Optimization of Intersection Signal Timing -Case Study of Shola Gebeya and Estifanos - Stadium - Laghar Intersection.

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This is to certify that the thesis prepared by Tadele Girma, entitled: Optimization of Intersection Traffic Signal Timing: Case Study on Shola Gebeya and Estifanos - Stadium - Laghar intersection and submitted in partial fulfillment of the requirements for the degree of Master of Science (Civil and Environmental Engineering) complies with the regulation of University and meets the accepted standards with respect to originality and quality.

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DECLARATION

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Abstract

In Addis Ababa, recent problems of traffic delay at traffic signal intersection is becoming increase and the optimizations of traffic signal timing is being the effective tool toward reduction of total delay, travel time and fuel consumption. This paper contains the overall structure and the various components of a simulation-based optimization system to optimize the parameters of fixed time traffic signal intersection. This study examines method for optimization and simulation by using Synchro and Sim traffic software packages.

The optimization of intersections (Estifanos - Stadium - Laghar and Shola Gebeya) were done to propose a better cycle length, green split, offset and phase sequences. Traffic simulation model was developed for assessment purposes. Coordination of network intersection from Estifanos - Stadium - Laghar was done for increasing bandwidth through network.

This study evaluated method of improving performance over the existing condition at (Estifanos - Stadium - Laghar and Shola Gebeya) intersections. The parameters of measure of effectiveness (MOE) were determined after simulation of traffic movements. The reductions of delays between 17% to 60% were obtained. The travel time reductions between 18% to 53% were obtained. In addition the fuel consumption reductions between 17% to 46% were obtained.

Overall, the evaluation of the (Estifanos - Stadium - Laghar and Shola Gebeya) showed that the current Synchro and Simtraffic signal optimization procedure significantly improved the performance of network operations. Thus, the study recommended that AACRA continue using its procedure for developing new timing plans but that it evaluate its signal timing plan regularly so that it does not become outdated.

Table of Contents

List of Figure

List of Tables

List of Acronym

AACRA:	Addis Ababa City Road Authority
C:	Cycle Length
D:	Average Delay
CBD:	Central Business District
ERA:	Ethiopia Road Authority
FFS:	Free Flow Speed
LOS:	Level of Service
HCM:	Highway Capacity Manual
MOE:	Measure of Effectiveness
MDOT:	Michigan Department of Transportation
MnDOT:	Minnesota Department of Transportation
NEMA:	National Electric Manufacturer Association
OD:	Origin Destination
PCE:	Passenger Car Equivalent
TRANSYT:	Traffic Network Study Tool
T:	Flow Period in Hours
X:	Degree of Saturation

1. INTRODUCTION

1.1 Background of the Study

On current dates, most metropolitan area in the world are experiencing traffic congestion during commuting hours, more than any times and to alleviate such inconvenience authorizes aim on planning of infrastructure projects., in upgrading or constructing new facility (Kidus Ayalneh, 2016)[1]. Specifically it states that, in Addis Ababa with traffic engineering perspective, new infrastructure measures did not prove to be adequate in that inefficiency is predominant.

Vehicular traffic passing through an intersection must to be controlled in order to overcome the conflicts arising between different directional flows of traffic. The installation and operation of traffic lights provide an effective method of controlling traffic when vehicle flows are relatively high. A problem the traffic engineer then faces is to determine the timing of these traffic lights so as to optimize the flow of traffic (Khewal Bhupendra Kesur, 2007)[2]. Traffic signal timing optimization has been recognized as one of the most cost-effective methods for improving mobility within the urban transportation system. Inappropriate signal timing plans can cause not only discomfort (extra delay) to drivers but also increased emissions and fuel consumption (B. Brian Park and Schneeberger, 2002)[3].

Traffic congestion is a severe problem at an intersection in Addis Ababa, having been causing many critical problems and challenges in city. Indeed, signal operation is the technique to appropriately decide the signal cycle and effective green time or other factors, with respect to various intersections, phases, traffic flows and others for the reduction of traffic congestion. Therefore, the study on traffic signal optimization at intersection in Addis Ababa traffic networks is important to improve service function of road. The issues include reviewing and updating the phasing sequence, displays, timing parameters and other related operational aspects of individual signalized intersections within a city.

The signal must be effective and functional for a wide variety of users that include pedestrians, automobiles, bicyclists, transit, and large trucks. Signalized intersections provide for the organized control of conflicting traffic movements in a safe manner; however, these intersections can be a source of frustration for motorists due to delays. As Addis Ababa city continues to develop, travel patterns have changed over the years (Transport policy of Addis Ababa, 2011)[4],leading to outdated traffic signal timings that account for a significant amount of delay on roadways throughout the town. By updating signal timings and installing new technology, benefits can be achieved at a relatively low cost. Updated signal timings and equipment have the potential to reduce vehicular delay and thereby improve air quality through reduced emissions and less time spent idling at an intersection.

This report summarizes the results of the studies conducted at some Addis Ababa City Road Authority controlled intersections throughout Addis Ababa.

1.2 Statement of Problem

Now a days the traffic congestion at intersections in Addis Ababa become increasing. There is high delay of traffic at intersections. This affects the life of people. There is also large number of traffic accident in the city which is caused by high traffic congestion at intersection. Specifically, the intersections at Shola Gebeya and Estifanos-Stadium-Laghar have inadequate distribution of signal parameters between demands and service. These intersections are highly congested and traffic delay at these intersections is high when compared to other intersections in Addis Ababa. The other problem that has been seen in almost all intersections is that the through vehicles in opposite direction could not use the road at the same time due to the absence of updated protected phases (see Figure 1). Although through vehicles in both directions can use the road at same phases, but the problem that is obviously seen is the additional delay of through vehicles when the vehicles in opposite direction are under movement. This increment of total delay increases the number of vehicles in queue.

Figure-1 Vehicles delay at Shola Gebeya

In general at traffic signal intersection in Addis Ababa, there are high traffic congestions which are leading to high queue and delay. These affect the traffic smooth flow which reduce the safety and comfort of the passengers and incur additional time.

1.3 Research Questions

The major research questions are listed below.

- What are the optimum cycle length, phase sequences and green split?
- What are the values of MOE parameters before and after optimization?
- How can optimization of signal parameters reduce MOE parameters?
- Is the present cycle length, offset, phase sequence and green split are properly provided or not?
- What are the possible mitigation measure used to be taken to reduce the delay and travel time at intersection?

1.4 The Objective of Study

1.4.1 General Objective: The overall goal of this study is to assess the performance of traffic signal and develop the optimum signal timing parameters at test intersection in Addis Ababa.

1.4.2 Specific Objectives: The specific objectives of this research include:

- To evaluate delay, number of vehicle in queue, fuel consumption, air pollution and travel time.
- To develop simulation model of traffic movement.
- To assess the performance of existing signal timing parameter at test intersection in Addis Ababa.
- To compute the optimum values of cycle length, offset, phase and green split.

1.5 Limitation of Study

The first limitation of this study is that since the traffic count is taken by video there may be traffic which is hidden by larger vehicle. The second limitation is the impacts of adjacent intersections (Intersections at Mexico and Biherawi) which are closer to the Estifanos – Stadium –Laghar corridor were not quantified. The third limitation is during the counting of traffic; the peak hour was identified based on mixed flow even if it should depend on Passenger Car Unit by using the assumption that, traffic composition stay constant, on a given interval. The fourth limitation is due to the difficulty of recording video on total network, Origin-Destination volumes were not considered and any extra vehicles in network were taken as mid-block vehicles.

1.6 Significance of Research

In Addis Ababa, the traffic accidents and traffic jam is becoming uncontrollable scenario. The importance of research is that by using software called SYNCHRO after calibrating and validating to local condition, an optimization of fixed signal parameter had been done, which has potential of delay reduction at intersection. Also traffic movement simulation is modeled by using sim traffic software package to assess the performance of intersections.

1.7 Thesis Organization

This research contains five chapters and related appendixes and they are listed as follow.

The First Chapter contains introduction part of research such as background of the study, statement of problem, research questions, objectives, limitations, significance of research and research structure.

The Second Chapter contains a thorough review of literature which was related to optimization. In this chapter different reference materials were reviewed and different software programs which are used to optimize are deeply discussed.

The Third Chapter contains the methodologies that are used during data collection. Here method of traffic data count, video, measurement from site, description of study area and location, each input data during analysis are discussed in detail.

The Fourth Chapter contains results and its discussion. By using synchro and sim traffic data collected in chapter three, the different values of cycle length, green split, phase sequences and offset are determined. Also delay, fuel consumption, travel time, and air pollutions are calculated. Then the new values were compared with the previous and percentages of reductions were calculated.

The Fifth Chapter contains conclusions and recommendations depending on the result discussed in chapter four. This includes optimized cycle length, phase sequence, green split and offset.

At the end there will be Appendixes which are used as extra references. In this part there are Tables and Figures which are determined from traffic analysis and extracted from software.

2 LITERATURE REVIEW

2.1 Introduction

Signal optimization for congested conditions has been studied since the 1960s. In 1963 (Gazis and Potts, 1963)[5], proposed a "store and forward" strategy for dealing with oversaturated traffic signals. This strategy refined and presented later uses time varying traffic demands combined with a mathematical programming approach to optimally store and dissipate queues at signals where demand exceeds capacity. This store and forward approach does not account for the effects of queue variation within a cycle and offsets. (Rahmann, 1973)[6] said that queuing would be a norm during peak periods and presented the idea of designing signals as storage/output devices, even during under saturated conditions. (Pignataro et al, 1978)[7] attempted to define congestion and oversaturation in terms of their causes and scope, and proposed guidelines for dealing with such conditions. As reported by (Lee et. 1975)[8] the primary objective of the queue-control policy was to delay or eliminate intersection blockage (Nadeem A. Chaudhary, Chi-Leung Chu, Srinivasa R. Sunkari, and Kevin N. Balke, 2010)[9].

In most cases, the models' derived splits are usually long enough to service the uniform arrival of traffic demand during the plan period, and accommodate a desired two-way progression. Selecting the right splits, however, requires good familiarity with all critical movement in the network. A critical movement is defined as one that can affect several other related movements in a negative way. For example, a short split for a left-turn movement with a less than desired storage bay could result in a spillover that would affect the adjacent and upstream through movements, thus causing unusual lane utilization at and upstream of the intersection. Such an occurrence could compromise efficiency and safety for these movements (Ziad A. Sabra and Keith Riniker, 1999)[10].

The mathematical model was used to determine the green splits and the level of metering, whereas TRANSYT was used for simulating the dynamic processes within the cycle and for offset optimization. Other instances include the works of (Choi, 1997)[11] and (Lieberman and Chang, 2005)[12] who refined the internal metering models and demonstrated their real-time application. Kim and Messer developed mathematical models for controlling congested interchanges and arterials with single critical intersections. These models managed queues at external approaches to a system, while preventing spillback and reducing delay on the interior links (Nadeem A. Chaudhary, Chi-Leung Chu, Srinivasa R. Sunkari, and Kevin N. Balke, 2010)[9].

Traffic signal optimization and coordination is a cost effective way to improve the flow of traffic along a corridor. At signalized intersections it is important for the signal timing plans to match existing traffic patterns within the corridor. The coordination and optimizing of traffic signals is a way to maximize the capacity of the intersections within the corridor without having to perform costly infrastructure improvements (Clough harbor, 2012)[13].

The primary goal of the signal timing optimization and coordination project is to reduce average fuel consumption and emissions by improving traffic mobility, decreasing travel times, traffic delays and number of vehicle stops at signalized intersections. (Alliant engineering enc, 2012)[14]

However, Webster's green split calculation method, which treats each signalized intersection separately, may not be valid when queues from one intersection start to influence an adjacent signal. To accommodate such situations, the second model uses an iterative method for simultaneously calculating green splits and offsets for all signals in the system. The procedure automatically determines if the system is under saturated or oversaturated. If the system is oversaturated, it pushes all excess demand to the user-defined boundary of the system, while keeping the interior links clear of detrimental queues (Clough Harbor, 2012)[13]

The difference in opinion on the *Cycle Length* issue is more related to the philosophy of signal timing rather than on short versus long. Those who truly believe that maximizing coordination on major thoroughfares should dictate the determination of the optimum cycle length usually favor long cycles. Under these circumstances, the coordinated traffic movements receive more green time than the side street movements, and those who are delayed on the side streets usually will make up the delay if they are destined for the coordinated movements. Also, the ratio of the sum of fixed time intervals (Y+AR) to a cycle length is lower, when a cycle length is increased, than the same ratio for a shorter cycle length. Therefore, one could argue that increasing the cycle length for progression is more efficient than reducing it, and also has a lesser occurrence probability on the type of crashes that are associated with the change intervals (Ziad A. Sabra and keith rinker1999)[10].

2.2 Benefits of Optimization

According to (Cough harbor, 2012)[13] the benefits of optimization are

- Reduction in travel time and delays
- Reduction in stops and traffic slowdowns could reduce accident potential
- Reduction in fuel consumption (i.e., less idling time) and vehicle emissions
- Potential to delay/eliminate the need for intersection widening
- Reduced driver frustration by having to stop at numerous closely spaced signalized intersection

According to (Srinivara Sunkari, 2004)[15] some result of optimization project were listed below.

- As per the Institute of Transportation Engineers (ITE), traffic signal improvements reduce travel time by 8 to 25 percent. The reduction in travel time also reduces fuel consumption and emissions.
- The Fuel Efficient Traffic Signal Management Program in California demonstrated a benefit to cost ratio of 58:1. The program optimize 3,172 signals, resulting in 15-percent savings in delays, 8.6-percent savings in fuel consumption, 16-percent savings in stops and 7.2-percent savings in travel time.
- The Traffic Light Synchronization (TLS) Program in Texas showed a benefit to cost ratio of 62:1. By retiming traffic signals with the TLS program, Abilene, TX, experienced reductions of 14 percent in travel time and 37 percent in delays. Overall, the program resulted in a 24.6-percent reduction in delays, a 9.1-percent reduction in fuel consumption and a 14.2-percent reduction in stops.
- In Kitchener-Waterloo, Canada, 89 intersections that included arterials in commuter and commercial routes and central business district areas were retimed. The project demonstrated savings of 10 percent in travel time, 27 percent in delays and 20 percent in stops.

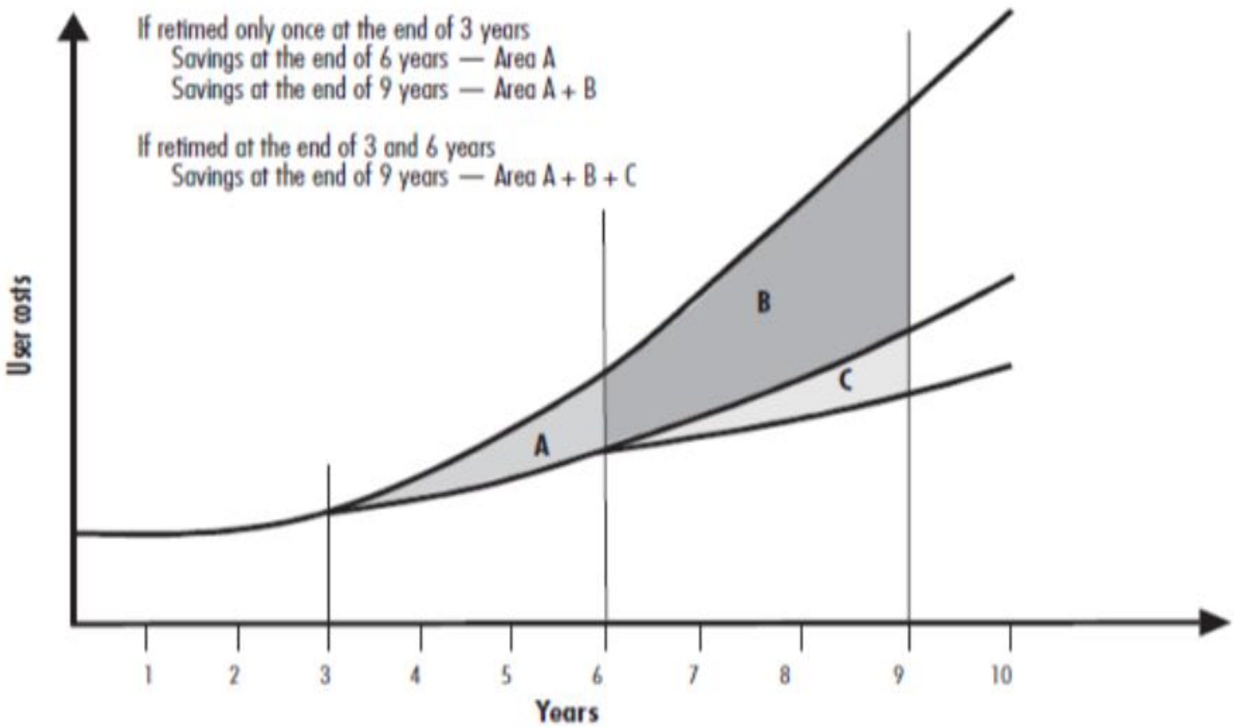


Figure 2 Benefit of optimization graphically (Source srinivari Sunkari, 2004)[15]

2.3 Passenger Car Equivalent

The significant impact Heavy Vehicles (HV) have on freeway operations has been identified since the first edition of the Highway Capacity Manual (HCM). The method of incorporating their impact in freeway capacity calculations has changed through the years. The HCM 2000 used Passenger Car Equivalent (PCE) values and percent of trucks/buses and Recreational Vehicles (RV) to account for HV effect on capacity. However PCE values in the most recent HCM edition rely on a limited field database and extensive simulation runs based on this information; they were calibrated on ‗steady-flow traffic operations. The objective of this effort was to identify and quantify HV characteristics that have an impact of freeway throughput at various congestion levels on level, urban freeways using 1.2 million individual vehicle observations, with an emphasis on operations at LOS E and F. It was desired to use the products of this effort as recommended inputs for future simulation runs of congested freeway flow conditions. Passenger Car (PC) and HV headways were found to increase with HV presence in the traffic stream (Umama Ahmed, 2009) [16].

There are different methods of calculating PCE among these methods using the spatial headway methodology, (Seguin et al, 1982)[17] suggested PCE as the ratio of average headway for Vehicle types: average truck headway divided by the average headway for passenger cars:

$$PCE_{ij} = \frac{H_{ij}}{H_{pcj}} \text{ -- Equation 1}$$

Where PCE_{ij}, = the PCE of vehicle type i under conditions j,

H_{ij} = average headway for vehicle type i,

Hpcj = the average headway for passenger car for conditions j.

(Green shields et al, 1947)[18] Estimated PCU value by the following equation. This method is known as basic headway method.

$$PCU_i = \frac{H_i}{H_c} \text{ -- Equation 2}$$

Where PCU_i = passenger car unit of vehicle type,

H_i = average headway of vehicle type,

H_c = average headway of passenger car.

(Werner and Morrall, 1976)[19] Recommended determining PCE using headways when the roadway is sufficiently congested on level terrain:

$$PCE = \frac{\frac{HM}{HB} - PC}{PT} \text{ ---Equation 3}$$

Where, HM = is the average headway for a sample including all vehicle types, HB = is the average headway for a sample of passenger cars only, PC = is the proportion of cars, PT = is the proportion of trucks.

(Demarchi and Setti, 2003)[20] Proposed the PCE's formula to eliminate the possible error for mixed heavy vehicles in the traffic stream, including interaction between multiple trucks types:

$$PCE = \frac{1}{\sum_i^n Pi}\left(\frac{qB}{qM} - 1\right) + 1 \quad \text{--- Equation 4}$$

Where Pi= proportion of trucks of type i out of all trucks n in the mixed traffic flow

q_B= equivalent passenger car only flow rate for a given v/c ratio,

q_M= mixed flow rate

2.4 Delay Estimation

Signalized intersections were developed in England in the early 20th century. With the introduction of these controls to maneuver conflicting streams of vehicular and passenger traffic, researchers have concentrated on estimating delays due to these controls and in developing the optimum signal timings to minimize delay especially for pre-timed signals. Webster's equation is one of the fore most delay equations developed in 1958 assuming practical distributions like Poisson (random) arrivals with uniform discharge headways (Anil Kamarajugadda and Byungkyu B Park, 2003)[21]. They also recommend the following formula which is based on Webster equation.

$$d = \frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x2}{2q(1-x)} - 0.65\left(\frac{c}{q2}\right)^{0.5} x^{2+5\lambda} \quad \text{-------------------------------------Equation 5}$$

Where,

d is the average delay per vehicle,

c is the cycle time,

λ is the ratio of the effective green to the cycle length,

q is the flow rate,

s is the saturation flow rate, and

x is the degree of saturation.

According to (zonghi li and lili Du, 2016)[22] among all the essential assumptions and approximations, the delay formula at signalized intersections is potentially the most important one. They conclude same signal settings considered under different cost assumptions may provide totally different theoretical properties and result in completely different results. From the classic deterministic queuing model to the Shock wave delay model, different models have different assumptions made with different behaviors in both uncongested and congested situations.

A research conducted by (Dion et al. ,2004)[23] compared vehicle delays provided by a number of analytical delay models with delays estimated by microscopic traffic simulator on a one-lane approach to a pre-timed signalized intersection approach for traffic conditions ranging from under-saturation to over-saturation. According to (Zonghi li and lili Du ,2016)[22], all types of delay formulations, the time-dependent stochastic delay models were reported to provide strong consistency with the microscopic simulation method approach under both under-saturated and over-saturated conditions.

As one of the most widely accepted time-dependent stochastic delay models, 2010 HCM delay model employed the incremental queue accumulation procedure to calculate the uniform delay instead of the first item of the Webster's formula (Strong and Rouphail, 2006) [24].

According to (MnDOT, 2013)[25] Delay studies at individual intersections are valuable in evaluating the efficiency or effectiveness of a traffic control method. Other factors include crashes, cost of operation, and motorists' desires. It also lists the factors which affect delay at intersections which include:

- Physical factors such as number of lanes, grades, widths, access control, turning provisions, transit stops, etc.
- Traffic factors such as volume on each approach, driver characteristics, turning movements, pedestrians, parking, approach speeds, etc.

Delay estimate measures reflect the driver discomfort, frustration, fuel consumption and lost travel time. Numerous equations have been developed for the estimation of delay. In the U.S., delay is estimated using the Highway Capacity Manual (HCM). The HCM equation is a function of multiple input parameters arising from geometry, traffic and signal conditions. The HCM procedure for a signalized intersection uses average demand flow rate and saturation flow rate in order to estimate volume to capacity ratio and the corresponding performance measure, delay. The Level of Service (LOS) is then determined from a predefined range of average control delay values (Anil Kamarajugadda, and Byungkyu Brian Park, 2003)[21]. They also imply that an average delay value obtained from the data collection may not reflect the actual performance of the intersection. In other words, an average delay value of say 35 seconds per vehicle (LOS D) obtained on the data collection day has no significance if the 95th percentile confidence interval of delay varies from 20 to 50 seconds per vehicle (LOS B to LOS D). Further, LOS is not a good performance measure when the delay value lies border line of two LOS categories. For example, an average delay of 34.9 seconds is considered as LOS C, while 35.0 is LOS D.

