ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Modeling the Impact of Traffic Factors on Queue Length at Consecutive Signalized Intersections

A Thesis Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science in Road & Transport Engineering.

BY: DEREJE NIGUSSIE DERESEH

ADVISOR: YONAS MINALU (Dr.Eng)

February 12, 2025

ADDIS ABABA, ETHIOPIA

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The undersigned have examined the thesis entitled "Modeling the Impact of Traffic Factors on Queue Length at Consecutive Signalized Intersections" presented by **DEREJE NIGUSSIE DERESEH**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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I certify that the research work titled "**Modeling the Impact of Traffic Factors on Queue Length at Consecutive Signalized Intersections**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Signature

Dereje Nigussie Dereseh

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Tables of Contents

List of Figures

List of tables

List of Acronyms

AADT	Annual Average Daily Traffic
HCM	Highway Capacity Manual
ITS	Intelligent Transportation Systems
LOS	Level of Service
OD	Origin-Destination
PSI	Paired Signalized Intersections
RT	Right Turn
LT	Left Turn
SUMO	Simulation of Urban Mobility
TAZ	Traffic Analysis Zone
UD	Uniform Delay
V/C	Volume-to-Capacity Ratio
g/c	Green time to cycle time ratio
OSM	Open Street Map
SFR	Saturation Flow Rate
STL	Start-up lost time
RMSE	Root Mean Square Error
MLR	Multiple Linear Regression
MAPE	Mean absolute percentage error

Abstract

Urbanization and increased vehicular traffic have led to significant challenges in managing traffic flow at signalized intersections, particularly in developing countries. This study focuses on optimizing traffic flow at consecutive signalized intersections by analyzing the factors affecting queue dynamics. The objectives include modeling the impacts of pedestrian volume, signal green time allocation and proportion of buses presence on queue length.

Using a simulation-based approach with SUMO software, the study modeled various traffic scenarios to understand the interactions between vehicles, pedestrians and signal timing. Key findings highlight that queue lengths are heavily influenced by signal timing inefficiencies, pedestrian crossing during vehicle gaps and the presence of proportion of buses, which exacerbate delays and reduce intersection capacity.

A multinomial linear regression model was developed to assess the impact of pedestrian volume, proportion of buses and green time on the performance of two consecutive signalized intersections. The regression model yielded significant results, with an adjusted R square value close to one, indicating a strong correlation between pedestrian volume, proportion of buses and green time and probability of queue formation. Factors such as pedestrian volume, proportion of buses and green time were found to be statistically significant in influencing the performance.

This research contributes to the field of urban traffic management by providing insights into optimizing signal timing strategies and addressing the unique challenges of mixed traffic conditions.

Key words: Consecutive signalized intersections, Queue length, Simulation of urban mobility (SUMO), Weibull distribution, urban traffic congestion.

Chapter One

1. Introduction

1.1. Background

In recent years, urbanization, economic growth, and increased welfare have led to a consistent rise in both urban populations and urban traffic, particularly in developing nations (Guojing Hu, 2020). This growing trend has necessitated significant changes in number of vehicles. In order to accommodate this number of vehicle cities use a signalized traffic control system at junctions of road network. Signals and other modern transportation advancements have enhanced travel in terms of safety, speed, reliability and convenience, they have also exposed to several negative consequences, such as traffic congestion, accidents and environmental pollution (Lele Zhang, 2020).

(Zhu Hong, 2021) Proposed Adjustment models which significantly reduce estimation errors for saturation flow rate (SFR) and start up lost time (SLT). He demonstrates the effectiveness of SFR and SLT in improving traffic flow predictions under varying conditions using experimental data on saturation flow rate (SFR) and start-up lost time (SLT) from signalized intersections. His focusing was mainly queue lengths and traffic conditions. Capacities of intersections can be reduced by downstream intersections (Hong Zhu H. N.-A., 2020). The study involves simulating traffic flows using three different models to evaluate downstream influence on upstream congestion.

Signalized intersections are key components of urban transportation systems, serving a critical role in managing traffic flow. However, these intersections often face challenges related to congestion and delays, which are particularly noticeable in dense urban areas. The interaction between upstream and downstream traffic flows is a major factor influencing intersection performance. (Zhu Hong, 2021)Downstream intersections can create queues that spill back into upstream intersections. Factors such as high traffic volumes, short spacing between intersections, varied vehicle compositions, and insufficient effective green time further contribute to these problems, complicating efforts to enhance traffic conditions.

Many researchers studied factors affecting the performance of consecutive signal intersections in urban area. (Haddad J., 2014) Concludes the optimal timing plan offsets can significantly enhance traffic flow and intersection capacity. The study investigates paired signalized intersections focusing on, the impact of timing plan offsets on movement capacity, considering factors like queue spillbacks and saturation flow rates at both upstream and downstream intersections. Intersection conditions, traffic volume data and queue storage capacity are essential for evaluating capacity expansion treatments. The study outlines a four-step evaluation process to assess the impact of downstream queues on upstream capacity and determine the feasibility of proposed treatments (YU Xin, 2012). Minimizing downstream queue spillback is key to achieving effective capacity expansion and improving traffic flow at urban intersections.

The downstream disturbances significantly impact delays at upstream intersections, underscoring the importance of control parameters in mitigating these delays (Ahmed K., 2005). The study involves developing macroscopic models to estimate delays caused by downstream disturbances and validating these models using microscopic traffic simulations. Traffic flow characteristics, geometric parameters and control parameters, including vehicle arrival rates, link lengths and effective green times at intersections, are analyzed.

The coordinated signal model significantly reduces total delay compared to the uncoordinated model under high traffic demand and closer intersection spacing, while the performance difference diminishes under low demand or greater spacing (Ronghan Yao, 2020). The study uses traffic demand data, geometric configurations of paired intersections, and parameters such as left-turn bay lengths and effective green times for each phase. Two optimization models are developed for paired intersections with uncoordinated and coordinated signals, followed by sensitivity analysis and simulation experiments using VISSIM software to evaluate performance.

Most researchers try to study the factors affecting two consecutive signalized intersections, but there is not enough study on the effect of vehicle type, pedestrian volume and signal timing on the performance of two consecutive signalized intersections. In the following chapters, we will embark on a comprehensive exploration of the signalized intersections and the myriad factors that influence its performance. We will consider the implications of these findings for traffic engineers ultimately aiming to propose a model for predicting the situation.

1.2. Statement of the problem

Traffic congestion and environmental pollution remain significant challenges in urban areas, contributing to longer travel times, increased economic costs and environmental degradation. Effective traffic management, particularly at intersections with consecutive traffic signals, is crucial for mitigating congestion and enhancing the efficiency of urban transportation networks. The performance of these consecutive signals, which regulate the flow of traffic in close proximity, is influenced by a variety of factors such as signal timing, traffic volume and road geometry.

The interaction between consecutive traffic signals and its impact on traffic flow remains an area of limited study. While factors such as signal timing, pedestrian volume and vehicle types are known to influence traffic conditions, their combined effects on queue lengths, delays and overall system performance have not been comprehensively analyzed. A deeper understanding of these interdependencies is essential for optimizing signal coordination, reducing congestion and enhancing the macroscopic efficiency of road networks. By addressing these gaps, researchers can develop more effective traffic management strategies that improve mobility and sustainability in urban environments.

This research aims to address this gap by identifying and analyzing the key factors that affect the performance of consecutive signals, focusing on how these factors influence queue lengths. By exploring these factors, this study aims to provide valuable insights into the optimization of traffic signal strategies, contributing to improved traffic management, reduced congestion and enhanced urban mobility.

1.3. Objective

1.3.1. General Objective

The main objective of the research is to investigate the factors affecting the queue length of consecutive signals intersections.

1.3.2. Specific Objective

- To investigate the effect of pedestrian volume at the time of vehicles green time and also its consequence on queue length.
- To develop a probabilistic prediction model for queue length by considering the impact of pedestrian volume, proportion of buses and green time.
- To recommend the effect of pedestrian volume, proportion of buses and green time in proposing possible applications for coordinating consecutive signals.

1.4. Research Question

Research questions have been developed in order to help us address the objectives of the study. These questions will help us focus on what we need to find out. They guide us to the solutions we seek.

- What are the key factors affecting the queue length of consecutive signals intersections in urban traffic corridor?
- How does pedestrian volume during the vehicle green time affect queue length at signalized intersections?
- How can a probabilistic model be developed to predict queue length while accounting for pedestrian volume, the proportion of buses and green time?
- What are the potential applications of pedestrian volume, the proportion of buses and green time in optimizing the coordination of consecutive traffic signals?

1.5. Significance of the study

The significance of this study lies in its potential to address key challenges in urban traffic management and contribute to the optimization of transportation systems. Traffic congestion and environmental pollution remain pressing issues in urban areas, and improving the performance of consecutive traffic signals is crucial for mitigating these problems. By identifying the factors that influence the performance of consecutive signals, this research aims to enhance traffic flow, reduce congestion and improve overall network efficiency.

The findings of this study will provide valuable insights for traffic engineers and urban planners in optimizing signal timing strategies, thereby reducing delays, improving vehicle throughput, and minimizing environmental impacts. By understanding how factors such as signal timing, traffic demand and road geometry interact, this research will help in the development of more efficient traffic management practices that can lead to smoother and more sustainable urban mobility.

In addition, this study will contribute to the broader goal of creating smarter urban transportation systems by providing a deeper understanding of how traffic signal coordination can enhance overall network efficiency. As cities continue to experience rapid urbanization and increasing traffic volumes, the need for intelligent and adaptive traffic management strategies becomes more critical. By analyzing the interplay between pedestrian volume, vehicle composition, and signal timing, this research offers insights that can be applied to optimize signal control, reduce congestion, and improve mobility. Furthermore, the study supports the development of data-driven and predictive traffic management models, which can be integrated into intelligent transportation systems (ITS) to enable real-time traffic adjustments. Such advancements help cities manage traffic flow dynamically, reducing delays, fuel consumption, and greenhouse gas emissions. Ultimately, by minimizing negative economic consequences such as productivity loss due to congestion and mitigating environmental impacts from vehicle emissions, this study contributes to the foundation of sustainable and efficient urban mobility solutions.

In addition from this study, the broader aspect will be bettered by this study in smarter urban transportation systems having a much better understanding of how traffic signal coordination can improve overall network efficiency. Nowadays, cities are witnessing the next-level rapid urbanization and increasing densities of traffic, making much more importance about intelligent adaptive traffic management systems. This research assigns the contribution of analysis on the interaction of pedestrian volume and vehicle composition with signal timing towards optimization of signal control, congestion reduction and mobility improvement. It further contributes to developing data-driven and predictive traffic management models that would feed into intelligent transportation systems (ITS) for real-time traffic adjustment. With such improvements, cities will dynamically manage their traffic flows whilst also minimizing delays, fuel usage and greenhouse gas emissions. Thus, the objective of this study on minimizing some negative economics like lost productivity due to congestion and mitigating environmental impact dipped from vehicle emissions is purely based on sustainable and efficient urban mobility solutions.

1.6. Thesis Outline

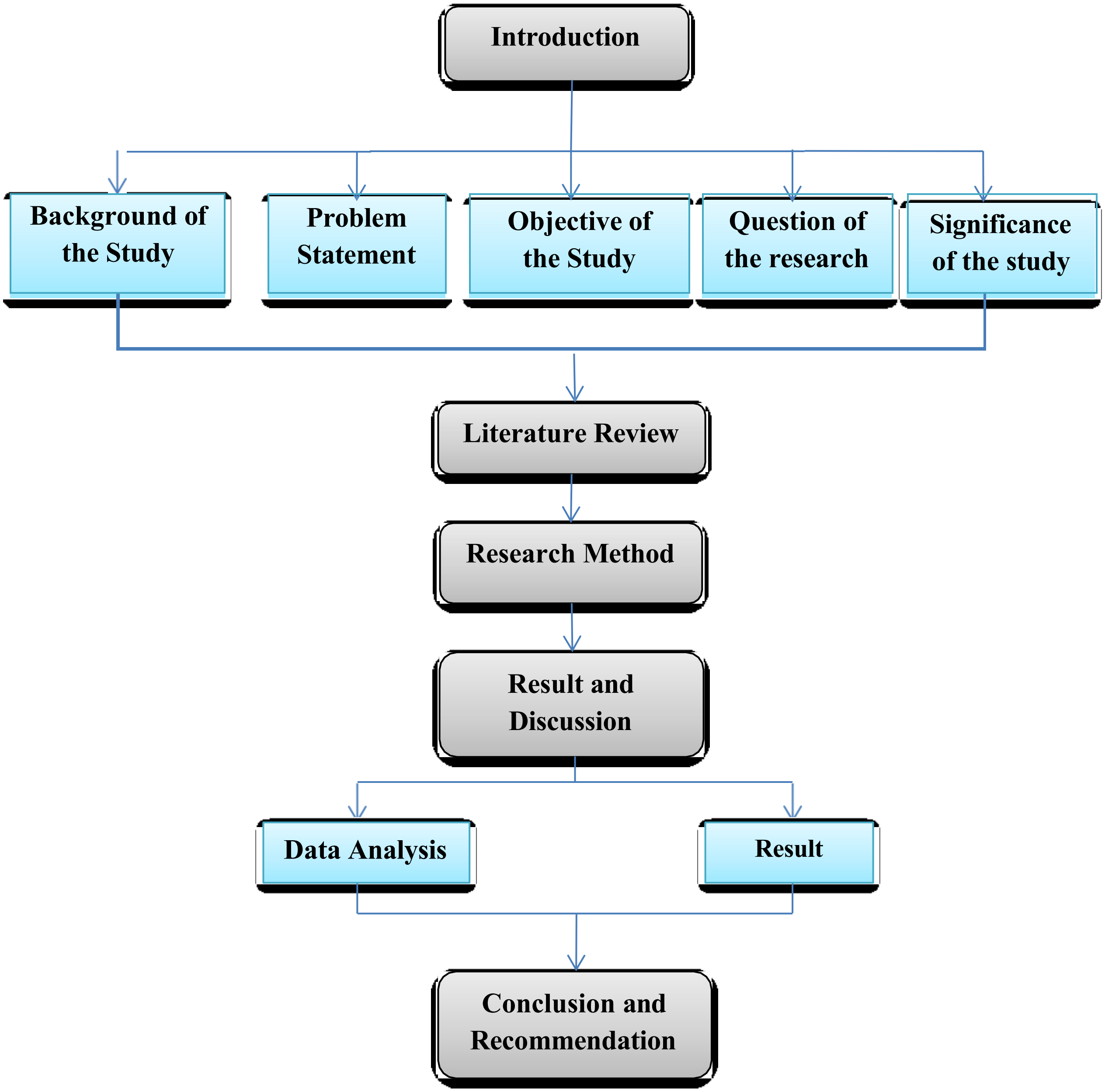


Figure 1: Thesis outline

Introduction: The background of the study, the problem statement, the objective, the hypothesis, and the significance of the study.

Literature review: Analyzes a variety of literature about the factors that affect consecutive signalized intersections. Definition and concept of the signalized intersections, historical development and key theories, previous works on factors affecting the signalized intersections, gaps in the existing literature, etc. would be reviewed.

Methodology: Discuss the methodology, data collection, analysis and simulation.

Result and discussion: At this part, a review of the research outputs including interpretation and theoretical implications, practical implications in transport planning and an implementable model have been discussed.

Data analysis: The data gathered from urban mobility simulations (SUMO) and weibull distribution are analyzed and organized to be suitable for modeling.

Conclusion and recommendation: The concluding section incorporates a summation of the key findings, emphasizes the relevance of these research findings, presents limitations of the study, discusses implications of the findings for policy change and suggests future research directions.

Chapter Two

2. Literature Review

2.1. Fundamental Theories of Traffic Flow

The basic theories of traffic flow become a very extensive and intricate form to enable analysis of understanding traffic dynamics through three broad spectra: macroscopic theories, microscopic theories, and mesoscopic theories. Each provides its own perspective of how traffic systems behave on various scales (Garber, 2009).

2.1.1. Macroscopic Traffic Flow Theory

Recognizing the phenomenon of traffic, macro traffic flow theory treats it as a single entity, unlike the micro approach that focuses on individual vehicles and analyzes their collective characteristics. This method adopts the gamut of important measures such as flow, density, speed, and so on to analyze segments of the roadway and develop a better understood whole concerning traffic phenomenology. With perfect understanding of this collective measurement, a transport engineer may derive really important relationships, such as the equation of flow-density, which is very rich for specifying and showing traffic behaviors and patterns under varying conditions. The proposed model is relevant to the predictions and planning of traffic systems since it defines the forms of congestion yet optimizes road designs for better performance in the traffic system. As macro view approaches, it is necessitated to take all traffic analysis to be improved at a larger scale, being the basis for developing plans that will entail improvement of traffic flow and thus reducing delays within highways and cities (Garber, 2009).

2.1.2. Microscopic Traffic Flow Theory

In a very contrasting way to the macro view of traffic flow, microscopic flow theory concerns itself with the minute behaviors of individual vehicles and their drivers, probing and going deeper into intricate processes that occur between the vehicles within a roadway. Microscopic models are pivotal in such developments as arising shock waves, which occur when one suddenly brakes and transmits the disturbance through the flowing stream of traffic (Garber, 2009). This theory would precisely analyze the movements of vehicles in terms of their critical actions, which are mainly integrated in all flow channels. There are cases of acceleration, deceleration, and lane changing. Microscopic traffic flow models will thus give a better understanding of complicated transport phenomena and more particularly, of shockwaves in their different forms they are the ripples that develop in the ongoing traffic whenever a vehicle brakes suddenly. Improved understanding of individual interactions would boost the development of more accurate and advanced traffic simulation models, which finally would improve the timing of traffic signals and, hence, the resultant safety and efficiency of the roads. Such detailed analysis would make possible intelligent and self-organizing transport systems that automatically tune themselves to the traffic condition in real time and thus increase the overall efficiency of the transport network.

2.1.3. Mesoscopic Traffic Flow Theory

The theory of mesoscopic traffic flow is a link between macroscopic and microscopic theories and serves as a bridge aiding in understanding the comprehensive analytical framework that takes groups of vehicles and their interactions on a larger scale than microscopic analysis and finer resolution than that under macroscopic analysis (Garber, 2009). This kind of modeling is most useful for traffic scenarios found in complex networks where behavior and interaction of clusters of vehicles are studied and not just on the individual vehicles or the whole traffic flow. The functionality of mesoscopic models simulates the same inherent non-linear traffic interaction in the settings that characterize cities in which different streams of traffic interact most of the time and very complicatedly. The inclusion of the basic elements found in both macroscopic and microscopic theories offers mesoscopic traffic flow theory a sophisticated and multidimensional view of traffic dynamics in order to allow the better formulation of better methods of dealing with traffic and well thought out infrastructure projects that help in addressing the different variations in need in urban transportation: the different variations in need with respect to urban transportation.

2.2. Traffic Congestion in Urban Areas

A big problem that exists within cities is that of traffic congestion, especially with regard to critical junction points. There are quite a number of causes as to what gives rise to traffic congestions, including but not limited to:

2.2.1. The Impact of Insufficient Road Capacity on Traffic Flow

A shortage of road capacity occurs when the number of vehicles using a road exceeds the space available for them, resulting in an extremely high imbalance between the amount of demand and supply (Tiaprasert K., 2015). Where it is unable to move freely and is forced to wait, such a condition results in a bottleneck and causes several increased delays. The picture of congestion becomes clearer when more vehicles enter the network, especially when the peak periods are on. It can ripple throughout the entire traffic system. This not only adds to frustration as people want to move from one point to another but also contributes to increased travel time and emissions.

2.2.2. Inefficiencies of Traffic Signal Systems

The uncoordinated traffic control systems have many shortfalls in adapting to the changing dynamics of present traffic situations. It becomes clearer during the peak hours, caused primarily by their inefficiencies, thus worsened by congestion. Most of these systems are pre-timed, meaning that the operations are based on fixed time schemes that do not take into consideration the real-time flow of the roadway into the traffic signal. These create unnecessary delays and longer waiting times at intersections. Such static time shows no awareness of current traffic conditions, enabling vehicles to queue up at the signal, which adds to the current bottlenecks in moving vehicles (Tiaprasert K., 2015).

2.2.3. The increase in urban traffic

Increased urbanization and population parallel massive traffic on city roads. These have become conditions where several vehicles are bumping into one another on roads with limited capacity, thus resulting in longer waiting times at junctions and increased time taken by commuters to reach their destinations (Tiaprasert K., 2015). The competition for space may create congestion, especially at major intersections where the traffic signals are unable to cater to the higher traffic due to the volume.

2.2.4. Limitations of Traditional Traffic Detection Techniques

Traditional traffic detection techniques like loop detectors have a few or many significant disadvantages that block effective traffic management. Such limitations include cost in installation and maintenance for these detectors and, therefore, are not a viable option very often for networks that are much more extensive (Tiaprasert K., 2015). They also have some limitations on site-specific data collection, which would result in a lack of complete visibility into the entire network.

2.3. Intersections

Intersections are crucial substructures of road systems projecting high complexity in that they are the points at which various roads intersect in order to allow vehicle and pedestrian traffic with possibly conflicting traffic patterns. These junctions have a major impact on the ability, productivity, and reliability of metropolitan transportation networks. Traffic signals, which are normally installed at the intersections, are timed devices that control traffic by switching on the green and red lights to prompt different phases in which conflicting traffic movements are avoided to promote the safety of all traffic entities. They include, among others, traffic volume, signal timings and layout of roads, which are fundamental in the analysis of traffic flow at an intersection (Boumediene A., 2009).

Figure 2: Traffic intersections

2.4. Traffic Signals

Traffic light systems are essentially part and parcel of an urban transportation system. It enables safe and efficient movement of vehicles, pedestrians, and sometimes cyclists at intersections. Right-of-way is usually granted at these intersections using phased lights like red, yellow and green indicating when each type of road user would have to stop, prepare to stop or proceed through the intersection. Traffic signals are hardly critical isolated entities; some parameters can be considered for evaluating their effectiveness; this includes signal timings, traffic volume and arrival patterns for both vehicles and pedestrians. Timed signals reduce delays, increase safety and improve traffic flow, while poorly timed signals can create traffic jams and frustration; for example, an adaptive signal control system uses real-time data from traffic patterns to be optimized by changing traffic conditions to modify the timing of the signal. Furthermore, one might find justifications in favor of traffic signals under perspectives of large-scale urban planning such as intelligent transportation systems, emissions reductions from smoother traffic flow and thereby, promoting multimodal transport. All these would ensure periodic and appropriate updates and maintenance to make traffic signals effective in meeting today's challenges and demands from traffic (Boumediene A., 2009).

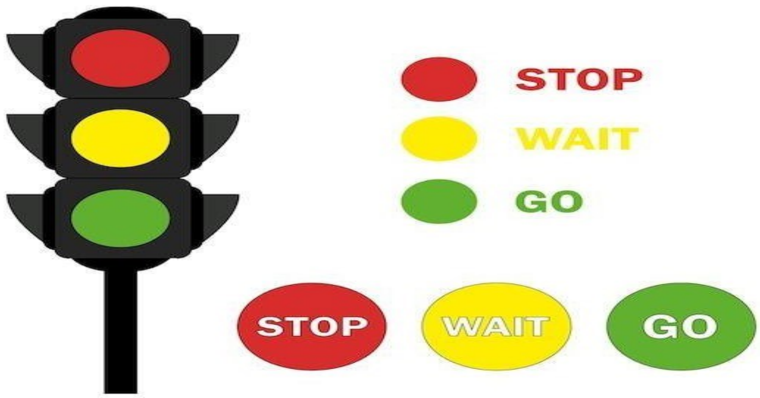


Figure 3: Traffic signals

2.4.1. Paired Signalized Intersections

Paired signalized intersections refer to the function of two closely spaced signalized junctions, exhibiting different operational characteristics related to proximity effects. Such intersections are characterized by queue spillbacks and upstream intersection blockage, complicating flow-capacity analysis. For instance, when queues build up at the downstream intersection, vehicles will be unable to discharge smoothly from the upstream intersection under green phasing, leading to diminished capacity and increased delays. The two intersections under review will require a specific timing plan to optimize the flow of traffic because, if poorly synchronized, the effective use of a given green light duration will include lesser movement through both intersections (Haddad J., 2014).

2.4.2. Isolated and Paired Signalized Intersections

Isolated signalized intersections function independently, meaning their traffic signals and operations do not interact with those of nearby intersections. This independence allows for straightforward management of traffic flow at each intersection without considering the effects of adjacent signals. In contrast, paired signalized intersections are located close to each other and can significantly influence one another. For instance, if a vehicle queue builds up at the downstream intersection, it can create delays at the upstream intersection due to the interaction of traffic flows. This relationship is critical in traffic management, as disturbances at one intersection can lead to increased delays and reduced capacity at the other. The paper highlights that understanding these dynamics is essential for accurate level-of-service and capacity analyses, as the delays caused by downstream disturbances must be factored into traffic models to ensure effective traffic control and planning (Ahmed K., 2005).

2.5. Car-following system

Car-following models are models that describe the behavior of vehicles following each other while on the road, such as during discharge queues at controlled intersections, including many parameters states like the speed of the lead vehicle, vehicle length, as well as spacing between vehicles, to replicate reality in-car driving behavior. According to the study, the modified Pitt car-following model assumes that parameters in queues discharge situations should be different from those in interactions or conditions of uninterrupted flow man-made complex heterogeneous traffic streams and lane changing behaviors (Cohen, 2002). Mathematically, the system is represented by equations that take into account the dynamics of vehicle acceleration-deceleration and define safety distance for following vehicles while employing speed adaptation along the route. This kind of application proves to provide information about the discharge headways and many other factors, such as free-flow speed and vehicle characteristics, affecting traffic flow, and thus is seen as an extension far beyond simpler models.

2.6. Traffic Counts

For the analysis and management of roadway systems, traffic counting is the most effective because it can be treated as a snapshot of vehicle/user volumes per specific location. Counts could either be 24-hour weekly counts or peak period movement counts for understanding traffic distribution throughout the day or the week (Curtis E., 2014). It's known by all that traffic counts do not stay constant and vary drastically from day to day as well as seasonally, hence necessitating continuous monitoring of counting points. The count data covers user types such as: motor vehicles, non-motorized, pedestrian and cyclers by intersection approach and movement type. Special users, like school-age children or frail elderly pedestrians, should also be recorded for consideration of traffic management strategy in respect of all road users.

2.7. Mixed Flow Conditions

Mixed flow conditions encounter various vehicles and road users running on the same pathway. Such circumstances create a complex and often chaotic environment with respect to traffic (Saha A., 2017). The said environment usually comprises different categories of vehicles, like cars, buses, trucks, motorcycles, bicycles and pedestrians, all with free and loosely defined lane boundaries.

2.7.1. Heterogeneity

Factors considered include the different types of vehicles in various shapes and sizes. In these conditions, the enormous differences in speeds, the varying ages, and types of vehicles are unfortunate features of mixed flow and can make it a little more complicated in delay for flows at intersections (Saha A., 2017).

2.7.2. Traffic management challenges

Predicting behavior among traffic flows becomes very complex, owing to the variables above; for instance, larger vehicles may block smaller ones and/or non-motorized users, say cyclists or pedestrians, may very well affect the movements of motorized vehicles (Saha A., 2017).

2.7.3. Effect on delay

The nature of traditional traffic models is usually those they take off from homogeneous, mixed traffic conditions and are of less predictive value when estimating delay. Developing countries could use such situations in either a more positive or negative way because traffic behavior might not resemble those of developed nations.

