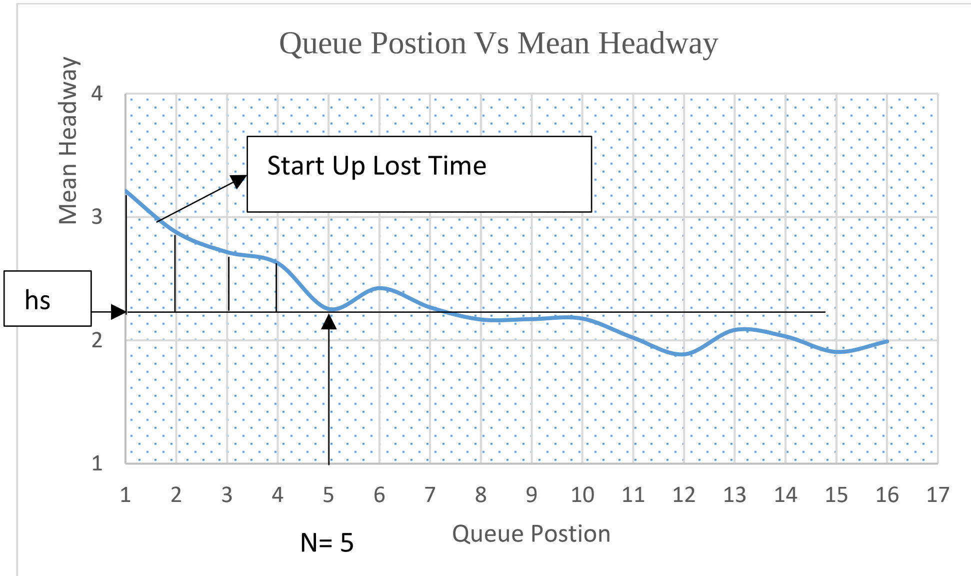


Figure C-5. German to Jemo Left Turn Queue Position Vs Mean Headway

Table C-6: Descriptive Statistics Queue Position Vs Mean Headway(s) for Jemo to Mexico Through Movement

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	71	3.3496	0.3971	2.5976	3.2946	4.4418	0.79
	2	71	2.8661	0.482	1.7337	2.9159	3.7523	-0.39
	3	71	2.7527	0.5146	1.4441	2.7759	3.9587	-0.06
	4	71	2.6066	0.6424	1.4784	2.6041	4.7479	0.97
	5	71	2.145	0.547	1.25	2.083	4	1.09
	6	71	2.1762	0.5784	1.25	2	4.583	1.8
	7	70	2.1391	0.5975	1.333	2.083	4.416	1.89
	8	70	2.0989	0.5982	0.834	1.917	4.25	1.32
	9	70	2.1371	0.5998	1.167	2.083	4.584	1.72
	10	70	2.1686	0.7168	1.3006	2	5.833	2.71
	11	70	2.1898	0.5889	1.083	2.083	4.333	1.86
	12	68	2.2005	0.6933	1.25	1.9585	4.917	1.35
	13	65	2.1625	0.6156	1.25	2	4.333	1.35
	14	63	2.1292	0.4716	1.416	2	3.75	1.01

	15	59	2.1049	0.4731	1.25	2	3.583	0.9
	16	56	2.1201	0.5475	1	2.0415	3.833	0.97
	17	45	2.217	0.955	1.167	2.083	7.583	4.24

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.1529 sec and as shown in the table 5th queued vehicle mean discharge headway is 2.145 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t test for 5th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
71	2.1450	0.5470	0.0649	(2.0155, 2.2745)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1529$
Alternative hypothesis	H_1: $\mu \neq 2.1529$

T-Value	P-Value
-0.12	0.903

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

T test for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
71	2.6066	0.6424	0.0762	(2.4545, 2.7587)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1529$
Alternative hypothesis	H_1: $\mu \neq 2.1529$

T-Value	P-Value
5.95	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Jemo to Mexico Through movement.

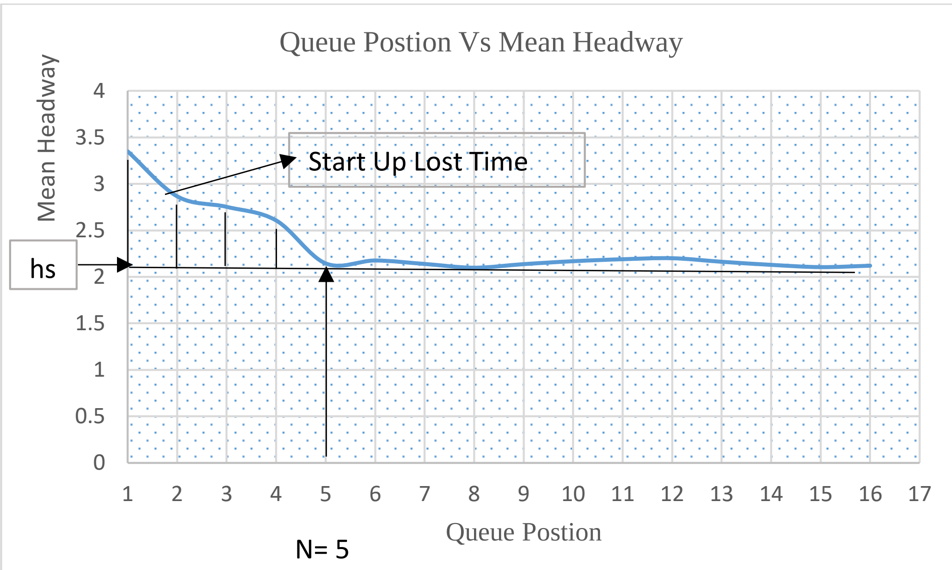


Figure C-6: Queue Position Vs Mean Headway for Jemo to Mexico Through Movement

Table C-7: Descriptive Statistics Queue Position Vs Mean Headway (sec) for Mexico to German Left Turn

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	41	3.4528	0.4701	2.6812	3.3575	4.785	0.85
	2	40	2.822	0.818	1.575	2.649	5.302	1.13
	3	40	2.733	1.101	1.444	2.567	6.435	2.09
	4	39	2.507	1.05	1.144	2.411	7.035	2.51
	5	37	2.07	0.5068	1.25	2.25	3.111	0.14
	6	35	2.06	0.4484	1.5	2.166	3.5	0.78
	7	30	2.054	0.3133	1.5	1.9585	2.584	0.1
	8	27	2.0016	0.4136	1.25	1.916	2.75	0.33
	9	22	1.97	0.582	1.25	1.836	4.25	2.99
	10	15	1.92	0.435	0.667	1.666	2.425	-0.43
	11	14	1.9522	0.3096	1.666	1.8318	2.583	0.96

	12	10	1.98	0.388	1.55	1.917	2.833	1.19
	13	7	1.75	0.2407	1.333	1.833	2	-0.76
	14	5	1.7332	0.2075	1.583	1.667	2.083	1.67
	15	3	1.864	0.372	1.508	1.833	2.25	0.37

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 1.9413sec and as shown in the table 5th queued vehicle mean discharge headway is 2.07 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t test for 5th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
37	2.0700	0.5068	0.0833	(1.9010, 2.2390)

μ: mean of Sample

Test

Null hypothesis H_0: $\mu = 1.9413$

Alternative hypothesis H_1: $\mu \neq 1.9413$

T-Value	P-Value
1.54	0.131

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

T test for 4th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
39	2.507	1.050	0.168	(2.167, 2.847)

μ: mean of Sample

Test

Null hypothesis H_0: $\mu = 1.9413$

Alternative hypothesis H_1: $\mu \neq 1.9413$

T-Value	P-Value
3.36	0.002

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Mexico to German Left Turn.

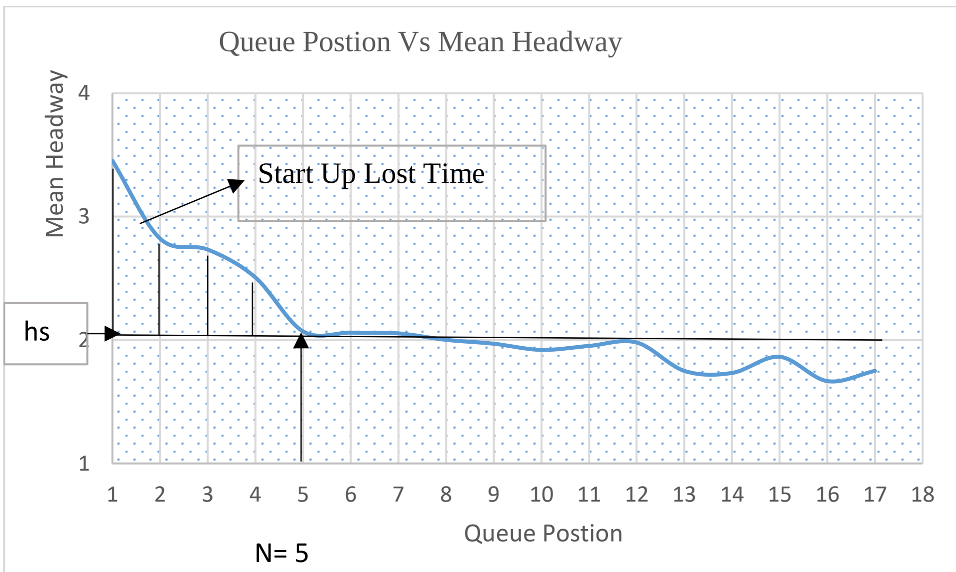


Figure C-7. Queue Position Vs Mean Headway for Mexico to German Left Turn

Table C-8: Descriptive Statistics Queue Position Vs Mean Headway (s) for Mexico to Jemo Through Movement

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	40	3.4318	0.4281	2.8289	3.385	4.451	0.64
	2	40	2.766	0.5459	1.8875	2.6942	4.1478	0.52
	3	40	2.394	0.733	1.146	2.315	4.09	0.37
	4	40	2.166	0.5998	1.211	2.0692	3.7507	0.64
	5	39	2.0679	0.5935	1.167	1.917	4.584	2.12
	6	39	2.1206	0.359	1.417	2.1651	3.25	0.51
	7	39	2.1623	0.4644	1.167	2.083	3.417	0.51

8	38	2.0977	0.4439	1.25	2.1245	3.5	0.89
9	38	2.0608	0.4493	1.2558	2.083	3.083	0.52
10	38	2.1262	0.4668	1.2464	2.084	3.75	1.45
11	36	2.085	0.4152	1.333	2.083	3.083	0.38
12	34	1.9729	0.4036	1.166	2.083	2.833	0.03
13	30	1.9402	0.4347	0.8474	1.9	2.917	0.14
14	24	2.0164	0.317	1.334	2.084	2.583	-0.48
15	18	1.877	0.448	0.583	1.85	2.5	-1.29
16	14	1.9557	0.3168	1.416	1.9885	2.583	0.02
17	11	1.816	0.426	1.25	1.833	2.417	-0.2
18	9	1.888	0.468	1.167	1.917	2.584	-0.05

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.0133 sec and as shown in the table 5th queued vehicle mean discharge headway is 2.0679 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t test for 5th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
39	2.0679	0.5935	0.0950	(1.8755, 2.2603)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.0133$
Alternative hypothesis	H_1: $\mu \neq 2.0133$

T-Value	P-Value
0.57	0.569

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

t test for 4th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
40	2.2666	0.5998	0.0948	(2.0748, 2.4584)

μ: mean of Sample

Test

Null hypothesis	H_0: μ = 2.0133
Alternative hypothesis	H_1: μ ≠ 2.0133

T-Value	P-Value
2.67	0.011

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Mexico to Jemo Through movement.