(Akcelik, 1988)[26] further developed the delay equation by utilizing the coordinate transformation technique to obtain a time-dependent equation that is applicable to signalized intersections. A generalized delay equation of the form is shown in Equation 6 below was developed that embraces the Australian and Canadian delay formulas as well.

$$d = \frac{0.5c(1-u)^2}{1-ux} + 900Tx^n\left(\left|\lfloor x-1\rfloor + \sqrt{(x-1)2 + m(x-xo)/QT}\right|\right) \text{------------------Equation 6}$$

Where,

d = average overall delay

u = g/C (ratio of effective green to the cycle length)

x = degree of saturation

T = flow period in hrs.

m, n = calibration parameters

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

c = signal cycle time in seconds

In the U. S., the Highway Capacity Manual (HCM) delay equation is utilized in delay computations. The HCM 2000 propounds that delay be computed using the following equation (Anil Kamarajugadda and Byungkyu Brian Park, 2003)[21].

$$D=\frac{0.5*C\left(1-\frac{g}{C}\right)2}{1-(\min(1,X)g/C}+900*T((X-1)+\sqrt{(X-1)2+8kIX/cT)})$$ -----------------Equation 7

Where,

C = cycle length in seconds,

g = effective green time in seconds,

X = degree of saturation (v/c),

v = demand volume in vehicles/hour

T = duration of analysis period hours,

k = incremental delay factor, 0.5 for pre-timed signals,

i = upstream filtering/metering adjustment factor, 1 for isolated intersection, and

c = capacity in vehicles per hour.

2.5 Optimization Methods

Traffic signal timing optimization has been recognized as one of the most cost-effective methods for improving mobility within the urban transportation system. Inappropriate signal timing plans can cause not only discomfort (extra delay) to drivers but also increased emissions and fuel consumption. Thus, it is important to investigate the practice of signal optimization methodology to ensure that newly developed timing plans will improve the system performance. The investigation can be conducted via either field testing or the use of a reliable simulation tool (B Brian Park and J.D Shneeberger, 2002)[3].

According to (Khewal Bhupendra Kesur, 2007)[2] following measures are typically used as indicators of performance when evaluating signals timing schemes:

- Vehicular delay
- Number of vehicle stops
- Capacity
- Queue lengths
- Fuel consumption
- Vehicle Emissions

Optimal signal settings are obtained by minimizing or maximizing the combination of the above measures. The most commonly used criterion is delay. Typically, two different types of delay are considered (Khewal Bhupendra Kesur, 2007)[2]

- ✓ Stopped delay: This is the total time spent by a vehicle in a stationary queue.
- ✓ Control delay: This is the difference between the actual travel time and the uninterrupted travel time. It includes the delay from deceleration, stopped delay, queue move-up time and acceleration delay.

Various simulation programs and optimization techniques have evolved that aid the traffic engineer in the optimization process. Delay and its derivative are used as the objective function in most optimization software. For example, SYNCHRO optimizes based on the percentile delay while TRANSYT-7F optimizes based on factor that involves the average delay called the disutility index. However, delay is a stochastic variable and optimizing a signalized intersection for an average value fails the system performance for extreme demand conditions. Thus, a methodology that optimizes the intersection considering stochastic variability would be very useful (Anil Kamarajugadda and Byungkyu Brian Park, 2003)[21].

According to (Tamir Balasha, 2014)[27] the parameters of optimization commonly considered are four basic groups which are: cycle length, green splits, phase sequence and offsets.

(Loannis Psaros, 2016)[28] Define each parameter as follows

- ✓ Cycle length: Cycle length is the total time to complete one sequence of all movements around an intersection
- ✓ Split: An individual (movement) split is the sum of the green time + yellow interval + red clearance interval for a particular movement
- ✓ Actuated Traffic Control: Fully-actuated signals have detectors on all of the approaches and semi-actuated signals only have detectors at some of the approaches.
- ✓ Signal Coordination**:** Process to synchronize start of the "green light" along the major corridor so that a group of vehicles can travel together ("platoon") through multiple signals with minimal or no stopping
- ✓ Offset**:** Time between start of the "green light" at one intersection and the start of "green light" at another intersection (the offset defines the movement of traffic along the corridor/major road, also referred to as "progression)

2.5.1 SYNCHRO

SYNCHRO, developed by Traffic ware Inc., is a software package that can model and optimize traffic signal timings. SYNCHRO minimizes a parameter called percentile delay in its optimization. The Percentile Delay is the weighted average of a delay corresponding to the 10th, 30th, 50th, 70th and 90th percentile volumes. SYNCHRO accommodates for progression by calculating the progression factor (PF) used in the delay equation using the ratio of uniform delay calculated by SYNCHRO with coordination and uniform delay calculated by SYNCHRO assuming random arrivals. Furthermore, SYNCHRO uses quasi-exhaustive search in offset optimization (Anil Kamarajugadda and Byungkyu Brian Park, 2003)[21].

It is a macroscopic and deterministic model for optimizing traffic signal timing plans. Synchro can optimize cycle lengths, green splits, phase sequences, and offsets. Splits are optimized by percentile, with Synchro attempting to provide enough green time to serve 90% of the flow from a lane group. If there is not enough cycle time to serve the 90% flow, 70%, 50%, etc., flow is then tried (Synchro Manual, 1999)[29]. Any extra green time goes to the main street. Synchro attempts to determine the shortest cycle length that clears the critical percentile traffic when optimizing cycle lengths. Offset optimization is conducted through a semi-exhaustive search. It is not possible to perform an exhaustive search for every second. Instead, Synchro uses three steps to eliminate "bad" offset areas. The first step looks at every 8 seconds for offset values. The bad areas are eliminated. Second, it looks at every 4 seconds, eliminating the bad areas (B BRIAN PARK, 2002).

SYNCHRO is a macroscopic traffic software program that is based on signalized intersection capacity analysis as specified in HCM 2000. Macroscopic level model represents in terms of aggregate measure for each movement of the intersection. Equation is used to determine measure of effectiveness such as delay and queue length (Synchro Manual, 1999)[29].

Major steps used in Synchro program as per (Synchro Manual, 1999) [29]

1) Lane calibration: under this step travel time, LT, RT, ThT ,lane width, ideal saturated flow, right turn factor, left turn factor, lane utilization factor, storage lane, storage length, approach grade and CBD factor are those adjusted under lane calibration.

2) Volume calibration**:** Peak hour factor, growth rate, bus blockage factor Fbb, number of parking maneuvers, traffic from mid-block (%), O-D movement for intersection, traffic in shared lanes are the parameters which are adjusted under volume setting.

3) Node setting: type of controller, Cycle length minimum is adjusted under node setting.

4) Timing calibration: under this step turn type, lost time adjust, lead/lag optimization, recall nodes, are the variables which are filled in this step.

5) Phasing calibration: minimum gap, vehicle extension, time before reduce, time to reduce, pedestrian phases, walk time, flash don’t walk pedestrian calls, visualization of phasing are determined under this step.

6) Simulation calibration: Taper length, lane alignments, blocked intersection, median width, link offset for Estifanos intersection, crosswalk width. TWLTL, headway factor, mandatory distance, positioning distance are adjusted under simulation setting step.

2.5.2 TRANSYT-7F

TRANSYT (Traffic Network Study Tool) is a macroscopic traffic signal model originally developed in the United Kingdom by the Transportation and Road Research Laboratory and later modified by the University of Florida Transportation Research Center for the Federal Highway Administration (FHWA). The basic premise of the analysis procedures used in TRANSYT-7F is the macroscopic, step-wise modeling of platoon progression and dispersion as it travels through a series of adjacent intersections. The TRANSYT-7F software has been designed to serve two primary functions. The first of these is the simulation of traffic as it flows through an arterial or network. The second is the development of optimized traffic signal timing plans. In both cases, it is required that all signals operate with consistent cycle lengths, though double cycling can be incorporated (MnDOT Manual, 2013)[25].

According to Steven W. Davis the two major functions of TRANSYT-7F are to simulate traffic flow and optimize the traffic signal timing plans. In order to do this the user must input the following data, so the traffic model may represent traffic flow within the network, accurately:

1 Network Data
- A Intersection
- B Block Length

2 Signal Timing Data
- A Signal Sequence
- B Cycle Length
- C Offset

3 Capacity Parameter
- A Saturation flow
- B Extension of effective green time
- C Start up lost times

TRANSYT-7F employs a more realistic modeling of network traffic flow than other models. A simulation model is employed to track flow patterns from external links all the way through the network. Through and turning proportions are specified at the end of each link. (Khewal Bhupendra Kesur, 2007)[2].

TRANSYT 7F is a mesoscopic-deterministic model for analyzing and optimizing signal timings on arterials and networks. Like CORSIM, TRANSYT 7F has been developed and tested over a period of several decades and has gained acceptance from the user community as a sound model. TRANSYT 7F uses a combination of exhaustive, hill-climbing, and GA based optimization methods. TRANSYT 7F uses a delay-based traffic model. In other words, it is primarily designed to select signal timings that produce minimum system delay and stops. In addition, it provides a capability to select several secondary objectives, including minimization of stops and maximization of progression opportunities. During its optimization process, TRANSYT 7F generates second-by-second flow profiles of vehicles on all links in the network. Then, it analyzes these profiles to determine MOEs (Texas Transportation Institute, 2006)[30].

TRANSYT 7F has two delay-based traffic models. The first model (original model) performs the optimization in a link-wise fashion by optimizing timings for one link at a time. This model does not accurately account for queue buildup because it treats a queue of vehicles as an upward stack at the stop bar. However, it works well for under saturated traffic conditions. Users all over the world have extensively validated this model.

The second model was recently added to remove the limitations of the first model. This model takes into consideration the formation and dissipation of queues in space. In addition, it accounts for flow interactions on adjacent links through a step-by-step analysis of all links in the system (Texas Transportation Institute, 2006)[30].

TRANSYT-7F produces various other measures of effectiveness that are useful in evaluating intersection performance. These include total travel and total travel time, stops, and fuel consumption. In addition to these intersection MOEs, TRANSYT-7F outputs also provide route and system MOEs. These include total travel and travel time; uniform, random, total, average, and passenger delays; stops; speeds; fuel consumption; operating costs; and performance index (MnDOT, 2013)[25].

2.5.3 Genetic Algorithmic

Genetic Algorithms are search algorithms based on the mechanics of natural selection and evolution. John Holland, his colleagues and his two students at the University of Michigan developed these algorithms (Goldberg, 1989)[31].

According to (Anil Kamarajugadda and Byungkyu Brian Park, 2003)[21] A genetic algorithm process starts with a random set of individuals called the population. The individuals in a population are represented in the form of binary strings. These strings are then acted upon by operators, which produce a different population every generation, and then this cycle is repeated until certain termination criteria are met.

A simple genetic algorithm is composed of three operators:

- Reproduction
- Crossover
- Mutation

The reproduction is a process in which individuals are selected based upon their fitness value or the objective function. This operator is an artificial version of natural selection, the survival of the fittest. The reproduction operator is implemented in algorithmic form in a number of ways. Roulette wheel selection and tournament selection are some of them. After reproduction, a crossover operator involving two steps is operated. (Anil Kamarajugadda and B Brian Park, 2003)[21].

Genetic algorithms are computational models based on the mechanisms of natural selection and evolutionary theory. A primary application of genetic algorithms has been in the field of function optimization. Genetic algorithms are capable of handling both continuous and combinatorial optimization problems (Khewal Bhupendra Kesur, 2007)[2].

Genetic algorithms (GAs) belong to a class of algorithms known as evolutionary algorithms which have been developed fairly recently. A GA starts with a subset of scenarios (some members of a population) and applies principles of natural selection (mating, gene mutation, etc.) to generate a new or revised set of scenarios (called the next generation). A GA-based optimization model uses a specified traffic simulation model to evaluate the fitness of each member (i.e., a signal timing scenario) in the current population. Then, it generates a new population by combining the characteristics of selected pairs of scenarios (members). The principles of natural selection ensure that the characteristics of the fittest members (i.e., those with higher bandwidths or lowest delays, depending on the objective of optimization) have a high probability of transmission to the next generation (Texas Transport Institute, 2006)[30].

2.6 Microsimulation

Microsimulation is the dynamic and stochastic modeling of individual vehicle movements within a system of transportation facilities. Each vehicle is moved through the network of transportation facilities on a split second by split second basis according to the physical characteristics of the vehicle (length, maximum acceleration rate) the fundamental rules of motion (e.g. acceleration times time equals velocity, velocity times time equals distance) and rules of driver behavior such as car following rules, lane changing rules, etc. (Richard Dowling, 2004)[33].

Additionally the (Richard Dowling, 2004)[33] define micro simulation as the modeling of individual vehicle movements on a second or sub second basis for the purpose of assessing the traffic performance of highway and street systems, transit, and pedestrians.

The microscopic traffic simulation models are based on the reproduction of the traffic flows simulating the behavior of the individual vehicles, this not only enables them to capture the full dynamics of time dependent traffic phenomena, but also to deal with behavioral models accounting for drivers' reactions (venter, 2001)[34].

The method for developing a micro simulation model can best be thought of as the building up of several layers of the model until the model has been completed. The first layer (the link/node diagram) sets the foundation for the model. Additional data on traffic controls and link operations are then added on top of this foundation. Travel demand and traveler behavior data are then added to the basic network. Finally, the simulation run control data are input to complete the model development task (FHWA, 2004)[35].

The type of information that micro simulation can provide for a further analysis is beyond the capabilities of traditional static models. The average flows from sections to sections turning movements) for the allowed movements at selected intersections in the model, speeds and delays for every simulated time interval can be obtained. The dynamic analysis for a time period is completed with values for other traffic variables or indicators of the quality of service as number of stops, time delayed at stops, average queue lengths (Venter, Vermeulen and J Barceló2)[36].

Examples of micro simulation software are: Aim sum, CORSIM, Paramics, Simtraffic, Transmodeller, VISSIM, WATSIM, etc. They stochastically model individual vehicle movements as a function of time and space (Richard Dowling, Joseph Holland and Allen Huang, 2002)[37].

2.6.1 Simtraffic

Simtraffic is a microscopic model used to simulate a wide variety of traffic controls, including a network with traffic signals operating on different cycle lengths or operating under fully actuated condition. Sim traffic also models unsignalized intersection roundabout and channelized right turn lane. In sim traffic, each vehicle in the traffic system is individually tracked through the model and comprehensive operational measures of effectiveness are collected on every vehicle during each 0.1 sec of simulation. Driver behavior characteristics (ranging from passive to aggressive are assigned to each vehicle by the model affecting the vehicles free flow speed queue discharge, headway, and other behavioral attributes the variation of each vehicle's behavior is simulated in manner reflecting real world operation (Simtraffic Training Course, 2005)[38]. Also it says, since sim traffic is microscopic the model measure the full impact of queuing and blocking.

Simtraffic is especially useful for analyzing complex situations that are not easily modeled macroscopically including (Synchro Manual, 2013)[39].

- ✓ Closely spaced intersections with blocking problems
- ✓ Closely spaced intersections with lane change problems

- ✓ The effects of signals on nearby un signalized intersections and driveways
- ✓ The operation of intersections under heavy congestion.

Simtraffic Measures of Effectiveness MOEs (synchro manual, 2013):

- Stopped Delay
- Stops
- Queue Lengths
- Travel Time and Distance
- Fuel Consumption and Efficiency
- Exhaust Emissions

2.7 Signal Coordination

(Loannis Psarros, 2016)[28] defines Signal coordination as process to synchronize start of the "green light" along the major corridor so that a group of vehicles can travel together ("platoon") through multiple signals with minimal or no stopping.

In urban area where traffic signal are nearby, the coordination of adjacent signal is important and gives great benefit to road user by increasing the utilization per unit time in peak hour (Naveen Kumar and Santha kumar)[40].

There are four major areas of consideration for signal coordination (Tom V Mathew, 2014)[41]:

1. Benefits
2. Purpose of signal system
3. Factors lessening benefits
4. Exceptions to the coordinated scheme

1 Benefit

It is common to consider the benefit of a coordination plan in terms of a cost or penalty function; a weighted combination of stops and delay, and other terms as given here:

Cost = A × (total stops) + B × (total delay) + (other terms). The object is to make this dis-benefit as small as possible.

2 Purpose of the signal system

The physical layout of the street system and the major traffic flows determine the purpose of the signal system. It is necessary to consider the type of system, whether one-way arterial, two-way arterial, one-way, two-way, or mixed network. The capacities in both directions on some streets, the movements to be progressed, determination of preferential paths

3 Factors lessening benefits

Among the factors limiting benefits of signal coordination are the following:

- Inadequate roadway capacity
- Existence of substantial side frictions, including parking, loading, double parking, and multiple driveways
- Wide variability in traffic speeds
- Very short signal spacing
- Heavy turn volumes, either into or out of the street

4 Exceptions of the Coordinated Scheme

All signals cannot be easily coordinated. When an intersection creating problems lies directly in the way of the plan that has to be designed for signal coordination, then two separate systems, one on each side of this troublesome intersection, can be considered. A critical intersection is one that cannot handle the volumes delivered to it at any cycle length.

Time-Space Diagram and Ideal Offsets

The time-space diagram is simply the plot of signal indications as a function of time for two or more signals. The diagram is scaled with respect to distance, so that one may easily plot vehicle positions as a position of time.

2.8 Measure of Effectiveness (MOE)

All traffic signal timing and analysis models produce at least some estimates of performance, or measures of effectiveness (MOEs). There are two general classes of MOEs: 1) estimates of performance measures that allow the analyst to evaluate the quality of the system, and 2) performance measures which in fact serve as the explicit objective function of the optimization. Only optimization models are concerned with the later type (MnDoT manual, 2013)[25].

Degree of Saturation

Degree of Saturation is defined as:

$$\mathrm{x} = \frac{vC}{sg} \quad \text{……………………………………………………..Equation 8}$$

where,

X = degree of saturation expressed as a decimal value or multiplied by 100 to form a percentage,

v = volume in vph,

C = cycle length in seconds,

s = saturation flow in vph, and

g = effective green (split time - lost time).

According to (MDOT Manual, 2008)[42] Measures of Effectiveness (MOEs) should be evaluated to determine the effectiveness of the optimization process. MOEs to be considered vary between the local intersection and network levels:

Local Intersections: Intersection *control delay* and *level of service* (Highway Capacity Manual method) should be evaluated as the primary MOE at the local intersection level. Wherever possible, Level of Service (LOS) C or better should be achieved for all approaches.

However, judgment should be used to balance approach levels of service based on relative traffic demand.

Network/System:

Progression *bandwidth* should be evaluated as the primary MOE at the network/system level. The optimization should aim not only to provide the maximum bandwidth along major corridors, but to position the band to provide progression for the leading vehicles in the platoon (leading edge bandwidth) wherever possible. While other MOEs should be evaluated as part of the optimization and reporting process, the above MOEs should be considered the most important when making optimization decisions and Adjustments (MDOT Manual, 2008)[42]. Also SimTraffic Measures of Effectiveness (MOEs) should be collected and evaluated as a means of further assessing the results of the optimization process. The following MOEs should be reviewed during the analysis process, and should be included in the final report

- ✓ Total Network Delay
- ✓ Average Network Speed
- ✓ Total Network Travel Time
- ✓ Total Network Stops.

3 RESEARCH METHODOLOGY

In this chapter the research methods, material and procedures are presented. Different methodologies and procedures are used during data collection. Both field investigation and office works were involved. Quantitative data and analysis were used to determine the level of service of the selected intersections and MOE parameters. As a data source both primary data or direct field measurements and secondary data were the main sources of quantitative data. The primary data were collected by direct measurement. Also qualitative data from questionnaire were also used to determine whether the delay in selected intersection is significant or not and to assess other related parameters.

3.1 Description of Study Area

In order to conduct this study, seven intersections in Appendix C were taken as general representative of intersections in Addis Ababa. These intersections were selected by using simple random sampling method. But the following two intersections had been selected from seven intersections in Appendix C based on traffic delay and queue at intersections. Ranking of the intersection by their delay and queue was done depending on the result of questionnaire in Appendix C. The most congested intersections are the following.

These are:

1) Isolated intersection at Shola Gebeya
2) Network intersection from Estifanos – Stadium - Laghar

3.1.1 Description of Isolated Intersection at Shola Gebeya

This intersection found at 9°1’27.99’’ latitude and 38°47’45.55’’ longitude on the way from 4-kilo to Megenanya next to British embassy at the altitude of 2397m asl. The intersection is a four legged intersection which contain four approaches which come from Megenanya approach, 4 kilo approach, northbound and southbound approach. The intersection has fixed time type of controller. The Figure 3 shows the general top view of Shola Gebeya intersection.

Figure 3 Shola Gebeya Intersections.

3.1.2 Description of Network Intersection from Estifanos – Stadium – Laghar

This is the corridor which is found from Estifanos to Laghar. It contains three intersections and each intersection has fixed time control type. It has the total length of 1067m. The general top view of Estifanos – Stadium – Laghar was indicated in Figure 4.

Figure 4 Top View of the Network

3.1.2.1 Estifanos Intersection

Estifanos intersection is found at the latitude 9º0'39.41'' and longitude of 38º45'45.96'' near the Estifanos church. It has elevation of 2345m above sea level. It has four legs namely 4-kilo approach, Laghar approach, Bole approach and Hayahulet approach. This intersection has fixed time type of controller. The general top view of intersection at Estifanos was indicated in Figure 5.

Figure 5 Top view of Estifanos Intersection

3.1.2.2 Stadium Intersections

This intersection is found at 9º 0'41.42'' latitude and 38º45'33.5''longitude around Ethiopian national Stadium. It is found at altitude of 2348m asl. It has four approaches which are called Estifanos approach. Biherawi approach, Laghar approach and Gotera approach. It has fixed time type of controller. The general top view of intersection at Stadium was indicated in Figure 6.

Figure 6 Top View of Stadium Intersection

3.1.2.3 Laghar Intersection

This intersection is found at latitude of 9° 0’ 42.5’’ and longitude of 38°45’11.16’’ around Laghar bus station at the elevation of 2357m above sea level. It has four approaches which are Piazza approach, bus station approach, Stadium approach and the leg which come from Mexico. It has pre timed type of controller. The general top view of intersection at Laghar was indicated in Figure 5.

Figure 7 Top View of Laghar Intersection

3.2 Study Design

The methodology of this research includes both quantitative and qualitative data collection method. Under quantitative there were different data which have been used during optimization. These are traffic volume, signal data and geometrical data which were used as input during analysis. The video of mixed traffic flow was taken on 3/20/2017, 4/10/2017 4 and then counted during office work in order to determine the traffic volume. Number of each vehicle depending on their types was also determined. The data were collected by direct measurement during field investigation. The secondary data such as speed, traffic growth were collected from AACRA manuals. Under qualitative data collection, site observation was conducted and qualitative data such as turn types, phase sequences, lanes sharing were collected. Since the data were primary, they are accurate and efficient to optimize and evaluate the actual performance of each test intersection. Also questionnaire in (Appendix C) was taken whether there was traffic delay on segments and to identify the time of a day at which the traffic flow becomes high.

Since it is impossible to optimize each isolated and network intersections in Addis Ababa, by using random sampling method, some selected intersections were only considered. In this research, as much as possible the study tried to represent the traffic signal intersection in Addis Ababa, under these two types namely isolated intersection and network intersection.

Under isolated intersection the intersection at Shola Gebeya was selected depending on response from questionnaire Appendix C. As general representatives of network intersection the arterial intersection from Estifanos – Stadium - Laghar was selected, depending on the feedback from road user.

The analysis of data was done by one of the optimization software packages, which is called synchro and simulation was done by simtraffic software packages. The opportunity of using Synchro is that it is simple and straight forward. It optimizes the fixed traffic signal parameter. Its limitation is that it does not show platoon dispersion along corridor (Synchro Manual, 2013)[39]. The simtraffic simulates the movement of individual vehicles by considering the driver behavior and the data can be transferred from synchro.

The limitation of sim traffic is that it does not consider the age and type of vehicle when calculate the fuel consumption and emission (Synchro Manual, 2013)[39] .

Each data entry and calibration was done properly to mitigate the statement problems which were discussed in introduction part. The analysis has been done effectively to optimize all parameters of fixed cycle signal control.

3.3 Data Collection Methods

The data collections methods had been mostly done by direct measurement from field during field works.

