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Effect of Cycle Time and Signal Phase on Average Time Delay and safety at Road Intersection in Addis Ababa City: A case study at Hager Astedader Signalized Intersection

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ABSTRACT

High urbanization rate and Economic development has caused many challenges to transportation systems. Among these, long time delay and high fuel consumption of vehicles at congestion places are a few to mention. Many literatures have revealed that road traffic congestions are caused by inadequate infrastructures, long signal cycle time and poor traffic management, such as incapable roads, inefficient public transit, and high travel demand.

The research study was focused on the effect of traffic congestion on average time delay at selected signalized road intersection in Addis Ababa. The study area was Hager Astedadrer road traffic signal intersection. The methodology that has been employed for the study was a quantitative descriptive research design method. The data needed were; road geometry data, signal data, traffic vehicles flow data including the pedestrian data were collected on peak hours (with 15minutes interval) from 7:30 - 9:30 AM and from 5:30 – 7:30 PM for the four consecutive working days. Data of traffic classes were extracted manually on separate worksheet. The volume of each vehicle category was converted to the same vehicle category using passengers' car unit (PCU) of each vehicle class. Data analysis and processing have been performed using SIDRA (Signalized and unsignalized Intersection Design and Research Aid) intersection software in order to know the traffic flow condition at the intersection. The result of average time delay was used to know the corresponding level of service and operational performance of the intersection.

The outcomes of the research work minimize average time delay by adopting different improving strategies that range from low-cost measures such as improvements to signal timing and phase numbers, to high-cost measures such as intersection reconstruction which be applied in the study area after knowing the amount of average time delay as well as the types of the level of service within the intersection area.

Key words: Congestion, Peak Period, Passenger car unit, SIDRA, Level of Service, Signals.

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ACRONYMS /ABBREVIATIONS

A.A	Addis Ababa
AACRA	Addis Ababa City Road Authority
AASHTO	American Association of State Highway and Transportation Officials
ERA	Ethiopian Road Authority
HAI	Hager Astedader Intersection
HCM	Highway Capacity Manual
JIT	Jimma Institute of Technology
LOS	Level of Service
PCE	Passengers Car Equivalency
PCPH	Passenger Cars per Hour
PCU	Passenger Car Unit
PHF	Peak Hour Factor
PHV	Peak Hour Volume
SFR	Saturation Flow Rate
USA	United State of America
$V_{p15min.}$	Peak 15minute Volume
VTPI	Victoria Transport Policy Institute

CHAPTER ONE

INTRODUCTION

1.1 General

The cities and traffic have been developing parallel, because of human settlements. Though their magnitude or patterns are more complex today, cities still provide access to various social and economic activities, such as services, goods, markets, network, and these which determines the development of the urban areas [13].

Urban economic activities and movements have a direct relationship. Adequate transport system is needed to facilitate a greater choice of the peripheral areas if urban transport provided by the government. Because of most socio-economic activities concentrated in the center of city, many urban mobility problems are created, such as increasing car usage and trips to the central business districts. This in turn leads to high demand for parking, urban environmental problem (pollution, noise), and in-abilities to handle these problems results in congestion [6].

Fast growth in urbanization and industrializations demand the use of more vehicles group which leads to an imbalance between the infrastructures availability and mobility. In the third world, the roads are narrow or incapable to accommodate a heavy traffic that slow the traffic flow, accompanied by in adequate provision for parking and loading and boarding facilities, sidewalk along the street, and mixed land use [5].

1.2. Background

Traffic Congestions become common characteristics in urban road transportation system of cities in developing countries which result in high operating cost, loss of users' productive time, and more fuel consumption among others [4].

Due to numerous factors, congestion is becoming more serious problem in Addis Ababa city from time to time, such as population growth- in addition to natural growth, pull factor that immigrate people from different part of the country to the city in searching for livelihood. To sustain the city, it is clear that these added portions of the society also need transport service to sustain their day to day activities. However, the city is unable to cope up with the existing high transport services demand because of poor infrastructure, high population and absence of well traffic management which are the major reasons why traffic congestion exist.

Understanding of the present situation of vehicles traffic congestion is very important area of consideration in order to make the right decision to solve the issues [3].

1.3. Statement of the Problem

In Ethiopia, traffic congestion is becoming a new phenomenon. It has an economic cost on the city communities and economy. The road traffic congestion is becoming more serious problems in a day to day activity of people in some parts of the Addis Ababa city, specifically, in the morning and evening peak hours. The traffic congestion is an outcome of various factors. These are; insufficient capacity of the roads to cope up with the existing traffic volume, long travel time (delay) at intersections, insufficient traffic management in the city are the major problems that related to road traffic congestion in the city. In addition, long travel time to reach destination that affect business users productivity time, increasing fuel consumption are the main impacts of road traffic congestion which still prevail [3]. Therefore, this research was intended to assess the level of delay in all approaches as a result of road traffic congestion at Hager Astedader signalized intersection in Addis Ababa.

1.4 Research Questions

The research questions are based from the specific objectives:

1. What was the average time delay of the vehicles at the intersection of study area?
2. What was the level of service (LOS) of the intersection?
3. How to evaluate the operational performance of the intersection?
4. What are the contributory parameters of traffic congestion in the study area?
5. What possible measures to be under taken to reduce average time delay at study area?

1.5. General Objectives

The main objective of this study was to assess the effect of traffic congestion on average time delay incurred on the travelers and motorists within the identified intersection in City of Addis Ababa.

1.5.1 Specific Objectives

1. To determine the average time delay of the motorists at Hager Astedader intersection.
2. To determine the Level of Service (LOS) of Intersection and approaches, using SIDRA.

3. To evaluate the operational performance of the intersection in all approaches.
4. To identify the contributory parameters to the traffic congestion in the study area.
5. To suggest some possible solutions to the traffic problem within the study area.

1.6 Significance of the Study

It would be known that the output of this study will be added to the existing academic knowledge and enable to understand the subject matter as it paves the way for further investigation on the topic. Moreover, it can benefit other parties of the society through:-

Firstly: It will allow the researcher to assess the current condition and impact of traffic congestion on the economic activities of the city thereby build academic knowledge and provide base for further career improvement.

Secondly: It would also benefit Jimma Institute of Technology (JIT) in attaining its objective as a center of academic excellence and accelerate the national development through provision of problem solving research output to the policy and decision makers.

Lastly: The City Administration of Addis Ababa can use the finding of this research work for its social and economic policy formulation and right decision making based on factual and latest information, so that, the City Administration would use its resources efficiently to ensure the City's sustainability by solving the problem of traffic congestion delay.

1.7 Limitation of the study

There was limitation of budget in conducting field survey. All data collection was very sensitive to professionals, especially types of vehicles and turning movement type at the intersection. Lack of related study from concerned body like Addis Ababa city Road Authority and Ethiopian Road Authority. The software analysis was also need more time and some parameters used as input for the software were difficult to collect from the study area manually

CHAPTER TWO

REVIEW OF RELATED LITERATURES

2. Introduction

Currently, traffic congestion problem has become the research agenda for a growing community of researchers particularly in developing countries, like Ethiopia. This problem represents directly or indirectly about relationships with on-street parking management, roadway geometric configurations, characteristics or traffic capacity. These can affects peoples in various ways such as long travel time and vehicles more fuel consumption [3].

2.1 Urban Mobility characteristics

Concerning the field of mobility, a study by [32] stated that numerous unique characteristics of Addis Ababa and other cities in developing countries include:

- Rapid growth of motorization
- Intense desire for car ownership
- Travel demand that far exceeds the supply of facilities
- High share of trips by public transit
- Urban structure incompatible with motorization
- Greater differences in vehicle performance
- Inadequate street and highway maintenance
- Irregular response to impacts of new construction
- Fewer legal constraints on the use of new technologies
- Very limited agreement on planning approaches
- Scarcity of capital and operating subsidies are difficult to sustain
- Local transportation development is more centralized in the hands of a few elite players.

Developing countries are characterized by a high travel demand, chaotic traffic behavior and a low supply of networks and means. Although fewer legal restraints exist for the use of new technologies, the transport sector as a whole is not likely to be innovative: capital is scarce and the number of involved stakeholders is limited.

The lack of adequate and accessible transportation options causes a high share of non-motorized transport. This stressed mobility situation in developing countries results in premature congestion, a deteriorating environment and a high incident rate. It has to be noted that these characteristics concern the average of the whole group of developing countries.

2.2 Urban Traffic Flow Characteristics

Improvement of mobility is a function of vehicular flow. Flow is defined as the rate at which vehicles pass a given point on a road way, and it is in normally given in terms of vehicles per hour. Flow can be categorized into two:

1. Uninterrupted flow- This is the flow that is regulated by vehicle to vehicle interactions and interactions between the road way environment and the geometry of the road.
2. Interrupted flow- This refers to flow that is regulated by external means; normally planned traffic means. The means could include traffic signals and traffic signs. Devices such as signals allow designated traffic movements to occur at a certain time.

To ensure adequate mobility, both interrupted and uninterrupted flows should be kept at an absolute minimum. This is done by minimizing points of traffic conflicts within a traffic facility hence ensuring free flow of traffic within the facility. Free flow of traffic means that vehicular traffic needs to attain free flow speeds. According to the Highway Capacity Manual 2000, free flow speeds are defined as the speed of traffic under conditions of low volume and low density. It is the speed at which drivers feel comfortable traveling under the physical, environmental, and

Traffic control conditions existing on an uncongested section of multilane highway. Increase of traffic conflicts points minimizes free flow of traffic thereby causing traffic congestion [3]. In addition, in developing cities, high traffic volume is occurred during peak period in which large number of traveling is concentrated because activities starts up in the early part of the day, so numerous journeys take the same time. A similar pattern occurred in the afternoon when business activities end .While the portion of trip was made by single occupant (small size) vehicles by commuters [7,8].

2.3. Categories of Traffic Congestion and its impact

Traffic congestion can be defined as the saturation of road network capacity due increased traffic volume or interruptions on the road that cause an increase in travel times. It is characterized by slower speeds, longer trip times, and increased vehicular queuing. When traffic demand on a road facility is great, such that the interaction between vehicles slows the speed of the traffic stream, congestion results. Traffic congestion can be categorized into two:

2.3.1 Recurrent congestion**:** A regular congestion that occurs in hourly, daily, weekly or annual cycle. There are three main reasons that cause recurrent type of congestion. They include:

- Insufficient capacity

  Occurs when the existing road network simply cannot cope up with the demand. This is simply due to the annual increase in traffic.
- Unrestrained demand

  The demand results from more and more people buying private vehicles because they can afford to, because the public transport system is insufficient, or because there is a geographic necessity for it. The result of this demand is a growth that outstrips the rate at which public transport systems can be expanded or adjusted.
- Ineffective capacity management

  Where traffic control systems are ill-timed, out of order or badly placed. "Roads are operated near to their maximum capacity and saturated intersection can quickly give rise to queues whose upstream propagation can swamp local roads and intersections" [13].

2.3.2 Non-recurrent congestion**:** -It is a spontaneous type of congestion. It depends on occurrence of events on traffic facilities such as accidents and disabled vehicles. Such events cause interruptions to free flow of traffic on the road [3].
The five main non-recurrent causes for traffic congestion are as follows:

- Incidents and accidents: **-** Road accidents have the ability to shut down lanes on a highway as the vehicles involved block the route. Rubber-necking which involves other drivers who slow down to take a look, is encouraged and this effectively creates a bottleneck.
- Work zones

  Road maintenance and construction near a road requires space and a reduction in speed of vehicles. The result, again, is a bottleneck.
- Weather

  Bad weather hampers visibility (in the case of rain or snow) and therefore cautious drivers will slow down to avoid accidents. The result of this is formation of slow moving traffic queues hence congestion.

- Special events

  An influx of a few thousand people to a sports or entertainment venue such as a stadium creates traffic congestion in the surrounding access roads.

- Emergency situations: - Locations in the world where large-scale disasters, such as hurricanes or severe snowstorms, occur regularly, experience severe traffic congestion. The congestion occurs as people evacuate the area before the outcome of the disaster.

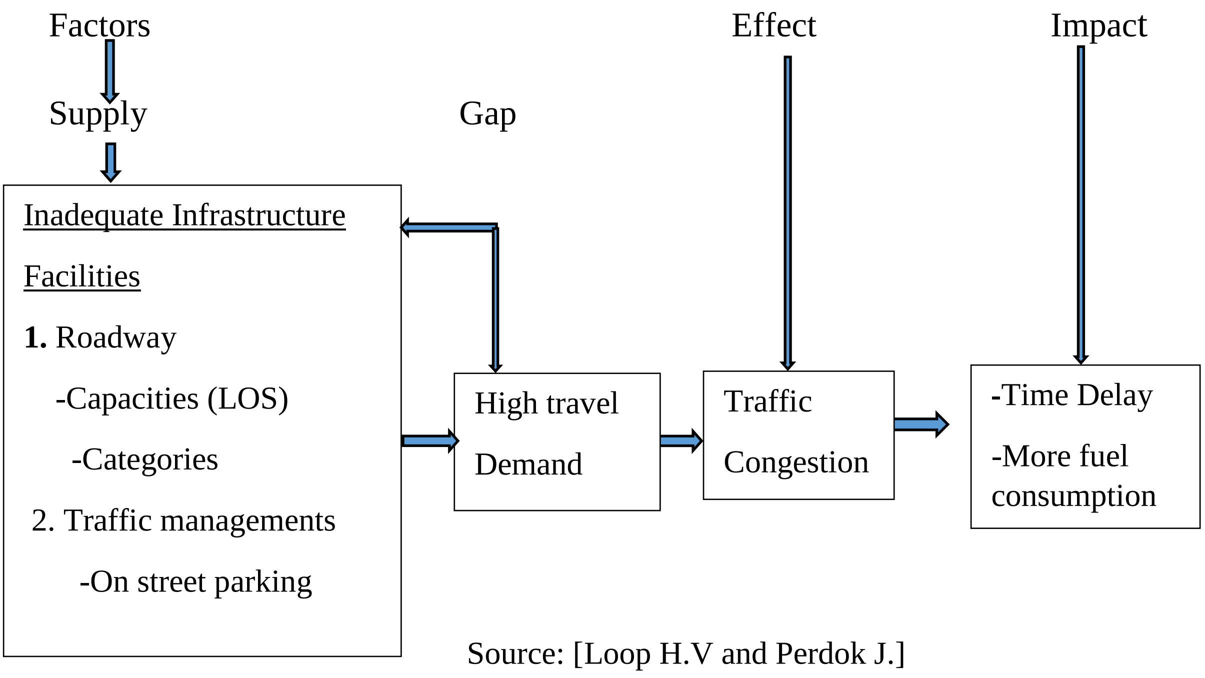


Figure 2.1 Conceptual framework explaining congestion and its impact

Many conceptual frameworks explain the causes and the impact of vehicle traffic congestion in different way. Most writers argued that traffic congestion has an impact on time delay and fuel consumption, among them "traffic congestion can causes more fuel to be used" [8, 13].

