



ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF GRADUATE STUDIES
SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING

**Evaluating the Performance of Signalized Intersection
and the Associated Economic Impact of Congestion:
(A Case Study on Ras Mekonnen Street of Addis
Ababa, Ethiopia)**

By:

Tsegaye Gashaw Abate

**A Thesis Submitted to the School of Graduate Studies of Addis Ababa
University in Partial Fulfillment of the Requirements for the Degree of Master
of Science in Civil Engineering (Road and Transport Engineering)**

Advisor:

Dr. Bikila Teklu Wodajo

June, 2018

Addis Ababa, Ethiopia

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UNDERTAKING

I, the undersigned, certify that this research work titled “Evaluating the Performance of Signalized Intersection and the Associated Economic Impact of Congestion: (A Case Study on Ras Mekonnen Street of Addis Ababa, Ethiopia)” is my original work performed under the supervision of my research advisor Dr. Bikila Teklu and has not been presented elsewhere for assessment and for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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ABSTRACT

Now days, traffic congestion is becoming a serious issue in developing countries in general and in Addis Ababa in particular. Its major impact on the day-to-day activities of road users is not limited to delay in travel time but also it affects the consumption of fuel and the surrounding environment in terms of greenhouse gas emissions.

In Addis Ababa, the issue is becoming a great concern to its residents, transport managers and planners. In light of the interest in testing the performance and optimization of Signalized Intersection timing plan on major route corridor of Addis Ababa, this study was designed and conducted on Ras Mekonnen Street between the Addis Ababa School of Commerce and Estifanos, as it is the most signalized route corridor in the city and serves the city's major traffic population.

Based on Highway Capacity Manual (HCM), the current performances of signal operation on the route corridor were assessed along with the cost of congestion results from Travel Time Lost, Extra Fuel Consumption and Environmental Pollution costs. In addition, using Synchro 9 Simulation Software, the Intersection signal timing plan was optimized with respect to network cycle length, offset and splits.

The result of performance evaluation indicated that all the Signalized Intersections under this study serve the traffic demand with poor level of service (LOS F) resulting in costs to the Country amounted to **ETB:1,191,442.76 per hour** due to extra fuel consumption and travel time loss and also affected by pollutions with emission rate of 31,668.64 g/h of NO_x; 206,796.22 g/h of HC; 2,874,467.45 g/h of CO; 3,736,807.68 g/h of PM_{2.5} and 134,375,604.21 g/h of CO₂ attributed by the on roads traffic on the study segment.

The simulation analysis for optimization shows that service quality at intersections can be improved by changing the current operating cycle length of 188sec to 270sec for Estifanos Intersection and 230sec for all other Intersections in the network, together with phase sequences and splits. As a result, more than 550 vehicles will be relieved from stopping at intersections and also extra 654 liters of fuel consumed by the vehicles per hour will also be saved owing to improvement of signal timing.

KEY WORDS: Level of Service, HCM Manual, Synchro Software, Congestion, Optimization, Signalized Intersection

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LIST OF ABBREVIATIONS

AACRA:	Addis Ababa City Roads Authority
AAiT	Addis Ababa Institute of Technology
CBD:	Central Business District
CO:	Carbon Mono Oxide
ECT:	Emission Control Technologies
EL:	Exclusive Left Turn Lane
ER:	Exclusive Right Turn Lane
ETB:	Ethiopian Birr
FHWA:	Federal Highway Administration of the United States
GHG	Green House Gases
HCM:	High Way Capacity Manual
IPCC	Intergovernmental Panel on Climate Change
Kg/h:	Kilogram per Hour
Km:	Kilometer
LOS:	Level of Service
LT:	Left Turn
LTOR:	Left Turn on Red
LTV:	Left Turn Volume
m:	Meter
Nox:	Nitrogen Oxide
pc/h/ln:	Passenger Car per Hour per Lane
PCU:	Passenger Car Unit
PHF	Peak Hour Factor
PLT:	Proportion of Left Turn Volume
PRT:	Proportion of Right Turn Volume
RT:	Right Turn
RTOR	Right Turn on Red
RTV:	Right Turn Volume
sec:	Second
SL:	Shared Left Turn Lane
SR:	Shared Right Turn Lane
U.S:	United States
V/C:	Volume to capacity Ratio
Veh/h:	Vehicle per Hour
VOC:	Vehicle Operating Cost
WHO	World Health Organization

CHAPTER ONE

INTRODUCTION

1.1 Research background

Traffic congestion is a widely recognized transport cost which affects transport system performance and transport planning. When the road reaches its capacity, each additional vehicle imposes more total delay on others than they bear. Congestion tends to increase travel time, fuel consumption, pollution emission and also reduce life satisfaction (Bhosale, 2016).

Retiming traffic signal is one of the most cost effective tasks that an agency can do to improve traffic flow. Signal optimization requires current traffic flow data and knowledge of field observations. In the real world signal timing process begins with trigger event like field adjustments and system retiming (Henry, 2005).

The rapid increase of road transportation requirements has brought significant challenges to the wellbeing and development of societies of developing countries. Accordingly, most of the urban residents of these countries in general and in Addis Ababa in particular suffer from poor transport facilities resulting in high cost of productivity losses from traffic delays, traffic accidents, traffic jams, pollutions and noise. In the meantime, the considerable financial losses caused by the traffic congestion become a large and growing burden on the nation's economy (Mekonnen, 2015).

Because of the rapid population and economic growth in the cities, traffic flow increment is an important issue. Accordingly, improving the transport facilities to reduce delay and travel time is considered as an effective countermeasure against traffic congestion (ChunhuiYu, 2017).

Addis Ababa, the capital city of Ethiopia serves more than half of the country's vehicular population without well-organized transport facilities. In connection to this, it is common to see congestions, long queues and associated delays in almost all of Addis Ababa's junctions, especially at signalized intersections. The Ras Mekonnen Street of Addis Ababa (between the Addis Ababa School of Commerce and Meskel Square) is one of the route corridor mainly affected by the jammed traffic flows. The traffic at the junctions of the route are served by four

signal controlled intersections that makes the route corridor to be one of the most signalized section of route in the city.

This paper examines the improper traffic signal timing process and defines methods that can be used to minimize the cost of delays. In addition, optimizations of the signal timing were conducted to improve the intersection capacity. This effort primarily emphasizes on efficiently updating signal timing on the selected route corridor with lowest possible costs.

1.2 Problem statement

Currently, the increase in traffic volume of roadways has led transport expertise to explore different means of control mechanisms. Among the different control mechanisms, traffic signals are the most preferable in urban areas where there is right of way limitation and highly accident prone roadway junctions. However, the inappropriate traffic forecasting and/or installation of traffic signals can actually result in an extended long queue, traffic crash, congestion and delays on the road ways. In this connection, the day to day activities of the urban residents has been negatively affected and this effect has also direct impact on the nation's economy.

In the city of Addis Ababa, the effect of traffic congestion with regard to limited mobility of road uses, fuel consumption and productivity loses due to travel time delay are significant especially, at the city's major intersection. Consequently, the sustainable development and satisfaction of road users of the city's population has been significantly affected.

This being the case, to utilize effective and efficient flow of traffic and to overcome the associated economic impact, there is a need to assess the possible causes of traffic jam, updating the traffic signal timing plan.

1.3 Objectives of the Study

1.3.1 General Objective

The general objective of the study was to examine the service quality of junctions' especially signalized intersections and evaluate the impact of congestion in the city of Addis Ababa. For the accomplishment of stated objectives, all necessary data for signal evaluation, for congestion cost estimation were identified.

1.3.2 Specific Objectives

The specific objectives of this study were to evaluate the performance of signals, optimizing timing plans and its economic impact in the city of Addis Ababa; by focusing on a case study on Ras Mekonnen Street (between the Addis Ababa School of Commerce and Meskel Square), in relation to constraint analysis of congestion, fuel consumption, level of service and outline a framework on the performance of the route. Particularly, the study has the following specific-objectives:

- To evaluate the performance of each isolated signalized intersection on Ras Mekonnen street;
- To determine the cost of congestion due to jammed traffic; and
- To optimize the road network (i.e. Ras Mekonnen street) signal timing.

1.4 Scope of research

The scope of the study covers performance analysis of the Ras Mekonnen Street of Addis Ababa, considering all physical and timing parameters, mainly focused on the existing travel flow patterns. An evaluation framework was presented for performance analysis based on HCM, 2010 methodologies. Performance analysis was manipulated using Microsoft excel. Synchronization of traffic signals has been done using SYNCHRO version-9 micro simulation software which is the most preferable and recommended software by FHWA of U.S Department of Transportation. Further, Costs relating to traffic congestions were formulated from field data and software output.

Nevertheless, the existing traffic flow pattern, driver behavior in relation to perception reaction time and red signal time enforcement, economic development of area and land use pattern can vary from corridor to corridor in the city and it would be difficult to transfer to other routes.

The collection of data was a challenging task as the research area involves mixed traffic population. In the end, conclusions were presented and recommendations were formulated based on the major findings of the research.

1.5 Thesis Contribution

This research can be significantly used in many ways. First, it focuses on least cost intersection performance enhancement, so that the traffic management agencies can use the output for synchronizing the segment.

1.6 Thesis organization

The report consists of five chapters. Following this chapter of Introduction, chapter two deals with Literature Review revising the works that have been done in the area of traffic signal coordination with respect to selection of best simulation model, signal timing plan evaluation and performance parameters considered during analysis, design and road way congestion impacts and costs. Chapter three focuses on research approach and plan to assess the performance evaluation. The framework includes the data collection methods together with selection of appropriate simulation model.

In addition, Chapter four focuses on results and discussions with respect to the Signal Timing performance evaluations, Signal Timing Optimizations and congestion cost estimations. The last chapter, Chapter five summarizes the major conclusions of the research and identifies the areas of future research. At the end future research areas, as found relevant have been proposed.

CHAPTER TWO

LITERATURE REVIEW

2.1 Signalized Intersection

Intersections in urban roadways form bottlenecks. Accordingly, to accommodate the traffic and to eliminate the queue formed on that limited space, different junction types are considered to be provided in accordance with warrants in the Manual on Uniform Traffic Control Devices (P.Roess, 2004).

From those junctions control forms signalized intersection is the ultimate control method for at grade intersection. Signals can increase the traffic capacity of the intersection, reduce frequency and severity of crashes, particularly right-angle crashes, and interrupt heavy traffic flow to permit other motor vehicles, pedestrians and bicycles to cross the street. Signalized intersection is an operational strategy which facilitates the movement of traffic on the intersections by reducing crossing accidents and giving pedestrian safe feeling.

Unwarranted or poorly timed traffic signals can have negative impacts, including excessive delay to vehicular and pedestrian traffic, disrespect for traffic control devices in general, increased “cut through” traffic on inappropriate routes, and increased frequency of crashes (MassDOT, 2012).

At signalized intersection, signal time is mounted to reduce, eliminate and segregate the potentially conflicting movements. The signal design incorporates cycle and phase parameters. The cycle is a time required for one complete sequence of signal indicators and a phase is a portion of cycle allocated to any combination of traffic movement (Tarekegn Kumala, 2016).

The two most important features considered during evaluation of signalized intersection are capacity and delay. Capacity is the maximum rate of flow where the vehicle is expected to traverse a road way under prevailing traffic, control and road way condition whereas; delay is the additional time experienced by the driver beyond the time required to traverse a road way (Bhosale, 2016).

Delay and level of service are an indication of the potential capacity and performance measure of an intersection and it is found that the main contributory factors for moderate traffic congestion

in the same area is the allocation of long cycle time and inappropriate phase plan at the intersections (Tarekegn Kumala, 2016).

In order to conduct an operational analysis of signalized intersection geometric, traffic and signalization data are required. The geometric conditions includes area type, number of lanes, average lane width, grade, existence of exclusive left turn or right turn lanes, length of stoppage bay left turn or right turn lanes and parking. Traffic conditions also includes demand volume by movement, Base saturation flow rate, Peak-hour factor, Percent heavy vehicles, Approach pedestrian flow rate, Local buses stopping at intersection, Parking activity, Arrival type, Proportion of vehicles arriving on green and Approach speed. In addition, the input parameters of Signalization conditions includes Cycle length, Green time, Yellow-plus-all-red change-and-clearance interval, Actuated or pre-timed operation, Pedestrian push-button, Minimum pedestrian green, Phase plan and Analysis period (HCM, 2010).

2.1.1 Level of service

The Minnesota Department of Transportation (MN Dot) of the United States (US) in 2013, defined the level of service of signalized intersection in terms of delay; which is a measure of driver discomfort, frustration, fuel consumption and lost travel time. The delay is the difference between actual travel time and ideal travel time (no traffic control, no geometric delay, no incident and no vehicle in the road).

Level of Service (LOS) is somewhat a subjective indication of the general acceptability of delay to drivers and directly related with average control delay and volume to capacity ratios. When delay levels are unacceptable but v/c ratios are relatively low, the cycle length and/ or the phase plan may be inefficient (Koonce, 2008).

It should be noted, however, that when signals are part of a coordinated system, the cycle length at individual intersections is determined by system considerations, and alterations at isolated locations may not be practical. When both delay levels and v/c ratios are unacceptable, the situation is critical. Delay is already high, and demand is near or over capacity. In such situations, the delay may increase rapidly with small changes in demand. The full range of potential geometric and signal design improvements should be considered in the search for

improvements (HCM, 2010).

LOS is average control delay per vehicle in a lane group. The average control delay of the intersection is estimated by aggregating average delay weighted by adjusted flow of intersection. The adjusted flow of intersection (v_p) is the maximum hourly flow rate based on the full hourly volume.

$$V_p = \frac{V}{PHF} \quad \text{Equation 2.1}$$

Where,

V_p = Adjusted flow of intersections;

V = traffic volume; and

PHF = Peak hour factor

Table 2-1: Level of Service based on HCM 2010 manual

I.No.	Control Delay per vehicles (s/Veh)	LOS
1.	<10	A
2.	>10-20	B
3.	>20-35	C
4.	>35-55	D
5.	>55-80	E
6.	>80	F

2.1.2 Delay

Traffic delays and queues are principal performance measures of intersection LOS, in the evaluation of the adequacy of lanes, and in the estimation of fuel consumption and emissions. The delay measures include the Control Delay, the Queue Delay and the Total Delay. The Control Delay is uniform delay plus the impacts of coordination and incremental delays. Queue Delays are the additional delay caused by reduction in capacity due to spillback and starvation. Control delay mainly affected by cycle time, ideal saturation flow, inters green time, number of phasing, number of lane, and left turn on red (LTOR) (Abojaradeh, 2014).

Total delay includes delay introduced by the presence of the signal (control delay) and other delay due to various roadway and traffic characteristics. Control delay includes deceleration as the vehicle approaches a queue or a red signal indication, acceleration away from the signal to get back to free flow speed, and the time spent stopped at a red indication (Avdiu, 2013).

Control delay of signalized intersection is the weighted average of delay of each approach where the approach delay is also the weighted average of delay of each lane group. In addition, the average control delay per vehicle is determined by HCM Methodology for Signalized Intersection (HCM, 2010).

$$d=d1(PF)+d2+d3 \quad \text{Equation 2.2}$$

Where, d = control delay per vehicle;

d1= uniform control delay assuming uniform arrivals (s/veh);

$$d1 = \frac{0.5c(1-\frac{g}{c})^2}{1-[\min(1,x)\frac{g}{c}]} \quad \text{Equation 2.3}$$

c = cycle length (s); cycle length used in pre-timed signal control, or average cycle length for actuated control

g = effective green time for lane group (s)

x = v/c ratio or degree of saturation for lane group

PF = uniform delay progression adjustment factor

$$PF = \frac{f_{PA}(1-P)}{1-(\frac{g}{c})} \quad \text{Equation 2.4}$$

P = proportion of vehicles arriving on green

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

Where, d2= incremental delay

$$d_2 = 900T[(x - 1) + \sqrt{(x - 1)^2 + \frac{8kIX}{cT}}] \quad \text{Equation 2.5}$$

T = duration of analysis period (h)

k = incremental delay factor that is dependent on controller settings

I = upstream filtering/metering adjustment factor

C = lane group capacity (veh/h)

x = lane group v/c ratio or degree of saturation

Where, d_3 = initial queue delay

$$d_3 = \frac{1800Qb(1+u)t}{cT} \quad \text{Equation 2.6}$$

Q_b = initial queue at the start of period T (veh)

c = adjustment lane group capacity (veh/h)

T = duration of analysis period (h)

t = duration of unmet demand in T (h)

u = delay parameter

The study also used the above Highway Capacity Manual (HCM) methodology in order to evaluate the signal timing.

2.2 Essentials of signal coordination

Providing effective real time traffic signal control for a large complex network is an extremely challenging task. Hence, in order to get efficient movement of traffic on streets and highways, optimization of traffic signals is required (Shamshirband, 2008). Using integration microscopic traffic simulation and assignment model the traffic signal re-timing by coordinating isolated signals is essential in order to reduce the variance of the speed, total delay, stopped delay,

acceleration and deceleration delay and average crash risk (H. Rakha, 2016).

2.3 Overview of signal Coordination Design

Even if traffic signals are one of the most effective and flexible active traffic control mechanisms that require simple geometric design, it causes large stopped delays, and complexity in the design and implementation. The three parameters that determine the coordination of signalized network are cycle length, offset, and split. Optimization of the network signal timing plan (cycle, splits, and offsets) can be handled by Synchro software. However, when the computer model is not available, the following procedures are suggested by FHWA (Koonce, 2008). Synchro 9 software is also preferred for this study.

- a) Calculate cycle length for each primary intersection using Webster equation;

$$\text{Optimum Cycle Length} = (1.5 * L + 5) / (1.0 - Y) \quad \text{Equation 2.7}$$

- b) determine the minimum pedestrian cycle length: Pedestrian Minimum Cycle = $LT + 14 + Wm/ SP + Wc/ SP + Y$
- c) Using the average distance between intersections, calculate the potential resonant cycles; Cycle = $2 * \text{Distance} / \text{Speed}$; Cycle = $4 * \text{Distance} / \text{Speed}$; Cycle = $26 * \text{Distance} / \text{Speed}$; Cycle = $8 * \text{Distance} / \text{Speed}$ and then select the shortest resonant cycle. The cycle time should be max (optimum Cycle Length, Pedestrian Minimum Cycle and shortest resonant cycle)
- d) Calculate the offset using Kell method
- e) Split is calculated for the primary intersection using the Critical Movement Method.

Phase difference plan is a better tool in order to coordinate traffic signals for closely spaced intersection having 1000m center to center distance by adopting uniform cycle length. On this study, two different phase plans are designed depends on average speed of vehicle, distance, geometric properties of link and assigned to all consecutive intersections (Bhosale, 2016).

2.4 Traffic Congestion

Traffic congestion becomes a serious problem at road intersections. The costs due to unproductive time loss, extra fuel consumption, accidents, environmental pollution are the major problems accounted for in relation to traffic congestion. On rush hours, the intersections cause long queue which imposes economic impacts on the urban dwellers in many aspects (Agajie, 2015). Mekonnen, 2012, mentioned traffic congestion is the most annoying feature which substantially consumes travel time, creates difficulty for scheduling and on time delivery and occurs at times of high travel demand.