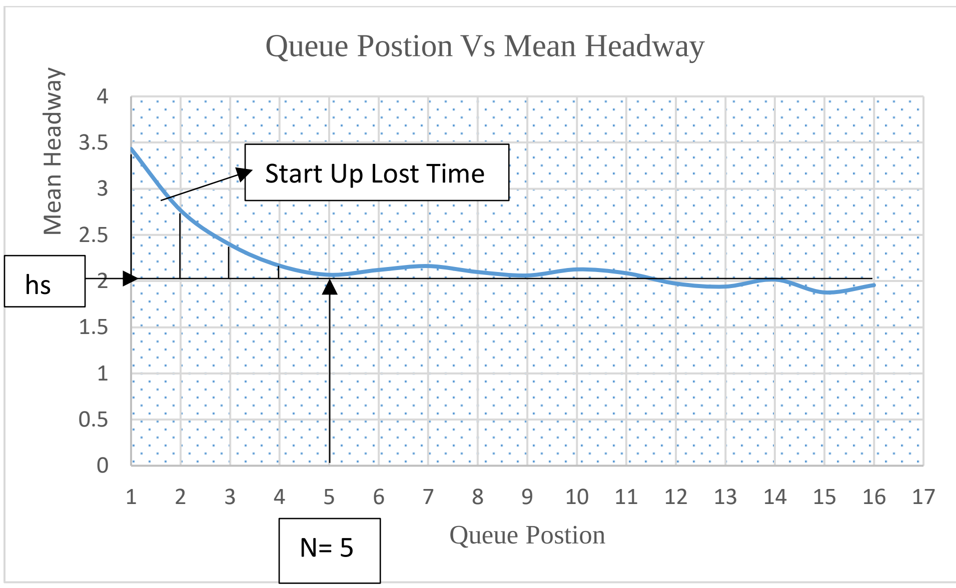


Figure C-8. Queue Position Vs Mean Headway for Mexico to Jemo Through Movement

Table C-9: Descriptive Statistics Queue Position Vs Mean Headway (sec) for German to Kaliti Through Movement

Variable	Queue Position	Total	Mean	StDev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	157	3.5189	0.4818	2.7607	3.4606	5.3611	1.28
	2	157	2.9993	0.7137	1.0498	2.9175	4.8605	0.18

	3	157	2.9804	0.9139	1.2621	2.852	6.9416	1
	4	157	2.6391	0.8341	1.226	2.5442	5.8319	0.76
	5	157	2.2971	0.5973	1.068	2.202	4.805	0.71
	6	157	2.2714	0.6142	1.102	2.102	4.78	1.16
	7	157	2.2632	0.5312	1.134	2.202	3.925	0.54
	8	156	2.287	0.5603	1.168	2.202	5.105	1.28
	9	156	2.3635	0.6305	1.1535	2.269	4.871	0.83
	10	152	2.4077	0.6444	1.068	2.3055	5.103	1.2
	11	151	2.3485	0.699	1.034	2.205	5.839	1.57
	12	139	2.4993	0.8752	1.4506	2.237	7.107	2.31
	13	116	2.4864	0.7112	0.767	2.3525	5.672	1.58
	14	91	2.3486	0.5714	1.3385	2.1845	3.97	0.91
	15	78	2.4829	0.6936	1.4305	2.403	5.939	1.73
	16	57	2.4416	0.7507	1.268	2.369	4.5663	1.01

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.3747 sec and as shown in the table 5th queued vehicle mean discharge headway is 2.2971 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t test for 5th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
157	2.2971	0.5973	0.0477	(2.2029, 2.3913)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.3747$
Alternative hypothesis	H_1: $\mu \neq 2.3747$

T-Value	P-Value
-1.63	0.106

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

t test for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
157	2.6391	0.8341	0.0666	(2.5076, 2.7706)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.3747$
Alternative hypothesis	H_1: $\mu \neq 2.3747$

T-Value	P-Value
3.97	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for German to Kaliti Through movement.

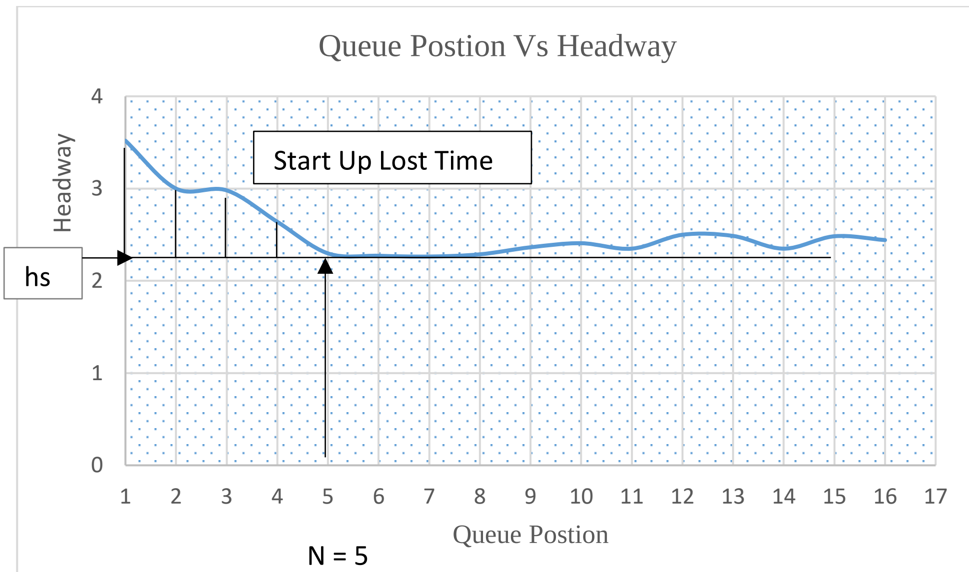


Figure C-9. Queue Position Vs Mean Headway for German to Kality Through Movement

Table C-10: Descriptive Statistics Queue Position Vs Mean Headway (sec) for Kaliti to German Through Movement

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	132	3.0218	0.4337	2.1224	2.9741	4.885	1.09

	2	132	2.6255	0.6706	1.1568	2.5272	5.4298	1.04
	3	132	2.3322	0.6814	1.3096	2.2698	4.0526	0.54
	4	132	2.1673	0.6934	1.0861	2.0695	4.3447	1
	5	132	1.9629	0.5219	0.634	1.8954	3.834	0.67
	6	130	1.9876	0.5349	0.9219	1.9525	3.77	0.9
	7	130	1.9341	0.4976	0.701	1.869	3.306	0.28
	8	129	1.8367	0.4276	0.917	1.833	3.003	0.32
	9	127	1.9425	0.4474	0.667	1.916	3.2378	0.06
	10	123	1.8791	0.4784	0.9909	1.834	3.504	0.6
	11	115	1.9516	0.5302	0.901	1.936	4.138	0.64
	12	103	1.8479	0.4609	1.034	1.75	4.167	1.58
	13	93	1.9301	0.5099	1.0826	1.835	4.004	1.09
	14	81	1.7611	0.4852	0.834	1.702	4.583	2.46
	15	67	1.7901	0.5171	0.5	1.769	3.583	0.41
	16	50	1.9249	0.6872	0.801	1.8293	4.8966	1.9
	17	36	1.8597	0.5375	0.9464	1.7358	3.07	0.55
	18	28	1.7913	0.5026	1.034	1.7425	3.17	0.63
	19	20	1.9511	0.3506	1.2632	1.9585	2.5288	-0.22
	20	15	1.647	0.411	0.634	1.735	2.235	-0.95

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 1.8748 sec and as shown in the table 5th queued vehicle mean discharge headway is 1.9629 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t test for 5th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
132	1.9629	0.5219	0.0454	(1.8730, 2.0528)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 1.8748$
Alternative hypothesis	H_1: $\mu \neq 1.8748$

T-Value	P-Value
1.94	0.055

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

T test for 4th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
132	2.1673	0.6934	0.0604	(2.0479, 2.2867)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 1.8748$
Alternative hypothesis	H_1: $\mu \neq 1.8748$

T-Value	P-Value
4.85	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Kaliti to German Through movement.

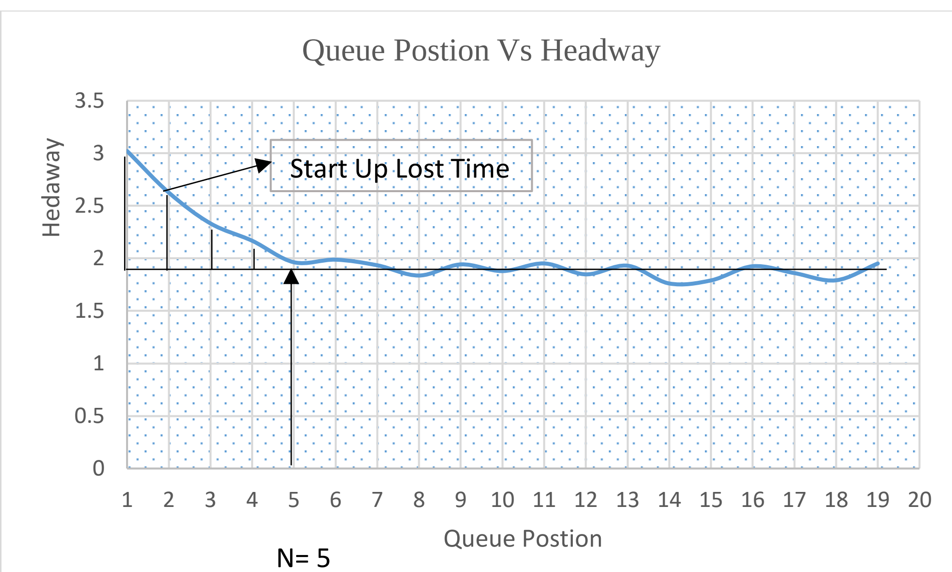


Figure C-10. Queue Position Vs Mean Headway for Kaliti to German Through Movement

Table C-11: Descriptive Statistics of Queue Position Vs Mean Headway (sec) for Kaliti to Sebeta Left Turn

Variable	Queue Position	Total	Mean	StDev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	54	2.8729	0.4549	2.0272	2.7809	4.4783	1.21
	2	53	2.4914	0.5301	1.424	2.4859	3.7938	0.29
	3	54	2.37	1.034	1.08	2.082	5.528	1.14
	4	54	2.268	0.991	0.969	1.868	5.585	1.51
	5	54	1.987	0.6662	1.083	1.909	5.108	1.95
	6	54	1.9794	0.5315	1.034	1.969	3.5	0.52
	7	53	1.9553	0.4842	1.084	1.902	3.6111	0.81
	8	50	1.7864	0.4687	0.867	1.7425	2.733	0.15
	9	48	1.9351	0.5071	0.935	1.917	3.17	0.19
	10	45	1.6875	0.452	0.889	1.569	3.334	1.14
	11	42	1.7169	0.4591	0.401	1.735	2.569	-0.5
	12	36	1.7825	0.4427	1.135	1.776	2.75	0.43
	13	34	1.7734	0.5607	0.7693	1.7595	3.404	0.51
	14	30	1.6549	0.4943	0.917	1.451	2.75	0.73
	15	23	1.856	0.621	1	1.833	3.081	0.47
	16	18	1.86	0.536	0.916	2.208	3.103	-0.38
	17	14	1.784	0.504	1.167	1.749	2.903	0.65
	18	11	1.832	0.435	1.167	1.833	2.667	0.42
	19	10	1.835	0.736	1.083	1.624	3.603	1.73

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 1.8283 sec and as shown in the table 5th queued vehicle mean discharge headway is 1.987 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t Statistics for 5th queued vehicles

t test for 5th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
54	1.9870	0.6662	0.0907	(1.8052, 2.1688)

μ: mean of Sample

Test

Null hypothesis	H_0: μ = 1.8283
Alternative hypothesis	H_1: μ ≠ 1.8283

T-Value	P-Value
1.75	0.086

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

T test for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
54	2.268	0.991	0.135	(1.998, 2.538)

μ: mean of Sample

Test

Null hypothesis	H_0: μ = 1.8283
Alternative hypothesis	H_1: μ ≠ 1.8283

T-Value	P-Value
3.26	0.002

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Kaliti to Sebeta Left Turn movement.