The effect of large or slow-moving vehicles such as buses and trucks on traffic flow efficiency includes increasing headways, reducing the capacity of an intersection, and causing platoon dispersion. Heavy vehicles usually accelerate and decelerate more slowly, contributing to a longer lag at signal changes, thus increasing intersection delays and reducing throughput. Motorcycles and bicycles travel through traffic differently and may be a source of safety or flow disruption for other vehicles. In any series of intersections, not only the performance of each individual intersection but also the progression of traffic from intersection to intersection is affected by the composition of vehicles. A large percentage of heavy vehicles in a platoon may therefore lengthen travel times and delays at the downstream intersections, thereby distorting signal coordination into longer queues. On the other hand, higher proportions of smaller, more nimble vehicles such as motorcycles may allow faster clearance times but result in more conflict at the intersections. Knowledge of vehicle composition is important in signal timing optimization for smoother progression and the increased effective capacity of the entire cross-section (Saha A., 2017).

2.8. Traffic Signal Synchronization

Synchronizing traffic signals is a method for the optimization of traffic flow through the coordination of the times at which signals operate along a street. The principal aim here is to create a "green wave," which allows cars to travel through succeeding intersections at a prescribed speed without stopping and delays (Martin, 1964). Changes in timing must be made for green and red signals so that, when a vehicle passes the first signal at the right time, it can proceed smoothly to the subsequent ones. Traditionally, this is done by having even cycle lengths for all signals, as well as green times set to maximize flow in both directions.

Effective synchronization therefore considers factors like speeds of vehicles, distances between intersections, and the exact time of a signal phase. While simpler systems like one-way streets can just as easily have their offsets done, two-way systems require more complex balancing schemes for efficient traffic flow in both directions. One such is the simultaneous system where all signals turn green at the same time. Although this would improve flow, it tends to lead toward speeding or extended stop-and-go periods. If traffic signal coordination is given a good run in urban areas, it will probably reduce congestion and, at the same time, enhance overall efficiency and improve the commuting experience of the drivers.

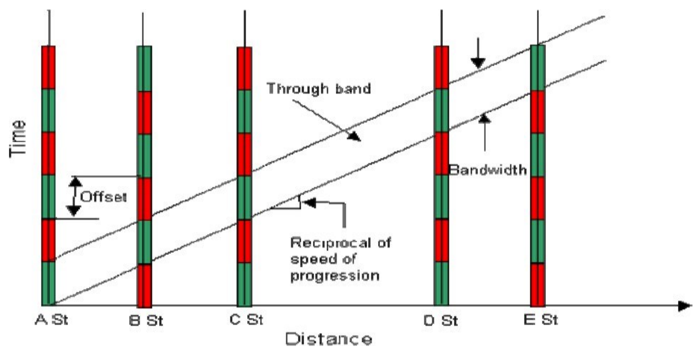


Figure 4: Time-space diagram showing important elements of coordination

2.9. Queuing Theory and Traffic Flow Analysis

Queuing theory is simply a branch of mathematics which helps study the manner and behavior of waiting lines also known as queues with respect to customer arrivals, service processes and characteristics of the total performance of the system (Gunes F., 2020). Its purpose is to provide a thorough understanding of all factors that affect the efficiency of service systems, especially in conditions such as the arrival rates and service times. Queuing theory has its applications in traffic engineering, especially in modeling the vehicle flow at signalized intersections. Queuing theory can predict future delays and helps improve signal timings. The basic assumption about the models is that arrivals can be treated as stochastic. The commonly used process to follow these is the Poisson distribution (Lian Xue, 2010).

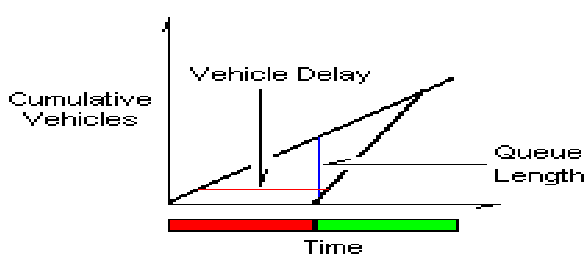


Figure 5: Queue theory

2.9.1. Queue Length at Signal Intersections

The length of a queue at a signalized intersection serves as a very important measure regarding traffic management since it influences the movement of vehicles as well as the performance of an intersection. Vehicles arrive at a signalized intersection over time, at rates either varying or constant; however, the ways in which they arrive day over day may be different. Their arrival takes place during the red phase of the signal, and hence, an increase in queues is caused. Queue maxima can then be computed as the initial overflow from the last cycle that arrived, plus those vehicles arriving during the effective red time. As it happens, it will matter for accumulation during times of heavy congestion because, among other things, it could add to delays and increases in the length of queues around an intersection (Joubert, 2002).

When the signal turns green, then the vehicles' discharge begins from the queue at saturation flow rate. The saturation flow rate will then be lower than the arrival rate for vehicles such that eventually the queue reduces. If, however, the arrival rate is equal to or more than the saturation flow, the queue will not completely discharge but will flow over to the next signal cycle.

2.9.2. Queue Discharge at Signalized Intersections

Queue discharge is the rate of exit of vehicles from a queue at a certain point of a signalized intersection after a signal turns green. This rate thus becomes essential for understanding the traffic flows and delays at intersections since typically the queue discharge rate will settle down after just a few vehicles have passed through the stop line, thus being termed the saturation flow rate. Recent studies have shown that the abovementioned discharge rates can vary dramatically, mainly caused by driver behavior, the length of the green signal and the specific characteristics of the intersection concerned. For instance, observations indicate that in many cases, the drivers will tend to speed up so as to clear the intersection before the green signal changes into red. This leads to an increasing queue discharge rate as the green phase progresses. Thus, the correlation established between queue discharge and green time shows that the longer the green cycle, the earlier the dissipation of the queue and thus overall delay calculations would be contingent upon these dynamics. These issues become even more important in developing appropriate models for estimating delays, which should reflect more accurately the patterns of signalized intersections in real-world conditions (Ranjitkar S. M., 2013).

2.9.3. Queue Length Estimation

This algorithm proposed can predict queue lengths reliably in a number of different situations, both where there is a saturated volume under pre-timed signal control and with actuated signal control, asserting it efficacy beyond any traditional input mechanisms. (Tiaprasert K., 2015)The queue length estimation model makes use of the location and speed data of individual vehicles as collected from connected vehicles and does not require any of the traditional inputs, specifically the signal timing and traffic volume, which are mostly very difficult to source. It consists of three steps: collecting vehicle data, determining if the vehicle is stopped or moving depending on the vehicle speed, and estimating using this data queue length, enhanced with the application of discrete wavelet transform for more accurate results.

2.9.4. Queue Discharge Characteristics

The data characterizing the headway, speed and acceleration of vehicles during the queue discharge were collected at various signalized intersections with an emphasis on the following broad vehicle categories: two-wheelers, three-wheelers and cars. (Pratim D.P, 2013)The acceleration characteristics were analyzed through a method in which non-uniform acceleration was used; this is where the relationship of acceleration and speed can be stated as $(dv/dt) = \alpha - \beta v)$; average values for coefficients α and β were computed for each type of vehicle. The pattern of gradual compression in headways whilst queue discharge led to an understanding of acceleration characteristics of vehicles released from stop at the green signal, contributing to better insights in the traffic flow dynamics under mixed conditions.

2.9.5. Shockwave Analysis

Analysis of shock waves is intended to express the pattern or behavior of traffic queues or delays in terms of changes in traffic density along a roadway. Unlike simple input-output models that discuss only the difference between total arrivals and departures, shockwave analysis takes the additional dimension of time into space and, thus, brings a broader understanding of how queues form and dissolve. It also traces the queues traveling along a congestion-induced reaction as a vehicle in effect gives the dynamics of traffic flow. This method can be very useful for the analysis of bottlenecks as it describes how queues develop and the consequences on travel times experienced by vehicles within those queues. This analysis is based on the interaction between vehicles and congestion behavior (Yi Ping, 2008).

2.10. Capacity and Critical Movement Analysis

Capacity and critical movement analysis is concerned with the remedial measures in traffic engineering, concentrating on the improvement of the flow of vehicles through signalized intersections (Koonce P., 2008). Capacity is defined as the maximum rate at which vehicles can pass through a specific point in an hour. It is also dependent on parameters such as saturation flow rate and timings for traffic signals. The exact capacity has to be known for timing signals properly, as it directly controls the effectiveness of that intersection to take the load of traffic demand. However, the critical movement analyzes those movements at an intersection that cannot happen simultaneously and require a maximum time to serve the demand. This analysis helps calculate the respective green times for the various movements.

2.10.1. Basic Operational Principles of Traffic Flow

The basic principles of operation with respect to vehicles travelling under conditions of signalized intersections are based on the functioning of traffic signals with respect to vehicle flow. When the signal is red, vehicles build up and form a queue, and flow does not exist during this time. Once the signal turns green, drivers wait for only a very short duration known as start-up lost time, generally of around 2 seconds while recognizing the signal and starting to move. After this initial lost time, the flow rate is quickly firmed to be saturation flow rate, which is defined as the maximum flow rate at which given vehicles can be pumped through an intersection under optimal conditions. It continues at this steady flow until the point at which the signal changes from green; some fraction of outflow will be passed during the yellow change interval, which is called yellow extension. The effective green time is the total green time minus start-up lost time and clearance lost time (the time that goes unused during both yellow and red intervals) (Koonce P., 2008).

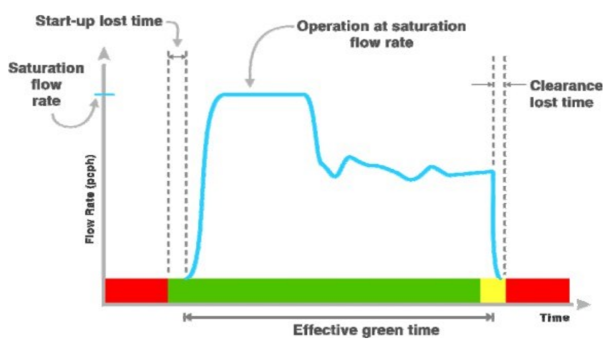


Figure 6: Typical flow rates at a signalized movement

2.10.2. Saturation Flow Rate

The threshold defined by the number of vehicles that can pass through a signalized intersection under ideal situations, with no interference caused by pedestrians and cyclists or blocking by parked vehicles (Koonce P., 2008). Saturation flow rate is defined by the time interval, termed as headway, within which two vehicles are released by the stop line. Mathematically, the saturation flow rate is stated as

$$\mathbf{S} = \frac{\mathbf{3600}}{\mathbf{h_s}}$$

Where

S = saturation flow rate

3600 = number of seconds per hours

h_s= saturation headway

Where (h_s) denotes the average gap time between automobiles during peak flow conditions (Pratim D.P, 2013): saturation headway. But achieving the actual saturation flow rate under real-world situations is very difficult for several reasons: heavy motor or short queues and fluctuating traffic demands. This study had mentioned that there is no constant value for a saturation flow rate; it will vary widely from one cycle to another based on the specific conditions at an intersection.

2.10.3. Lost Time in Traffic Signal Operations

In traffic signal operations, lost time refers to the time vehicles are prevented from moving owing to different signal phases. It includes periods of inactivity at the beginning of each green phase and time taken during yellow change and red clearance intervals, which are otherwise not usable by vehicles. The computation of total lost time is imperative because the capacity of any intersection is computed after deducting that amount from the cycle length to estimate effective green time available to vehicles. The highway capacity manual prescribes a default lost time of 4 seconds per phase, which covers both startup lost time and clearance lost time. Startup lost time occurs when the very first few vehicles queuing at a signal delay take advantage of its green aspects before accelerating into traffic; clearance lost time, on the contrary, happens during the signal change phases where no critical movement is expected or occurring. These components define most of the total lost time as possible, directly affecting traffic flow or efficiently using intersections (Koonce P., 2008).

$$\mathbf{g = G + Y + R - (\boldsymbol{l_1} + \boldsymbol{l_2})}$$

Where

g = is the effective green time

G = is the actual green interval

Y = is the actual yellow change interval

R = is the actual red clearance interval

l_1= is the start-up lost time

l_2= is the clearance lost time

2.10.4. Capacity in Traffic Management

The traffic capacity is the maximum rate at which vehicles can pass through an intersection or roadway under current conditions. Several factors affect it, such as the saturation flow rate, which is the number of vehicles that can pass a point in an hour, and the effective green time available for vehicles to enter the intersection. The capacity is mathematically expressed in relation to the above-mentioned elements, in which capacity c is denoted by saturation flow rate s, effective green time g, and cycle length C of the traffic signal. According to this definition, capacity at signalized intersections is determined by the maximum number of vehicles that can pass through a point in an hour and the ratio of time they can enter that intersection. Besides the configuration of lanes, the presence of conflicting traffic and signal timing, capacity can be greatly affected. (Koonce P., 2008) Capacity is thus an important aspect to consider for any of the design and optimization activities related to traffic signals for effective traffic management and reduction in delays at intersections. The Highway Capacity Manual gives methodologies for capacity estimation and level of service determination for different types of transport infrastructures such as intersections and roadways.

$$\mathbf{c} = \boldsymbol{s}\left(\frac{\boldsymbol{g}}{\boldsymbol{C}}\right)$$

Where

c = is the capacity;

s = is saturation flow rate of the lane group in (veh/hr);

g = is the effective green time (sec);

C = is the cycle length (sec).

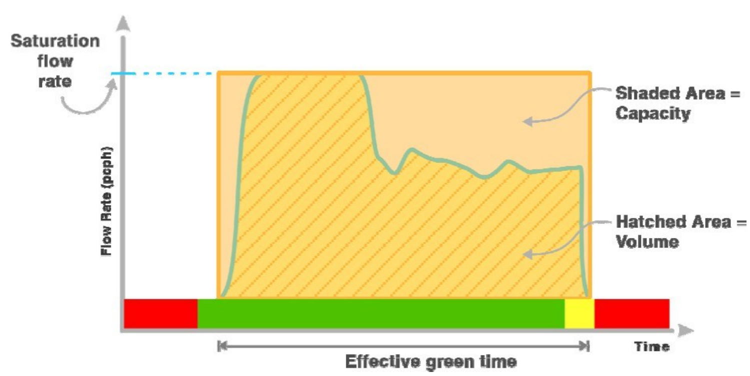


Figure 7: Volume and capacity of a signalized movement

2.10.5. Volume-to-Capacity Ratio

The volumetric-to-capacity (v/c) ratio keeps score against the orders of management of traffic towards discovering the relations between the spaces being demanded of a certain road with that which is allocated as capacity for it. The demand volume (v), which is devised to indicate the amount of vehicles that are expected to use a roadway or intersection toward a specific time period, is divided up into its capacity (c), which means the maximum number of vehicles that can pass through that point under optimum conditions. For ratios of v/c between 0.85 and 1.00, they signify the transition to lesser stable operations of fluctuations, where traffic can induce the formation of queuing, and a v/c ratio below 0.85 is associated with stable traffic flow operating efficiently. It means that a v/c that is above 1.00 indicates that demand continues to exceed capacity, which will in turn induce congestion, queue buildup and possibly delay over time. It would then mean that such a ratio is very relevant to traffic signal timing and overall traffic management through evaluating flow efficiency and determining how much adjustment is needed to better an intersection point (Koonce P., 2008).

$$v/c = \frac{v}{s\left(\frac{g}{C}\right)} = \frac{vC}{sg}$$

Where

v = is the demand volume of the subject movement (veh/hr)

2.10.6. Arrival Flow in Traffic Systems

The flow of vehicles arriving at an intersection over a time interval is denoted v and is defined as the rate at which vehicles arrive at an intersection. This is one of the key measures for handling the traffic coming into an intersection, as it identifies the level of performance of the intersection in handling incoming traffic. In a simple model, saturation flow, denoted as S, is the maximum possible rate at which vehicles can enter into an intersection when the traffic signal is green. The subsequent relationship between arrival flow and saturation flow can be drawn as a ratio v/c, where c represents capacity: The arrival flow exceeds the saturation flow, leading to congestion and delay when the intersection cannot operate effectively with all arriving vehicles. This is the worst scenario during peak hours of traffic when the arrival flow reaches the highest level, causing overflow delays and longer queues at the intersection (V.F.Hurdle, 1984).

2.11. Types of Delay at Signalized Intersections

Delay is the most important operational quality determinant at signalized intersections, with queue length sometimes considered a secondary indicator. Although delay can be determined from field studies, the very nature of measuring it makes it an extremely complex and subjective process, as different observer will interpret and assess delay differently. For these reasons and for practical purposes, it is very convenient to have predictive delay models. However, it should be remembered that delay can be quantified in various forms (Alkaissi, 2022). Typically, the most commonly used forms of delay may be defined in terms of:

- Stopped-time delay
- Approach delay
- Travel-time delay
- Time-in-queue delay
- Control delay

The different delay measures at signalized intersections depend on the conditions around the site. Fig shows the differences in time stopped, approach delay and travel time delay for a single vehicle by depicting the desired path and progress of that vehicle, including a stop at a red signal.

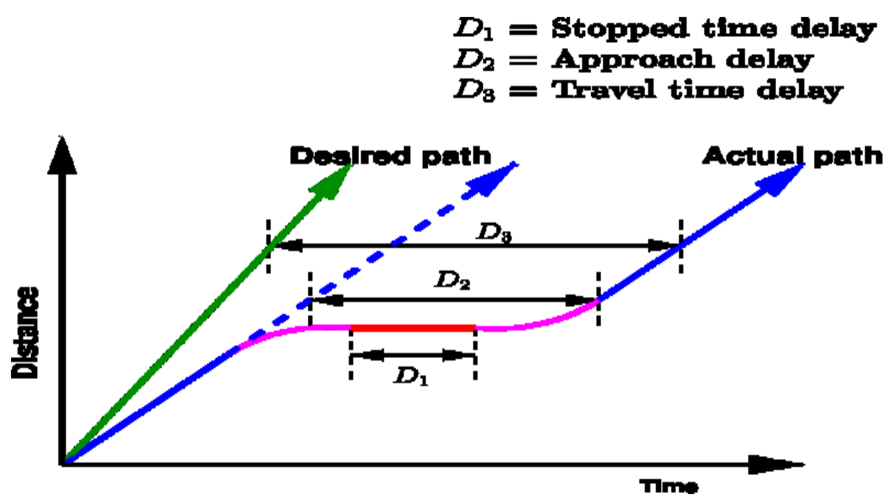


Figure 8: Types of delay measures

2.11.1. Stopped-time Delay

This type of delay is measured by the period a vehicle stands still in a queue waiting for a chance to cross an intersection (Alkaissi, 2022). Delays can be measured as the average amount of time spent by all vehicles waiting during a reference period, which gives an idea of the nature of traffic flow within the segment. Stopped-time delay is a simple form of measure, which only considers the total time of vehicles at a standstill, the clock starting once a vehicle becomes stationary and stopping when it is moving again.

2.11.2. Approach Delay

Thus, the notion of approach delay includes, besides the already discussed stopped-time delay, the time losses incurred by a vehicle as it decelerates from its approach speed to a full stop, followed by the necessary reacceleration to the accepted speed. It is a much more valuable delay because it reflects the whole series of actions concerned in deceleration and reacceleration-more comprehensively reflecting the time lost extensively at the intersection, thus preserving the holistic approach. To average the approach delay, the same calculations are taken for all vehicles operating over a common time period while simultaneously considering the variability of driver behavior and traffic conditions (Alkaissi, 2022).

2.11.3. Time-in-Queue Delay

This kind of delay records the total time a vehicle spends in a queue, from the moment it enters the queue at an intersection until it crosses the stop line on exiting (Alkaissi, 2022). As opposed to stopped-time delay, the whole experience, including any time spent in queuing and waiting is comprised in time-in-queue delay, thus rendering it an essential parameter for a more thorough understanding of the nature of congestion experienced at intersections. Just as the average time-in-queue delay is calculated for all vehicles in a specific time period, it will give remarkably valuable insights into the traffic pattern and the congestion level.

2.11.4. Travel Time Delay

Travel time delay is a rather conceptual concept that defines the difference between the expected travel time by an intersection and the actual time spent crossing it for a driver. Most often, this sort of delay does not find its place in any application owing to the difficulties in defining a "desired" travel time that is universally accepted (Alkaissi, 2022). It offers, nevertheless, a philosophical way to understand the more wide-ranging consequences of delays in traffic flow.

2.11.5. Control Delay

Control delay is precisely defined as the time waste that has been caused by the presence of a control device, such as a traffic sign or stop signal, regulating the movement of vehicles in an intersection. This type of delay is approximately equivalent to the summation of the time-in-queue delay plus the contributions due to acceleration and deceleration (Alkaissi, 2022). Control delay is an eminent concern in traffic engineering, as it is directly proportional to the effectiveness offered by traffic control plans to improve and help vehicle flow at critical points of the intersection.

2.12. Components of Delay

It's important to know the different components of delay while predicting traffic performance, especially regarding the behavior of vehicles waiting at intersections. There can be three independent components of delay: uniform delay, random delay and overflow delay.

2.12.1. Uniform Delay

The basis of delay modeling is known as uniform delay. This component rests on the premise of uniformly distributed vehicle arrivals at an intersection and stable traffic flow. Thus, vehicles are assumed to arrive consistently during the entire length of time without any cycle failure during analysis. In such cases, delay prediction could be performed quite simply since it is assumed that every vehicle will receive timely service without interruption. This type of delay is very important as a baseline for all traffic flow analyses because it provides uncomplicated information regarding the interaction of vehicles with traffic signals (Bullen, 1999).

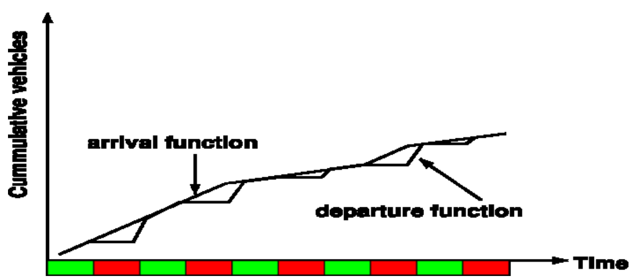


Figure 9: Uniform delay

2.12.2. Random Delay

The addition of random delay increases the complexity of analysis through variations in the arrival of vehicles. Unlike the homogeneous delay, under which traffic flow is consistent but exceeds the expectation from the homogeneous model, random delay manifests when the traffic occasionally departs from a pattern. This variability may stem from several reasons, such as fluctuations in the traffic volume or unforeseen events leading to the disturbance of the normal flow (Bullen, 1999). As far as isolated intersections are concerned, vehicle arrivals tend to have much more randomness; thus, inter-arrival times may closely fit an exponential distribution with an average parameter. This extra measure of delay is extremely important because it reflects the real world in which traffic is, at times, not entirely regular and predictable but consists of strange behaviors with respect to moment-to-moment activities.

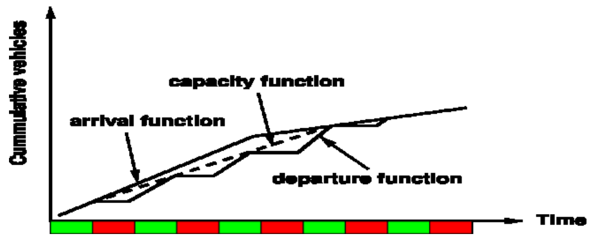


Figure 10: Random delay

2.12.3. Overflow Delay

The third element of this is flow over delay. This is especially relevant for cases in which the demand for motor vehicle movement exceeds the capacity of the intersection (Bullen, 1999). These situations are encountered when the flow rate of arrivals surpasses the storage capacity provided by the various phases of the traffic signals, producing queues that grow longer and longer. An overflow delay includes a uniform delay plus part of a random delay since it presumes that some signal phases have been inadequate to carry the flow of traffic into the intersection. It thus represents the summation effects of those vehicles that have been kept waiting far too long, in other words, due to non-saturation conditions at the intersection.

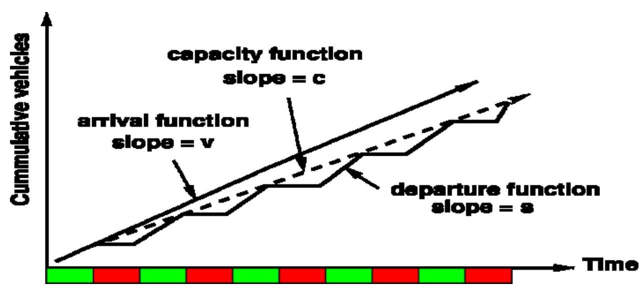


Figure 11: Overflow delay

2.13. Delay Models at Signalized Intersections

Models for estimating delay at signalized intersections are defined as the difference between the actual travel time experienced by vehicles and that expected under conditions of free-flow travel with no signal control. Estimation of delay is affected by probabilistic distributions of arrival flow, signal timings, and queue discharge rate, to name just a few factors, and all have variations; hence the problem of estimating delay is complicated further. Delay defines the critical component in determining service levels at intersections, which grade the performance from A, the best, to F, the worst, based on the amount of time delay experienced by motorists. Models like that developed by Webster have been criticized for overestimation of delay, especially in congested conditions where the assumptions of uniform arrival and departure rates do not hold. The most modern models now incorporate random delay components and use empirical modifications to better represent the situation on the road. For example, such models that consider variable queue discharge rates were found to be capable of improving accuracy concerning delay predictions at signalized intersections by decreasing their estimates by around 5 to 6% (Ranjitkar M. S., 2013).

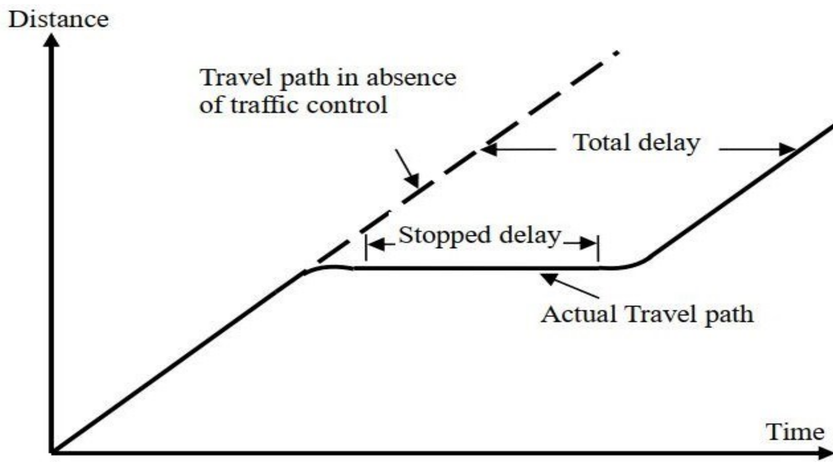


Figure 12: Delay at signalized intersections

2.13.1. Webster Delay Models

The Webster Delay Equation is a very fundamental equation used for estimating the average delay experienced by a vehicle at an intersection (Alkaissi, 2022). The basic variables are cycle length (c), flow of vehicles (q), effective green time (g), and saturation (x) where the latter stands for the ratio of actual flow to maximum flow (Cheng, 2003). The equation looks as follows: different delays are included as follows: the first term accounts for the German doom under the assumption of uniform arrivals; the second term includes randomness in terms of arrivals; and the conclusion is an empirical correction made with the aim of accuracy improvements under varying flow conditions (Ranjitkar M. S., 2013). As an example, it is mentioned that an absurd result is obtained when the degree of saturation approaches one-an infinite delay-which emphasizes its applicability limits in the extreme high volume of traffic mobilized. These have led to the improvement and refinement of delay models like the 'HCM 2000,' which gives improved modeling on the overtaking-beyond-the-capacity condition and random failures in the traffic stream (Balla, 2021).

$$\boldsymbol{d} = \frac{\boldsymbol{c}(\mathbf{1}-\boldsymbol{\lambda})^2}{\mathbf{2}(\mathbf{1}-\boldsymbol{\lambda x})} + \frac{\boldsymbol{x}^2}{\mathbf{2}\boldsymbol{q}(\mathbf{1}-\boldsymbol{x})} + \mathbf{0.65}\left(\frac{\boldsymbol{c}}{\boldsymbol{q}^2}\right)^{1/3}\boldsymbol{x}^{(2+5\lambda)}$$

Where

d = average delay per vehicle on the particular lane group of the intersection, sec/veh

c = cycle length, sec

q = flow, veh/sec

λ = proportion of the effective green with respect to cycle length (i.e. g/c and g is effective green, sec) and

x = the degree of saturation. This is the ratio of the actual flow to the maximum flow which can be passed through the intersection from this lane group, and is given by $x = g/\lambda s$, where s is the saturation flow in veh/sec.