Hours are lost or delayed due to various factors, among which the direct indicators are the imbalance between demand and supply, number of vehicles or traffic volume, like trucks, buses, private cars etc. exceeds the existing road capacity [15].Moreover, the capacity of a road can be measured by comparing the traffic volume and travel time. However, traffic management, like on-street parking conditions can affect the performance of the roads.

2.4. Measurement of Traffic Congestion.

In many countries, road transport has a great share than other modes in transporting goods and passenger. It is also confronts with serious issues, most notable is- traffic congestion which is a result of excessive utilization of the road infrastructures due to high number of pedestrians, small road network length, a high portion of the population engaged in informal business sector, and poor public transport supply are not based on peak hour demand which result in long travel journey period or delay [9].Congestion can be measured in various ways, including roadway level-of-service (LOS), and average congestion delay compared with free-flowing traffic [7]. The capacity of a road depends on various design factors, such as lane widths and intersection configurations among others. As indicated in various literatures, it is possible to conclude that capacity of a given road at the intersection (junction) can be determined by actual traffic volume to capacity ratio for the vehicles, delay and level-of service (LOS) of the intersection [2].

2.4.1. Traffic volume to Capacity ratio and LOS

Congestion also can be measured by a volume-to-capacity ratio (V/C Ratio).The Victoria Transport policy Institute report states that traffic congestion impacts can be measured in terms of volume to capacity ratio. For the purposes of congestion calculations, congestion levels are defined as:

V/C Ratio greater than 1.0 = Severe Congestion

V/C Ratio of 0.75 to 1.0 = Heavy Congestion

V/C Ratio of 0.5 to 0.74 = Moderate Congestion

V/C Ratio of less than 0.5 = Low or No Congestion

The volume to capacity ratio hence requires computation of traffic volumes and determination of capacities of road facilities [14].

The capacity of a road depends on various factors, some of which are lane width and configurations of the roadway. The Author [11] stated that the traffic volume of roadways for different lane as follows.

Table2.1 Maximum Traffic Volume (Passenger Cars per Hour)

Type of way	LOS A	LOS B	LOS C	LOS D	LOS E	LOS F
4-lane freeway	700	1100	1550	1850	2000	Unstable
2-lane freeway	210	375	600	900	1400	Unstable
4-lane highway	720	1200	1600	1940	2200	Unstable

Source: [Orn H. 2007].

Table 2.1 shows the traffic volume per lane for different type of roadways. The traffic volume is increasing when one passes from "A" to "F" level-of-service for different lanes. The road capacity is positively proportioned to the number of lanes [10], and as traffic congestion is a non- linear function in its characteristics, it will represent a small change in a traffic volume which can result in a proportional change traffic flow. For example, a 5% reduction in traffic volume on a congested highway may reduce delays by 10 – 30% even if the size of vehicles directly related with the road space [11].

Delay has a direct relationship with the volume of traffic. Level-of-service (LOS) indicates traffic volume on the road [16] and delay becoming more and more serious when go from "A" to "F" level-of-services.

For example, except for F- level of service, which is so flexible or undetermined, delay for "A" level- of service is less than 15 seconds, and for "B" level-of-service is between 15 and 30 seconds and so on[1]. Similar to the idea of [16] and Transportation Research Board of USA (2000) stated time delay with in each level-of -service at different geometrical road designs as follows.

Table 2.2 Delay in each LOS for an intersections and Street/roads.

LOS	At signalized intersection (second/vehicle)	At un-signalized intersection (second/vehicle)	At Streets/roads (Using queue)	Remark
A	<10	<10	<15	There is higher delaines at Streets than un-signalized & signalized intersections
B	10 - 20	10 – 15	15 - 30	
C	20 - 35	15 – 25	30 - 55	
D	35 - 55	25 – 35	55 - 85	
E	55 - 80	35 – 50	85 - 120	
F	>80	>50	Unstable	

Source [Highway Capacity Manual 2010]

Level-of-service A: Individual vehicles flow freely, not affected by others.

Level-of-service B: though it is in a stable flow condition, obviously additional vehicles joining the traffic stream will affect traffic movement. However, there is a relatively freedom in speeds.

Level-of-service C: There is a relative stable flow, but individual vehicles influence the flow immediately and become significantly affected by interactions with other vehicles in the traffic stream.

Level-of-service D: It is a crowded roadway situation as mobility and a stable flow is restricting with a large number of vehicles. Speed and freedom to movement are harshly restricted.

Level-of-service E: Roadway accommodates nearly to its full capacity, low speed, and small increment in the traffic volume will affect the traffic movement more.

Level-of-service F: Vehicles move in a locked each other with in front and beyond condition. Speed is mostly to zero, and the travel time cannot be predicted.

All level of service types are shown below graphically.

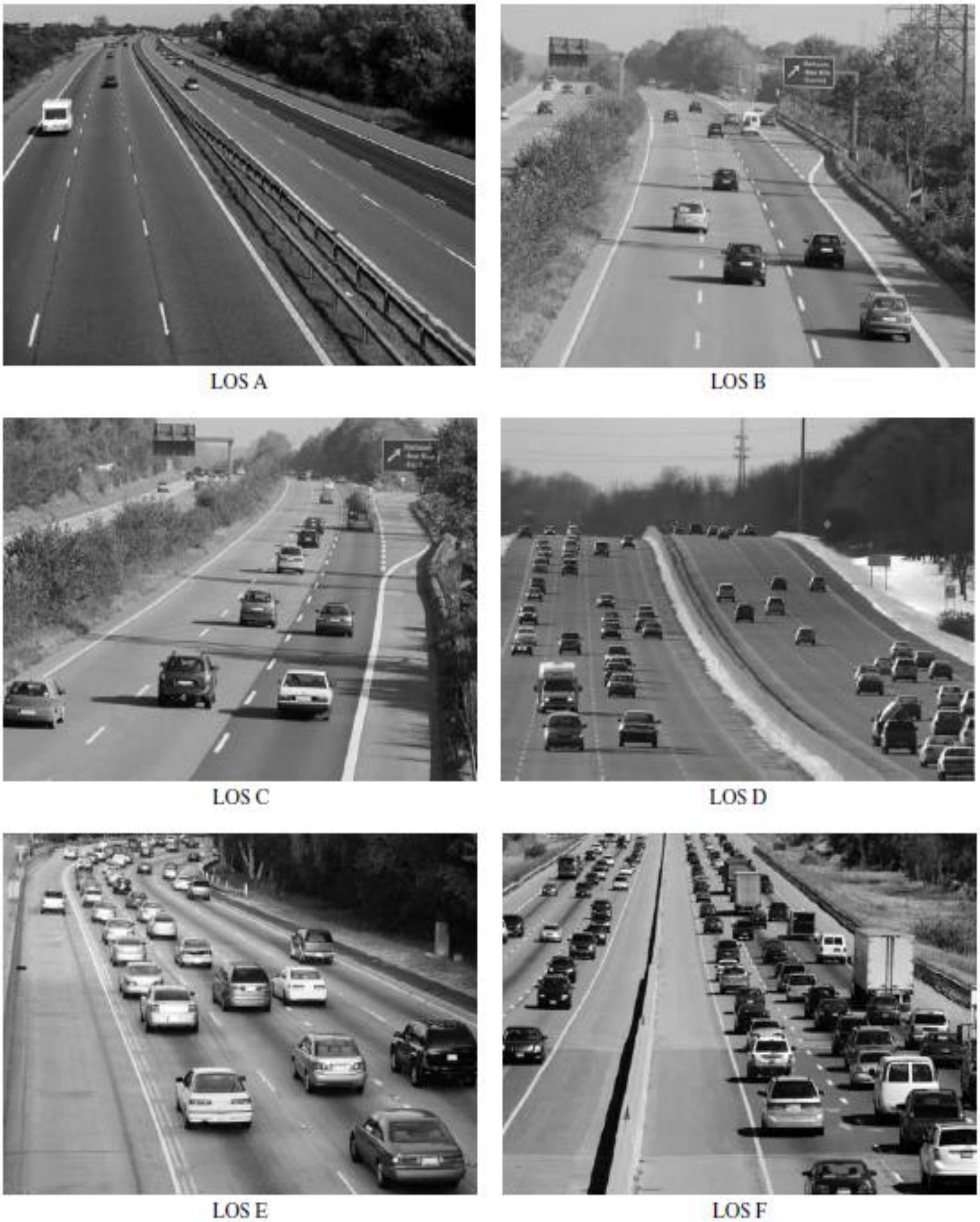


Figure 2.2 Different classes of level of service type. (Source HCM 2000)

The base criteria for determining the average delay per vehicle and delay per pedestrian as per the Highway Capacity manual have been using in our country Ethiopia since Level of Service (LOS) is directly related to the delay value. The standard criteria as per the Highway Capacity

Manual for the signalized intersection are listed in the following table 2.3 and table 2.4 for vehicles and pedestrian respectively.

Table2.3 Level of Service criteria for vehicles at signalized intersection

LOS Types	Delay per Vehicle (sec/veh)
A	≤ 10
B	10 - 20
C	20 - 35
D	35 - 55
E	55 - 80
F	>80

Source [Highway Capacity Manual 2010]

Table 2.4 Level of Service criteria for pedestrian at signalized intersection

LOS Types	Pedestrian Delay (sec/p)
A	≤ 10
B	10 - 20
C	20 - 30
D	30 - 40
E	40 - 60
F	>60

Source [Highway Capacity Manual]

The service measure is the average delay experienced by a pedestrian. Many researches indicate that the average delay of pedestrians at signalized intersection crossing is not constrained by capacity. LOS criteria for pedestrians at signalized intersection is based on pedestrian delay. When pedestrians experience more than a 30-s delay, they become impatient, and engage in risk-taking behavior [1]. This research work tried to investigate the average delay time conditions at the study area for both pedestrians and vehicles flow condition using the SIDRA Intersection software.

CHAPTER THREE

MATERIALS AND METHODOLOGY

3.1 Introduction

Addis Ababa is located at 9°1′48″N 37°47′24″E and lies at an altitude of 2,369 m (7,546 ft.) with estimated area of 530 square kilometers. Based on the preliminary 2008-07-01 census results, Addis Ababa has a total population of 3,147,000, with an annual growth rate of 2.8% of which migration accounted for 1.98% [12].

3.1.1 Climate

The study area is located on the high land of Ethiopian plateau that is having an altitude of over 2000m above mean sea level. Therefore; it is considered as Dega. The effective temperature is lower than 25c°, which is good for life.

3.1.2 Rainfall

The study area receives mean annual rainfall in the range of 918 – 1567mm and the rainy period is between June and September, although occasional shower is expected in the month of March, April, October and November.

The mean monthly rainfall of the study area gets maximum during the month of July and August that is 254 and 280mm and gets minimum values during the month of November and December 8 and 9mm respectively.

Table 3.1 Mean monthly Rainfall (mm) of the study area

month	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
Rainfall	17.6	39.7	68.5	91.3	77.3	119.2	253.6	279.6	172.2	39.2	8.1	9.2

Source: [National Meteorology Agency]

3.1.3 Temperature

For the study area, the monthly temperature is maximum during the months of March and May, about 27.5C° and its minimum in the months of November through January, about 4.7C° for last five years.(Source: National meteorology Agency)

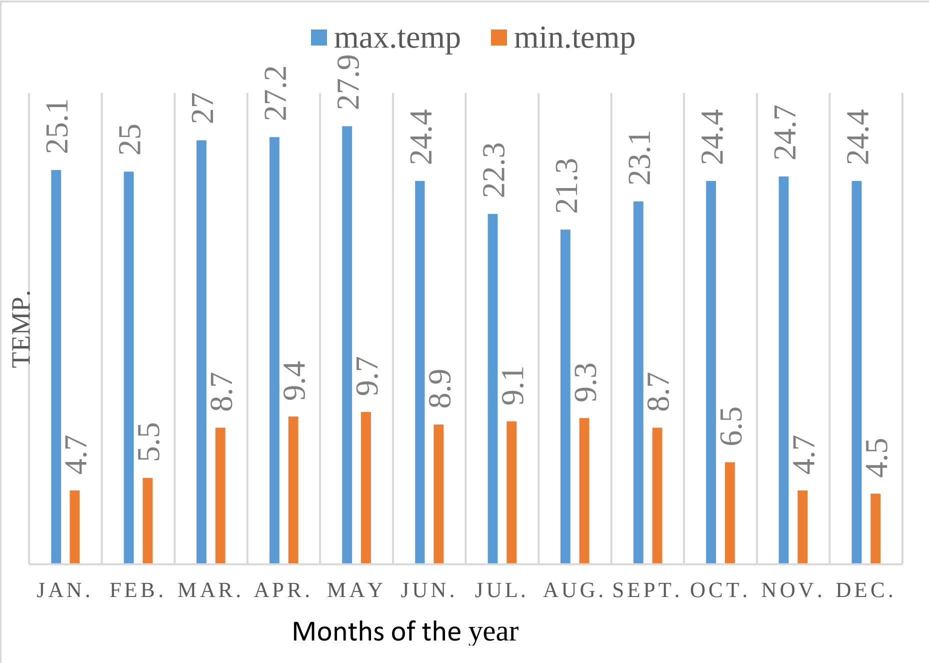


Figure 3.1 Monthly maximum and minimum temperature (C^0) of the study area.

3.1.4 Study Area Description

Addis Ababa has five main roads that radiate from North-South and East-West directions linking to external Towns of the country. Among them, Addis Ababa - Akaki main road is the one which runs through Lafto sub-city and contributes the traffic congestions on the study area. The study area is located at East of Tikur Anbessa, North of Biheraw, South of Piazza, and Northwest of Filhuha (Posta Bet). The study intersection is comprised of two right turn and four approaches of which traffic flow enters/dissipates. The study area is almost located at the center of the city where high commercial/ business activities are highly experienced which result traffic congestion during the two peak hours.

Figure 3.2 Location of the study area (Source Google Map 2015)

A junction, or intersection, is the general area where two or more roads join. The study area is located on Churchill Avenue (street).The availability of nearby institutions such as Black Lion Hospital, Immigration Head Office, Ethiopian Telecommunication Head Office, Ethiopia Broadcast Corporation, Black Lion School, Ministry of Transportation and public bus stopping for those Office has an important effect on the intersection. Detail drawing of the intersection is as follows.