Sugiyanto, 2010, on the study on estimation of Congestion Cost of Private Passenger Car Users in Malioboro, Yogyakarta, has developed a model for estimating congestion cost considering that congestion cost is a generalized cost (i.e. vehicle operating cost, travel time cost and pollution cost) in actual condition (8 km/h) less in perceived condition (30 km/h). However, the study is limited to Private Passenger Car.

Side effects of traffic congestion give rise to various resource costs that can be expressed in monetary terms: time costs of delays, health costs caused by air pollution, productivity losses due to lives lost in traffic accidents, abatement costs due to climate impacts of transport, etc. In this study, the two major delay related costs; cost of travel time and Cost of Extra Fuel Consumption will be estimated (Institute, 2017).

Mahmud, 2012, used different approach to estimate the congestion cost. He gave estimates of congestion cost for different vehicle and carriage way type considering fuel and travel time cost as a major congestion related cost.

2.5 Congestion Cost

2.5.1 Cost of Travel Time Lost

The value of travel time is a critical factor in evaluating the benefit of transportation infrastructure investment and saving of it mainly depend on the condition of the trip (US Department of Transportation, 2011).

Travel time savings are major benefits arising from transport infrastructure and its estimation in a

monetary value is important in order to forecast level of traffic which is attracted and to include so value of savings in cost benefit analysis (Bank, 2016).

The value of time lost is an opportunity cost caused by waiting at congested road intersection. The lost time may be either working time or nonworking time. The lost time is determined from the hourly income of individuals, average vehicle occupancy, the number of vehicles delayed and average waiting time as developed below (Agajie, 2015).

$$C_T = L_d * W_d * t_a * V_p \quad \text{Equation 2.8}$$

Where; C_T = Cost of time lost per hour, ETB;

L_d = Difference in number of vehicles waiting in the system;

W_d = Difference in average waiting time per vehicle;

t_a = Average value of time, sec; and

V_p = Vehicle occupancy, pass/veh.

2.5.2 Cost of Extra Fuel Consumption Due to Waiting at Signalized Intersections

Most drivers are less aware of fuel their vehicles consume while traversing a short segment of road but it is important to measure the effectiveness of signalized intersection. The impact of extra fuel consumption on the city economy is significant if it is estimated over multiple vehicles and forecasted to a given service time. The amount of fuel consumption is estimated by the PC-travel software based on the velocity and acceleration of vehicle (Meyer, 2010).

Abajie (2015), stated that the cost of fuel lost is rapidly increasing if vehicles are waiting longer time. According to him, the following are considered to estimate the cost of extra fuel consumed by the vehicles: determination of differences in waiting time, number of vehicles waiting, average fuel consumption of single vehicle and the unit cost of fuel are required. In addition, he illustrated in his research that about ETB: 560,000 is wasted due to consumption of extra fuel while vehicles are delayed at junctions for about 6minutes. In order to determine this cost he

used the following mathematical equation (Abajie, 2015).

$$\text{ECEFC} = \text{Cu} * \text{Fc} * \text{wd} * \text{Ld} \quad \text{Equation 2.9}$$

Where;

ECEFC= Expected Cost of Extra Fuel Consumed per hour;

Cu = Unit cost of fuel, ETB;

FC= Extra fuel consumed for one vehicle per hour;

Wd = Difference in average waiting time at the road Junction; and

Ld = The difference in number of vehicles waiting at the road junction.

2.5.3 Emission from Inroad Vehicles

In Ethiopia, there are no emissions standard that vehicles imported into Ethiopia must meet. This is true for both new and used vehicles. For this reason, even newer vehicles emit severe exhaust smoke pollution because they enter the country without the modern Emissions Control Technology (ECT) installed. In line with the vehicle usage in Addis Ababa, many vehicles are not maintained and are actually overloaded, thus these conditions worsen vehicle exhaust smoke emissions (Shankute, 2015).

The average age of Ethiopia's vehicle fleet is 15-20 years. Vehicles are considered to be second-hand ten years after their production date, compared to the global norm of four years after production. The rate of growth of used vehicles registration is greater than that of brand new ones. Excessive import of used vehicles would by far contribute to CO₂ emission, as used vehicles consume more fuel than brand new (Dr. Thomas Schiller, 2017).

Ethiopian Ministry of Health disclosed that air pollution level in the Ethiopian capital has exceeded the limit set by WHO and reached a critical stage. It is observed that 300 microgram Particulate matters were there rather a maximum of 20 microgram set by WHO.

There are two types of GHG emission estimation method. One is Fuel consumption base (IPCC,

USA, Japan), and the other is Vehicle traveling kilometer base (Australia). But most countries have used methods developed on the basis of the IPCC Guideline. For this study the IPCC guideline were used (Jiun-Horng Tsai, 2010).

CHAPTER THREE

MATERIALS AND RESEARCH METHODOLOGY

3.1 Research Methodology

This chapter will present the materials and research methodology used in this research. This provides framework in which this research was carried out and presented. The knowledge gained from the literature review and the HCM, 2010 methodology in the area of Signalized Intersection and past research carried out in the field of traffic flow problem on multi-lane intersection (Economic analysis), form the main building block for this evaluation framework. This chapter focuses on creating that understanding to formulate a framework for the performance evaluation together with cost estimation of the signalized intersection. In addition, modeling and evaluation using Synchro software has been discussed.

3.2 Study area description

Addis Ababa is the capital city of the Federal Democratic Republic of Ethiopia (FDRE) and a seat for heads of the African Union which is also a seat for the United Nations Economic Commission for Africa (UNECA) as well as a center of economic and political activities of the country. The total population of Addis Ababa is more than 4 million and expected to grow to 10 million in the year 2040 (Gauff Ingenieure GmbH & Co. KG -JBG, 2017). The high economic growth and growth of residential, business and settlements is leading the city's streets to overcrowding by traffic jams.

The Ras Mekonnen Street (between the Addis Ababa School of Commerce and Meskel Square) of Addis Ababa is the Principal Arterial Road with three (3) lanes in each flow direction and is one of the main corridors of the city which serves the east-west traffic of the city in a jammed situation. The Street is one of the route corridors in the city with high traffic density along the East to West corridor. Along the corridor, there is also a light transit railway that overpasses the route.

This study covers the Ras Mekonnen Street of Addis Ababa. The corridor includes three (3) major signal controlled intersections located at Addis Ababa University School of Commerce, Leghar and Meskel Square. In addition, because of its great significance during optimization, the

Estifanos Signalized intersection which is located at Jomo Kenyatta Street has also been included in this study. The study corridor is approximately 1975m in length as measured along the route (i.e. from the start of the Mexico intersection up to Estifanos intersection).

Figure 3-1 below shows the geographical location of the route;

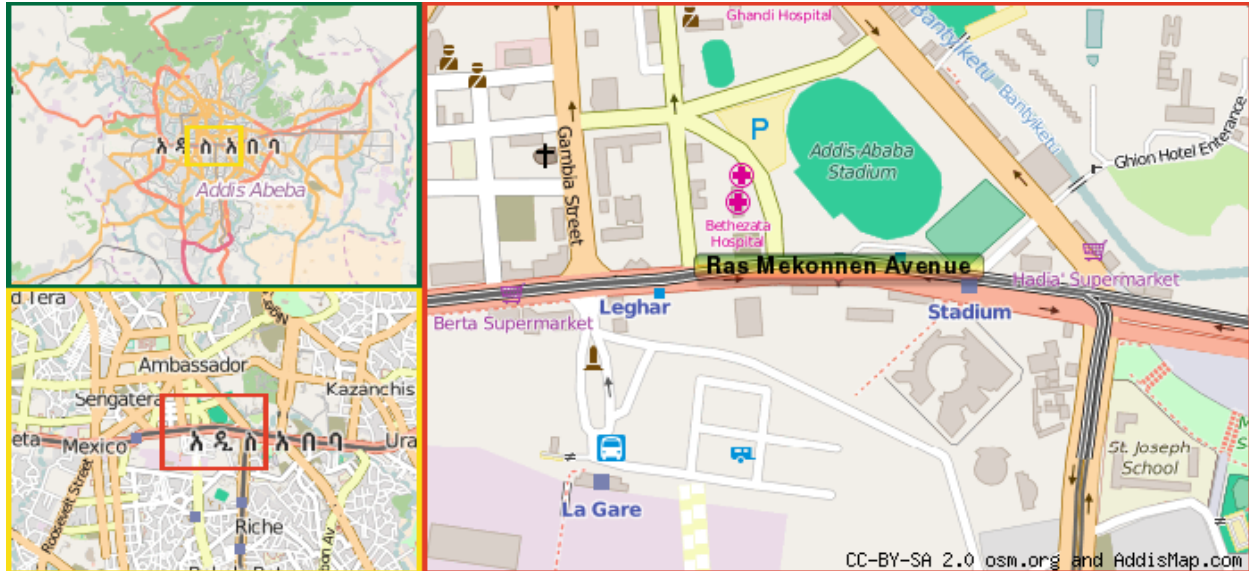


Figure 3-1: Map of the Ras Mekonnen Street together with Jomo Kenyatta Street

The details of aforementioned intersections are described here in below;

Addis Ababa University School of Commerce Intersection

The Addis Ababa University School of Commerce intersection is a three armed junction located near the Mexico light rail station. The junction serves the Commerce approach, Bunana Shay approach and Leghar approach. The Leghar and Mexico approach traffic is served by three lanes of which two are for through movement and the other is shared lane for right and/or left turn movement. However, the Commerce approach is an island type approach which accommodates the traffic by two lanes for both right and left turn movement. Figure 3-2 below shows the layout of Addis Ababa University School of Commerce intersection.

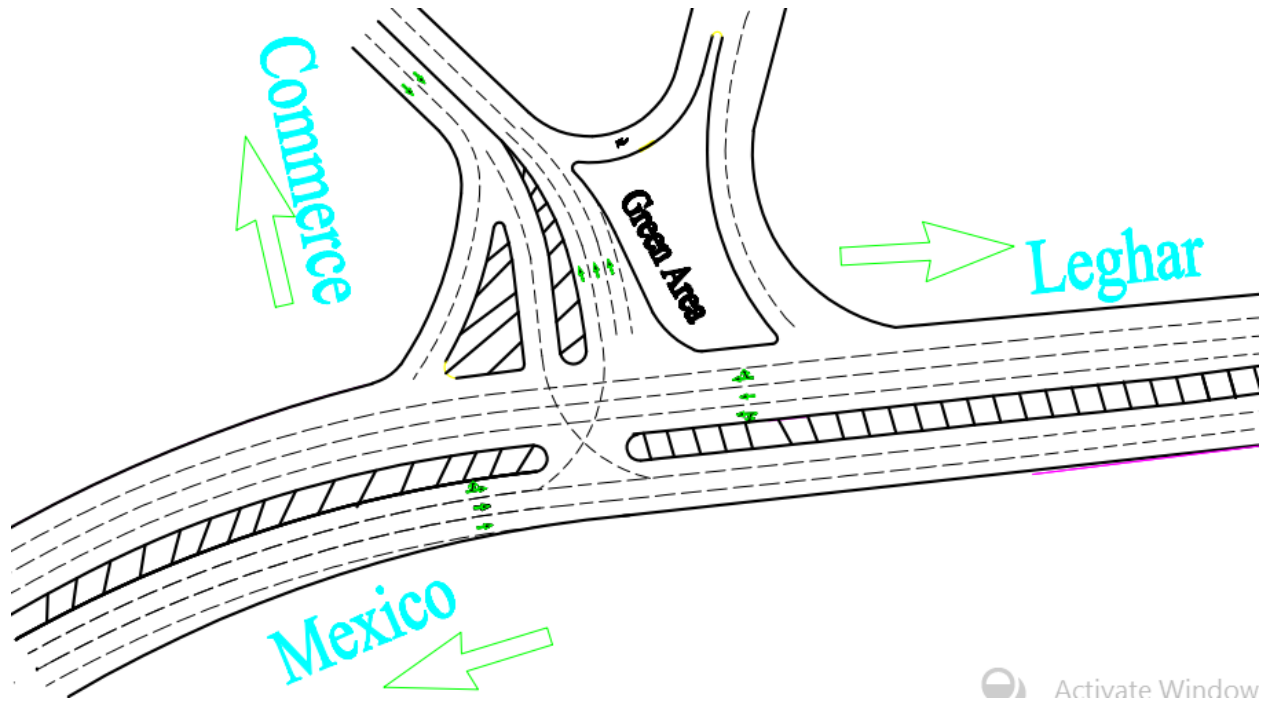


Figure 3-2: Layout of the Commerce Intersection

Leghar Intersection

Leghar Intersection is a four armed junction located around the old Ethio- Djibouti Railway Station. The junction serves the Piassa approach, Leghar approach, Stadium approach and Mexico approach traffic. At his intersection all right turns are not channelized. Figure 3-3 below shows the layout of Leghar intersection.

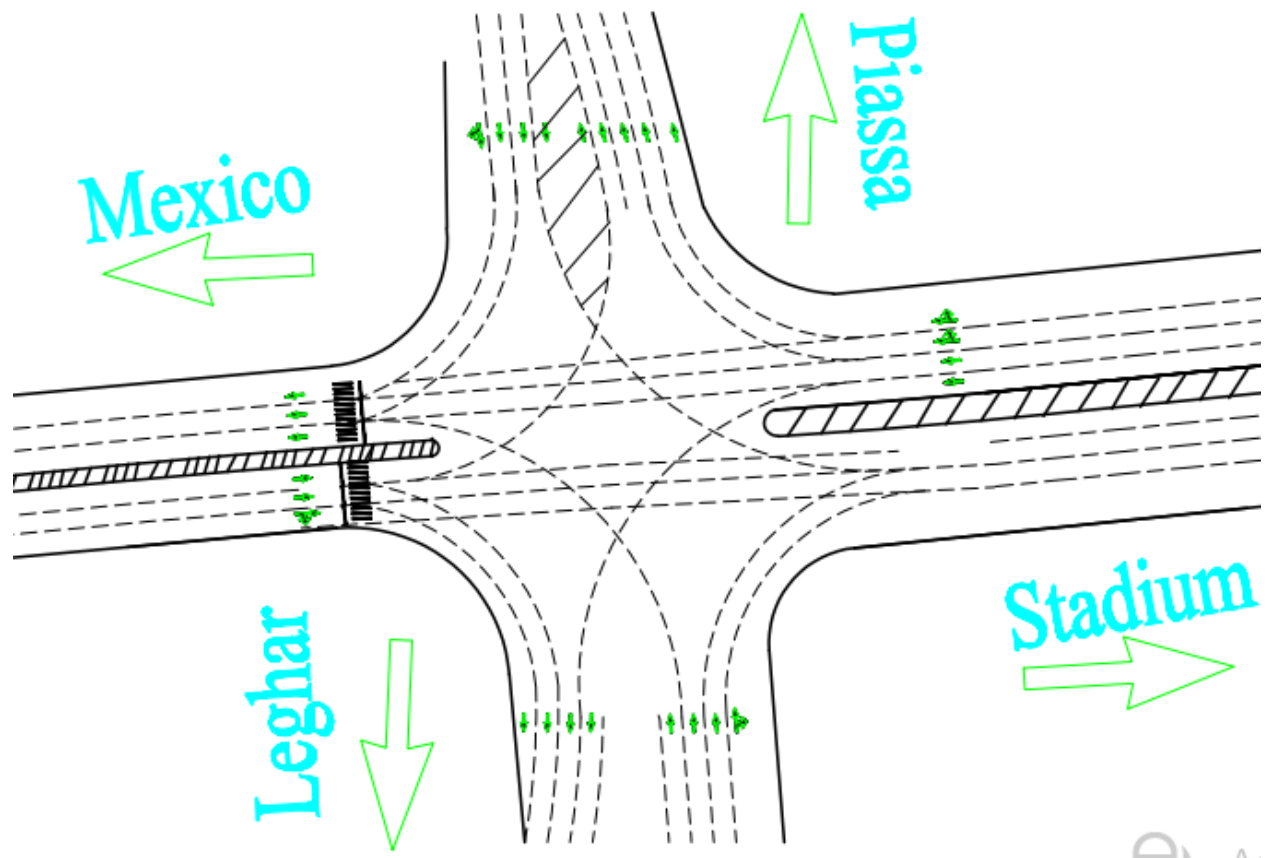


Figure 3-3: Layout of the Leghar Intersection

Meskel Square Intersection

It is a four armed junction that serves traffic flow which comes from Stadium approach, Saris approach, Estifanos approach and Piassa approaches. Except the Stadium approach, all others have a channelized right turn lane. This intersection serves a great number of traffic volumes as compared to other intersections in the corridor. Figure 3-4 below shows the layout of Meskel Square intersection.

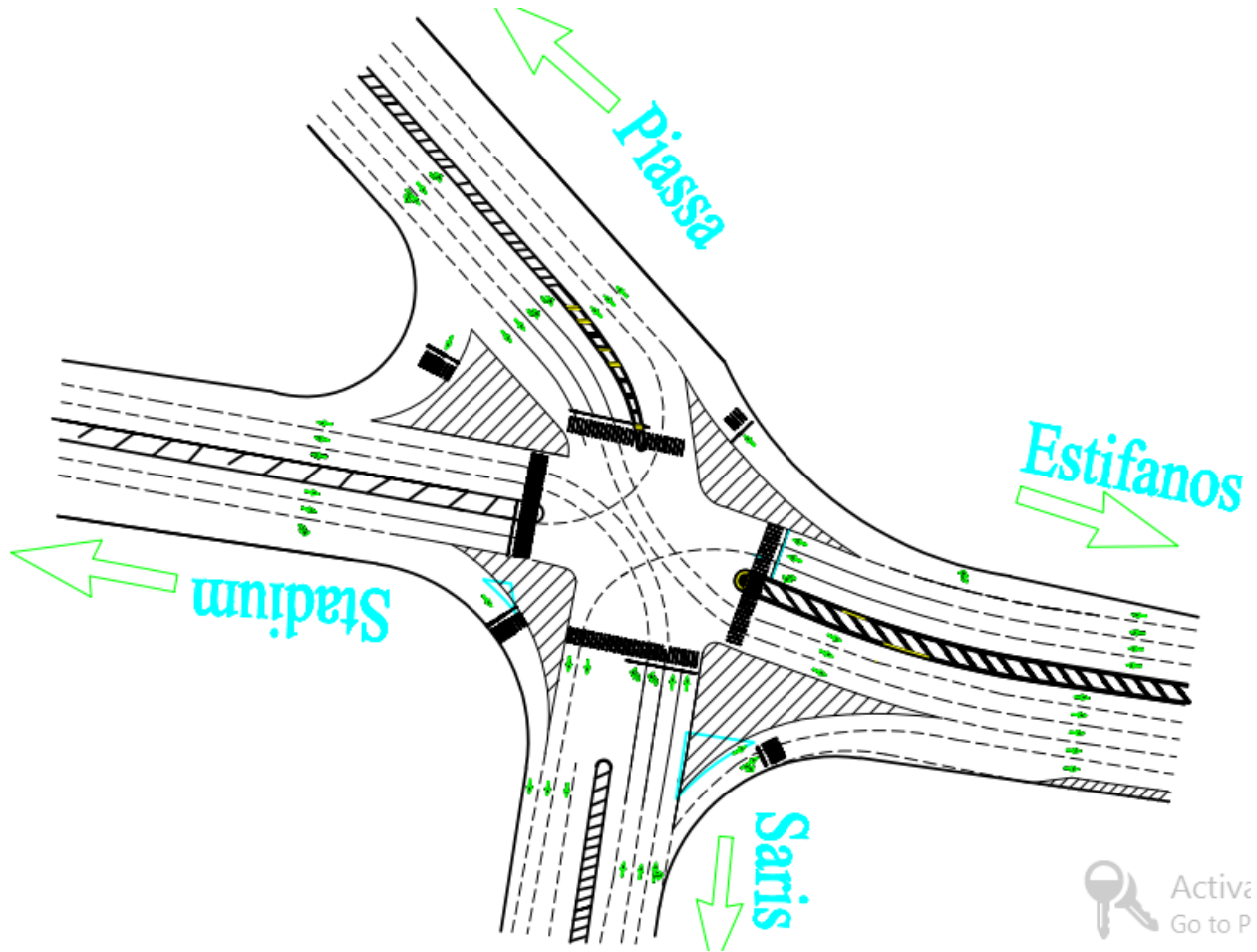


Figure 3-4: Layout of the Meskel Square Intersection

Estifanos Intersection

It is also a four armed junction that serves traffic flow which comes from Meskel Square approach, Haya Hulet approach, Estifanos approach and Bole approach. The centerline of North and South approach at this intersection does not coincide and having around 20m offset. All approaches except the Meskel Square has a channelized right turn. Figure 3-5 below shows the layout of Estifanos intersection.