2.13.2. Akcelik Delay Model

The Akcelik Delay Model estimates traffic delays in relation to signalized intersections, and it has elements to provide an all-inclusive analysis of vehicle delay with the signal cycle. Among those is the so-called uniform delay (UD), which is based on the number of arrivals during the cycle and the length of the cycle. The model makes the calculation of uniform delay quite simple, which means that in providing certain adjustable parameters such as the arrival flow rate and effective green time, the model can be more accurate. The model will also include some additional components, like overflow delays that have to be added to the uniform delay to represent the total experience of the vehicle in the entire period of evaluation. The aggregate delay is obtained as the area under the arrival and departure curves, and that shows the importance of green time in traffic signal design (Curtis E., 2014). It is further noted in the model that the lengths of red phases are directly proportional to the overall delay, whereby longer red phases tend to achieve higher aggregate delays (Ranjitkar M. S., 2013).

$$\mathbf{OD} = \frac{cT}{4}\left[(\mathbf{X}-1)+\sqrt{(X-1)^2+\frac{12(X-X_0)}{cT}}\right]$$

$$X_0 = 0.67 + \left(\frac{sg}{600}\right)$$

Where
T = analysis period, h
X = v/c ratio
C = capacity, veh/h
S = saturation flow rate, veh/sg (veh/s of green)
G = effective green time, s

2.13.3. HCM 2000 delay models

The HCM 2000 delay equation models control delays per vehicle using an isolated intersection as a study case. Different components include uniform control delay from uniform arrivals of vehicles (d1), an incremental delay from random, oversaturated arrivals (d2), and initial queue denial (d3) resulting from queues present before the start of analysis. A zero initial queue assumption simplifies calculations in this study. Unlike Webster's original delay equation, HCM 2000 has a more realistic approach to accounting for scenarios with high saturation because Webster's model predicts infinite delays, which can obviously be considered impractical. The HCM 2000 also accounts for random failures as well as short-term saturation, making it more applicable to real roads. This defines the extent of saturation showed delay because using hayets, HCM 2000 has offered wider stability in deferring the delay estimate compared to Webster, especially just before saturation reaches one. Such flexibility, for example, would be necessary to supplement the accurate measurements of traffic flow from which signal timing optimization at intersections would be derived (Cheng, 2003).

$$\mathbf{d} = \boldsymbol{d_1 PF + d_{2+} d_3}$$

$$\boldsymbol{d_1} = \frac{\boldsymbol{c}}{\mathbf{2}} \times \frac{\left(\mathbf{1} - \frac{\boldsymbol{g}}{\boldsymbol{c}}\right)^{\mathbf{2}}}{\mathbf{1} - \left[\mathbf{min}(\mathbf{1}, \boldsymbol{X})\left(\frac{\boldsymbol{g}}{\boldsymbol{c}}\right)\right]}$$

$$\boldsymbol{d_2} = \mathbf{900T}\left[(\mathbf{X} - \mathbf{1}) + \sqrt{(\boldsymbol{X} - \mathbf{1})^2 + \frac{\mathbf{8}\boldsymbol{KlX}}{\boldsymbol{cT}}}\right]$$

Where

d = control delay, s/veh

d_1= uniform delay component, s/veh

PF = progression adjustment factor

d_3 = delay due to pre-existing queue, s/veh

T = analysis period, h

X = v/c ratio

C = cycle lengths, s

K = incremental delay factor for actuated controller settings; 0.50 for all pre-timed controllers

I = upstream filtering/ metering adjustment factor; 1.00 for all individual intersection analyses

C = capacity, veh/h

2.13.4. Australian Delay Model

The Australian Delay Model is a popular model frequently used to estimate delay at signalized intersections. It uses a distinct formulation, which incorporates specific coefficients for the overflow delay term. It is, however, distinguished by its prediction of zero overflow delay at low degrees of saturation and is appropriate for under-saturated traffic conditions. While comparisons are made between the performances of the Australian delay model and other delay models, this model often highlights its strengths and limitations in certain traffic scenarios (Bullen, 1999).

$$d = \frac{C(1-\lambda)^2}{2(1-\lambda x)} + 900T\,[(x-1) + \sqrt{(x-1)^2 + 12(\frac{x-x_0}{cT})}]$$

$$X_0 = 0.67 + \frac{sg}{600}$$

Where

d = average overall delay

C = cycle time (sec)

λ= green ratio

x = degree of saturation

c = capacity (vph)

x_0 = degree of saturation below which the second term delay is zero

sg = capacity per cycle (veh/cycle)

2.13.5. Canadian Delay Model

The Mr. Whiting-indicted Canadian Delay Model is another stand-alone model that best predicts delays at signalized intersections. Like the Australian model, it involves terms of uniform and overflow delays. The Canadian Model only differs in its coefficients for the overflow delay term, thus causing the delay estimation to differ from that of the other models. It is reputed in fluxes of traffic across Canada and has gained credibility for its efficiency at estimating delay across a wide variety of conditions (Bullen, 1999).

$$d = \frac{C(1-\lambda)^2}{2(1-\lambda x)} + \frac{15t_e}{c}[(v-c) + \sqrt{(v-c)^2 + \frac{240v}{t_e}})]$$

Where

t_e= evaluation time (minutes)

v = arrival flow rate

After some manipulation, the Canadian delay model is expressed as:

$$d = \frac{C(1-\lambda)^2}{2(1-\lambda x)} + 900\mathrm{T}\,[(x-1) + \sqrt{(x-1)^2 + \frac{4x}{\mathrm{cT}}})]$$

2.13.6. Burrows' Generalized Delay Model

Burrow's generalized delay model is yet another versatile and advanced framework that goes beyond the simple estimation of delays at signalized intersections (Bullen, 1999). It takes into consideration the whole range of parameters affecting traffic flow, including vehicle types, driver behavior, and environmental conditions. Burrow's model incorporated wider perspectives of flow, thus yielding better and more reliable estimates of delays and a progressive improvement in developing effective traffic management strategies. The model serves as an excellent remedy in highly congested urban locations, where classical models are likely to lose their accuracy.

$$d = 0.5\frac{C(1-\lambda)^2}{2(1-\lambda x)} + 900\mathrm{T}x^n\,[(x-1) + \alpha + \sqrt{(x-1)^2 + \frac{m(x+\beta)}{\mathrm{cT}}})]$$

Where

m, n, α, β = calibration terms

2.14. Traffic Signal Optimization

Traffic signal optimization is the major part of improving a traveler's efficiency and effectiveness in the entrapment of modern mobility's dimensions, burying a person under the weight of modern travel. This is a very complex activity, as it involves the strategic and methodical management of traffic signals to reduce delays for motorists, decrease the levels of congestion drastically, and finally create a smooth overall flow of traffic in urban areas. It denotes the high importance of lumping traffic assignment mechanisms and signal control strategies into one critical whole because they are mutually interdependent; changes in the timings of signals have a great effect on the development of traffic flows, while on the other hand, the traffic volume on roads may heavily deem the determination of optimal settings for traffic signals already in place. In that context, this paper stresses the need for a cyclically time-expanded network representation model that is capable of accurately and effectively detailing the complex interdependencies existing between such variables. This leads to an entirely holistic and integrated view of managing urban traffic systems (Strehler, 2018).

2.14.1. Benefits of Signal Timing Optimization

The optimization of signal timing entails many benefits for traffic flow and congestion alleviation. One such major advantage is the energy conservation and environmental preservation; improvement in signal timings may translate into fuel consumption savings of 6 to 12% and into reductions in harmful emissions, such as carbon dioxide and hydrocarbons, by 13% and 10%, respectively (Lloyd J., 2006). Besides, because optimized signal timings would be more able to maintain flow at a preferred speed on arterial roads, the argument can be made for it being a tool to control speed. It would also encourage platoons, smooth flows, and reduced differentials in speed with shorter queues. Lower operating costs resulted from reduced idling or slow-moving traffic for vehicles, which again contributed to reduced accident costs. Another major benefit from this is that queue lengths are shorter, relieving spillback at intersections, thereby reducing overall congestion in the area. They also provide users with fewer costs incurred by fewer stops and delays, as well as more efficient travel time.

2.14.2. Traffic and Emissions Optimization in Transportation

Traffic and emissions optimization in transportation is an enhancement process by integrating vehicular traffic models with emission models to achieve efficiency in transportation systems while reducing environmental incidences. The optimization utilizes advanced techniques such as simulation-based optimization procedures that allow for designing traffic control measures in accordance with traditional traffic metrics like travel time and congestion and environmental metrics such as emission levels. Furthermore, experts can also develop signal plans using high-resolution microscopic models, which consider detailed interactions of vehicles with their environment, to find out the much-needed best traffic signal plans for an entirely better performance. Comparing alternative traffic-designed plans, this approach shifts from evaluation of expected strategies towards solution designing in congestion and emission mitigation (Nanduri, 2014).

2.14.3. Green-Wave Traffic Theory Optimization

The data on vehicle fuel consumption with details on the relationship between fuel consumption, driving state and speed is set up in a structured format for analytical evaluation. The Green-Wave traffic theory is optimized here through two-phase signal control particularly for cross intersections and T-intersections in order to improve traffic flow and safety. The enhancement of the Green-Wave traffic theory is expected to improve road safety, reduce vehicle fuel consumption and minimize vehicle emissions, thus positively contributing to environmental sustainability (Xiaoping Wu, 2014).

2.15. Signal Systems

Systems of signals are more prevalent at intersections for the management of traffic flow. They are diversified based on their particulars, operational complexities and adaptability to varying traffic conditions. Here are some common types of signal systems:

2.15.1. Fixed-Time Traffic Signals

Traffic signals that are fixed-time are some of the varieties of traffic management measures, which operate on pre-specified timings and switch the light phases after regular predefined intervals, disregarding the present real-time traffic conditions. These traffic signals are highly designed to manage the movements of vehicles and pedestrians at intersections using green, yellow, and red lights at a particular, fixed cycle. Fixed-time signals can be corrected by applying the so-called mathematical models, such as the cyclically time-expanded network model, trying to optimize traffic efficiency and travel time in real life. Although fixed-time signals can efficiently regulate traffic during stable conditions, they might not respond well to increases and decreases in traffic flow, resulting in delays at the times of maximum demand. For this reason, the integration of fixed-time signals with more adaptive systems like actuated signals can surely improve their performance and the response to actual traffic needs (Thunig T., 2019).

2.15.2. Fully Actuated Signal Control Systems

A fully actuated signal control system is a very modern form of traffic management system that has adapted itself to have the signal phases change dynamically based on the real-time data that is collected from either external loop detectors embedded in the roadway. These systems enable intersections to optimize their control by sensing the actual flow of vehicles through them and responding to that rather than following strict timing schedules. This means that the waiting time goes down, with the possibility of better traffic efficiency, because the signals change with the different traffic situations. Unfortunately, those systems suffer from the difficulty of forecasting the signal phase time because several factors, such as the number of vehicles on the road and the timing of previous phases, should be considered. However, recently developed methods for machine learning prediction have been introduced to improve the prediction of signal phases using historical data to forecast the future signal timings, thereby increasing the performance of fully actuated signal control systems employed in urban traffic management (Genser A., 2022).

Actuated Signals Optimization

Actuated signals convert live conditions into indications that maximize vehicle flow at intersections. Optimization includes determining the critical base parameters of maximum green and minimum green for adjusting around a specific traffic volume to minimize delay. Maximum green tells how long a signal stays green after a vehicle was detected, while a minimum green should count on what traffic is expected, optimally set at about 60% of the main street capacity (Jiang X., 2011). The optimization treatment uses analytical models and computer simulations to evaluate the effects of different volume-to-capacity ratios on control delays at intersections and applicable alignment from the real-world data. This dynamic acclimation to traffic conditions is seeking to lessen delay and elevate overall system efficiency.

Actuated Signal Control Systems for Coordinated Intersections Based on Platoons

Platoon-based actuated signal control systems, as the title suggests, operate on the principle of detecting and controlling the movement of vehicle platoons, or groups of vehicles traveling in close proximity to one another, to provide better traffic flow for signalized intersections. These systems make use of conventional loop detectors to display traffic volume and occupancy measurements. However, such technologies can only be applied in identifying platoons on approaches to intersections. The invention consists in the fact that it employs the call mechanism, as it is known in usual terms for vehicles, by changing it to the recognition of platoons instead of individual vehicles, allowing more optimized signal timing adjustments to be utilized for reducing their delays on both left turns and through traffic under platooned conditions. Thus, the system is proposed to be the main point of coordination between adjacent intersections. This can ensure that traffic flow is smooth for arterials with high corridor volumes. By presenting such conditional logics to switch between original coordinated control and the new platoon-based actuated control, the system allows for managing varying traffic conditions, hence ensuring that timings are optimized based on actual real-time traffic data (Xing Wu, 2019). Traffic signals thus operate far more efficiently, but this is just the start for developing the systems over an entire urban road network with conventional detection technologies.

2.15.3. Semi-Actuated Traffic Signals

The semi-actuated traffic sign mechanism provides the traffic light management control facility, wherein mainline traffic is going to flow uninterrupted while accommodating the needs of the side street vehicles. In this case, the mainline will have a predetermined fixed cycle length, and any signal will be activated for side streets only when there is an actual demand, such as during the waiting period of an automobile or vehicle at an intersection. This means there would be a continuous flow to the mainline traffic until a side street vehicle presses for a green light, and the intensive mainline happens to keep primary traffic being expected with longer waits for side street traffic. This usually happens because if the mainline is very busy, many of the phases for the side street would probably be skipped and only activated if there is an indicated call during some times in the cycle. It could even just lead to inefficient use of time served to a side street by not giving more than a fully actuated time, which adjusts its phases for the side streets according to real-time usage. Generally, such a condition can cause delays to side street vehicles having longer waiting times, often getting less green than pretimed signals as mainline traffic is almost continuously heavy. Even though the semi-actuated signal works great for the traffic, it cannot be said to favor each and everyone's use while passing through the road, especially the ones on side streets (P.E., 1993).

2.16. Intelligent Transportation Systems (ITS)

Intelligent transportation systems are a collection of advanced technologies that can significantly improve the safety and efficiency of transportation systems. Thus, these systems have several components, like sensor, communication, and data modeling technology, to monitor and manage traffic flow. The main objective of ITS is to improve existing traffic management by real-time traffic conditions, such as analyzing vehicle density, speed and travel time to optimize traffic signals and thus, accomplish reductions in congestion (Kharat, 2018). It takes a number of different forms, providing travelers with real-time information via an array of different channels for instance, SMS or via internet platforms-so that consumers know their current traffic conditions and whether they are likely to face delays on the roads. The need to implement ITS is borne out of the demands of increasing numbers of vehicles on and growth of cities, which are making their input in terms of traffic congestion and environmental pollution. All this integration of different technologies intends to make a general response and efficiency of the transport networks better, enforce, and enhance the overall experience of travel for users, including drivers and pedestrians alike. The worldwide dimension of ITS development has been reflected in the significant projects implemented all over the world. It would be typical of what a project could expect to be able to demonstrate in traffic management within cities and road safety.

2.17. Weibull Distribution

The historical development of the distribution, as well as its different formulations, is due to Waloddi Weibull. Actually, Weibull presented three such formulas, while a fourth one had been derived by error from a printer's error, resulting in confusion amidst the literature on applications and interpretations. A complete and thorough review of the historical context and mathematical formulations pertaining to the Weibull distribution, including cumulative distribution functions, probability density functions and hazard rates, has been around. However, one must be cognizant of the different formulations to avoid confusion among many practitioners and also promote effective employment of the Weibull distribution in all fields, especially reliability engineering and failure analysis. Graphical methods for estimating the parameters obtained from Weibull graph paper that simplifies the process of parameter estimation. In short, therefore, it actually becomes an excellent resource for practitioners by providing a clear historical background and by detailed comparison of essential formulas related to the Weibull distribution (Jr., 2018).

Thus, while research on the univariate distributions involving two Weibull distributions has gained much attention, there appears to be a significant gap in research regarding distributions involving three or more Weibull distributions and it calls for further exploration in this regard. Application of Weibull probability plots on the selection of model and parameter estimation, although it mentions the limitations of these graphical methods in respect to statistical theory for small samples. Focus on the basic Weibull distribution and its various extensions, which are important for modeling complex lifetime data that are beyond the capabilities of the basic model (C. D. Lai, 2011).

2.18. Intelligent Driver Model (IDM)

The intelligent driver model (IDM) analyzes how driving strategies tend to influence traffic capacity, especially in scenarios depicting adaptive cruise control (ACC) vehicles. Hence, the investigation states that while the original IDM addresses microscopic driving behavior effectively by producing unrealistic results in lane-crossing events, where real gaps between vehicles are often less than required to be "ideal". So, the authors introduce a new heuristic that considers constant accelerations for more realistic driving behavior within critical situations. The novel IDM combines the pros of the original model with the new approach to establish a more relaxed driving style similar to that done by humans. Simulations are used to examine how different proportions of ACC vehicles turn into traffic "phenomena" or flow characteristics, showing the research that about 0.3% of traffic capacity improvement can increase 1% in ACC cars added. This study proves that adaptable driving strategies should, when integrated, use them in future traffic systems to improve overall efficiency and reduce congestion (KESTING A., 2010).

It looks at the Intelligent Driver Model (IDM), a second-order car-following model and demonstrates how negative initial conditions develop into negative velocity and divergence to negative infinity, detrimental to realistic traffic simulations. Employs an extensively rigorous analysis of the well-posedness of the IDM model. The IDM is shown through the provision of counterexamples to display certain undesirable behaviors and the authors propose modifications to its individual acceleration function to avoid said undesirable behaviors. The findings show that simple amendments guarantee the well-posedness of the model, which would help in implementing numerical simulations with more reliability while keeping its core features intact and enhancing its applicability to different traffic scenarios (Albeaik S., 2022).

2.19. Summary and Gaps of Literature review

This present literature provides a wide breadth of various traffic flow theories, signalized intersection operations and queuing dimensions from metropolitan urban environments. It portrays the various modeling approaches to macroscopic, microscopic and mesoscopic states: understanding the behavior of traffic flows. It also notes some of the typical reasons leading to congestion such as limited capacity of roadways, poor traffic signals and increasing urban traffic and furthermore presents how delays get created at intersections, queue lengths as a result of different signal timing and arrival patterns, and compares a number of delay estimation models such as Webster, Akcelik and HCM 2000. The complexity of traffic management is widely attributed to mixed traffic conditions, especially in the developing countries, with regard to heterogeneous vehicle types and unpredictable pedestrian behavior.

Despite the extensive research, several gaps are identified. Most studies focus on isolated factors affecting intersection performance, with limited attention to how pedestrian volume, proportion of buses and green time interact to influence queue length at consecutive signalized intersections. Research often emphasizes developed country settings and lacks representation of mixed traffic conditions typical in cities like Addis Ababa. Moreover, the application of advanced simulation tools such as SUMO is limited and there is a need for more probabilistic models to account for real-world traffic variability. These gaps highlight the need for integrated studies that better reflect complex, real-life traffic systems and support more effective traffic signal coordination strategies.

Chapter Three

3. Methodology

This research methodology chapter spells out the kind of arrangement when it comes to accomplishments of the study's objectives and answering the research questions. It begins by describing the process of research and putting an orderly framework for the progression of the investigation. The chapter then gets into the sampling and evaluation methods utilized at intersection traffic signals. First, there is a survey of existing methods to highlight the best technique that could address the research objectives. A thorough explanation of the methodology follows once the mode of testing is decided. The chapter, finally, talks about the types of data used and the manner in which they were acquired to ensure a proper understanding of the methodological underpinnings of the study.

3.1. Research flow

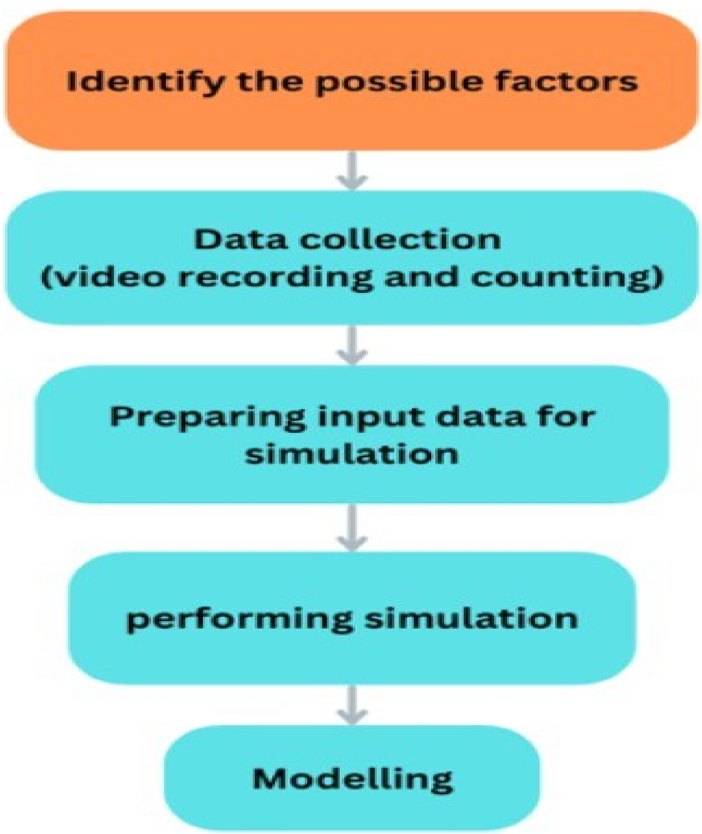


Figure 13: Research flow

The initial phase of the study is dedicated to data extraction and preparation, which are crucial steps for simulating traffic along consecutive signalized intersections within a corridor. This involves extracting and refining road network data to represent the geometry, signal timing and operational characteristics of the selected intersections accurately. Using SUMO (Simulation of Urban Mobility), the prepared corridor is simulated to model traffic flow along the corridor, capturing the interactions between vehicles and their response to signals at consecutive intersections. This approach provides a detailed understanding of how traffic propagates and behaves across the corridor, enabling insights into the performance and potential optimization of signalized intersections in the corridor.

3.2. Study Area

The capital city of Ethiopia, Addis Ababa, is considered one of the largest and most dynamic cities in all of Africa. Located at 9.005401° N and 38.763611° E, it sits at an altitude of 2,355 meters above the sea and has a unique topography and climate. The city serves as a hub for transport, trade and commerce and it forms a very important part of the economy of the country and the region. The increasing rapid urbanization and growing population are consequently making a lot of demands on the already overstretched road infrastructure and traffic management, making it the prime focus of transportation studies and urban planning programs.

Among the selected locations within Addis Ababa, Kebela 24, Shola Gebeya and Post Office intersections differ by the variety of traffic conditions and intersection photographs they represent. These locations are ideal for analyzing how intersection features can influence overall traffic performance because they vary across flows of vehicles, pedestrian activity and road geometry. Studying different intersections concerning their operational and structural characteristics would help researchers understand the performance of traffic and develop challenges and opportunities for enabling urban mobility at a wider scope.

Figure 14: Addis Ababa road network map

Regarding the selection of corridors for study, conducting consecutive signalized intersections throughout the whole city is difficult in terms of time through this research; therefore, representative road signal intersection corridors are selected considering a number of engineering factors.

3.3. Sample Size

In line with the objectives of the study, roads experiencing major delays at consecutive signalized intersections were selected. The sample size is determined by the number of shortlisted road corridors with vehicle counts taken between one traffic signal intersection analysis zone and the next consecutive signal intersection. These include vehicle counts at each of the various road segments in the study important to know in terms of the variation therein in traffic conditions and delays.

Purposive sampling was highly appropriate in this study as it enables a focus on locations and circumstances most representatives of the phenomena being studied. With adequate criteria and logic in the selection of these samples, the technique led to a much focused investigation into the high-impact traffic performance factors. Purposive sampling or judgment sampling was applied and it is selecting samples that are highly relevant to the research objectives. This method is used to identify certain signalized junctions, based on contributing parameters that cause longer queue lengths and delays.

Sampling of road intersections in Addis Ababa focuses on three consecutive signal intersections, which means six (6) intersections in important areas:

- Kebela 24 consecutive signal intersections
- Shola Gebeya consecutive signal intersections
- Post Office consecutive signal intersections

The three consecutive signalized intersections were chosen for study because of these perennial traffic congestions that arise out of the high volume of vehicles and pedestrians, closely spaced signalized intersections and poor traffic control. The area experiences heavy traffic all day long of private vehicles, public transport and non-motorized road users. A large number of pedestrians cross the road frequently and this also disturbs traffic movement.

The proximity of closely spaced signalized intersections at the three intersections would be among the factors for their choice. The short distance between nodes signals causes spillback to block traffic movement and thus the efficiency of the road by waiting vehicles at one intersection extending into the next. Extended delays prove frustratingly long, queues increase and stop-and-go conditions become observable. Thus, the site is very appropriate for studying the impact of intersection spacing on urban traffic congestion.

Another contributing factor in these three areas is the absence of dedicated pedestrian facilities. Pedestrians tend to cross at random without the availability of crossing signals and only rely on interruptions in the flow of vehicles. This causes interruptions with traffic, causes vehicles to stop suddenly and diminishes effective green time with motorized traffic. Collectively, they increase travel time and worsen congestion.

Moreover, the three consecutive signalized intersections represent the general type of traffic challenge that can face most urban areas with an intricate road network. The study can draw inferences for improving mobility and optimizing signal timing and pedestrian infrastructure in some similar intersections of Addis Ababa and other cities using the result obtained from this location. Urban traffic can improve mobility and decongestion due to the lesser economic and environmental impacts opposite congestion through traffic management improvement strategies.

Data’s collected on the following factors.

- To get a wide range of traffic observations, observations on locations taken at peak times.
- The data will include information about signal timings, lane configurations, pedestrian numbers, traffic volume, vehicle speeds and vehicle types.
- The pedestrian, public transport, private car, truck, truck trailer and motorcycle were taken into account.

Kebela 24 consecutive intersection | Post office consecutive intersection | Shola gebeya consecutive intersection

Figure 15: Study area

3.4. Data Collection

The data was collected using the primary method of phone video recordings and counts of traffic and pedestrian volumes at a consecutive of signalized intersections. The high resolution camera smartphones are mounted onto stable structures such as buildings or raised platforms to provide a clear view of the entry and exit of vehicles and pedestrians, turning movements, pedestrian cross movement and queue lengths. Recordings were done at the same time across intersections such that length measurement was done at synchronized time to keep track of traffic participation and interaction. Data was collected during peak hours to account for traffic variability and then reviewed manually for performance analysis of a consecutive traffic signal intersection using counted vehicles and pedestrian numbers, speed, cycle time of signals and compositions of vehicles.