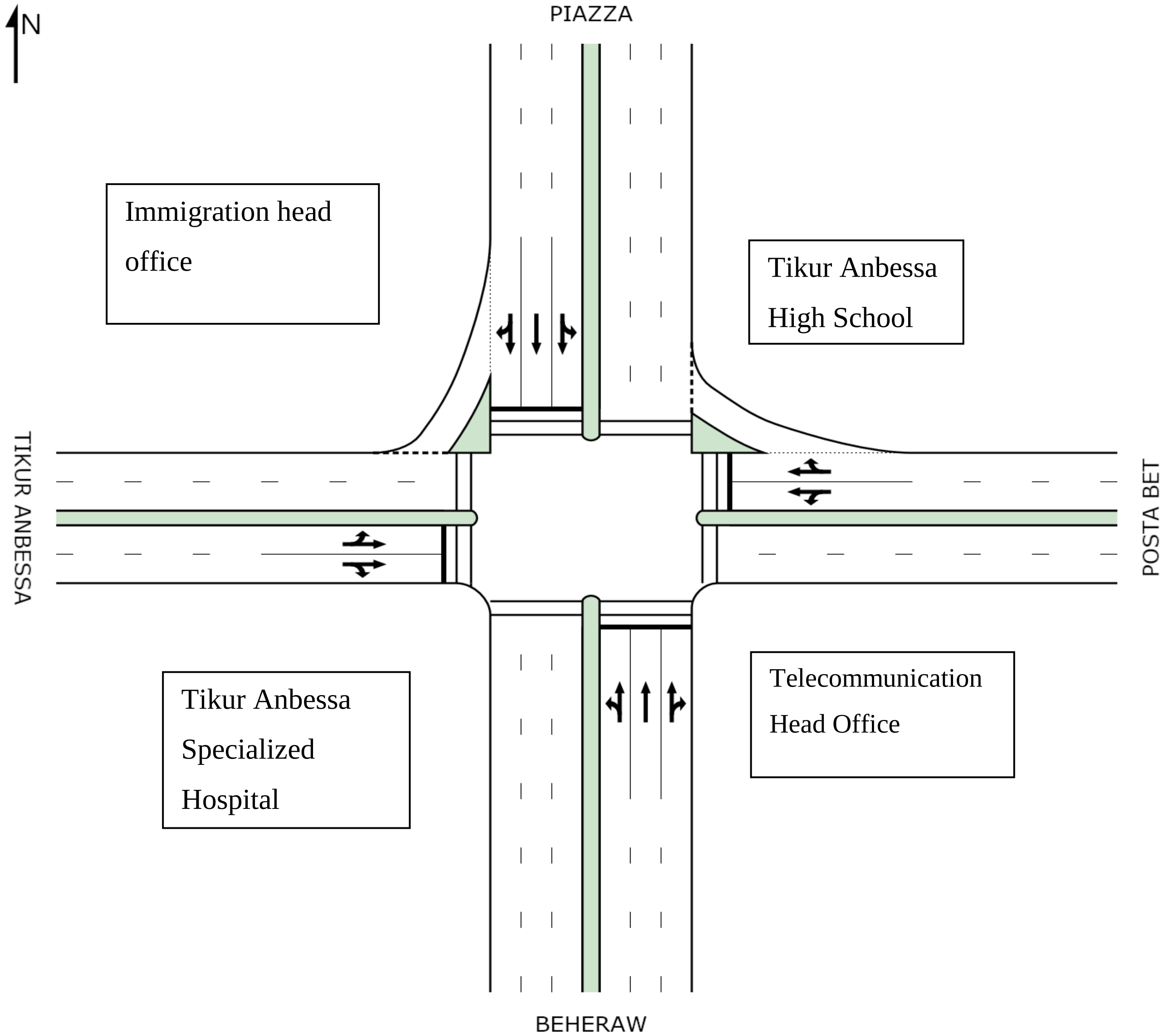


Figure 3.3 Detail drawing of the study area with lane configuration.

Figure 3.3 represents the drawing of the intersection with its lane disciplines which incorporates four legs and two right turn lanes. Intersection design can vary widely in terms of size, shape, number of travel lanes, and number of turn lanes. Each roadway radiating from an intersection is called a "leg." Most intersections have four legs, which is generally accepted as the maximum recommended number for safety and capacity reasons [17].

The Churchill Avenue (street Biheraw to Piazza) is 10.8 m wide with three lanes in each direction. The other Road from Tikur Anbessa to Posta Bet is 14.4 m wide with two lanes in each direction. At the intersection, no separate left turn lanes from Biheraw and Tikur Anbessa legs, but there are right turn lanes provided from Posta Bet and Piazza legs.

As shown in the following photo, the intersection is a basic 4-way intersection (four leg intersection)

Figure 3.4 Photo shows the study area at the junction

Generally, the lane width of the study intersection in each direction is 3.6m with three lanes in the direction of south to north with 2m median width and two lanes in the west to east directions with 1.8m median width. Total area of the intersection is more than 23.6m x 16.2m on which the vehicles can follow their directions.

3.2 Materials

The materials used for accomplishment of the thesis work are:

1. Video camera to take actual traffic movements and other information.
2. Laptop Computer for analyzing and storing necessary data
3. Stopwatch for registering signal times

4. GPS for determining altitude and approaching grades
5. Google Map of the study area to show the exact area
6. Tape meter to measures dimensions of road geometry and different distances

3.3 Study Period

The study period of the research work was started from May 2015-October 2015 for the different activities including thesis writing.

3.4 Data Requirements

Those information or data used as inputs for the software were obtained from:

1. Motorized mixed traffic flow data in each leg of the intersection.
2. Pedestrian data in each leg of the intersection.
3. Signal data.
4. Road geometry data

3.5 Study variables and descriptions

Dependent variable: Effect of traffic congestion on an average time delay.

Independent variable: The independent variables for this research include: - volume, speeds, saturation flow rate, signal data, Road geometry data.

On an interrupted flow facility, flow usually is dominated by points of fixed operation, such as traffic signals and stop signs. These controls have different impacts on overall flow. The operational state of traffic at an interrupted traffic-flow facility is defined by the following measures [1].

- Volume and flow rate
- Saturation flow and departure headways
- Control variables (stop or signal control)
- Delay and Queuing

3.5.1 Volume

Volume is the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval; it can be expressed in terms of annual, daily, hourly, or sub hourly periods.

Three basic variables—volume, speed, and density; can be used to describe traffic conditions on any roadway. Since speed and density are more given attention on highway segment or for uninterrupted-flow conditions, more attention was given to traffic volume of the intersection and queuing on each approach.

Volume or traffic flow is a parameter common to both uninterrupted- and interrupted-flow characteristics, but speed and density apply primarily to uninterrupted flow. Some parameters related to flow rate, such as spacing and headway, also are used for both types of flow characteristics; other parameters, such as saturation flow or gap, are specific to interrupted flow. Thus, the research work primarily focuses on volume, saturation flow rate, signal data and road geometry data [1].

3.5.1.1 Peak Hour Volume and Flow rate

The peak hour volume is the volume of traffic that uses the approach, lane, or lane group; during the hour of the day that observes the highest traffic volumes for that intersection. The peak hour volume would be the sum of four peak volume of passenger car units for the given hour. The peak hour volume is normally given in terms of passenger car units, since changing turning all vehicles into passenger car units makes these volume calculations more representative of what is actually going on.

Flow rate: - is the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than 1 hour, usually 15 minute. Peak flow rates and hourly volumes produce the peak-hour factor (PHF).

3.5.1.2 Peak Hour Factor

It is simply the ratio of total hourly volume to four times the peak fifteen-minute volume. For example, during the peak hour, there will probably be a fifteen-minute period in which the traffic volume is denser than during the remainder of the hour. That is the peak fifteen minutes, and the volume of traffic that uses the approach, lane, or lane group during those fifteen minutes is the peak fifteen-minute volume. The peak hour factor is given below.

$$PHF = \frac{\text{Hourly Volume(V)}}{\text{Peak flow rate within hour}} \qquad (3.1)$$

The peak hour volume is just the sum of the volumes of the four 15 minute intervals within the peak hour [20].

If 15-min periods are used, the PHF may be computed by Equation:-

$$PHF = \frac{V}{4 * V15min} \qquad (3.2)$$

Where

PHF= peak-hour factor,

V= hourly volume (veh/h),

V_{15} = volume during the peak 15 min of the peak hour (veh/15 min).

When the PHF is known, it can convert a peak-hour volume to a peak flow rate by Equation:-

$$v = \frac{V}{PHF} \tag{3.3}$$

Where

v= flow rate for a peak 15-min period (veh/h),

V = peak-hour volume (veh/h), and

PHF= peak-hour factor

The flow rate then can be computed simply as 4 times the maximum 15-min count.

Peak-hour factors in urban areas generally range between 0.80 and 0.98. Lower values signify greater variability of flow within the subject hour, and higher values signify little flow variation. Peak-hour factors over 0.95 are often indicative of high traffic volumes, sometimes with capacity constraints on flow during the peak hour [20].

3.5.2 Speed (s)

Speed is defined as a rate of motion expressed as distance per unit of time, expressed as miles per hour (mi/hr.) or kilometer per hour (km/hr.) for interrupted-flow conditions. Delay rather than speed is the primary measure of traffic operations at the intersections [20]. However, speed measures similar to those for uninterrupted flow are helpful in determining the added travel time due to deceleration, movement in queues, and acceleration of vehicles passing through an intersection. Since speed is primarily applicable for uninterrupted traffic flow facilities, the SIDRA intersection software needs only the operating speed of the intersection. Thus the value used in the software was operating speed of 60km/h for through movement which is obtained from Addis Ababa City Road Authority (AACRA) design manual volume 11 and 40km/h. for left and right turn movements as per software users' guide.

3.5.3 Saturation flow rate and departure headways.

Saturation Flow Rate is the number of vehicles that would pass through the intersection when the approach signal were stay with green for an entire hour. Obviously, certain aspects of the traffic and the roadway will affect the saturation flow rate of the approach. If the approach has very narrow lanes, traffic will naturally provide longer gaps between vehicles, which will reduce our saturation flow rate. If there are large numbers of turning movements, or large numbers of trucks and busses, your saturation flow rate will be reduced [10].

On another hand, the saturation flow rate (s) for a lane group is the maximum number of vehicles from that lane group that can pass through the intersection during one hour of continuous green under the prevailing traffic and roadway conditions.

The saturation flow rate is normally given in terms of straight-through passenger cars per hour of green. Most design manuals and textbooks provide tables that give common values for trucks and turning movements in terms of passenger car units (pcu).[1]

Thus, Saturation flow rate is defined as the flow rate per lane at which vehicles can pass through a signalized intersection and can be computed by the following equation:-

$$S = \frac{3600}{h} \tag{3.4}$$

Where 3600= number of seconds per hour, s = saturation flow rate (veh/hr.), and

h = saturation headway (s).

For this research work, no need of calculating the values of saturation flow rate on each approach manually, because the SIDRA intersection software by itself generates the values after the raw data are fed to the software.

Departure headway is the elapsed time between the front wheels of the first and that of the second vehicles over the stop line at the intersection. The first headway will be comparatively long, as a result. The second vehicle in the queue follows a similar process, except that the reaction and acceleration period can occur while the first vehicle is beginning to move. The second vehicle will be moving faster than the first as it crosses the stop line, because it has length in which to accelerate. Its headway will generally be less than that of the first vehicle.
In general, the saturation (departure) headway is the amount of time that a vehicle in the stopped queue takes to pass through a signalized intersection on the green signal, assuming that there is a continuous queue of vehicles moving through the intersection.
For interrupted flow, headway represents the time between the passage of the front axle of one vehicle and of the front axle of the next vehicle over a given cross section of the roadway [1].

3.5.4 Signal Control

The most significant source of fixed interruptions on an interrupted-flow facility is the traffic signal. Traffic signals periodically halt flow in each movement or set of movements. Movement on a given set of lanes is possible only for a portion of the total time, because the signal prohibits movement during some periods. Only the time during which the signal is effectively green is available for movement [25].

Some movements are allowed to proceed during a phase even though they cause conflicts. Pedestrians are commonly allowed to proceed across intersections even though right-turn movements are occurring. These movements are called permitted, while protected movements are those without any conflicts.

In any case, the movements at an intersection can be grouped, and then these groups can be served during separate phases.

The basic timing elements within each phase include the green interval, the effective green time, the yellow or amber interval, the all-red interval, the inter green interval, the pedestrian WALK interval, and the pedestrian crossing interval. Each of these elements is described below [18].

The green interval is the period of the phase during which the green signal is illuminated the yellow or amber interval is the portion of the phase during which the yellow light is illuminated

The effective green time was contained within the green interval and the amber interval. The effective green time for a phase, was the time during which vehicles are actually discharging through the intersection. The all-red interval is the period following the yellow interval in which all of the intersection's signals are red. The inter-green interval is simply the interval between the end of green for one phase and the beginning of green for another phase. It is the sum of the yellow and all-red intervals.

The pedestrian WALK interval is the portion of time during which the pedestrian signal says WALK. This period usually lasts around 4-7 seconds and is completely encompassed within the green interval for vehicular traffic. Some pedestrian movements in large cities are separate phases unto themselves.

Finally, the pedestrian crossing time is the time required for a pedestrian to cross the intersection. This is used to calculate the inter-green interval and the minimum green time for each phase.

Therefore; in this research work the signal values obtained from field surveys used in the SIDRA Intersection software are discussed under the topic of data collection

3.5.5 Delay

Delay is a critical performance measure on interrupted-flow facilities. There are several types of delay. They are:

I. Stopped time delay,
II. Approach delay,
III. Control delay; but in this research work, control delay is the principal service measure for evaluating LOS at signalized intersections.

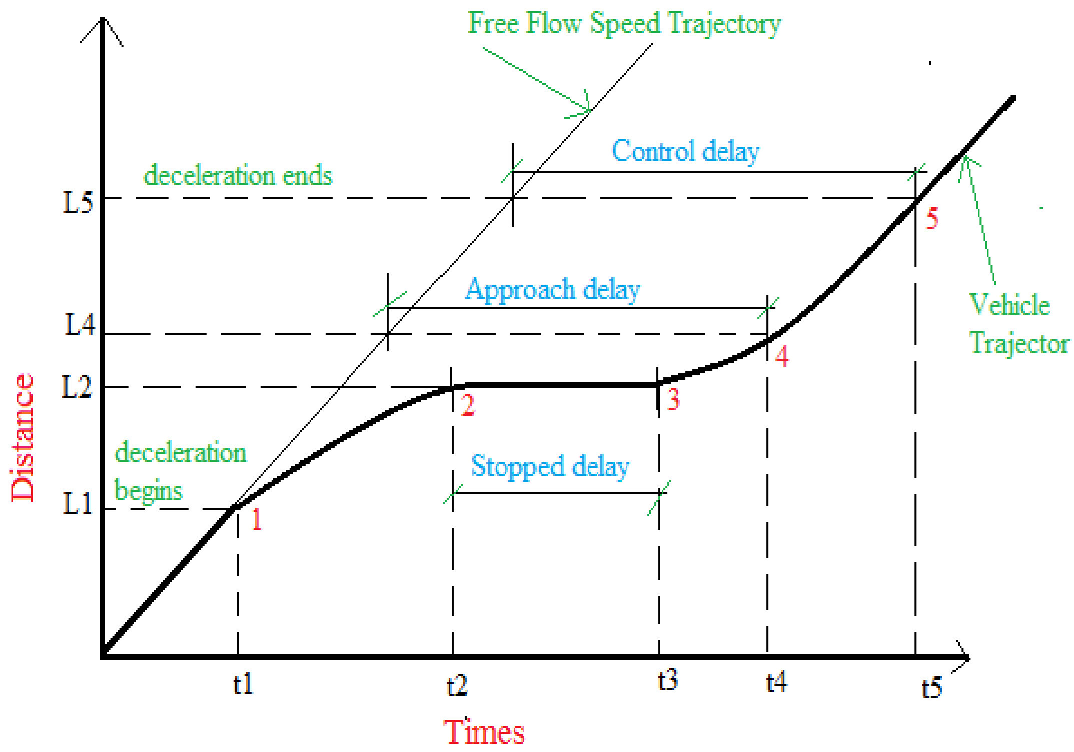


Figure 3.5 Delay types at a signalized intersection

The distance (L_1, L_2 . . . L_5) along y-axis shows the distance moved by the vehicles and the time (t1, t_2 …t_5) along x-axis represents time taken by the vehicles to travel the corresponding distances.