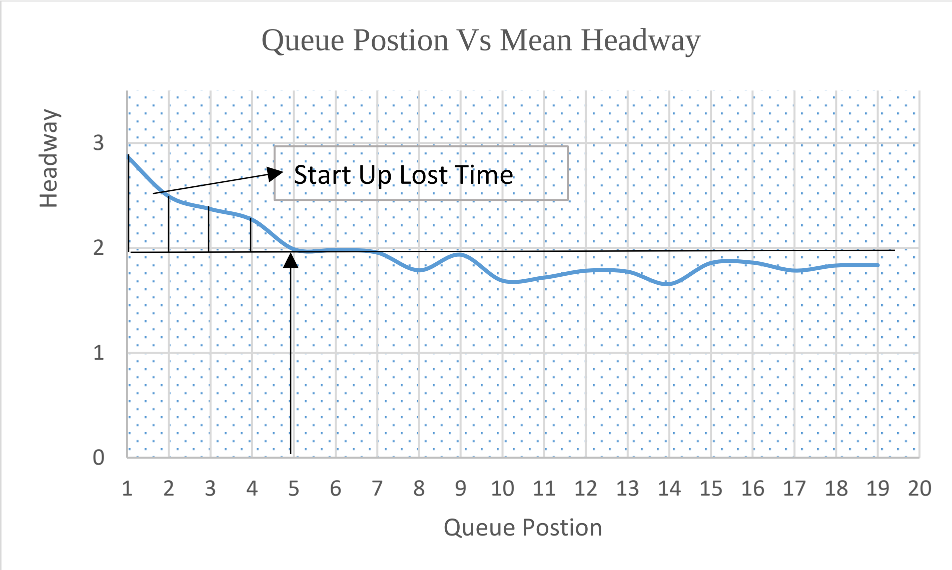


Figure C-11. Queue Position Vs Mean Headway for Kaliti to Sebeta Left Turn

Table C-12: Descriptive Statistics Queue Post Vs Mean Headway (sec) for Sebeta to German Left Turn

Variable	Queue Position	Total	Mean	St.Dev	Minimum	Median	Maximum	Skewness
Headway (sec)	1	94	3.2439	0.7285	1.9228	3.1002	5.601	0.82
	2	94	2.7782	0.7415	1.4667	2.8116	5.0154	0.52
	3	94	2.7092	0.7666	1.0009	2.5931	4.9068	0.45
	4	93	2.5356	0.7005	1.0587	2.5545	4.7247	0.46
	5	92	2.2293	0.6334	1.235	2.102	6.042	3.12
	6	92	2.2737	0.5206	0.8234	2.161	3.937	0.64
	7	92	2.2347	0.7098	1.168	2.1185	6.54	2.93
	8	92	2.1539	0.4908	1.135	2.1123	3.804	0.65
	9	92	2.18	0.4514	1.001	2.158	3.671	0.6
	10	89	2.1826	0.5454	1.108	2.083	4.6285	1.33
	11	86	2.2187	0.4805	1.268	2.1995	4.237	0.99
	12	80	2.1792	0.4578	1.368	2.1525	4.6272	1.92
	13	72	2.0586	0.4624	1.001	2.069	3.503	0.34
	14	62	2.1776	0.5275	0.868	2.147	4.0841	0.74
	15	51	2.1862	0.5509	1.333	2.102	4.905	2.5
	16	38	2.0344	0.3271	1.301	2.035	2.836	0.12
	17	28	1.9645	0.3609	1.201	1.9265	2.773	0.16
	18	23	1.8416	0.3513	1.135	1.833	2.469	-0.14
	19	15	2.046	0.527	1.168	1.938	3.236	0.45

Analysis of variance is conducted to locate the position of saturation begins. To conduct this, t – test is executed for the mean discharge headway of after the 4th vehicles and 5th queued vehicle.

The mean discharge headway of all the vehicles after the 4th queued vehicle is 2.1307 sec and as shown in table 5th queued vehicle mean discharge headway is 2.2293 sec. t- test is conducted to test that 5th queued vehicle mean discharge headway is the same or significantly different with the mean discharge headway of all the vehicles after the 4th vehicle

Ho: all the mean discharge headway after the 4th vehicle are equal

H1: all the mean discharge headway after the 4th vehicle are not equal

t Statistics for 5th queued vehicles

N	Mean	StDev	SE Mean	95% CI for μ
92	2.2293	0.6334	0.0660	(2.0981, 2.3605)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1307$
Alternative hypothesis	H_1: $\mu \neq 2.1307$

T-Value	P-Value
1.49	0.139

P value > 0.05 which means that no ground to reject the null hypothesis and entails that the mean discharge headway of all the vehicles after the 4th vehicle are the same to the mean discharge headway of the 5th vehicle means that the vehicles after the 4th vehicle are under saturation condition. Further, it is necessary to test the vehicle in 4th queued and the result of the test is shown herein below.

T Statistics for 4th queued vehicle

N	Mean	StDev	SE Mean	95% CI for μ
93	2.5356	0.7005	0.0726	(2.3913, 2.6799)

μ: mean of Sample

Test

Null hypothesis	H_0: $\mu = 2.1307$
Alternative hypothesis	H_1: $\mu \neq 2.1307$

T-Value	P-Value
5.57	0.000

P value is less than 0.05 which means that the vehicle in the 4th queue position mean discharge headway is significantly different with the mean discharge headway of the vehicles after 4th queued vehicle. To conclude this, the test indicates that the vehicles after the 4th queue position are under saturation condition while the first four vehicles are under start up lost time condition and the figure below shows the mean discharge headway vs queue position for Sebeta to German Left Turn movement.

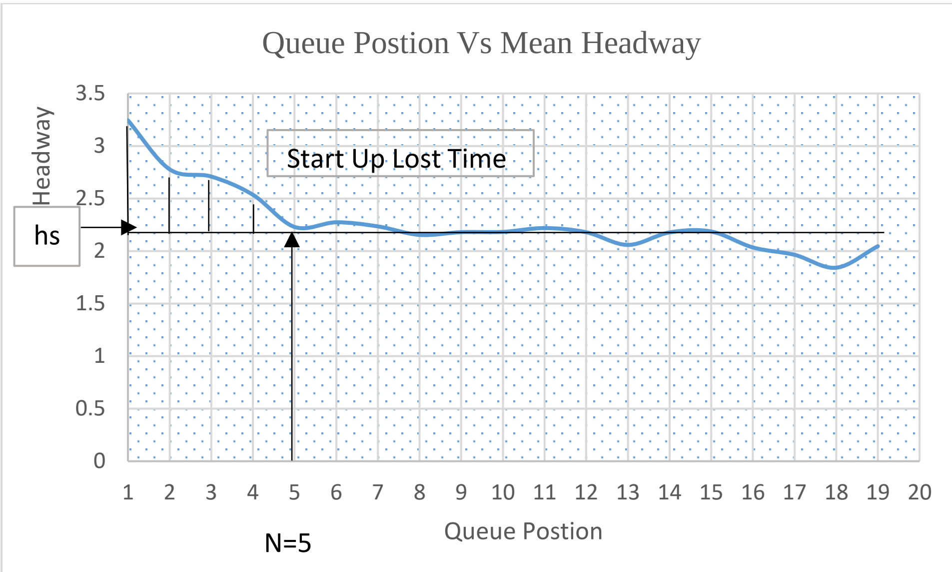


Figure C-12. Queue Post Vs Mean Headway for Sebeta to German Left Turn

APPENDIX D

Outlier Plot and Outlier Determination

Determining outliers in the data set

Before conducting the analysis, the data set should be tested for outliers and influential points. This could be done with Grubbs' test with the following Null and Alternate Hypothesis.

Ho: - All the data values come from the same normal population

H1:- Smallest or largest data value is an outlier

Significance Level: - 0.05

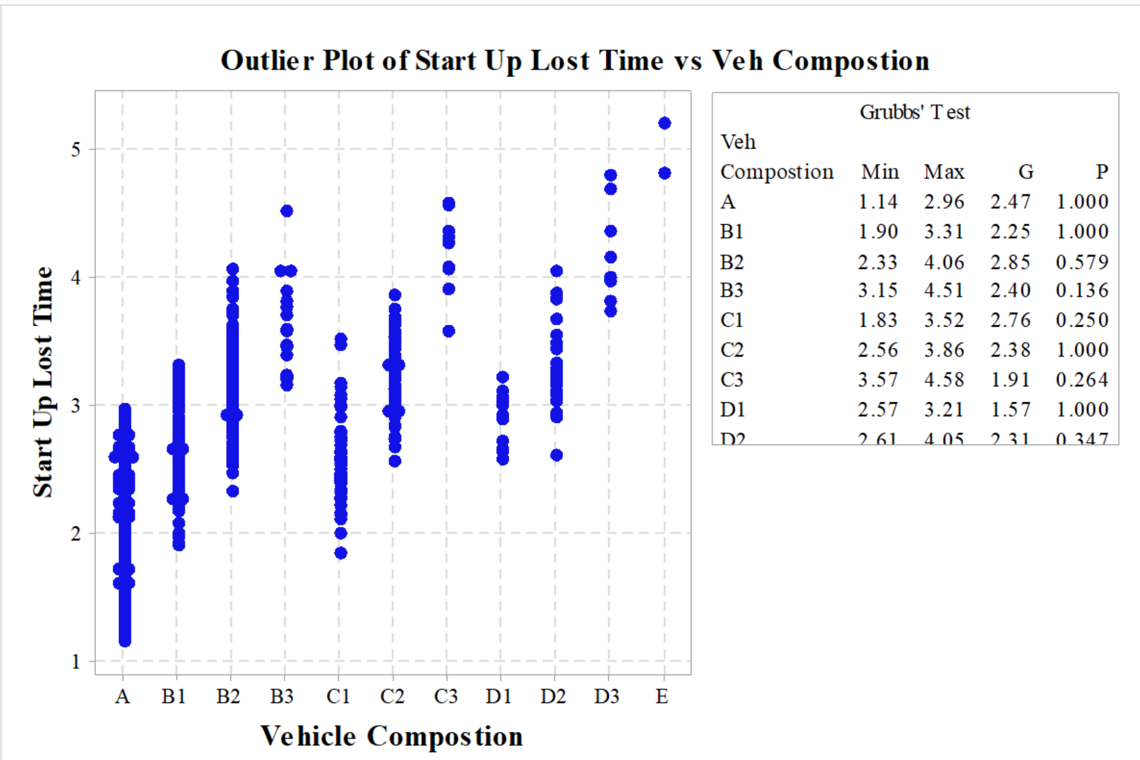


Figure D -1: - Outlier plot of all Vehicular Compositions

As we see from the outlier plot of Start-up Lost data for each vehicular composition, there are no observations pointed out as an outlier. The p-value from Grubbs test result in the above table for the all variables is greater than 0.05 (level of significance) which leads to the conclusion to accept the Null hypothesis which is All the data values come from the same population.