Data was collected during the peak hour between 11:00 AM and 12:00 PM. This specific time was chosen because it represents a period of high traffic activity. Many employees leave their workplaces during this hour to go home, which significantly increases the number of vehicles and pedestrians on the roads.

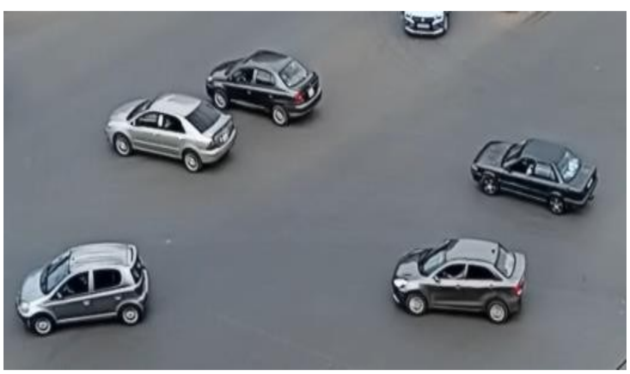
- Traffic volume at signalized intersections denotes the number of vehicles moving through the intersection within a specific time period.

Figure 16: Traffic volume

- Vehicle composition is the mix of different types of vehicles (i.e. cars, buses, trucks, motorcycles and bicycles…) significantly affects traffic dynamics at a signalized intersection, as well as at consecutive intersections.

Bus

Truck trailer



Car

Mini- bus

Heavy truck

Pick-up

Small Truck

Motor Cycle

Figure 17: Types of vehicles

Table 1: Vehicle composition count data

Intersections Area	Upstream or Downstream	Approach	Traffic volume	Car (%)	Mini-bus & Pickup (%)	Bus (%)	Small Truck (%)	Heavy Truck (%)	Truck Trailer (%)	Motor (%)
Kebela 24	Upstream	LT (1)	1204	67.4	18.1	9.3	5.3	0.0	0.0	0.0
Kebela 24	Upstream	RT (2)	1204	68.0	18.1	8.7	5.3	0.0	0.0	0.0
Kebela 24	Upstream	Through & RT (3)	604	67.0	16.5	10.8	1.3	0.0	0.0	4.4
Kebela 24	Upstream	Through (4)	604	50.2	35.5	9.6	2.2	0.0	0.6	1.9
Kebela 24	Upstream	Through & LT (5)	1204	63.3	28.6	11.0	1.5	4.1	0.5	1.0
Kebela 24	Downstream	Through & RT (6)	564	52.8	37.7	0.0	1.9	0.0	0.0	7.5
Kebela 24	Downstream	Through (7)	664	65.4	28.9	1.1	2.8	0.0	0.2	1.6
Kebela 24	Downstream	Through & LT (8)	564	52.2	40.7	1.0	3.8	0.0	1.6	0.5
Shola Gebeya	Upstream	LT (1)	1212	71.4	17.5	9.5	1.5	0.0	0.0	0.0
Shola Gebeya	Upstream	RT (2)	1012	40.7	33.3	9.8	2.9	2.9	0.0	10.5
Shola Gebeya	Upstream	Through & RT (3)	912	78.0	7.0	1.5	4.7	0.0	0.0	8.8
Shola Gebeya	Upstream	Through (4)	912	55.8	32.3	1.9	6.1	2.0	0.0	2.0
Shola Gebeya	Upstream	Through & LT (5)	912	61.5	22.1	9.7	3.0	1.2	0.0	2.5
Shola Gebeya	Downstream	Through & RT (6)	860	70.8	21.9	4.3	1.7	0.0	0.0	1.3
Shola Gebeya	Downstream	Through (7)	860	40.9	42.9	4.5	5.4	2.0	0.0	4.4
Shola Gebeya	Downstream	Through & LT (8)	760	60.2	26.5	1.6	6.4	1.6	0.0	3.6
Post Office	Upstream	LT (1)	1392	52.1	31.0	2.5	0.0	0.0	0.0	14.4
Post Office	Upstream	RT (2)	1392	49.2	29.7	4.5	2.7	0.0	0.0	13.9
Post Office	Upstream	Through _1 (3)	792	49.2	38.3	1.0	2.1	1.1	0.2	8.0
Post Office	Upstream	Through _2 (4)	792	58.6	30.6	1.1	3.2	1.4	0.3	4.8
Post Office	Upstream	Through & LT (5)	1592	50.4	43.8	2.8	0.8	1.0	0.3	1.0
Post Office	Downstream	Through _1 (6)	576	37.5	35.1	8.1	2.1	0.8	0.2	16.2
Post Office	Downstream	Through _2 (7)	576	53.1	24.0	1.1	2.9	1.7	0.5	16.8
Post Office	Downstream	Through & LT (8)	1176	60.0	26.9	8.6	2.6	0.0	0.0	1.9

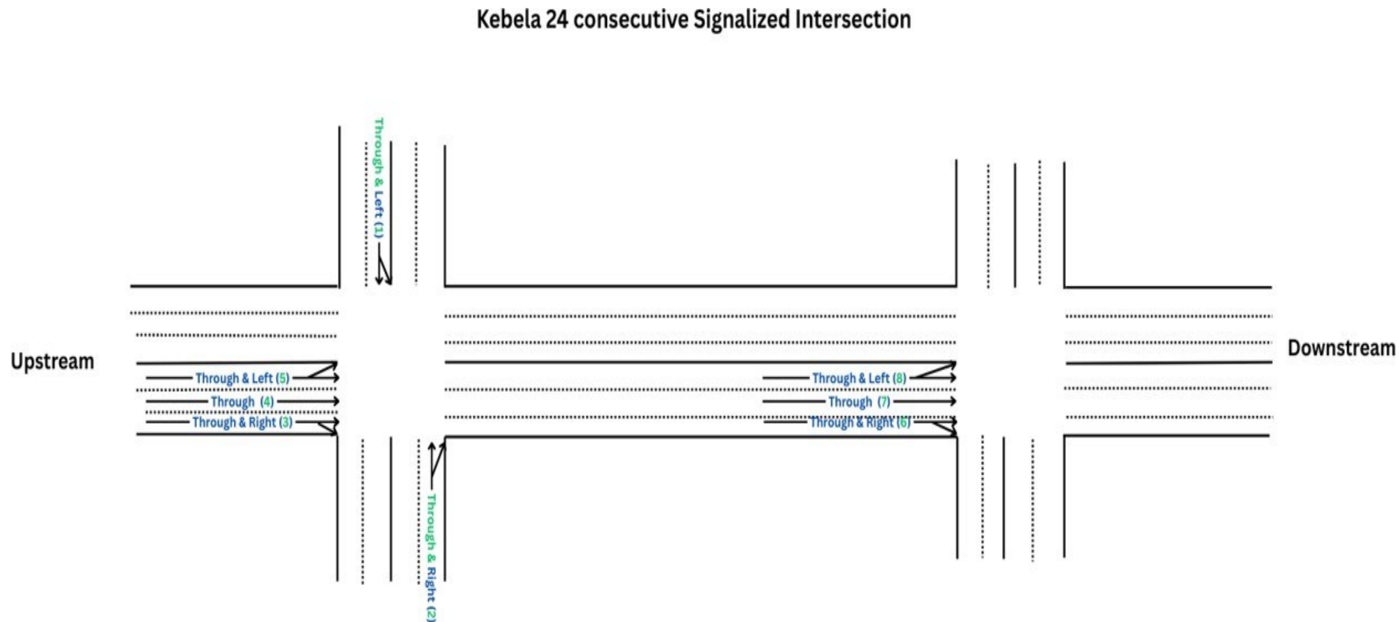
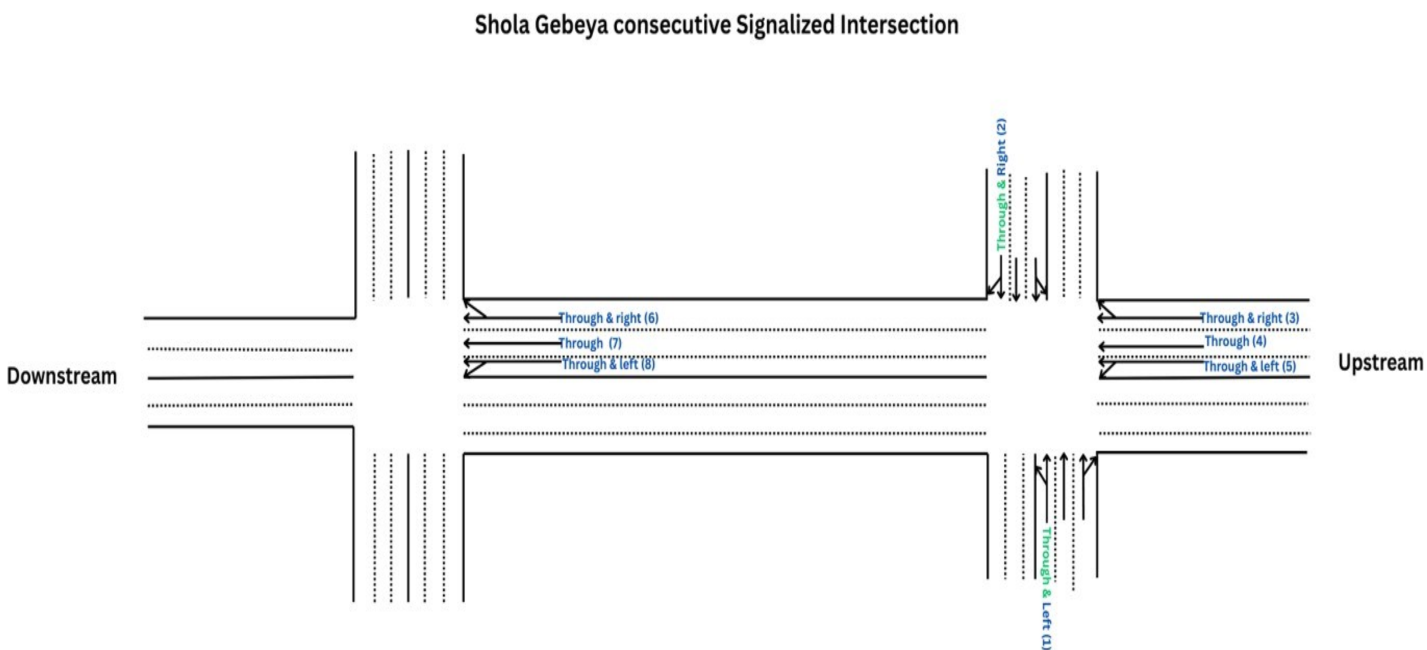
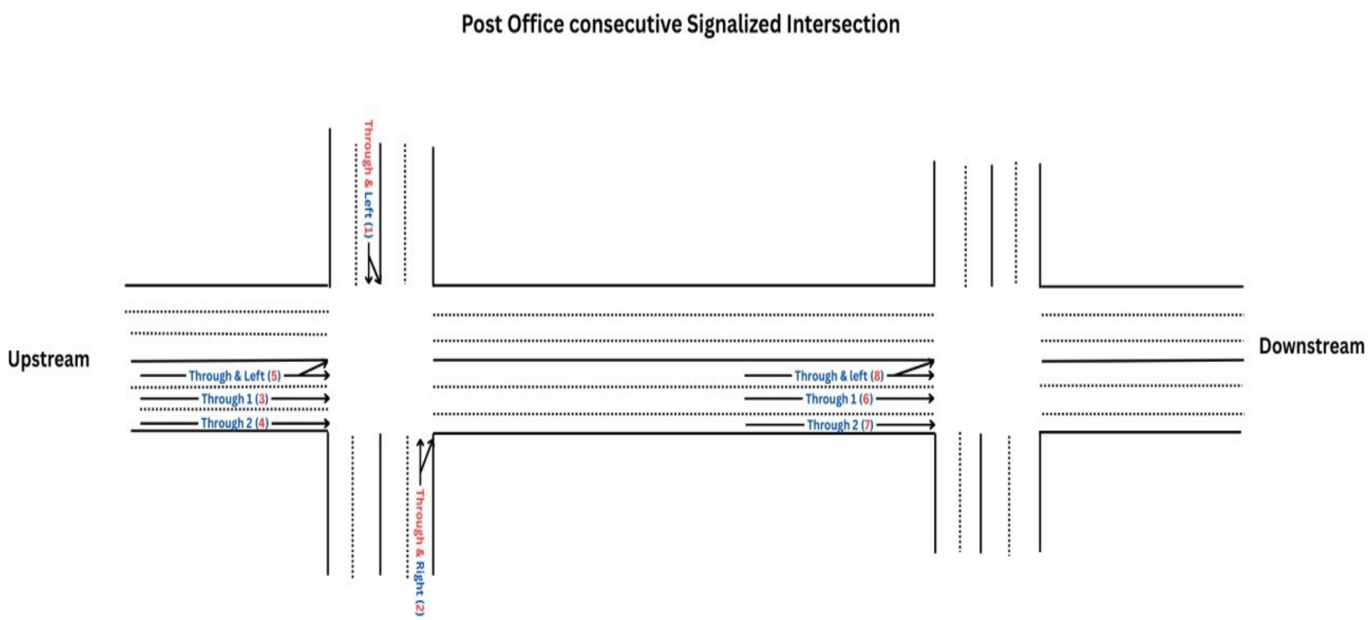


Figure 18: Kebela 24 consecutive signalized intersections directions of each lane



Figures 19: Shola gebeya consecutive signalized intersections directions of each lane



Figures 20: Post Office consecutive signalized intersections directions of each lane

- Green time in signalized intersections represents the duration within a traffic signal cycle when a specific direction of traffic has a green light, allowing vehicles or pedestrians to proceed. It is a critical component of signal timing and is typically optimized based on traffic volume, intersection capacity and safety considerations to minimize delays and improve traffic flow efficiency. As shown in the table 2 and 3 below, the direction of each approach is shown in the above figures 20, 21 and 22.

Table 2: Green time count data

Intersections Area	Upstream or Downstream	Approach	Green time (sec)
Kebela 24	Upstream	LT (1)	17
Kebela 24	Upstream	RT (2)	30
Kebela 24	Upstream	Through & RT (3)	68
Kebela 24	Upstream	Through (4)	68
Kebela 24	Upstream	Through & LT (5)	20
Kebela 24	Downstream	Through & RT (6)	61
Kebela 24	Downstream	Through (7)	61
Kebela 24	Downstream	Through & LT (8)	61
Shola Gebeya	Upstream	LT (1)	25
Shola Gebeya	Upstream	RT (2)	40
Shola Gebeya	Upstream	Through & RT (3)	45
Shola Gebeya	Upstream	Through (4)	45
Shola Gebeya	Upstream	Through & LT (5)	45
Shola Gebeya	Downstream	Through & RT (6)	40
Shola Gebeya	Downstream	Through (7)	40
Shola Gebeya	Downstream	Through & LT (8)	40
Post Office	Upstream	LT (1)	17
Post Office	Upstream	RT (2)	25
Post Office	Upstream	Through _1 (3)	63
Post Office	Upstream	Through _2 (4)	63
Post Office	Upstream	Through & LT (5)	20
Post Office	Downstream	Through _1 (6)	65
Post Office	Downstream	Through _2 (7)	65
Post Office	Downstream	Through & LT (8)	25

Table 3: Green Time to Cycle Time Ratio (g/c)

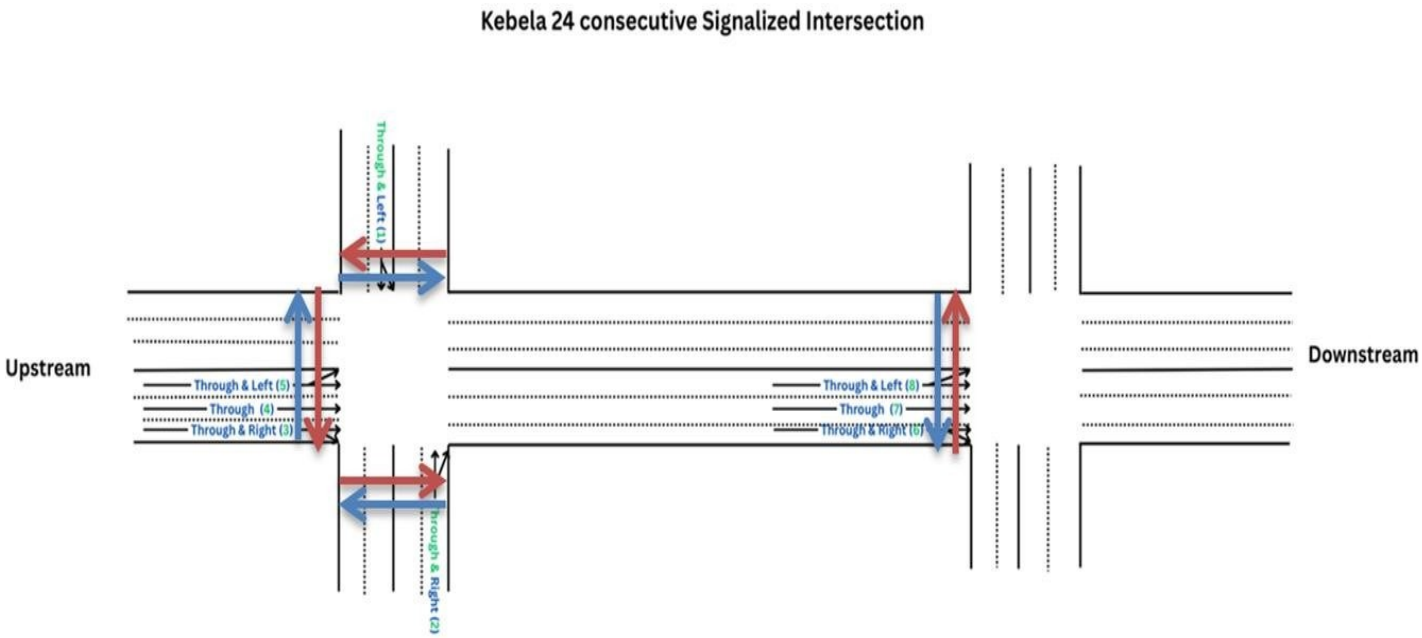
Intersections Area	Upstream or Downstream	Approach		g/c
Kebela 24	Upstream	LT	(1)	0.094444444
Kebela 24	Upstream	RT	(2)	0.166666667
Kebela 24	Upstream	Through & RT	(3)	0.377777778
Kebela 24	Upstream	Through	(4)	0.377777778
Kebela 24	Upstream	Through & LT	(5)	0.111111111
Kebela 24	Downstream	Through & RT	(6)	0.338888889
Kebela 24	Downstream	Through	(7)	0.338888889
Kebela 24	Downstream	Through & LT	(8)	0.338888889
Shola Gebeya	Upstream	LT	(1)	0.157232704
Shola Gebeya	Upstream	RT	(2)	0.251572327
Shola Gebeya	Upstream	Through & RT	(3)	0.283018868
Shola Gebeya	Upstream	Through	(4)	0.283018868
Shola Gebeya	Upstream	Through & LT	(5)	0.283018868
Shola Gebeya	Downstream	Through & RT	(6)	0.203045685
Shola Gebeya	Downstream	Through	(7)	0.203045685
Shola Gebeya	Downstream	Through & LT	(8)	0.203045685
Post Office	Upstream	LT	(1)	0.11038961
Post Office	Upstream	RT	(2)	0.162337662
Post Office	Upstream	Through _1	(3)	0.409090909
Post Office	Upstream	Through _2	(4)	0.409090909
Post Office	Upstream	Through & LT	(5)	0.12987013
Post Office	Downstream	Through _1	(6)	0.451388889
Post Office	Downstream	Through _2	(7)	0.451388889
Post Office	Downstream	Through & LT	(8)	0.173611111

- In signalized intersections, pedestrian volume is the number of people who cross the streets or walk through the intersection during a specific period of time. That includes anyone walking in either direction across the crosswalks during specified times when vehicular and pedestrian movement is controlled by the traffic signal. Measuring pedestrian volume is an additional tool in understanding how busy an intersection is and how well it handles foot traffic. In studies on pedestrian safety, determining needs for wider sidewalks or longer crossing times or checking traffic signal timing to control possible conflicts around pedestrian-vehicle interactions, pedestrian volume will be of great help.

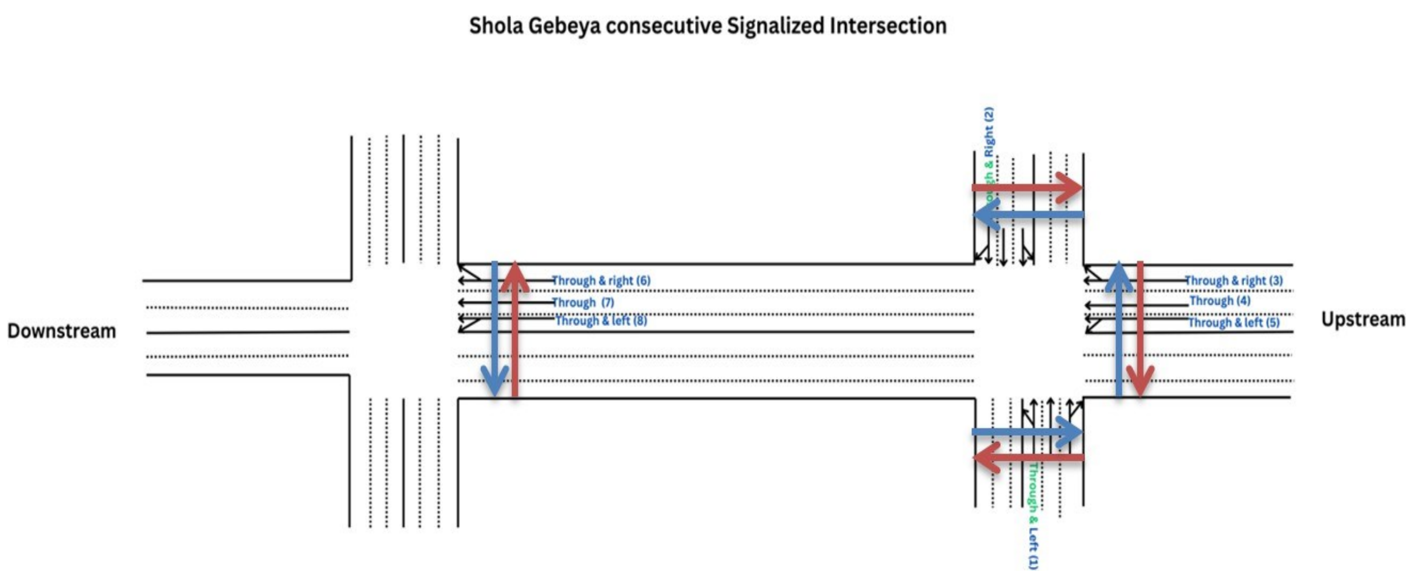
Figure 21: Pedestrian random crossing

Table 4: Pedestrian volume count data

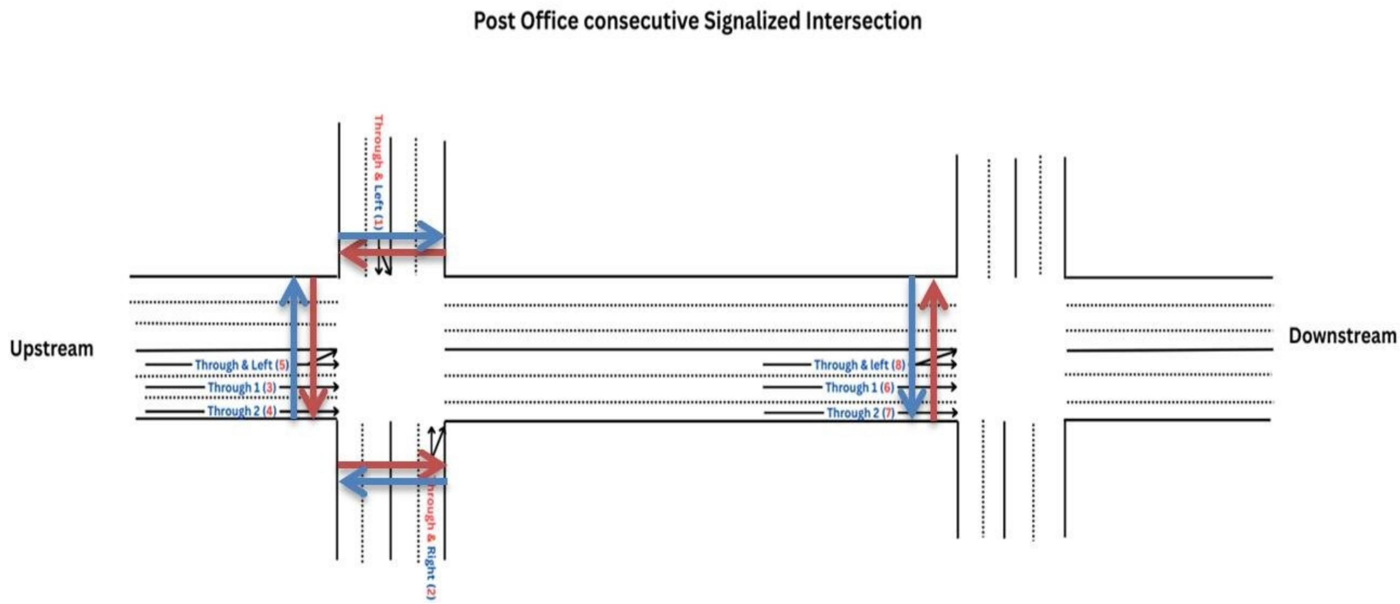
Intersections Area	Upstream or Downstream	Approach	Pedestrians volume (/hr)
Kebela 24	Upstream	LT (1)	495
Kebela 24	Upstream	RT (2)	732
Kebela 24	Upstream	Through & RT (3)	432
Kebela 24	Upstream	Through (4)	432
Kebela 24	Upstream	Through & LT (5)	285
Kebela 24	Downstream	Through & RT (6)	148
Kebela 24	Downstream	Through (7)	148
Kebela 24	Downstream	Through & LT (8)	148
Shola Gebeya	Upstream	LT (1)	1084
Shola Gebeya	Upstream	RT (2)	1084
Shola Gebeya	Upstream	Through & RT (3)	884
Shola Gebeya	Upstream	Through (4)	684
Shola Gebeya	Upstream	Through & LT (5)	684
Shola Gebeya	Downstream	Through & RT (6)	912
Shola Gebeya	Downstream	Through (7)	912
Shola Gebeya	Downstream	Through & LT (8)	912
Post Office	Upstream	LT (1)	411
Post Office	Upstream	RT (2)	394
Post Office	Upstream	Through _1 (3)	472
Post Office	Upstream	Through _2 (4)	472
Post Office	Upstream	Through & LT (5)	872
Post Office	Downstream	Through _1 (6)	192
Post Office	Downstream	Through _2 (7)	192
Post Office	Downstream	Through & LT (8)	315



Figures 22: kebela 24 consecutive signalized intersection pedestrians crossing



Figures 23: Shola gebeya consecutive signalized intersection pedestrians crossing



Figures 24: Post office consecutive signalized intersection pedestrians crossing

- Speed using stopwatch between two defined points

Figure 25: Speed measurement using stopwatch

Table 5: Vehicle speed calculation from stopwatch data

Number	Time (sec)	Distance (m)	Speed (m/s)
1	27.52	46.3	1.68
2	23.76	46.3	1.95
3	17.26	46.3	2.68
4	15.76	46.3	2.94
5	13.75	46.3	3.37
6	12.76	46.3	3.63
7	12.29	46.3	3.77
8	11.79	46.3	3.93
9	10.78	46.3	4.29
10	11.03	46.3	4.2
11	10.48	46.3	4.42
12	9.78	46.3	4.73
13	10.04	46.3	4.61
14	9.25	46.3	5.01
15	9.28	46.3	4.99
16	9.29	46.3	4.98
17	9.99	46.3	4.63
18	10.04	46.3	4.61