3.5.5.1 Stopped time delay

Stopped-time delay is defined as the time a vehicle is stopped in queue while waiting to pass through the intersection. It begins when the vehicle is fully stopped and ends when the vehicle begins to accelerate.

3.5.5.2 Approach delay

Approach delay includes stopped-time delay but adds the time loss due to deceleration from the approach speed to a stop and the time loss due to re-acceleration back to the desired speed.

3.5.5.3 Control delay

Control delay is the delay caused by a control device, either a traffic signal or a STOP-sign. It is approximately equal to time-in-queue delay plus the acceleration-deceleration delay component [39].

Control delay involves movements at slower speeds and stops on intersection approaches, as vehicles move up in the queue or slow down upstream of an intersection. Control delay is the total elapsed time from a vehicle joining the queue until its departure from the stopped position at the head of the queue. The control delay also includes the time required to decelerate to a stop and to accelerate to the free-flow speed [1].

The final output of the control delay had been generated by the software under result and discussion topics.

3.5.6 Queuing

When demand exceeds capacity at an approach to a signalized intersection at the start of an effective green period, a queue forms. Because of the arrival of vehicles during the red phases, some vehicles might not clear the intersection during the given green phase. Queuing is simply a closely spaced collection of vehicles.

During the red interval, the line of vehicles waiting at the intersection begins to increase. The queue reaches its maximum length at the end of the red interval. When the signal changes to green, the queue begins to clear as vehicles depart from the intersection at the saturation flow rate [26] as seen in the figure 3.5 below.

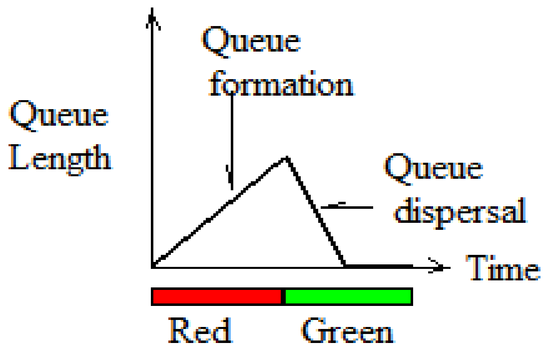


Figure 3.6 Queue Length versus Time

To predict the characteristics of a queuing system, it is necessary to specify the characteristics of the following parameters.

- Arrival pattern
- Service pattern

The plot for the arrival pattern and departure (service) times for each vehicle become the following figure below.

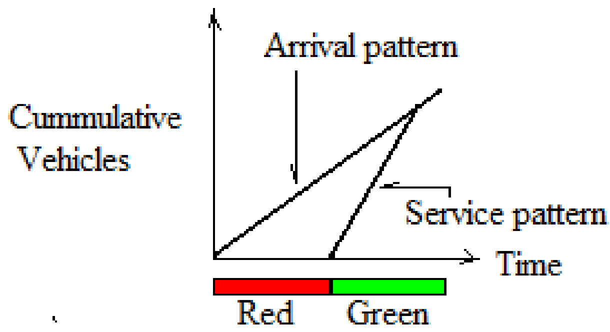


Figure 3.7 Vehicles versus Time

For a given time, the difference between the arrival pattern and the service pattern is the queue length. For a given vehicle, the difference between the service pattern and the arrival pattern is the vehicle delay. In addition, the area of the triangle is equivalent to the total delay for all of the vehicles [26]. See figure below.

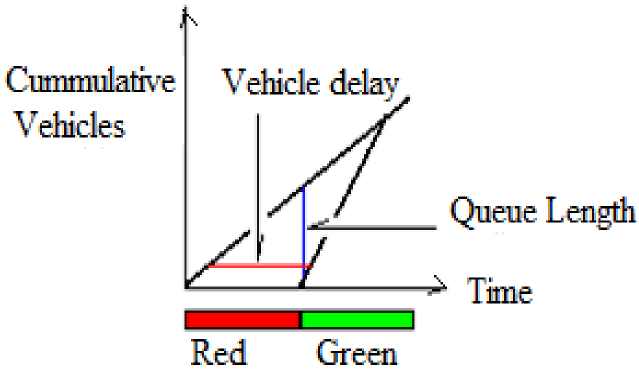
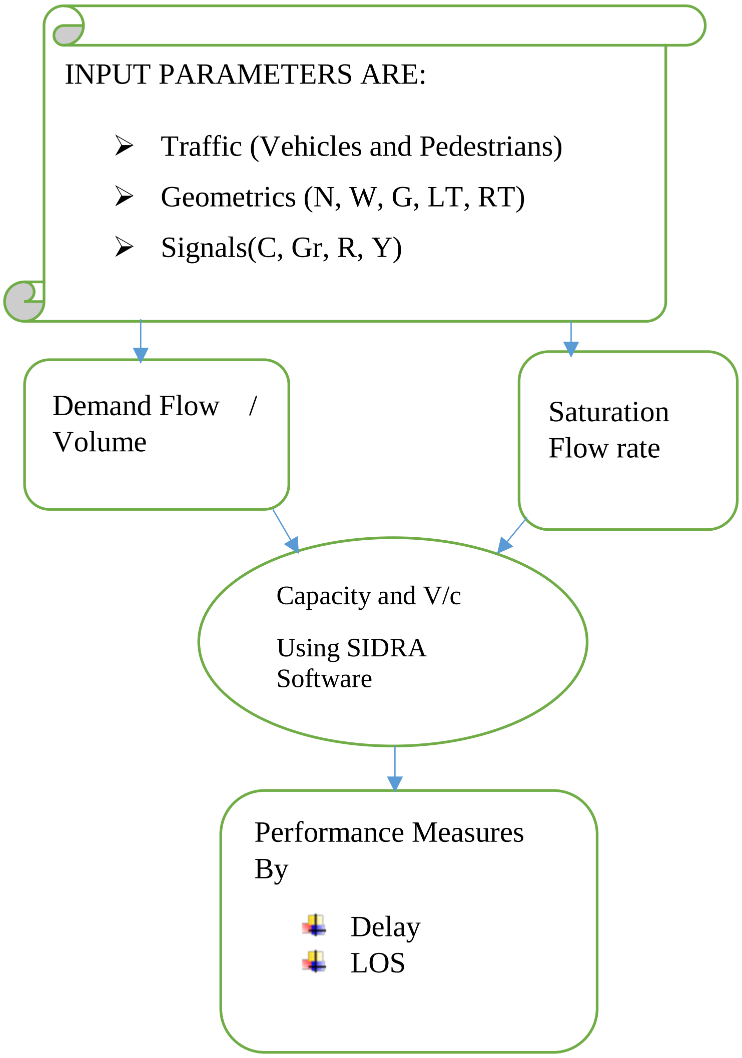


Figure 3.8 Properties for vehicle delay and queue length.

The first vehicle to be stopped by the red signal experiences the most delay. In addition, the queue is longest just before the green interval begins [1].

3.6 Research methodology

The quantitative descriptive research design used for the purpose of this study which enable the research to interpret the finding adequately and accurately. Consequently, the research work consisted the relevant data collection, intensive data analysis by some suitable tools and describe the effect of road traffic flow, the pedestrian data, road geometry data and signal characteristics with the relationship of average time delay and the Level-Of-Services by using the software at the signalized road intersection quantitatively. Below is the flow chart of the research work.



Where: N = number of lanes

W = lane width (m)

G = gradient (%)

LT = left turns

RT = right turns

C =cycle length (sec.)

Gr = green time (sec.)

R =red time (sec.)

Y = yellow time (sec.)

V = volume (veh.)

c= capacity

Figure 3.9 Sequential research work output procedures.

3.7 Data Collection methods.

Data acquisition and field investigation provides an understanding of the physical and operational characteristics of the study intersection and identify factors that contribute to its deficiencies. All the input data for the SIDRA intersection software for this research work, mixed or heterogeneous traffic data, geometric data and signal parameters data are essential described in section 3.7.2, 3.8.1 and 3.8.2.

For this research work, SIDRA which stands for (**S**ignalized and Unsignalized **I**ntersection **D**esign and **R**esearch **A**id) intersection software was used. SIDRA intersection software is a professional tool for the purpose of capacity, level of service and operating performance analysis of road traffic. SIDRA can also be used to analyze signalized and unsignalized intersections, single-point urban interchanges, and signalized midblock crossings for pedestrians. Unlike HCM which uses lane group concept in intersection analysis, SIDRA has a capability of performing lane-by-lane analysis at the intersection.

Field survey of traffic data (both vehicles and pedestrians), geometric data and traffic signal parameter's observations were conducted from July 20 to 24/2015during the two peak hours (7:30 – 9:30 a.m. and 5:30– 7:30 p.m.). While the secondary data (different books, journals, related thesis) were collected from different relevant bodies, such as Addis Ababa City Transport Branch Office, Internets, related thesis and Road Authorities(AACRA).

3.7.1 Field Investigation

Field investigations were performed to observe the rough flow conditions, safety and operating conditions. The researcher visited the site to evaluate and recommend improvements to an intersection by considering three perspectives.

1. Intersection perspective: This deals with operational performance, geometric configuration, movements types with high delay/overcapacity, signal timing, signing, and pavement markings.

2. User perspective: visibility of road marking, driver ability, decision making, level of service, and conflicts for all user types.

3. System perspective: impacts of upstream/downstream intersections influence of adjacent driveways, location relative to other facility types.

Figure 3.10 Photo shows field observation and recording values of road geometry

3.7.2 Methods of Conducting Volume Counts.

Traffic volume counts were conducted using manual method: Manual counting involved ten persons for recording observed vehicles on each approaches using tally marks.

The counts were undertaken at the study area to record the proportion of vehicles as well as the traffic volume, where the volume of the various turning movements were existed. With this type of count, intersection counts were taken to determine vehicle classifications, through movements, and turning movements (right and left) appeared at intersections. These data are used mainly in determining the volume of signalized intersection. Note that, the inclusion of pickups and light trucks with four tires in the category of passenger cars would have significant deficiencies in the collected data, since the performance characteristics of these vehicles aren't similar to those of passenger cars. Therefore; the research work used some heavy vehicles adjustment factors in order to convert into passenger's car unit (PCU).

3.7.3 Vehicles Classification

Based on Addis Ababa City Road Authority (AACRA) Design Vehicle grouping and traffic manual with annual vehicles population growth rate (1.6%), mixed traffic vehicles classifications are summarized as follows.

P. Cars= (Standard Car+ Wagon & Pickup+ Minibus Van, Single Rear Axle)

= Light vehicles (Lv.).

Mini buses include public buses with 12 seats up to 18 seats categorized under passenger cars. Medium buses/Medium Trucks include Higer buses, other buses with 24 seats, coaster buses and medium freight vehicles and Large buses/Large Trucks include Anbessa buses, other public buses more than 24 seats and heavy trucks.

Medium = (Buses, Dual Rear Axle Trucks)
Heavy = (4 Axle Trucks) = Heavy Vehicles (HV.)
Articulated = (Large Trucks)

Finally; in the research work, the counted vehicles were grouped into three different vehicle types: Passengers car, Buses and heavy vehicles.

For the purpose of intersection analysis, SIDRA INTERSECTION defines a Heavy Vehicle as any vehicle with more than two axles or with dual tyres on the rear axles.

The three options (rules) used for specifying and displaying the Vehicles data in using the software are:

I. Separate LV & HV: Separate volumes for Light Vehicles (LVs) and Heavy Vehicles (HVs) will be specified, e.g. LVs 900 veh/hr. and HVs 100 veh/hr.
II. Total Vehicles & HV (%): Total volume and per cent Heavy Vehicles will be specified, e.g. total 1000 veh/hr. and 10 per cent HV, and
III. Total Vehicles & HV (veh): Total volume and Heavy Vehicle volume will be specified, e.g. 1000 veh/h and 100 veh/hr.

Therefore; the research work used the third option and the traffic data arranged accordingly.

3.7.4 Passenger Car Unit (PCU)

The Passenger Car Unit (PCU) or Passenger Car Equivalent (PCE) is the universally adopted unit of measure for traffic volume or capacity. Thus, the traffic flow with any vehicular composition can be expressed in terms of its equivalent Passenger Car Unit. Many studies have been performed to determine reasonable values for PCE under different road and traffic conditions. In the HCM 2000 the definition of PCE is given as "The number of passenger cars displaced by a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions" [1].

The PCU value of a passenger car was identified as 1.0 because of ease maneuverability in any directions. Each vehicle type was given a single PCU equivalent to represent its relative disturbance to the flow under the prevailing traffic condition. Sometimes a set of PCU values is assigned to a particular type of vehicle to represent the various disturbances in its presence in different traffic situations [28].

The factors influencing the PCU value of a vehicle are mainly related to the physical and mechanical characteristics of a vehicles like, overall length, overall width, engine power, weight, acceleration, deceleration, braking and other maneuvering characteristics [33]. The PCU values of buses and trucks is summarized in the following table based on movement types.

Table 3.2 Different PCU values used at a signalized intersection.

Movement types	Left		Through		Right	
Vehicles types	Buses	Trucks	Buses	Trucks	Buses	Trucks
PCU ranges	1.34-2.4	2.4-3.38	1.28-1.77	1.83-2.82	1.68-2.27	2.25-2.83
Aveg. Pcu	1.87	2.89	1.5	2.3	1.98	2.68

[Source: J.W.Z Warteveen, 2011]

Using the above factors, each vehicle classes converted to the total pcu for each 15min. intervals in the peak hour traffic condition that is used as input for the software. The raw data for vehicles traffic volume and pedestrians' data were collected simultaneously.

Therefore; the researcher selected the third rule for specifying and displaying the factored vehicles data of four days as inputs for the software. The following figure shows summarized factored vehicles traffic data of the study area.