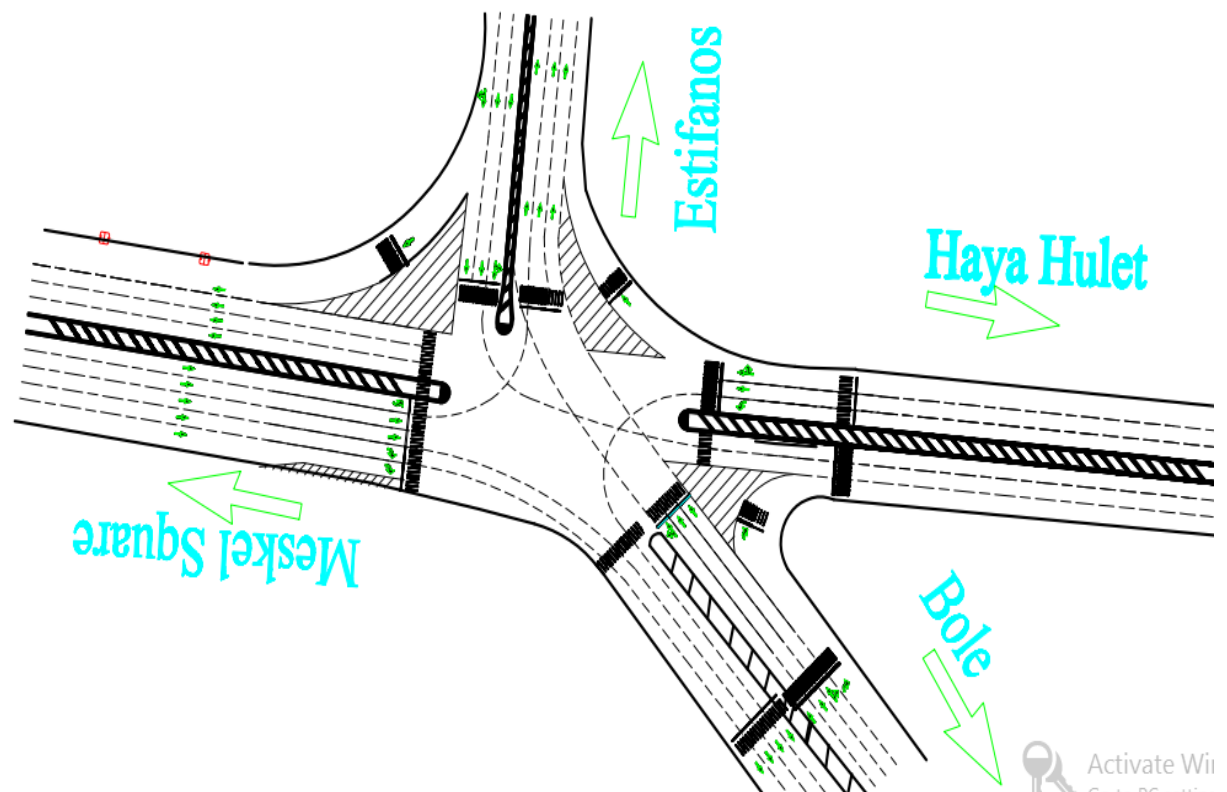


Figure 3-5: Layout of the Estifanos Intersection

3.3 Method of data collection

3.3.1 Traffic Data

The traffic data collection for this study was based on the Guideline of the Manual for Traffic Studies, Part 3; Traffic Counts it stated in the manual that the count period should avoid special event or compromising weather conditions and may range from 5 minutes to 1 year. Typical count periods are 15 minutes or 2 hours for peak periods, 4 hours for morning and afternoon peaks, 6 hours for morning, midday, and afternoon peaks, and 12 hours for daytime periods. The traffic counts during Monday mornings rush hour and Friday evenings rush hour may show exceptionally high volumes and were not normally used in analysis; therefore, counts are usually conducted on a Tuesday, Wednesday, or Thursday (Smith, 2002).

In this study, the count was conducted for six hour period on each date. The period of count was selected based on field observation. During field observation, it is found that weekdays are very susceptible for high traffic jams and peak traffic flows were also observed at morning, midday

and afternoon periods at each intersection. Accordingly, traffic data at the study area were collected from 10/10/17 to 12/10/17 by manual count method on Tuesday, Wednesday, Thursday for six hours per day at 15 (fifteen) minutes interval. The count data then were recorded separately for each movement in accordance to direction of flow.

The targets were the vehicles that entered into intersection as per their direction of flow. The Count started and ended at the same time at each intersection. Further, volume of pedestrians at cross and sidewalks were counted for one hour period.

Collection of traffic flow was collected using video cameras by fixing the cameras at Buildings and pedestrian bridges of light transit railway having clear view to the intersection. The following are sample Figures taken during Video recording. Figure 3-6 and 3-7 below show the Estifanos and Leghar Intersections Traffic Flow respectively.

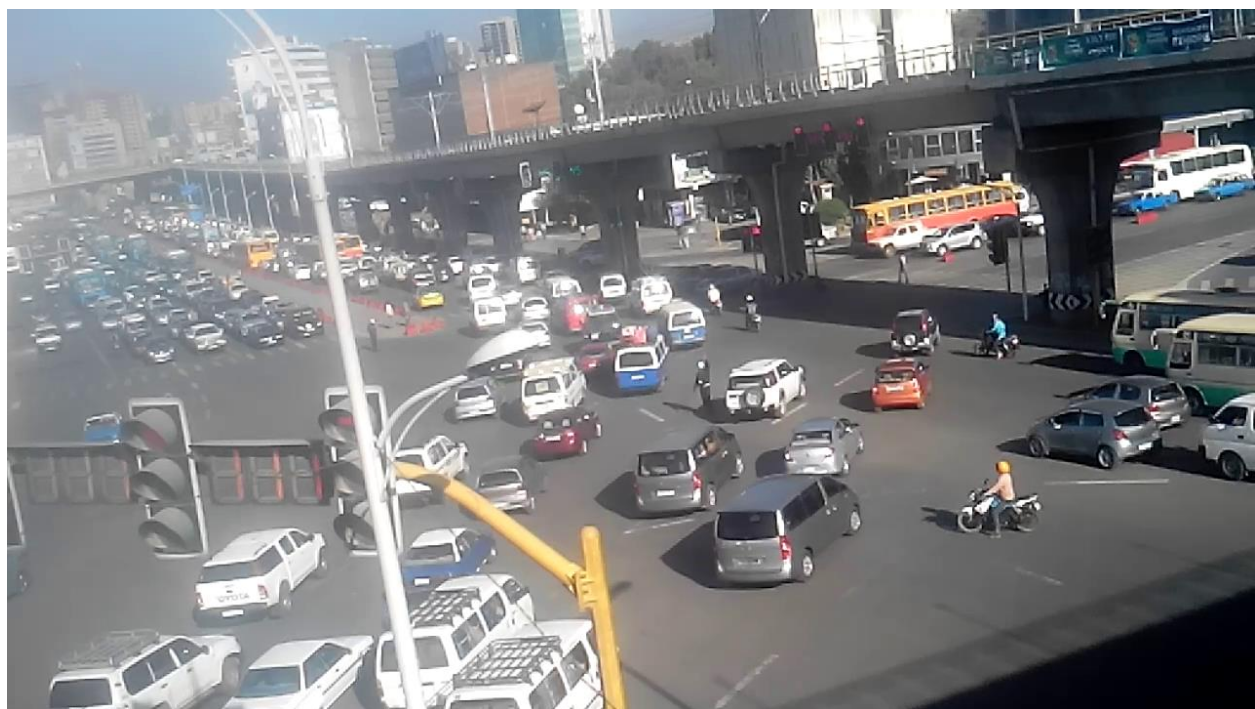


Figure 3-6: Estifanos Intersection flow



Figure 3-7: Leghar Intersection flow

Traffic flow count is categorized into six classes by visual assessment of the vehicle sizes, Corresponding passenger car equivalent values were considered as depicted in Table 3-1 below. Table below depicts the vehicle types and passenger Car unit (PCU) considered in this study (Andrew, 2003).

Table 3-1: PCU equivalent by Vehicle Type

I.No.	Vehicle Type	PCU
1.	Cycle	0.2
2.	M/ cycle	0.5
3.	Car (includes Passengers Cars, Station wagons, pick-up, M/bus)	1
4.	S/Truck includes Small buss 18-48 passenger occupancy and small truck	2
5.	M/Truck includes Medium Trucks and Large Bus 48-62 passenger occupancy	2.5
6.	H/Truck includes double cargo city busses and Truck and Trailer	3

Source: Botswana, Ministry of Works and Transport

From the collected traffic data, it was noted that the route corridor serves a peak flow during 10/10/2017 and 11/10/2017; which, for Commerce and Leghar Intersection flow 10/10/2017

during morning peak period and for Meskel Square and Estifanos Intersections Night is a peak period of traffic flow on 11/10/2017. The Collected traffic data together with the peak period is summarized in the Table 3-2 herein under;

Table 3-2: Traffic data

I.No	Name of Intersection	Approach Leg	Peak Period	V (Pc/h) (A)	PHF (B)	Vp (Adjusted flow rate) (pc/h) (C=A/B)	Maximum Intersection Flow (Pc/hr)
1.	Commerce	Mexico	10-10-17, M, @ 8:45-9:45	1976	0.93	2106	4733
		Leghar		2108	0.39	2244	
		Commerce		649	0.91	713	
2.	Leghar	Mexico	10-10-17, M, @ 8:45-9:45	1950	0.93	2092	5395
		Stadium		2030	0.89	2286	
		Piassa		668	0.85	784	
		Leghar		747	0.82	912	
3.	Meskel Square	Stadium	11-10-17, N, @ 5:15-6:15	3217	0.86	3725	9282
		Estifanos	11-10-17, N, @ 4:45-5:45	3966	0.92	4294	
		Piassa	11-10-17, N, @ 4:45-5:45	902	0.96	942	
		Saris		1197	0.96	1244	
4.	Estifanos	Meskel Square	10-10-17, N, @ 2:15-3:15	3398	0.91	3721	7822
		Estifanos	10-10-17, N, @ 1:45-2:45	1337	0.92	1446	
		Bole		2065	0.96	2150	
		22		1022	0.92	1103	

3.3.2 Pedestrians at Cross Walk

Analysis of LOS for Pedestrians crossing at signalized intersection is more complicated to analyze than a mid-block intersection; because, it involves intersecting side walk flows, pedestrian crossing the street and others waiting for the signal to change (Dr. Tom V, 2014).

In this study the effect of pedestrian and bicycle blockage to the vehicles waiting for the signal has been analyzed. The number of pedestrians on the cross walks were collected using separate spread sheet prepared for the same. The count was done irrespective of pedestrian characteristics

(i.e. age, physical ability, sex etc.) because the parameters used for pedestrian and bicycle blockages at conflict zone do not require such kind of data. The volumes were counted for a peak of one hour period at each conflict zones of leg of intersections of the road segment.

3.3.3 Saturation headway

The HCM suggests that for a given flow which have no local data, a base saturation flow rate of 1900 passenger cars per hour of green time per lane (pc/h/ln). However, this default value is not applicable to all locations since it differs with driver behavior, traffic conditions and road conditions. HCM clearly adds a disclaimer emphasizing the need for adopting locally measured values outside of U.S.

The base saturation flow rate assumes a set of “ideal” conditions that include: 12-ft lanes, no heavy vehicles, no turning vehicles, no local buses, level terrain, non- central business district (CBD) location, among others. It is usually impossible to find a location that has all of these conditions. In calibrating a base saturation flow rate, a location is sought with near ideal physical conditions. An approach with three or more lanes is recommended, as the middle lane can provide for observations without the influence of turning movements. Heavy vehicles cannot be avoided, but sites that have few heavy vehicles provide the best data. Even where data is observed under near ideal physical conditions, all headways observed after the first heavy vehicle must be discarded when considering the base rate (P.Roess, 2004).

The following criteria were set in order to obtain best sample intersection which satisfies the HCM requirements.

- Lane width 3.5m, the HCM requires 3.65 m;
- No parking adjacent to a travel lane within 250 ft (75 m) of the stop line;
- No disturbance from right turning vehicles on the other approaching lane;
- No disturbance from left turning vehicles on the other approaching lane;
- The approach’s grade is level;
- Approach with three or more lanes; and
- Middle lane serves through traffic without disturbance.

Accordingly, the Estifanos intersection at the Meskel Square approach has satisfied the abovementioned requirements and was selected for further study. After selecting the stated intersection, the data already recorded in video were analyzed to check the time for highest percentage of through-moving traffic. Accordingly, the following task has been done to extract information required for headway calculation. More than 30 signal cycles were used during the study which is almost above double of the requirement by HCM. The following steps were followed according to HCM;

- Measure the geometry of the area;
- Check that a minimum of eight vehicles were queued initially; and
- Record the time taken to traverse the ideal stop line (walk way end) by the fourth, tenth and last queued vehicle.

After doing so, the following information were extracted;

- The number of queued vehicles in PCU; and
- The time required to traverse the ideal stop line (walk way end) by the fourth, tenth and last queued vehicle.

Accordingly, the above information was obtained, the following formula was used and calculations performed to determine the saturation headway per vehicle.

$$\text{Saturation Headway} = \frac{TQ - T4}{VQt - 4} \quad \text{Equation 3.1}$$

Where,

TQ= time required by last queued vehicle (s);

T4= time required by fourth queued vehicle (s); and

VQt=total queued vehicle.

3.3.4 Bus Stoppage and Parking Maneuver Rates

Parking maneuver rates represents the count of influential parking maneuvers that occur at an intersection leg, as measured during the analysis period. An influential maneuver occurs directly adjacent to movement group, within a zone that extends from the stop line to a point 75m upstream of it. Maneuvers were recorded separately for each approach leg (HCM, 2010).

Whereas, bus stoppage rate represents the number of local buses that stop and block traffic flow in a movement group within a zone that extends from the stop line to a point 75m both at downstream and upstream of it. Local buses are buses which pick up and discharge passengers at a bus stop. Stoppage counts were also recorded separately for each approach leg (HCM, 2010).

In order to conduct the volume count, formats were prepared used for manual record. The volumes were counted for a peak of one hour period at each leg of intersections on the upstream and downstream of road segment.

3.3.5 Geometric Data

The geometric features or cross section elements of intersections; traffic lane width, number of travel lanes in each direction, divisional island, side walk, cross walk etc. we taken from the design documents of the route corridor from the Addis Ababa City Roads Authority as tabulated here in under;

Table 3-3: Geometric Data

I.No	Name of Intersection	Approach Leg	No of Lanes	Lane Width (m)	Median	Sidewalk width (m)	Crosswalk Width (m)
5.	Mexico	Mexico	3	3.5	Raised	4.0	4.3
		Leghar	3	3.5		4.3	4.3
		Commerce	4	3.5		8.0	4.3
6.	Leghar	Mexico	3	3.5	Raised	4.0	4.3
		Stadium	3	3.5		4.0	4.3
		Piassa	4	3.5	Only Marked	5.4	4.3
		Leghar	4	3.5		5.0	4.3
7.	Meskel Square	Stadium	3	3.5	Raised	4.0	4.3
		Estifanos	4	3.5		10.0	4.3
		Piassa	3	3.5		4.0	4.3
		Saris	3	3.5		4.5	4.3
8.	Estifanos	Meskel Square	5	3.5	Marked	10.0	4.3
		Estifanos	4	3.5		4.0	4.3
		Bole	4	3.5	Raised	4.5 ²	4.3
		22	3	3.5	Marked	4.0 ¹	4.3

¹ the other side is 10m wide, ² the other side is 4m wide

3.3.6 Signal data

The signal time of the existing signalized intersections; cycle time, allocated green time for each approach, all red and any other allocated signal timing is taken from the Addis Ababa Traffic Management Agency. The agency has a dongle; software with a flashed link which is used to adjust the signal timing. The signal timing of the route corridor is as summarized in the Tables 3.4, 3.5, 3.6 and 3.7 below;

Table 3-4: Mexico Intersection Signal data

Phase	Approach green time (sec)			Remark
	Mexico	Leghar	Commerce	
1	25			all movement
2	80	80		Through only
3	40	40		Left
4			25	all movement

There are two schedules, but, the second schedule is after 5:00 PM which is flashing only

Table 3-5: Leghar Intersection Signal data

Phase	Approach green time (sec)				Remark
	Mexico	Leghar	Piassa	Stadium	
1				44	all movement
2	44			44	Through only
3	43				all movement
4			25		all movement
5		20			all movement

Table 3-6: Meskel Square Intersection Signal data

Phase	Approach green time (sec)				Remark
	Saris	Stadium	Estifanos	Piassa	
Schedule 1				12:30 --6:00 PM @local	
1			32		all movement
2		63	63		Through only
3		15			all movement
4	30				all movement
5				27	all movement
Schedule 2				6:00-3:30PM @local	

1			36		
2		60	60		all movement
3		14			Through only
4	25				all movement
5				32	all movement
					all movement

Table 3-7: Estifanos Intersection Signal data

Phase	Approach green time (sec)				Description
	Haya Hulet	Meskel Square	Bole	Estifanos	
1	21				all movement
2	37		37		Through only
3			30		all movement
4				35	all movement
5			50		all movement

3.3.7 Travel time and fuel consumption data

The conditions of travel time together with fuel consumption at congested intersections were analyzed from questionnaires distributed to road users that pass the Ras Mekonnen Street. The questionnaire sheet include: job, income, purpose of trip, number of trips per day, travel speed, travel time lost and characteristics of respondents. In addition to the above, amount of fuel consumed on the study route and users understanding on source of congestion were also included in questionnaires. The respondents were selected using cluster sampling strategies because it is better to organize the collected data into clusters like passengers, drivers and private car owners.