3.5. Simulation of Urban Mobility (SUMO)

SUMO stands for Simulation of Urban Mobility and is a flexible, freely available, agent-based, and time-continuous traffic simulation system for large-scale networks. It enables the traffic behavior to be described more precisely at the road network and particularly in the urban environment. Through SUMO vehicle dynamics, mobility, interaction and actions like changing lanes, accelerating or decelerating can be simulated. Through surveys and empirical studies, the software is mainly designed by the Institute of Transportation Systems at the German Aerospace Center. The latter depends on the network data used for simulation, which need to be translated into the SUMO format and normalized for all networks. Below are the steps for extracting a SUMO network:

3.5.1. Extraction of a SUMO network

Transform networks from the simulation source to SUMO network and edit them to treat each other equally. The following are procedures to extract a sumo network:

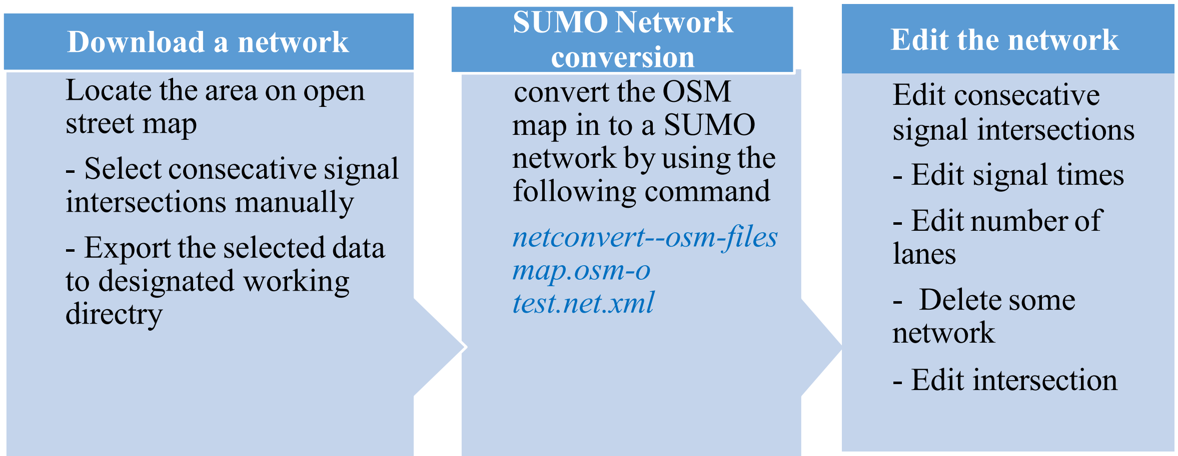


Figure 26: SUMO network extraction

The SUMO network specifies characteristics of the road network and mainly consists of:

- Hierarchy of roads
- Number of lanes and ways
- Speed limit and width of road

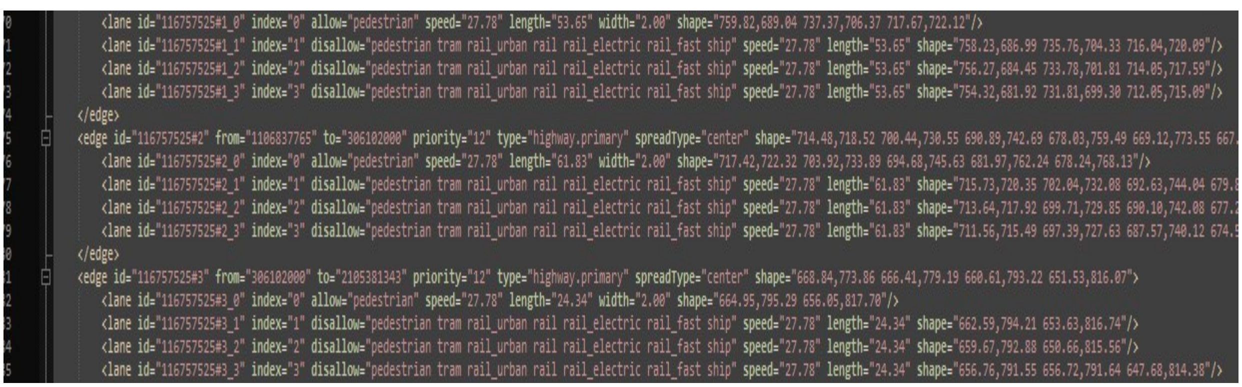


Figure 27: Converted SUMO network file

3.5.2. Origin Destination Matrix

In SUMO, an OD matrix describes movement between particular origins and destinations within the network of the transport system. It plays an important role in the transportation planning and traffic survey, helping to understand the travel demand and vehicle traffic by different zones. An OD matrix is used to determine the demand occurring between the zones or areas in the simulated network and acts as a key element in establishing valid traffic scenarios and for further analyses. According to SUMO, the OD matrix is defined as explained in the following figure.

```
$O;D2
* From-Time To-Time
0.00 1.00
* Factor
1.00
*
* some
* additional
* comments
          1     2     350
          1     3     250
```

Figure 28: Origin and destination definition

The first two columns, 1 and 2 stand as the names of traffic analysis zones, and the last column, 350, represents traffic flow.

The source and the target addresses of each subsequent signal intersection are different. This differential allows all corridors to be justified so that equal comparisons and evaluations of traffic flow in different corridors can be made. The source and destination for each traffic analysis of consecutive signal intersections are depicted below in the figure.

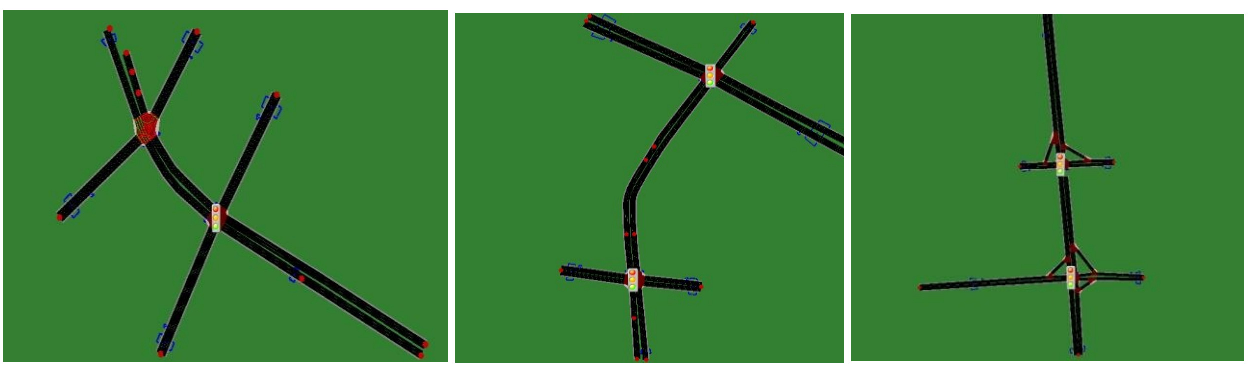


Figure 29: Traffic Analysis zone (TAZ)

A Traffic Analysis Zone (TAZ) is a part of the network where traffic flows in and out of the corridor and where vehicles originate or reach their destination. From (Lopez, 2018), a TAZ is characterized by its ID (a simple name) and by lists of source and destination edges. Here is how to construct a TAZ:

<tazs>

<taz id="<TAZ_ID>" edges="<EDGE_ID> <EDGE_ID>..."/>

... additional traffic analysis zones...

</tazs>

Every TAZ must have at least one source edge and one destination edge, and every edge must be distinct, being identified by its ID. These edges are to and from the corridor.

3.5.3. Trip Generation

This step is important for modeling the travel demand and evaluating transportation system performance and trip generation is creating virtual trips in a simulated transportation network to account for the movement of vehicles or people from one point within SUMO to another by the network choosing to source another.

Figure 30: Trip generation

```
<trip id="car417" depart="77.89" from="E4" to="200567869#2" type="car" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
<trip id="car110" depart="81.63" from="E4" to="E2" type="car" fromTaz="1" toTaz="3" departLane="free" departSpeed="max"/>
<trip id="car222" depart="101.43" from="E4" to="-E0" type="car" fromTaz="1" toTaz="5" departLane="free" departSpeed="max"/>
<trip id="car507" depart="105.18" from="E4" to="200567869#2" type="car" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
<trip id="car398" depart="106.94" from="E4" to="200567869#2" type="car" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
<trip id="car366" depart="107.93" from="E4" to="200567869#2" type="car" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
<trip id="car369" depart="114.89" from="E4" to="200567869#2" type="car" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
<trip id="car128" depart="116.59" from="E4" to="E2" type="car" fromTaz="1" toTaz="3" departLane="free" departSpeed="max"/>
<trip id="car317" depart="150.40" from="E4" to="200567869#2" type="car" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
<trip id="car545" depart="153.28" from="-E3" to="200567869#2" type="car" fromTaz="2" toTaz="6" departLane="free" departSpeed="max"/>
```

Figure 31: Trip file

3.5.4. Route Assignment

Traffic assignment is the term used to describe the process of assigning a certain set of trip interchanges to a particular transportation system. Replicating the pattern of vehicle movements that would be seen on the transportation system when the trip matrix or matrices, to be assigned represents the travel demand is the primary goal of the trip assignment procedure.

(Lopez, 2018)The simulation must identify routes through the network (list of edges) that are utilized to reach the destination from the origin edge for a given set of vehicles with origin-destination relations (trips). The easiest way to locate these routes is to use routing algorithms to calculate the paths in the network. Even though SUMO can use a different type of assignment methods, this research uses the default assignment method in SUMO due to there is only one route for each vehicle from origin to destination.

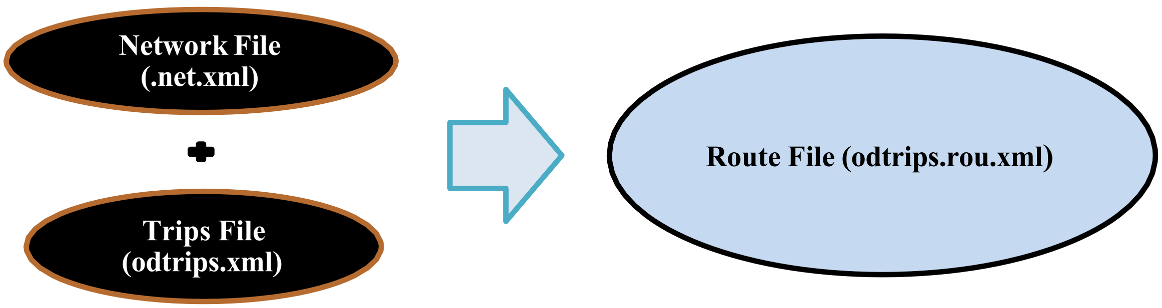
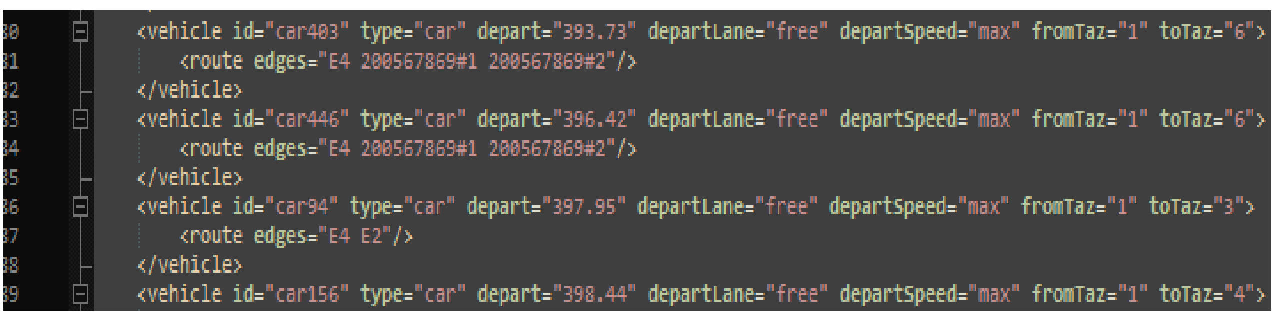


Figure 32: Trip assignment



```
<vehicle id="car403" type="car" depart="393.73" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
    <route edges="E4 200567869#1 200567869#2"/>
</vehicle>
<vehicle id="car446" type="car" depart="396.42" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
    <route edges="E4 200567869#1 200567869#2"/>
</vehicle>
<vehicle id="car94" type="car" depart="397.95" departLane="free" departSpeed="max" fromTaz="1" toTaz="3">
    <route edges="E4 E2"/>
</vehicle>
<vehicle id="car156" type="car" depart="398.44" departLane="free" departSpeed="max" fromTaz="1" toTaz="4">
```

Figure 33: Vehicle route file

3.5.5. Lane Based Flow and Queue Length

The output simulation can be generated at the level of lanes, edges or groups of edges. The simulations output for each consecutive signalized intersection corridor forms the dataset for optimizing queue lengths at signalized intersections. The outputs comprise queuing time and queuing length.

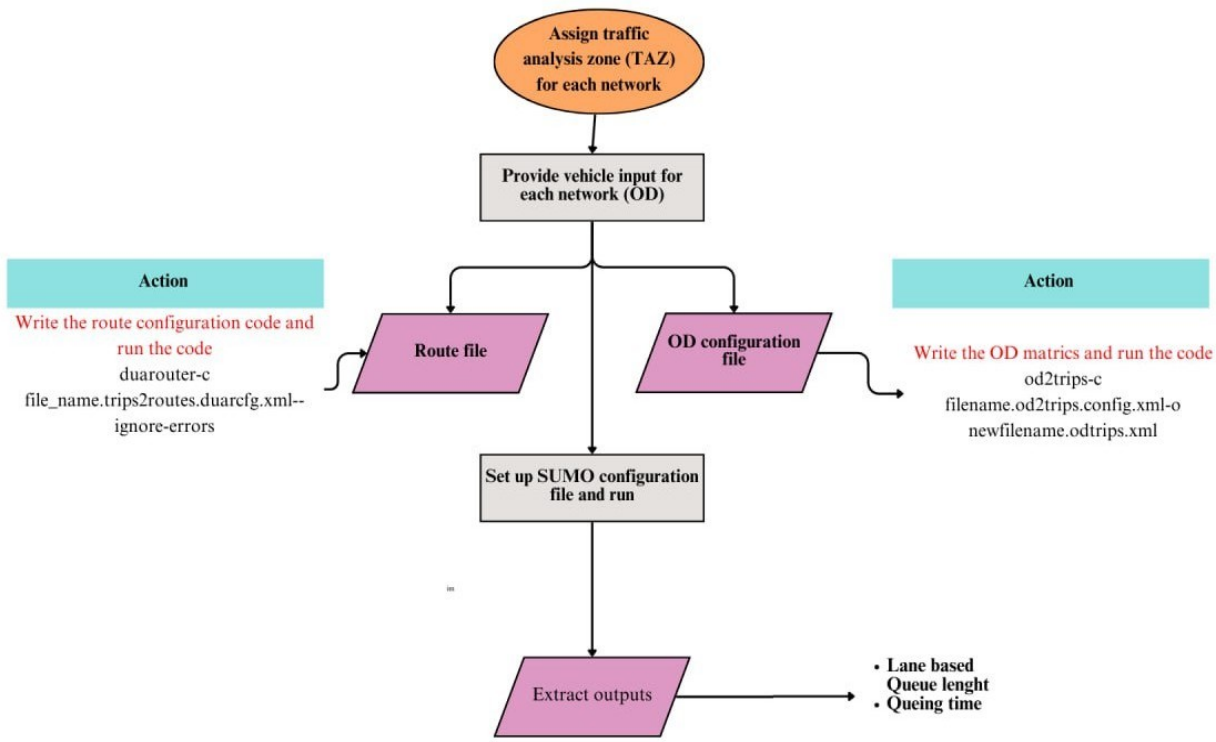


Figure 34: Data extraction flow chart from SUMO

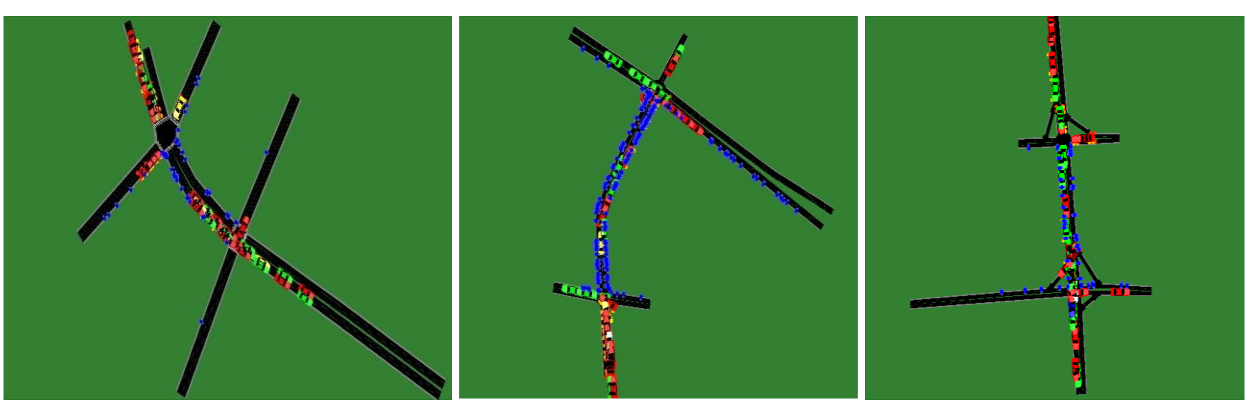


Figure 35: Simulation operating in SUMO (through the SUMO interface)

```xml
        <lanes>
            <lane id="-E2_2" queueing_time="67.00" queueing_length="9.00" queueing_length_experimental="9.00"/>
            <lane id="-E3_1" queueing_time="54.00" queueing_length="20.60" queueing_length_experimental="20.60"/>
            <lane id="200567869#1_1" queueing_time="64.00" queueing_length="57.82" queueing_length_experimental="57.82"/>
            <lane id="200567869#1_2" queueing_time="67.00" queueing_length="58.81" queueing_length_experimental="58.81"/>
            <lane id="200567869#1_3" queueing_time="65.00" queueing_length="61.61" queueing_length_experimental="61.61"/>
            <lane id="E4_3" queueing_time="6.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
        </lanes>
    </data>
    <data timestep="707.00">
        <lanes>
            <lane id="-E2_2" queueing_time="68.00" queueing_length="9.00" queueing_length_experimental="9.00"/>
            <lane id="-E3_1" queueing_time="55.00" queueing_length="20.60" queueing_length_experimental="20.60"/>
            <lane id="200567869#1_1" queueing_time="65.00" queueing_length="57.81" queueing_length_experimental="57.81"/>
            <lane id="200567869#1_2" queueing_time="68.00" queueing_length="58.81" queueing_length_experimental="58.81"/>
            <lane id="200567869#1_3" queueing_time="66.00" queueing_length="61.61" queueing_length_experimental="61.61"/>
            <lane id="E4_2" queueing_time="0.00" queueing_length="0.00" queueing_length_experimental="7.61"/>
            <lane id="E4_3" queueing_time="7.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
```


Figure 36: Simulation output

Chapter Four

4. Result and Discussion

4.1. Data Analysis

The Weibull distribution is among the most commonly employed lifetime distributions in reliability engineering. Its flexibility allows it to mimic the features of other distributions depending on the value of the shape parameter.

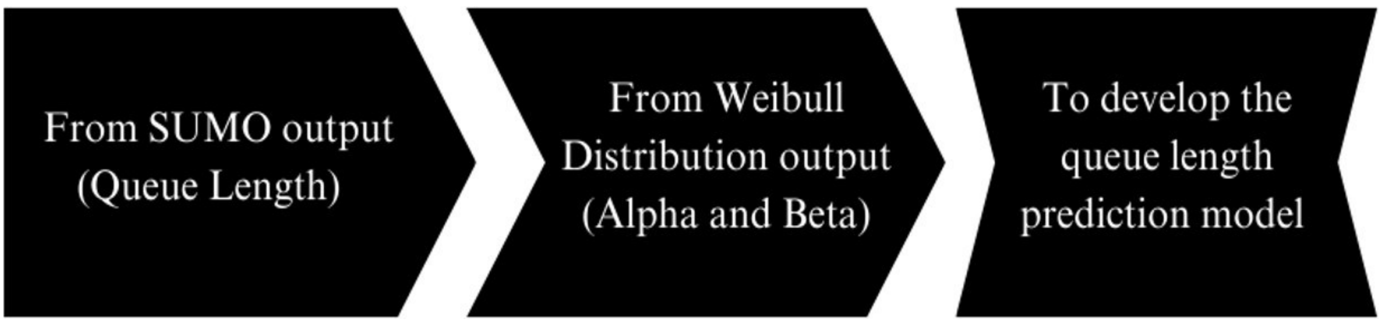


Figure 37: weibull distribution input and output

The two parameter weibull

$$f(t) = \frac{\beta}{\alpha}\left(\frac{t}{\alpha}\right)^{\beta-1} e^{-\left(\frac{t}{\alpha}\right)^{\beta}}$$

Where

Β = shape parameter (or slope)

α = scale parameter or characteristic life

t = time

4.1.1. Characteristics of the Weibull Distribution

The weibull distribution has become one of the most used tools in reliability engineering and life data analysis. As for other distributions, weibull distribution is capable of modeling multiple life behaviors of objects depending on the values of the parameters. At this point, however, this discussion presents how the values of shape parameter, β and scale parameter, α impact on curve shape, reliability and the failure rate. It is worth keeping in mind that the 3-parameter Weibull distribution will continue to be used. The required changes to convert to other forms such as the two-parameter form where alpha equals zero or one where beta equals C and similar variations are readily available.

Effects of the Shape Parameter (β)

The Weibull shape parameter, denoted β is commonly known as a slope since its value actually corresponds to the slope of the regression line in a probability plot. Different values of β significantly influence the characteristics of the distribution. Notably, certain values of this parameter can simplify the Weibull distribution equations into that of other well-known distributions. For instance, when β = 1, the probability density function (pdf) of the 3-parameter Weibull distribution simplifies to the pdf of the 2-parameter exponential distribution.

$$\boldsymbol{f(t) = \frac{1}{\alpha} e^{-\frac{t-\gamma}{\alpha}}}$$

Where $\frac{1}{\alpha} = \lambda$ =failure rate. The parameter β is dimensionless, meaning it has no units. The figure below illustrates how varying the shape parameter β influences the form of the probability density function (pdf). As shown, the pdf can adopt a range of shapes depending on the value of β.

Weibull pdf with 0 < β < 1, β = 1, and β > 1

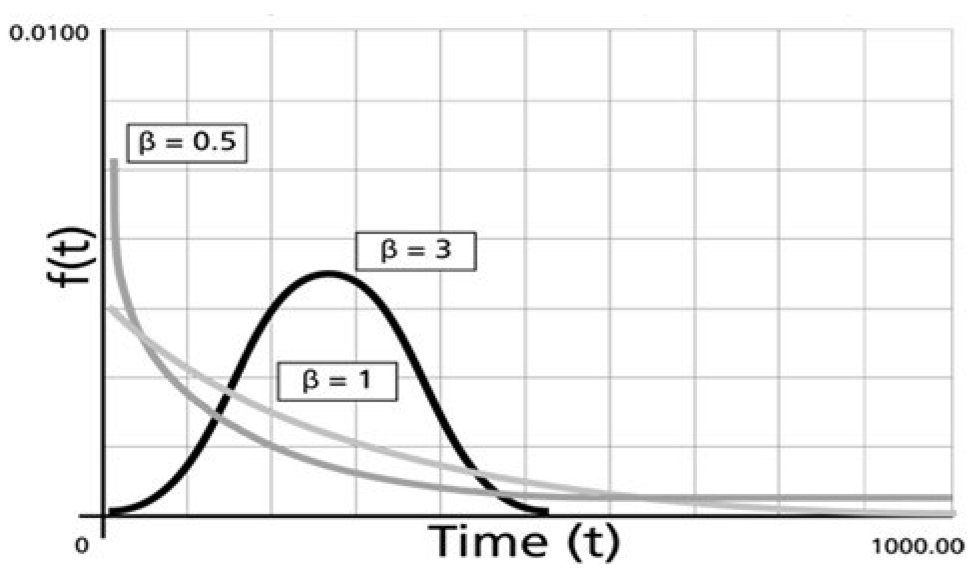


Figure 38: The shape parameter of beta

Effects of the Scale Parameter

Changing the value of the scale parameter α changes the distribution and likewise changes the scale of the horizontal axis. If β is fixed and α is increased, the pdf expands horizontally. Since the area under the pdf curve must remain unity, the height of the pdf top must decrease when α increases, as shown in the below figure.

Weibull pdf plot with varying values of α

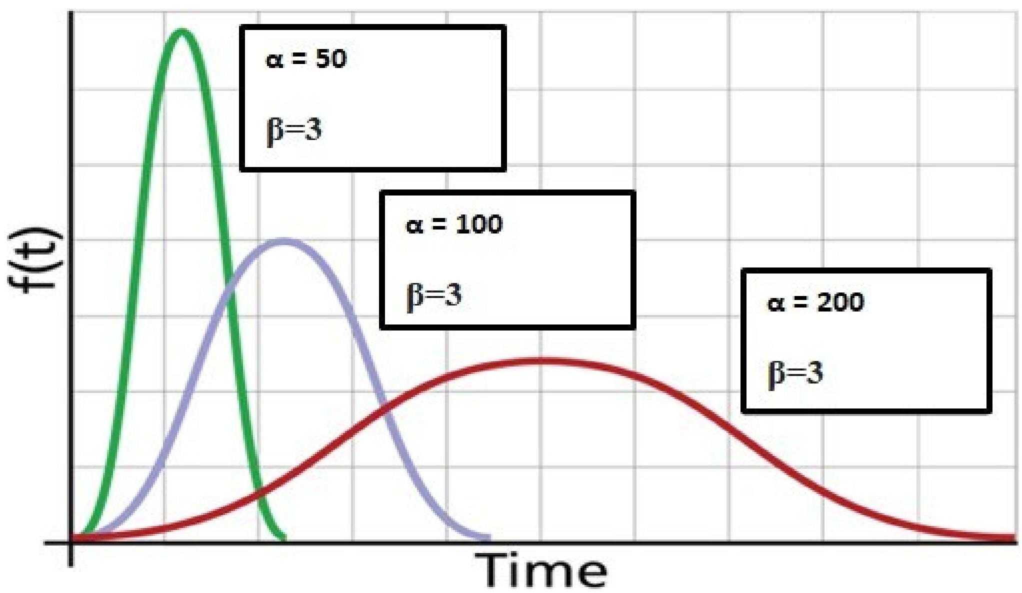


Figure 39: The scale parameter of alpha

- If α is increased while β and γ are kept the same, the distribution gets stretched out to the right and its height decreases, while maintaining its shape and location.
- If α is decreased while β and γ are kept the same, the distribution gets pushed in towards the left (i.e., towards its beginning or towards 0 or γ) and its height increases.
- α has the same units as t, such as hours, miles, cycles, actuations, etc.