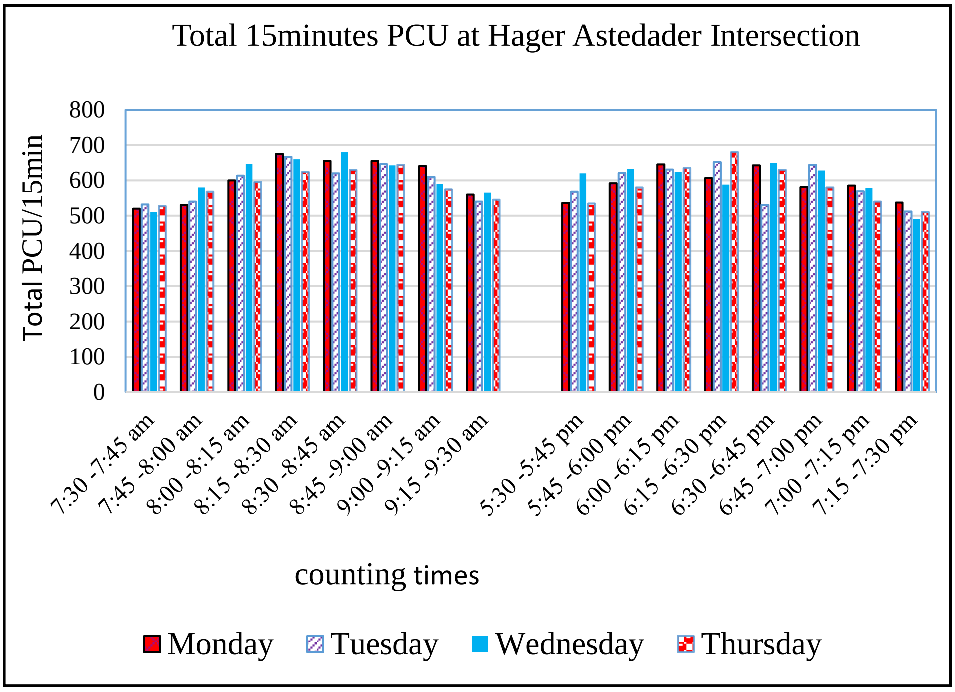


Figure 3.11 Total 15 minutes passenger's car unit versus counting times at the study area

Based on observation taken at the study area, the above figure 3.10 shows that the total passenger car unit per 15 minutes at Hager Astedader Signalized intersection within observation of the days. The observations were taken for four consecutive working days at the two peak hours (7:30-9:30 pm and 5:30-7:30 pm). The highest 680 passenger car unit per 15 minutes at Hager Astedader signalized intersection was observed on Wednesday and Thursday at 8:30am to 8:45 am and 6:15 pm to 6:30 9m respectively in the morning and afternoon which justified that the two peak hours really occurs at the morning and afternoon times.

On the other hand, the lower counts of passenger car unit per 15 minutes were observed relatively on Monday and Thursday than that of Tuesday and Wednesday. This may be because of these days are near to the resting day (Sunday) or it may be an occasion.

Since the research works used the total Vehicles and Heavy Vehicle volume methods as input in the software as described above in section 3.7.3, the average four days traffic movements on each approaches of the intersection according to the movement directions are summarized as follows.

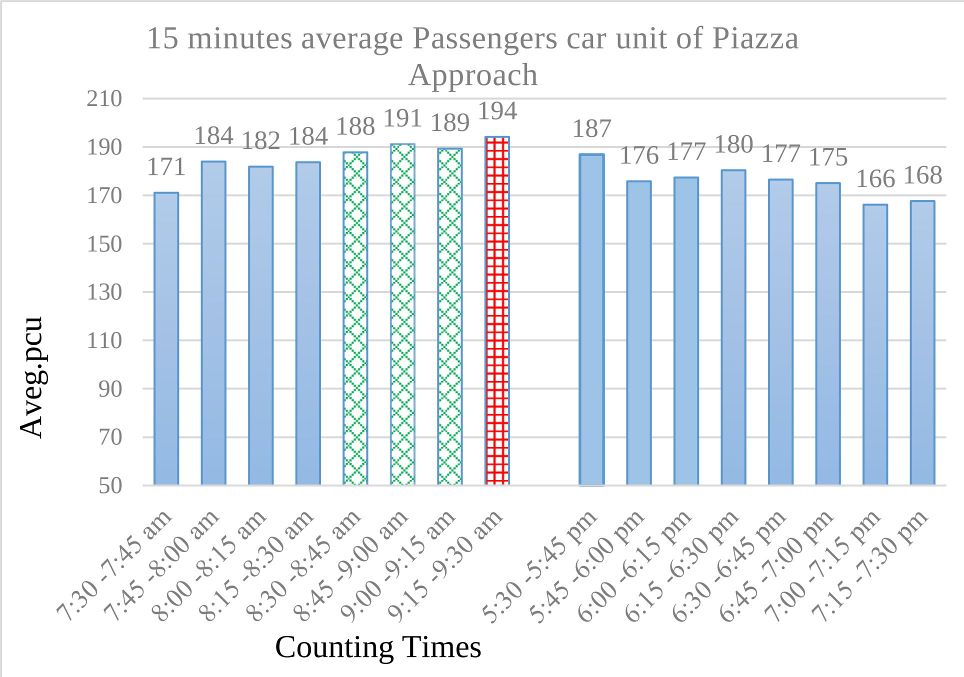
Table 3.3 summarized four days average traffic movements per hour on each approaches.

Dates 20 - 23/07/2015
Time 7:30 am - 9:30 am and 5:30 pm - 7:30 pm

Leg Name.	Left		Through		Right		Total
	Total Veh	HV	Total Veh	HV	Total Veh	HV	PCU/hr
Piazza	81	8	302	46	59	12	442
Posta Bet	69	6	174	38	22	5	265
Beheraw	74	10	286	50	51	4	411
T. Anbesssa	55	8	180	25	35	7	270
							1388

The values in the above table is the final factored traffic input data used in the software. The vehicles are classified as total vehicles and heavy vehicles volume in the three movement directions. As it was observed, there are four legs at the intersection in which the vehicles can enter or/and dissipates. Among these legs, Piazza and Beheraw had relatively higher passenger cars unit than the two rest legs. Piazza and Posta Bet have separate right turn lanes on which vehicles counted data have less effect on the intersection average delay.

The rest two approaches (Beheraw and Tikur Anbessa) have no separate right turn lanes, but they have inclusive shared right turn lanes at the intersection. From all approaches, Beheraw and Paizza approaches had high traffic movements while Posta Bet and Tikur Anbessa approaches had relatively low traffic movements as shown in the above table 3.3. Additionally, average passenger car unit for four days with respect to 15 minutes time described using bar graphs for each approach in the following figures.



Peak 15 minutes volume of the approaches

Peak 15minutes volumes

Figure 3.12 Four days average pcu versus counting times at Piazza approach

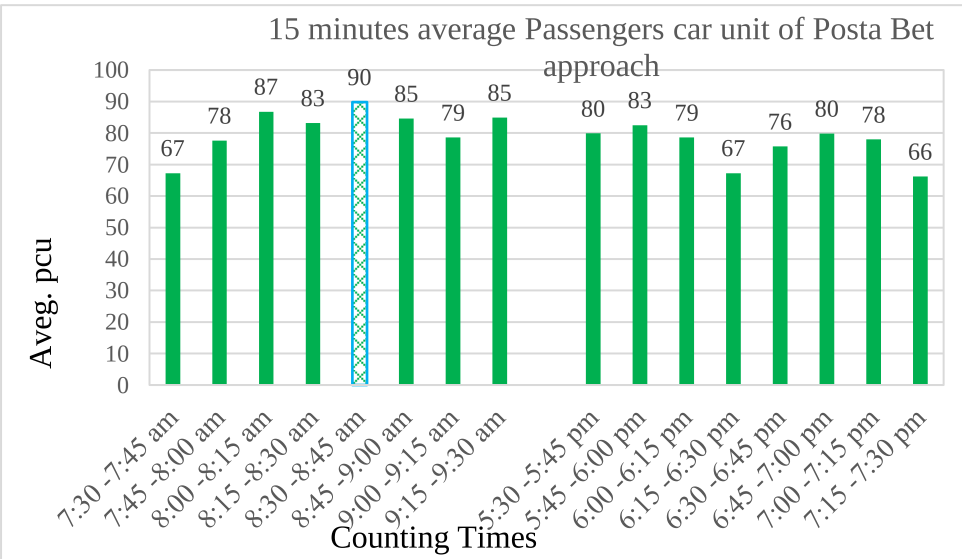


Figure 3.13 Four days average pcu versus counting times at Posta Bet approach

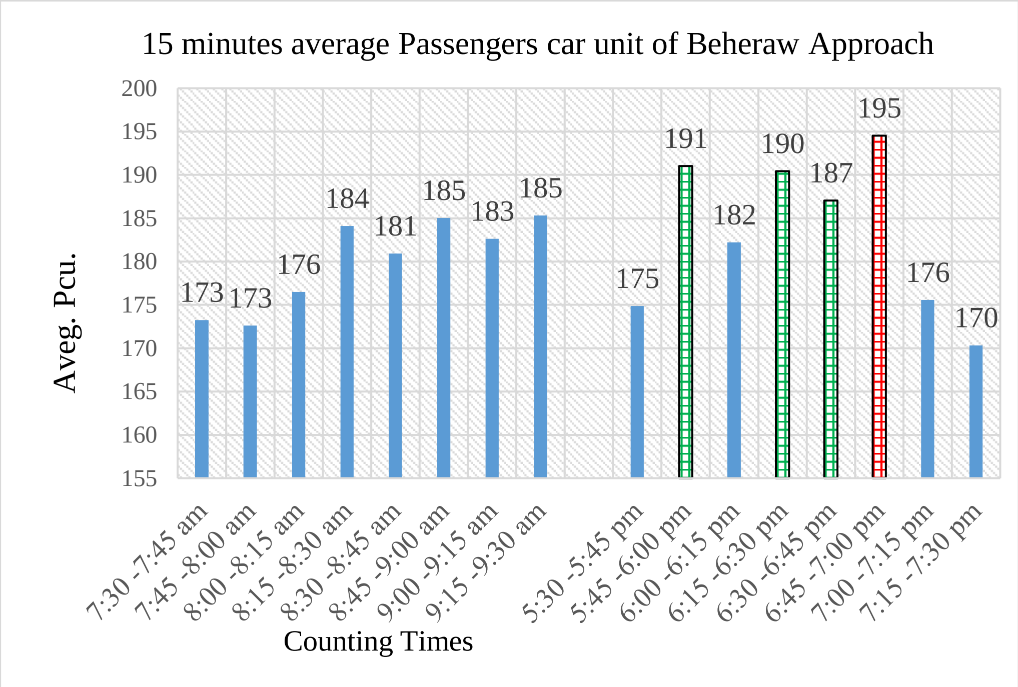


Figure 3.14 Four days average pcu versus counting times at Beheraw approach

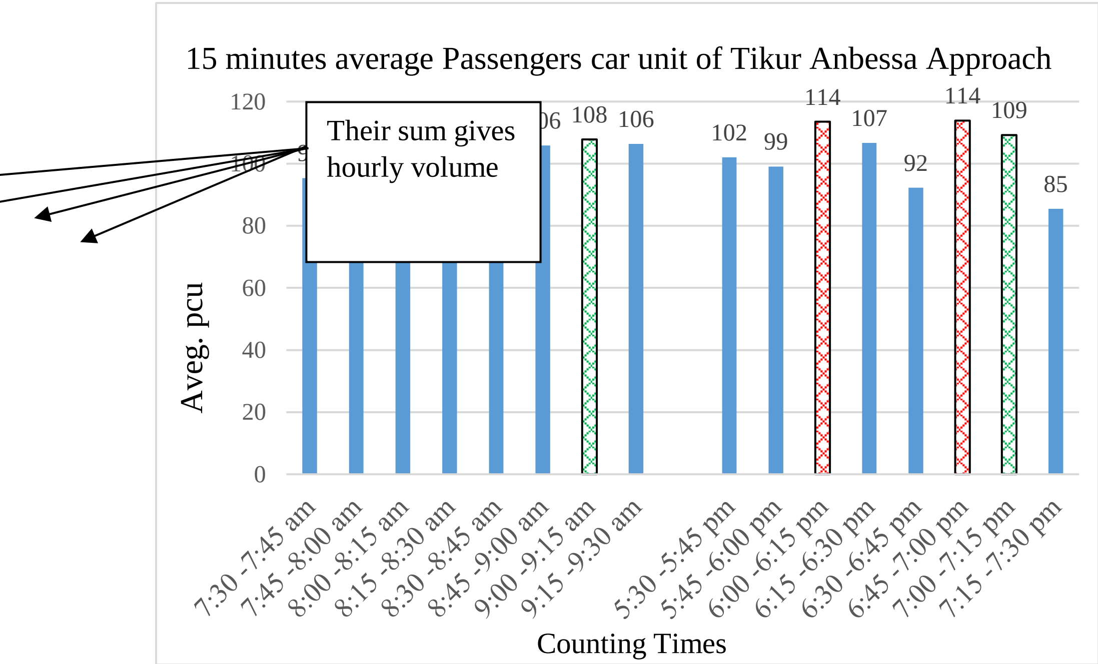


Figure 3.15 Four days average pcu versus counting times at Tikur Anbessa approach

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As shown the above figure 3.11 to figure 3.14, the values along the y-axis shows that four days average 15minutes vehicle movements and the x-axis represents 15minutes counting periods of the movement. The arrows in figure 3.11 and figure 3.12 represents that, the peak 15 minutes volumes of Piazza and Posta Bet approaches which were observed in the peak time of the morning, but for the rest two approaches (Beheraw and Tikur Anbessa) approaches peak 15 minutes volumes were observed in the afternoon. All approaches have different peak 15minutes volumes ($V_{15min.}$), hourly volumes (V), peak hour factors (PHF), and actual (design) flow rates. The peak hour volume of each approaches was obtained by summing up the largest four 15 minute volumes of the intervals within the peak hours. The peak hour factor (PHF) was found by dividing the peak hour volume to four times the peak 15 minute volume and the actual (design) flow rate was also calculated by dividing the peak hour volume to the PHF or simply by multiplying the peak 15 minute volume by four.

Sample calculation for Piazza approach based on vehicles counted data:

Peak hourly volume (PHV) = sum of four peak 15minutes= 194+191+189+188=762pcu

Peak 15minuts volume ($V_{p15min.}$) = maximum 15minute volume =194pcu.

Peak Hour Factor (PHF) = PHV/ (4*$V_{p15min.}$) = 762/ (4*194) = 0.98

Flow rate (FR) = PHV/PHF = 4*$V_{p15min.}$ = 4*194 = 776pcu = design flow rate

Similarly, for the rest approaches the sample calculation was summarized in the following table.

Table 3.4 Summarized flow conditions for each approaches used as input for the software.

Leg Name	PHV (pcu.)	V15min (pcu.)	PHF	Flow rate (pcu.)
Piazza	762	194	0.98	776
Posta Bet	347	90	0.97	360
Beheraw	763	195	0.98	780
T. Anbesssa	445	114	0.98	540

Peak-hour factors over 0.95 are often indicative of high traffic volumes, sometimes with capacity constraints on flow during the peak hours [1].