APPENDIX E

Probability Distribution Fitting for all Vehicular Compositions

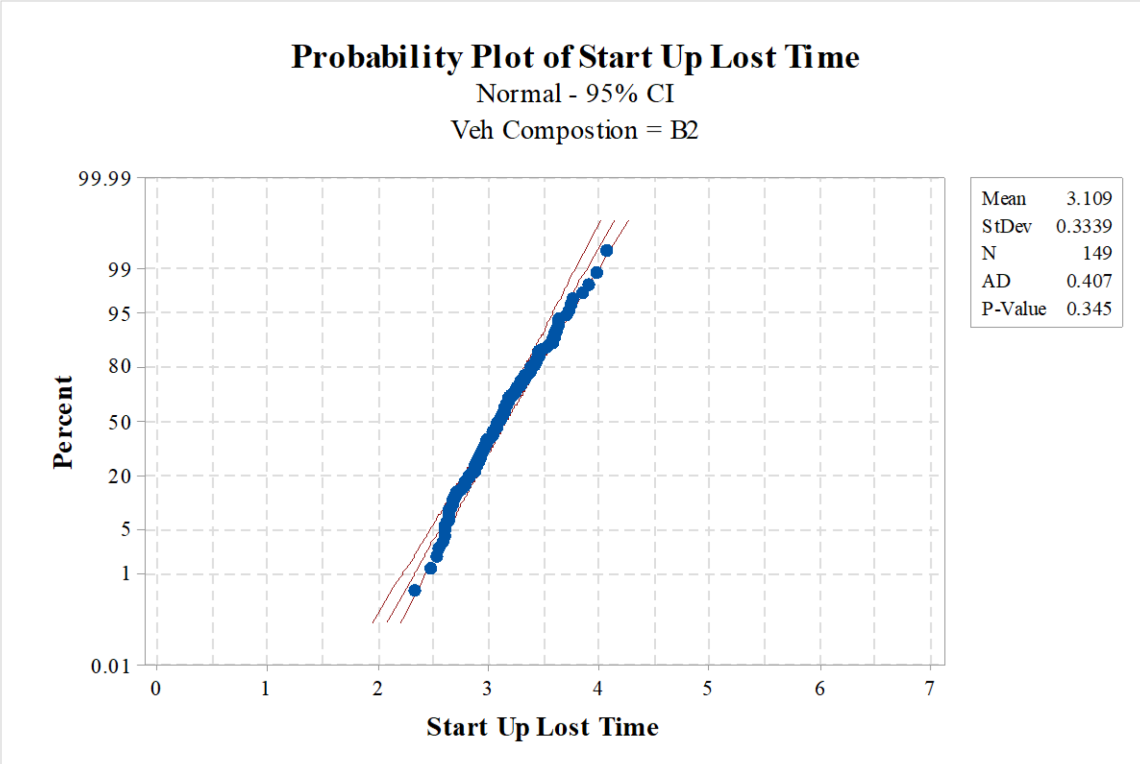


Figure E-1:- Normal Probability Distribution for Vehicular Composition B2

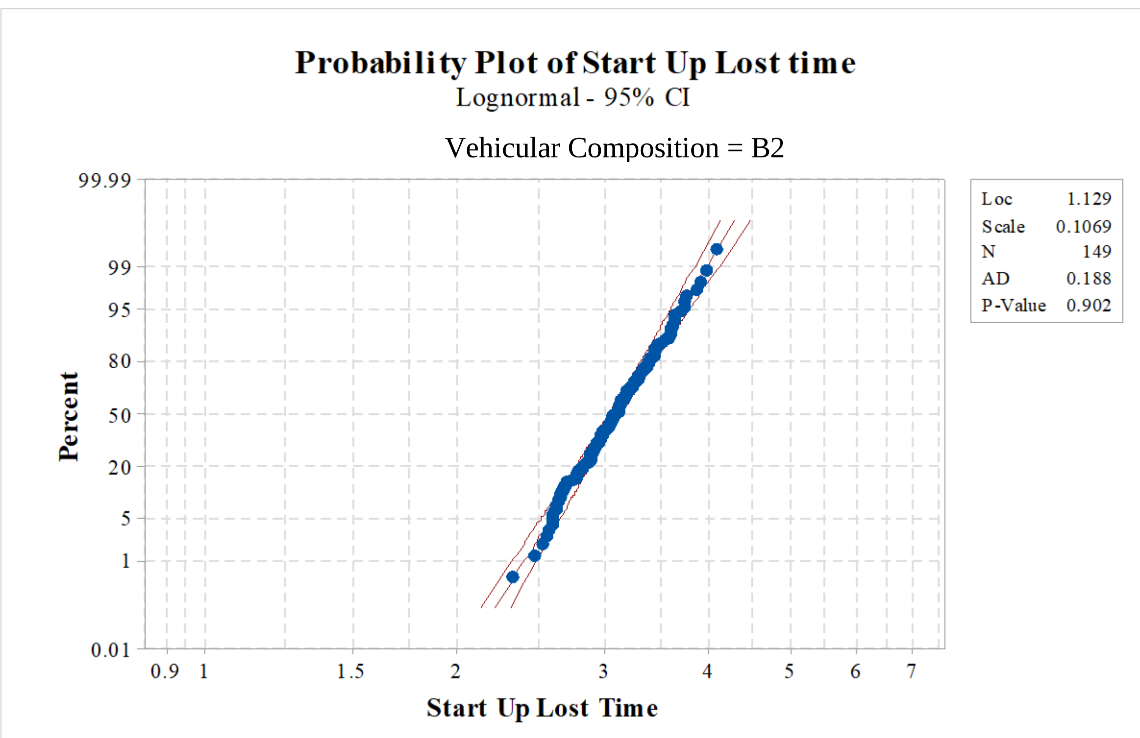


Figure E-2: Lognormal Distribution of Vehicular Composition B2

As indicated in the figure E-1 and E-2; the probability distribution of start Up Lost time for Vehicular Composition B2 can be represented by the normal and Lognormal Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, P value of Normal Distribution < Lognormal Distribution and AD value of Normal Distribution > AD value of Lognormal Distribution for Vehicular composition B2 as indicated in Figure A1. Hence, Lognormal Probability Distribution is the best fitted Probability Distribution for Vehicular composition B2.

Lognormal Distribution can be represented by the Equation

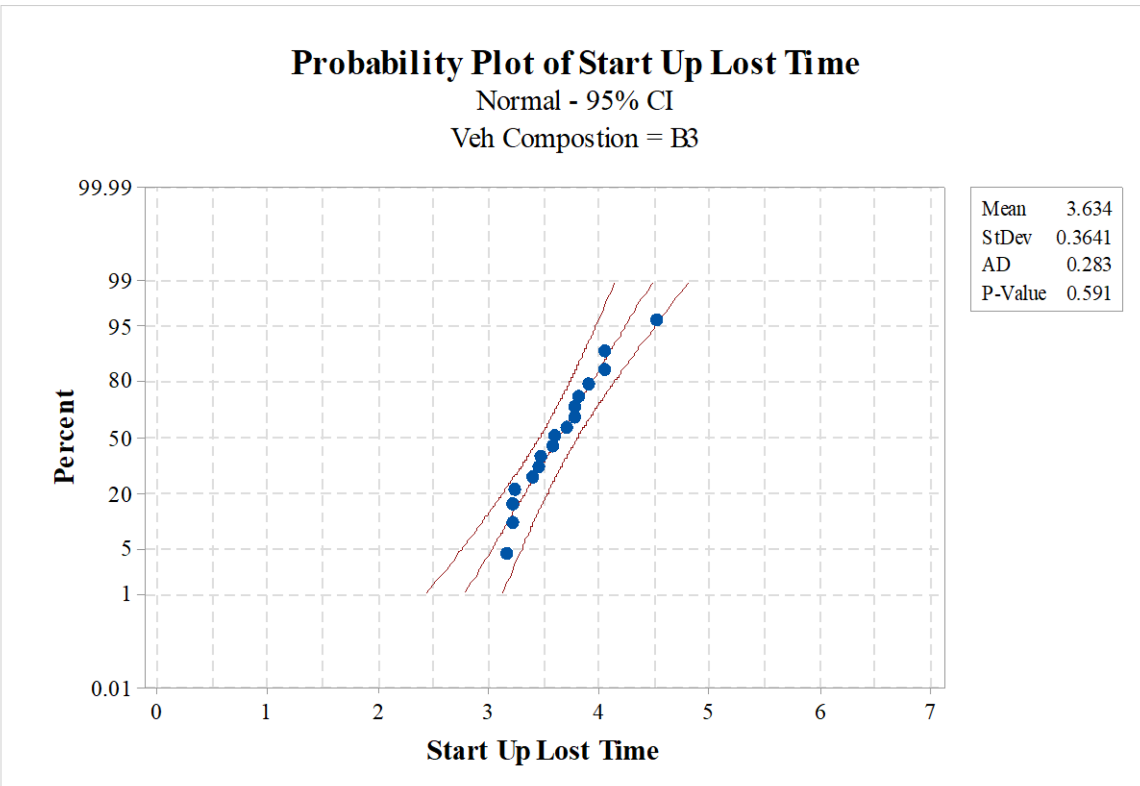


Figure E-3:- Normal Probability Distribution for Vehicular composition *B3*

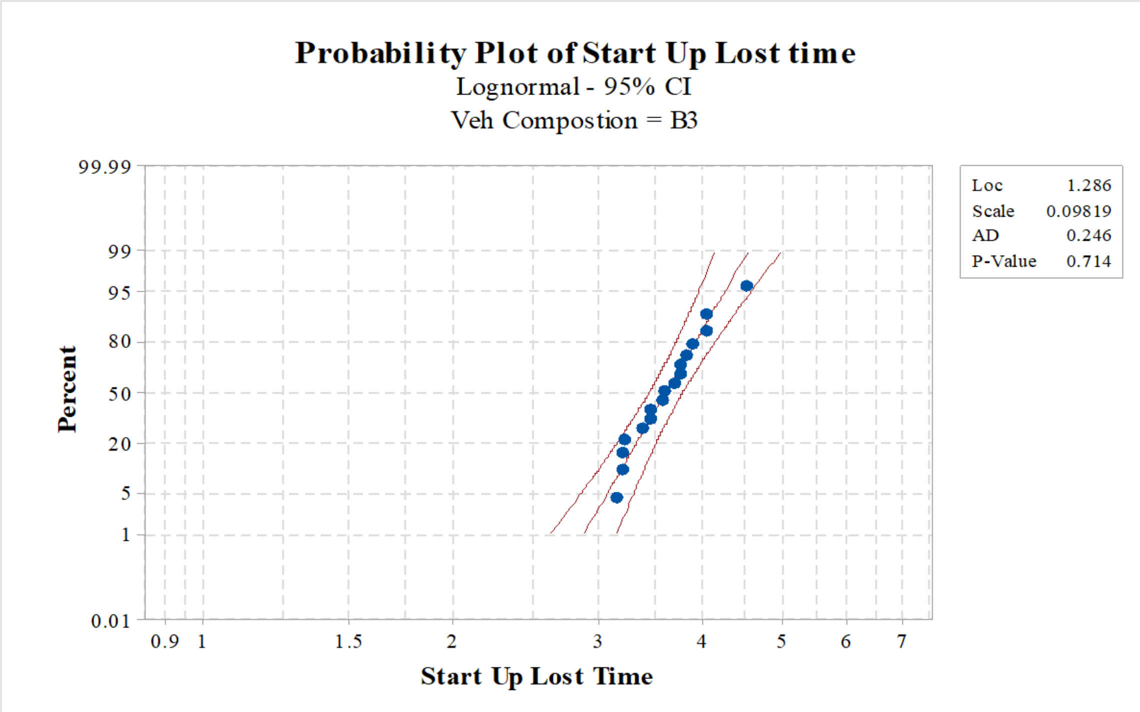


Figure E-4: Lognormal Probability Distribution of Vehicular Composition B3

As shown in the above figures, the probability distribution of start Up Lost time for Vehicular Composition B3 can be represented by the normal and Lognormal Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, P value of Normal Distribution < Lognormal Distribution and AD value of Normal Distribution > AD value of Lognormal Distribution for Vehicular composition B3 as indicated in Figure A1. Hence, Lognormal Probability Distribution is the best fitted Probability Distribution for Vehicular composition B3.