4.1.2. Modeling Traffic Queue Lengths Using the Weibull Distribution

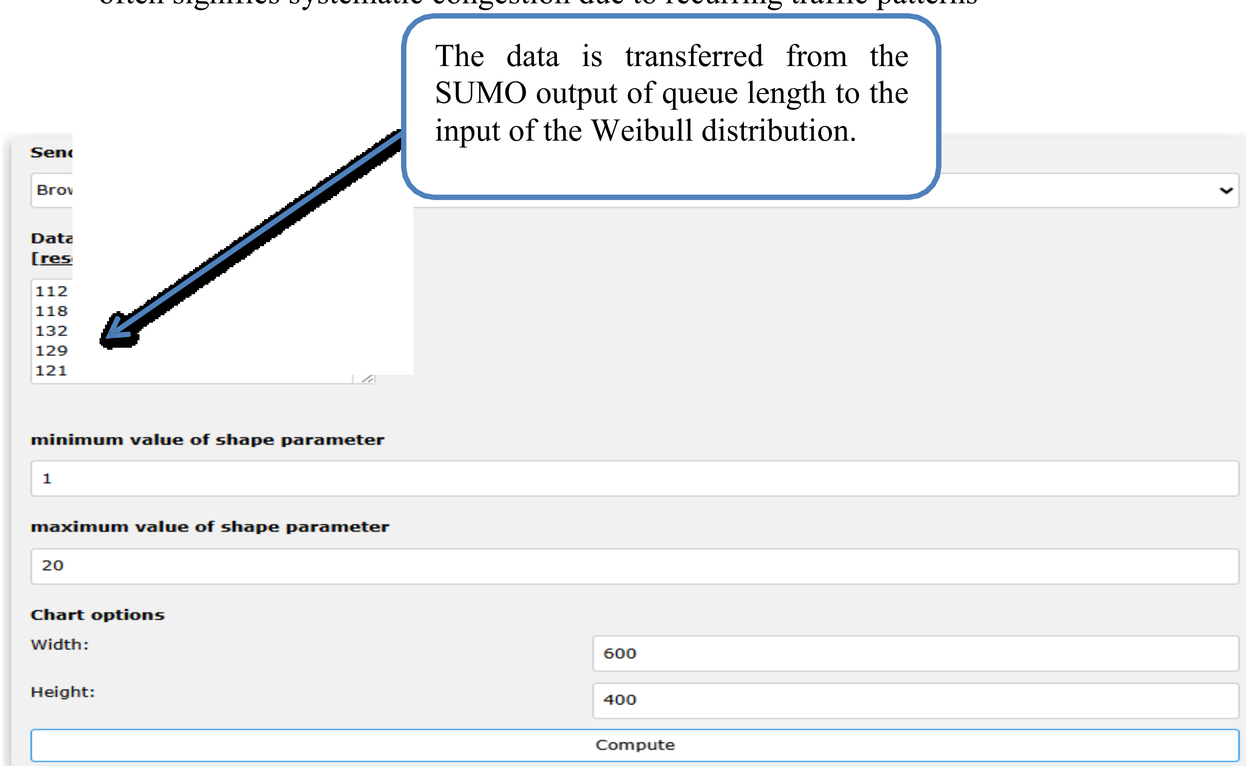
The Weibull distribution is often used to model queue lengths in traffic studies, including those derived from SUMO simulations. The parameters α (scale) and β (shape) of the Weibull distribution can be estimated from the queue length data output by SUMO. The scale parameter, α, reflects the typical magnitude of the queue lengths, while the shape parameter, β indicates how the queue lengths are distributed whether they are more uniform, spread out or skewed. By fitting the Weibull distribution to the queue length data, you can analyze and predict traffic patterns, helping to understand congestion behavior and optimize traffic flow.

The Weibull distribution is a valuable tool for modeling queue lengths in traffic systems, particularly when analyzing data from SUMO simulations. The distribution is defined by two key parameters: the scale parameter (α) and the shape parameter (β), which provides meaningful insights into traffic behavior.

Scale Parameter (α): This parameter represents the average or typical size of queue lengths observed at intersections or along road segments. A higher value of α indicates longer queues, reflecting higher congestion levels, while a lower α corresponds to shorter queues and smoother traffic flow.

Shape Parameter (β): The shape parameter defines the variation or spread of the queue lengths. It helps to characterize how the queue lengths are distributed. For instance:

- When β<1, queue lengths tend to be shorter and more variable, indicating a high likelihood of transient or sporadic congestion.
- When β=1, the distribution takes an exponential form, suggesting a steady decrease in the probability of longer queues.
- When β>1, queue lengths are more consistent, clustering around certain values, which often signifies systematic congestion due to recurring traffic patterns



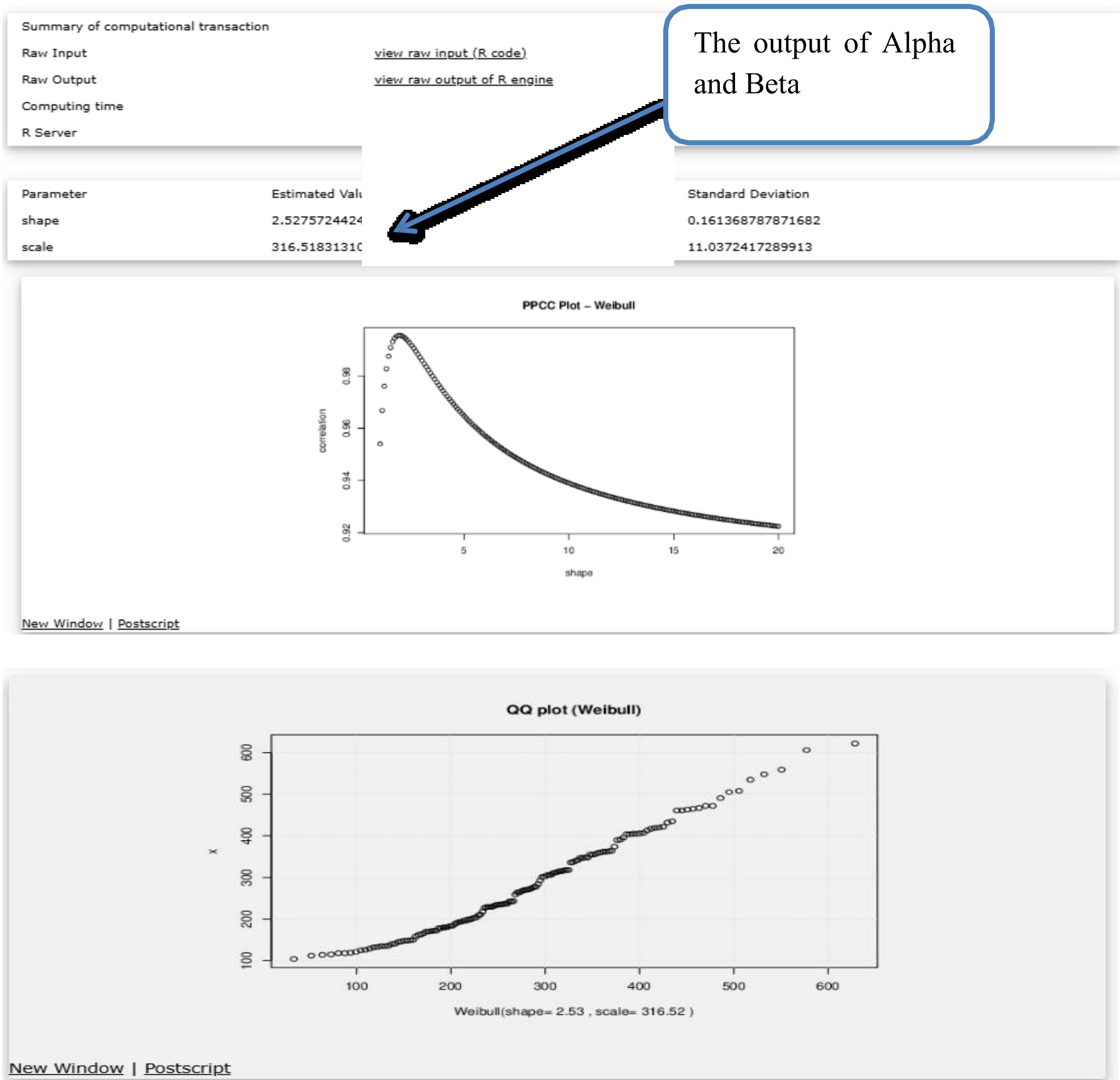


Figure 40: Maximum-likelihood Fitting - Weibull distribution software (calculator)

Table 6: The output of Weibull distribution alpha and beta results

Intersections Area	Upstream or Downstream	Approach	β	α
Kebela 24	Upstream	Left turn	1.850	55.014
Kebela 24	Upstream	Right turn	2.644	58.006
Kebela 24	Upstream	Through & Right turn	1.668	40.087
Kebela 24	Upstream	Through	2.051	35.550
Kebela 24	Upstream	Through & Left turn	2.334	60.050
Kebela 24	Downstream	Through & Right turn	1.034	20.622
Kebela 24	Downstream	Through	1.079	18.292
Kebela 24	Downstream	Through & Left turn	1.039	24.537
Shola Gebeya	Upstream	Left turn	2.807	72.606
Shola Gebeya	Upstream	Right turn	2.396	70.135
Shola Gebeya	Upstream	Through & Right turn	1.975	37.125
Shola Gebeya	Upstream	Through	2.126	42.643
Shola Gebeya	Upstream	Through & Left turn	2.031	41.308
Shola Gebeya	Downstream	Through & Right turn	1.869	50.743
Shola Gebeya	Downstream	Through	1.934	49.261
Shola Gebeya	Downstream	Through & Left turn	2.100	45.422
Post Office	Upstream	Left turn	1.902	42.090
Post Office	Upstream	Right turn	1.700	50.859
Post Office	Upstream	Through _1	1.300	30.322
Post Office	Upstream	Through _2	1.276	31.958
Post Office	Upstream	Through & Left turn	2.036	41.230
Post Office	Downstream	Through _1	1.670	42.124
Post Office	Downstream	Through _2	1.025	30.495
Post Office	Downstream	Through & Left turn	2.000	38.644

4.1.3. Multiple Linear Regression

Multiple linear regressions is said to be a technique for the study of a dependent variable with more than one independent variable. In fact, it is used to predict the value of a dependent variable based on already known values of independent variables. The method assigns weight (coefficient) to each independent variable, indicating its effect on the dependent variable. These weights are computed in such a way that the error difference between predicted and actual dependent variable values gets minimized. Then, the relative sizing of weights represents the strength as well as the direction (positive or negative) in which every independent variable contributes towards the prediction. Hence, this technique would be widely used in various sectors to analyze the complex and informed forecast.

$$\mathbf{y} = \boldsymbol{a_0} + \boldsymbol{a_1 x_1} + \boldsymbol{a_2 x_2} + \boldsymbol{a_3 x_3} \ldots\ \boldsymbol{a_n x_n}$$

Where

y is the dependent variable,

x_1, x_2, x_3, …x_n are the independent variables.

a_0 is the y-intercept.

a_1, a_2, a_3,…a_n are the coefficients (weights) of the independent variables

4.2. Results

This section seeks to present and interpret the findings of the research study. It addresses some trends, patterns, and correlations identified, as well as the context in which the analyzed data are interpreted. They relate the findings to the research questions or hypotheses and finally, the said findings develop into a predictive model.

4.2.1. Factors Affecting Consecutive Signal Intersections

Green Time

The synchronization of green time in terms of upstream and downstream signals at consecutive signalized intersections should ensure the smooth passage of vehicles. It does this by adjusting the beginning of the green phase at the downstream signal according to the time that a vehicle traveling from the upstream signal takes to arrive at the downstream signal, also known as the offset. When such an adjustment is done, if it takes 10 seconds for vehicles to get from one intersection to another, then the beginning of the green phase at the downstream signal should start 10 seconds after the green phase at the upstream signal goes on. This is what "green wave" is referred to as, and it helps to minimize stopping, reduce actual time delay and improve general efficiency.

The following graph is generated using the green time data from Table 2 and the alpha & beta parameters obtained from the Weibull distribution output in Table 6.

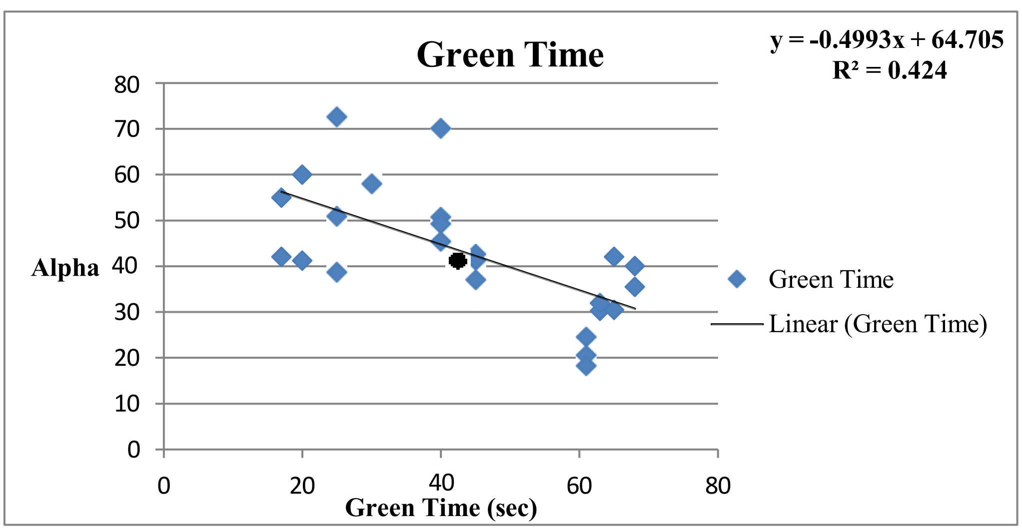


Figure 41: Relationship between alpha and green time

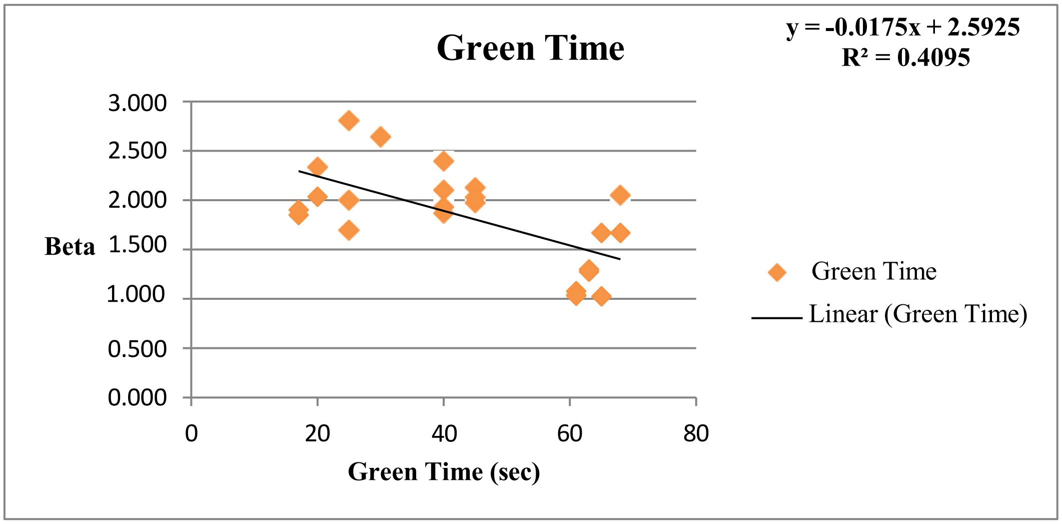


Figure 42: Relationship between beta and green time

As the figures of 37 and 38 shows, the dependent variable (alpha and beta) and green time are inversely correlated. If the green time at a signalized intersection increases, more vehicles can pass the intersection on each cycle, which reduces the probability of queue formation and facilitates the traffic flow. Thus, the increased green time which mitigates the traffic density or delays is highly efficient in improving the flow so that the value of alpha and beta will decrease as well.

Pedestrians Volume

There is no specific pedestrian signals available meaning that pedestrians have to literally navigate across busy intersections. This practice affects vehicle flow quite a bit as some drivers tend to slow down if not completely stop, in more cases just to allow the pedestrian to pass. This hinders traffic movement as delays are increased in the number of vehicles passing each intersection. The result is caused by the quantity of pedestrians crossing with a reliance on the gaps created with vehicles this leading to unpredictable interruptions and serious congestion loss in front of both pedestrian routes.

The following graph is generated using the pedestrian volume data from Table 4 and the alpha & beta parameters obtained from the Weibull distribution output in Table 6.

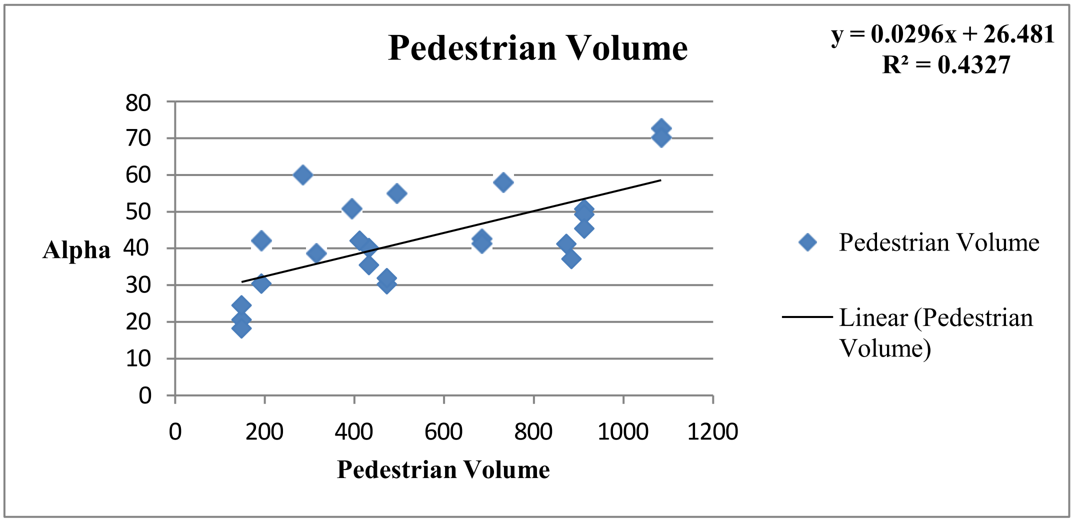


Figure 43: Relationship between alpha and pedestrian volume

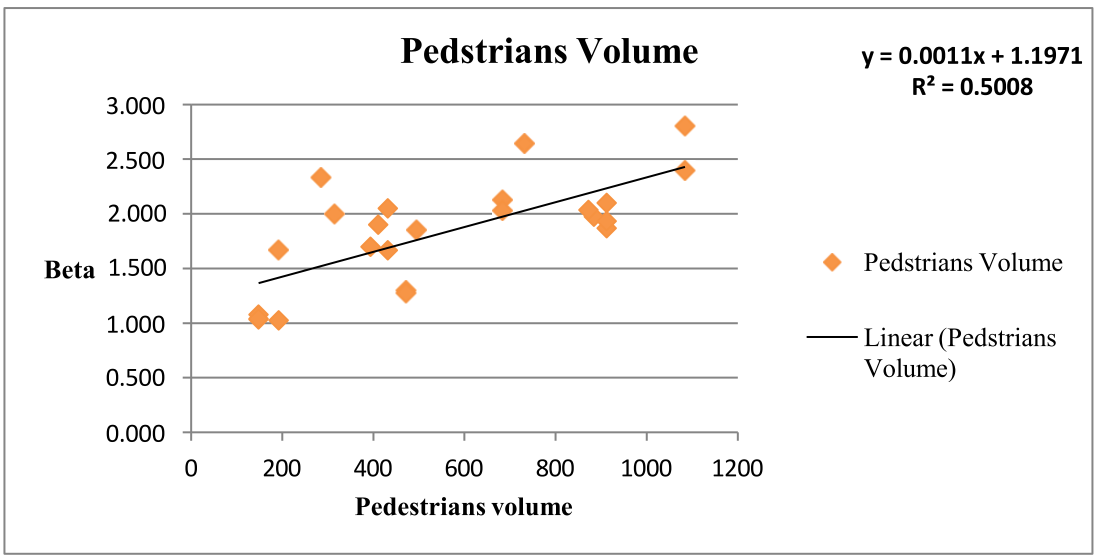


Figure 44: Relationship between beta and pedestrian volume

As the figures of 39 and 40 shows, the relationship between pedestrian traffic and the dependent variable (alpha and beta) is direct. If there is high pedestrian traffic during vehicle green phases, pedestrians will cross while vehicles are still moving; then this affects the vehicular flow and reducing the effective green time for vehicles. More vehicles join the queues, resulting in a higher probability of queue formation and increasing alpha and beta.

Proportion of Buses

The presence of slower or larger vehicles, especially the city buses, disrupts the smooth functioning of traffic flow at intersections causing greater delays, reducing the capacities of intersections. This is because they exhibit slower acceleration and deceleration properties which generate longer headways. These headways restrict the number of vehicles which may pass through an intersection during the green light phase. Moreover, buses tend to create large gaps-periodically-in traffic, inhibiting the formation of natural platoons of vehicle formation. From their frequent stops and slow movements, the gaps further get spread and are inefficient as well. The startup lost time, which the time is taken for the signal change to allow delay in movement of the vehicles at the stop, is increased by having buses lead in the queue at a stop. Other factors related to the age of vehicles, which affect acceleration and their lengths straining the traffic stream additionally decrease effective intersection capacity and serve to increase congestion.

The following graph is generated using the proportion of buses data from Table 1 and the alpha & beta parameters obtained from the Weibull distribution output in Table 6.

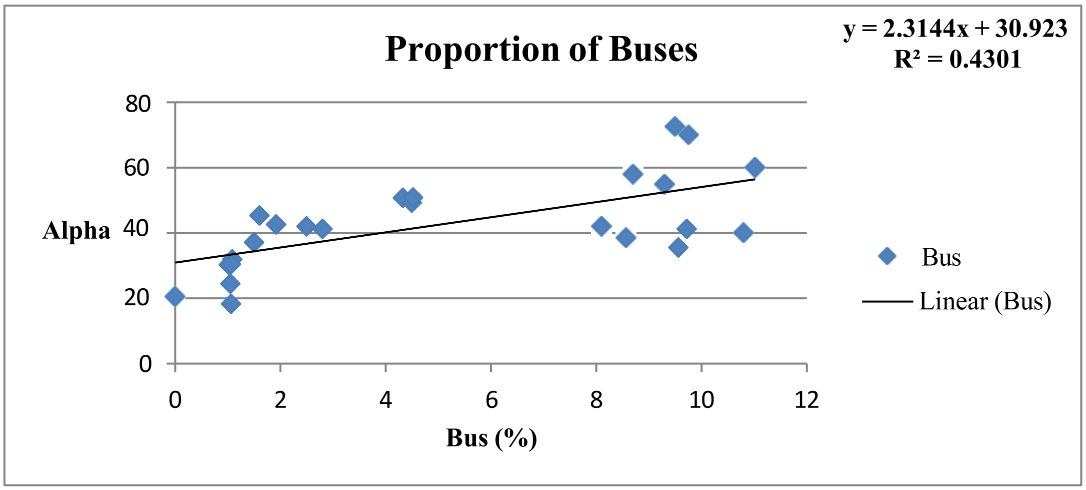


Figure 45: Relationship between alpha and proportion of buses

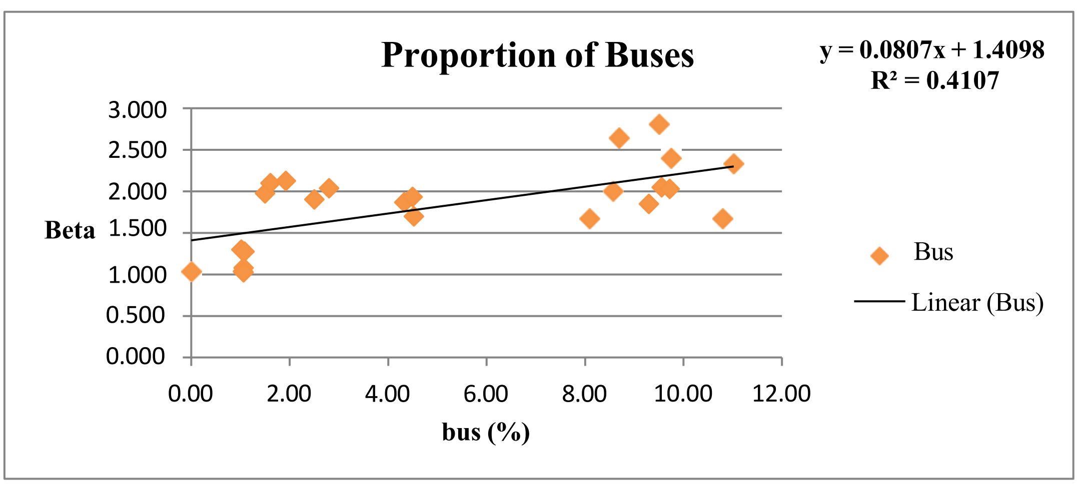


Figure 46: Relationship between beta and proportion of buses

As the figure of 41 and 42, proportion of buses in a vehicle composition and alpha and beta has direct relationship. Therefore, the increasing number of buses will also bring an increase in alpha and beta. Since buses occupy more space, move slower and have a long startup time compared to small vehicles, there will be more delays at the intersections during the phases of green light. This reduced net flow rate of vehicles leads to a longer probability of queue formation.

Green Time to Cycle Time Ratio (g/c)

The Green Time to Cycle Time Ratio (g/c), as a key factor in the design and analysis of a traffic signal, stands for the fraction of the whole cycle period during which the green light is displayed at a signal for a particular movement or approach. It directly contributes to the intersection capacity because it specifies how much time in the total cycle a vehicle or pedestrian uses to pass through the signalized junction. High g/c ratios imply a larger portion of the cycle is allocated to green time it generally leads to reduced vehicle delay and improved traffic flow for the movement receiving more green time. Conversely, low ratios will tend to indicate less green time available, therefore creating possible congestion or delay. Balancing these across the competing approaches is important in optimizing the intersection performance to ensure good traffic operations.

The following graph is generated using the green time to cycle time ratio (g/c) data from Table 3 and the alpha & beta parameters obtained from the Weibull distribution output in Table 6.