The primary data were collected by

- Recording video
- Direct measurement/field observation
- Traffic count method
- Stop watch to measure travel time
- Questionnaire

Questionnaires are distributed among different road user, mostly traffic police and driver around intersections. From this questionnaire peak flow in the time of day is morning from 7:45am to 8:45am and 4:00pm to 6:00pm. Depending on this feedback from road user, the traffic flow data was taken by video from 7:00am to 10:00am and 4:00pm to 6:00 pm to consider the fluctuation of traffic flow**.** Each data are discussed below in detail.

3.3.1 Traffic Count Data

First the video was taken from 7:00am to 10:00 am during morning peak and 4:00 to 6:00pm during afternoon peak. As sample for the report, the intersection at Estifanos was discussed here in detail because the same methodology was used for other intersections. The data for the left intersection were listed in Appendix A.

The video of Estifanos intersection was taken on March 20, 2017 from 7:00am to 10:00am morning peak and 4:00pm to 6:00 pm afternoon peak. Then the mixed flow data was counted for all movement types such as (EBL, EBT, EBR, WBL, WBT, WBR, NBL, NBT, NBR, SBL, SBT, SBR) for both morning and afternoon peak period by the interval of 15 minutes. Estifanos intersection was selected below to reduce redundancy since the other intersections were done in similar way. For other intersection traffic count data were found in Appendix A.

Table 1 : Mixed flow traffic data at Estifanos

time		turning types										
start time	end time	EBL	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
7:00	7:15	46	139	12	217	20	125	81	5	12	91	59
7:15	7:30	82	197	9	222	21	139	122	11	23	136	81
7:30	7:45	118	215	29	215	17	237	167	8	25	122	86
7:45	8:00	87	219	16	200	9	290	199	8	35	148	103
8:00	8:15	89	313	40	208	6	205	153	4	55	183	115
8:15	8:30	109	334	33	233	5	371	75	3	35	255	121
8:30	8:45	78	318	38	227	10	303	163	2	26	177	118
8:45	9:00	70	379	24	191	9	354	198	9	32	173	98
9:00	9:15	85	298	16	318	17	296	159	9	43	196	119
9:15	9:30	82	323	29	206	15	315	189	7	37	165	107
9:30	9:45	57	329	27	184	13	337	161	9	34	178	111
9:45	10:00	64	276	12	168	7	290	142	6	39	154	72
4:00	4:15	84	239	18	180	11	291	172	8	42	159	94
4:15	4:30	75	200	26	147	16	261	117	6	39	167	83
4:30	4:45	79	296	21	261	15	184	158	7	27	139	93
4:45	5:00	72	247	27	277	10	196	167	11	35	151	96
5:00	5:15	67	355	26	306	9	252	164	6	39	165	94
5:15	5:30	83	302	21	208	11	263	172	14	20	172	68
5:30	5:45	77	340	24	229	12	271	198	8	28	225	82
5:45	6:00	76	262	14	198	14	231	156	12	27	251	83

After the count for each movement, the consecutive four intervals were added to identify the interval of the maximum peak hour. To add mixed flow before changing to passenger car, this method was appropriate to determine the peak hour by using the assumption that, traffic flow compositions remain constant through time. For example, for the east bound through (EBT) vehicles, the maximum flow is determined by adding four consecutive 15 minutes and selection of the largest traffic volume.

Similarly the count was conducted to Shola Gebeya isolated intersection. The data was taken on April 10, 2017 for both morning and afternoon peak hour. Similar procedure was used to determine the peak flow hour. Table 2 show the mixed traffic data for each movement type.

Table 2: Mixed Traffic Flow of Shola Gebeya

time		turning types											
start time	end time	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
7:00	7:15	2	241	57	53	231	1	119	1	24	1	1	3
7:15	7:30	8	299	106	55	356	1	142	1	28	1	2	1
7:30	7:45	3	326	119	64	395	1	158	1	35	1	1	3
7:45	8:00	5	349	115	79	432	2	142	1	53	0	1	1
8:00	8:15	5	374	139	70	400	3	131	2	51	1	0	1
8:15	8:30	2	385	130	86	374	2	128	1	48	1	1	0
8:30	8:45	12	374	94	97	335	2	149	1	63	2	0	1
8:45	9:00	6	319	89	106	332	2	124	1	89	1	2	3
9:00	9:15	8	396	139	64	406	2	104	1	49	1	1	3
9:15	9:30	9	377	125	81	378	1	121	1	67	1	0	4
9:30	9:45	5	331	114	88	368	0	117	2	57	1	3	2
9:45	10:00	4	361	105	79	361	1	109	1	54	0	1	3
4:00	4:15	8	355	83	86	266	2	89	3	78	1	0	4
4:15	4:30	10	382	117	91	318	3	127	1	69	2	1	2
4:30	4:45	11	384	113	62	326	4	119	2	64	1	0	5
4:45	5:00	7	371	121	60	311	3	123	2	59	2	0	3
5:00	5:15	7	367	107	45	290	1	139	1	48	0	1	1
5:15	5:30	9	360	112	67	333	0	129	0	46	1	0	1
5:30	5:45	13	379	97	53	349	1	147	1	43	0	0	2
5:45	6:00	8	386	77	42	314	0	122	1	58	0	1	2

From the Table 2 for example for East Bound Left movement the maximum mixed flow is 37 vehicles which are found from 5:00pm to 6:00 pm. But since the above figures do not consider the effect of heavy vehicle on passenger car it would not be used directly in analysis. Because each traffic data should be changed to passenger car with respect to their passenger car unit factor. So the next step is changing the selected peak hour flow to passenger car.

By using the assumption that traffic composition is similar through time, then traffic count depending on their type, is done for only the time range of (4:30 to 5:30 pm) for EBT movement in order to consider the effect of heavy vehicles on passenger cars at Estifanos. For other turning types depending on their maximum range the count is done as follows Table 4 and for other intersections data were listed in Appendixes A.

The vehicles data in Table 4 below added, depending on their type, to determine the maximum hourly flow of each vehicle. The next step is determining Passenger Car Unit (PCUi) and converting each vehicles effect to passenger car equivalent.

There are different methods of determining the PCU. Among this from literature review the researcher uses spatial headway methodology Equation 1,

$PCE_i = H_{ij}/HPC_j$

Where PCE_i is passenger car equivalent of vehicle i

H_{ij} is time headway of vehicle i in condition j

HPC_j is time headway of passenger car in condition j

The next is by using the video that was taken previous; the headway for each vehicle is measured. The sample of measuring the headways is taken ten (10) times in order to consider the effect of one vehicle following other vehicle and keeping the accuracy of data. As much as possible the researcher tried to include the effect of one vehicle following another. e.g. (passenger car following large bus, passenger car following passenger car, heavy vehicle following large bus, medium bus following two wheels etc.).

Below there is the sample of time headway measured at Estifanos intersection and for the other intersections it was discussed in Appendixes A. By using the above Equation 1, passenger car units were calculated and listed under last row of Table 3.

Table 3: Time Headway (sec) and PCE at Estifanos

no trial	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
1	0.82	0.61	1.16	2.03	2.64	2.64	3.25	3.15	5.68
2	*	1.81	0.91	2.64	2.65	2.24	2.99	3.01	8.26
3	*	2.03	1.94	2.08	2.95	3.35	5.03	3.41	7.01
4	*	1.22	2.13	2.48	3.25	2.51	5.35	3.46	*
5	*	1.18	1.53	2.18	2.23	2.96	5.07	2.75	*
6	*	1.72	2.24	1.53	4.07	3.45	3.7	3.15	*
7	*	1.52	4.06	3.35	3.14	3.25	4.87	4.07	*
8	*	1.22	2.44	1.95	2.03	3.15	4.46	4.47	*
9	*	1.22	3.79	5.15	2.23	2.46	4.17	4.57	*
10	*	0.72	1.93	1.63	2.14	3.51	3.97	2.64	*
Avg	0.82	1.325	2.213	2.502	2.733	2.952	4.286	3.468	6.983
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12

* represent no vehicles

From above Table 3 under bicycle, and large vehicle column since there were no enough vehicles the researcher could not get ten samples and passenger car unit were calculated only for available data. By using the above calculated PCEi and multiplying it with corresponding number of vehicles Table 5 the effect of each vehicle was changed to passenger car. At last by summing up all vehicles type, the vehicle per hour was determined as follows in Table 4.

Table 4: Passenger Cars at Estifanos

time	EBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:30-7:45	0	3	53	26	20	6	12	4	0
7:45-8:00	1	2	30	23	19	5	11	3	0
8:00-8:15	0	5	36	25	12	5	13	2	0
8:15-8:30	0	3	48	27	15	3	10	3	0
sum	1	13	167	101	66	19	46	12	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	0.37	7.787	167	114.13	81.18	25.46	89.24	18.84	0
sum(pc)	504								
	EBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:30-4:45	2	6	113	88	51	18	18	8	0
4:45-5:00	1	7	110	83	46	17	15	7	0
5:00-5:15	0	8	101	71	42	16	15	6	0
5:15-5:30	2	8	138	59	63	19	17	8	0
sum	5	29	462	301	202	70	65	29	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	1.85	17.371	462	340.13	248.46	93.8	126.1	45.53	0
sum(pc)	1335								
	EBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:00-8:15	2	7	118	90	57	19	14	8	0
8:15-8:30	0	9	122	98	60	20	15	9	0
8:30-8:45	1	7	120	99	57	19	14	10	0
8:45-9:00	2	8	137	114	68	23	17	9	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	1.85	18.569	497	453.13	297.66	108.54	116.4	56.52	0
sum(pc)	1550								
	WBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:00-8:15	0	1	18	8	0	2	1	0	0
8:15-8:30	0	0	14	6	7	2	1	0	0
8:30-8:45	1	1	17	9	6	2	2	1	0
8:45-9:00	0	0	11	6	4	1	1	1	0
sum	1	2	60	29	17	7	5	2	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	0.37	1.198	60	32.77	20.91	9.38	9.7	3.14	0
sum(pc)	138								

Table 4: Passenger Cars at Estifanos (continued)

	WBT vehicles								
Time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:30-4:45	0	8	95	73	52	17	12	7	1
4:45-5:00	2	6	20	81	49	17	12	9	0
5:00-5:15	0	8	110	83	61	18	14	10	1
5:15-5:30	1	5	92	40	42	12	9	7	1
sum	3	27	317	277	204	64	47	33	3
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	1.11	16.173	317	313.01	250.92	85.76	91.18	51.81	9.36
sum(pc)	1136								
	WBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:00-7:15	0	2	9	4	5	2	1	1	0
7:15-7:30	0	1	9	6	2	1	1	1	0
7:30-7:45	1	2	7	4	3	1	1	0	0
7:45-8:00	0	0	4	2	2	0	0	0	0
sum	1	5	29	16	12	4	3	2	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	0.37	2.995	29	18.08	14.76	5.36	5.82	3.14	0
sum(pc)	80								
	NBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:15-8:30	2	8	143	104	60	22	17	12	0
8:30-8:45	1	7	113	93	47	19	14	9	0
8:45-9:00	2	8	158	80	71	22	16	11	1
9:00-9:15	2	6	101	86	59	18	13	10	0
sum	7	29	515	363	237	81	60	42	1
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	2.59	17.371	515	410.19	291.51	108.54	116.4	65.94	3.12
sum(pc)	1531								
	NBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:30-8:45	1	5	62	41	33	10	7	4	1
8:45-9:00	0	7	77	50	40	12	9	5	0
9:00-9:15	1	5	61	39	32	9	7	4	0
9:15-9:30	1	4	73	47	39	11	8	5	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	1.11	12.579	273	200.01	177.12	56.28	60.14	28.26	3.12
sum(pc)	812								

Table 4: Passenger Cars at Estifanos (continued)

	NBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
5:00-5:15	0	1	3	2	1	0	0	0	0
5:15-5:30	1	2	6	3	3	1	1	0	0
5:30-5:45	0	0	4	2	3	0	0	0	0
5:45-6:00	0	1	6	2	2	1	1	1	0
sum	1	4	19	9	9	2	2	1	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	0.37	2.396	19	10.17	11.07	2.68	3.88	1.57	0
sum(pc)	51								
	SBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
9:00-9:15	0	1	16	11	9	3	2	2	0
9:15-9:30	1	2	14	9	7	2	2	2	0
9:30-9:45	0	1	12	10	7	2	1	1	0
9:45-10:00	1	1	14	11	8	2	2	1	1
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	0.74	2.995	56	46.33	38.13	12.06	13.58	9.42	3.12
sum(pc)	182								
	SBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
5:00-5:15	1	5	63	42	32	10	9	6	0
5:15-5:30	0	3	66	44	34	10	8	6	0
5:30-5:45	2	6	81	60	46	13	10	8	1
5:45-6:00	1	7	78	68	50	15	11	9	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	1.48	12.579	288	241.82	199.26	64.32	73.72	45.53	3.12
sum(pc)	930								
	SBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:45-8:00	1	3	40	25	21	6	5	3	0
8:00-8:15	1	4	44	29	22	7	5	3	0
8:15-8:30	0	5	53	30	24	7	5	5	0
8:30-8:45	1	4	52	22	24	7	6	4	0
sum	3	16	189	106	91	27	21	15	0
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12
pc	1.11	9.584	189	119.78	111.93	36.18	40.74	23.55	0
sum(pc)	532								

In addition to passenger car, the following parameters are also used in optimization

Heavy Vehicles (%): is the percentage of large truck and large bus. As sample EBT was taken its heavy vehicle average is 5.59% in total flow.

Pedestrian per Hour: is numbers of pedestrian per hour that conflict with right turn movement during permitted phases. This number affect right turn factor and saturated flow rate.

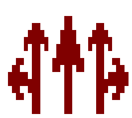
Bus Blockages: is the number of buses which stop and block the traffic flow. This also affects the saturated flow rate.

Bicycle per Hour: is through bicycle which may become obstacles to right turn vehicles. This includes through bicycle only.

3.3.2 Geometric and Signal Primary Data

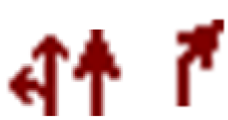
In addition to the above traffic data there are many other data which should be collected for input during optimization using synchro and sim traffic software packages. Researcher use sample intersection which is found at Estifanos for brief explanation and reduce redundancy. For the left intersections the same procedures were used to measure and collect the necessary data.

Lane sharing SB (the lane from 4-kilo approach) it has one left and thru shared lanes, one thru lane, one right turn exclusive lanes with 3.6m width except exclusive of EBR which is 6m. Others are also shown in Figure 8.



A) Northbound Lane

B) Eastbound Lane



C) Westbound Lane

D) Southbound Lane

Figures 8: lane sharing at Estifanos

Storage lanes and length: All approach leg at Estifanos intersection has no storage lanes and it has zero storage length.

Right channelization and add lanes (#): SB lanes have right channelization with one (1) add lanes. Also WB has right channelization and no adds lanes.

Right Turn On Red: Yes because there is right turn movement during red phases for EB, SB and WB.

Adjacent parking lanes (#) and park maneuvering (#/hr): On all approaching lanes there is no adjacent parking lanes and parking maneuvering is 0veh/hr.

Turn type: Since the left turn use the protected phases of adjacent thru the left turn type is split type. The right turns since it has acceleration lanes at downstream and it yields to pedestrian the turn is free type.

Protected phases:

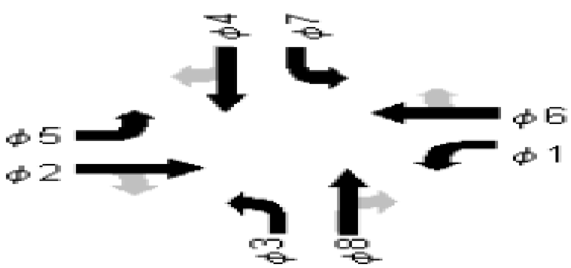


Figure 9: Type of turning phases

All turning phases are split phases because the left turning vehicle uses thru green time on similar directions without yielding to the ongoing thru vehicles.

Permitted phases: The right turns phases are permitted phases because they can move always by yielding to pedestrian.

Control Type: the control type is pre timed

Pedestrian Phases: There is no a pedestrian phase.

Cycle Length:

The existing/baseline/ timing of the Estifanos intersection at the time data was collected is as follows.

Yellow times…………….3 sec

Red times………………151 sec

All red……………………0.5 sec

Green times………………34 sec

Taper Length: There is no taper and taper length is zero**.**

Median Width: The median width is measured as 1.8m for SB, 4m for EB and WB, 3m for NB lanes.

Cross Walk Width: The width of crosswalk/zebra is 3.4m for NB and SB and it is 3m for EB and WB approaches. It is determined by direct measurement using meter.

Turning Speed: This is used during simulation by sim traffic and synchro manual recommends to left turn the speed of 15mph and for right turn 9mph.

Design Period. The design period of traffic light control depends on addition of extra intersection and at most it should be inspected yearly.

Grade: The alignment of road is almost a level or slight slope at Estifanos.

Ideal Saturated Flow: As local study indicated by (Kidus Ayalneh, 2016)[1], it recommends saturation flow rate from 1789 to 1820 and saturation headway of 1.97sec to 2.01se which is almost similar to HCM recommendation of 1900 vph and 1.9 sec saturation headway.

Link Origin-Destination Volumes: according to synchro manual OD volumes are used only if the distance between the nodes or intersections is less than 300 ft. So it is unnecessary in our case because the link is greater than 300 ft.

Enter Blocked Intersection This is not allowed entering intersection when it is blocked by traffic jam.

Links Offset: It was determined by measuring distance using meter from the projected straight center of south bound to centerline of northbound. It is around 15m.

Background Image: even if it has no effect on result it simplifies the process of adding network and link and the following is taken for all three intersections Estifanos-Stadium-Laghar.

Figure 10: Background image of network

All the above listed data are similarly done for all intersection at Shola Gebeya, Stadium and Laghar.

4 RESULTS AND DISCUSSION

In this chapter the optimization of the selected intersections is done by using the synchro and simtraffic software packages. The Synchro was used to optimize signal parameters using macro optimization. But the Simtraffic was used to simulate the micro properties of each individual vehicle. The different parameter of measure of effectiveness such as total delay, stop, fuel consumption and travel time were discussed in detail and the existing parameters is compared with the new parameters which were the result of optimization. In this research two types of intersections are selected, depending on the result of questionnaire (Appendix C), to represent the general features of Addis Ababa signal intersection. Among the respondents, 43% of them respond the high delay intersection from Estifanos – Stadium – Laghar and 38% responds Shola Gebeya. So the network intersection from Estifanos – Stadium – Laghar and isolated intersection at Shola Gebeya were selected as test segment.

4.1 Traffic Composition

Traffic compositions of these intersections were done as an aggregate of directional movement of vehicle based on traffic counted data during morning and afternoon peak. Below the traffic composition are indicated in chart form.

4.1.1 Traffic Composition and Passenger Car Equivalent at Shola Gebeya

Based on the peak hour flow of each turning movement the vehicle type versus their number is shown below in chart form.

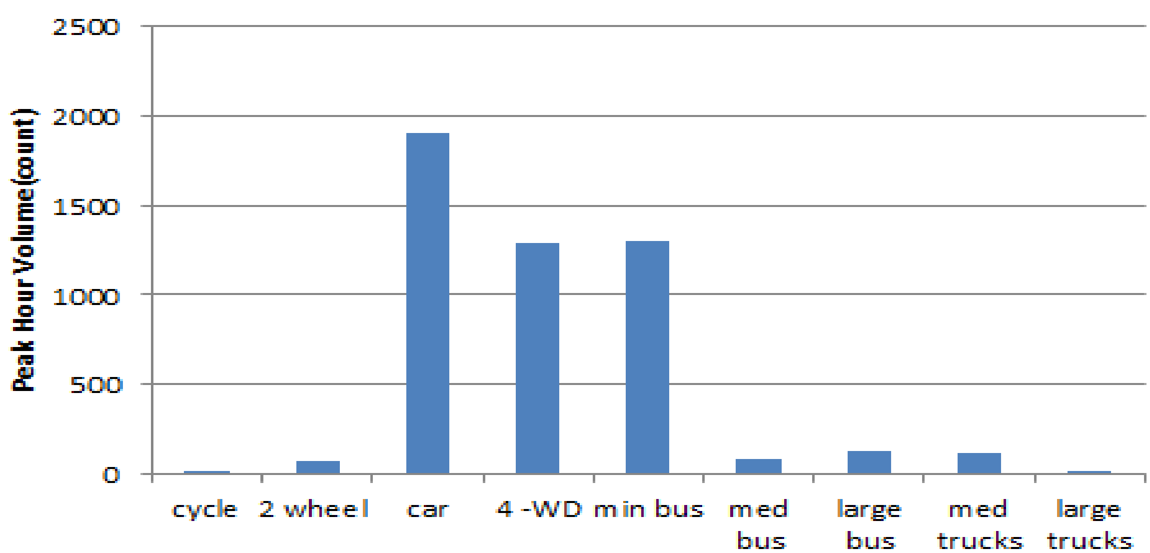


Figure 11: Vehicle Composition of Shola Gebeya

As indicated in methodology parts, the passenger car analysis of this intersection based on headway. There are different methods of determining PCU. Equation 1 was used to compute it. The following values of passenger car equivalents were determined for Shola Gebeya intersection.

Table 5: Passenger Car Equivalent at Shola Gebeya

vehicles	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med truck	large trucks
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29

4.1.2 Traffic Composition and Passenger Car Equivalent at Estifanos

Depending on the peak hour flow data the traffic composition of Estifanos intersection is shown in Figure 12 by chart format. The passenger car equivalents of each vehicle at Estifanos are indicated in Table 6.

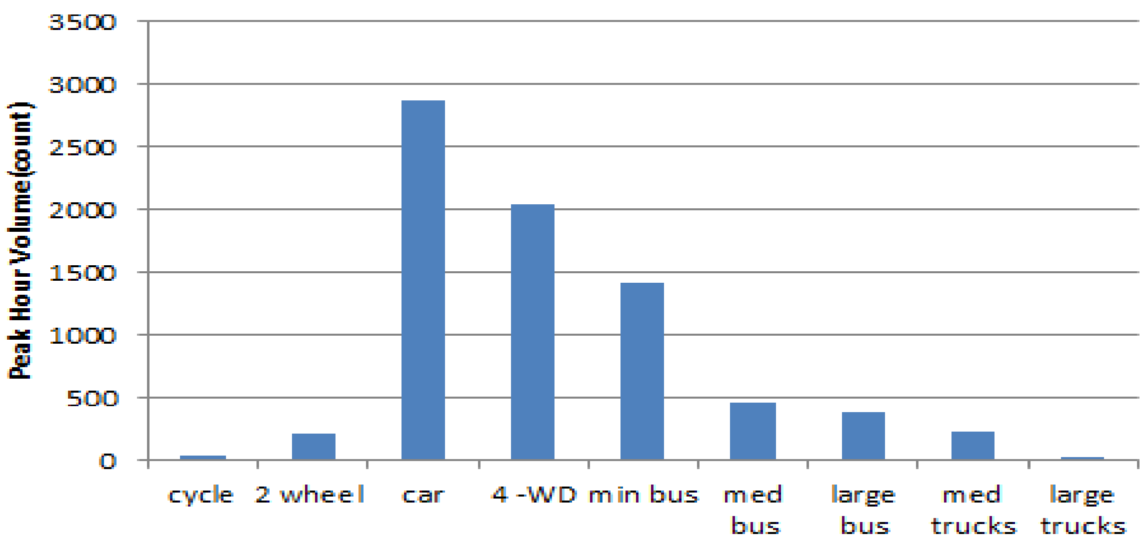


Figure 12: Traffic Composition at Estifanos

Table 6: Passenger Car Equivalent at Estifanos

vehicle	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
pcui	0.37	0.599	1	1.13	1.23	1.34	1.94	1.57	3.12

4.1.3 Traffic Composition and Passenger Car Equivalent at Stadium

Based on traffic count data collected during the morning peak and afternoon peak hours the traffic composition of the vehicles at Stadium intersection was shown in Figure 13.