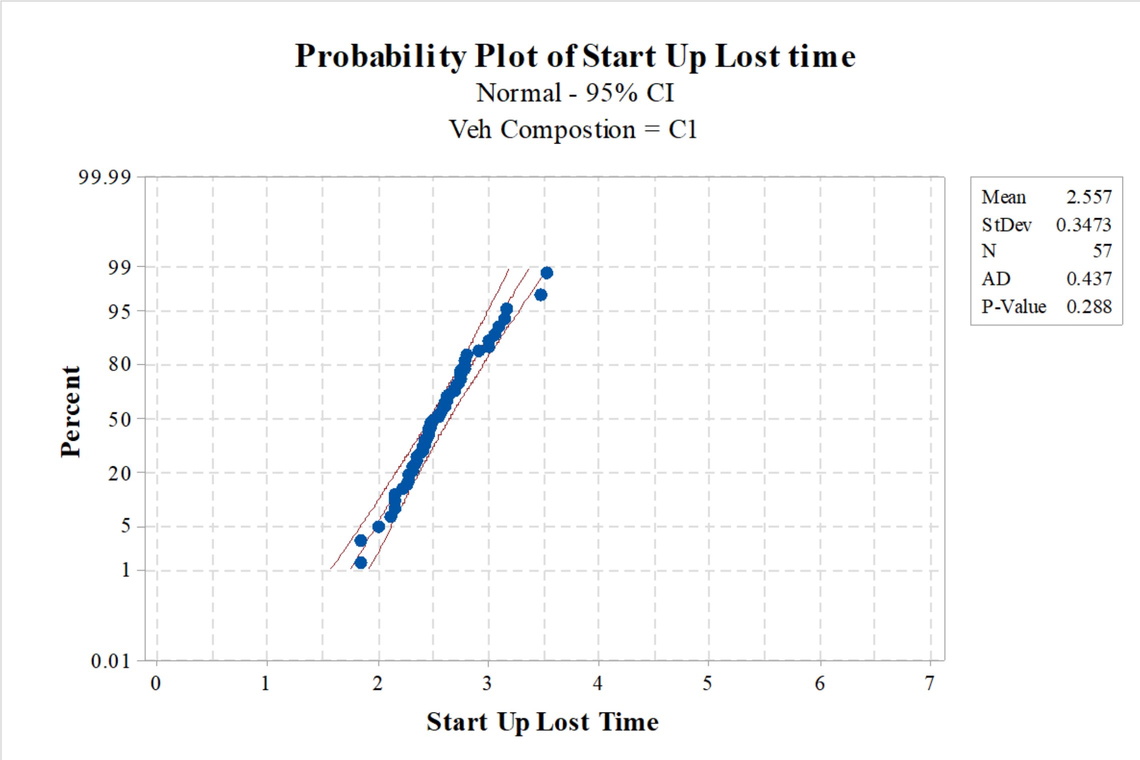


Figure E-5: Normal Probability Distribution for Vehicular composition C1

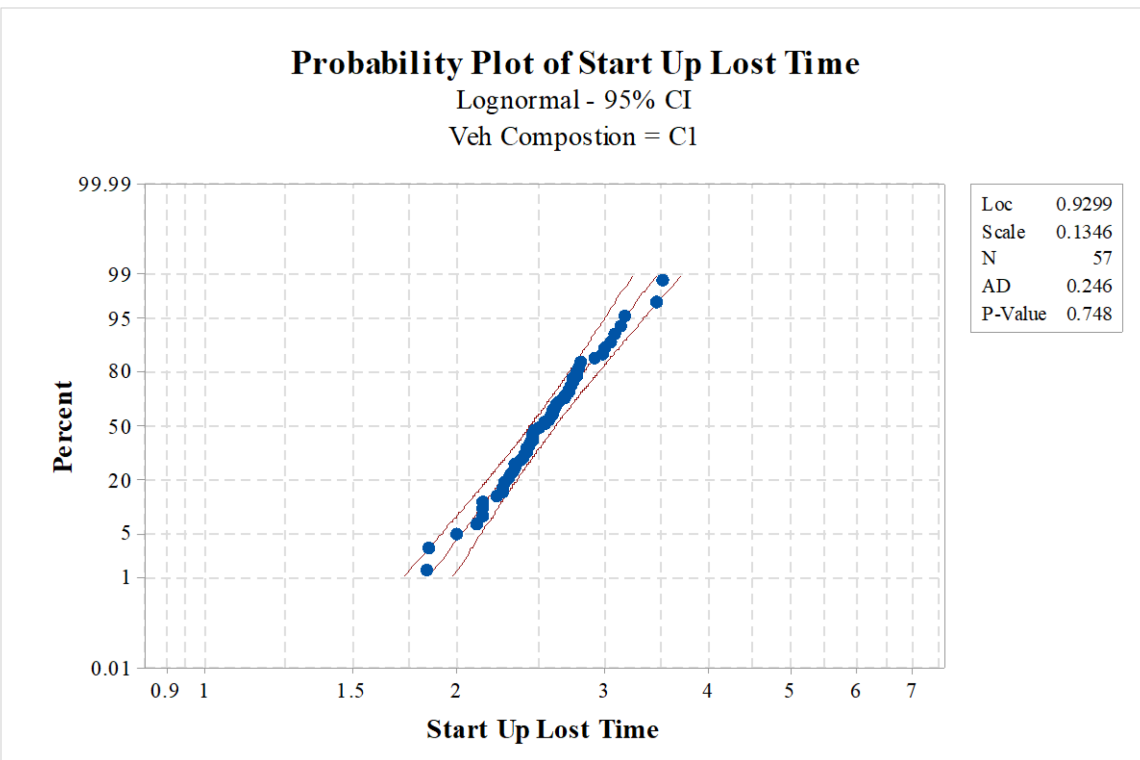


Figure E-6:- Lognormal Probability Distribution of Vehicular Composition C1

Figure E-5 and E-6 entails that, the probability distribution of Start Up Lost time for Vehicular Composition C1 can be represented by the normal and Lognormal Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, P value of Normal Distribution < Lognormal Distribution and AD value of Normal Distribution > AD value of Lognormal Distribution for Vehicular composition C1 as indicated in Figure E-5. Hence, Lognormal Probability Distribution is the best fitted Probability Distribution for Vehicular composition C1.