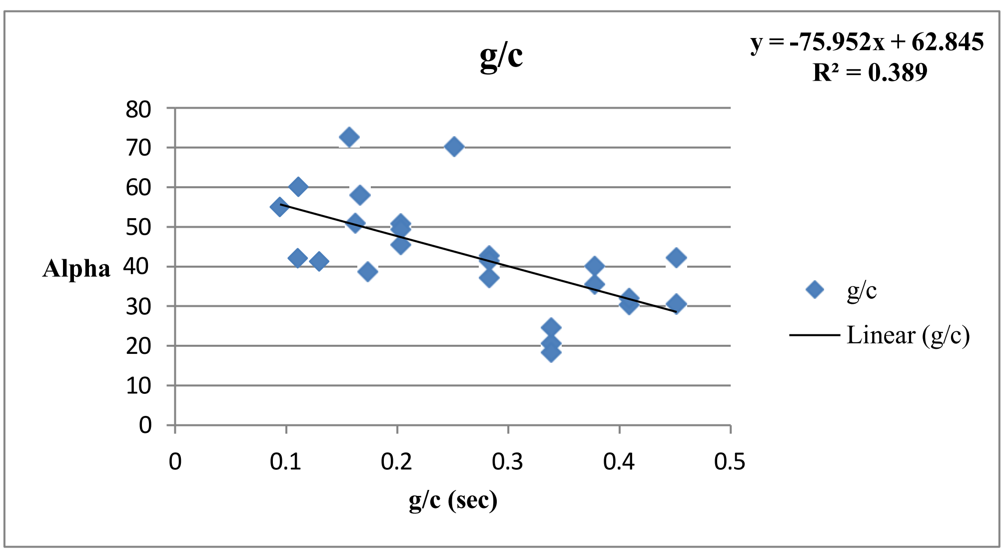


Figure 47: Relationship between alpha and g/c

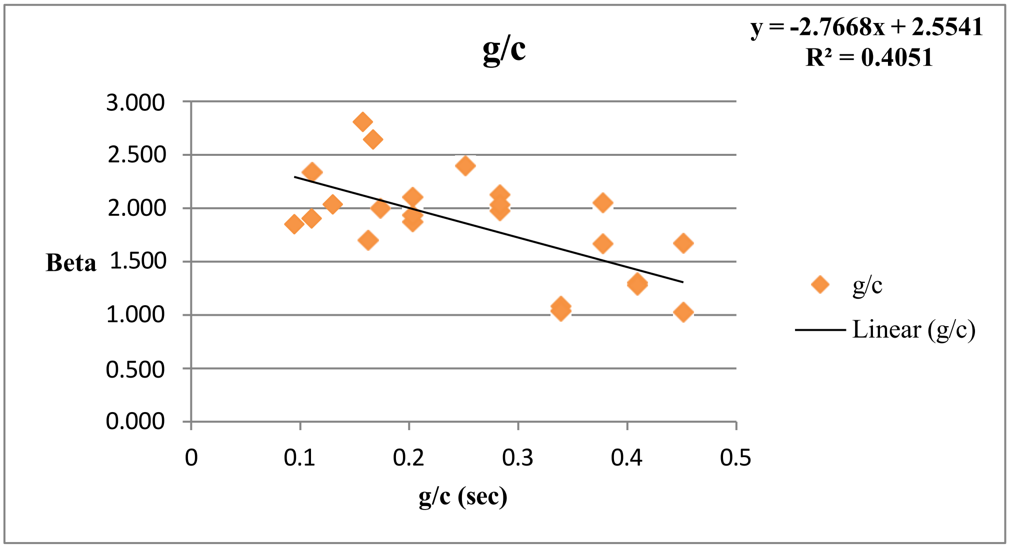


Figure 48: Relationship between beta and g/c

As the figures of 43 and 44 shows, the Green Time to Cycle Time Ratio (g/c) and the dependent variable (alpha and beta) have an opposite relationship. Increasing the value of (g/c) is likely to decrease the value of alpha and beta because more green time during every signal cycle permits more vehicles to pass the intersection, thus leading to less waiting time. Those low waiting times, leading to short probability of queues formation and hence a low alpha and beta value. Contrary to this, with a low g/c ratio, little green time is provided and leads to the accumulation of more vehicles at the intersection, increasing both the alpha and beta.

4.2.2. Investigating Variable Correlations

Correlation analysis is important step in analyzing data before modeling, leading to the detection of multicollinearity between a set of independent variables manifested by strong correlations. Particularly, the multicollinearity effects on regression models manifest as an unstable regression coefficient, increased standard errors, and unreliable results. The correlated variables then serve in the selection of the best variables for a model. Knowledge of relationships between variables is critical when building predictive models so that the most impactful variables can be selected for the target variable. The candidate returned strong correlations with the target variable because they are likely to contain information relevant to prediction.

Table 7: Summary of variable correlations

Factors	β	α	Green time	Pedestrian volume	P. Buses (%)	g/c
β	1					
α	0.85675346	1				
Green time	-0.6399279	-0.6511379	1			
Pedestrians volume	0.70767662	0.65783418	-0.398139878	1		
P. Buses (%)	0.640844	0.65582855	-0.289057579	0.175192312	1	
g/c	-0.6364795	-0.6237199	0.967156905	-0.424098638	-0.2879941	1

Green time, Pedestrian volume and proportion of buses were found as independent variables developing their own identity as none of them had weak statistical correlation with any other predetermined independent variables, keeping a significant correlation existing with alpha and beta. In a data set with highly correlated variables, it's critical to find a core set of predictor variables that is not unfairly effacing one another's coefficient. In this case green time, pedestrian volume and proportion of buses are giving unique, reliable contributions to the model without redundancy. Green Time has negative correlation with alpha and beta. Pedestrian volume and proportion of buses has a positive correlation. Green time, pedestrian volume and proportion of buses will construct harmonious and less complicated but stable integrated model without adding the high collinearity complication.

4.2.3. Quean length probabilistic Prediction Model

The prediction model helps to quantify the relationships between the independent variables (Green time, pedestrian volume and proportion of buses) and the dependent variable (Alpha and Beta). It reveals the impact of change in each independent variable on the alpha and beta. So once the model created, it can make predictions about alpha and beta in scenarios where there are values for the independent variables but want to estimate the corresponding alpha and beta.

A predictive model was created using multiple linear regressions (MLR) to forecast the alpha and beta within a corridor level. The data analysis tool employed for this purpose was the MS-Excel Data Analysis Tool Pak. The model was constructed using several variables: Green time, pedestrian volume and proportion of buses were considered explanatory factors, while the alpha and beta served as the dependent variable. A dataset consisting of three consecutive signalized intersections (six intersections), as outlined in the table below, was utilized to formulate and refine this predictive model.

Table 8: Summary of dependent and independent variables

Intersections Area	Upstream or Downstream	Approach	β	α	Green time	Pedestrians volume	Bus (%)
Kebela 24	Upstream	LT	1.850	55.014	17	495	9.30
Kebela 24	Upstream	RT	2.644	58.006	30	732	8.70
Kebela 24	Upstream	Through & RT	1.668	40.087	68	432	10.80
Kebela 24	Upstream	Through	2.051	35.550	68	432	9.56
Kebela 24	Upstream	Through & LT	2.334	60.050	20	285	11.02
Kebela 24	Downstream	Through & RT	1.034	20.622	61	148	0.00
Kebela 24	Downstream	Through	1.079	18.292	61	148	1.07
Kebela 24	Downstream	Through & LT	1.039	24.537	61	148	1.05
Shola Gebeya	Upstream	LT	2.807	72.606	25	1084	9.50
Shola Gebeya	Upstream	RT	2.396	70.135	40	1084	9.76
Shola Gebeya	Upstream	Through & RT	1.975	37.125	45	884	1.50
Shola Gebeya	Upstream	Through	2.126	42.643	45	684	1.92
Shola Gebeya	Upstream	Through & LT	2.031	41.308	45	684	9.72
Shola Gebeya	Downstream	Through & RT	1.869	50.743	40	912	4.33
Shola Gebeya	Downstream	Through	1.934	49.261	40	912	4.50
Shola Gebeya	Downstream	Through & LT	2.100	45.422	40	912	1.61
Post Office	Upstream	LT	1.902	42.090	17	411	2.50
Post Office	Upstream	RT	1.700	50.859	25	394	4.52
Post Office	Upstream	Through _1	1.300	30.322	63	472	1.02
Post Office	Upstream	Through _2	1.276	31.958	63	472	1.08
Post Office	Upstream	Through & LT	2.036	41.230	20	872	2.80
Post Office	Downstream	Through _1	1.670	42.124	65	192	8.10
Post Office	Downstream	Through _2	1.025	30.495	65	192	1.06
Post Office	Downstream	Through & LT	2.000	38.644	25	315	8.57

Multiple Linear Regression model is used to predict Alpha and beta. This model was developed using the MS-Excel Data Analysis Tool Pak. The model has a form of the equation:

$$\mathbf{y = a_0 + a_1x_1 + a_2x_2 + a_3x_3 \ldots\ a_nx_n}$$

Where

y is Alpha and Beta (a dependent variable).

$x_1, x_2, x_3, \ldots x_n$ are the independent variables.

a_0 is the y-intercept.

$a_1, a_2, a_3 \ldots\ldots, a_n$ are the coefficients of the independent variables.

Table 9: Multiple liner regression output Summary of alpha

Regression Statistics	
Multiple R	0.9078547
R Square	0.82420016
Adjusted R Square	0.79783018
Standard Error	6.25831837
Observations	24

ANOVA

	Df	*SS*	*MS*	*F*	*Significance F*
Regression	3	3672.480653	1224.1602	31.25525	9.56254E-08
Residual	20	783.3309764	39.166549		
Total	23	4455.81163			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	34.4502342	5.737766567	6.0041192	7.18E-06	22.48146292	46.419006	22.4814629	46.4190056
Green time (x_1)	-0.2584919	0.080794956	-3.199356	0.004503	-0.42702718	-0.089957	-0.42702718	-0.08995653
Pedestrians volume (x_2)	0.01976688	0.004609792	4.2880203	0.000359	0.010151024	0.0293827	0.01015102	0.02938274
P.Bus (%) (x_3)	1.69897593	0.346426513	4.9042896	8.57E-05	0.97634289	2.421609	0.97634289	2.42160898

The fitted regression model gives the equation for multiple linear regressions

$$\mathbf{y} = \boldsymbol{a_0 + a_1 x_1 + a_2 x_2 + a_3 x_3 \ldots\ a_n x_n}$$

$$\boldsymbol{\alpha = 34.4502342 - 0.2584919x_1 + 0.01976688x_2 + 1.69897593x_3}$$

Where

α is the dependent variable

x_1 is green time (second)

x_2 is pedestrians volume

x_3 is bus (%)

Table 10: Multiple liner regression output Summary of beta

Regression Statistics	
Multiple R	0.9219018
R Square	0.8499029
Adjusted R Square	0.8273883
Standard Error	0.2064311
Observations	24

ANOVA

	Df	*SS*	*MS*	*F*	*Significance F*
Regression	3	4.825886753	1.6086	37.749	1.9956E-08
Residual	20	0.852276047	0.0426		
Total	23	5.6781628			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	1.4397252	0.189260667	7.6071	2.5E-07	1.04493433	1.834516	1.04493433	1.834515995
Green time (x_1)	-0.0083395	0.002665028	-3.129	0.00528	-0.0138987	-0.00278	-0.01389866	-0.00278036
Pedestrians volume (x_2)	0.0008113	0.000152054	5.3353	3.2E-05	0.00049408	0.0011284	0.00049408	0.001128442
P.Bus (%) (x_3)	0.0584919	0.011426905	5.1188	5.2E-05	0.03465575	0.082328	0.03465575	0.082327958

The fitted regression model gives the equation for multiple linear regressions

$$\mathbf{y = a_0 + a_1x_1 + a_2x_2 + a_3x_3 \ldots\ a_nx_n}$$

$$\boldsymbol{\beta = 1.4397252 - 0.0083395x_1 + 0.008113x_2 + 0.0584919x_3}$$

Where

β is the dependent variable

x_1 is green time (second)

x_2 is pedestrians volume

x_3 is bus (%)

A multiple linear regression model was used in this analysis to investigate the relationships between alpha and beta (the dependent variable) and independent variables like green time, pedestrian volume and proportion of buses. It was evaluated in terms of goodness of fit and significance of independent variables. The higher adjusted R^2 value shows that the model explains more variation in the dependent variable. As a result, this choice served well as an appropriate model for predicting the impact of the key factors on alpha and beta.

The result from the multiple linear regressions shows two key relationships. First, it presents a positive relationship between the pedestrian volume and proportion of buses with the dependent variable (alpha and beta). This means the greater the pedestrian volume or the proportion of buses, the higher the probability that the queue formation will be longer. Second, there is a negative relation between green time and the dependent variable (alpha and beta) where the higher the green time, the lesser is the probability with which the queues formation would be longer. These findings underscore the importance of considering various factors in understanding and optimizing consecutive traffic signals, with green time playing facilitative roles, while higher pedestrian volume and proportion of buses optimal flow.

The regression summary statistics revealed a multiple R value of the dependent variable (alpha and beta) 0.9078 and 0.9219, signifying the multiple correlations between the response (dependent) variable and the three explanatory (independent) variables. This demonstrates the strength of the combined predictor set with respect to the response variable. It establishes a relationship between a dependent variable and its linear combination of predictors rather than with just one predictor alone. The value 0.9078 and 0.9219, which is very close to 1, indicates a strong relationship between the response variable and the combined set of independent variables.

The regression gave an R^2 value of 0.8242 and 0.8499, which indicates a goodness fit of the model with the input data. This means roughly 82.42 and 84.99 percent of variations or changes in the dependent variable. In this case, the probability queue formation could be explained by the changes in the three independent variables (green time, pedestrian volume and proportion of buses). The adjusted R^2 value, 0.7978 and 0.8273, states that it accounts for 79.78 and 82.73 percent of this variation while correcting for biases developed when several independent variables are included in the model. This value for adjusted R^2 is particularly useful in this case because it gives a more accurate measure of the performance of the model since it was adjusted by the number of predictors included, ensuring that the model's explanatory power is accurately captured.

According to the ANOVA output, since the significance F equals $9.56254E^{-08}$ and $1.9956E^{-08}$, which is less than 0.05, it can be concluded that the model is statistically significant. Also, the P-value of all the independent variables in the equation is less than 0.05, which implies that the independent variables in the model, except those variables not found in the equation, are statistically significant to predict the alpha and beta.

The coefficients of the multiple linear regressions show that, the scale at which the respective independent variables (Green time, Pedestrian volume and proportion of buses) affect the dependent variable (Alpha and beta). As a result the pedestrian volume and proportion of buses increases there effect by alpha (0.0197 and 1.6989) and beta (0.0008 and 0.0584) respectively. Conversely, the increase of Green time will reduce alpha and beta by 0.258 and 0.008.

Table 11: Summary of alpha and beta

Dependent Variables	Independent Variables	Coefficients	P-value	Adjusted R Square
α	Intercept	34.45023	7.18E-06	0.7978302
	Green Time (x_1)	-0.2584919	0.0045028	
	Pedestrian Volume (x_2)	0.01976688	0.0003588	
	P. Buses (x_3)	1.69897593	8.57E-05	
β	Intercept	1.4397252	2.50E-07	0.8273883
	Green Time (x_1)	-0.0083395	0.0052824	
	Pedestrian Volume (x_2)	0.0008113	3.20E-05	
	P. Buses (x_3)	0.0584919	5.23E-05	

$$f(Q) = \frac{\beta}{\alpha}\left(\frac{Q_L}{\alpha}\right)^{\beta-1} e^{-\left(\frac{Q_L}{\alpha}\right)^{\beta}}$$

Where

$\alpha = 34.4502342 - 0.2584919x_1 + 0.01976688x_2 + 1.69897593x_3$

$\beta = 1.4397252 - 0.0083395x_1 + 0.008113x_2 + 0.0584919x_3$

x_1 = green time

x_2 = pedestrian volume

x_3 = proportion of buses

Q_L = the probability of queue length

The root mean square error (RMSE) and mean absolute percentage error also calculated. As we can see below the values of both RMSE and MAPE, the average magnitude of error produced by a model (MAPE) and the average difference between values predicted by a model and the actual value are small. All RMSE, MAPE and adjusted R^2 tell the independent variables are good in explaining the dependent variables.

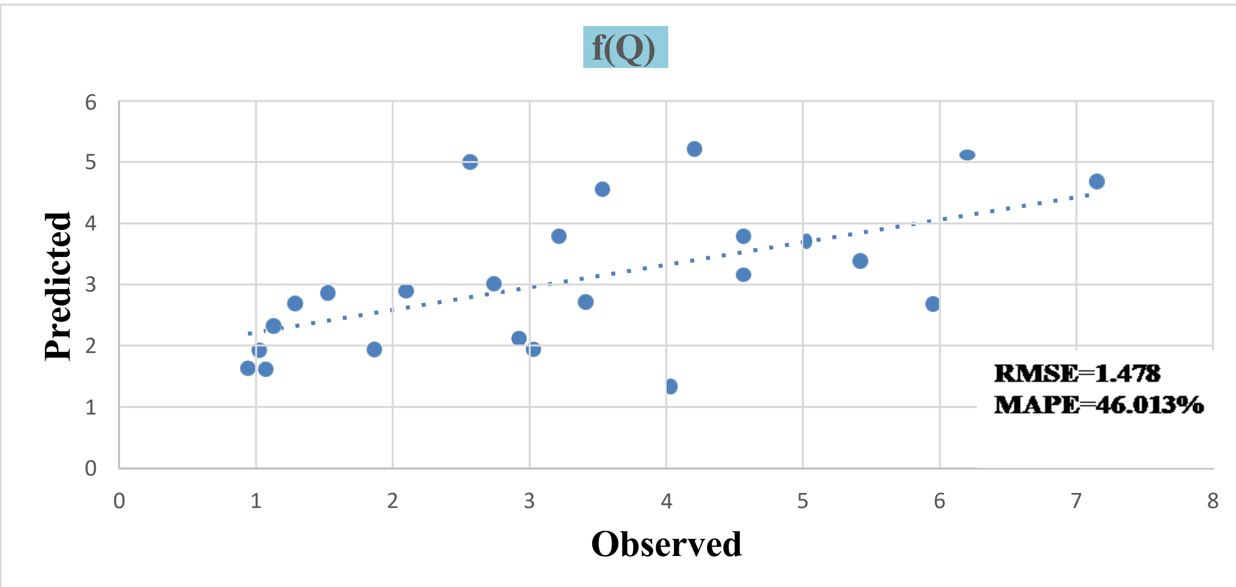


Figure 49: RMSE and MAPE of f(Q)

4.2.4. Effect of green time on probability of queue formation

As shown in the graph below when the green time allocation to an intersection is 68 seconds, vehicles are given more time to pass through the intersection, so that the probability of formation of lower queues is higher. In contrast as shown in the graph below when the green time becomes 17 second the probability of lower queue formation is low. Accommodate the incoming traffic demand, particularly during peak hours or periods of high vehicle volume. Therefore, optimizing the green time allocation based on traffic flow patterns is crucial to reducing queue lengths and improving intersection performance.

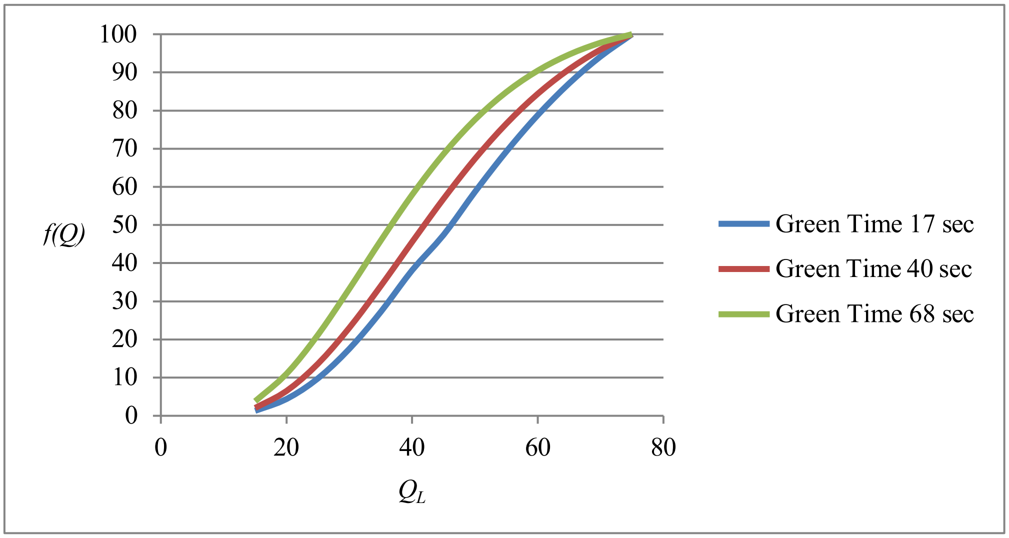


Figure 50: The probability of queue formation of green time

Table 12: The probability of queue formation of green time calculation

Effect of green time on probability of queue formation											
Green time 17 sec				Green time 40 sec				Green time 68 sec			
alpha	beta	f(Q)	Q_L	alpha	beta	f(Q)	Q_L	alpha	beta	f(Q)	Q_L
55.641	5.858	0.003	15.000	49.696	5.666	0.006	15.000	42.458	5.433	0.011	15.000
55.641	5.858	0.008	20.000	49.696	5.666	0.013	20.000	42.458	5.433	0.022	20.000
55.641	5.858	0.014	25.000	49.696	5.666	0.021	25.000	42.458	5.433	0.030	25.000
55.641	5.858	0.020	30.000	49.696	5.666	0.027	30.000	42.458	5.433	0.036	30.000
55.641	5.858	0.025	35.000	49.696	5.666	0.032	35.000	42.458	5.433	0.038	35.000
55.641	5.858	0.028	40.000	49.696	5.666	0.033	40.000	42.458	5.433	0.036	40.000
55.641	5.858	0.024	45.000	49.696	5.666	0.033	45.000	42.458	5.433	0.032	45.000
55.641	5.858	0.029	50.000	49.696	5.666	0.030	50.000	42.458	5.433	0.027	50.000
55.641	5.858	0.027	55.000	49.696	5.666	0.027	55.000	42.458	5.433	0.022	55.000
55.641	5.858	0.025	60.000	49.696	5.666	0.023	60.000	42.458	5.433	0.017	60.000
55.641	5.858	0.022	65.000	49.696	5.666	0.019	65.000	42.458	5.433	0.013	65.000
55.641	5.858	0.018	70.000	49.696	5.666	0.015	70.000	42.458	5.433	0.009	70.000
55.641	5.858	0.015	75.000	49.696	5.666	0.012	75.000	42.458	5.433	0.007	75.000

4.2.5. Effect of pedestrian volume on probability of queue formation

As shown in the graph below when the pedestrian volume to an intersection is 684, there is more pedestrian volume to cross through the intersection, so that the probability of formation of higher queues is higher. In contrast as shown in the graph below when the pedestrian volume becomes

285 the probability of lower queue formation is low. Accommodate the incoming traffic demand, particularly during peak hours or periods of high vehicle volume.

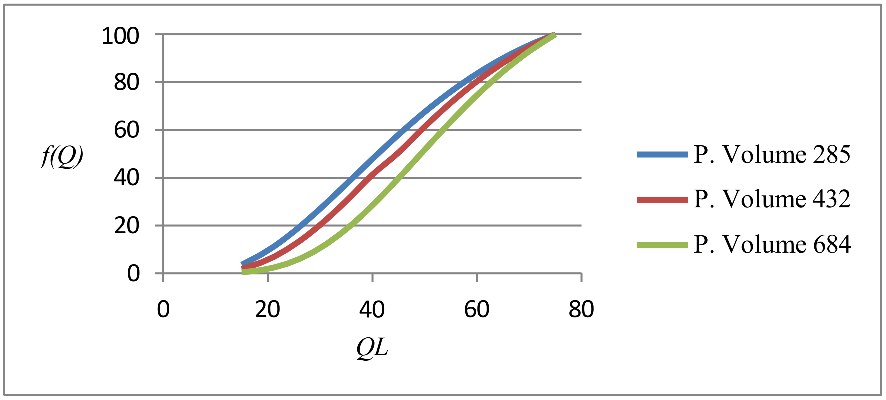


Figure 51: The probability of queue formation of pedestrian volume

Table 13: The probability of queue formation of pedestrian volume calculation

Effect of pedestrian volume on probability of queue formation											
Pedestrian volume 285				Pedestrian volume 432				Pedestrian volume 684			
alpha	beta	f(Q)	Q_L	alpha	beta	f(Q)	Q_L	Alpha	beta	f(Q)	Q_L
51.490	4.154	0.076	15.00	54.396	5.347	0.009	15.00	59.377	7.391	0.000188	15.00
51.490	4.154	0.125	20.00	54.396	5.347	0.018	20.00	59.377	7.391	0.000636	20.00
51.490	4.154	0.169	25.00	54.396	5.347	0.030	25.00	59.377	7.391	0.00142	25.00
51.490	4.154	0.201	30.00	54.396	5.347	0.040	30.00	59.377	7.391	0.002444	30.00
51.490	4.154	0.218	35.00	54.396	5.347	0.048	35.00	59.377	7.391	0.003513	35.00
51.490	4.154	0.222	40.00	54.396	5.347	0.052	40.00	59.377	7.391	0.004426	40.00
51.490	4.154	0.215	45.00	54.396	5.347	0.044	45.00	59.377	7.391	0.005042	45.00
51.490	4.154	0.200	50.00	54.396	5.347	0.052	50.00	59.377	7.391	0.005306	50.00
51.490	4.154	0.180	55.00	54.396	5.347	0.048	55.00	59.377	7.391	0.005236	55.00
51.490	4.154	0.159	60.00	54.396	5.347	0.043	60.00	59.377	7.391	0.0049	60.00
51.490	4.154	0.136	65.00	54.396	5.347	0.037	65.00	59.377	7.391	0.004386	65.00
51.490	4.154	0.115	70.00	54.396	5.347	0.031	70.00	59.377	7.391	0.00378	70.00
51.490	4.154	0.096	75.00	54.396	5.347	0.026	75.00	59.377	7.391	0.003153	75.00

4.2.6. Effect of proportion of buses on probability of queue formation

When the proportion of buses in a vehicle composition crossing an intersection is 9.3 the probability of formation of higher queues is higher. In contrast as shown in the graph below when the bus proportion becomes 0 the probability of lower queue formation is high.

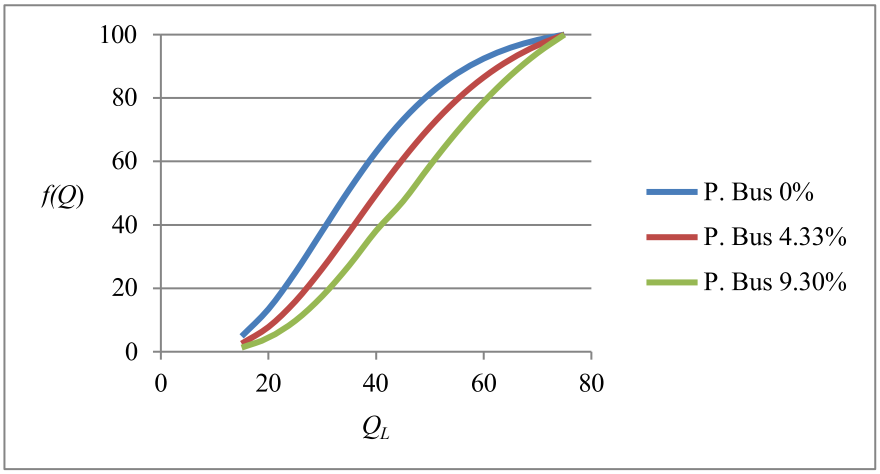


Figure 52: The probability of queue formation of bus

Table 14: The probability of queue formation of bus calculation

Effect of proportion of buses on probability of queue formation											
Bus 0%				Bus 4.33%				Bus 9.30%			
alpha	beta	f(Q)	Q_L	Alpha	beta	f(Q)	Q_L	alpha	beta	f(Q)	Q_L
39.840	5.314	0.015	15.000	47.195	5.567	0.008	15.000	55.641	5.858	0.003	15.000
39.840	5.314	0.027	20.000	47.195	5.567	0.016	20.000	55.641	5.858	0.008	20.000
39.840	5.314	0.036	25.000	47.195	5.567	0.024	25.000	55.641	5.858	0.014	25.000
39.840	5.314	0.040	30.000	47.195	5.567	0.031	30.000	55.641	5.858	0.020	30.000
39.840	5.314	0.040	35.000	47.195	5.567	0.035	35.000	55.641	5.858	0.025	35.000
39.840	5.314	0.037	40.000	47.195	5.567	0.036	40.000	55.641	5.858	0.028	40.000
39.840	5.314	0.031	45.000	47.195	5.567	0.034	45.000	55.641	5.858	0.024	45.000
39.840	5.314	0.025	50.000	47.195	5.567	0.030	50.000	55.641	5.858	0.029	50.000
39.840	5.314	0.020	55.000	47.195	5.567	0.026	55.000	55.641	5.858	0.027	55.000
39.840	5.314	0.015	60.000	47.195	5.567	0.021	60.000	55.641	5.858	0.025	60.000
39.840	5.314	0.011	65.000	47.195	5.567	0.017	65.000	55.641	5.858	0.022	65.000
39.840	5.314	0.008	70.000	47.195	5.567	0.013	70.000	55.641	5.858	0.018	70.000
39.840	5.314	0.005	75.000	47.195	5.567	0.010	75.000	55.641	5.858	0.015	75.000

4.3. Possible Application of the Model

Signal coordination is a traffic management technique that synchronizes traffic lights at consecutive signalized intersections to ensure the smooth movement of vehicles. This is done by adjusting the start of the green phase at the downstream signal based on the time it takes for vehicles to travel from the upstream signal. This timing adjustment, known as the offset, is calculated using factors such as vehicle speed, travel time and distance between intersections. When properly implemented signal coordination creates a green wave, allowing vehicles to move through multiple intersections with minimal stopping, thereby reducing congestion and improving traffic flow.