In connection, the sample size was determined based on National Education Association formula as presented by Krejcie (2012) on his paper on determining sample size for research activities as follows;

$$s = \frac{X^2 NP(1-P)}{d^2(N-1)} + X^2 P(1 - P) \quad \text{Equation 3.2}$$

Where;

s = required sample size;

X^2 = chi-square value for 1 degree of freedom at the desired confidence level (3.841);

N = the population size;

P = the population proportion (assumed to be 0.5 since this would provide the maximum sample size); and

d = the degree of accuracy expressed as proportion (0.05).

Accordingly, about 200 copies of the questionnaire were prepared and distributed to road users, of which 100 for passengers and 100 for drivers and private car owners. From the distributed questionnaires, 60 from drivers and private car owners and 59 from passengers were returned.

3.3.8 Emission

Inroad vehicles emission can be measured directly by directing the vehicles the road side test locations and also using measuring instrument that measure component of atmosphere. Using vehicles emission control technologies (ECT) fixed with the vehicles and equipment like wayer smoke opacity of vehicles can be estimated (Shankute, 2015).

Determination of number vehicles is not sufficient to estimate vehicle emissions. Depending on the fuel type used in vehicles amount of pollutants vary. The number of gasoline vehicles registered in Ethiopia becomes constant from 2002-2009 with a share of 27.7% and 28.1% in the first and last four years of the aforementioned year respectively from the total registered vehicles. The proportion of vehicles estimated by AAiT should also be used in this study (AAiT, 2012).

In addition to the above, the other important parameter while determining amount of the pollutants is emission factor. Emission factors depend on type and compound of the fuel and ignition system. Taking into considerations of those factors Hamit SOLMAZ have determined emission factors depending on types of vehicle for gasoline and diesel fuel (SOLMAZ, 2012).

Neway (2016), in his study on Investigation on Pollution Caused by Gasoline and Diesel fuelled Vehicles determined emission factors for Ethiopian conditions. In the study, lots of parameters were taken into account such as, fuel consumption, age of vehicles, average vehicles kilometers

traveled in a year and accordingly CO, NO_x, CO₂, PM and HC pollutants emission factors were determined. According to his study, the highest level of NO_x emissions were estimated at 3.44 g/km, HC emissions at 6.53 g/km, CO at 13.9 g/km, PM_{2.5} at 1.3 g/km and 35.96 g/km of CO₂. As the research was relatively current the estimated emission factors were used in this study.

Having this data the emission from vehicles is estimated by multiplying average vehicles kilometers traveled by emission factor.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Signal Timing Performance Evaluation

4.1.1 Saturation Flow rate

Saturation Flow rate is the maximum hourly service rate of the intersection under prevailing traffic conditions like lane width, number of lanes, presence of heavy vehicle and others, assuming that the green time is displayed at full hour. It may vary significantly at locations depending on traffic composition and different communities.

The studies in U.S show that Saturation Flow rate ranges from 1500 to 2000 Pc/h/ln. Accordingly, HCM established default value of Base Saturation Flow rate is 1900 pc/h/ln. However, determination using local data is recommended in order to get more accurate result for the local conditions. In order to determine Saturation Flow rate, estimation of base Saturation Flow rate is required (HCM, 2010).

The base Saturation Flow rate can be calculated using the following formula (HCM, 2010);

$$S = \frac{3600}{h} \quad \text{Equation 4.1}$$

Where,

S=base saturation flow rate (Veh/h), and

h=Saturation headway (s)

Saturation headway is an average amount of time of vehicles at queue takes to pass through a signalized intersection at green time occurring after forth vehicle pass a signalized intersection assuming that there is continuous queue of vehicles moving through intersection.

The Saturation headway can be calculated using the following formula (HCM, 2010);

$$h = \frac{T}{N} \quad \text{Equation 4.2}$$

Where,

T=Queue discharge time (s); and

N= Number of through vehicles in queue passing intersection minus four vehicles.

Kidus Ayalneh, (2014) found that the saturation flow rate on Addis Ababa's multi-lane highways

range from 1789 to 1820. During his study he used two roundabouts and three unsignalized intersections. However, in this study a signalized intersection which satisfies the requirements stated in the HCM was selected and data were updated and was adopted during assessment as a base rate selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located.

The Meskel Square approach leg towards the Estifanos Intersection was selected for the study. The approach has 5 lanes having 3.5m width, of which the middle lane serves only through traffic. Thirty two cycle times were selected to estimate the Saturation headway at this intersection and Saturation headways at 4th, 10th and the last queued vehicle were estimated as presented in Table 4-1 herein under.

Table 4-1: Saturation headway at 4th, 10th and the last queued vehicle

	Discharge Headway with Reference to the Last Surveyed Vehicle	Discharge Headway with Reference to the Tenth Vehicle	Discharge Headway with Reference to the Fourth Vehicle
Minimum Discharge Headway (s)	1.6	1.5	1.5
Maximum Discharge Headway (s)	2.1	2.3	2.8
Average Discharge Headway (s)	1.8	1.9	2.1
Saturation Flow Rate (pc/h/ln)	1973	1879	1695

In Table 4-1 above, it is to be noted that no major difference is observed on the saturation headway with reference to last surveyed and tenth vehicle. However, the value observed at fourth vehicle has major difference from the other. In addition, it is noted that the saturation flow rate ranged from 1.6 to 2.05 with average value of 1.825. The current estimation of the saturation flow value is higher than the HCM default value; this shows the driver's in the city have more aggressive nature towards accepting the traffic signal. Accordingly, the base Saturation flow for intersection is adopted to be 1973 Veh/h/ln. Figure 4.1 below illustrates the saturation flow, 4th vehicle.

The detail relation is also shown in the chart shown below;

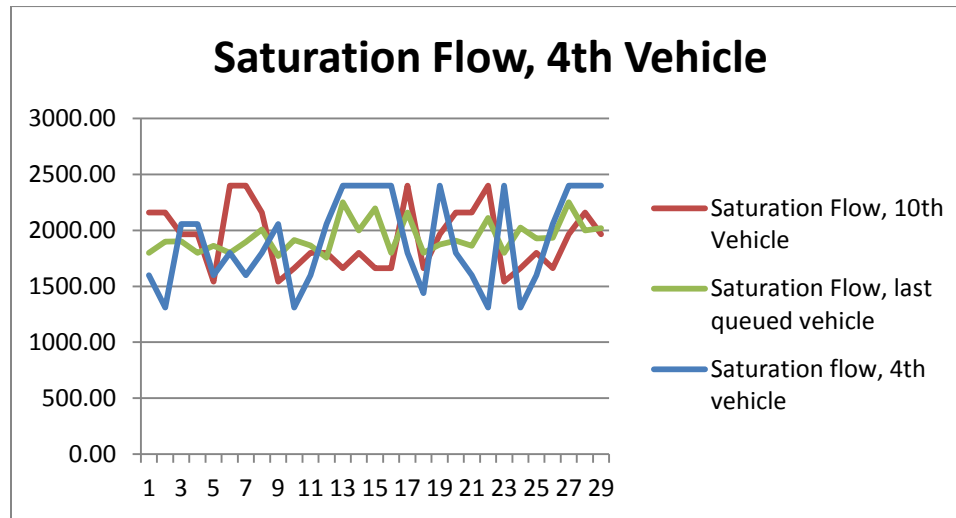


Figure 4-1 Saturation headway at 4th, 10th and the last queued vehicle

The prevailing conditions were determined according to the HCM and accordingly, the adjusted saturation flow rate of the study area intersections were estimated using the following equation;

$$S = so * fw * fHV * fg * fp * fbb * fa * fLU * fLT * fRT * fLpb * fRpb \quad \text{Equation 4.2}$$

Where,

S = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);

so = base saturation flow rate per lane (pc/h/ln);

fw = adjustment factor for lane width;

fHV = adjustment factor for heavy vehicles in traffic stream;

fg = adjustment factor for approach grade;

fp = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;

fbb = adjustment factor for blocking effect of local buses that stop within intersection area;

fa = adjustment factor for area type;

fLU = adjustment factor for lane utilization;

fLT = adjustment factor for left turns in lane group;

fRT = adjustment factor for right turns in lane group;

fLpb = pedestrian adjustment factor for left-turn movements; and

fRpb = pedestrian-bicycle adjustment factor for right-turn movements.

The above listed adjustment factors are described as follow;

I. Adjustment for Lane Width

The lane width adjustment factor (f_w) accounts for the negative impact of narrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes. Table 4-2 below presents the adjustment for f_w .

Table 4-2: Adjustment for Lane Width (f_w)

Average lane width (m)	Adjustment factor (f_w)
<3	0.96
3.0-3.9	1.00
>3.9	1.04

All lane groups of the entire route intersections have 3.5m wide lanes. Hence a lane adjustment factor of 1 was adopted in the study.

II. Adjustment for Heavy Vehicles, f_{HV}

The heavy-vehicle adjustment factor (f_{HV}) accounts for the additional space occupied by heavy vehicles and for the difference in their operating capabilities, compared with passenger cars. This factor does not address local buses that stop within the intersection area.

$$f_{HV} = \frac{100}{100+PHV(ET-1)} \quad \text{Equation 4.3}$$

Where;

f_{HV} = heavy-vehicle adjustment factor;

PHV =percent heavy vehicles in the corresponding movement group (%); and

ET =equivalent number of through cars for each heavy vehicle = 2.0.

III. Adjustment for Grade

The grade adjustment factor (f_g) accounts for the effects of approach grade on vehicle performance.

$$f_g = 1 - \frac{pg}{200} \quad \text{Equation 4.4}$$

where,

f_g = grade adjustment factor; and

pg= grade of segment.

This factor applies to grades ranging from -6.0% to +10.0%.

The fg and fHV factors are summarized in the Table 4.3 below.

Table 4-3: Adjustment for Heavy Vehicles (HV) and grade

I.No	Name of Intersection	Approach Leg	% of HV	fHV	fg
1.	Commerce	Mexico	1.6	0.98	1
		Leghar	3.0	0.97	1
		Commerce	1.3	0.99	1
2.	Leghar	Mexico	3.5	0.97	1
		Stadium	3.8	0.96	1
		Piassa	4.3	0.96	1
		Leghar	7.6	0.93	1
3.	Meskel Square	Stadium	5.0	0.95	1
		Estifanos	4.4	0.96	1
		Piassa	1.3	0.99	1
		Saris	2.4	0.98	1
4.	Estifanos	Meskel Square	4.5	0.96	1
		Estifanos	2.0	0.98	1
		Bole	1.4	0.99	1
		Haya Hulet	3.1	0.97	1

IV. Adjustment for Parking

The parking adjustment factor (fp) accounts for the frictional effect of a parking lane on flow in the lane group adjacent to the parking lane. It also accounts for the occasional blocking of an adjacent lane by vehicles moving into and out of parking spaces. If no parking is present, then this factor has a value of 1.00.

$$fp = \frac{N - 0.1 - \frac{18Nm}{3600}}{N} > 0.05 \quad \text{Equation 4.5}$$

Where;

fP= parking adjustment factor;

Nm = parking maneuver rate directly adjacent to the lane group and within 75m upstream of the stop line (maneuvers/h); and

N = number of lanes in lane group.

V. Adjustment for Bus Blockage

The bus-blockage adjustment factor (f_b) accounts for the impact of local transit buses that stop to discharge or pick up passengers at a near-side or far-side bus stop within 75m of the stop line (upstream or downstream). If on both cases of vehicles stoppage, the maximum value is adopted as both cases could not appear at the same time.

$$f_{bb} = \frac{N - \frac{14.4Nb}{3600}}{N} > 0.05 \quad \text{Equation 4.6}$$

Where;

f_{bb} = bus-blockage adjustment factor

N_b = bus blockage rate directly adjacent to the lane group and within both 75m upstream and downstream of the stop line (maneuvers/h), and

N = number of lanes in lane group.

The f_p and f_{bb} factors are summarized in Table 4-4 below.

Table 4-4: Adjustment for parking and Bus Blockage

I.No	Name of Intersection	Approach Leg	No of lanes	Nm (Veh/h)	Nb at Up Stream (Veh/h)	Nb at Down Stream (Veh/h)	f_p	f_{bb}
1.	Commerce	Mexico	3	11	0	0	0.95	1.00
		Leghar	3	60	42	155	0.87	0.79
		Commerce	4	No Parking	0	0	1.00	1.00
2.	Leghar	Mexico	3	No Parking	0	250	1.00	0.67
		Stadium	3	No Parking	0	0	1.00	1.00
		Piassa	4	6	0	0	0.97	1.00
		Leghar	4	10	0	31	0.96	0.97
3.	Meskel Square	Stadium	3	6	0	0	0.96	1.00
		Estifanos	4	17	0	0	0.95	1.00
		Piassa	3	31	0	30	0.92	0.96
		Saris	3	No Parking	0	85	1.00	0.89
4.	Estifanos	Meskel Square	5	31	0	69	0.95	0.94
		Estifanos	4	No Parking	0	86	1.00	0.91
		Bole	4	No Parking	0	69	1.00	0.93
		Haya Hulet	3	No Parking	0	0	1.00	1.00

VI. Adjustment for Area Type

The area type input is used to indicate whether the intersection is in central business district

(CBD) type of environment. An intersection in a CBD, or a similar type of area is considered when its characteristics include narrow street rights-of-way, frequent parking maneuvers, vehicle blockages, taxi and bus activity, small-radius turns, limited use of exclusive turn lanes, high pedestrian activity, dense population, and midblock curb cuts. The Study area has very limited rights-of-way, high pedestrian activity, high taxi and bus activity. Hence, this area is considered as central business district so that an adjustment factor value of 0.90 is adopted.

VII. Adjustment for Lane Utilization

The input lane utilization adjustment factor is used to estimate saturation flow rate for a lane group with more than one exclusive lane. If the lane group has one shared lane or one exclusive lane, then this factor is 1.0, as the demand exceed capacity the drivers have no opportunity to select their lane freely. Hence, a lane utilization factor of 1 is adopted for the entire intersection.

In the study area, the demand exceeds the intersection capacity such that the drivers have no opportunity to select their lane freely even if drivers tries to change their lane improperly due to their aggressive nature.

VIII. Adjustment for Right Turns

The right-turn adjustment factor (f_{RT}) is intended primarily to reflect the effect of right-turn path geometry on saturation flow rate.

$$f_{RT} = \frac{1}{ER} \quad \text{Equation 4.7}$$

Where;

f_{RT} = right-turn adjustment factor; and

ER is the equivalent number of through cars for a protected right-turning vehicle (= 1.18).

On situations in which an exclusive right-turn lane could have a higher flow rate than the adjacent through lane, the right turns that could occur simultaneously with a protected left-turn movement from the cross street should be deducted from the right-turn flow rate.

For shared lane

$$f_{RT} = 1 - 0.15P_{RT} > 0.05 \quad \text{Equation 4.8}$$

IX. Adjustment for Left Turns

The left-turn adjustment factor (fLT) is intended primarily to reflect the effect of left-turn path geometry on saturation flow rate.

$$fLT = \frac{1}{EL} \quad \text{Equation 4.9}$$

Where;

fLT= left-turn adjustment factor; and

EL is the equivalent number of through cars for a protected left-turning vehicle (= 1.05)

For shared lane

$$fLT = \frac{1}{1+0.05PLT} > 0.05 \quad \text{Equation 4.10}$$

Table 4-5: Adjustment for Right and Left Turn

I.No	Name of Intersection	Approach Leg	Lane Type	V (pc/h)	LTV (pc/h)	RTV (pc/h)	PLT (%)	PRT (%)	fLT	fRT
1.	Mexico	Mexico	SL&SR	2318	351	0	15.12	0.00	0.99	1.00
		Leghar	SL&SR	2032	0	510	0.00	25.10	1.00	0.96
		Commerce	EL&ER	598.3	269	330	44.89	55.10	0.95	0.85
2.	Leghar	Mexico	SL&SR	1951	340	124	17.40	6.30	0.99	0.99
		Stadium	SL&SR	2030	279	215	13.70	10.57	0.99	0.98
		Piassa	SL&SR	668	335	138	50.20	20.70	0.98	0.97
		Leghar	SL&SR	748	187	206	25.03	27.50	0.99	0.96
3.	Meskel Square	Stadium	EL&ER	3218	195	679	6.04	21.08	0.95	0.85
		Estifanos	EL&ER	3967	745	1156	18.80	29.13	0.95	0.85
		Piassa	EL&ER	902	621	11	68.80	1.20	0.95	0.85
		Saris	EL&ER	1197	417	283	34.80	23.60	0.95	0.85
4.	Estifanos	Meskel Square	EL&ER	3399	544	1532	16.00	45.07	0.95	0.85
		Estifanos	SL&ER	1337	92	541	6.80	40.43	1.00	0.85
		Bole	SL&ER	2066	1301	17	62.90	0.80	0.97	0.85
		22	EL&ER	1022	135	12	13.20	1.17	0.95	0.85

Note: SL= Shared left turn lane, SR= Shared right turn lane, EL= Exclusive left turn lane, ER= Exclusive right turn lane. PLT = percentage of left turn volume, PRT= percentage of right turn volume, LTV= left turn volume and RTV= right turn volume

X. Adjustment for Pedestrians and Bicycles

In order to estimate the pedestrian-bicycle adjustment factor fLpb, fRpb, first we need to determine the pedestrian volume at cross walks. Table 4-6 to 4-9 below shows the Pedestrian volume at cross walks counted at all intersections.

Table 4-6: Pedestrian volume at cross walks (CW), Mexico Intersection

Time	Approach Legs		
	Mexico (CW-3)	Leghar (CW-4)	Commerce (CW1 &2)
1:45 - 2:00	110	154	305
2:00 - 2:15	105	162	303
2:15 - 2:30	154	147	345
2:30 - 2:45	97	159	247
<i>Total</i>	<i>466</i>	<i>622</i>	<i>1200</i>

Table 4-7: Pedestrian volume at cross walks (CW), Leghar Intersection

Time	Approach Legs			
	Mexico (CW-4)	M/Sq (CW-3)	Leghar (CW-2)	Piassa (CW-1)
11:00-11:15	161	282	315	187
11:15-11:30	175	250	274	190
11:30-11:45	191	300	291	200
11:45-12:00	117	284	264	175
<i>Total</i>	<i>644</i>	<i>1116</i>	<i>1144</i>	<i>752</i>

Table 4-8: Pedestrian volume at cross walks (CW), Meskel Square Intersection

Time	Approach Legs			
	Stadium (CW-4)	Estifanos (CW-3)	Piassa (CW-1)	Saris (CW-2)
1:45 - 2:00	130	270	254	374
2:00 - 2:15	127	224	277	357
2:15 - 2:30	174	145	315	322
2:30 - 2:45	201	198	154	297
<i>Total</i>	<i>632</i>	<i>837</i>	<i>1000</i>	<i>1350</i>

Table 4-9: Pedestrian volume at cross walks (CW), Estifanos Intersection

Time	Approach Legs			
	M/Sq (CW-4)	Estifanos (CW-1)	Bole (CW-2)	Haya Hulet (CW-3)
1:45 - 2:00	133	165	314	417
2:00 - 2:15	125	147	297	315
2:15 - 2:30	69	200	320	324
2:30 - 2:45	178	184	189	412
<i>Total</i>	<i>505</i>	<i>696</i>	<i>1120</i>	<i>1468</i>

The procedure to determine the left-turn pedestrian-bicycle adjustment factor (f_{Lpb}) and the right-turn pedestrian-bicycle adjustment factor (f_{Rpb}) was based on the concept of conflict zone

occupancy, which accounts for the conflict between turning vehicles, pedestrians, and bicycles. Relevant conflict zone occupancy takes into account whether the opposing vehicle flow is also in conflict with the left-turn movement. The proportion of green time in which the conflict zone was occupied was determined as a function of the relevant occupancy and the number of receiving lanes for the turning vehicles. Figure 4.2 below shows the Conflict Zone between pedestrians and vehicles at cross walk.