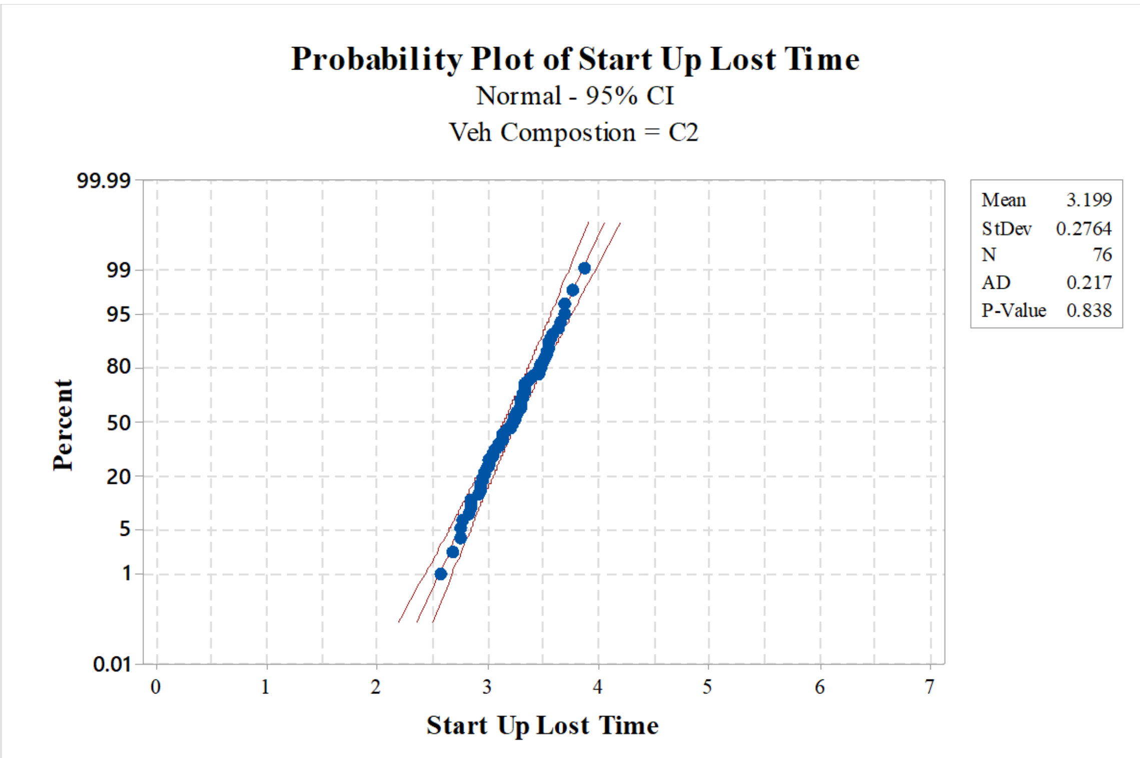


Figure E-7: - Normal Probability Distribution for Vehicular composition C2

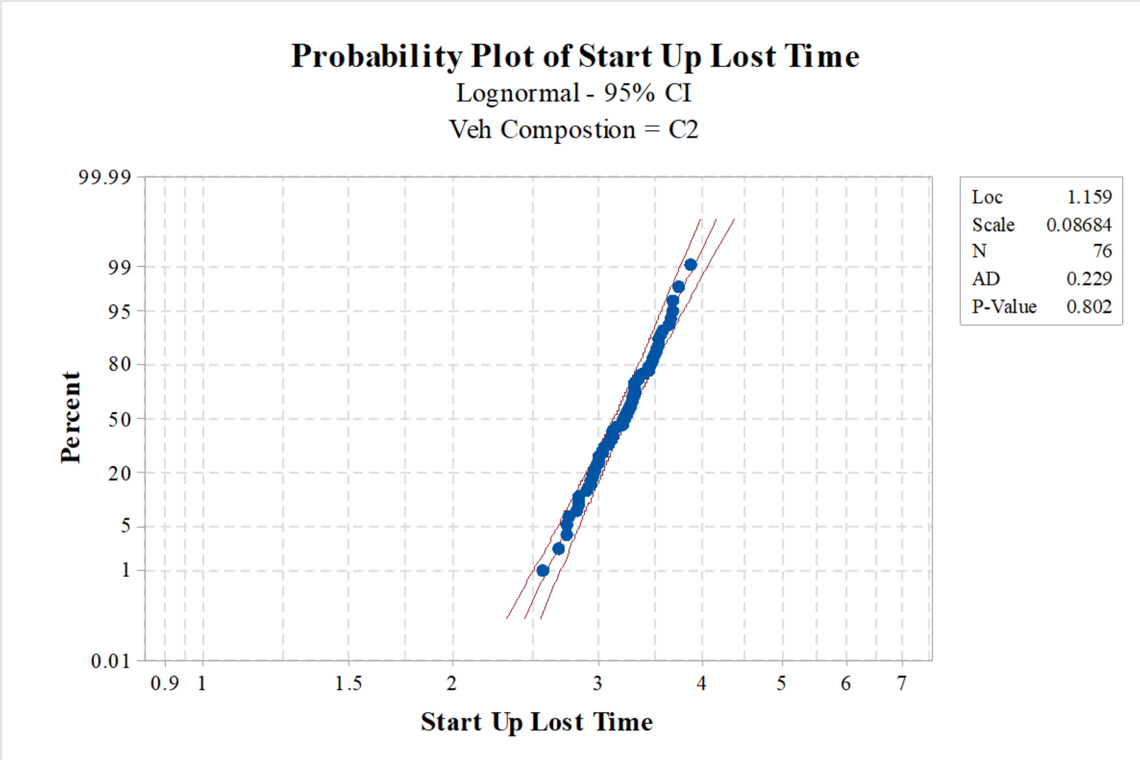


Figure E-8: - Lognormal Probability Distribution of Vehicular Composition C2

The probability distribution of Start Up Lost time for Vehicular Composition C2 can be represented by the normal and Lognormal Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, P value of Normal Distribution < Lognormal Distribution and AD value of Normal Distribution > AD value of Lognormal Distribution for Vehicular composition C2 as indicated in Figure E-7 and E-8. Hence, Lognormal Probability Distribution is the best fitted Probability Distribution for Vehicular composition C2.

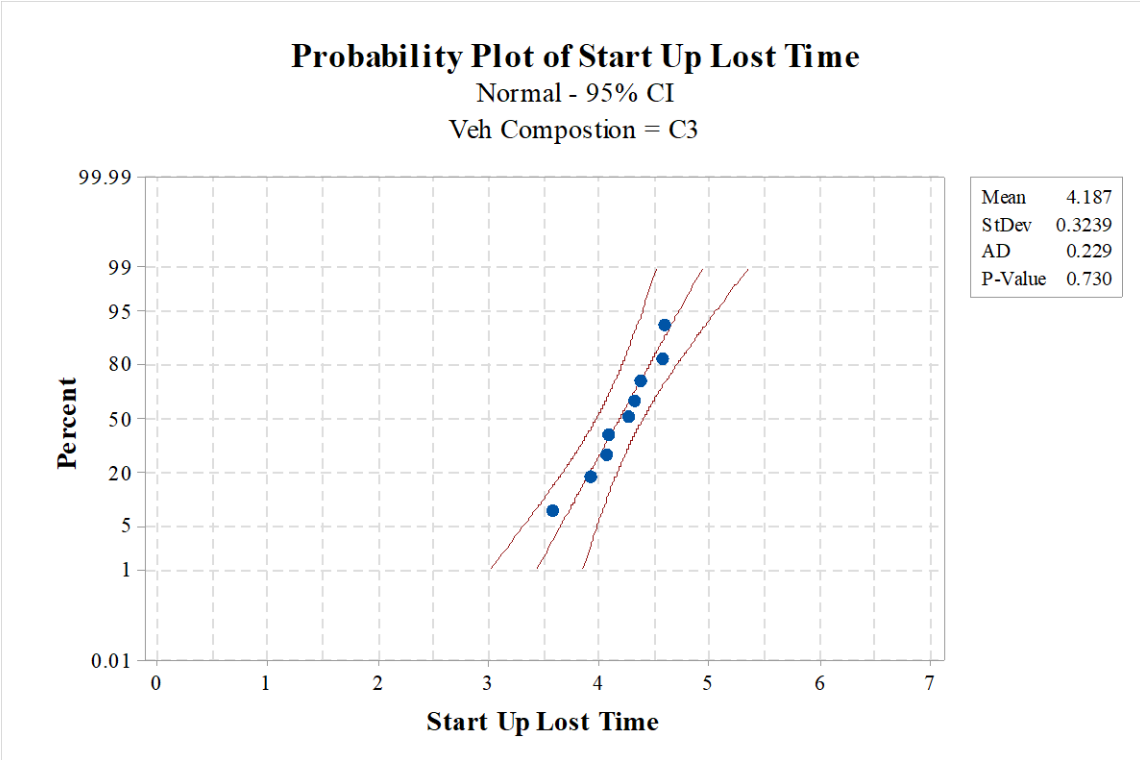


Figure E -9:- Normal Probability Distribution of Vehicular composition C3

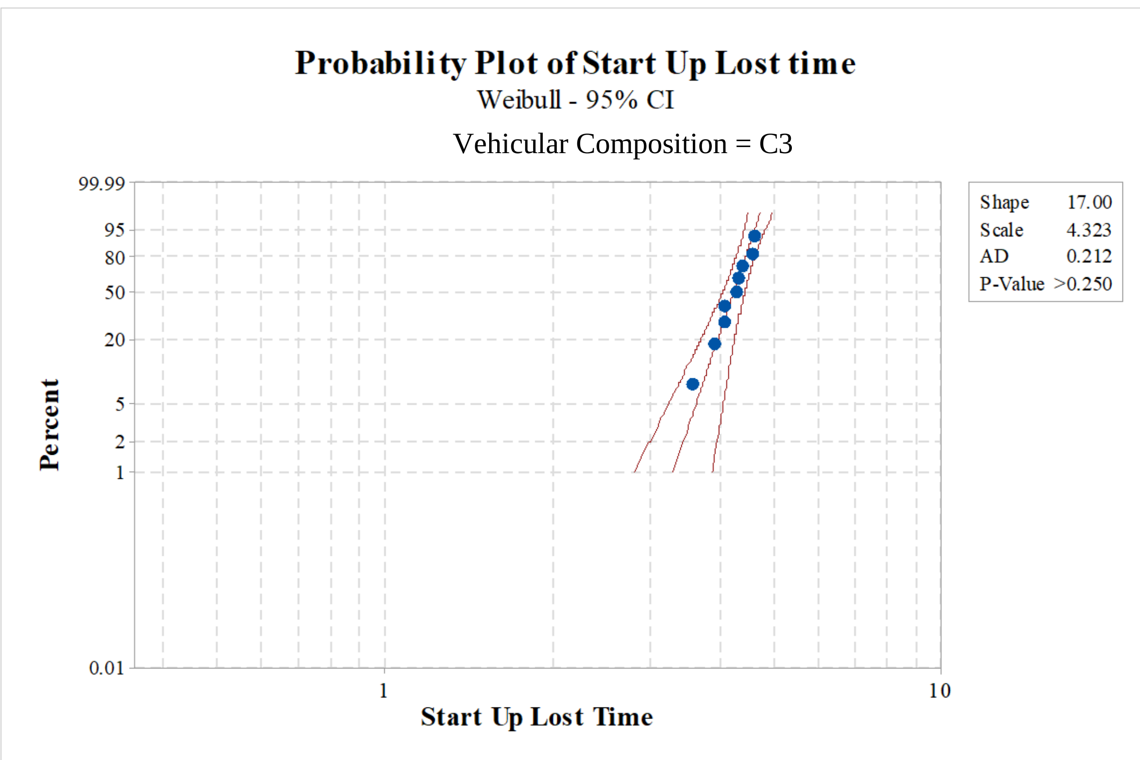


Figure E-10: Weibull Distribution of Vehicular Composition C3

The probability distribution of Start Up Lost Time for Vehicular Composition C3 can be represented by the normal and Weibull Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, AD value of Normal Distribution > AD value of Weibull Distribution for Vehicular composition C3 as indicated in Figure E-9 and E-10. Hence, Weibull Probability Distribution is the best fitted Probability Distribution for Vehicular composition C3.

Weibull Probability Distribution is Written as follows.

$$F(t) = \frac{\gamma}{\alpha}\left(\frac{x}{\alpha}\right)^{\gamma-1} exp\left[-\left[\frac{x}{\alpha}\right]^{\gamma}\right] \quad x \geq 0 \quad \text{.. } \textbf{Eqn E-1}$$