Therefore; all approaches had high traffic volumes based on the actual counted data as observed in the above table 3.4. Maximum flow rate (780 pcu) of the intersection is equal to rough estimated capacity of the intersection. The above values in table 3.4 were used as input for the software in the data analysis section.

3.7.5 Pedestrian Volume Counts.

Pedestrian counts also made during the two peak hours manually on a working day of a week at the same time with vehicles traffic data. These counts used for crash analysis, capacity analysis, and determining minimum signal timings at signalized intersections incorporating with vehicles data. Volume counts of pedestrians were made at locations such as subway stations, midblock, and crosswalks. The safe and efficient accommodation of pedestrians at intersections is equally important as the provisions made for vehicles. Pedestrian movements should be provided for and their locations controlled to maximize safety and minimize conflicts with other traffic flows. Often, pedestrians are a secondary consideration in the design of roadways, particularly at intersections in suburban areas [1, 18].

For signals (signalised intersections*)*, pedestrian movement data was used for estimating the effect of pedestrians on vehicle movement capacities and signal timings as well as estimating pedestrian performance in the software. The pedestrians' data counted during the peak hours of the working day of a week for 16 hours of each 15 minutes were added and then taking the averages of 16 hours. The following table 3.5 was counted pedestrians data per hour at the study area that used as input for the software.

The pedestrian movement data can affect the vehicles movement at the intersection when the speeds of elder and/or young pedestrians' will be high or small below the threshold value of walking speed (1.2m/s) as per Highway Capacity manual 2000.

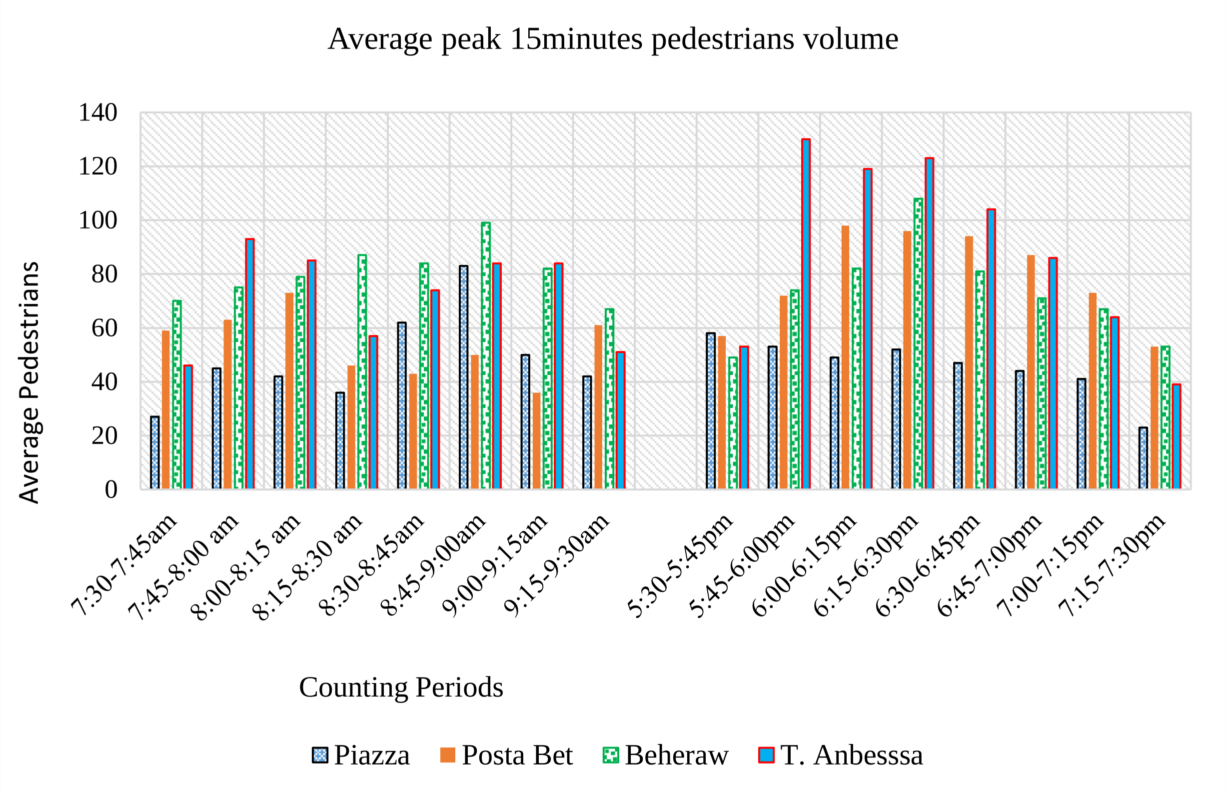

Figure 3.16 Average four days peak 15minutes pedestrians volume.

Based on the above figure 3.15, average 15minute pedestrians volume for each approaches varies from time to time and the largest value (130 pedestrians per 15minutes) was counted at Tikur Anbessa approach in the afternoon peak hour at the time of 5:45 pm to 6:00 pm. The reason for the highest counted pedestrian value on this approach, there are existing nearby governmental institutions like Immigration head office, Black Lion Hospitals and Ethiopian Broad cast Corporation and bus parking on this direction which have some effects on traffic flow conditions. The second largest counted pedestrian value was recorded at Beheraw approach in the afternoon and on the other hand the lowest value was counted at piazza approach. Pedestrian data organization is shown under the appendixes. Therefore; the SIDRA software requires average pedestrians' movement per hour of the peak times along each directions and the peak hour factors of each legs as input for the software in the data analysis. The research work used four days averages pedestrian per hour and those necessary data used as input for the software are summarized in the following figure 3.16 and table 3.5.

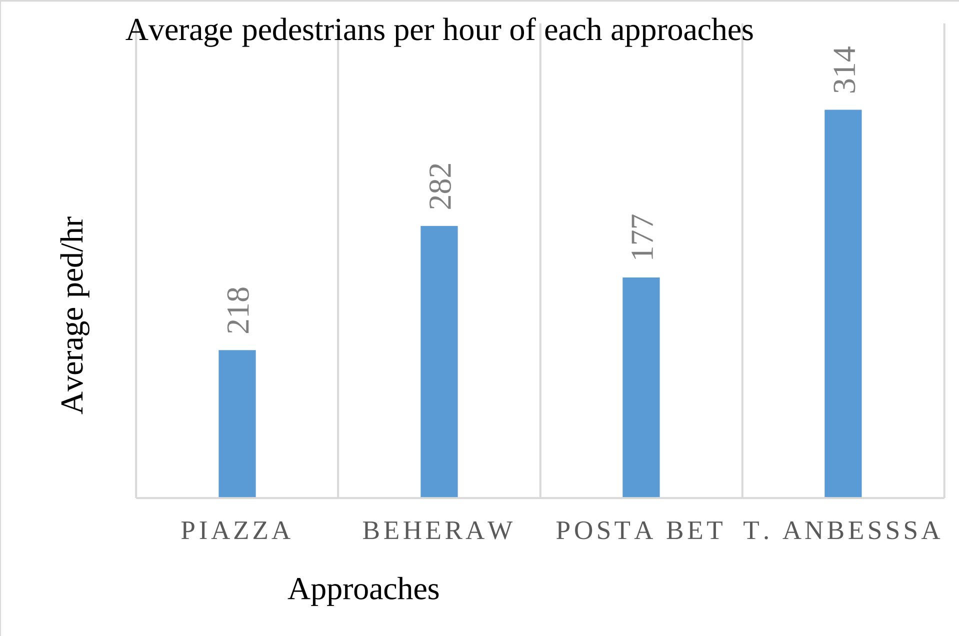

Figure 3.17 Average pedestrian counted data per hour of each approaches

Sample calculation for Piazza approach based on counted pedestrians data:

Peak hourly volume (PHV) = sum of four peak 15minutes= 93+72+68+63=296 pedestrians

Peak 15minuts volume ($V_{p15min.}$) = 93 pedestrians.

Peak Hour Factor (PHF) = PHV/ (4*$V_{p15min.}$) = 296/ (4*93) = 0.80

Flow rate (FR) = PHV/PHF = 4*$V_{p15min.}$ = 4*93 = 372 ped/hr.

Similarly, for the rest approaches the sample calculation was done and summarized in the following table 3.5.

Table 3.5 Summary of pedestrians flow value used as input for the software.

Leg Name	PHV (ped.)	Vp_{15min} (ped.)	PHF	Flow rate (ped.)	Average (Ped. /hr.)
Piazza	296	93	0.80	372	218
Posta Bet	435	118	0.92	472	117
Biheraw	488	133	0.92	532	282
T. Anbesssa	795	220	0.90	880	314

From the above table 3.5, the peak hour pedestrian volume for each approaches was summarized and the largest values for peak hour volume, 15 minute's volume, flow rates and total pedestrian per hour were observed from Tikur Anbessa approach while the smallest values for the same parameters were recorded on the Piazza approach as shown in the following figure 3.17.

Figure 3.18 Photo shows pedestrian platoon along Tikur Anbessa and Piazza approach.

3.8 Signal Data

Signal operation and timing have a significant impact on intersection performance. The development of a signal timing plan addresses all user needs at a particular location. Ideally, the length of the green display should be sufficient to serve the demand present at the start of the green phase for each movement and should be able to move groups of vehicles, or platoons, in a coordinated system. There are three types of traffic signal controlling modes.

a) Pre-timed, in which a sequence of phases is displayed in repetitive order. Each phase has a fixed green time change and clearance interval that are repeated in each cycle to produce a constant cycle length.

b) Fully actuated, in which the timing on all of the approaches to an intersection is influenced by vehicle detectors. Each phase is subject to a minimum and maximum green time, and some phases may be skipped if no demand is detected. The cycle length for fully actuated control varies from cycle to cycle.

c) Semi actuated, in which some approaches (typically on the minor street) have detectors and some of the approaches (typically on the major street) have no detectors [1].

Signal phasing can provide for protected, permitted, or not opposed turning movements. A permitted turning movement is made through a conflicting pedestrian or opposing vehicle flow. Thus, a left-turn movement concurrent with the opposing through movement is considered to be permitted, as is a right-turn movement concurrent with pedestrian crossings in a conflicting crosswalk. Protected turns are those made without these conflicts, such as turns made during an exclusive left-turn phase or a right-turn phase during which conflicting pedestrian movements are prohibited.

3.8.1 Basic Signal Timing Elements

At the study area of a signalized intersection, three signal indications are displaying: green, yellow, and red. The basic timing elements within each phase include the green interval, the effective green time, yellow or amber interval, and the all-red interval. Each of these elements is described below.

The green interval is the period of the phase during which the green signal is illuminated. The yellow or amber interval is the portion of the phase during which the yellow light is illuminated. The effective green time is contained within the green interval and the amber interval. The effective green time, for a phase, is the time during which vehicles are actually discharging through the intersection. The all-red interval is the period following the yellow interval in which all of the intersection's signals are red. The inter-green interval is simply the interval between the end of green for one phase and the beginning of green for another phase. It is the sum of the yellow and all-red intervals [25].

3.8.2 Cycle Length of the study area

Cycle length is composed of the total signal time to serve all of the signal phases including the green time plus any change interval. Longer cycles will accommodate more vehicles per hour but that will also produce higher average delays. The best way is using the shortest practical cycle length that will serve the traffic demand. The cycle length includes the green time plus the vehicle signal change interval for each phase totaled to include all signal phases. A number of methods have been used to determine cycle lengths [1]. When the cycle length has been determined the vehicle signal changes are deducted giving the total cycle green time which can be proportioned to each signal phase on the basis of critical lane volumes. The individual signal phase times are then the proportioned time plus the vehicle change interval on each phase [23]. The study area intersection is pre-timed or fixed time traffic signal with four (4) phasing in which flow from each approach is put into a single phase avoiding all conflicts. This type of phase plan is ideally suited in urban areas where the turning movements are comparable with through movements and when through traffic and turning traffic need to share the same lane [22]. No need of developing the equations to determine the signal timing because the software requires the actual field observation values. Since the signal of the intersection at the study area was digital display, the time for each signal and the cycle length along the Major and Minor Street were in table 3.6 below used as input for the software as shown.

Table 3.6 Values of signal timing at the study area**.**

	Major streets		Minor streets	
Signals	Piazza approach	Beheraw approach	Posta Bet approach	Tikur Anbessa approach
Green (sec.)	40	40	30	30
Yellow (sec.)	3	3	3	3
Red (sec.)	111	111	121	121
Cycle Length	160 seconds = sum of all green + yellow + (all- red =2sec.)			

[Source: field observation]

3.9 Road Geometric Data:

Road geometric data include all of the relevant information needed by the software as input like approaches grade, approach operating speed, the number and width of lanes, medians and exit operating speed conditions.

3.9.1 Approach Grades

Approach grades are needed for all approaches, and are expressed as a percentage, with positive values for upgrade approaches and negative for downgrade approaches as shown in the following figure 3.18. The grade affects the calculation of critical gaps **(**headways**)** that is the time in seconds, between two successive vehicles as they pass a point on the roadway, measured from the same common feature of both vehicles (for example, the front axle or the front bumper).

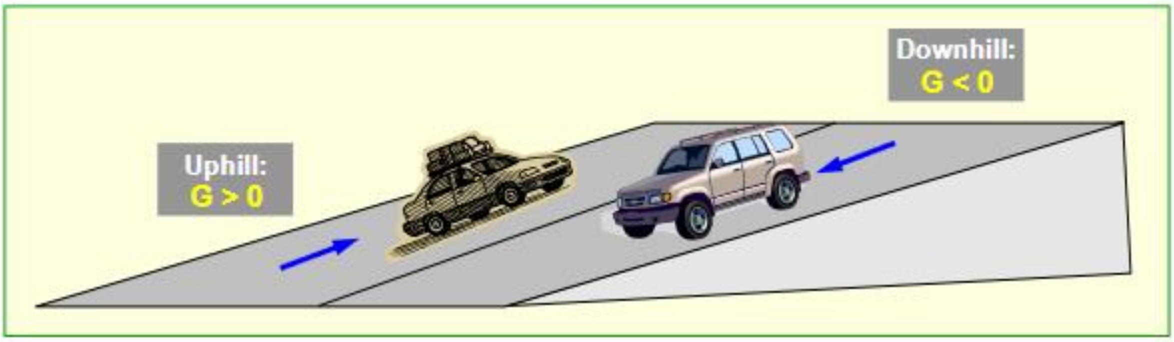

Figure 3.19 Grade definition at approach lanes and exit lanes

Therefore; based on Highway capacity manual the values of approach grades with speed limit of 30km/hr. for turning lanes and 60km/h for through lanes were used as input for the software.

The values for grades of each approaches was calculated by dividing the elevation differences of two points to the distance between the two points times 100 in order to put in percentage. The elevation difference of two points were obtained using Geographic Positioning system

(GPS).