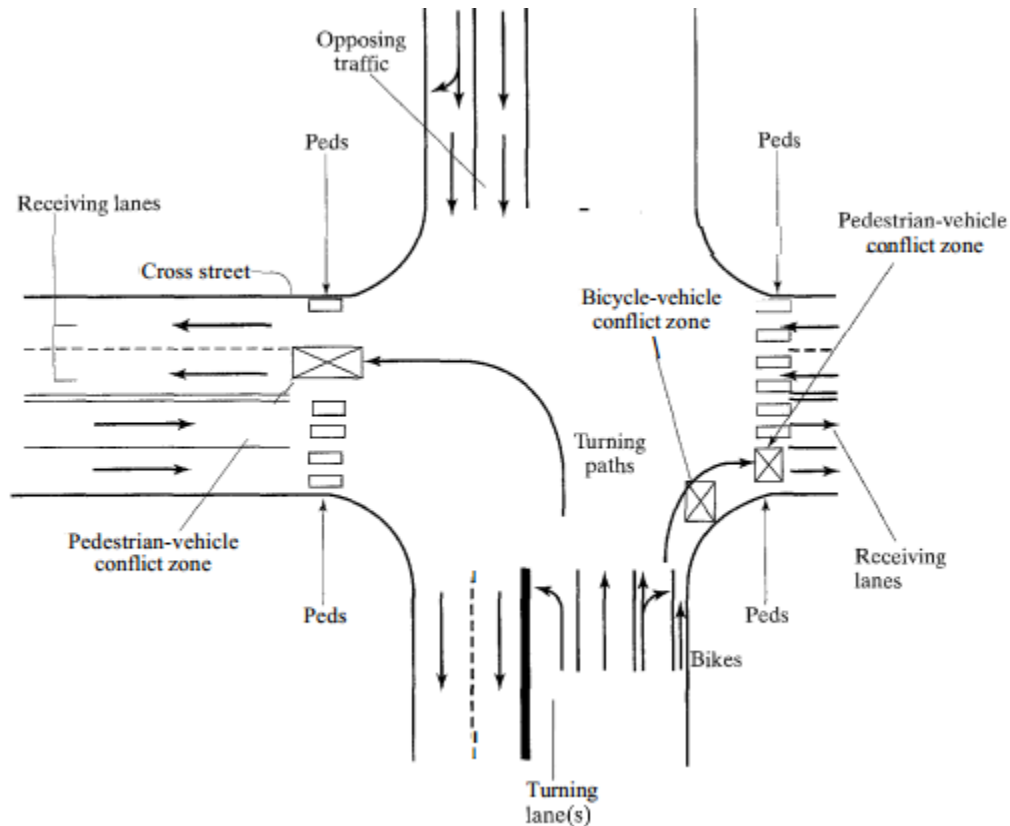


Figure 4-2: Conflict Zone

The adjustment factor for lane groups is determined as follows

- Determine Pedestrian Flow Rate During Service

The rate is adjusted to reflect the fact that pedestrians are moving only during the green phase of the signal. However, in actual scenarios of Addis Ababa, pedestrians do not wait for green phase.

$$V_{pedg} = V_{ped} \frac{c}{g_{ped}} < 5000 \quad \text{Equation 4.11}$$

Where,

V_{pedg} = pedestrian flow rate during the pedestrian service time (p/h);

v_{ped} = pedestrian flow rate in the subject crossing (walking in both directions) (p/h);

C = cycle length (s);
 gped= pedestrian service time (s); and
 Pc= pedestrians clear setting

If the phase providing service to pedestrians is actuated, then $g_{ped} = \min(g, Walk + PC)$. Otherwise, the pedestrian service time can be assumed to equal the effective green time for the phase (i.e., $g_{ped} = g$). If the minimum green time for the phase is adopted, the effect of pedestrians and bicycles on saturation flow is lower.

- Determine Average Pedestrian Occupancy

$$OCC_{pedg} = \frac{V_{pedg}}{2000}, \text{ if } V_{pedg} < 1000 \quad \text{Equation 4.12}$$

$$OCC_{pedg} = 0.4 + \frac{V_{pedg}}{10,000} < 0.9, \text{ if } V_{pedg} > 1000 \quad \text{Equation 4.13}$$

- Determine Relevant Conflict Zone Occupancy

For right-turn movements with no bicycle interference or for left-turn movements from a one-way street

$$OCC_r = \frac{g_{ped}}{g} * OCC_{pedg} \quad \text{Equation 4.14}$$

In our case, $OCC_r = OCC_{pedg}$, as the Pedestrians green time is equal with the effective green time.

- Determine Unoccupied Time

If the number of cross-street receiving lanes is equal to the number of turn lanes, then turning vehicles will not be able to maneuver around pedestrians or bicycles.

$$ApbT = 1 - OCC_r \quad \text{Equation 4.15}$$

If the number of cross-street receiving lanes exceeds the number of turn lanes, turning vehicles will more likely maneuver around pedestrians or bicycles. In this situation, the effect of pedestrian and bicycles on saturation flow is lower.

$$ApbT = 1 - 0.6 * OCC_r \quad \text{Equation 4.16}$$

- Determine Saturation Flow Rate Adjustment Factor

$$fR_{pb} = ApbT \quad \text{Equation 4.17}$$

$$fL_{pb} = ApbT \quad \text{Equation 4.18}$$

Detail computation of Adjustment for Pedestrians and Bicycles discussed herein above is also summarized in Table 4-7 and 4-8 below.

Table 4-10: Adjustment for Pedestrians and Bicycles for left Turn Conflict zone

I.No	Name of Intersection	Approach Leg	Pedestrian volume (Vp/h)	Gp (sec)	C (sec)	Vpedg (p/h)	OCCped	OCCr	ApbT
1.	Commerce	Mexico	1200	120	188	766	0.38	0.38	0.77
		Leghar	0	40		132	0.07	0.07	0.96
		Commerce	622	25		160	0.08	0.08	0.95
2.	Leghar	Mexico	752	43		147	0.07	0.07	0.96
		Stadium	1144	44		261	0.13	0.13	0.92
		Piassa	1116	25		100	0.05	0.05	0.97
		Leghar	644	25		152	0.08	0.08	0.95
3.	Meskel Square	Stadium	1000	78		262	0.13	0.13	0.92
		Estifanos	1350	95		423	0.21	0.21	0.87
		Piassa	837	27		144	0.07	0.07	0.96
		Saris	632	30		215	0.11	0.11	0.94
4.	Estifanos	Meskel Square	696	67		180	0.09	0.09	0.95
		Estifanos	1468	35	130	0.06	0.06	0.96	
		Bole	505	50	298	0.15	0.15	0.91	
		Haya Hulet	1120	58	453	0.23	0.23	0.86	

Table 4-11: Adjustment for Pedestrians and Bicycles for Right Turn Conflict zone

I.No	Name of Intersection	Approach Leg	Pedestrian volume (Vp/h)	Gp (sec)	C (sec)	Vpedg (p/h)	OCCped	OCCr	ApbT
1.	Commerce	Mexico	1200	120	188	0	0.00	0.00	1.00
		Leghar	0	40		255	0.13	0.13	0.92
		Commerce	622	25		62	0.03	0.03	0.98
2.	Leghar	Mexico	752	43		262	0.13	0.13	0.92
		Stadium	1144	44		176	0.09	0.09	0.95
		Piassa	1116	25		86	0.04	0.04	0.97
		Leghar	644	25		148	0.07	0.07	0.96
3.	Meskel Square	Stadium	1000	78		560	0.28	0.28	0.83
		Estifanos	1350	95		505	0.25	0.25	0.85
		Piassa	837	27		91	0.05	0.05	0.97
		Saris	632	30		134	0.07	0.07	0.96
4.	Estifanos	Meskel Square	696	67		399	0.20	0.20	0.88
		Estifanos	1468	35	94	0.05	0.05	0.97	
		Bole	505	50	390	0.20	0.20	0.88	
		Haya Hulet	1120	58	215	0.11	0.11	0.94	

The estimated value is also presented in the Table 4-9 below;

Table 4-12: Saturation Flow rate

I.No	Name of Intersection	Approach Leg	Number of lanes, N	So (veh/h/ln)	s (veh/h/ln)
1.	Commerce	Mexico	3	1973	1242
		Leghar	3		1010
		Commerce	4		1304
2.	Leghar	Mexico	3		989
		Stadium	3		1460
		Piassa	4		1471
		Leghar	4		1329
3.	Meskel Square	Stadium	3		1001
		Estifanos	4		970
		Piassa	3		1157
		Saris	3		1115
4.	Estifanos	Meskel Square	5		1024
		Estifanos	4	1259	
		Bole	4	1080	
		22	3	1124	

4.1.2 Determine Proportion of vehicles arriving During Green

Control delay and queue size are highly dependent on Proportion of vehicles arriving during Green time. When Proportion Arriving during Green is high it results lower queue size and minimize control delay.

$$P = R_p \frac{g}{c} \quad \text{Equation 4.19}$$

Where;

P = proportion of all vehicles in movement arriving during green phase;

R_p= platoon ratio;

C = cycle length (s); and

g= effective green time for movement or lane group (sec)

In order to determine the quality of progression, six arrival types were defined by HCM together with its relation with platoon ratio.

When delay is estimated for future situations involving coordination, particularly in the analysis of alternatives, it is advisable to assume Arrival Type 4 as a base condition for coordinated lane groups (except left turns). Arrival Type 3 should be assumed for all uncoordinated lane groups. Movements made from exclusive left-turn lanes on protected phases are not usually provided with good progression. Thus, Arrival Type 3 is usually assumed for coordinated left turns. In addition as the study area is synchronized, intersection type 4 arrival is considered for estimation. Hence, the platoon ratio should be 1.33.

The g/C ratio for each lane group, the effective green time divided by the cycle length, is also computed and used to compute lane group capacity. Effective green times can be taken as equal to the actual green time plus the change-and-clearance interval minus the lost time for the movement.

$$g_i = G_i + Y_i - tL_i \quad \text{Equation 4.20}$$

Where,

g_i = effective green time for movement(s) i, s ;

G_i = actual green time for movement(s) i, s ;

Y_i = sum of yellow and all red intervals for movement(s) i, s , ($Y_i = y_i + a_{ri}$);

y_i = yellow interval for movement(s) i, s ;

a_{ri} = all-red interval for movement(s) i, s ; and

tL_i = total lost time for movement(s) i, s ($tL_i = l_1 + l_2, l_2 = y + ar - e$).

HCM recommended using start-up lost time (L_1) and e to be equal to 2.

Field condition shows that a value of $y=3s$ and $ar=2s$.

$$tL = l_1 + y - ar - e = 2 + 3 + 2 - 2 = 5s$$

$$Y_i = y_i + a_{ri} = 3 + 2 = 5s$$

Hence, the effective green time and actual green time for movements were equivalent and accordingly, the same was adopted during calculation.

Table 4-13: Proportion of all vehicles in movement arriving during green phase

I.No	Name of Intersection	Approach Leg	Rp	g/C	P
1.	Commerce	Mexico	1.33	0.77	1.03
		Leghar		0.64	0.85
		Commerce		0.13	0.18
2.	Leghar	Mexico		0.46	0.62
		Stadium		0.47	0.62
		Piassa		0.13	0.18
		Leghar		0.11	0.14
3.	Meskel Square	Stadium		0.41	0.55
		Estifanos		0.51	0.67
		Piassa		0.14	0.19
		Saris		0.16	0.21
4.	Estifanos	Meskel Square		0.36	0.47
		Estifanos	0.19	0.25	
		Bole	0.27	0.35	
		22	0.31	0.41	

4.1.3 V/c Estimation

The v/c ratio, or degree of saturation, is a principal output measure from the analysis of a signalized intersection. It is a measure of the sufficiency of available capacity to handle existing or projected demands. Obviously, cases in which V/C > 1.00 indicate a shortage of capacity to handle the demand (Roess, 2004). HCM sets single variable, X, to express v/c ratio and the same was adopted in this study. For a given lane group, X is determined as;

$$Xi = \frac{vi}{ci} = \frac{vi}{si \left(\frac{gl}{c}\right)} \quad \text{Equation 4.21}$$

Where,

- Xi = (v/c) = ratio for lane group i;
- vi = actual or projected demand flow rate for lane group i (veh/h);
- si = saturation flow rate for lane group i (veh/h);
- gi = effective green time for lane group i (s); and
- C = cycle length of the intersection (s).

The actual demand flow rate was given in Table: 3.2 above. Effective green is the time that may be used by vehicles at saturation flow rate and can be taken as equal to the actual green time plus the change-and-clearance interval minus the lost time for the movement and were calculated using the following formula;

$$g_i = G_i + Y_i - tL \quad \text{Equation 4.22}$$

Where,

g_i = effective green time for lane group i (sec);

G_i = allocated green time for lane group i (sec);

Y_i = change-and-clearance interval/ allocated yellow time for lane group i (sec); and

tL = lost time (sec).

The lost time is the sum of start-up lost time (11) and clearance lost time (12), HCM recommends 11 to be 2sec. However, 12 was estimated from yellow and all red interval, (i.e. $y+ar-e$ in which $e=2\text{sec}$).

The degree of saturation of the intersection is tabulated herein under;

Table 4-14: Degree of saturation of the intersection

I.No	Name of Intersection	Approach Leg	V, (Pc/h) (A)	g_i/C (B)	Si, (veh/h) (C)	Ci, (veh/h) (D=B*C)	Xi (E=A/D)
1.	Commerce	Mexico	1976	0.77	3725	2873	0.69
		Leghar	2108	0.64	3031	1935	1.09
		Commerce	672	0.13	5217	694	0.97
2.	Leghar	Mexico	1950	0.46	2968	1373	1.42
		Stadium	2030	0.47	4379	2050	0.99
		Piassa	668	0.13	5885	783	0.85
		Leghar	747	0.11	5314	565	1.32
3.	Meskel Square	Stadium	3217	0.41	3004	1246	2.58
		Estifanos	3966	0.51	3879	1960	2.02
		Piassa	902	0.14	3472	499	1.81
		Saris	1197	0.16	3344	534	2.24
4.	Estifanos	Meskel Square	3398	0.36	5120	1825	1.86
		Estifanos	1337	0.19	5036	938	1.43
		Bole	2065	0.27	4319	1149	1.80
		22	1022	0.31	3373	1040	0.98

4.1.4 Delay estimation

Delay in HCM is defined as the difference between the travel time actually experienced by the driver and the reference travel time that would result during ideal conditions. The ideal conditions includes but not limited to absence of traffic control, absence of any incidents, freeway road. Delay estimation includes movement at slower speed and stops in intersection due to queue at upstream. The delay measures include the uniform control delay, incremental delay

and initial queue delay.

Total delay can be computed as;

$$d = d1(PF) + d2 + d3 \quad \text{Equation 4.23}$$

Where;

d = control delay per vehicle (s/veh);

$d1$ = uniform control delay assuming uniform arrivals (s/veh);

PF = uniform delay progression adjustment factor, which accounts for effects of signal progression;

$d2$ = incremental delay to account for effect of random arrivals and oversaturation queues, adjusted for duration of analysis period and type of signal control; this delay component assumes that there is no initial queue for lane group at start of analysis period (s/veh); and

$d3$ = initial queue delay, which accounts for delay to all vehicles in analysis period due to initial queue at start of analysis period (s/veh)

Progression adjustment factor is the proportion of vehicles that join the queue when green phase is displayed. It used to adjustment vehicles arrival on the green time. signal progression is mainly affect the uniform delay. When delay is estimated for future situations involving coordination, particularly in the analysis of alternatives, it is advisable to assume Arrival Type 4 as a base condition (HCM).

$$PF = (1 - P)fpA/(1 - \frac{g}{c}) \quad \text{Equation 4.24}$$

Where,

PF = uniform delay progression adjustment factor;

$P = \min(Rp * \frac{g}{c}, 1)$; and

fPA = Progression adjustment factor 1.5 for Arrival type 4.

4.1.4.1 Uniform delay

Uniform delay is computed by assuming uniform arrivals, stable flow and no initial queue. If the degree of saturation of the intersection is more than one ($x > 1$) the oversaturation portion is not considered for calculation.

$$d1 = \frac{0.5c(1-\frac{g}{c})^2}{1 - [\min(1, X)\frac{g}{c}]} \quad \text{Equation 4.25}$$

where,

d_1 = uniform control delay assuming uniform arrivals (s/veh);

C = cycle length (s); cycle length used in pretimed signal control, or average cycle length for actuated control for signal timing estimation of actuated control parameters);

g = effective green time for lane group (s); green time used in pretimed signal control, or average lane group effective green time for actuated control for signal timing estimation of actuated control parameters); and

X = v/c ratio or degree of saturation for lane group.

Table 4-15: Uniform control delay

I.No	Name of Intersection	Approach Leg	X_i	g_i/C	C (s)	d_1 , (s/veh)
1.	Commerce	Mexico	0.69	0.77	188	10.47
		Leghar	1.09	0.64		34.00
		Commerce	0.97	0.13		81.10
2.	Leghar	Mexico	1.42	0.46	188	50.50
		Stadium	0.99	0.47		49.57
		Piassa	0.85	0.13		79.70
		Leghar	1.32	0.11		84.00
3.	Meskel Square	Stadium	2.58	0.41	188	55.00
		Estifanos	2.02	0.51		46.50
		Piassa	1.81	0.14		80.50
		Saris	2.24	0.16		79.00
4.	Estifanos	Meskel Square	1.86	0.36	188	60.50
		Estifanos	1.43	0.19		76.50
		Bole	1.80	0.27		69.00
		Haya Hulet	0.98	0.31		64.49

4.1.4.2 Incremental Delay

It is a delay arises owing to non-uniform arrivals, temporary cycle failures as well as sustained period of oversaturation. This delay component assumes there is no initial queue delay and adjusted for duration of analysis period and type of signal control.

$$d_2 = 900T [(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}] \quad \text{Equation 4.26}$$

Where;

d_2 = incremental delay, s;

T = duration of analysis period (h), $T=0.25h$ (15min) is used for this study;

k = incremental delay factor;

I = upstream filtering/metering adjustment factor;

c = lane group capacity (veh/h); and

X = lane group v/c ratio or degree of saturation.