Where:-

F(t) is the Weibull distribution

γ is the shape parameter

α is the scale parameter

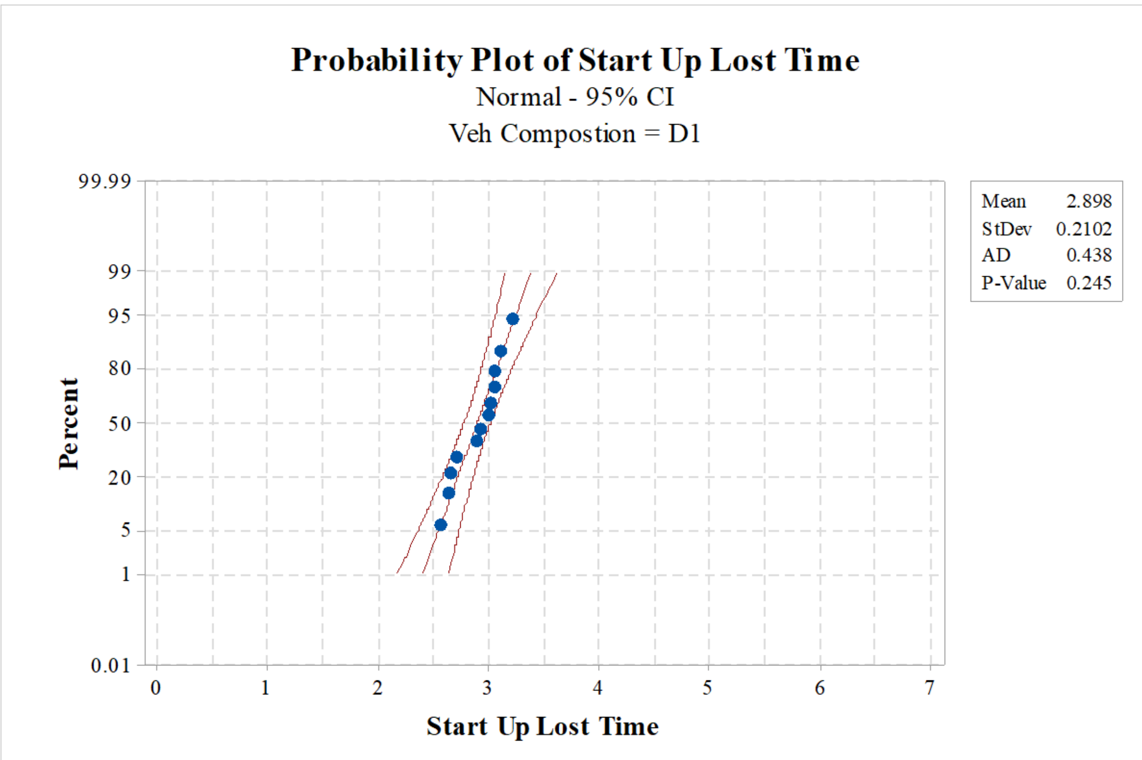


Figure E -11:- Normal Probability Distribution for Vehicular composition D1

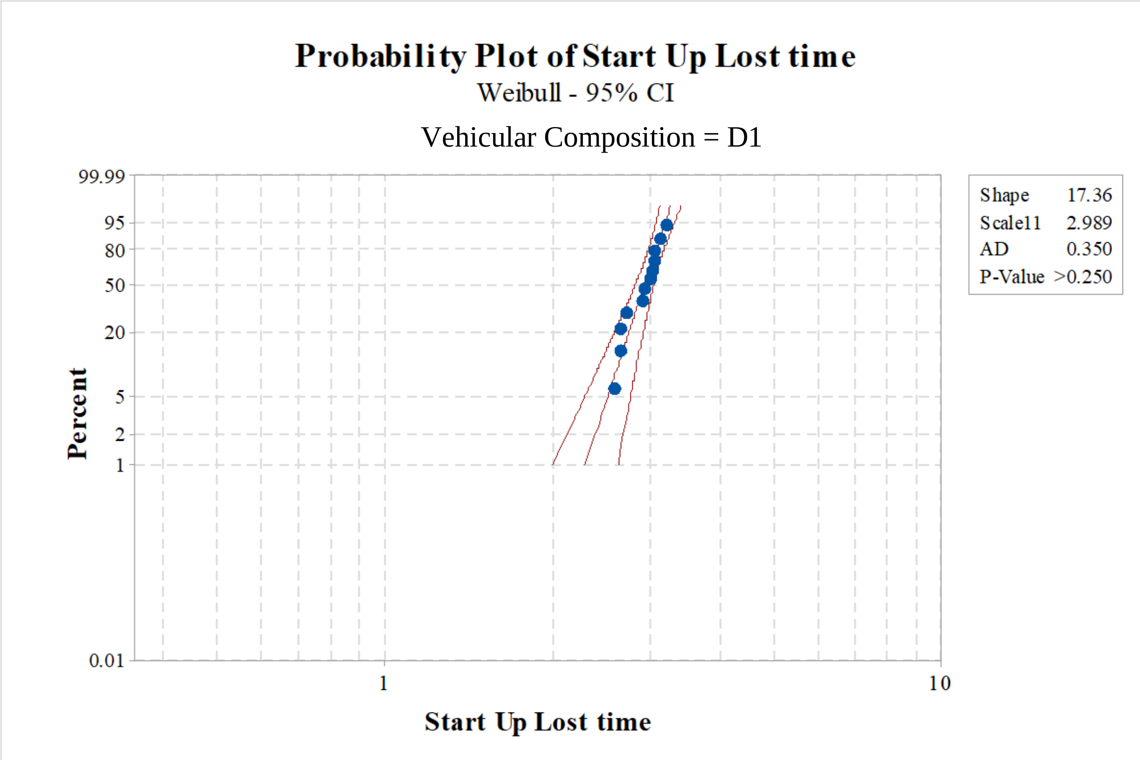


Figure E-12:- Weibull Distribution of Vehicular Composition D1

The probability distribution of Start Up Lost time for Vehicular Composition D1 can be represented by the normal and Weibull Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, AD value of Normal Distribution > AD value of Weibull Distribution for Vehicular composition D1 and also P value of Normal distribution < P value of Weibull Distribution as indicated in Figure E-11 and E-12. Hence, Weibull Probability Distribution is the best fitted Probability Distribution for Vehicular composition D1.

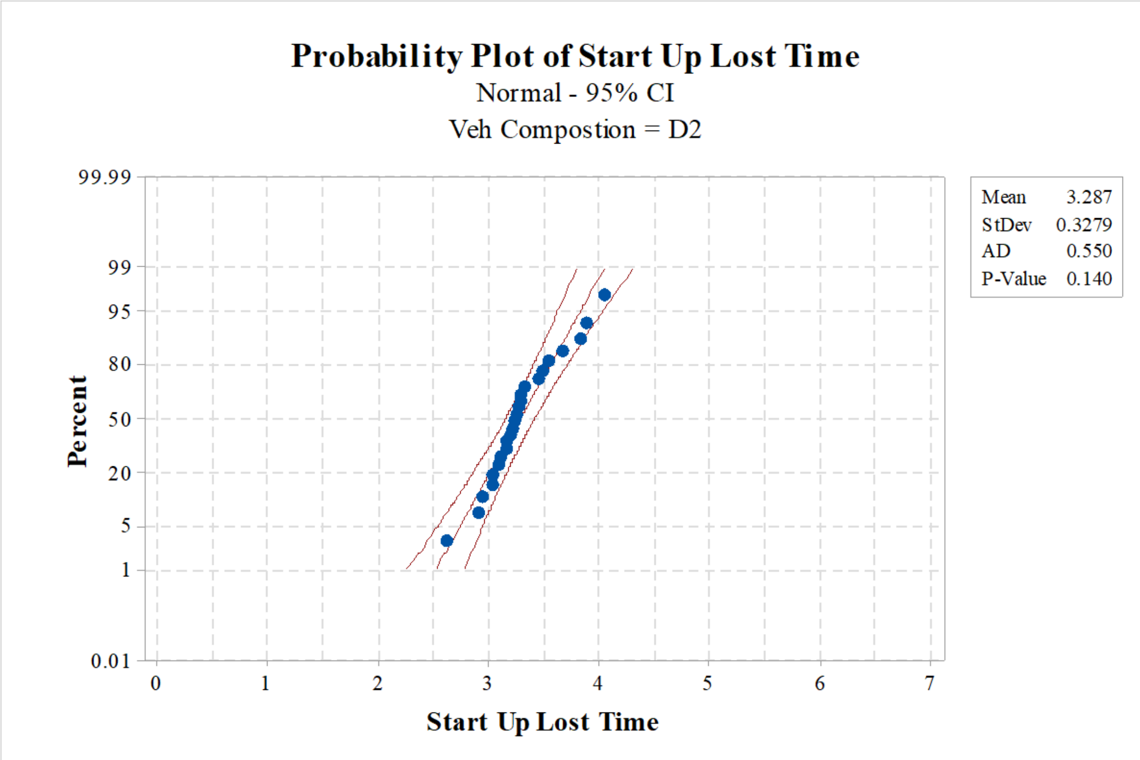


Figure E-13: Normal Probability Distribution for Vehicular composition of D2

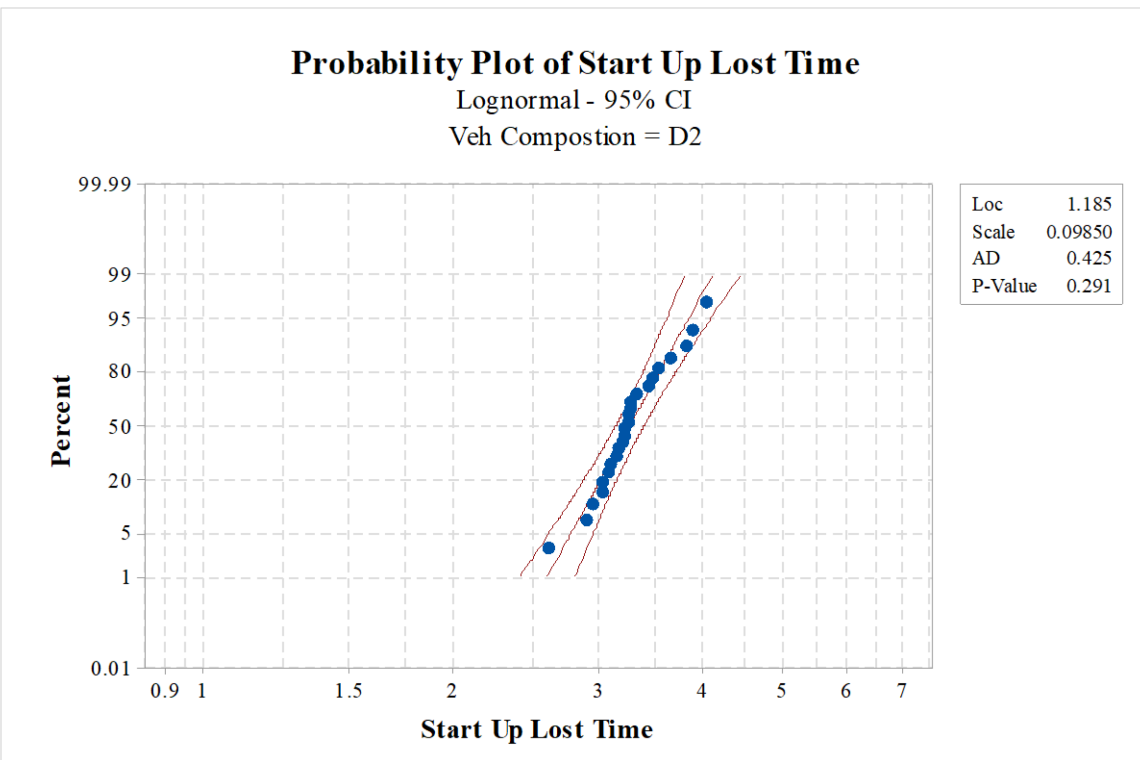


Figure E-14: Lognormal Probability Distribution of Vehicular composition of D2

The probability distribution of Start Up Lost Time for Vehicular Composition D2 can be represented by the normal and Lognormal Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, P value of Normal Distribution < Lognormal Distribution and AD value of Normal Distribution > AD value of Lognormal Distribution for Vehicular composition D2 as indicated in Figure E-13 and E-14. Hence, Lognormal Probability Distribution is the best fitted Probability Distribution for Vehicular composition D2.