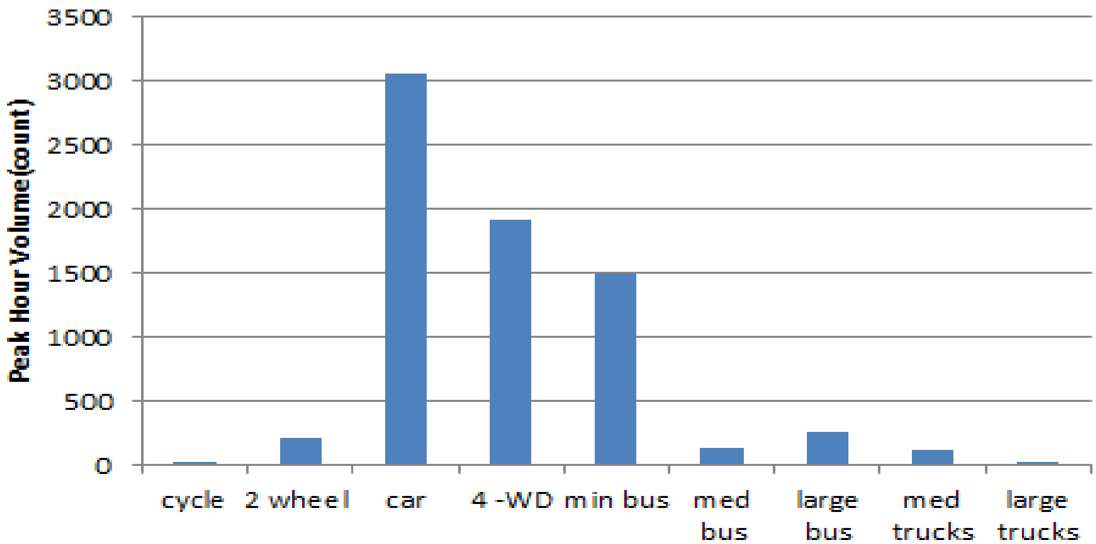


Figure 13: Traffic Composition at Stadium

Passenger car equivalent of intersection at Stadium is also calculated similar to the above method. It is shown in Table 7.

The chart shows the traffic composition at Stadium intersection. The vertical axis shows the number of vehicle in one hour using the intersection from all approach.

As indicated in previous part here also time headway based methods are used on the notion that passenger car following larger vehicles may have higher headways compared to time headway between two successive passenger cars at saturated flow conditions. The ratio of these two quantities may give PCE value.

In Table 7, a cycle passenger car equivalent is 0.41 and large trucks passenger car equivalent is 2.83. As different factors affect the passenger car equivalent, it is different for different intersection. By multiplying the passenger car equivalent of vehicle i with total number of vehicle i, the vehicle per hour would be determined.

Table 7: Passenger Car Equivalent at Stadium

vehicle	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83

4.1.4 Traffic Composition and Passenger Car Equivalents at Laghar

Similar to above method the traffic composition of Laghar intersection during the peak hour of each turning movement is shown in Figure 14. The passenger car equivalent used to change the effect of heavy vehicle to passenger car equivalent is shown in Table 8.

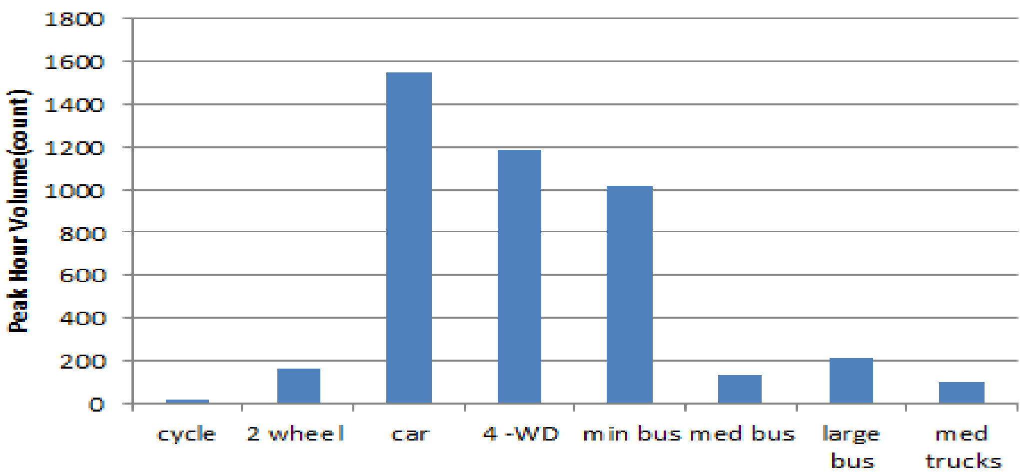


Figure 14: Traffic compositions at Laghar

Table 8: passenger car equivalent at Laghar

vehicles	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
pcui	**0.42**	**0.57**	**1**	**1.31**	**1.45**	**2.02**	**2.65**	**1.92**	**3.45**

The above figure indicates the number of each vehicle which uses the intersection at laghar during peak hour from all approaches. The traffic volume before 7:00am is low in all intersection and it start increasing at 7:00am. This continues up to 9:00 am. After 9:00 the traffic volume becomes similar and there is no extensive change overtime.

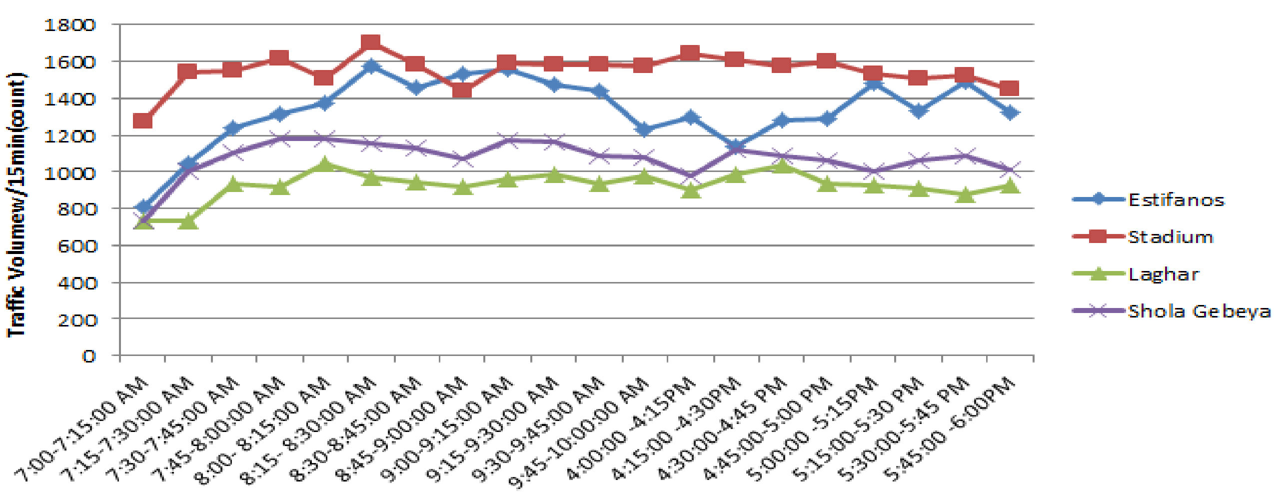


Figure 15: Traffic volume during time of day

4.2 Time Space Diagram

The Time-Space Diagrams can be used to graphically see how flow between intersections traffic. Synchro time-space diagrams display time along the horizontal axis and distance along the vertical axis. This diagram also shows the speed and position of the vehicles. Each line represents one or more vehicles. The slope of the line is proportional to the vehicles' speed. Horizontal lines represent stopped cars. The best timing plans are the ones with the fewest and shortest horizontal lines. The triangles of horizontal lines represent stopped vehicles queued at a red light. The width of the triangle is the longest waiting time. The height of the triangle represents the maximum queue. The higher the height of triangle, the longer queue and the shorter stop. A short, wide trapezoid represents a few vehicles making a long stop

4.2.1 Time Space Diagram of Intersection at Shola Gebeya

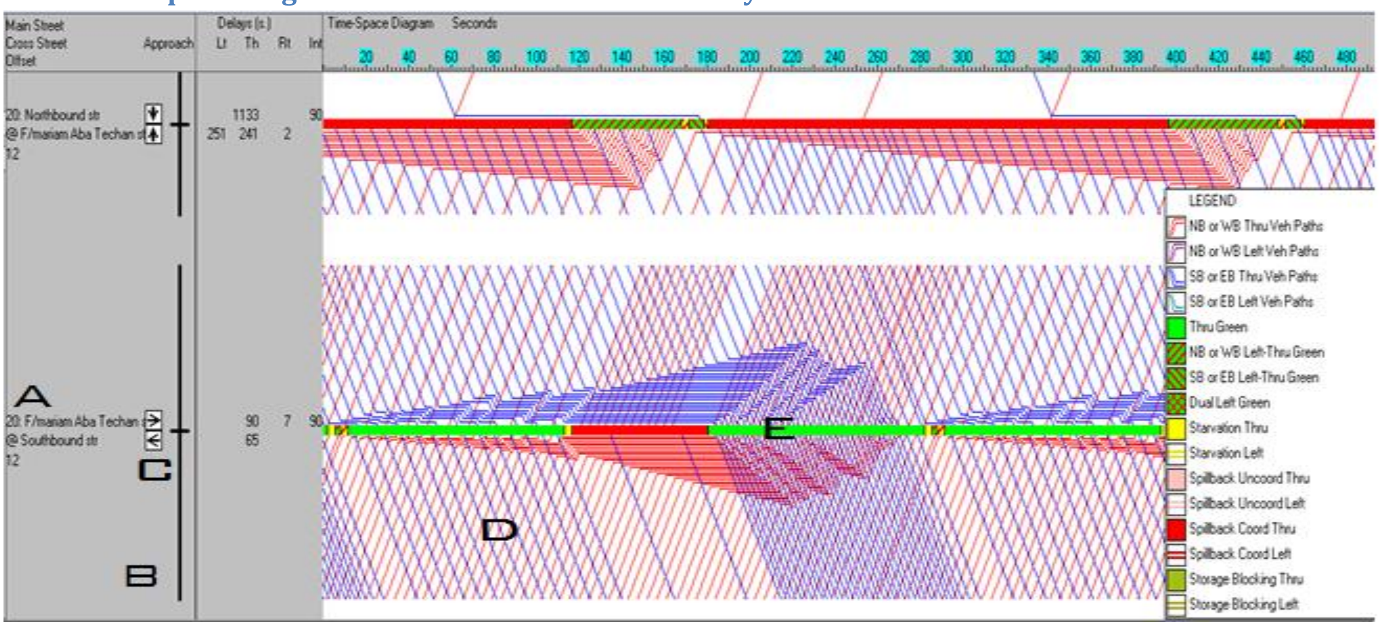


Figure 16: Time Space diagram at Shola gebeya

From the above diagram

A: - Represents Street Names - These are the street names of the intersection shown. The top name is the name of the street with the time-space diagram being shown. The bottom name is the name of the cross street.

B: -Street and Intersection Diagram- The vertical line represents the street with the time-space diagrams. The horizontal lines are crossing streets. These lines also show where the diagrams stop, when two or more streets are shown in intersection view.

C: - Represents Direction - These icons indicate the direction of the street in question. The top icon shows the direction of traffic moving downward in the diagram, usually Southbound or Westbound. The bottom icon shows the direction of traffic moving in the upward direction in the diagram, usually Northbound or Eastbound.

D: - Represents Traffic flow lines or Traffic Density Diagram- The diagonal and horizontal lines show traffic flow.

E: - Timing Bands. The red, green, and yellow bands indicate the phase of the signal for each part of the cycle. The different colors and markings are as follows:

- Green represents a green phase for through traffic in both the upwards and downwards direction.
- Red represents a red phase for both directions through and left movements. So from time space diagram it can be read easily without confusion.

4.2.2 Time Space Diagram at Estifanos

From the output of synchro signal timing optimization software packages the following diagram was extracted as time space diagram of intersection at Shola Gebeya. The Estifanos intersection is represented by the jomo kenyata and minilik II ave. It is found at the top for north south lanes and bottom line for east west lanes of networks. This intersection has both NB or WB left and thru green and spill back coordinates thru phases.

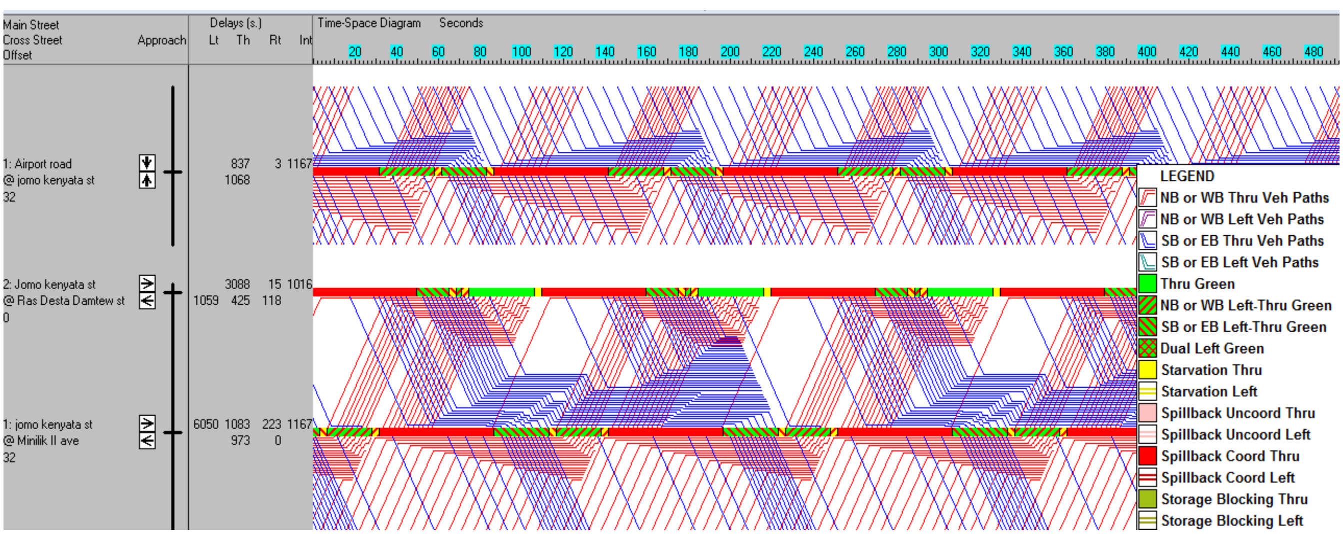


Figure 17: Time Space Diagram at Estifanos

Similar description is used as Section 4.2.1, Even though there is difference in number of street.

4.2.3 Time Space Diagram at Stadium

Below there is time space diagram of intersection at Stadium which is extracted from synchro. This intersection is found at the middle of networks. From figure 18 this intersection behaves as partial split phases and protected phases. There is also spill back coordinate thru phases. For more description refer Section 4.2.1.

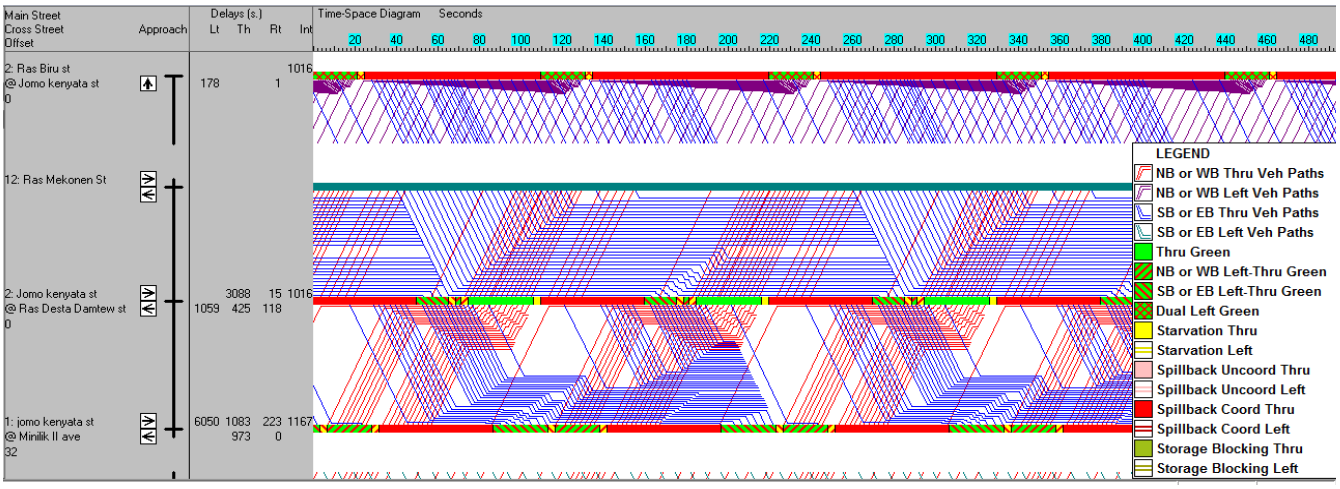


Figure 18: Time Space Diagram at Stadium

4.2.4 Time Space Diagram at Laghar

The following represents time space diagram at Laghar intersection. It is referred between Yilma Demissew and Ras Mekonnen Street. It contains NB or WB left thru green. The north south bound has lesser vehicle when compared with east west part. This is why large number of red phase is in north south direction. For more description refer Section 4.2.1.

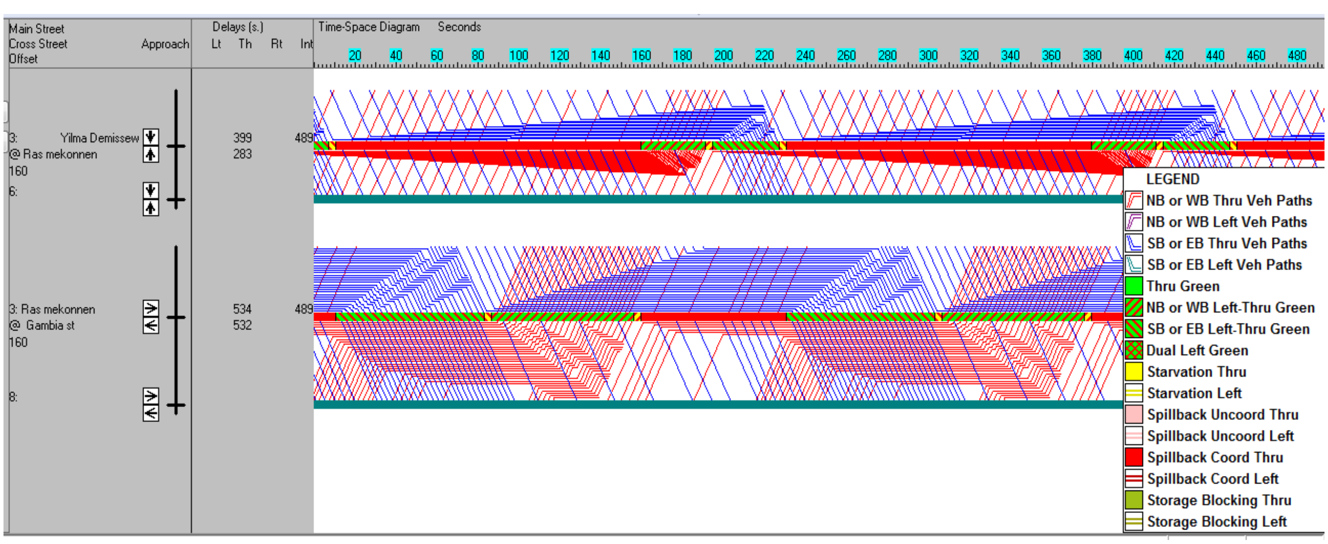


Figure 19: Time Space Diagram at Laghar

4.3 Data Input and Calibration

During optimization by synchro data can be entered, edited and viewed with the data entry setting buttons after links and nodes have been created in the map view. The data entry buttons are inactive and not accessible until either a link or node is selected on the map. Each data entry steps were discussed below.

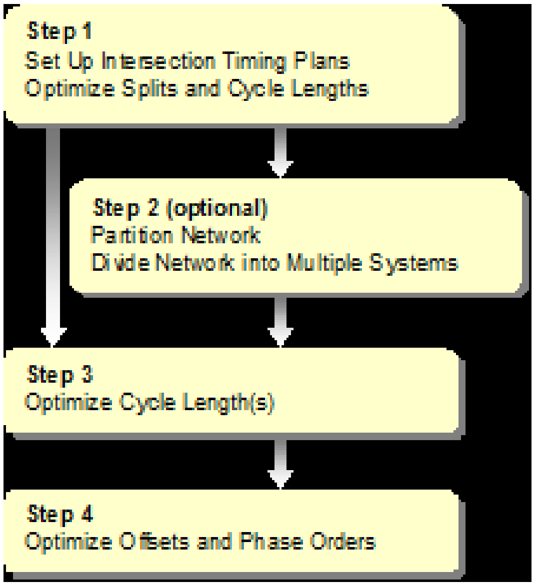


Figure 20: Step in optimization (source synchro manual, 2013) [39]

Because of it is difficult to list or include each step in this report the sample of Shola Gebeya was discussed in detail. But the optimization step is done for all intersection.

4.3.1 Background Map of Shola Gebeya Intersection

The basic layout of the map view has been enhanced to provide users with the ability to customize the location and content. For other intersections the map is shown in Appendix B.

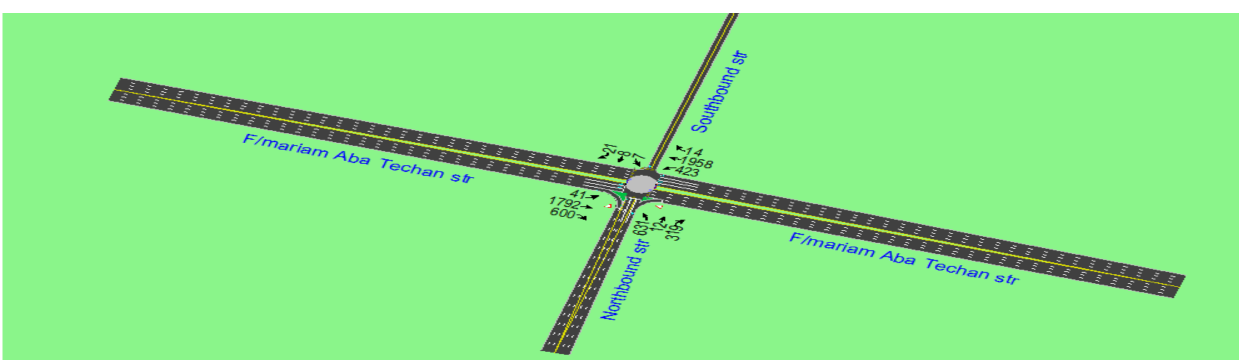


Figure 21: Map setting of shola intersection

4.3.2 Lane Data Entry and Calibration at Shola Gebeya

For each lane group the number of lanes and type of movements are identified and each exclusive and shared lane had been selected from synchro software package. The following Figure 22 is lane data entry format.

Use the following rules to determine which lanes belong to a lane group.

- Shared lanes always count as through lanes.
- Only exclusive turning lanes count as turning lanes.

So EB contains one shared lanes, two thru and one exclusive right lane. The WB contains two shared and two thru lanes. The NB contains two exclusive left turn lanes and one shared lane. The SB contains only one shared lanes because the traffic volume from SB is low. If an approach has a shared and an exclusive turning lane, the approach can be modeled by Synchro. The exclusive lane is in the turning group, and the shared lane is in the through group.