Figure 3.120 Photo shows while taking the elevation of Beheraw approach with GPS.

Here under is the sample grade calculation for Beheraw approach that was used as input for the software.

Elevation at stopping line of Beheraw approach = 2369m

Elevation at 100m back of stopping line of Beheraw approach = 2367m

Then grade of Beheraw approach (G) = $\frac{\Delta y}{\Delta x}$ = $(\frac{2369-2367}{100})*100 = 2\%$

Similarly, for the rest approaches the values of approach and exit grades are summarized in the following table 3.7.

Table 3.7 Values of approach and exit grades at the study area.

Name of approaches	Approaches grade values (%)	Exit grades values (%)
Piazza	-3.5	3.5
Posta Bet	2.3	-2.3
Beheraw	2.4	-2.4
TikurAnbessa	-3.6	3.6

Grade data for approach and exit lanes on the same intersection leg should have opposite signs and the grade parameter was used for saturation flow estimation for signalised intersections only.

3.9.2 The Number and Width of Existing Lanes

A traffic lane is a part of the roadway set aside for the normal movement of a single stream of vehicles. At the study area the existing number of lanes are three lanes-two way direction from piazza to Beheraw (North to South) with 3.6m lane width and 2m width of medians while two lanes-two way direction from Tikur Anbessa to Posta Bet with 3.6m lane width and 1.8m median width. The lane width and numbers could be affected by traffic volume and vehicles dimensions. Thus, numerical values of the numbers and width of lanes including median width values were needed as input data for the software.

The lane discipline at the study area indicates the movements allocated to the lane in accordance with their turn designations. There are exclusive lanes which involves (left, through and right turns) and shared lanes which includes (left and through, through and right, left, through and right turns). Figure 3.20 shows the lane discipline or flow movement at every approaches in the study area.

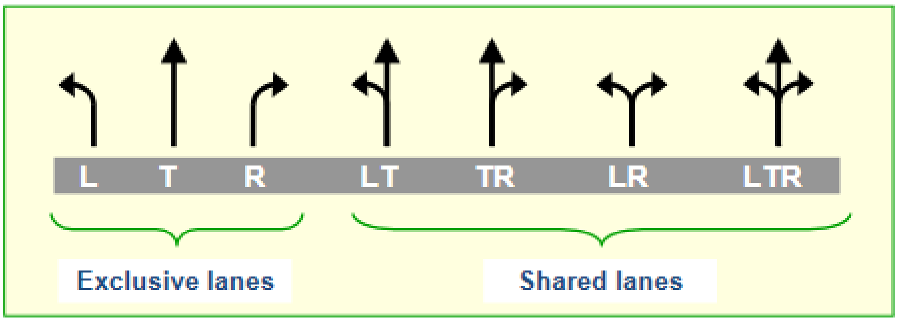

Figure 3.21 Lane discipline combinations (L = Left, T = Through, R = Right)

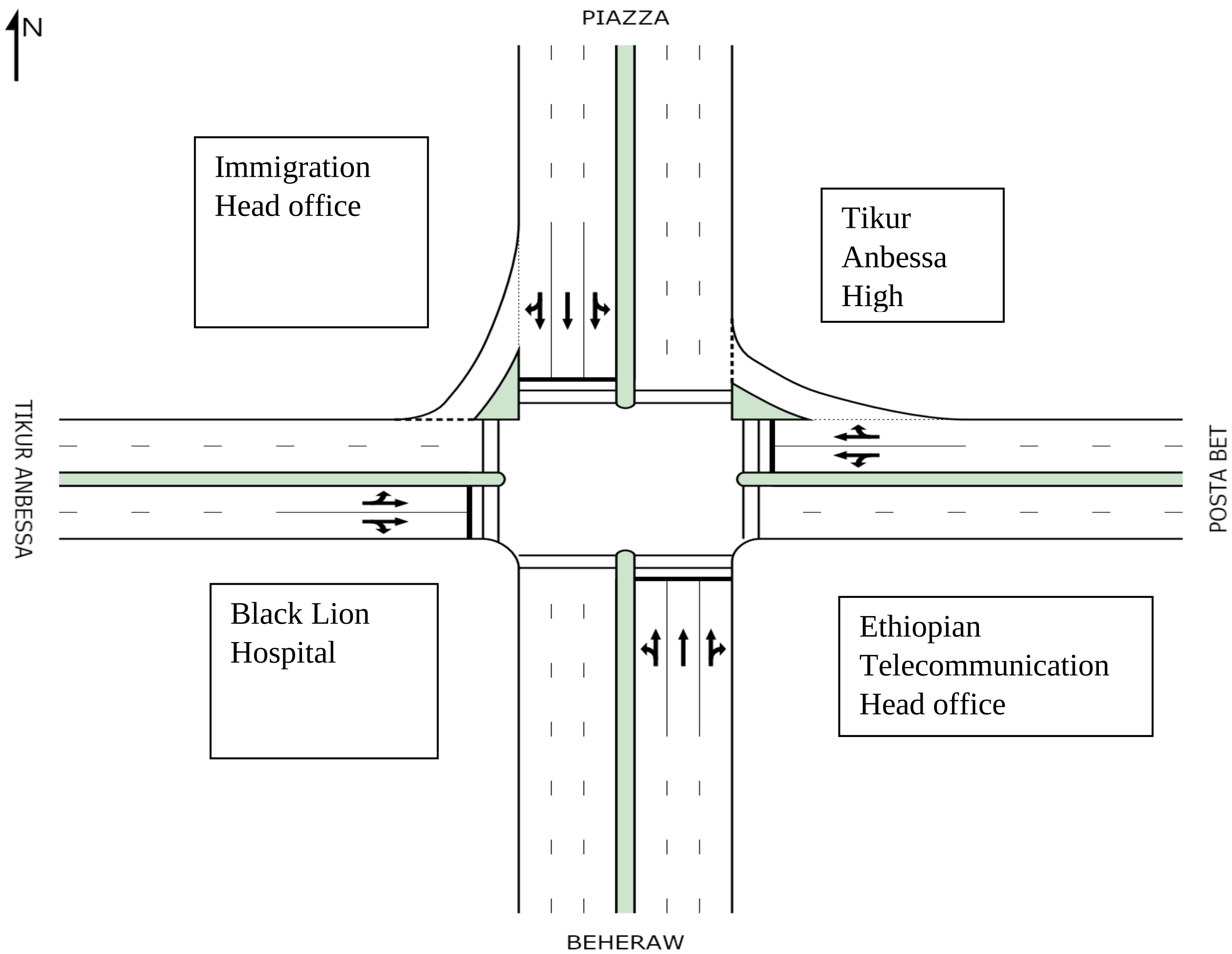

Figure 3.22 Layout of existing intersection in the study area.

3.10 Basic saturation flow rates of Vehicles and pedestrians

SIDRA intersection software uses basic saturation flow values in through car units per hour (tcu/h) for signalised intersections. Saturation flow is a macro performance measure and an indication of the potential capacity of a junction operation. According to the Highway Capacity Manual 2000, the adjustable value of saturation flow rate at an intersection and the recommendable values ranges in between 1500 and 2000 tcu/h/lane with the corresponding headway of 2.4 and 1.9 seconds. Since, there was no related research done for standard saturation flow rate in Ethiopia, the research work used the following saturation flow rate equation developed in South Africa in 2007, by Bester, C.J. and Meyers, W.L which is majorly depends on the following factors.

- Speed limits – urban traffic intersection speed limits of 30 km/h for gradient and 50 km/h for flat area was used.
- Gradient - intersections on different gradients were observed for traffic flow up hill and traffic flow downhill. Thus the research work used grades obtained from the field.
- Number of through lanes.

The equation used for the estimation of saturation flow rate at the study area is:

$$SFR = 990 + 288TL + 8.5SL - 26.8G \qquad (3.5)$$

Where SFR = Saturation flow rate, TL = Number of through lanes

SL = Speed limit, G = Gradient in percent

Therefore, using the data collected from the study area and linear regression analysis, the saturation flow rate and the corresponding saturation headway are tabulated in the following table 3.8 for each approaches by taking 30km/h of speed limit.

Table 3.8 Saturation flow rate of each approaches at study area

Approaches	Number of exclusive through lanes	Gradient (%)	Speed limit(km/h)	Saturation flow rate(tcu./h)	Headway(sec.) H=3600/SFR
Piazza	1	-3.5	30	1627	2.2
Posta Bet	0	2.3	30	1184	3.0
Beheraw	1	2.4	30	1469	2.5
T.Anbessa	0	-3.6	30	1341	2.6

The calculated saturation flow rate for Piazza and Beheraw approaches were the maximum saturation flow rates at the intersection. Thus, the saturation headway in the above table 3.8 obtained using equation (3-5), that is the ratio of the number of seconds in an hour (3600) to saturation flow rate of the corresponding approaches.

An idealized view of saturation flow at a signalized junction is illustrated in the following figure 3.22.

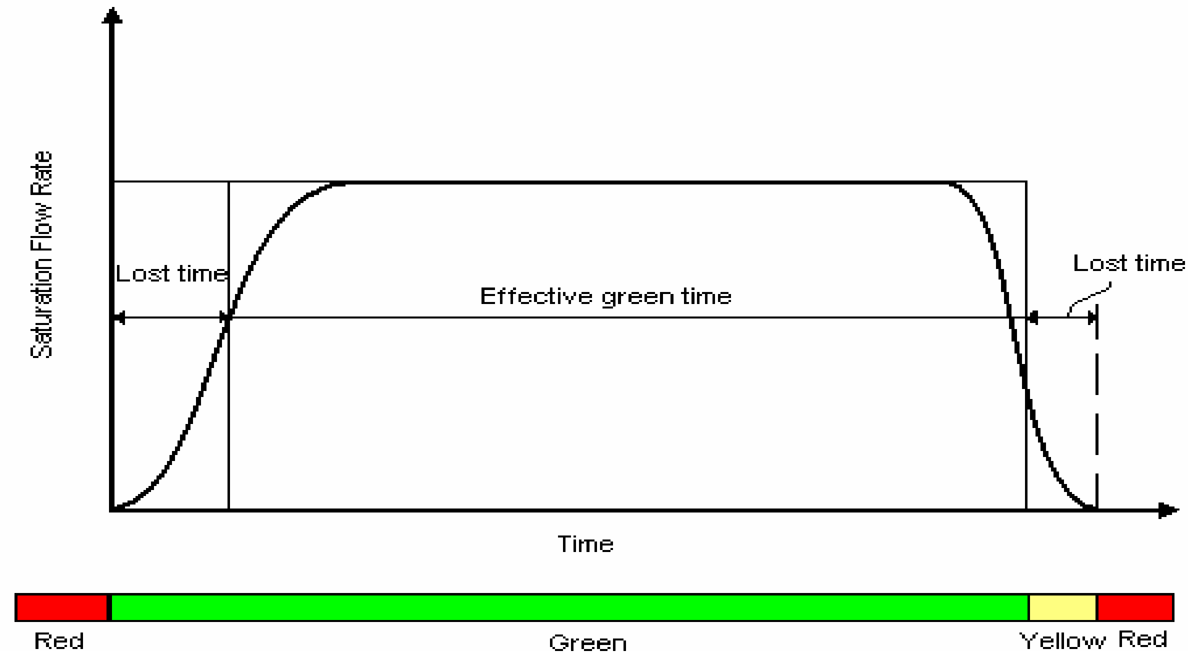

Figure 3.23. The flow of traffic during the green period from a saturated approach

Note from the figure that as the traffic signal shows green, there is first a very short gap as the first driver reacts to the signal change. The rate of vehicles crossing the stop line increases as vehicles accelerate to the speed determined by the cars they are following. Vehicles soon reach a state where they are following one another at a constant headway. In a saturated junction, the queue formed in the red time will be too long to clear in the green period and so cars will follow each other at constant spacing during the green period. The flow rate will start dropping at an increasing rate when the signals are in yellow time and then stop when the signals turn red. The lost time is the time from the start of green to a point where vehicles are flowing at half the maximum flow plus the time from where vehicles are flowing at half the maximum flow at the end of saturation to the beginning of red time.

For pedestrian base saturation flow, the software was developed to use the maximum saturation flow of 12,000 ped/h. Therefore, the research work used these value for the analysis of the software.

3.11 Optimum Cycle Length of the existing intersection

Optimum cycle time is the time at which relatively small vehicles delay recorded. This means, if the cycle is too long, delays will be lengthened, as vehicles wait for their turn to discharge through the intersection. Figure 3.23 provides a graphical representation of this phenomenon.

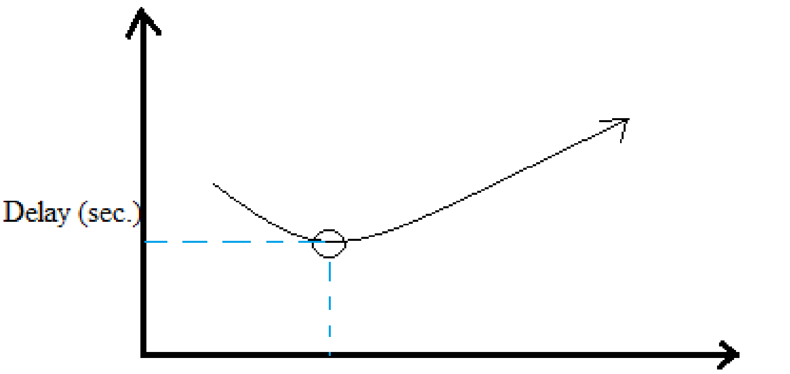

Figure 3.24 Cycle Length versus Delay

Several methods for solving this optimization problem have already been developed, but Webster’s equation is the most prevalent. Webster's equation, which minimizes intersection delay, gives the optimum cycle length (C_O) as a function of the lost times and the critical flow ratios. Many design manuals use Webster's equation as the basis for their design and only make minor adjustments to suit their purposes. Webster's equation is shown below.

$$Co = \frac{1.5L + 5}{1 - \sum (V/SFR)} \quad (3.6)$$

L = (Starting loss + all-red) * number of phase.

= (2sec. + 2sec)* 4 = 16 sec.

Y_C = V/SFR = Ratio of flow rate (PCU/h) to the saturation flow rate. This value can be obtained from the following table 3.9 based on tables 3.3 and 3.8.

Table 3.9 Critical flow ratio for each approaches

Approaches	Through volume(veh/h)	SFR(tcu/h)	Y_C (sec.)
Piazza	302	1627	0.19
Posta Bet	174	1184	0.15
Beheraw	286	1469	0.20
Tikur Anbessa	186	1341	0.14
Summation of Y critical			= 0.68sec.

Then, using the above equation 3.6, the optimum cycle length became:

$C_o = [1.5(16) +5]/ (1-0.68) = 91$ seconds, But the analysis of the revised intersection was done by 96 seconds as optimum cycle length. Because the signal system is pre-timed so that it needs some extra few seconds which guarantees for lost time within different cycles.