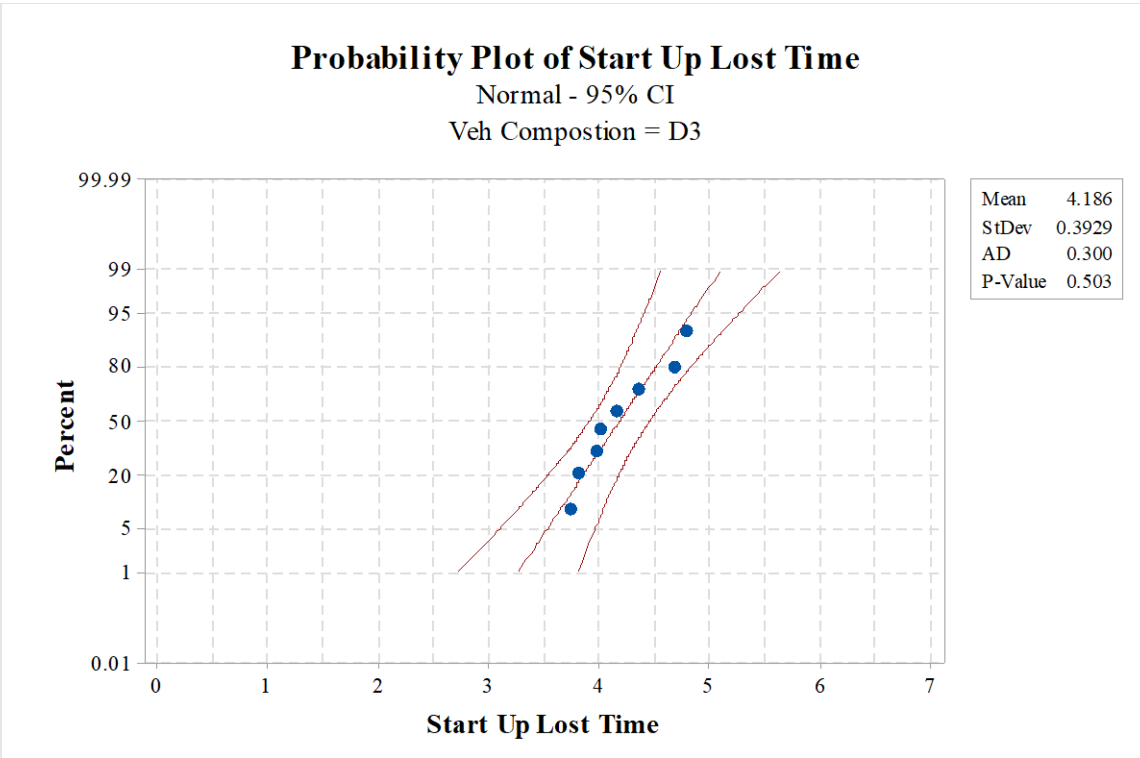


Figure E -15:- Normal Probability Distribution for Vehicular composition D3

The same to other vehicular composition, the probability distribution of Start Up Lost time for Vehicular Composition D3 can be represented by the normal and Lognormal Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, P value of Normal Distribution < Lognormal Distribution and AD value of Normal Distribution > AD value of Lognormal Distribution for Vehicular composition D3 as indicated in Figure E-15 and E-16. Hence, Lognormal Probability Distribution is the best fitted Probability Distribution for Vehicular composition D3.

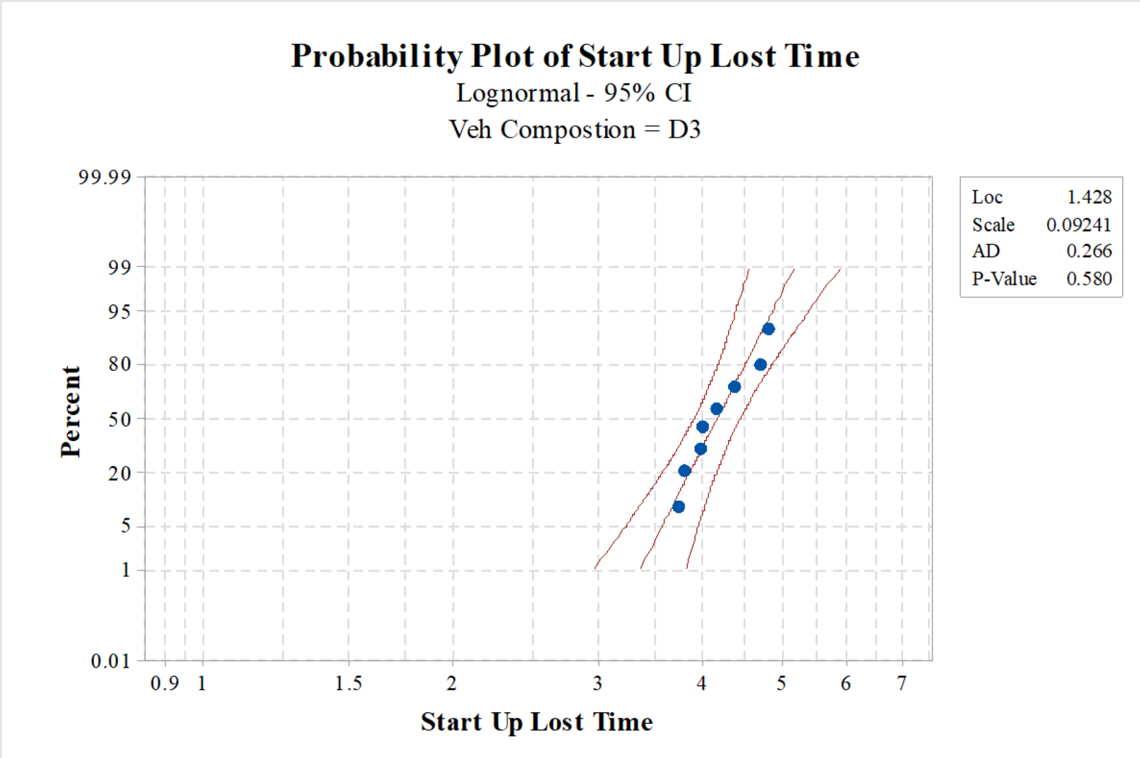


Figure E:16- Lognormal Probability Distribution of Vehicular composition D3

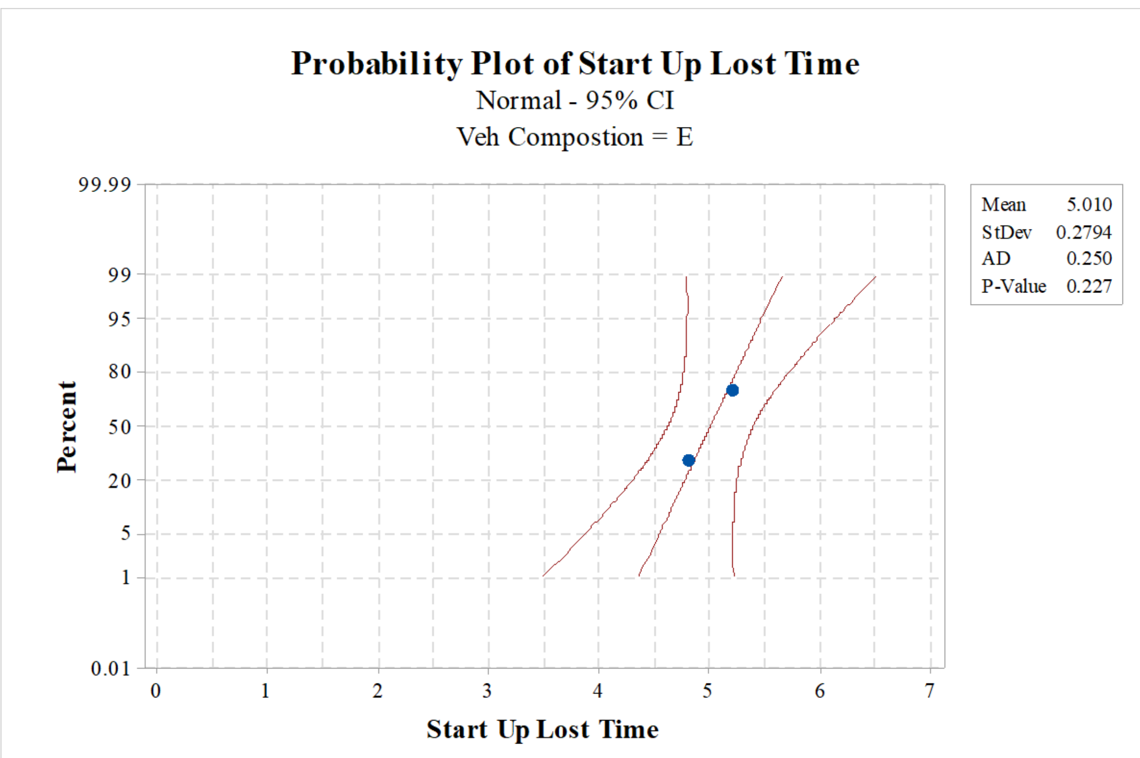


Figure E -17: Normal Probability Distribution for Vehicular Composition E

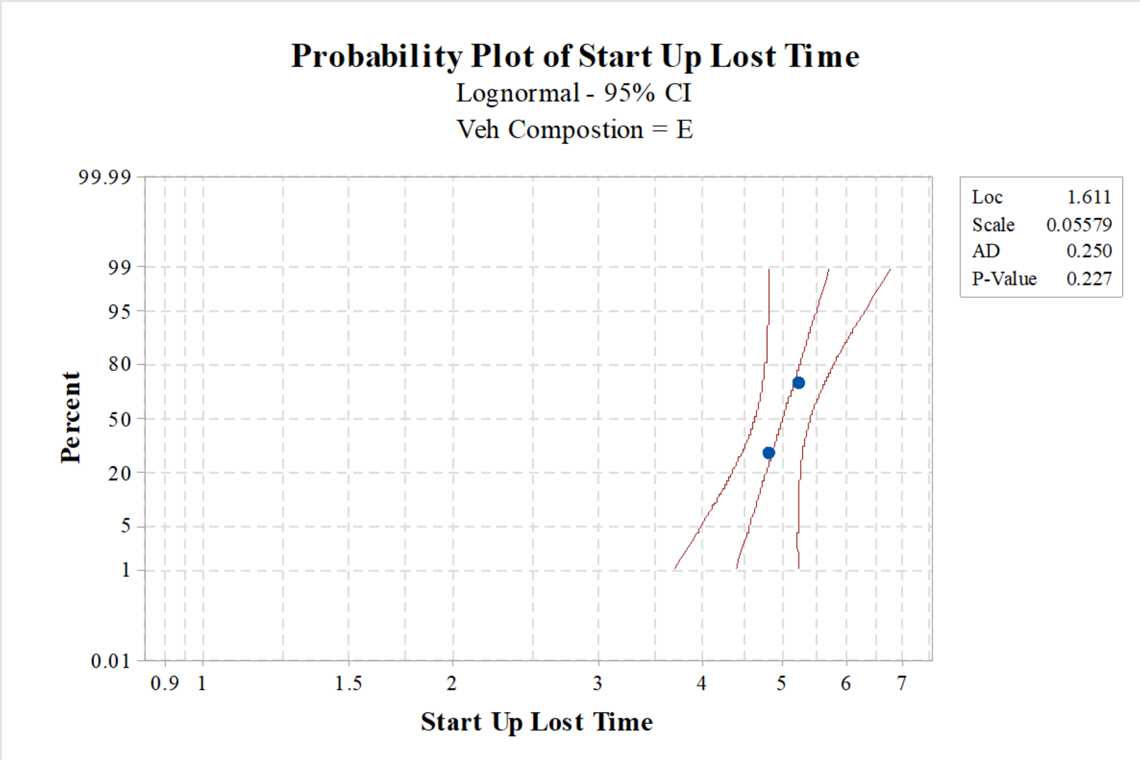


Figure E -18: - Lognormal Probability Distribution of Vehicular Composition E

The probability distribution of Start Up Lost time for Vehicular Composition E can be represented by the normal and Lognormal Distribution while it is better to determine the best fit probability distribution by comparing the Anderson Darling Value and P-Value. The probability distribution with Large P- value and small AD value is the best fitted probability distribution. Accordingly, P value of Normal Distribution < Lognormal Distribution and AD value of Normal Distribution > AD value of Lognormal Distribution for Vehicular Composition E as indicated in Figure E-17 and E-18. Hence, Lognormal Probability Distribution is the best fitted Probability Distribution for Vehicular Composition E.