LANE SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)												
Traffic Volume (vph)	39	1707	571	403	1865	13	600	11	304	7	8	20
Street Name	F/mariam Aba Techan str			F/mariam Aba Techan str			Northbound str			Southbound str		
Link Distance (ft)	—	1325	—	—	1352	—	—	709	—	—	759	—
Link Speed (mph)	—	31	—	—	31	—	—	31	—	—	31	—
Set Arterial Name and Speed	—	EB	—	—	WB	—	—	NB	—	—	SB	—
Travel Time (s)	—	29.1	—	—	29.7	—	—	15.6	—	—	16.7	—
Ideal Satd. Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width (ft)	12	12	12	12	12	12	12	12	12	12	12	12
Grade (%)	—	0	—	—	0	—	—	1	—	—	1	—
Area Type CBD	—	☑	—	—	☑	—	—	☑	—	—	☑	—
Storage Length (ft)	0	—	0	0	—	0	0	—	0	0	—	0
Storage Lanes (#)	—	—	—	—	—	—	—	—	—	—	—	—
Right Turn Channelized	—	—	Yield	—	—	None	—	—	Yield	—	—	None
Curb Radius (ft)	—	—	50	—	—	—	—	—	50	—	—	—
Add Lanes (#)	—	—	1	—	—	—	—	—	1	—	—	—
Lane Utilization Factor	0.91	0.91	1.00	0.86	0.86	0.86	0.97	1.00	1.00	1.00	1.00	1.00
Right Turn Factor	—	1.000	0.850	—	0.999	—	1.000	0.855	—	—	0.922	—
Left Turn Factor (prot)	—	0.999	1.000	—	0.991	—	0.950	1.000	—	—	0.990	—
Saturated Flow Rate (prot)	—	4174	1294	—	5030	—	3111	670	—	—	1070	—
Left Turn Factor (perm)	—	0.661	1.000	—	0.704	—	0.950	1.000	—	—	0.833	—

Figure 22: Lanes Data Entry and Calibration of Shola Gebeya

4.3.3 Volumes Data Entry and Calibration at Shola Gebeya

Traffic volumes for design period were entered and the pedestrian numbers using the road are also entered. Here the design period is taken as one year since longest inspection period is one year (Sunkari, April 2004)[15]. During volume data entry and calibration, conflicting pedestrian, peak hour factor, growth factor, heavy vehicle (%) and bus blockage were entered. The outputs are adjusted flow, lane group flow and adjusted saturation flow.

Conflicting pedestrian is taken from site during traffic count. It is the number of pedestrian cross the road during peak hour. Growth factor is the value by which the traffic volumes change within the design period.

VOLUME SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)												
Traffic Volume (vph)	39	1707	571	403	1865	13	601	11	304	7	8	20
Conflicting Peds. (#/hr)	1057	—	684	684	—	1057	2343	—	2173	2173	—	2343
Conflicting Bicycles (#/hr)	—	—	2	—	—	4	—	—	0	—	—	0
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Growth Factor	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
Heavy Vehicles (%)	0	3	2	4	4	0	0	0	4	0	0	0
Bus Blockages (#/hr)	0	59	23	14	99	0	4	0	15	0	0	0
Adj. Parking Lane?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Parking Maneuvers (#/hr)	—	—	—	—	—	—	—	—	—	—	—	—
Traffic from mid-block (%)	—	0	—	—	0	—	—	0	—	—	0	—
Link OD Volumes	—	—	—	—	—	—	—	—	—	—	—	—
Adjusted Flow (vph)	45	1948	652	460	2129	15	686	13	347	8	9	23
Traffic in shared lane (%)	—	—	—	—	—	—	—	—	—	—	—	—
Lane Group Flow (vph)	0	1993	652	0	2604	0	686	360	0	0	40	0

Figure 23: Volume Data Entry and Calibration of Shola Gebeya.

4.3.4 Time Data Entry and Calibration at Shola Gebeya

In time setting both turn left and right types were entered. For instance, in the following Figure 24 EBL turn has split phases before optimization and protected phases after optimization. Also EBR has similar permitted phase before and after optimization because RTOR is allowed and the vehicle should yield to pedestrian. Green split was obtained after time data entry (Figure 24).

NODE SETTINGS	
Node #	20
Zone:	shola
X East (ft):	14140689
Y North (ft):	3289290
Z Elevation (ft):	7863
Description	shola
Control Type	Pretimed
Cycle Length (s):	280.0
Lock Timings:	☐
Optimize Cycle Length:	Optimize
Optimize Splits:	Optimize
Actuated Cycle(s):	280.0
Natural Cycle(s):	65.0
Max v/c Ratio:	2.63
Intersection Delay (s):	493.1
Intersection LOS:	F
ICU:	1.33

TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	PED	HOLD
Lanes and Sharing (#RL)													—	—
Traffic Volume (vph)	39	1707	571	403	1865	13	601	11	304	7	8	20	—	—
Turn Type	Prot	—	Perm	Prot	—	—	Split	—	—	Perm	—	—	—	—
Protected Phases	7	4		3	8	—	2	2	—		6	—		
Permitted Phases			4			—			—	6		—	—	—
Permitted Flashing Yellow	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Detector Phases	7	4	4	3	8	—	2	2	—	6	6	—	—	—
Switch Phase	0	0	0	0	0	—	0	0	—	0	0	—	—	—
Leading Detector (ft)	—	0	0	—	0	—	0	0	—	—	0	—	—	—
Trailing Detector (ft)	—	0	0	—	0	—	0	0	—	—	0	—	—	—
Minimum Initial (s)	5.0	5.0	5.0	5.0	5.0	—	5.0	5.0	—	1.0	1.0	—	—	—
Minimum Split (s)	20.0	20.0	20.0	20.0	20.0	—	20.0	20.0	—	4.5	4.5	—	—	—
Total Split (s)	20.0	169.0	169.0	20.0	169.0	—	83.0	83.0	—	8.0	8.0	—	—	—
Yellow Time (s)	3.0	3.0	3.0	3.0	3.0	—	3.0	3.0	—	2.0	2.0	—	—	—
All-Red Time (s)	0.5	0.5	0.5	0.5	0.5	—	0.5	0.5	—	0.0	0.0	—	—	—
Lost Time Adjust (s)	—	3.0	3.0	—	3.0	—	3.0	3.0	—	—	3.0	—	—	—
Lagging Phase?	☐	☑	☑	☐	☑	—	—	—	—	—	—	—	—	—

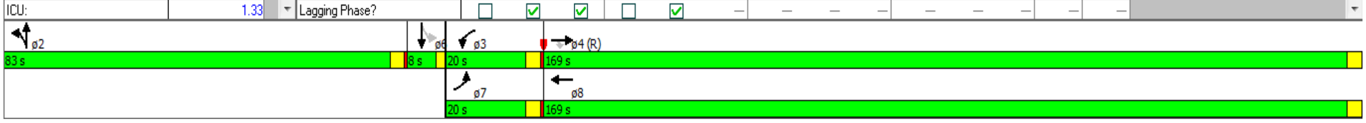


Figure 24: Time Data Entry and Calibration of Shola Gebeya

The types of turn treatment are:

1. Permitted (Perm): Left or right turn movements are not protected and vehicles must yield to oncoming traffic and pedestrians in the crosswalk.

2. Protected (Prot): Left turn movements are protected by a dedicated signal and turning traffic can only move during the arrow indication of this signal.

3 Split: Left and through traffic share a single protected phase. This type of phasing is commonly used if a lane is shared between left and through traffic. Split phasing insures that shared left-turn lanes are protected and offer a greater level of protection compared with permitted left-turns.

4 Free: A free right turn movement yields to pedestrians and is not assigned a signal phase. The permitted phase is automatically set to Free by the Free turn type.

4.3.5 Phase Data Entry and Calibration at Shola Gebeya

During phase setting there are five scenarios modeled; these are the 90th, 70th, 50th, 30th, and 10th percentiles. Traffic volumes for each approach are adjusted up or down to model these percentile scenarios. If traffic is observed for 100 cycles, the 90th percentile would be the 90th busiest, the 10th percentile would be the 10th busiest, and the 50th percentile would represent average traffic.

NODE SETTINGS	
Control Type	Pretimed
Cycle Length (s):	280.0
Lock Timings:	☐
Optimize Cycle Length:	Optimize
Optimize Splits:	Optimize
Actuated Cycle 90th (s):	280.0
Actuated Cycle 70th (s):	280.0
Actuated Cycle 50th (s):	280.0
Actuated Cycle 30th (s):	280.0
Actuated Cycle 10th (s):	280.0
Natural Cycle(s):	65.0
Max v/c Ratio:	2.63
Intersection Delay (s):	493.1
Intersection LOS:	F
ICU:	1.33
ICU LOS:	H
Offset (s) :	0.0
Referenced to:	Begin of Green
Reference Phase:	4 - EBT
Master Intersection:	☑
Yield Point:	Single

PHASING SETTINGS	2-NBTL	3-WBL	4-EBT	6-SBTL	7-EBL	8-WBT
Minimum Split (s)	20.0	20.0	20.0	4.5	20.0	20.0
Maximum Split (s)	83.0	20.0	169.0	8.0	20.0	169.0
Yellow Time (s)	3.0	3.0	3.0	2.0	3.0	3.0
All-Red Time (s)	0.5	0.5	0.5	0.0	0.5	0.5
Lagging Phase?	—	☐	☑	—	☐	☑
Allow Lead/Lag Optimize?	—	☑	☑	—	☑	☑
Optimize Phs Weights - Delays	1.0	1.0	1.0	1.0	1.0	1.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0
Minimum Gap (s)	3.0	3.0	3.0	3.0	3.0	3.0
Time Before Reduce (s)	0.0	0.0	0.0	0.0	0.0	0.0
Time To Reduce (s)	0.0	0.0	0.0	0.0	0.0	0.0
Recall Mode	Max	Max	Max	Max	Max	Max
Pedestrian Phase	☐	☐	☐	☐	☐	☐
Walk Time (s)	—	—	—	—	—	—
Flash Dont Walk (s)	—	—	—	—	—	—
Pedestrian Calls (#/hr)	—	—	—	—	—	—
Dual Entry?	☑	☑	☑	☑	☑	☑
Fixed Force Off?	☑	☑	☑	☑	☑	☑
90th %ile Green Time (s)	80 mr	17 mr	166 cd	6 mr	17 mr	166 cd
70th %ile Green Time (s)	80 mr	17 mr	166 cd	6 mr	17 mr	166 cd
50th %ile Green Time (s)	80 mr	17 mr	166 cd	6 mr	17 mr	166 cd

Figure 25: Phase Data Entry and Calibration Shola Gebeya.

4.3.6 Simulation Data Entry and Calibration at Shola Gebeya

In simulation setting number of storage lanes and its length in (feet) were entered. Also link offset, crosswalk width and median width are entered.

The **Storage Length** is the length of a turning bay in (meters) and is the same value found in the LANE settings. If an intersection has a left turn storage bay of (45 meters), enter ("45") in this box. If the left or right turn extends back to the previous intersection, enter "0".

Storage Lanes Code the number of lanes in the right or left storage bay. This value only appears when the Storage Length is greater than 0. By default, the number of **Storage Lanes** is equal to the number of turning lanes.

Lane Width is the width of a single lane in feet (meters).

Enter Blocked Intersection setting controls simulation modeling gridlock avoidance. The four options for modeling blocked intersections are "Yes", "No", "1" and "2". The default value is "No", for intersections.

The **Median Width** is used to set the width of the median. Left turn lanes are considered to be positioned in the median even if they are not defined as storage lanes.

SIMULATION SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)												
Traffic Volume (vph)	39	1707	571	403	1865	13	601	11	304	7	8	20
Storage Length (ft)	0	—	0	0	—	0	0	—	0	0	—	0
Storage Lanes (#)	—	—	—	—	—	—	—	—	—	—	—	—
Taper Length (ft)	—	—	—	—	—	—	—	—	—	—	—	—
Lane Alignment	Left	Left	Right	Left	Left	Right	Left	Left	Right	Left	Left	Right
Lane Width (ft)	12	12	12	12	12	12	12	12	12	12	12	12
Enter Blocked Intersection	No	No	No	No	No	No	No	No	No	No	No	No
Median Width (ft)	—	13	—	—	13	—	—	0	—	—	0	—
Link Offset (ft)	—	0	—	—	0	—	—	0	—	—	0	—
Crosswalk Width (ft)	—	13	—	—	13	—	—	13	—	—	13	—
TWLTL Median	—	☐	—	—	☐	—	—	☐	—	—	☐	—
Headway Factor	1.14	1.27	1.29	1.14	1.30	1.14	1.16	1.15	1.15	1.15	1.15	1.15
Turning Speed (mph)	15	—	9	15	—	9	15	—	9	15	—	9
Mandatory Distance (ft)	—	1166	—	—	1166	—	—	1166	—	—	1166	—
Positioning Distance (ft)	—	1365	—	—	1365	—	—	1365	—	—	1365	—
Mandatory Distance 2 (ft)	—	910	—	—	910	—	—	910	—	—	910	—
Positioning Distance 2 (ft)	—	1820	—	—	1820	—	—	1820	—	—	1820	—

Figure 26: Simulations Data entry and calibration at shola Gebeya.

4.4 Optimization

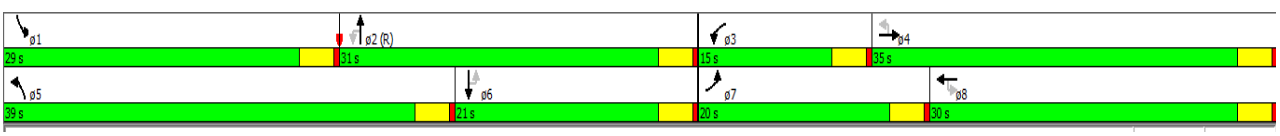
The parameters which will be optimized are cycle length, green split, phase sequences and offset from master intersection. In this research different values of parameter are used and among these the one with low fuel consumption, travel time, delay and low emission of COx would be selected as optimized parameter.

The signal optimization routine within Synchro has been enhanced to allow users the ability to favor specific phases during the optimization routine. This is accomplished by applying weighting factors (by phase) at individual intersections or throughout the entire network. Synchro contains a number of optimization functions

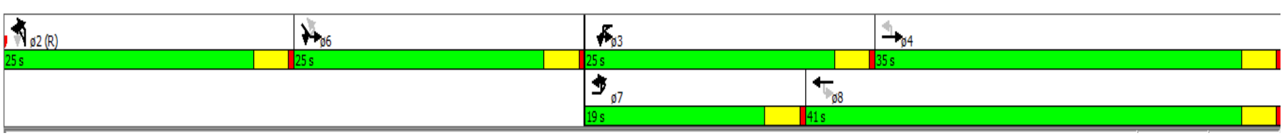
4.4.1 Green Split and Phase Sequence Optimization

Based on traffic volume, type of left and right phases, lanes sharing, and minimum split, the optimize-intersection split command automatically set the split for all turning movement. Time is divided based on each lane group traffic volume divided by its adjusted saturated flow rate. The split optimization will respect the minimum split setting for each phases wherever possible.

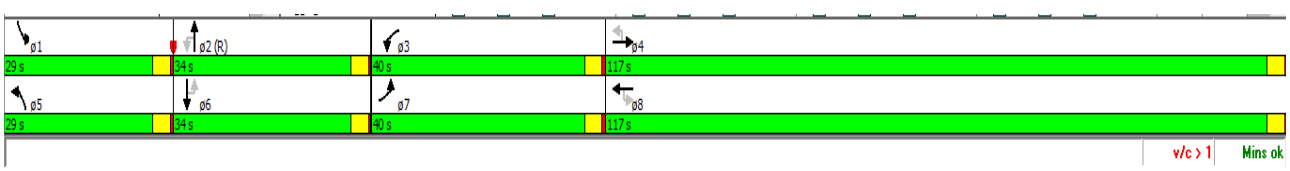
A Phase split and sequence at Shola Gebeya intersection after optimization



B Phase split and sequences at Estifanos



C Phase split and sequence at Stadium



D Phase split and sequences at Laghar

Figure 27: Green split and phase sequences of intersection

4.4.2 Cycle Length Optimization

Synchro attempts to provide enough time during split optimization to clear 95th percentile traffic flow. This ensures enough time is available for volume fluctuation of traffic. During the optimization of cycle length the green split automatically optimized with the newest cycle length. In some cases a longer cycle length will give lower delays or other performance benefits. The cycle length optimization attempts to determine the shortest cycle length with acceptable performances.

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. If no acceptable cycle length is found, the cycle length is set to the cycle length with the lowest PI. To set the Maximum Cycle Length, use the Options → Network Settings command.

With one or more movements at or over capacity, 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.

4.4.2.1 Shola Cycle Length Optimization

Since this intersection is isolated intersection there is no coordination effect and coordination factor is zero. Here it is possible to optimize by direct command on node setting or by network cycle length and manual method. But the measure of effectiveness should be compared and the efficient one should be taken. In this intersection there is previous cycle length of 130sec. But after optimization the cycle length changed to 280 with the following MOE parameters. Here is how synchro calculates MOE for different cycle length and optimize.

During selection of cycle length the synchro by itself shows the most efficient cycle length with lowest Performance index. In the following the synchro indicates that 280 sec is the one with low total delay, delay/vehicle, and fuel consumption.

Table 9: Selection of Cycle Length at Shola Gebeya

Cycle Length	Perform Index	Queue Delay (hr)	Total Delay (hr)	Delay / Veh (s)	Total Stops	Stops / Veh	Fuel (gal)
220	333	0	323	200	3611	0.62	313
230	334	0	324	200	3615	0.62	314
240	329	0	319	197	3636	0.62	310
250	330	0	320	198	3634	0.62	311
260	329	0	319	197	3643	0.63	311
270	328	0	318	196	3654	0.63	310
280	327	0	317	196	3668	0.63	309
290	329	0	319	197	3665	0.63	310
300	329	0	319	197	3671	0.63	310
310	329	0	318	197	3685	0.63	310
320	330	0	320	198	3690	0.63	311
330	332	0	321	199	3687	0.63	312
340	332	0	322	199	3691	0.63	313
350	333	0	323	200	3702	0.64	314
360	334	0	323	200	3711	0.64	314

From Existing cycle length the following measure of effectives MOE in Table 10 is taken from synchro. The command of create report and measure of effectiveness gives the result for baseline cycle length. Different parameters such as total delay, stops, and travel time and fuel consumptions were extracted and shown below.

For example total delay/vehicle is 219sec. This means one vehicle delay 219 sec due to the installation of traffic light on this intersection. The other is total travel time which is 398 hr. Here this figure represents total travel time of all vehicles from all approach on the interval which was represented by one hour flow. The fuel consumed is 336 gal for existing cycle length and the average stop is 3572 vehicles.

Table 10: MOE Parameter for Shola Gebeya

MOE parameter	Before optimization	After optimization
Direction	All	All
Vehicles	5828	5828
Control delay(sec/veh)	219	89
Total delay(hr)	354	143
Stops(#)	3572	3548
Total travel time(hr)	398	187
Distance traveled(mi)	1359	1359
Fuel consumed(gal)	336	181
CO emmission(kg)	23.47	12.66
NOx emmision(kg)	4.57	2.46

The command of optimization and cycle length gives the cycle length of 280 sec.

After optimization of cycle length, phase sequence and green split the following MOE is determined from synchro. The optimized cycle length is 280 sec as indicated in Table 9. The total delay /vehicle is 89 sec. This means one vehicle delay only 89 sec after optimization which is more efficient when compared with vehicle delay before optimization which is 219 sec. so due to optimization delay /vehicle is reduced by 60%. Also the travel time after optimization is 187 hr which is best when compared with travel time before optimization 398 hr.

4.4.2.2. Cycle Length Optimization of Arterial Intersection from Estifanos –Stadium- Laghar

This network contains three nodes or intersection and the optimization of the network is done by using half cycle length to have the same or half intersection of one intersection with others. Here the network cycle length is done manually as follows from synchro.

By using the command of optimization→ network cycle length→un coordination (never (0))→allow half cycle length→the following cycle length of 110 sec is determined. In this network the coordination factor is 122, which is determined from synchro→coordination factor command. Then since the coordinate factor is greater than 80, for which according to synchro manual coordination intensity is high. So using the command allow uncoordinated would be (never (0)) which coordinate all intersection.

The factors used to determine coordinatablity are the following

- ✓ Travel time
- ✓ Traffic to storage space
- ✓ Proportion of traffic in platoon
- ✓ Main street volume
- ✓ Increase in cycle length needed for coordination

The higher the coordinate factor the higher the benefit of network intersection from coordination.

Table 11: Selection of Cycle Length for Arterial Intersection

Cycle Length	Perform Index	Queue Delay (hr)	Total Delay (hr)	Delay / Veh (s)	Total Stops	Stops / Veh	Fuel (gal)
90	7689	0	7628	1133	21798	0.90	5939
100	7278	0	7222	1073	20179	0.83	5632
110	7196	0	7144	1061	18957	0.78	5567
120	7267	0	7215	1072	18656	0.77	5618
130	7516	0	7467	1109	17837	0.74	5798
140	7638	0	7589	1127	17474	0.72	5885
150	7743	0	7696	1143	17055	0.70	5961
160	7951	0	7904	1174	16940	0.70	6113
170	8185	0	8139	1209	16627	0.69	6283
180	8291	0	8245	1225	16519	0.68	6361
190	8496	0	8451	1255	16146	0.67	6509
200	8638	0	8594	1277	15813	0.65	6612
210	8800	0	8757	1301	15548	0.64	6730
220	9071	0	9028	1341	15481	0.64	6928
230	9177	0	9134	1357	15462	0.64	7006
240	9377	0	9334	1387	15270	0.63	7152
250	9628	0	9585	1424	15283	0.63	7336
260	9762	1	9720	1444	15105	0.62	7433
270	10037	1	9996	1485	14999	0.62	7635
280	10056	0	10015	1488	14773	0.61	7648
290	10369	0	10328	1534	14679	0.61	7876
300	10792	0	10751	1597	14702	0.61	8187
310	10781	0	10741	1596	14524	0.60	8178

From the Table 11, the newest cycle length of the network is 110sec, which is lower when compared with the previous cycle length 190 sec. The above intersection is not by itself fulfilling the optimization criteria and the 3rd intersection (Laghar) MOE is greater than the previous MOE parameter's. This indicates that the 110 cycle length is not enough to Laghar intersection. Then by using the double cycle intersection 220 sec it is more preferable. The MOE of each intersection before and after optimization are indicated.

Before optimization the cycle length of Estifanos intersection is 190 sec. The total delay is 1319sec/veh. The average stop is 5409 veh during the interval of simulation. The other is total travel time. The travel time for the current condition is 3435 hr. This represent the total travel time from all approach. The fuel consumption is 2579 gal and for other parameter the detail is indicated in the following Table 12.

Table 12: MOE Parameters at Estifanos

MOE parameter	Before optimization	After optimization
Direction	All	All
Vehicles	9226	9226
Control delay(sec/veh)	1319	1074
Total delay(hr)	3379	2751
Stops(#)	5409	6546
Total travel time(hr)	3435	2807
Distance traveled(mi)	1723	1723
Fuel consumed(gal)	2579	2125
CO emmission(kg)	180.2	148.7
NOx emmision(kg)	35	28.9

After optimization the total delay/vehicle is 1074se/veh which is low when compared with 1319 sec/veh before optimization. The travel time reduced from 3435hr to 2708 after optimization. But the vehicle stop after optimization is greater than the vehicle stop before optimization. This occurs due to the decrement of cycle length. When cycle length decrease the number of yellow time and loss of time increase. That is why the number of stop vehicle after optimization is greater than stop vehicle before optimization. In the following Table 13, all MOE parameters are shown in detail.

Before optimization the cycle length of stadium intersection is 190 sec. The total delay is 1706sec/veh. The stop is 6030 veh during the interval of simulation. The other is total travel time. The travel time for the current condition is 4350 hr. This represent the total travel time from all approach. The fuel consumption is 3262 gal and for other parameter the detail is indicated in the following table.