Incremental delay factor (k) is dependent on controller settings; for pre-timed signals, a value of $k=0.5$ is used. On the other hand, for actuated signals, smaller value is adopted for $X<1$, and $k=0.5$ is used when $X=1$. For this study $k=0.5$ is used as all signals are pre-timed signals.

Upstream filtering/metering adjustment factor (I) is used for the effect of filtered arrival from upstream intersections. An I value of 1.0 is used for isolated intersection (1.6km away from the upstream signalized intersection).For non-isolated intersections the value of I is depend on weighted V/C ratio (X_u) of the upstream movements.

For $X_u \geq 1.0$, $I=0.09$,

$X_u < 1.0$, $I = 1 - 0.91X_u^{2.68}$ is used

Where,

$$X_u = \frac{\sum_{k=0}^n V_i * X_i}{\sum_{k=0}^n V_i}, \quad V_i = \text{upstream movement contributed to the subject lane group}$$

Table 4-16: Upstream factor

I.No	Name of Intersection	Approach Leg	Upstream effect	V_i	$x_i * v_i$	Upstream ($X_i * V_i$)	$(x_i * v_i) / v_i$	I
1.	Commerce	Mexico	No	1976	1359	0	0	1
		Leghar	all Leghar approach	2108	2297	6336	1	0.09
		Commerce	No	672	650	0	0	1
2.	Leghar	Mexico	all Mexico approach	1950	2770	4306	1	0.09
		Stadium	all M/s approach	2030	2009	20646	2	0.09
		Piassa	No	668	569	0	0	1
		Leghar	No	747	988	0	0	1

I.No	Name of Intersection	Approach Leg	Upstream effect	Vi	xi*vi	Upstream (Xi*Vi)	(xi*vi)/vi	I
3.	Meskel Square	Stadium	all Leghar approach	3217	8305	6336	1	0.09
		Estifanos	all Estifanos approach	3966	8026	12951	2	0.09
		Piassa	No	902	1631	0	0	1
		Saris	No	1197	2684	0	0	1
4.	Estifanos	Meskel Square	all Meskel Square approach	3398	6329	4306	2	0.09
		Estifanos	No	1337	1906	0	0	1
		Bole	No	2065	3713	0	0	1
		Haya Hulet	No	1022	1004	0	0	1

Having the above values, the incremental delays were calculated as follows.

Table 4-17: Incremental Delay

I.No	Name of Intersection	Approach Leg	Xi	Ci (veh/h)	I	d2, (s/veh)
1.	Commerce	Mexico	0.69	2873	1	1.4
		Leghar	1.09	1935	0.09	41.3
		Commerce	0.97	694	1	27.2
2.	Leghar	Mexico	1.42	1373	0.09	189.5
		Stadium	0.99	2050	0.09	4.1
		Piassa	0.85	783	1	11.4
		Leghar	1.32	565	1	156.8
3.	Meskel Square	Stadium	2.58	1246	0.09	711.8
		Estifanos	2.02	1960	0.09	460.8
		Piassa	1.81	499	1	371.7
		Saris	2.24	534	1	565.3
4.	Estifanos	Meskel Square	1.86	1825	0.09	388.2
		Estifanos	1.43	938	1	197.8
		Bole	1.80	1149	1	362.5
		Haya Hulet	0.98	1040	1	23.9

4.1.4.3 Initial queue delay

When a residual queue from a previous time period causes an initial queue to occur at the start of the analysis period (T), additional delay is experienced by vehicles arriving in the period since

the initial queue must first clear the intersection.

$$d1 = \frac{1800 Qb(1+U)t}{cT} \quad \text{Equation 4.27}$$

Where,

Qb= initial queue at the start of period T (veh);

c = adjusted lane group capacity (veh/h);

T = duration of analysis period (h);

t = duration of unmet demand in T (h); and

u = delay parameter.

$$t = 0 \text{ if } Qb = 0, \text{ else } t = \min\left\{T, \frac{Qb}{c[1-\min(1,X)]}\right\}$$

$$u = 0 \text{ if } t < T, \text{ else } u = 1 - \frac{cT}{Qb[1-\min(1,X)]}$$

All initial queues are estimated during field observation, a value of Qb=0 is used for lane groups that serve flow at green time during the start of the analysis period.

Table 4-18: Initial queue delay

I.No	Name of Intersection	Approach Leg	Qb (veh)	Ci (veh/h)	t (se)	u	d3, (s/veh)
1.	Commerce	Mexico	35	2873	0.04	0	3.4
		Leghar	0	1935	0	0	0
		Commerce	0	694	0	0	0
2.	Leghar	Mexico	0	1373	0	0	0
		Stadium	0	2050	0	0	0
		Piassa	0	783	0	0	0
		Leghar	0	565	0	0	0
3.	Meskel Square	Stadium	0	1246	0	0	0
		Estifanos	0	1960	0	0	0
		Piassa	0	499	0	0	0
		Saris	0	534	0	0	0
4.	Estifanos	Meskel Square	0	1825	0	0	0
		Estifanos	0	938	0	0	0
		Bole	0	1149	0	0	0
		Haya Hulet	0	1040	0.25	0.69	0

The aggregated delay together with LOS is tabulated herein under Table 4-16 and 4-17.

Table 4-19: Total Delay

I.No	Name of Intersection	Approach Leg	d1	d2	d3	PF	d (d1*PF+d2+d3) (s/veh)
1.	Commerce	Mexico	10.47	1.4	3.4	0.00	4.79
		Leghar	34.00	41.3	0	0.48	57.65
		Commerce	81.10	27.2	0	1.00	108.28
2.	Leghar	Mexico	50.50	189.5	0	0.82	231.02
		Stadium	49.57	4.1	0	0.82	44.56
		Piassa	79.70	11.4	0	1.00	91.09
		Leghar	84.00	156.8	0	1.00	240.81
3.	Meskel Square	Stadium	55.00	711.8	0	0.88	760.26
		Estifanos	46.50	460.8	0	0.76	496.20
		Piassa	80.50	371.7	0	1.00	452.16
		Saris	79.00	565.3	0	1.00	644.31
4.	Estifanos	Meskel Square	60.50	388.2	0	0.94	445.10
		Estifanos	76.50	197.8	0	1.00	274.32
		Bole	69.00	362.5	0	1.00	431.48
		22	64.49	23.9	0	0.98	87.16

Table 4-20: Level of service of the study area

I. No.	Name of Intersection	Intersection Delay	LOS
1.	Commerce	42.59	D
2.	Leghar	144.24	F
3.	Meskel Square	606.57	F
4.	Estifanos	365.41	F

4.2 Traffic Signal Optimization

4.2.1 Components of Signal operation

An intersection may be controlled independently (isolated operation) or have the ability to synchronize to multiple intersections in a coordinated operation. Isolated and coordinated intersections can operate either in pre-timed (fixed) or actuated mode, where detectors will monitor traffic demand. At pre-timed operation, the total green time allocated to a phase will always have a preset time, regardless of demand.

During coordinated operation, multiple signalized intersections are synchronized to enhance the progression of vehicles on one or more directional movements in a system. Pre-timed coordination provides better progression from a driver standpoint, but higher delay is also experienced. Actuated coordination is more efficient, but progression is not consistently achieved (TDOT Traffic Design Manual, 2016).

Traffic Signal Optimization used to provide smooth flow of traffic by reducing travel times, stops, delays and vehicle operating costs along the streets. Intersections spaced within 0.5mile of each other will benefit from coordination, especially during periods of large traffic demand when platoons of vehicles may form, besides if they spaced one mile or more apart may benefit from coordination if there is minimal access turbulence on segments. During traffic signal coordination, system cycle length, splits and offsets should have prior considerations (Henry, 2005).

System Cycle Length

In coordination, all intersections included in a system must have a common cycle length in order to maintain a consistent time-based relationship between intersections. Software use a given set of inputs (including the range of preferred cycle lengths) and estimated performance measures to determine an optimal cycle length. System cycle lengths are mainly selected to address operations at a critical (or highest volume) intersection in a group of coordinated signalized Intersections (Henry, 2005).

Synchro starts with a short cycle length and optimizes the splits for that cycle length. If the splits for each phase are not able to clear the critical percentile traffic, Synchro will try a higher cycle length until the critical percentile traffic is cleared. For cycle lengths above 90, the v/c ratios simply need to be less than 1. If no acceptable cycle length is found, the cycle length is set to the cycle length with the lowest performance index. The PI is calculated as follows (Trafficware, 2014).

$$PI = [(D * 1) + (St * 10)] / 3600 \quad \text{Equation 4.28}$$

Where,

PI = Performance Index;

D = Total Delay (s); and

St = Vehicle Stops.

For over capacity intersections, the standard optimization procedure is to try successively longer and longer cycle lengths. Longer cycle lengths add additional capacity because a lower proportion of the cycle is used by yellow and total lost time. When cycle lengths exceed 120 seconds, however, further increases to cycle length start to have less impact on intersection capacity, while incremental delay continues to increase. Congestion lasting for 1 hour or longer tends to favor longer cycle lengths than spot congestion for 15 minutes. Changing to a longer analysis period may cause the cycle length optimization to recommend longer cycle lengths.

Splits

Splits are the portion of the system cycle allocated to each phase, including the green interval, yellow change, and red clearance intervals. Splits are selected based on individual intersection phasing and expected demand. Therefore, splits may vary from intersection to intersection. During implementing splits on a traffic signal controller, the sum of the phase splits must be equal to or less than the programmed cycle length (Henry, 2005).

Offsets

Offset is the time that elapses between the master clock and the offset reference point at each local intersection included in the system. Therefore, each signalized intersection will also have a relative offset to each other. It is through this association that the coordinated phase is aligned between intersections to create the relationship for synchronized movements. Offsets should be chosen based on the actual or desired travel speed between intersections, distance between signalized intersections and traffic volumes (Henry, 2005).

4.2.2 The simulation and Evaluation

System analysis software should be used for arterials and networks of multiple signalized and/or stop/yield-controlled intersections. Simulation software should be utilized to aid in determining storage lengths, verifying geometry and lane continuity, and to identify overall network operations. However, according to the research conducted by North California department of

Transportation it was noted that all system analysis software is not effective on all types of intersections. For multiple intersections, isolated signalized and isolated unsignalized intersections Synchro, for traffic simulation SimTraffic, VISSIM and CORSIM and also for Roundabouts SIDRA and VISSIM are the preferred software (NCDOT, 2015).

Wisconsin Department of Transportation, 2017, advised the traffic Engineers to use Synchro Version 9 software in order to implement Signal optimization, and/or simulation of a given traffic analysis for an existing or proposed signalized intersection mentioning that the software is developed based on HCM methodology and preferable for signalized intersection.

4.2.3 Modeling in Synchro

Synchro studio is a software package for modeling traffic system, analyzing and optimizing timing plans.

The 1.98 km section of part of Ras Mekonnen Street and Jomo Kenyatta Street of Addis Ababa was modeled with Synchro version 9 software based on the data obtained from traffic survey as discussed in Chapter- 3 of this paper. Figure 4-3 below shows a snapshot of the entire network in Synchro starting from Mexico Intersection being west end of the corridor at the left end of picture towards Estifanos Intersection.

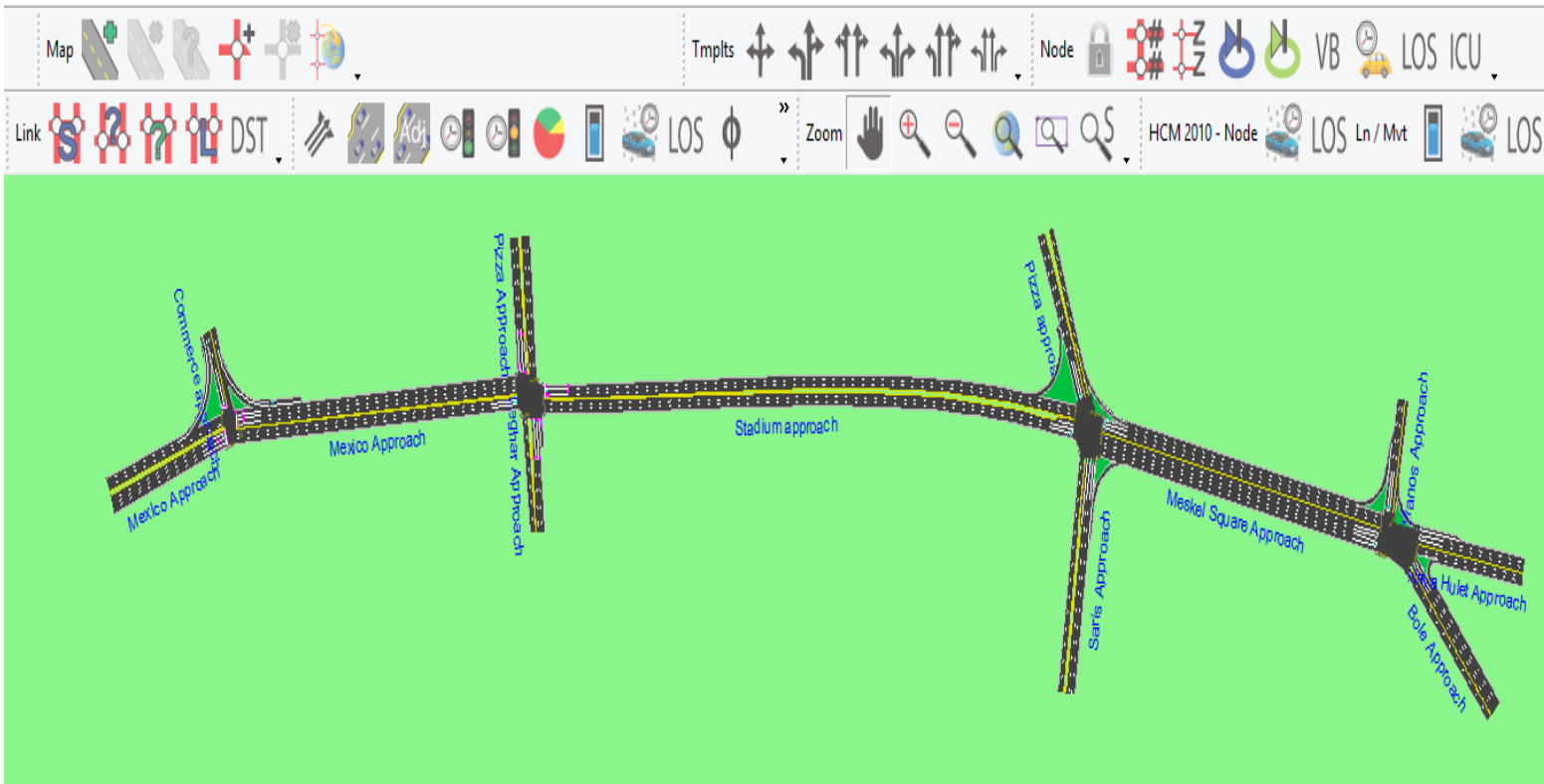


Figure 4-3: Network Modeling

4.2.3.1 Input parameters

The data requirement of the model consists of geometric, traffic and signalization information. The geometric conditions includes: area type, number of lanes, lane width, grade, existence of exclusive LT or RT lanes, length of storage lane etc.. Traffic conditions also include Demand volume by movement, Base saturation flow rate, Peak-hour factor, Percent heavy vehicles, Approach pedestrian flow rate, Local buses stopping at intersection, Parking activity, approach speed, etc..

In addition, the input parameters of Signalization conditions include: Cycle length, Green time, Yellow-plus-all-red change-and-clearance interval, Actuated or pre-timed operation, etc. The analysis period is set on the software as a default value of 15 minutes. The data used for traffic optimization is almost similar to the data used for performance evaluation of the individual signalized intersections except the traffic volume which traverses along the road. The Traffic data used for optimization should be collected at a time that serves by the entire network at same time. For example, if the aggregate peak traffic data of the entire network is at morning the

synchronization should be done for that particular period.

a) Traffic data

From the collected traffic data, it is noted that once at a time, the entire route serves peak traffic volume during 11/10/17 night time which is 25,948 PCU per hour. The Collected traffic data within the said peak period is summarized in the Table 4.18 herein under;

Table 4-21: Traffic data for Synchro

I.No	Name of Intersection	Approach Leg	V (Pc/h) (A)	PHF (B)	% of HV	Ideal Flow (Pc/hr)	Vp (Adjusted flow rate) (pc/h) (C=A/B)	Maximum Intersection Flow (Pc/hr)
1.	Commerce	Mexico	1761	0.95	1.6	1973	1849	4704
		Leghar	2001	0.89	3.0		2254	
		Commerce	555	0.95	1.3		675	
2.	Leghar	Mexico	1823	0.95	2.3		1928	4986
		Stadium	1735	0.92	17.6		1894	
		Piassa	670	0.85	3.6		790	
		Leghar	609	0.88	3.1		690	
3.	Meskel Square	Stadium	3218	0.86	5.0		3725	8962
		Estifanos	3967	0.92	4.4		4294	
		Piassa	907	0.93	1.4		966	
		Saris	1197	0.96	2.4		1244	
4.	Estifanos	Meskel Square	3398	0.91	4.5		3721	7733
		Estifanos	1337	0.92	2.0	1446		
		Bole	2065	0.96	1.4	2150		
		22	1022	0.92	3.1	1103		

Traffic data used in the analysis were labeled according to the direction of flow. The software has calculated the travel time using the link speed and link distance.


b) Geometric data

Each coordinate of nodes of intersections were taken from Google earth and feed to software. Accordingly, using those coordinate points the software calculates the link distances. Other parameters like lane width, grades, and storage lanes were similar to input data as presented in Chapter-3 of this report.

Table 4.19 below shows the geometric and traffic data of the Estifanos Intersection.

Table 4-22: Geometric and traffic data of Estifanos Intersection

Lanes and Geometrics
 1: Bole Approach & Meskel Square Approach/Haya Hulet Approach & Estifanos Approach



Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	SBL2	SBL	SBR	NWL	NWR	NWR2
Lane Configurations												
Ideal Flow (vphpl)	1973	1973	1973	1973	1973	1973	1973	1973	1973	1973	1973	1973
Lane Width (m)	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
Grade (%)		1%			1%			1%		1%		
Storage Length (m)	0.0		0.0	0.0		50.0		0.0	50.0	0.0	0.0	
Storage Lanes	1		2	1		1		3	1	2	2	
Taper Length (m)	2.5			2.5				2.5		2.5		
Right Turn on Red			Yes			Yes			Yes			Yes
Link Speed (k/h)		50			50			50		50		
Link Distance (m)		385.1			155.5			101.1		174.3		
Travel Time (s)		27.7			11.2			7.3		12.5		

Intersection Summary
 Area Type: CBD
 Description: Estifanos Intersection

c) Timing data

The route corridor signalized intersection is being served by pre-timed control type having cycle length of 188 sec. The already existed traffic timing data is fed to software with respect to leading and/or lagging time. The following picture shows the timing plan of Estifanos Intersection.