For example, if it takes 15 seconds for vehicles to travel from one intersection to the next, the green phase at the downstream signal should begin exactly 15 seconds after the upstream signal turns green. This synchronization minimizes the number of stops, reduces overall delay and enhances efficiency by ensuring steady traffic movement. By reducing stop-and-go driving, signal coordination also decreases fuel consumption and emissions, making it a vital strategy for urban traffic management. However, achieving perfect synchronization requires careful planning and real-time adjustments to account for varying traffic conditions, such as pedestrian crossings and fluctuating vehicle volumes.

Chapter Five

5. Conclusion and Recommendation

5.1. Conclusion

The study aimed to investigate the factors influencing queue length at consecutive signalized intersections, focusing on the effects of pedestrian volume, green signal timing and proportion of buses. By employing the Simulation of Urban Mobility (SUMO) software, the research provided a detailed analysis of traffic dynamics at critical intersections in Addis Ababa. The findings revealed significant relationships between signal timing, traffic volume and vehicle composition, underscoring the complexity of managing urban intersections.

In this study, data from three consecutive signalized intersections (a total of six signalized intersections) was collected from real-world observations and simulated using the Simulation of Urban Mobility (SUMO) software. Using the simulation outputs, specifically queue lengths and the results of the Weibull distribution (alpha and beta parameters), the analysis was developed.

- The analysis of multiple linear regression coefficients reveals the impact of green time, pedestrian volume and the proportion of buses on queue length. A one unit increase in pedestrian volume and the proportion of buses leads to a rise in queue length by 0.0197 and 1.6989 for alpha and 0.0008 and 0.0584 for beta respectively. This suggests that higher pedestrian activity and a greater share of buses contribute to longer queues, likely due to frequent stops and pedestrian crossings disrupting traffic flow. Conversely, an increase in green time by one unit reduces queue length by 0.258 for alpha and 0.008 for beta, emphasizing its role in improving traffic movement by allowing more vehicles to pass through intersections efficiently.

- The results from the Multiple Linear Regression prediction model indicate that the independent variables pedestrian volume and the proportion of buses significantly contribute to the increase in queue length (represented by alpha and beta) at consecutive signalized intersections. These two factors were found to have a notable impact on traffic flow, as higher pedestrian volumes and bus frequencies tend to disrupt the smooth progression of vehicles, leading to longer queues. The model underscores the importance of addressing these variables when analyzing and optimizing traffic management strategies for consecutive signalized intersections, as their influence is both statistically significant and operationally critical.

- The Multiple Linear Regression prediction model reveals that the independent variable, green time has a significant impact on increasing queue length (represented by alpha and beta) at consecutive signalized intersections. While green time is typically expected to reduce queues by allowing more vehicles to pass, the findings suggest that improper allocation or insufficient synchronization of green time across intersections can lead to longer queues. This highlights the critical role of optimizing green time as part of traffic signal coordination to improve the overall efficiency of consecutive signalized intersections and reduce congestion.

The results demonstrated that proportion of buses and inadequate green signal timing substantially increase queue length. The absence of dedicated pedestrian crossing times was observed to increase delays, as pedestrians frequently disrupted vehicle flow. These findings emphasize the need for better pedestrian prioritization and green time allocation to ensure smoother traffic progression. Additionally, the study identified that intersections with higher pedestrian activity and higher number of buses experience longer delays.

5.2. Recommendation and gaps

To optimize the consecutive signalized intersection corridors in Addis Ababa and similar urban environments, it is recommended to prioritize infrastructure investments based on the significant impact of different factors that affect the consecutive signalized intersections. Continuous data collection and analysis of traffic patterns, coupled with the development of advanced prediction models, can enhance decision-making for traffic engineers.

From the research results, the pedestrian crosses at signalized intersections without dedicated time (cross through waiting vehicle gaps) for them which lead to vehicle movement disruption (preventing frequent stops for vehicles) and causes higher queue length and congestion or delays. Therefore providing dedicated pedestrian crossing times, constructing overpasses or underpasses uses pedestrians to cross without interrupting vehicle flow.

Using Intelligent Transportation Systems (ITS), such as cameras and sensors, traffic signals can dynamically adjust green times based on real-time conditions to optimize flow and reduce congestion. These systems monitor vehicle volumes, speeds, queue lengths and pedestrian activity at intersections, allowing adaptive traffic signal control to allocate longer green times to busier approaches while shortening them for less congested ones. This approach reduces delays, and improves fuel efficiency by minimizing stop-and-go traffic. Additionally, ITS can prioritize emergency vehicles or public transit and adjust to sudden traffic changes, ensuring smooth and flexible traffic management tailored to current conditions.

Banning older buses can significantly reduce start-up lost time at intersections, as these vehicles often experience delayed acceleration due to outdated engines and mechanical inefficiencies. Modern buses are equipped with advanced technology, better engines and improved powertrains that enable faster acceleration and smoother movement through intersections. By phasing out older buses, the overall time vehicles spend waiting behind slow-starting buses is minimized, reducing delays and improving intersection efficiency. This approach not only enhances traffic flow but also contributes to lower emissions and improved air quality, as modern buses are typically more environmentally friendly.

Future researchers should consider exploring additional factors that influence the performance of two consecutive signalized intersections beyond those addressed in this thesis. This could include variables such as intersection spacing, signal coordination strategies, pedestrian crossing patterns and the impact of non-motorized traffic. Additionally, future studies should examine multiple types of signalized intersections, including those with varying geometries, traffic volumes and control mechanisms, to gain a more comprehensive understanding of how different configurations affect traffic performance and flow dynamics. Subsequent scholars should build upon this study by exploring strategies for coordinating consecutive signalized intersections to improve traffic flow and reduce delays.

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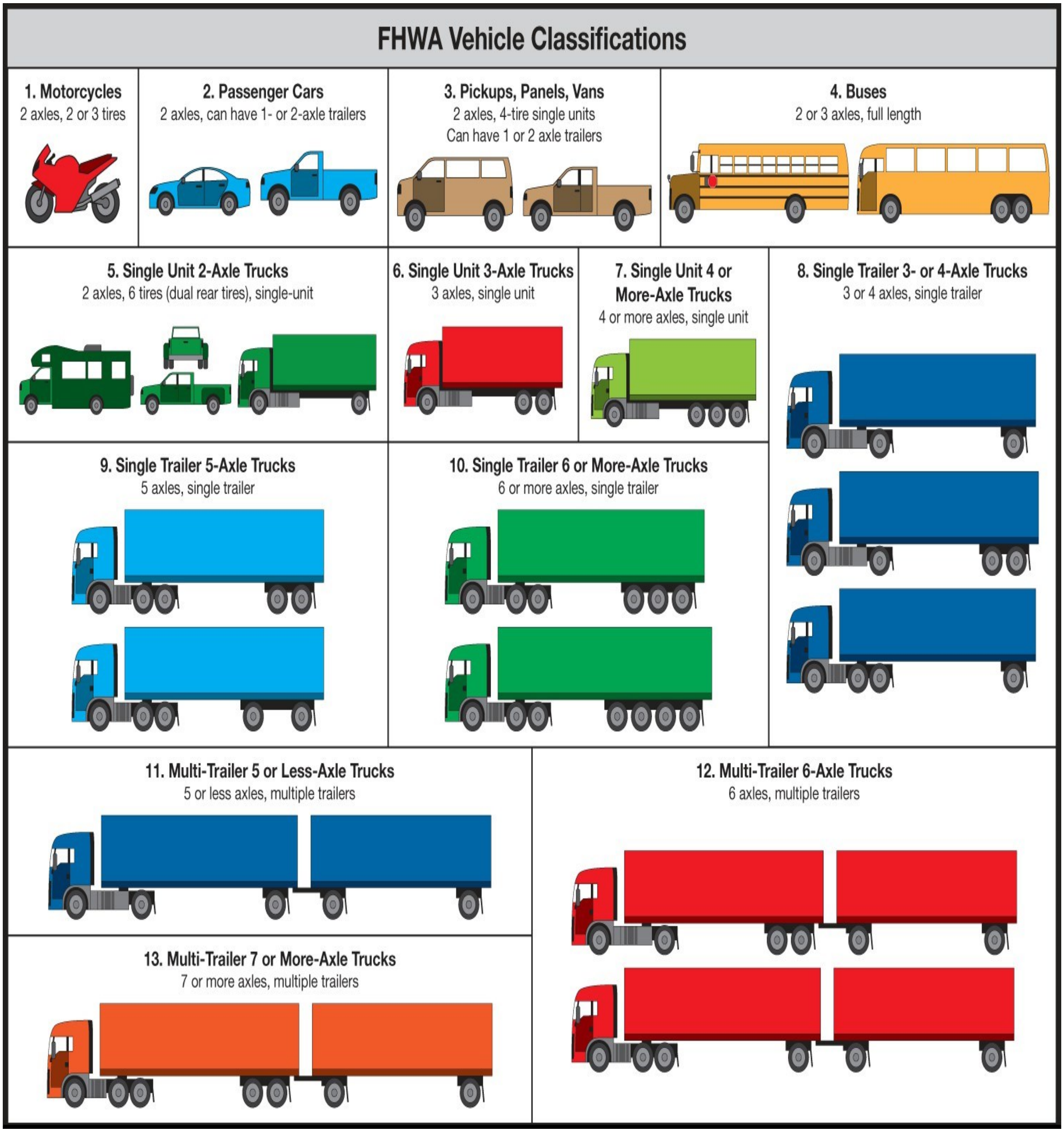
Appendices

Vehicle Classification According to FHWA



Traffic Volume (AADT) Counting Format

FHWA Vehicle Classification															
START	END	Motorcycle	Passenger Cars	Pickups	Buses	Single Unit 2-	Single Unit 3-	Single Unit 4	Single Trailer 3	Single Trailer 5	Single Trailer 6	Multi Trailer 5	Multi Trailer 6	Multi Trailer 7
AM	AM														
AM	AM														
PM	PM														
PM	PM														

Date: Location:

Speed Flow Calculations Format

	Time						
Classification	**Start Second**	**End Second**	**Duration Second**	**Hours**	**Meters**	**Distance Km**	**Speed Km/hr**

OSM network interface

```xml
<?xml version="1.0" encoding="UTF-8"?>

<!-- generated on 2024-11-15 20:14:28 by Eclipse SUMO netedit Version 1.19.0
<configuration xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/neteditConfiguration.xsd">

    <input>
        <sumo-net-file value="C:\Users\admin\Desktop\24\24.net.xml"/>
    </input>

    <output>
        <output-file value="C:\Users\admin\Desktop\24\24.net.xml"/>
    </output>

    <processing>
        <geometry.min-radius.fix.railways value="false"/>
        <geometry.max-grade.fix value="false"/>
        <offset.disable-normalization value="true"/>
        <lefthand value="0"/>
    </processing>

    <junctions>
        <no-internal-links value="false"/>
        <no-turnarounds value="true"/>
        <junctions.corner-detail value="5"/>
        <junctions.limit-turn-speed value="5.50"/>
        <rectangular-lane-cut value="0"/>
    </junctions>

    <pedestrian>
        <walkingareas value="0"/>
    </pedestrian>

</configuration>
-->

<net version="1.16" junctionCornerDetail="5" limitTurnSpeed="5.50" xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/net_file.xsd">

    <location netOffset="-476931.61,-995483.92" convBoundary="540.45,517.31,1055.55,889.39" origBoundary="38.790116,9.005657,38.803428,9.018857" projParameter="+proj=utm +zone=37 +ellps=WGS84

    <type id="highway.primary" priority="12" numLanes="2" speed="27.78" disallow="tram rail_urban rail rail_electric rail_fast ship" oneway="0"/>
```

Additional file interface

```xml
<?xml version="1.0" encoding="UTF-8"?>

<!-- generated on 2024-12-05 11:57:58 by Eclipse SUMO netedit Version 1.19.0
-->

<additional xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/additional_file.xsd">
    <!-- TAZs -->
    <taz id="1" shape="599.31,875.89 618.11,883.55 620.20,875.20 605.57,869.39 599.31,875.89" color="blue">
        <tazSource id="E4" weight="1.00"/>
        <tazSink id="E4" weight="1.00"/>
    </taz>
    <taz id="10" shape="752.77,669.64 743.60,669.89 748.18,676.55 755.64,675.39 752.77,669.64" color="blue">
        <tazSource id="200567869#1" weight="1.00"/>
        <tazSink id="200567869#1" weight="1.00"/>
    </taz>
    <taz id="11" shape="762.95,684.49 761.75,691.79 755.69,687.75 759.13,680.72 762.95,684.49" color="blue">
        <tazSource id="116757525#1" weight="1.00"/>
        <tazSink id="116757525#1" weight="1.00"/>
    </taz>
    <taz id="12" shape="582.68,701.72 585.56,697.69 588.82,700.19 586.14,703.64 582.68,701.72" color="blue">
        <tazSource id="-E3" weight="1.00"/>
        <tazSink id="-E3" weight="1.00"/>
    </taz>
    <taz id="13" shape="723.11,859.31 727.29,856.76 726.12,854.67 721.95,857.23 723.11,859.31" color="blue">
        <tazSource id="E2" weight="1.00"/>
        <tazSink id="E2" weight="1.00"/>
    </taz>
    <taz id="14" shape="686.93,550.05 691.46,547.96 693.03,551.10 688.50,553.37 686.93,550.05" color="blue">
        <tazSource id="E1" weight="1.00"/>
        <tazSink id="E1" weight="1.00"/>
    </taz>
    <taz id="15" shape="818.87,792.17 826.17,788.52 823.65,783.47 817.75,786.56 818.87,792.17" color="blue">
        <tazSource id="-E0" weight="1.00"/>
        <tazSink id="-E0" weight="1.00"/>
    </taz>
    <taz id="2" shape="546.82,694.09 560.08,673.70 568.23,681.18 555.66,701.90 546.82,694.09" color="blue">
        <tazSource id="-E3" weight="1.00"/>
        <tazSink id="-E3" weight="1.00"/>
        <tazSource id="E3" weight="1.00"/>
        <tazSink id="E3" weight="1.00"/>
    </taz>
    <taz id="3" shape="719.79,886.43 742.22,871.48 734.75,861.62 712.66,875.56 719.79,886.43" color="blue">
```

Origin Destination metrics interface

```
$O;D2
* From-Time To-Time
0.00 1.00
* Factor
1.00
*
* some
* additional
* comments
            1     2     84
            1     3     72
            1     6     252
            1     4     64
            1     5     40
            2     5     4
            3     6     56
            2     6     52
```

OD configuration file interface

```
<configuration>
<input>
<taz-files value="bus.add.xml"/>
<od-matrix-files value="bus.od.xml"/>
</input>
</configuration>
```

Trip file interface

```
<?xml version="1.0" encoding="UTF-8"?>

<!-- generated on 2024-12-05 12:17:05 by Eclipse SUMO od2trips Version 1.19.0
<configuration xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/od2tripsConfiguration.xsd">

    <input>
        <taz-files value="bus.add.xml"/>
        <od-matrix-files value="bus.od.xml"/>
    </input>

    <output>
        <output-file value="bus.odtrips.xml"/>
    </output>

    <processing>
        <vtype value="bus"/>
        <prefix value="bus"/>
    </processing>

</configuration>
-->

<routes xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/routes_file.xsd">
    <trip id="bus5" depart="55.65" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
    <trip id="bus7" depart="381.92" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
    <trip id="bus1" depart="476.94" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
    <trip id="bus3" depart="754.69" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
    <trip id="bus6" depart="1817.33" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
    <trip id="bus2" depart="1847.70" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
    <trip id="bus0" depart="2488.08" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
    <trip id="bus4" depart="2950.16" from="E4" to="200567869#2" type="bus" fromTaz="1" toTaz="6" departLane="free" departSpeed="max"/>
</routes>
```

Vehicle Type interface

```xml
<additional>
  <routes>
    <vType id="bus" accel="1.0" decel="3.8" sigma="0.5" length="12" minGap="2.3" maxSpeed="14" color="blue" emissionClass="HBEFA3/bus"/>
  </routes>
</additional>
```

Route File interface

```xml
<configuration>
  <input>
    <net-file value="bus.net.xml"/>
    <additional-files value="vtype_bus.add.xml"/>
    <route-files value="bus.odtrips.xml"/>
  </input>
  <output>
    <output-file value="bus.odtrips.rou.xml"/>
  </output>
  <report>
    <xml-validation value="never"/>
    <no-step-log value="true"/>
  </report>
</configuration>
```

```xml
<?xml version="1.0" encoding="UTF-8"?>

<!-- generated on 2024-12-05 12:18:04 by Eclipse SUMO duarouter Version 1.19.0
<configuration xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/duarouterConfiguration.xsd">

    <input>
        <net-file value="bus.net.xml"/>
        <additional-files value="vtype_bus.add.xml"/>
        <route-files value="bus.odtrips.xml"/>
    </input>

    <output>
        <output-file value="bus.odtrips.rou.xml"/>
    </output>

    <report>
        <xml-validation value="never"/>
        <ignore-errors value="true"/>
        <no-step-log value="true"/>
    </report>

</configuration>
-->

<routes xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/routes_file.xsd">
    <vType id="bus" length="12.00" minGap="2.30" maxSpeed="14.00" emissionClass="HBEFA3/Bus" color="blue" accel="1.0" decel="3.8" sigma="0.5"/>
    <vehicle id="bus5" type="bus" depart="55.65" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <routeDistribution last="0">
            <route cost="29.42" probability="1.00000000" edges="E4 200567869#1 200567869#2"/>
        </routeDistribution>
    </vehicle>
    <vehicle id="bus7" type="bus" depart="381.92" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <routeDistribution last="0">
            <route cost="29.42" probability="1.00000000" edges="E4 200567869#1 200567869#2"/>
        </routeDistribution>
    </vehicle>
    <vehicle id="bus1" type="bus" depart="476.94" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <routeDistribution last="0">
            <route cost="29.42" probability="1.00000000" edges="E4 200567869#1 200567869#2"/>
        </routeDistribution>
    </vehicle>
    <vehicle id="bus3" type="bus" depart="754.69" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <routeDistribution last="0">
            <route cost="29.42" probability="1.00000000" edges="E4 200567869#1 200567869#2"/>
        </routeDistribution>
```

```xml
<?xml version="1.0" encoding="UTF-8"?>

<!-- generated on 2024-12-05 12:18:04 by Eclipse SUMO duarouter Version 1.19.0
<configuration xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/duarouterConfiguration.xsd">

    <input>
        <net-file value="bus.net.xml"/>
        <additional-files value="vtype_bus.add.xml"/>
        <route-files value="bus.odtrips.xml"/>
    </input>

    <output>
        <output-file value="bus.odtrips.rou.xml"/>
    </output>

    <report>
        <xml-validation value="never"/>
        <ignore-errors value="true"/>
        <no-step-log value="true"/>
    </report>

</configuration>
-->

<routes xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/routes_file.xsd">
    <vType id="bus" length="12.00" minGap="2.30" maxSpeed="14.00" emissionClass="HBEFA3/Bus" color="blue" accel="1.0" decel="3.8" sigma="0.5"/>
    <vehicle id="bus5" type="bus" depart="55.65" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <route edges="E4 200567869#1 200567869#2"/>
    </vehicle>
    <vehicle id="bus7" type="bus" depart="381.92" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <route edges="E4 200567869#1 200567869#2"/>
    </vehicle>
    <vehicle id="bus1" type="bus" depart="476.94" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <route edges="E4 200567869#1 200567869#2"/>
    </vehicle>
    <vehicle id="bus3" type="bus" depart="754.69" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <route edges="E4 200567869#1 200567869#2"/>
    </vehicle>
    <vehicle id="bus6" type="bus" depart="1817.33" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <route edges="E4 200567869#1 200567869#2"/>
    </vehicle>
    <vehicle id="bus2" type="bus" depart="1847.70" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <route edges="E4 200567869#1 200567869#2"/>
    </vehicle>
    <vehicle id="bus0" type="bus" depart="2488.08" departLane="free" departSpeed="max" fromTaz="1" toTaz="6">
        <route edges="E4 200567869#1 200567869#2"/>
```

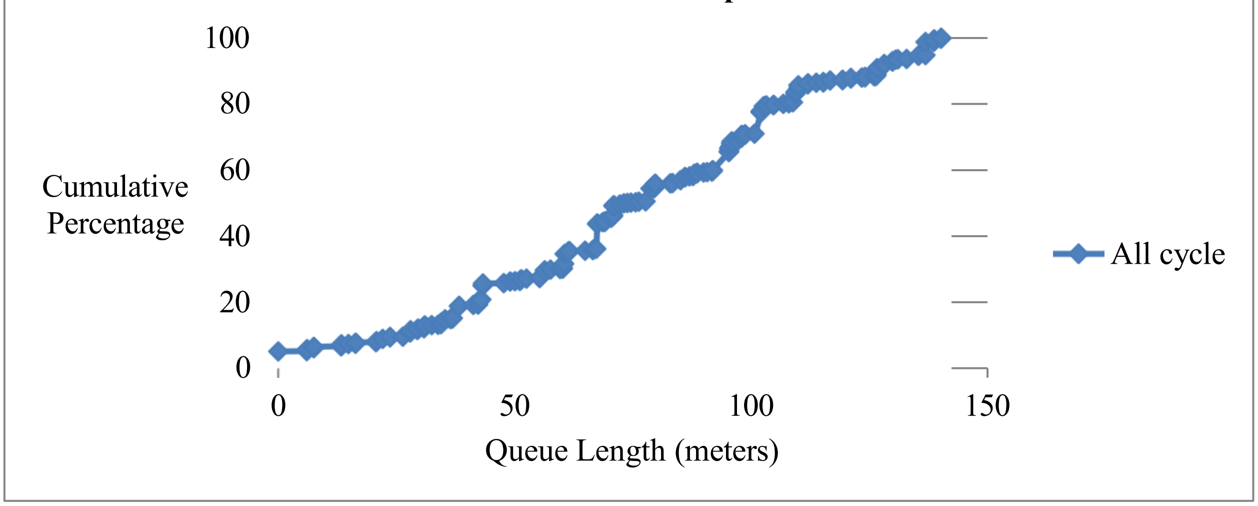
SUMO output (queue length) interface

```xml
<!-- generated on 2024-12-05 14:25:57 by Eclipse SUMO sumo Version 1.19.0
<configuration xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/sumoConfiguration.xsd">

    <input>
        <net-file value="all.net.xml"/>
        <route-files value="car.odtrips.rou.xml, bus.odtrips.rou.xml, hv.odtrips.rou.xml, motor.odtrips.rou.xml, pm.odtrips.rou.xml, pv.odtrips.rou.xml, st.odtrips.rou.xml, trailer.odtrips.rou.xml"/>
        <additional-files value="n.xml"/>
    </input>

    <output>
        <queue-output value="nee"/>
    </output>

    <time>
        <begin value="0"/>
        <end value="1000"/>
    </time>

</configuration>
-->

<queue-export xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance" xsi:noNamespaceSchemaLocation="http://sumo.dlr.de/xsd/queue_file.xsd">
```

```xml
        <lanes>
            <lane id="-E2_2" queueing_time="69.00" queueing_length="17.80" queueing_length_experimental="17.80"/>
            <lane id="-E3_1" queueing_time="94.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="200567869#1_1" queueing_time="62.00" queueing_length="77.71" queueing_length_experimental="77.71"/>
            <lane id="200567869#1_2" queueing_time="63.00" queueing_length="88.21" queueing_length_experimental="88.21"/>
            <lane id="200567869#1_3" queueing_time="60.00" queueing_length="79.86" queueing_length_experimental="79.86"/>
            <lane id="E4_1" queueing_time="6.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="E4_2" queueing_time="2.00" queueing_length="7.50" queueing_length_experimental="7.50"/>
            <lane id="E4_3" queueing_time="50.00" queueing_length="13.30" queueing_length_experimental="13.30"/>
        </lanes>
    </data>
    <data timestep="864.00">
        <lanes>
            <lane id="-E2_2" queueing_time="70.00" queueing_length="17.80" queueing_length_experimental="17.80"/>
            <lane id="-E3_1" queueing_time="95.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="200567869#1_1" queueing_time="63.00" queueing_length="77.71" queueing_length_experimental="77.71"/>
            <lane id="200567869#1_2" queueing_time="64.00" queueing_length="88.21" queueing_length_experimental="88.21"/>
            <lane id="200567869#1_3" queueing_time="61.00" queueing_length="79.86" queueing_length_experimental="79.86"/>
            <lane id="E4_1" queueing_time="7.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="E4_2" queueing_time="3.00" queueing_length="7.50" queueing_length_experimental="7.50"/>
            <lane id="E4_3" queueing_time="51.00" queueing_length="13.30" queueing_length_experimental="13.30"/>
        </lanes>
    </data>
    <data timestep="865.00">
        <lanes>
            <lane id="-E2_2" queueing_time="71.00" queueing_length="17.80" queueing_length_experimental="17.80"/>
            <lane id="-E3_1" queueing_time="96.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="200567869#1_1" queueing_time="64.00" queueing_length="77.71" queueing_length_experimental="77.71"/>
            <lane id="200567869#1_2" queueing_time="65.00" queueing_length="88.21" queueing_length_experimental="88.21"/>
            <lane id="200567869#1_3" queueing_time="62.00" queueing_length="79.86" queueing_length_experimental="79.86"/>
            <lane id="E4_1" queueing_time="8.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="E4_2" queueing_time="4.00" queueing_length="7.50" queueing_length_experimental="7.50"/>
            <lane id="E4_3" queueing_time="52.00" queueing_length="13.30" queueing_length_experimental="13.30"/>
        </lanes>
    </data>
    <data timestep="866.00">
        <lanes>
            <lane id="-E2_2" queueing_time="72.00" queueing_length="17.80" queueing_length_experimental="17.80"/>
            <lane id="-E3_1" queueing_time="97.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="200567869#1_1" queueing_time="65.00" queueing_length="88.21" queueing_length_experimental="88.21"/>
            <lane id="200567869#1_2" queueing_time="66.00" queueing_length="80.91" queueing_length_experimental="80.91"/>
            <lane id="200567869#1_3" queueing_time="63.00" queueing_length="79.86" queueing_length_experimental="79.86"/>
            <lane id="E4_1" queueing_time="9.00" queueing_length="6.00" queueing_length_experimental="6.00"/>
            <lane id="E4_2" queueing_time="5.00" queueing_length="7.50" queueing_length_experimental="7.50"/>
            <lane id="E4_3" queueing_time="53.00" queueing_length="13.30" queueing_length_experimental="13.30"/>
```

Sample of some output of simulation result distribution graph

Distribution Graph



Distribution Graph