Table 13: MOE Parameters at Stadium

MOE parameter	Before optimization	After optimization
Direction	All	All
Vehicles	9023	9023
Control delay(sec/veh)	1706	954
Total delay(hr)	4275	2392
Stops(#)	6030	6806
Total travel time(hr)	4350	2467
Distance traveled(mi)	2314	2314
Fuel consumed(gal)	3262	1887
CO emmission(kg)	228	132
NOx emmision(kg)	44.3	25.67

After optimization the total delay/vehicle is 954se/veh which is low when compared with 1706 sec/veh before optimization. The travel time reduced from 4350hr to 2467hr after optimization. But the vehicle stop after optimization is greater than the vehicle stop before optimization. This occurs due to the decrement of cycle length. When cycle length decrease the number of yellow time and loss of time increase. That is why the stop vehicle after optimization is greater than stop vehicle before optimization.

Before optimization the cycle length of Laghar intersection is 190 sec. The total delay is 514sec/veh. The stop is 3572 veh during the interval of simulation. The other is total travel time. The travel time for the current condition is 900 hr. This represent the total travel time from all approach. The fuel consumption is 704 gal. The total stop is 3572 veh, the CO emission is 49.21kg and for other parameter the detail is indicated in the following figure.

Table14: MOE parameters at Laghar

MOE parameter	Before optimization	After optimization
Direction	All	All
Vehicles	5986	5986
Control delay(sec/veh)	514	351
Total delay(hr)	855	584
Stops(#)	3572	3369
Total travel time(hr)	900	629
Distance traveled(mi)	1392	1392
Fuel consumed(gal)	704	504
CO emmission(kg)	49.2	35.2
NOx emmision(kg)	9.58	6.86

After optimization the total delay/vehicle is 351se/veh which is low when compared with 514 sec/veh before optimization. The travel time reduced from 900hr to 629hr after optimization. Since cycle length increase due to optimization the vehicle stop after optimization is less than the vehicle stop before optimization. This occurs due to the increment of cycle length. When cycle length increase the number of yellow time and loss of time decrease. That is why the stop vehicle after optimization is less than stop vehicle before optimization.

4.4.3 Offset Optimization

Offset is time difference between references phases of master intersection and the adjacent intersection. The master intersection was taken as Stadium. The reference phases are thru phase which is (Φ2+Φ6) and the start of green for these phases was taken as reference phases. So the lag of green time from master intersection is determined as follows.

4.4.3.1 Shola Offset Optimization

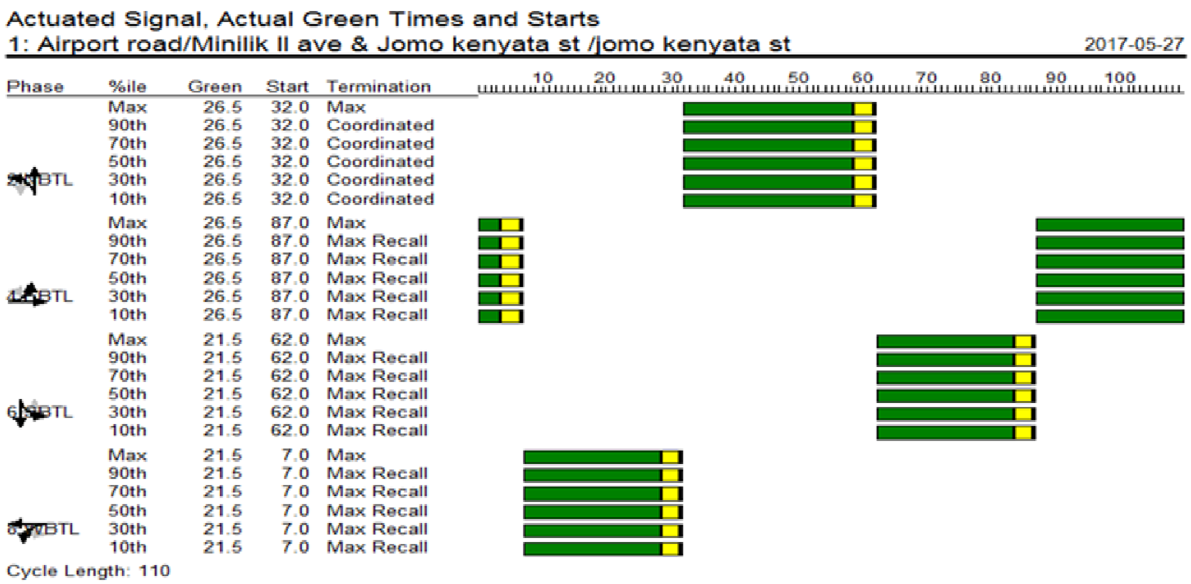
The intersection at Shola Gebeya is isolated intersection. Therefore it acts as it was master intersection because of this the offset optimization is unnecessary and beginning of green time can be at any time.

4.4.3.2 Offset of Estifanos and Laghar.

For these network intersections the master intersection was taken as Stadium nodes. The offset of this intersection is 0sec and the offset of other two intersections is measured from the relativity of Stadium intersection. Here the reference phase is beginning of green time and thru phases (Φ2+Φ6). From offset optimization command the offset of Estifanos intersection is 32 sec. This means the beginning of green time for Estifanos intersection should lag by 32 sec after the green time of Stadium start for above phases. This is time is enough to reach the Estifanos intersection on green time. For example for vehicles which moves by 50km/hr travel time from Stadium to Estifanos (L=383m) is L/V=28 sec. So the offset is optimum as one traffic stop for only 4sec for speedy vehicles.

Simultaneously the intersection of Laghar was optimized and the optimum offset is 160 sec. This means the green time of Laghar intersection for phases (Φ2+Φ6) should lag by 160 sec after the green time of Stadium intersection. So the number of vehicle stop is very low and the given offset is optimum.

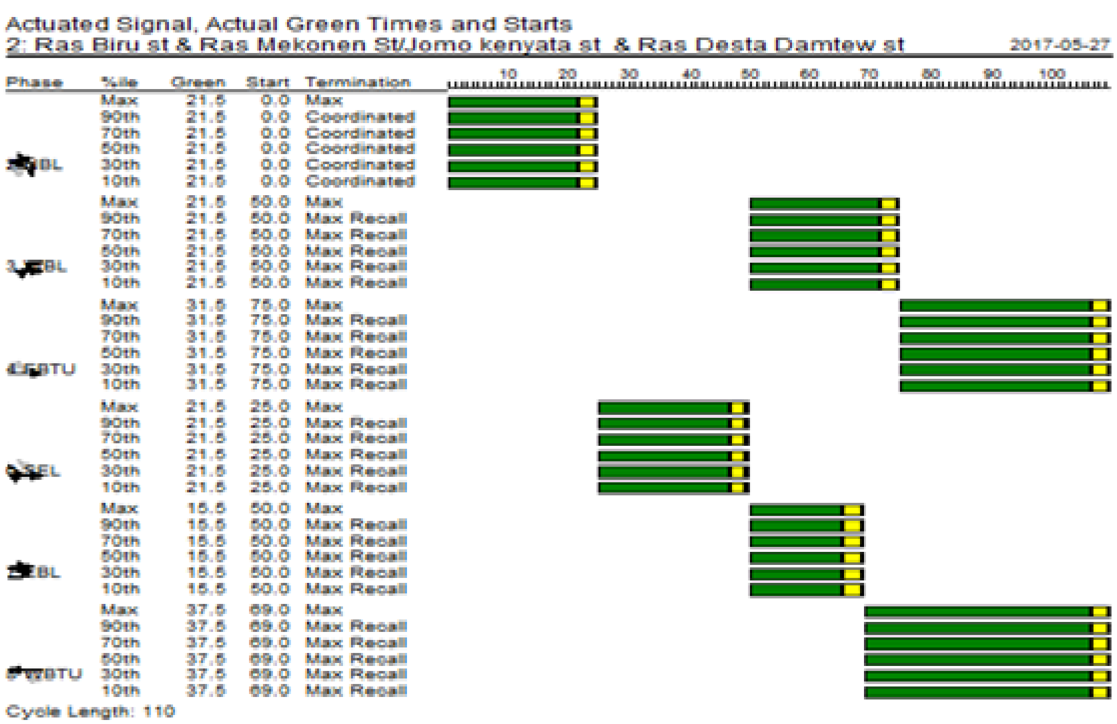
Green Bar and Start Time: The green bar and start time for each intersection is determined from create report and the followings are the green bar and start time for intersection at Estifanos for other intersection


Actuated Signal, Actual Green Times and Starts
1: Airport road/Minilik II ave & Jomo kenyata st /jomo kenyata st 2017-05-27

Phase	%ile	Green	Start	Termination
	Max	26.5	32.0	Max
	90th	26.5	32.0	Coordinated
	70th	26.5	32.0	Coordinated
	50th	26.5	32.0	Coordinated
2 BTL	30th	26.5	32.0	Coordinated
	10th	26.5	32.0	Coordinated
	Max	26.5	87.0	Max
	90th	26.5	87.0	Max Recall
	70th	26.5	87.0	Max Recall
	50th	26.5	87.0	Max Recall
4 BTL	30th	26.5	87.0	Max Recall
	10th	26.5	87.0	Max Recall
	Max	21.5	62.0	Max
	90th	21.5	62.0	Max Recall
	70th	21.5	62.0	Max Recall
	50th	21.5	62.0	Max Recall
6 BTL	30th	21.5	62.0	Max Recall
	10th	21.5	62.0	Max Recall
	Max	21.5	7.0	Max
	90th	21.5	7.0	Max Recall
	70th	21.5	7.0	Max Recall
	50th	21.5	7.0	Max Recall
8 WBTL	30th	21.5	7.0	Max Recall
	10th	21.5	7.0	Max Recall

Cycle Length: 110

Figure 28: Green Bar and Starting Time for Intersection at Estifanos


Actuated Signal, Actual Green Times and Starts
2: Ras Biru st & Ras Mekonen St/Jomo kenyata st & Ras Desta Damtew st 2017-05-27

Phase	%ile	Green	Start	Termination
	Max	21.5	0.0	Max
	90th	21.5	0.0	Coordinated
	70th	21.5	0.0	Coordinated
	50th	21.5	0.0	Coordinated
BL	30th	21.5	0.0	Coordinated
	10th	21.5	0.0	Coordinated
	Max	21.5	50.0	Max
	90th	21.5	50.0	Max Recall
	70th	21.5	50.0	Max Recall
	50th	21.5	50.0	Max Recall
3 EBL	30th	21.5	50.0	Max Recall
	10th	21.5	50.0	Max Recall
	Max	31.5	75.0	Max
	90th	31.5	75.0	Max Recall
	70th	31.5	75.0	Max Recall
	50th	31.5	75.0	Max Recall
4 EBTU	30th	31.5	75.0	Max Recall
	10th	31.5	75.0	Max Recall
	Max	21.5	25.0	Max
	90th	21.5	25.0	Max Recall
	70th	21.5	25.0	Max Recall
	50th	21.5	25.0	Max Recall
SEL	30th	21.5	25.0	Max Recall
	10th	21.5	25.0	Max Recall
	Max	15.5	50.0	Max
	90th	15.5	50.0	Max Recall
	70th	15.5	50.0	Max Recall
	50th	15.5	50.0	Max Recall
BL	30th	15.5	50.0	Max Recall
	10th	15.5	50.0	Max Recall
	Max	37.5	69.0	Max
	90th	37.5	69.0	Max Recall
	70th	37.5	69.0	Max Recall
	50th	37.5	69.0	Max Recall
WBTU	30th	37.5	69.0	Max Recall
	10th	37.5	69.0	Max Recall

Cycle Length: 110

Figure 29: Green Bar and Starting Time for Intersection at Stadium

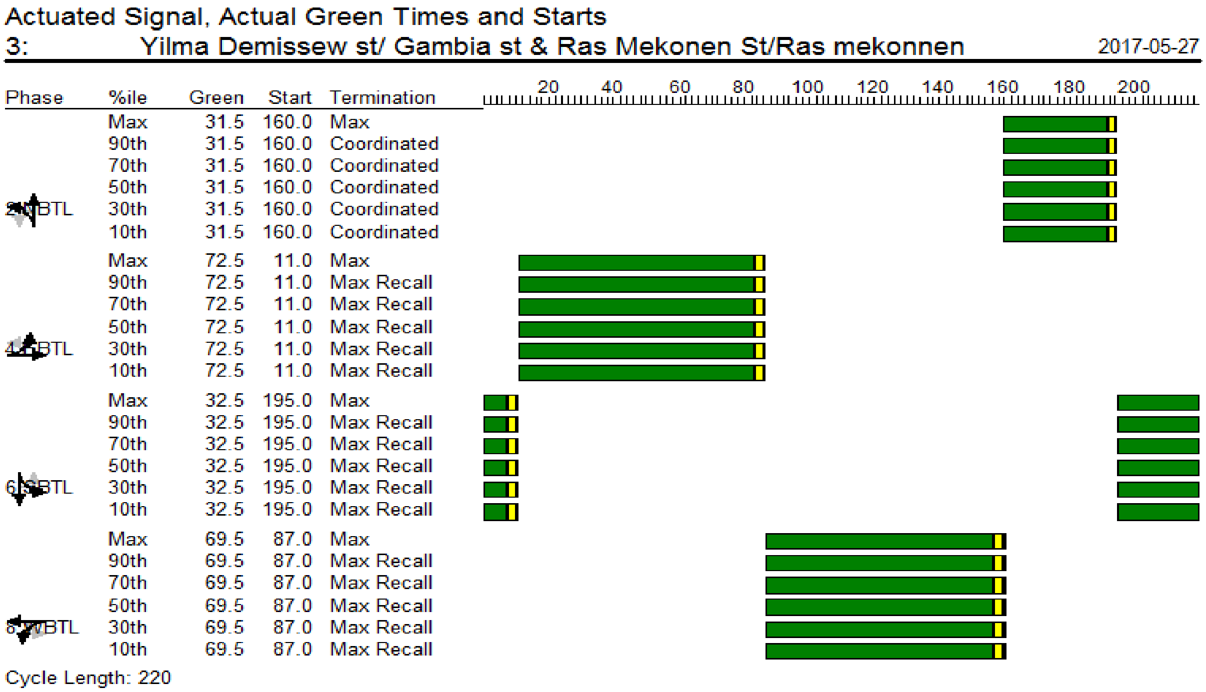


Figure 30: Green Bar and Starting time for Intersection at Laghar

4.5 Result Discussion

As the aim of this research is to identify the effect of optimization on traffic flow and evaluating performance, the above MOE parameters should be compared with the previous cycle length. Here the parameters are total delay; travel time and other are listed below.

The blue color represents the MOE parameter before optimization and red color represent parameter after optimization.

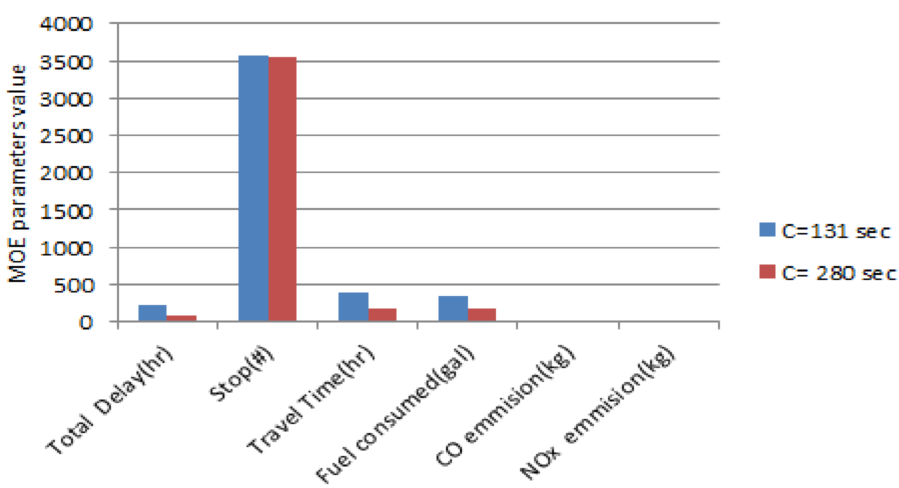


Figure 31: MOE Parameter for Shola Gebeya Intersection

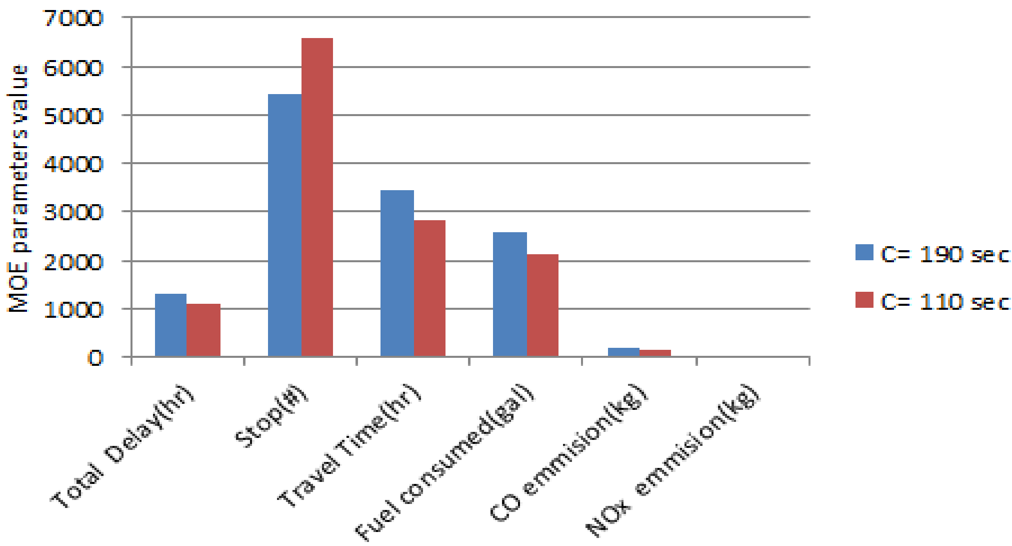


Figure 32: MOE Parameter for Estifanos Intersection

In Estifanos intersection the stop of the cycle length before optimization is less than stop after optimization. Some times synchro optimizes different parameter for different cycle length. When the cycle length become small a lot of yellow times appear which directly affect the number of vehicles stop.

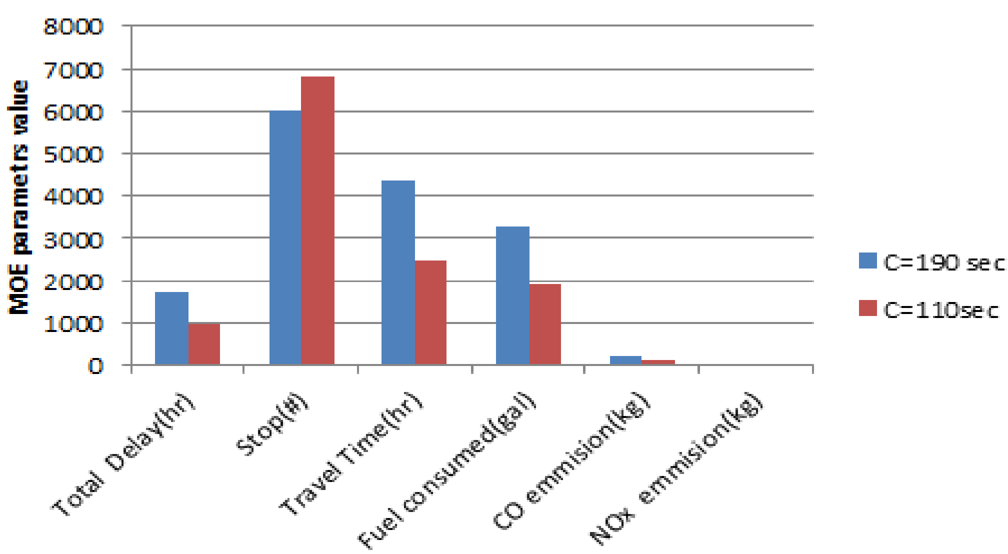


Figure 33: MOE parameter for Stadium Intersection

Also in this intersection the cycle length is less than the previous so that the present traffic stop is greater than the number of traffic stop before optimization. But other performance measurement satisfies the criteria of optimization.

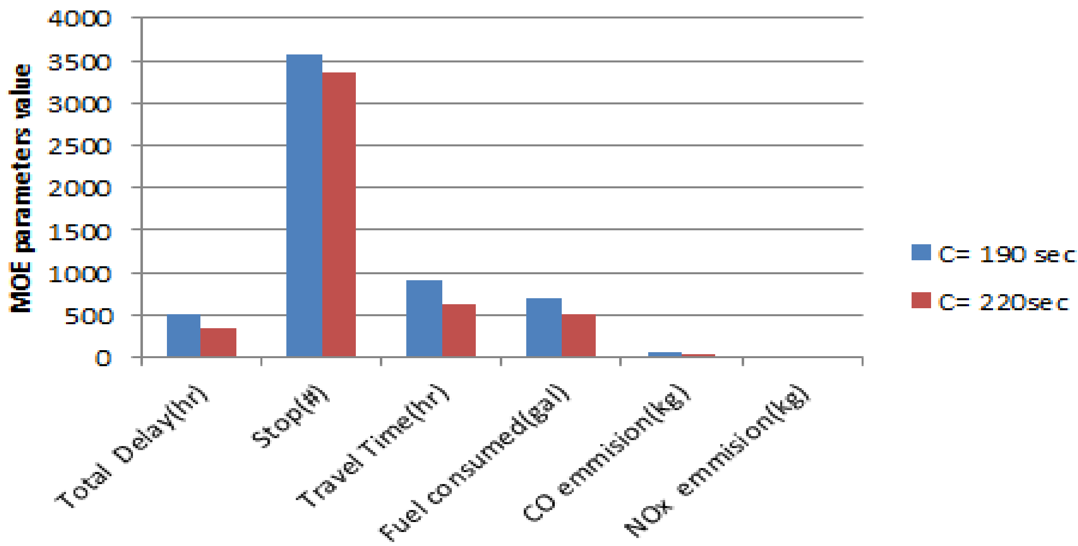


Figure 34: MOE Parameter for Laghar Intersection

The above chart can be summarized as follow depending on the percentage of reduction of each measure of effectiveness parameter. The negative sign indicates that due to optimization number of stop vehicle increase. It occurs when cycle length decrease by optimization.

Table 15: Efficiency of Optimization in Percentage

MOE parameters	Intersections			
	Shola Gebeya	Estifanos	Stadium	Laghar
Total Delay	60	18.6	44.1	31.7
Stop	0.6	-21.4	-12.9	5.7
Travel Time	53	18.3	43.3	30.1
Fuel consumed	46.13	17.6	42.2	28.4
CO emission	46	17.6	42.2	28.4
NOx emission	46	17.6	42.1	28.4

The HCM 2010 use the following range for classification of intersection depending on delays.

Table 16: Level of Service of Intersections (HCM 2010) [43]

LOS	A	B	C	D	E	F
Delay(sec)	<10	>10 and≤ 20	>20 and ≤ 35	>35 and≤ 55	>55 and ≤ 80	>80

According to above classification Shola Gebeya has 89sec delay. So it under LOS F. Intersection at Estifanos has 1074sec delay which is LOS F. The laghar intersection has 351sec delays and it is under LOS F. the intersection at Stadium has 954sec delays and it is also under LOS F.

Depending on the delay time interval from Synchro manual the level of service for each intersection is under LOS F. So even if delays were reduced by optimization, the better LOS could not be achieved. It may need construction additional facility.

The intersection capacity utilization after optimization for Estifanos intersection is 244.8%, the intersection capacity utilization for Stadium is 173.2% and for Laghar the intersection capacity utilization is 165.2%. This indicates that the intersections are served above their capacity. This ICU values gives insight to how intersection is functioning and how much extra capacity is available to handle traffic fluctuation and incidents.