1: Bole Approach & Meskel Square Approach/Haya Hulet Approach & Estifanos Approach 5/23/2018

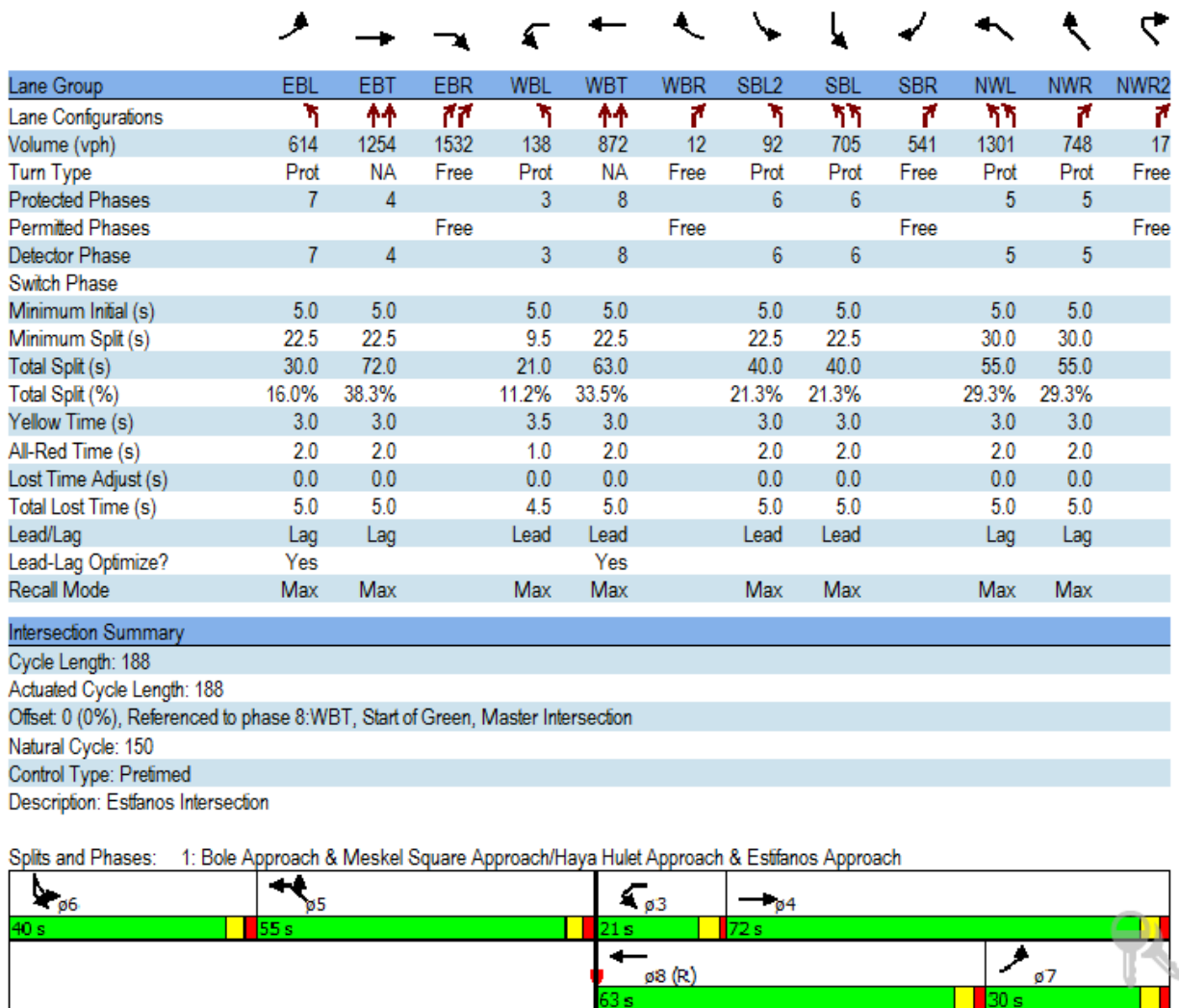


Figure 4-4: Timing data for Estifanos Intersection

All input data used for analysis is annexed in Appendix-2 and Appendix-4 of this report.

4.2.4 Intersections capacity before optimization

The software try to calculate the capacity of intersection in terms of delay; Control delay and Queue delay at 50th and 90th percentile, analyze Level of Service and estimate amount of fuel used and amount of emissions.

Analysis result shows that all approach of the entire network except Mexico Intersection serves the travel demand with poor level of service (i.e. LOS of F). West bound of Mexico Intersection

(Leghar Approach) have LOS C that makes the approach the most uncongested section of the network. Other approaches of Mexico Intersection; Mexico and Commerce approaches serve a traffic demand with LOS of E and LOS of D respectively.

In addition, fuel consumption and Emission of gases like CO, NOx and VOC are at significant amount. Details of analysis result are attached in Appendix-5 of the report.

Figure 4.5 below shows the time space diagram of the Estifanos Intersection

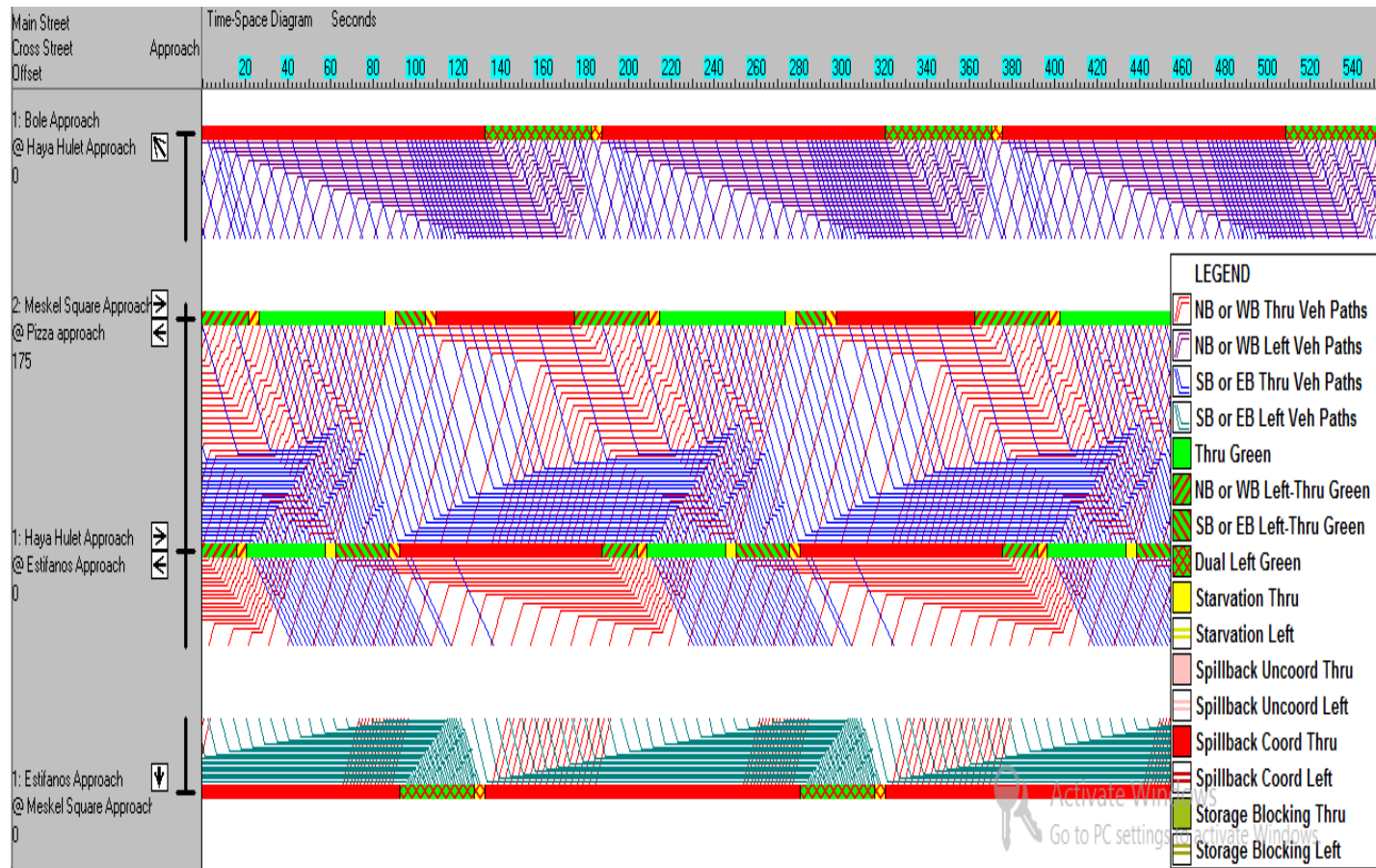


Figure 4-5: Time Space diagram of Estifanos Intersection

4.2.5 Intersections capacity after optimization

Signal optimization using Synchro software was accomplished by applying weighting factors (by phase) at individual intersection or throughout the network. More weighting factors and the green time were allocated to that particular phase. Optimization of signal was carried out for

network and intersection cycle length, network and intersection phase and intersection splits.

During optimization of network cycle length, minimum 50sec and maximum 360sec cycle length with fully coordinated network is set by allowing zero to uncoordinated intersection and found that a cycle length of 270sec for Estifanos Intersection and 230sec for all other intersections are preferable. Hence, by allocating a cycle length of 270sec for Estifanos Intersection and 230sec for all other intersections, the performance of the entire network will be improved and serve the traffic at maximum service rate regarding to reduction of effectiveness parameters like delay, fuel consumption, travel time, emission etc. without manipulating the existing geometry of network.

The following pictures show the recommended timing plan of the network after optimization.

Splits and Phases: 1: Bole Approach & Meskel Square Approach/Haya Hulet Approach & Estifanos Approach

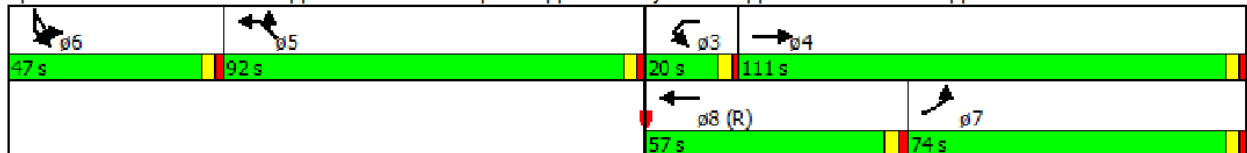


Figure 4-6: Estifanos Intersection Timing Plan

Splits and Phases: 2: Saris Approach/Pizza approach & Stadium approach/Meskel Square Approach

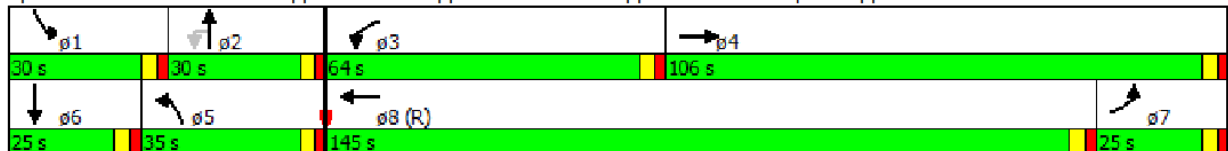


Figure 4-7: Meskel Square Intersection Timing Plan

Splits and Phases: 3: Leghar Approach/Pizza Approach & Mexico Approach/Stadium approach

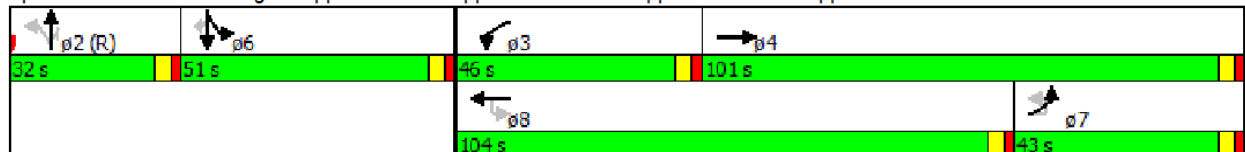


Figure 4-8: Leghar Intersection Timing Plan

Splits and Phases: 11: Mexico Approach & Commerce approach

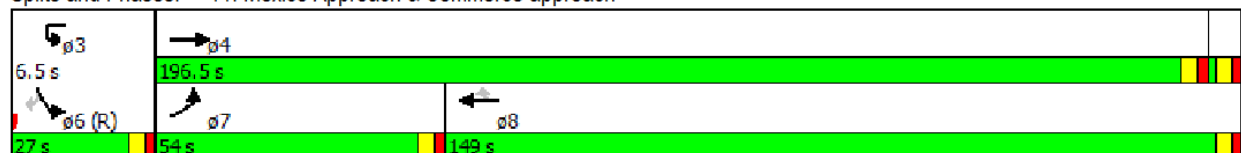


Figure 4-9: Mexico Intersection Timing Plan

In general the comparison of effectiveness of entire network in terms of performance parameters before and after optimization are extracted from software and summarized in the Table 4-20 below;

Table 4-23: Measure of Effectiveness of Total network

Detailed Measure of Effectiveness of Total network		
	Before Optimization @180sec cycle length	After Optimization @250sec cycle length
Number of Intersections	4	4
Number of delay/veh (s/v)	229	198
Queue delay/ veh (s/v)	0	0
Total delay/ veh (s/v)	229	198
Total delay (hr)	1713	1481
Stops/veh	0.62	0.57
Number of stops	16555	15193
Fuel consumed (l)	5992	5319
CO emissions (kg)	111.46	98.94
Nox emission (kg)	21.51	19.10
VOC emission (kg)	25.71	22.82
Performance index	1759.3	1523.7

From Table 4-20 above, all effectiveness parameters were improved by changing the cycle length from 188sec to 270sec for Estifanos Intersection and 230sec for all other intersections and phase sequences together with splits. More than 550 vehicles will be relieved from stopping at intersection and also extra 654 litter fuel consumed by the vehicles will also be saved owing to improvement of signal performance.

Figure 4-10 below to shows the difference between the effectiveness of network before and after optimization.

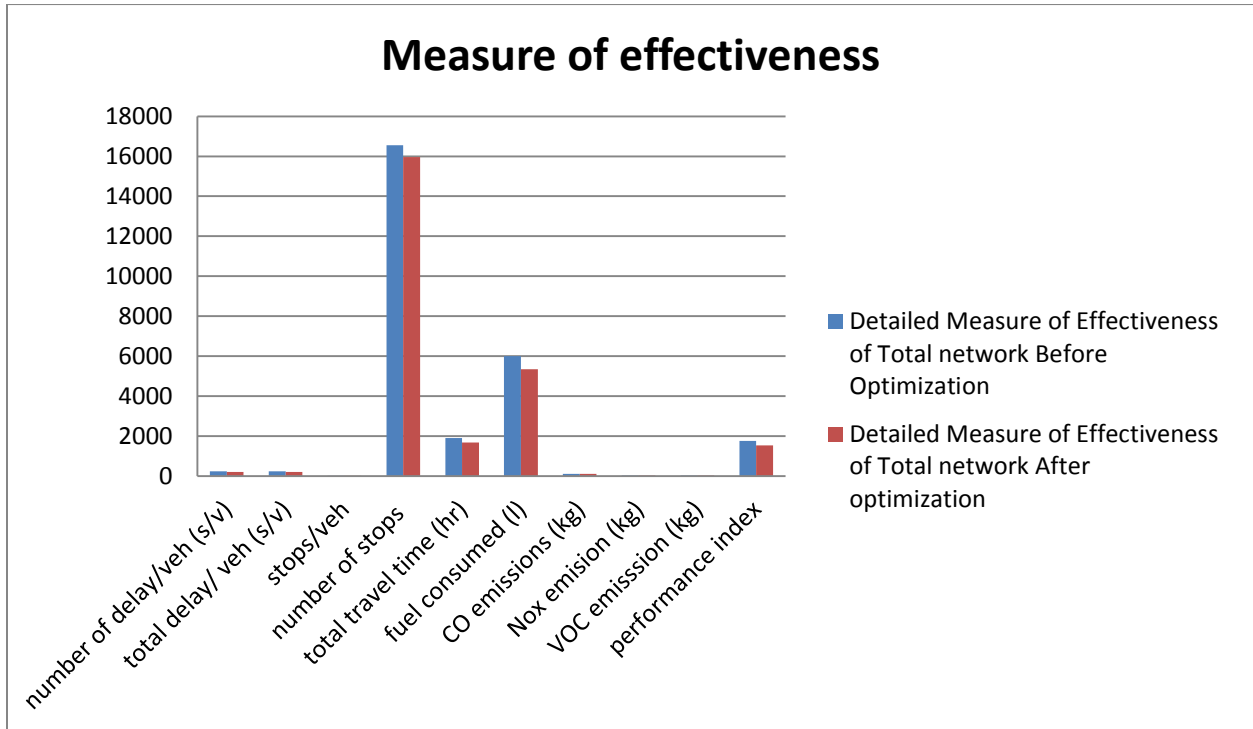


Figure 4-10: Measure of Effectiveness of Total network

Though, effectiveness parameters were improved due to optimization of cycle length, offsets and splits, the level of service of the network is not changed from LOS F to better service. This is happen due to the reason that the Arrival rate is too much to serve by the intersection. However, the Mexico Intersection service rate is improved from LOS D to LOS C.

To overcome the imbalance between arrival and service rate of signalized intersections on the network, it is recommended to look other options like serving the traffic by overpass, adding service lanes.

4.3 Congestion Cost Estimation

Traffic congestion give rise to various resource costs that can be expressed in monetary terms: time costs of delays, health costs caused by air pollution, productivity losses, life lost in traffic accidents.

This section contains a review of past studies that examined the effect of traffic congestion on the national economy and updates the findings for Ethiopian condition using current data. Most estimates of cost of congestion mainly focused on the travel time lost, excess fuel cost and environmental costs. Though congestion has many impacts on social, health and environment, in these study only social costs like Cost of Travel Time Lost and excess full are estimate.

As discussed in chapter three, sample survey were prepared and distributed to road users to identify their attitudes towards causes of congestion and also effects of congestion on their output. The major causes of congestion are pedestrian interruption, existence of high volume of public transport and long vehicles, existence of vehicles emerges from access road, inappropriate allocation of signal timing and low capacity of travel lane. Figure 4-11 below illustrates factors which are attributed to cause congestion from the respondents' perspective.