Table 17 Summary for the Intersections Parameters

Parameters	Estifanos	Stadium	Laghar	Shola Gebeya
Area types	CBD	CBD	CBD	CBD
Cycle length(sec)	110	110	220	280
Offset(sec)	32	0	160	0
Natural cycle length	90	80	90	70
ICU(%)	244.8	173.2	165.2	121.1
ICU LOS	H	H	H	H
Max V/C ratio	14.43	28	2.72	3.08
Control type	**pre timed**	**pre timed**	**pre timed**	**pre timed**

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Now a day the traffic congestion around urban signal intersection is become increasing. Similarly traffic accidents and depression increase due to the traffic jam and delay. So these problems should be alleviated by reducing traffic delay and congestion around signal intersection. One of the powerful methods of alleviating this problem is traffic signal optimization. It is a cost effective way to improve the flow of traffic along a corridor. The optimizing of traffic signals is a way to maximize the capacity of the intersection without having to perform costly infrastructure improvements.

In Addis Ababa, most of intersections are under high traffic delay with low level of service. But without building high cost infrastructure, methods which are used to reduce delays were stated in this research.

As methodology to conduct this research, a one single and one network intersection was taken as general representative for other intersection. The traffic and pedestrian flows was taken by video during morning and afternoon peak period. Time headway of each vehicle was also determined by stop watch. Primary data were collected by direct measurement during field investigation. Secondary data were also used for the case of optimization.

The analysis or optimization of test intersections (Estifanos - Stadium - Laghar and Shola Gebeya) has been done by Synchro software packages. A traffic simulation model was also developed for assessment purposes by Sim traffic software packages. Coordination of network intersection from Estifanos - Stadium - Laghar has been done. The level of service for each intersection is determined.

Optimization in this thesis was found to give improved performance over the existing condition at test intersections. The parameters of measure of effectiveness (MOE) were determined after simulation of traffic movements. The delay reductions between 17% to 60% were obtained. Also the travel time reductions between 18% to 53% were obtained. In addition even if type of car and its performance cost is not included the fuel consumption reductions between 17% to 46% were obtained.

5.2 Recommendations

- During the calculation of Passenger Car Equivalent, the weather condition is normal. But this is affected during rainy season. So it would be preferable if the future study consider harsh weather condition.
- Traffic signal timing is sensitive to traffic volume. The minimum change of traffic volume cause high effects on signal parameters. So it would be desirable to check and review traffic signal frequently as it is not become outdated.
- It would be desirable the future optimization includes the adjacent intersection such as Biherawi and Mexico signal intersection because the effect of these intersections has direct effect on the delay of selected corridor.
- Regarding to the applicability of this study for directly using on the site, the output of optimized cycle length, green split, phase sequence and offset discussed under Section 4.4 were recommended by the researcher for practical uses at each intersection. As it was preferable over the previous signal parameters, the optimized parameters have the potential to reduce delay, fuel consumption, travel time and so on.

5.3 Further Studies

As means of expanding research area for further study from this thesis, the following additional works needs to be done.

- Traffic based strategy of signal control
- Passenger car equivalents
- Testing the SYNCHRO and SIMTRAFFIC optimized and simulated parameter in other optimization approach such as TRANSYT, Genetic Algorithmic and CORSIM.

REFERENCES

1) Kidus Ayalneh Admasu, (2016), Development of Models of Interrupted Traffic Flow Condition of Addis Ababa, Addis Ababa University.

2) Khewal Bhupendra Kesur, (2007), Advances in Genetic Algorithm Optimization of Traffic Signals/ Johannesburg

3) B. Brian park, Ph.D., j. D. Schneeberger, (2002), Evaluation of Traffic Signal Timing Optimization Methods, Virginia Transportation Research Council.

4) FDRE Ministry of Transport, (2011), Transport Policy of Addis Ababa, Addis Ababa, Ethiopia.

5) Gazis D. and R. Potts, (1963), The Oversaturated Intersection in Proceedings of the 2nd International Symposium on Traffic Theory.

6) Rahmann, W, (1973), Storage/Output Design of Traffic Signals. Australian Road Research.

7) Pignataro, L., W. Mcshane, K. Crowley, and T. Casey, (1978), Traffic Control in Oversaturated Street Networks Nchrp Report 194, Transportation Research Board. Washington, D.C.

8) Lee et., K Crowley, and L. Pignataro, (1975), Better Use of Signals under Oversaturated Flows. Transportation Research Board. Special Report 153. Washington, D.C.

9) Nadeem A. Chaudhary, Chi-Leung Chu, Srinivasa R. Sunkari, And Kevin N. Balke, (2010), Guidelines for Operating Congested Traffi Signals,Texas Transportation Institute.

10) Ziad A. Sabra, Wang and Associates Keith Riniker, (1999), Maximizing Benefits of Signal Timing Optimization.

11) Choi, B-K, (1997), Adaptive Signal Control for Oversaturated Arterials. Ph.D., Thesis, Polytechnic University.

12) Lieberman E. And J. Chang, (2005), Optimizing Traffic Signal Timing through Network Decomposition. Transportation Research Record 1925.

13) Clough Harbor and Associates,(2012), Traffic Signal Optimization Project, 441 South Salina Street Syracuse, NY 13202.

14) Allient Engineering Inc,(2012), Signal Timing Optimization and Coordination, 233 Park Avenue South, Ste 300 Minneapolis, MN 55415

15) Srinivasa Sunkari, (2004), The Benefits of Retiming Traffic Signals/Ite Journal/ Texas Transportation Institute In College Station, Tx, USA.

16) Umama Ahmed, (2009), Passenger Car Equivalent Factors for Level Freeway Segments Operating Under Moderate and Congested Conditions, Marquette University.

17) Seguin, E., Crowley, K., and Zweig, W. (1982). Passenger Car Equivalents on Urban Freeways. Report DTFH61-80-C-00106.

18) Greenshields, B.D., Shapiro, D., and Ericksen, E.L. (1947).Traffic Performance at Urban Intersections, Technical Report No. 1. Bureau of Highway Traffic, Yale University.

19) Werner, A., & Morrall, J. F. (1976). Passenger car equivalencies of trucks, buses, and recreational vehicles for two-lane rural highways. Transportation Research Record, (615).

20) Demarchi, S. H., & Setti, J. R. (2003). Limitations of Passenger-Car Equivalent Derivation for Traffic Streams with more than one Truck type. Transportation Research Record: Journal of the Transportation Research Board, 1852(1), 96-104.

21) Anil Kamarajugadda Dr. Byungkyu Brian Park, (2003), Stochastic Traffic Signal Timing Optimization, a US. Dot University Transportation Center.

22) Zhonghi Li, Lili Du and Yei Lu (2016) Optimal Signal Timing Design For Urban Street Networks Under User Equilibrium Based Traffic Conditions, Illinois Institute of technology.

23) Dion et al, (2003), Analytical Delay Models Estimated by Microscopic Simulator.

24) Rouphail, N. Progression Adjustment Factors at signalized Intersections. Transportation Research Record 1225, TRB, National Research Council Washington, D.C., 1989,

25) Minneassota Departement of Transportation manual, (2013), Traffic Signal Timing and Coordination Manual, USA.

26) Akcelik, R. (1988) The Highway Capacity Manual Delay Formula for Signalized Intersections, ITE Journal.

27) Tamir Balasha, (2014) Simulation-based Optimization of Actuated Traffic Signal Plans, haaifa 32000, Israel.

28) Loannis Psarros, (2016), Presentation on Synchro Studio, The University of Memphis.

29) Synchro manual, (1999), Optimization and Simulation of Traffic Signal, Trafficware,522 Gillingham.

30) Texas Transportation Institute, (2006), PASSER V, Texas College Station, 47 Gilchrist Bldg.

31) Goldberg, D. E, (1989), Genetic Algorithms in Search, Optimization and Machine Learning. Addison-Wesley Publishing Co., Massachusetts.

32) Kanakabandi Shalini1, Brind Kumar, (2014), Estimation of the Passenger Car Equivalent: Vol 4. Published by Ijetae. U.P, India.

33) Richard Dowling, (2002),Guidelines for Applying Traffic Microsimulation Modeling Software, California Department of Transportation.

34) B D Ventor, M J Vermuelin and J Barcelo', (2001), The Advantage of Microsimulation in Traffic Modeling Universitat Politcnica Catalunya.

35) FHWA, (2004), Guidelines for Applying Traffic Microsimulation Modeling Software, US Department of Transportation

36) B D Ventor, M J Vermuelin and J Barcelo', (2001), The Advantage of Microsimulation in Traffic Modeling Universitat Politcnica Catalunya.

37) Richard Dowling, joseph Holland, Allen Huang, (2002),Guidelines for Applying Traffic Microsimulation Modeling Software, California.

38) Trafficware inc, (2014), traffic signal optimization and simulation, Sugarland TX 77487.

39) Synchro manual, (2013), optimization and simulation of traffic signal, trafficware,522 Gillingham.

40) Naveem Kumar, Santha Kumar and Samson Mathew, Traffic Signal Coordination as ana measure fo Urban Arterials in Chennai City, Tiruchippali.

41) Tom V. Mathew, (2014), Coordinated Traffic Signal, IIT Bombay, India

42) MDOT Manual, (2008), Michigan Signal Optimization Guidelines, 5th Edition, USA.

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

44) Mahendra Kumarmet kari, Anuj Kishor Budhkar, Akhilesh Kumar Maurya, (2012), Review of Passenger Car Equivalence Studies in Indian Context, Published By Ijca..

45) Solomon Shanko, (2015), Segmental Assessment of Level of Traffic Congestion on Kality Ring Road to Dukem Bridge/AAU, Ethiopia.

APPENDIXES

Appendix A: Traffic Count Data by Interval of 15 Minute and PCE.

Table A1: Traffic Count Data for Shola Gebeya Intersection

time		turning types											
start time	end time	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
7:00	7:15	2	241	57	53	231	1	119	1	24	1	1	3
7:15	7:30	8	299	106	55	356	1	142	1	28	1	2	1
7:30	7:45	3	326	119	64	395	1	158	1	35	1	1	3
7:45	8:00	5	349	115	79	432	2	142	1	53	0	1	1
8:00	8:15	5	374	139	70	400	3	131	2	51	1	0	1
8:15	8:30	2	385	130	86	374	2	128	1	48	1	1	0
8:30	8:45	12	374	94	97	335	2	149	1	63	2	0	1
8:45	9:00	6	319	89	106	332	2	124	1	89	1	2	3
9:00	9:15	8	396	139	64	406	2	104	1	49	1	1	3
9:15	9:30	9	377	125	81	378	1	121	1	67	1	0	4
9:30	9:45	5	331	114	88	368	0	117	2	57	1	3	2
9:45	10:00	4	361	105	79	361	1	109	1	54	0	1	3
4:00	4:15	8	355	83	86	266	2	89	3	78	1	0	4
4:15	4:30	10	382	117	91	318	3	127	1	69	2	1	2
4:30	4:45	11	384	113	62	326	4	119	2	64	1	0	5
4:45	5:00	7	371	121	60	311	3	123	2	59	2	0	3
5:00	5:15	7	367	107	45	290	1	139	1	48	0	1	1
5:15	5:30	9	360	112	67	333	0	129	0	46	1	0	1
5:30	5:45	13	379	97	53	349	1	147	1	43	0	0	2
5:45	6:00	8	386	77	42	314	0	122	1	58	0	1	2

Table A2: PCU and Time Headway at Shola Gebeya

no trial	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med truck	large trucks
1	0.82	0.74	1.43	1.11	1.85	3.46	5.99	2.84	5.39
2	0.52	0.5	1.83	1.93	1.47	3.04	5.19	2.13	6.49
3	0.71	0.92	1.22	1.93	1.52	4.98	6.09	2.85	11.69
4	0.81	1.36	2.34	1.83	1.32	5.61	11.57	4.06	9.56
5	*	0.5	1.11	1.43	2.45	3.64	2.84	3.35	3.86
6	*	1.12	1.53	1.93	1.63	6.01	4.04	3.38	12.07
7	*	1.42	2.14	1.73	1.22	4.78	2.74	2.74	6.7
8	*	0.81	1.22	1.73	2.54	5.38	7.73	1.32	5.69
9	*	0.67	1.63	2.44	2.34	2.97	4.57	3.24	3.68
10	*	*	2.43	1.72	1.93	1.83	5.49	2.33	*
Avg	0.715	0.893333	1.688	1.778	1.827	4.17	5.625	2.824	7.23666667
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29

* Represent no vehicle

Table A3: Traffic Data for Stadium Intersection

Time		turning types											
start time	end time	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
7:00	7:15	8	274	102	95	278	146	49	72	59	141	43	8
7:15	7:30	5	416	111	91	289	203	68	90	75	130	50	12
7:30	7:45	5	335	120	126	262	210	70	104	78	167	65	7
7:45	8:00	2	400	107	118	271	129	91	108	96	213	71	9
8:00	8:15	6	369	127	133	212	167	78	87	89	175	62	7
8:15	8:30	3	329	162	116	313	268	80	74	131	146	70	11
8:30	8:45	4	308	119	82	280	229	97	106	88	185	73	11
8:45	9:00	3	285	98	103	297	222	61	108	54	136	67	5
9:00	9:15	9	320	85	94	301	251	64	131	61	188	76	12
9:15	9:30	13	408	106	146	261	201	79	87	57	160	51	13
9:30	9:45	12	357	90	96	265	239	81	130	69	157	75	10
9:45	10:00	9	368	85	113	287	262	72	112	73	133	50	9
4:00	4:15	15	378	101	134	279	234	57	108	52	215	61	6
4:15	4:30	13	373	116	121	262	221	53	82	58	217	81	8
4:30	4:45	19	320	106	127	259	217	47	91	56	242	86	5
4:45	5:00	12	368	122	108	288	192	49	95	71	223	69	5
5:00	5:15	17	363	86	97	276	203	55	89	82	209	51	7
5:15	5:30	11	357	93	105	267	210	59	78	67	198	58	8
5:30	5:45	13	351	102	111	273	212	61	69	59	204	63	7
5:45	6:00	9	342	94	90	251	208	53	83	65	192	49	10

Table A4: PCU and Time Headway at Stadium

no trial	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
1	0.61	0.92	2.03	1.52	1.93	2.85	4.16	2.66	4.26
2	*	0.83	1.02	2.84	2.24	3.55	3.37	2.84	*
3	*	0.61	1.82	1.63	2.64	2.64	3.65	2.74	*
4	*	0.82	1.41	2.02	1.7	1.73	2.86	2.03	*
5	*	0.94	1.02	1.62	1.83	2.23	4.06	3.45	*
6	*	1.42	1.01	1.93	1.73	2.14	3.66	2.54	*
7	*	1.12	2.04	1.94	2.54	2.14	2.03	2.43	*
8	*	*	1.64	1.43	2.74	3.86	2.85	2.44	*
9	*	*	1.53	1.17	1.42	2.75	3.05	4.22	*
10	*	*	1.52	2.34	1.84	3.62	4.02	2.75	*
Avg	0.61	0.951429	1.504	1.844	2.061	2.751	3.371	2.81	4.26
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83

* represent no vehicle

Table A5: Traffic Data for Laghar Intersection

TIME		turning types											
start time	end time	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
7:00	7:15	55	281	15	29	212	13	14	25	23	47	8	12
7:15	7:30	47	279	8	27	208	9	10	35	22	66	14	10
7:30	7:45	66	328	11	63	261	11	14	53	29	51	20	25
7:45	8:00	61	329	9	58	299	13	4	30	13	70	13	21
8:00	8:15	64	360	14	71	329	18	9	45	23	73	21	15
8:15	8:30	85	363	11	60	226	17	12	53	32	58	17	32
8:30	8:45	52	294	21	65	252	31	19	63	35	69	23	23
8:45	9:00	74	315	13	72	237	17	11	52	39	53	15	22
9:00	9:15	77	309	12	83	255	21	15	46	38	55	24	23
9:15	9:30	85	273	19	79	263	25	6	59	33	88	27	31
9:30	9:45	59	269	21	74	260	27	15	52	36	59	26	36
9:45	10:00	47	290	18	81	266	32	14	58	49	63	34	25
4:00	4:15	37	284	22	44	221	27	19	48	41	94	39	29
4:15	4:30	62	298	17	68	243	35	16	43	35	83	45	37
4:30	4:45	88	312	21	71	283	31	10	35	39	75	34	34
4:45	5:00	76	276	16	70	214	28	20	49	38	78	37	32
5:00	5:15	71	287	19	63	249	22	15	44	31	69	32	27
5:15	5:30	49	317	22	39	198	19	18	55	42	81	42	25
5:30	5:45	57	291	14	48	229	24	14	47	33	72	33	18
5:45	6:00	63	301	18	57	215	27	16	51	36	76	40	24

Table A6: PCU and Time Headway at Laghar

no trial	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
1	0.51	0.53	2.13	1.13	2.23	2.03	3.15	2.54	5.09
2	0.68	1.12	1.32	1.52	2.03	2.74	4.47	4.16	4.57
3	*	0.55	1.32	2.34	1.12	2.44	2.34	1.74	*
4	*	0.81	0.72	2.54	2.13	1.73	2.44	2.64	*
5	*	0.92	1.21	1.52	3.45	3.45	3.91	3.55	*
6	*	0.71	1.52	1.25	1.33	2.84	3.66	2.23	*
7	*	0.91	0.81	1.42	1.53	3.76	3.25	3.66	*
8	*	*	2.13	1.93	1.83	3.63	5.69	1.93	*
9	*	*	1.72	2.54	2.43	1.87	3.86	2.44	*
10	*	*	1.13	2.24	2.03	3.86	4.36	1.98	*
Avg	0.595	0.792857	1.401	1.843	2.011	2.835	3.713	2.687	4.83
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45

* represent no vehicles

Table A7: Traffic Volume Based on Type at Shola Gebeya

	EBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
5:00-5:15	0	0	5	2	0	0	0	0	0
5:15-5:30	0	0	6	3	0	0	0	0	0
5:30-5:45	1	0	5	3	3	0	0	1	0
5:45-6:00	0	0	2	4	1	0	0	1	0
sum	1	0	18	12	4	0	0	2	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0.42	0	18	12.6	4.32	0	0	3.34	0
sum(pc)	38.68								
	EBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:15-4:30	1	3	139	92	130	6	8	5	0
4:30-4:45	1	4	135	102	123	3	5	9	2
4:45-4:00	0	2	127	99	115	8	11	13	1
5:00-5:15	0	4	123	104	110	7	11	5	0
sum	2	13	524	397	478	24	35	32	3
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0.84	6.89	524	416.85	516.24	59.28	116.55	53.44	12.87
sum(pc)	1707								
	EBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:30-7:45	1	0	50	33	25	2	1	3	0
7:45-8:00	0	0	48	35	17	7	3	9	0
8:00-8:15	0	2	57	42	28	2	3	5	0
8:15-8:30	0	3	60	40	20	1	4	2	0
sum	1	5	215	150	90	12	11	19	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0.42	2.65	215	157.5	97.2	29.64	36.63	31.73	0
sum(pc)	571								
	WBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:00-8:15	1	1	31	11	19	1	7	0	0
8:15-8:30	0	2	39	20	22	1	1	1	0
8:30-8:45	0	1	43	29	21	0	3	0	0
8:45-9:00	0	1	50	25	30	0	1	1	0
sum	1	5	163	85	92	2	12	2	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0.42	2.65	163	89.25	99.36	4.94	39.96	3.34	0
sum(pc)	403								
	WBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:30-7:45	1	5	149	99	104	11	18	9	0
7:45-8:00	2	6	162	116	110	10	23	3	0
8:00-8:15	1	7	155	124	84	8	10	11	0
8:15-8:30	0	5	139	104	96	8	11	11	0
sum	4	23	605	443	394	37	62	34	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	1.68	12.19	605	465.15	425.52	91.39	206.46	56.78	0
sum(pc)	1865								
	WBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	0	1	0	1	0	0	0	0
4:15-4:30	0	0	2	1	0	0	0	0	0
4:30-4:45	0	0	3	1	0	0	0	0	0
4:45-5:00	0	0	1	0	1	0	0	1	0
sum	0	0	7	2	2	0	0	1	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0	0	7	2.1	2.16	0	0	1.67	0
sum(pc)	13								

Table A7: Traffic Volume Based on Type at Shola Gebeya (continued)

	NBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:15-7:30	0	2	65	32	35	2	1	5	0
7:30-7:45	0	3	62	41	51	1	0	0	0
7:45-8:00	0	2	67	30	43	0	0	0	0
8:00-8:15	1	4	58	29	36	0	0	3	0
SUM	1	11	252	132	165	3	1	8	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0.42	5.83	252	138.6	178.2	7.41	3.33	13.36	0
sum(pc)	599								
	NBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	0	1	1	0	0	0	1	0
4:15-4:30	0	0	1	0	0	0	0	0	0
4:30-4:45	0	1	0	0	1	0	0	0	0
4:45-5:00	0	0	0	1	0	0	0	2	0
sum	0	1	2	2	1	0	0	3	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
Pc	0	0.53	2	2.1	1.08	0	0	5.01	0
sum(pc)	11								
	NBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	2	32	13	27	2	1	1	0
4:15-4:30	1	1	28	19	13	3	1	2	1
4:30-4:45	0	2	26	14	13	1	5	1	0
4:45-5:00	0	2	23	14	14	1	1	0	0
sum	1	7	109	60	67	7	8	4	1
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0.42	3.71	109	63	72.36	17.29	26.64	6.68	4.29
sum(pc)	304								
	SBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	0	0	0	0	0	0	1	0
4:15-4:30	0	0	0	1	1	0	0	0	0
4:30-4:45	0	0	1	0	0	0	0	0	0
4:45-5:00	0	1	0	0	0	0	0	1	0
sum	0	1	1	1	1	0	0	2	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0	0.53	1	1.05	1.08	0	0	3.34	0
sum(pc)	7								
	SBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:45-9:00	0	0	1	1	0	0	0	0	0
9:00-9:15	0	0	0	1	0	0	0	0	0
9:15-9:30	0	0	1	0	0	0	0	0	0
9:30-9:45	0	0	2	0	0	0	0	1	0
sum	0	0	4	2	0	0	0	1	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0	0	4	2.1	0	0	0	1.67	0
sum(pc)	8								
	SBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	0	2	0	1	0	0	1	0
4:15-4:30	0	0	1	1	0	0	0	0	0
4:30-4:45	0	0	3	2	0	0	0	0	0
4:45-5:00	0	0	2	0	2	0	0	2	0
sum	0	0	8	3	3	0	0	3	0
pcui	0.42	0.53	1	1.05	1.08	2.47	3.33	1.67	4.29
pc	0	0	8	3.15	3.24	0	0	5.01	0
sum(pc)	19								

Table A8: Traffic Volume Based on Type at Stadium

time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
	EBL vehicles								
4:15-4:30	0	0	6	7	0	0	0	0	0
4:30-4:45	0	1	10	5	1	0	0	2	0
4:45-5:00	0	1	4	7	1	0	0	0	0
5:00-5:15	0	0	10	6	1	0	0	0	0
sum	0	2	30	25	3	0	0	2	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0	1.26	30	30.75	4.11	0	0	3.74	0
sum(pc)	70								
	EBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:15-7:30	1	1	178	103	93	12	19	9	0
7:30-7:45	0	4	141	82	74	10	15	8	1
7:45-8:00	0	7	171	99	90	9	14	10	0
8:00-8:15	1	8	154	91	84	11	14	7	0
sum	2	20	644	375	341	42	62	34	1
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.82	12.6	644	461.25	467.17	76.86	138.88	63.58	2.83
sum(pc)	1868								
	EBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:30-7:45	1	4	37	20	49	4	3	2	0
7:45-8:00	0	4	33	20	40	5	2	3	0
8:00-8:15	1	5	39	24	48	5	2	4	0
8:15-8:30	1	6	48	31	61	7	4	4	0
sum	3	19	157	95	198	21	11	13	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	1.23	11.97	157	116.85	271.26	38.43	24.64	24.31	0
sum(pc)	646								
	WBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:30-7:45	0	4	60	35	11	6	8	2	0
7:45-8:00	1	3	53	33	10	6	8	3	0
8:00-8:15	1	4	63	37	12	7	8	1	0
8:15-8:30	0	4	55	32	10	6	7	2	0
sum	2	15	231	137	43	25	31	8	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.82	9.45	231	168.51	58.91	45.75	69.44	14.96	0
sum(pc)	599								
	WBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:15-8:30	1	5	147	99	42	5	12	3	0
8:30-8:45	1	4	132	88	38	4	11	2	0
8:45-9:00	2	4	140	93	40	4	12	2	0
9:00-9:15	1	5	141	95	40	5	11	3	0
sum	5	18	560	375	160	18	46	10	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	2.05	11.34	560	461.25	219.2	32.94	103.04	18.7	0
sum(pc)	1409								
	WBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:15-8:30	2	13	145	56	44	0	8	0	0
8:30-8:45	1	11	124	48	38	0	7	0	0
8:45-9:00	1	11	121	46	36	0	6	1	0
9:00-9:15	2	13	134	52	41	0	8	2	0
sum	6	48	524	202	159	0	29	3	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	2.46	30.24	524	248.46	217.83	0	64.96	5.61	0
sum(pc)	1094								

Table A8: Traffic Volume Based on Type at Stadium (continued)

	NBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:45-8:00	1	7	24	16	31	2	8	3	0
8:00-8:15	0	6	21	14	26	3	6	2	0
8:15-8:30	0	7	20	15	27	1	7	3	0
8:30-8:45	1	7	25	17	33	2	9	4	0
sum	2	27	90	62	117	8	30	12	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.82	17.01	90	76.26	160.29	14.64	67.2	22.44	0
sum(pc)	449								
	NBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:30-8:45	0	4	49	30	15	0	3	5	0
8:45-9:00	1	4	47	31	15	1	3	6	0
9:00-9:15	0	5	61	34	19	2	4	6	0
9:15-9:30	0	3	41	25	12	0	2	4	0
sum	1	16	198	120	61	3	12	21	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.41	10.08	198	147.6	83.57	5.49	26.88	39.27	0
sum(pc)	511								
	NBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:45-8:00	0	5	31	38	19	0	2	1	0
8:00-8:15	0	4	29	35	18	0	2	0	0
8:15-8:30	0	7	42	52	26	0	3	1	0
8:30-8:45	1	5	27	35	17	0	2	1	0
sum	1	21	129	160	80	0	9	3	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.41	13.23	129	196.8	109.6	0	20.16	5.61	0
sum(pc)	475								
	SBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	3	89	64	55	1	3	0	0
4:15-4:30	0	2	87	65	56	3	3	1	0
4:30-4:45	0	4	96	72	62	2	4	2	0
4:45-5:00	1	3	92	66	57	1	3	0	0
sum	1	12	364	267	230	7	13	3	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.41	7.56	364	328.41	315.1	12.81	29.12	5.61	0
sum(pc)	1063								
	SBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	2	20	19	19	0	1	0	0
4:15-4:30	1	1	31	24	21	1	2	1	0
4:30-4:45	0	2	36	26	22	1	1	0	0
4:45-5:00	0	1	29	21	18	0	1	0	0
sum	1	6	116	90	80	2	5	1	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.41	3.78	116	110.7	109.6	3.66	11.2	1.87	0
sum(pc)	357								
	SBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
9:00-9:15	0	1	5	2	4	0	0	0	0
9:15-9:30	0	2	4	3	4	0	0	0	0
9:30-9:45	1	1	3	2	3	0	0	0	0
9:45-10:00	0	0	4	3	2	0	0	0	0
sum	1	4	16	10	13	0	0	0	0
pcui	0.41	0.63	1	1.23	1.37	1.83	2.24	1.87	2.83
pc	0.41	2.52	16	12.3	17.81	0	0	0	0
sum(pc)	49								

Table A9: Traffic Volume Based on Type at Laghar

	EBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:15-4:30	0	2	25	17	16	0	2	0	0
4:30-4:45	0	3	20	18	43	0	3	1	0
4:45-5:00	0	3	19	19	26	1	5	3	0
5:00-5:15	1	1	31	17	15	2	3	2	0
sum	1	9	95	71	100	3	13	6	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.42	5.13	95	93.01	145	6.06	34.45	11.52	0
sum(pc)	391								
	EBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:30-7:45	0	7	112	100	79	9	18	3	0
7:45-8:00	1	5	117	105	70	8	16	9	0
8:00-8:15	2	6	124	112	96	7	8	5	0
8:15-8:30	0	6	145	109	90	5	4	4	0
sum	3	24	498	426	335	29	46	21	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	1.26	13.68	498	558.06	485.75	58.58	121.9	40.32	0
sum(pc)	1778								
	EBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	2	5	6	3	0	2	3	1
4:15-4:30	0	1	7	5	3	0	1	1	0
4:30-4:45	0	1	5	3	2	0	6	4	0
4:45-5:00	0	0	4	4	3	1	1	3	0
sum	0	4	21	18	11	1	10	11	1
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0	2.28	21	23.58	15.95	2.02	26.5	21.12	3.45
sum(pc)	116								
	WBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
9:00-9:15	1	0	22	20	40	1	0	0	0
9:15-9:30	1	1	20	18	33	3	4	0	0
9:30-9:45	0	3	19	15	30	3	2	2	0
9:45-10:00	0	2	31	21	23	2	1	1	0
sum	2	6	92	74	126	9	7	3	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.84	3.42	92	96.94	182.7	18.18	18.55	5.76	0
sum(pc)	419								
	WBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
7:30-7:45	0	7	97	71	50	11	19	6	0
7:45-8:00	0	8	111	81	57	13	22	7	0
8:00-8:15	1	9	122	89	63	14	24	8	0
8:15-8:30	1	6	84	62	43	10	15	5	0
sum	2	30	414	303	213	48	80	26	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.84	17.1	414	396.93	308.85	96.96	212	49.92	0
sum(pc)	1497								
	WBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	2	13	7	5	0	0	0	0
4:15-4:30	1	3	18	8	6	0	0	0	0
4:30-4:45	1	3	17	7	4	0	0	0	0
4:45-5:00	0	1	11	9	6	1	0	0	0
sum	2	9	59	31	21	1	0	0	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.84	5.13	59	40.61	30.45	2.02	0	0	0
sum(pc)	138								

Table A9: Traffic Volume Based on Type at Laghar (continued)

	NBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:45-5:00	0	0	6	4	2	2	4	2	0
5:00-5:15	0	1	5	3	2	2	0	2	0
5:15-5:30	0	2	7	4	3	1	0	1	0
5:30-5:45	0	1	6	3	3	0	1	0	0
sum	0	4	24	14	10	5	5	5	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0	2.28	24	18.34	14.5	10.1	13.25	9.6	0
sum(pc)	92								
	NBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
8:30-8:45	0	5	25	16	10	4	2	1	0
8:45-9:00	1	2	19	17	5	2	4	2	0
9:00-9:15	0	3	18	11	8	1	4	1	0
9:15-9:30	0	5	27	14	7	0	4	2	0
sum	1	15	89	58	30	7	14	6	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.42	8.55	89	75.98	43.5	14.14	37.1	11.52	0
sum(pc)	280								
	NBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
9:00-9:15	0	4	14	10	3	2	3	2	0
9:15-9:30	1	3	14	8	3	1	2	2	0
9:30-9:45	0	4	13	9	3	2	3	2	0
9:45-10:00	0	5	8	12	4	3	4	3	0
sum	1	16	49	39	13	8	12	9	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.42	9.12	49	51.09	18.85	16.16	31.8	17.28	0
sum(pc)	194								
	SBL vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	1	6	26	22	35	3	0	2	0
4:15-4:30	0	5	23	19	31	3	1	2	0
4:30-4:45	0	4	21	18	28	2	2	1	0
4:45-5:00	1	5	22	18	29	2	0	2	0
sum	2	20	92	77	123	10	3	7	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.84	11.4	92	100.87	178.35	20.2	7.95	13.44	0
sum(pc)	425								
	SBT vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	0	3	15	10	4	1	5	1	0
4:15-4:30	0	4	17	9	5	3	6	1	0
4:30-4:45	1	2	13	8	3	2	4	2	0
4:45-5:00	0	3	14	9	4	1	5	1	0
sum	1	12	59	36	16	7	20	5	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.42	6.84	59	47.16	23.2	14.14	53	9.6	0
sum(pc)	213								
	SBR vehicles								
time	cycle	2 wheel	car	4 -WD	min bus	med bus	large bus	med trucks	large trucks
4:00-4:15	1	3	11	8	7	0	0	0	0
4:15-4:30	0	4	19	10	4	0	0	0	0
4:30-4:45	0	5	12	11	3	1	2	0	0
4:45-5:00	0	3	15	8	4	0	1	1	0
sum	1	15	57	37	18	1	3	1	0
pcui	0.42	0.57	1	1.31	1.45	2.02	2.65	1.92	3.45
pc	0.42	8.55	57	48.47	26.1	2.02	7.95	1.92	0
sum(pc)	153								

Appendix B: Phasing Output

Table B1: Phasing's Output of Estifanos

Phasings
1: Airport road/Minilik II ave & Jomo kenyata st /jomo kenyata st 2017-05-27

Lane Group	EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR	NBU	NBL	NBT	SBU
Protected Phases		4	4			8	8			2	2	
Permitted Phases	4			Free	8			Free	2			6
Minimum Initial (s)	5.0	5.0	5.0		5.0	5.0	5.0		5.0	5.0	5.0	5.0
Minimum Split (s)	22.5	22.5	22.5		22.5	22.5	22.5		22.5	22.5	22.5	22.5
Total Split (s)	30.0	30.0	30.0		25.0	25.0	25.0		30.0	30.0	30.0	25.0
Total Split (%)	27.3%	27.3%	27.3%		22.7%	22.7%	22.7%		27.3%	27.3%	27.3%	22.7%
Maximum Green (s)	26.5	26.5	26.5		21.5	21.5	21.5		26.5	26.5	26.5	21.5
Yellow Time (s)	3.0	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0	3.0
All-Red Time (s)	0.5	0.5	0.5		0.5	0.5	0.5		0.5	0.5	0.5	0.5
Lead/Lag												
Lead-Lag Optimize?												
Vehicle Extension (s)	3.0	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0	3.0
Minimum Gap (s)	3.0	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0	3.0
Time Before Reduce (s)	0.0	0.0	0.0		0.0	0.0	0.0		0.0	0.0	0.0	0.0
Time To Reduce (s)	0.0	0.0	0.0		0.0	0.0	0.0		0.0	0.0	0.0	0.0
Recall Mode	Max	Max	Max		Max	Max	Max		Max	Max	Max	Max
Walk Time (s)												
Flash Dont Walk (s)												
Pedestrian Calls (#/hr)												
90th %ile Green (s)	26.5	26.5	26.5		21.5	21.5	21.5		26.5	26.5	26.5	21.5
90th %ile Term Code	MaxR	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord	MaxR
70th %ile Green (s)	26.5	26.5	26.5		21.5	21.5	21.5		26.5	26.5	26.5	21.5
70th %ile Term Code	MaxR	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord	MaxR
50th %ile Green (s)	26.5	26.5	26.5		21.5	21.5	21.5		26.5	26.5	26.5	21.5
50th %ile Term Code	MaxR	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord	MaxR
30th %ile Green (s)	26.5	26.5	26.5		21.5	21.5	21.5		26.5	26.5	26.5	21.5
30th %ile Term Code	MaxR	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord	MaxR
10th %ile Green (s)	26.5	26.5	26.5		21.5	21.5	21.5		26.5	26.5	26.5	21.5
10th %ile Term Code	MaxR	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord	MaxR

Intersection Summary

Cycle Length: 110

Actuated Cycle Length: 110

Offset: 32 (29%), Referenced to phase 2:NBTL, Start of Green

Control Type: Pretimed

Description: Estifanos

Table B2: Phasing Output of Stadium

Phasings
2: Ras Biru st & Ras Mekonen St/Jomo kenyata st & Ras Desta Damtew st 2017-05-27

Lane Group	EBU	EBT	EBR	WBU	WBL	WBT	WBR	NBU	NBL2	NBL	NBR	SEL
Protected Phases	7	4		3	3	8			2	2		6
Permitted Phases	4		Free	8			Free	2			Free	
Minimum Initial (s)	5.0	5.0		5.0	5.0	5.0		5.0	5.0	5.0		5.0
Minimum Split (s)	10.0	22.5		10.0	10.0	22.5		22.5	22.5	22.5		22.5
Total Split (s)	19.0	35.0		25.0	25.0	41.0		25.0	25.0	25.0		25.0
Total Split (%)	17.3%	31.8%		22.7%	22.7%	37.3%		22.7%	22.7%	22.7%		22.7%
Maximum Green (s)	15.5	31.5		21.5	21.5	37.5		21.5	21.5	21.5		21.5
Yellow Time (s)	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0		3.0
All-Red Time (s)	0.5	0.5		0.5	0.5	0.5		0.5	0.5	0.5		0.5
Lead/Lag	Lead	Lag		Lead	Lead	Lag						
Lead-Lag Optimize?	Yes	Yes		Yes	Yes	Yes						
Vehicle Extension (s)	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0		3.0
Minimum Gap (s)	3.0	3.0		3.0	3.0	3.0		3.0	3.0	3.0		3.0
Time Before Reduce (s)	0.0	0.0		0.0	0.0	0.0		0.0	0.0	0.0		0.0
Time To Reduce (s)	0.0	0.0		0.0	0.0	0.0		0.0	0.0	0.0		0.0
Recall Mode	Max	Max		Max	Max	Max		Max	Max	Max		Max
Walk Time (s)												
Flash Dont Walk (s)												
Pedestrian Calls (#/hr)												
90th %ile Green (s)	15.5	31.5		21.5	21.5	37.5		21.5	21.5	21.5		21.5
90th %ile Term Code	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord		MaxR
70th %ile Green (s)	15.5	31.5		21.5	21.5	37.5		21.5	21.5	21.5		21.5
70th %ile Term Code	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord		MaxR
50th %ile Green (s)	15.5	31.5		21.5	21.5	37.5		21.5	21.5	21.5		21.5
50th %ile Term Code	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord		MaxR
30th %ile Green (s)	15.5	31.5		21.5	21.5	37.5		21.5	21.5	21.5		21.5
30th %ile Term Code	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord		MaxR
10th %ile Green (s)	15.5	31.5		21.5	21.5	37.5		21.5	21.5	21.5		21.5
10th %ile Term Code	MaxR	MaxR		MaxR	MaxR	MaxR		Coord	Coord	Coord		MaxR

Intersection Summary

Cycle Length: 110

Actuated Cycle Length: 110

Offset: 0 (0%), Referenced to phase 2:NBL, Start of Green, Master Intersection

Control Type: Pretimed

Description: stadium

Table B3: Phasing Output of Laghar

Phasings
3: Yilma Demissew st/ Gambia st & Ras Mekonen St/Ras mekonnen 2017-05-27

Lane Group	EBU	EBL	EBT	WBU	WBL	WBT	NBU	NBL	NBT	SBU	SBL	SBT
Protected Phases		4	4		8	8		2	2		6	6
Permitted Phases	4			8			2			6		
Minimum Initial (s)	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Minimum Split (s)	22.5	22.5	22.5	22.5	22.5	22.5	22.5	22.5	22.5	22.5	22.5	22.5
Total Split (s)	76.0	76.0	76.0	73.0	73.0	73.0	35.0	35.0	35.0	36.0	36.0	36.0
Total Split (%)	34.5%	34.5%	34.5%	33.2%	33.2%	33.2%	15.9%	15.9%	15.9%	16.4%	16.4%	16.4%
Maximum Green (s)	72.5	72.5	72.5	69.5	69.5	69.5	31.5	31.5	31.5	32.5	32.5	32.5
Yellow Time (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
All-Red Time (s)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Lead/Lag												
Lead-Lag Optimize?												
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Minimum Gap (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Time Before Reduce (s)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Time To Reduce (s)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Recall Mode	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max
Walk Time (s)												
Flash Dont Walk (s)												
Pedestrian Calls (#/hr)												
90th %ile Green (s)	72.5	72.5	72.5	69.5	69.5	69.5	31.5	31.5	31.5	32.5	32.5	32.5
90th %ile Term Code	MaxR	MaxR	MaxR	MaxR	MaxR	MaxR	Coord	Coord	Coord	MaxR	MaxR	MaxR
70th %ile Green (s)	72.5	72.5	72.5	69.5	69.5	69.5	31.5	31.5	31.5	32.5	32.5	32.5
70th %ile Term Code	MaxR	MaxR	MaxR	MaxR	MaxR	MaxR	Coord	Coord	Coord	MaxR	MaxR	MaxR
50th %ile Green (s)	72.5	72.5	72.5	69.5	69.5	69.5	31.5	31.5	31.5	32.5	32.5	32.5
50th %ile Term Code	MaxR	MaxR	MaxR	MaxR	MaxR	MaxR	Coord	Coord	Coord	MaxR	MaxR	MaxR
30th %ile Green (s)	72.5	72.5	72.5	69.5	69.5	69.5	31.5	31.5	31.5	32.5	32.5	32.5
30th %ile Term Code	MaxR	MaxR	MaxR	MaxR	MaxR	MaxR	Coord	Coord	Coord	MaxR	MaxR	MaxR
10th %ile Green (s)	72.5	72.5	72.5	69.5	69.5	69.5	31.5	31.5	31.5	32.5	32.5	32.5
10th %ile Term Code	MaxR	MaxR	MaxR	MaxR	MaxR	MaxR	Coord	Coord	Coord	MaxR	MaxR	MaxR

Intersection Summary
Cycle Length: 220
Actuated Cycle Length: 220
Offset: 160 (73%), Referenced to phase 2:NBTL, Start of Green
Control Type: Pretimed
Description: lager

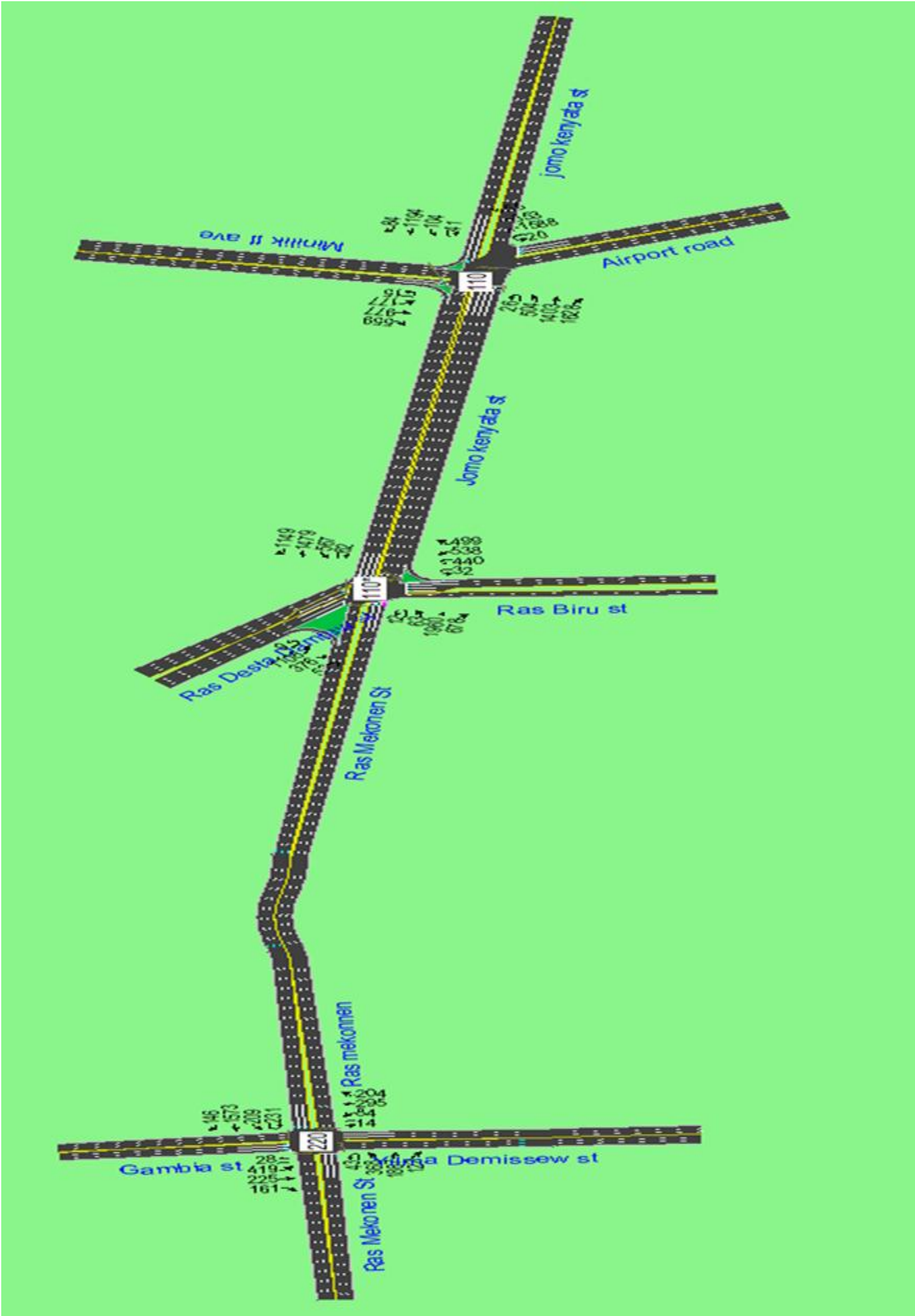


Figure B1 Map view of Estifanos- Stadium -Laghar

Appendix C: Questionnaire

DATE ………………..

Addis Ababa University

Addis Ababa Institute of Technology

School of Civil & Environmental Engineering

Dear Sir/Madam

I, the researcher, am student in Addis Ababa Institute of Technology, currently doing my M.Sc. in Civil Engineering under Road and Transport Engineering at Addis Ababa Institute of technology. I am doing my M.Sc. research/thesis entitled: optimization of intersection traffic signal in Addis Ababa with the aim of assessing the causes and impact of delay, quantify and evaluate performance of intersection in Addis Ababa.

Your genuine, honest and prompt response to the questionnaire will have contribution to the success of the research. Moreover, the information you provide will be used strictly for academic purpose.

Filling the questionnaire will not take more than 15 minutes. I thank you in advance for the time you devote, effort you make, and consideration you give in filling this questionnaire.
If you have any question concerning the items of the questionnaire.

Title: The optimization of intersection traffic signal timing parameter.

Objectives of research

- To evaluate the performance of signal intersection
- To develop simulation model for individual traffic movement
- To provide remedial measurement toward delay, travel time, stop reduction

Part-I: General Information

1. Respondent's Name (Optional):________________________________

2. Respondent's: (Use X)

   Sex____________

   Age: 20-30___ 30-40___ 40-50___ above 50___

3. Respondent's occupation:___________________________________

Part-II: Respondent's Perceptions toward Traffic delay in Addis Ababa city.

1) Do you think there is traffic congestion at intersections in Addis Ababa at some level?

   Yes____ No____

2). If your answer for question number 1 is yes. What do you think the level of traffic congestion at the intersections?

Very High___ High___ Moderate___ Low___ Very Low___

3. What do you think the cause of traffic delay at the intersection in the city?

A ____

B ____

C____

4. What do you suggest to minimize the delay at the intersections in the city?

A ____

B ____

C____

D ____

5) What is the time of day at which traffic flow become high?

____

6) Rank the following traffic signal intersection depending on their delay from highest to lowest?

Shola gebeya, Semien hotel, Kidest Mariam, Bête Mengist, Laghar,

Estifanos, Stadium.

I. ______________________________

II. ______________________________

III. ______________________________

IV. ______________________________

V. ______________________________

VI. ______________________________

VII. ______________________________

Thank You for Your Responses!!!!