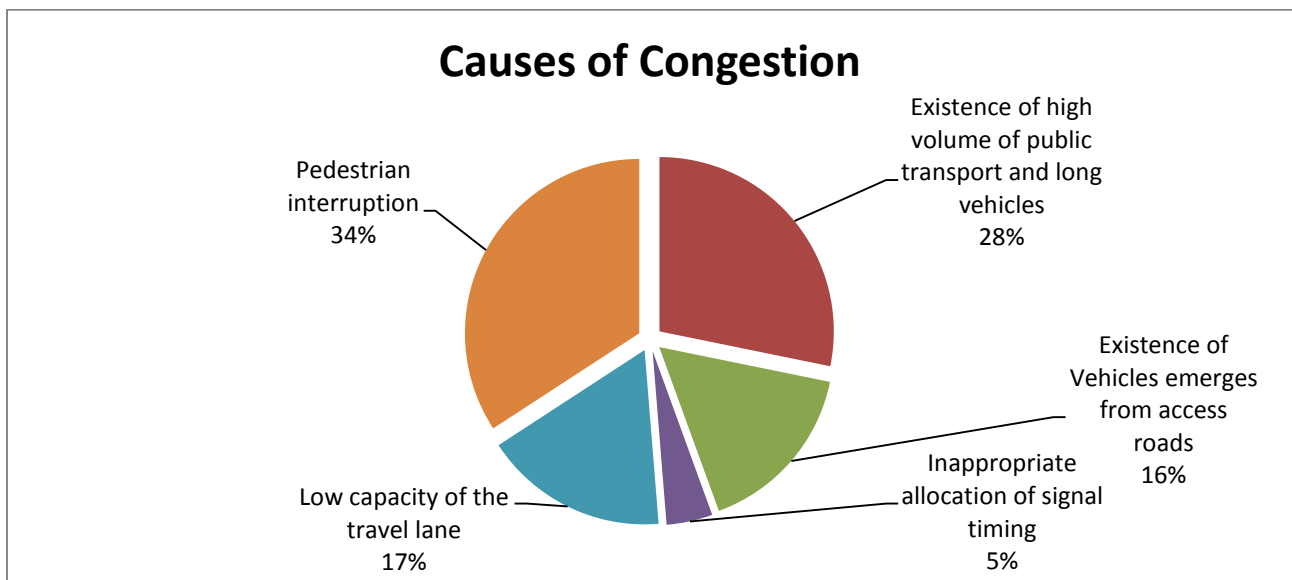


Figure 4-11: Causes of congestion

4.3.1 Cost of Travel Time Lost

One of the significant effects of traffic congestion is delaying the road users travel time which indirectly affects their productivity. In order to determine travel time lost the required inputs are explained below.

4.3.1.1 Hourly wage of road users

The hourly wage of road users is determined from the monthly income of person. From the distributed questionnaires' it was found that ETB: 34.33 is the average hourly wages of the road users. However, from the statistical data obtained from the Ethiopian Statistics Agency, the average hourly income of Ethiopians is ETB30, Hence considering the fact that the research is limited to some groups of road users it is better to use the countries data for calculating the same.

4.3.1.2 Average vehicle occupancy

The average vehicle occupancy is the number of persons in a vehicle including driver and passengers who did or did not complete the trip. Tadele (2017), on his research titled "*Analysis of traffic congestion and its economic cost in Addis Ababa a case study of Meskel Square to Kality Interchange*" has estimated the average number of vehicle occupancy on Meskel Square to Kality interchange segment. Based on his research, he has found that the average vehicle occupancy in the city of Addis Ababa were 13.7 passenger/Veh. In this paper an average vehicle occupancy value of 13.7 passenger/Veh were also used for estimating the congestion cost as the segment studied by Tadele (2017), is one of the approach legs for Meskel Square Intersection and due to similar nature of the vehicles traversing the routes.

4.3.1.3 Average Travel time

Due to congestion, travel time becomes variable and more uncertain which leads the travelers too late to arrive at their destinations. Travel time lost is the difference in cost between average travel at current congested level and totally uncongested conditions.

From the distributed survey data it was noted that there was an average of two (2) minutes time gap per intersection between the congested and uncongested conditions. In addition, the value of time lost by the road users may be working time or nonworking time to them. However, the data from survey is not accurate estimation as it is subjected to road users' opinion. Hence, further

study was carried out to get reasonable assumption.

Abaje (2013) and youn-soo Kang (2000), at their research and Prof. Tom.V.Mathew at his lecture note in Transportation System Engineering have all specified how to estimate the average waiting time of vehicles in the system as well as on the queue. In accordance to those dissertations the average waiting time of vehicles in the system mainly depend on traffic intensity, which is analogous to vehicle to capacity ratio (Agajie, 2015), the following formula was developed.

$$\rho = \frac{\lambda}{\mu} \quad \text{Equation 4.29}$$

Where;

ρ = traffic intensity;

λ = mean vehicle arrival rate; and

μ = mean vehicle service rate.

The main thing is determining of average amount of time a vehicle spends in a queue to use the service facility and the average time taken a vehicle to be served or leave the system. Once, traffic intensity is known the average waiting time of vehicles in the system is determined from the following relations, as provided by youn-soo Kang (2000);

$$L = \frac{\rho}{1-\rho} \quad \text{Equation 4.30}$$

$$Lq = \frac{\rho^2}{1-\rho} \quad \text{Equation 4.31}$$

$$W = \frac{L}{\lambda} \quad \text{Equation 4.32}$$

$$Wq = \frac{Lq}{\lambda} \quad \text{Equation 4.33}$$

Where;

L = Average number of vehicles in the system;

Lq= Average number of vehicles in the queue;

W= Expected waiting time in the system, sec; and

Wq= waiting time in the queue, sec.

Table 4-24: Average Travel time

Intersection Name	Approach	Arrival rate, μ	Service rate, λ	Traffic Intensity, ρ	No of Vehicles in the System, L	No of Vehicles in the Queue, Lq	Waiting time per Intersection, W in Sec
Mexico	Bunanashay	2873	1976	0.69	3	2	4.01
	Leghar	1935	2108	1.09	13	14	20.76
	Commerce	694	672	0.97	31	30	162.19
Leghar	Mexico	1373	1950	1.42	4	5	6.24
	Stadium	2050	2030	0.99	100	99	176.95
	Pizza	783	668	0.85	6	5	31.30
	Leghar	565	747	1.32	5	6	19.80
Meskel Square	Stadium	1246	3217	2.58	2	5	1.83
	Estifanos	1960	3966	2.02	2	4	1.79
	Pizza	499	902	1.81	3	5	8.93
	Saris	534	1197	2.24	2	5	5.43
Estifanos	Meskel Square	1825	3398	1.86	3	5	2.29
	Estifanos	938	1337	1.43	4	5	9.02
	Bole	1149	2065	1.80	3	5	3.93
	Haya Hulet	1040	1022	0.98	55	55	193.77
<i>Average waiting time per intersection (sec)</i>							43.2

From Table 4-21 above, it is inferred that a vehicle is obliged to wait 43.2sec per intersection and around 648.2sec in the system before leaving the segment intersections.

4.3.1.4 Number of vehicles waiting in the system

The starting point for computing the number of vehicle stops is the idealized concept of intersection behavior used in queuing theory. Specifically, vehicles are typically considered as being stored in a vertical queue at the downstream end of an approach link as shown in the time space diagram. When traffic demand is under saturated, vehicles arrive during the green period proceeds through the intersection without stopping besides correspondingly when Vehicle arrive during the red period, decelerates, and comes to a complete stop. When the signal turns green, the vehicle accelerates instantaneously to its cruising speed and leaves the intersection (Kang,

2000).

The idealized approximation differs from what happens in the field in a number of aspects. First, vehicles that queue upstream an intersection consume space and consequently vehicles arriving at the intersection should encounter the queue earlier than is predicted using queuing theory. Second, vehicles do not accelerate instantaneously; consequently vehicles experience numerous partial stops in many cases as opposed to complete stops. Alternatively, the queuing analysis considers only complete vehicle stops.

Youn-Soo Kang (2000), in his research also used the following equation to determine how to estimate the number of stopped vehicles in the queue assuming that the number of vehicle stops is computed as all vehicle arrivals when the traffic signal is red or when a queue exists at the approach stop line.

$$Ns = s * \frac{r}{c(s-q)} \quad \text{Equation 4.34}$$

Where:

Ns= number of stops per vehicle (stops/vehicle);

s = saturation flow rate (vehicles/sec);

C = cycle length (sec);

q = arrival flow rate (vehicles/sec); and

r = red interval (sec).

The number of vehicles waiting in the system is the maximum number of vehicles queued to the backward extent during the analysis period. HCM 2010 designates this parameter as a back of queue and estimate back of queue through the following formula is used.

$$Q = Q1 + Q2 + Q3 \quad \text{Equation 4.35}$$

Where,

Q= back of queue (veh/h);

$Q_1 = Nf$, number of fully stopped vehicles caused by phase sequence;

$Q_2 = \frac{CA}{3600 * N} * d_2$, due to oversaturation during analysis period;

Where,

CA = Average capacity (veh/h);

N = number of lanes in lane group;

d_2 = incremental delay (sec/veh);

$Q_3 = \frac{1}{NT} (tA * \frac{Q_b + Q_e - Q_{eo}}{2})$ due to initial queue;

tA= duration of unmet demand;

T = Analysis period;

Q_b = initial queue;

Q_e = queue at end of analysis period; and

Q_{eo} =queue at end of analysis period when $v \geq CA$ and $Q_b=0$

In addition, Synchro also developed two formulas to estimate back of queue depending on volume to capacity ratio.

If the volume to capacity ratio (v/c) is less than 1 then;

$$Q = \frac{v}{3600} * (R - 6) * \left[1 + \frac{1}{\frac{s}{v} - 1} \right] * \frac{L}{n * f * LU} \quad \text{Equation 4.36}$$

Where, R = Red time (sec);

s = Saturation Flow Rate (vph);

v = Arrival Rate (vph);

L = Length of vehicles, including space in between (ft);

n = Number of Lanes; and

fLU = Lane Utilization Factor.

If the volume to capacity ratio (v/c) exceeds 1, the queue length is theoretically infinite and accordingly the following equation was developed by Trafficware (2014), considering queue length as maximum after two cycles.

$$Q1 = v * (C - 6) + (v - s * \left(\frac{g}{c}\right)) * \frac{c}{3600} \quad \text{Equation 4.37}$$

The queue length along the study area is too long due to the reason that the arrival traffic volume exceeds the capacity of intersection.

From the above scenarios, estimating the number of vehicles stops using Synchro is found to be more acceptable as storage lengths and offsets between intersections are assumed. Hence, from Synchro simulation, the following data were obtained and the same were also be used for estimation of congestion costs.

Table 4-25: Total Stops Veh/h

I.No	Name of Intersection	Approach Leg	LOS	Stops (Vph)
1.	Mexico	Mexico	E	867
		Leghar	C	1066
		Commerce	D	413
2.	Leghar	Mexico	F	1407
		Stadium	F	1486
		Piassa	F	385
		Leghar	F	446
3.	Meskel Square	Stadium	F	1718
		Estifanos	F	2062
		Piassa	F	755
		Saris	F	906
4.	Estifanos	Meskel Square	F	1401
		Estifanos	F	633
		Bole	F	1262
		Haya Hulet	F	921
Total Stops veh/hr				15728

The parameters used to calculate cost of travel time lost are as described herein above. To this end, the formula used to determine the same was (Abejie, 2015);

$$\text{Cost of travel time lost (Ct)} = \text{Cr} * \text{Vn} * \text{Vt} * \text{Vo} \quad \text{Equation 4.38}$$

Where,

Ct=Cost of travel time lost

Cr= Hourly wage of road users (Cr); for passenger vehicles

Vn =Number of vehicles waiting in the system

Vt= Average waiting time per vehicle

Vo = Average vehicle occupancy

The total cost of travel time lost from Mexico to Estifanos road section was estimated to be **ETB: 1,163,916.56** per hour.

4.3.2 Cost of extra fuel consumption

To calculate extra fuel consumption due to waiting at congested section, the following parameters were required as inputs;

4.3.2.1 Average fuel consumptions of single vehicle per hour

Traffic congestion leads to excess fuel usage due to time spent idling in jam and the start-and-stop nature of travel in congested conditions, as travel at a lower speed consumes more fuel.

The Australian Government Department of Transportation on Working Paper No. 71, (2007) has developed graphical relation between travel speed and fuel consumption as illustrated in Figure 4-12 below.

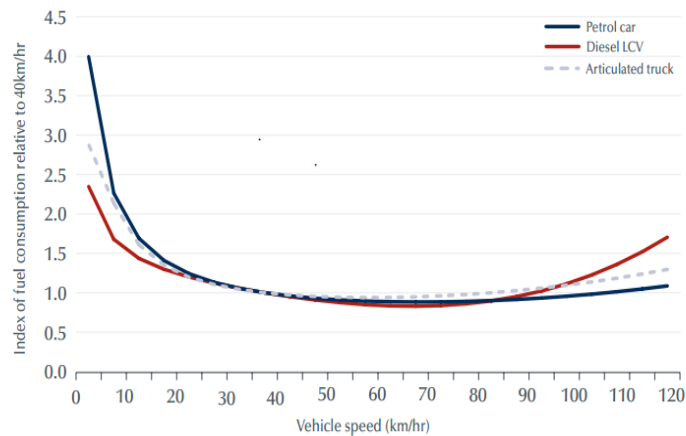


Figure 4-12: Speed Vs Fuel Consumption chart

Almost all vehicles operate with fuel efficiency when traveling between 50 and 80km/hr. Otherwise, vehicles consume increasingly more fuel the faster or the slower they travel. In this regard, the following chart was developed by Ardina (2006) to estimate the amount of fuel burnet by vehicle with respect to vehicle speed.

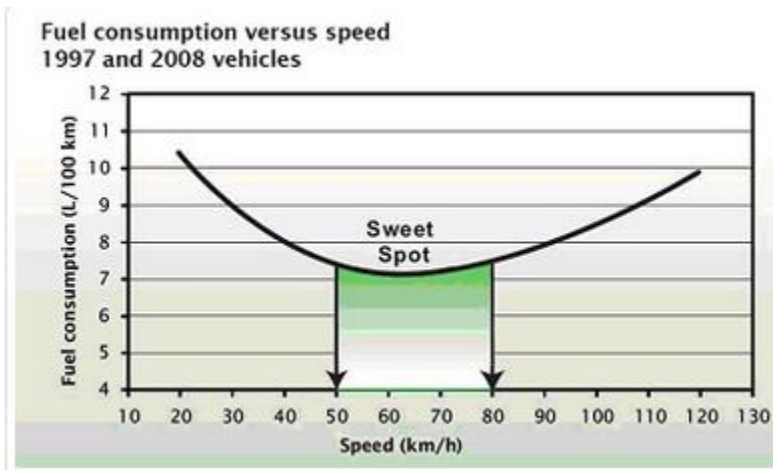


Figure 4-13: Speed Vs. Fuel Consumption chart

Ardina (2006) mentioned that for every 10 minutes of idling time, the car consumes around 100ml of petrol. In this study the same was adopted.

4.3.2.2 Cost of fuel per litter

Market value of fuel per liter was obtained from gas stations. For the Addis Ababa condition, the cost of gasoline is ETB: 16.20.

The parameters used to calculate cost of travel time lost were described above. Accordingly, the

same formula was used to determine the values in this case;

$$\text{Cost of extra fuel consumption (Cf)} = V_n * V_t * F_v * C_{fl} \quad \text{Equation 4.39}$$

Where;

Cf=Cost of extra fuel consumption;

V_n=Number of vehicles waiting in the system;

V_t =Average waiting time per vehicle (sec);

F_v=Average fuel consumptions of single vehicle per hour; and

C_{fl}=Cost of fuel per litter.

The cost of extra fuel consumption from Mexico to Estifanos road section was estimated to be **ETB: 27,526.20** per hour.

4.3.3 Emission

The highest level of emissions for Ethiopian condition Nox at 3.44 g/km, HC at 6.53 g/km, CO at 13.9 g/km, PM 2.5 at 1.3 g/km and CO₂ at 35.96 g/km was estimated by Neway (2016) on his study titled “*Investigation on Pollution Caused by Gasoline and Diesel Fuelled Vehicles*”. In addition to this other important parameter is vehicle kilometer travel. For the entire road segment 9206km has been covered by vehicles in peak one hour period. This parameter was directly taken from Synchro software. Having this data emission is calculated by;

$$\text{Emission} = EF * VKT \quad \text{Equation 4.40}$$

Where;

EF= Emission factors; and

VKT = vehicle kilometer travel.

Therefore, from Mexico to Estifanos road section on road vehicles emit 31,668.64 g/h of NO_x, 206,796.22 g/h of HC, 2,874,467.45 g/h of CO, 3,736,807.68 g/h of PM 2.5 and 134,375,604.21 g/h of CO₂.

Hence, the overall costs of congestion were estimated by adding **Cost of travel time lost (C_{tf})**

and **Cost of extra fuel consumption (Cf)**. Based on this specific study, region it was estimated that the country lost **ETB: 1,191,442.76** per hour from extra fuel consumption and travel time lost. In addition, it is also affected by pollution with emission rate of 31,668.64 g/h of NO_x, 206,796.22 g/h of HC, 2,874,467.45 g/h of CO, 3,736,807.68 g/h of PM_{2.5} and 134,375,604.21 g/h of CO₂.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This research focused on analyzing the impacts of improperly allocating traffic signal timing of Signalized Intersections on the flow of traffic and the effect of traffic congestion on road vehicles. The case study was the Ras Mekonnen street in Addis Ababa. For performance evaluation, the Highway Capacity Manual (HCM) was used and also for optimization of signal timing of Signalized Intersections, the Synchro software was used. In addition, the effect of congestion was estimated in monetary values with respect to the following parameters;

- Cost of extra fuel consumption; and
- Cost of travel time lost.

Further, the amount of emission from inroad vehicles with respect to NO_x, HC, CO, PM_{2.5} and CO₂ were also estimated.

Accordingly, the following results were obtained;

- The traffic flow at all intersections of the route corridor was served by poor Level of Service (LOS F).
- The results obtained using the HCM methodologies and Synchro software with respect to performance evaluation were almost the same.
- The saturation flow rate of the route corridor was high as compared to HCM default value which is mainly arising from aggressive nature of drivers.
- Due to the effect of congestion on extra fuel consumption and travel time lost, the country loses **ETB:1,191,442.76** per hour due to traffic jam on the route corridor.
- Due to the effect of congestion, the country is subjected to pollution with emission rate of: 31,668.64 g/h of NO_x, 206,796.22 g/h of HC, 2,874,467.45 g/h of CO, 3,736,807.68 g/h of PM_{2.5} and 134,375,604.21 g/h of CO₂.
- The service quality of the Intersections can be improved by changing the cycle length from 188 sec to 270 sec for Estifanos Intersection and 230 sec for all other intersections

and phase sequences together with splits. More than 550 vehicles will be relieved from stopping at the intersections and also extra 654 liters of fuel will be saved from improvement of signal performance.

- Though, effectiveness parameters were improved due to optimization of cycle length, offsets and splits, the level of service of the network is not changed from LOS F to better service. This is happen due to the reason that the arrival rates are too much to be served by the intersections.

5.2 Recommendations

- In order to improve the service quality of the corridor, only signal timing optimization is not enough by itself. Hence, another traffic management mechanism should be applied like overpass and introducing additional lanes to existing network.
- Using HCM default values leads inappropriate judgment, so calibration is required. For example, the value of saturation flow rate in our condition is higher than the HCM default value.

5.3 Recommendations for Future Research Areas

There are several potential areas which remain to be examined and can have significant influence for improvement of traffic signal operation. Some of the areas can be;

- Calibration of Level of Service thresholds for local conditions;
- Effect of driving behavior on saturation headway estimation;
- Formulation of other improvement methods other than signal re-timing to improve the level of service of road way;
- Further investigation should also be conducted to determine other types of congestion costs like noise cost; and
- Calibration of HCM to Ethiopian Condition.

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