3.12. Other necessary data needed by the software.

Those obtained from field observations, HCM, related research and the software users guides are as follow:

Estimated movement data from field observation

- ✓ Queue space for light vehicles = 5m
- ✓ Queue space for heavy vehicles = 12m
- ✓ Vehicles length for light vehicles = 4m
- ✓ Vehicles length for heavy vehicles = 10m
- ✓ Pedestrian queue space = 1m.
- ✓ Annual population vehicles growth rate =1.6%[12]
- ✓ Population growth rate of Addis Ababa City = 3.8% [3].

Gap Acceptance: The process by which an opposed (minor) stream vehicle accepts an available gap in the opposing (major) stream for entering (departing from queue or merging)

- ✓ Gap acceptance = 2seconds

Estimated Gap acceptance from HCM Critical Gap (Headway): The minimum time (Headway) between successive vehicles in the opposing (major) traffic stream that is acceptable for entry by opposed (minor) stream vehicles.

- ✓ Critical gap =5seconds

Follow-up Headway: - is the average headway between successive opposed (minor) stream vehicles entering a gap available in the opposing (major) traffic stream

- ✓ Follow up headway = 3seconds

End Departures represents the *maximum* number of vehicles that can depart after the end of the displayed green period at signalised intersections.

- ✓ End departures = 2 vehicles

Table 3.10 Pedestrian input data for calculating pedestrian timing data

Parameters	SIDRA Standard	HCM version
Minimum walking time	5s	7s
Crossing speed	1.2m/s	1.1m/s
Minimum crossing time	5s	5s
Pedestrian start loss	2s	2s
Pedestrian end gain	3s	4s

✓ Extra Bunching

The Extra Bunching parameter is a general parameter applicable to any type of intersection. The purpose of the *Extra Bunching* parameter is to adjust the proportion of free vehicles in the traffic stream according to the proximity of upstream signalised intersections. This parameter is used mainly in order to allow for the effect of upstream signals on capacity of signal-controlled intersections. Values for Extra Bunching are provided in the SIDRA INTERSECTION User's Manual as summarized in the following.

Table 3.11 Rough guide Extra Bunching Values for each approaches

Distance to upstream signals(m)	<100	100-200	200-400	400-600	600-800	>800
Extra bunching (%)	25	20	15	10	5	0

[Source: SIDRA user guide]

Therefore; for the study area only Beheraw approach have 15% of extra bunching because of upstream signal distance is found within the range of 200-400m and the rest approaches have zero values.

CHAPTER FOUR

ANALYSIS, RESULT AND DISCUSSION

4. Introduction

During the field survey, road geometry, signal characteristics and traffic flow conditions like vehicle queues and travel patterns through the intersection were examined. These were identified and noted whether it would interfere with upstream intersections by not using SIDRA intersection software. Since there was no precise methodology during the design phases of any intersection which usually guided by the geometry of the intersection, trial and error procedure usually adopted.

To summarize the research work; first the researcher has been tried to analyze the existing intersection using the collected data as it is (Do nothing) with the existing phase numbers and the cycle time. Then by improving those parameters (cycle time and phase numbers) the intersection was examined again. However, if the intersection did not show performance improvement after adjusting those parameters, the researcher had have to suggest improvement of the intersection geometry.

Therefore; the outputs of the SIDRA intersection software were presented below based on Fixed Time Signal, Practical Cycle time of 160 seconds and four phasing system.

4.1 Analysis of the Intersection (Do nothing)

4.1.1 Capacity of "do nothing"

The maximum number of vehicles that can reasonably be expected to pass over a study area of the intersection in one direction during a given time under signalization conditions can be used as a reference to gauge the current operation of the intersection. Total capacity per movement represents the Total Flow / Degree of Saturation (veh/hr.). Therefore; the capacity of each legs of fixed time signalized intersection at the study area using practical cycle time of 160 seconds is shown in the following figure 4.1.

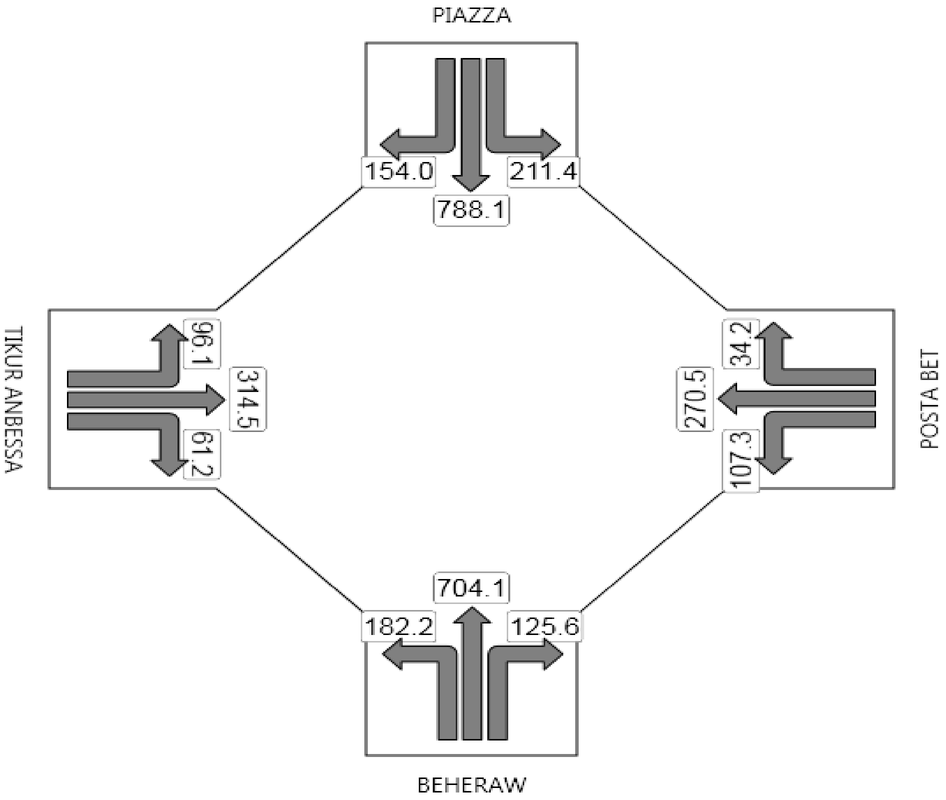

Figure 4.1 Capacity of the existing Hager Astedader signalized intersection

The above figure 4.1 shows the maximum sustainable flow rate at which vehicles were traversed at the study area intersection during a specified time period. The maximum capacity of the study area intersection was determined from the maximum capacity of each legs or each lane groups which is summarized in the following table 4.1.The maximum numerical value in any lane group represents the capacity of the intersection.

Table 4.1 Summarized capacity of the existing intersection and approaches

Entering	South	East	North	West	Intersection
Capacity(pcu/hr)	704.1	270.5	788.1	314.5	788.1

As it is seen from the above figure 4.1, the software output for capacity of the existing signalized intersection was 788.1 pcu/h. This shown that the software justified the Webster's rule which said the capacity of a given intersection is equal to the maximum actual flow rate (780pcu/h) of each approaches based on table 3.4. The maximum passenger cars unit per hour had observed from North or piazza approach that is also the capacity of the existing study area intersection under 160 seconds of cycle length with four phasing.

4.1.2. Traffic Queue for "do nothing"

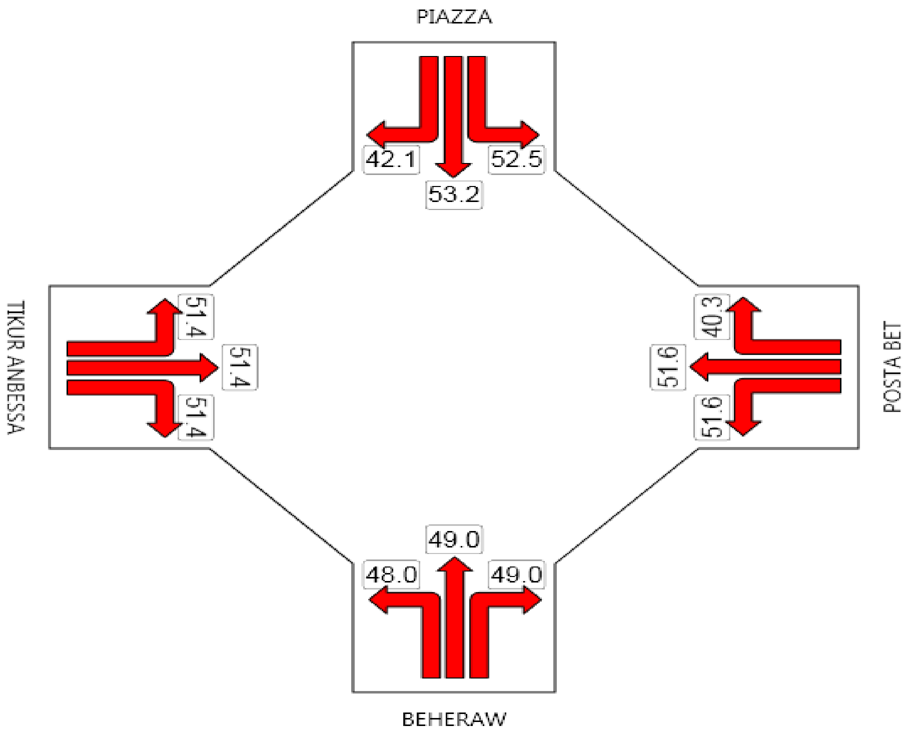

Figure 4.2 Queue for each lane groups used by movement (vehicles)

The figure 4.2 shows the closely spaced collection of vehicles at the intersection approaches during the red interval and was formed when demand exceeds capacity of the signalized intersection from the start of an effective green period ended. Figure 4.2 represents large number of vehicles queued at an approach to a signalized intersection due to long cycle time. Therefore; queue from the Northern (Piazza approach) was the governing one which was considered as the queue of the existing intersection and summarized as follows.

Table 4.2 Summarized queue of the existing intersection and approaches

Entering	South	East	North	West	Intersection
Queue (veh.)	49	51.6	53.2	51.4	53.2

4.1.3. Degree of saturation (V/C) for "do nothing"

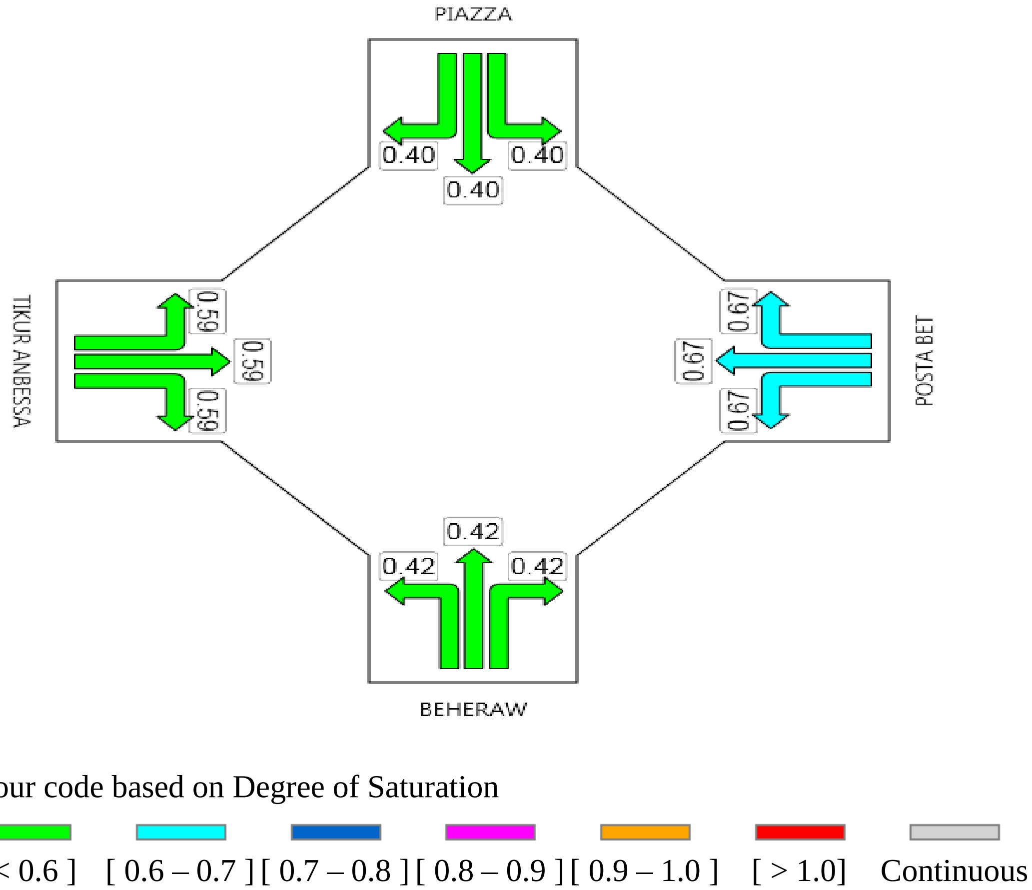

Figure 4.3 Demand Volume to Capacity (v/c) ratio at Hager Astedader signalized intersection.

Figure 4.3 referred to as the ratio of demand volume to capacity of each lane groups. The volume to capacity ratio from the software output for Hager Astedader intersection was less than one as shown in the above figure 4.3 which represents the current traffic operation at the study area is moderately congested flow condition. Based on Highway Capacity Manual, when v/c ratios are less than 1.0, the intersection is serving under capacity. Even if the v/c ratio is less than 1.0, there may be considerable delay by looking at its level of service. For an individual lane group or for the overall intersection, departure volumes may not equal to arrival volumes due to the existing signal or phase allocations. If so, changes or modification in either or both should be considered especially on cycle length.

The degree of saturation for each lane group can determine the degree of saturation for the signalized intersection. This means that the maximum degree of saturation of any lane group from all approaches represents the degree of saturation of the intersection which is summarized in the following table 4.3.

Table 4.3 Summarized degree saturation of the existing intersection and approaches

Entering	South	East	North	West	Intersection
Degree of Saturation	0.42	0.67	0.40	0.59	0.67

For the study area signalized intersection, demand volume to capacity ratio was higher in the Eastern leg than the others. In fact all approaches were near stable flow results in a wide range of delay even if the capacity exceeds the available demand of the intersection which brings excessive delays as per volume to capacity ratio threshold.

Table 4.4 Volume-to-capacity ratio threshold at signalized intersection

V/C ratio	Assessment
< 0.5	Intersection is operating under capacity. Excessive delays are not experienced
0.5 – 0.74	Intersection is operating near its capacity. Higher delays may be expected, but continuously increasing queues should not occur, Moderate congestion can be occurred. Signal improvement will be required to reduce delay.
0.75 – 1.00	Unstable flow results in a wide range of delay. Intersection improvements will be required soon to avoid excessive delays.
>1.00	The demand exceeds the available capacity of the intersection. Excessive delays and queuing are anticipated.

Source [Victoria Transport Policy Institute]

Therefore; the existing signalized intersection requires signal or/and phase improvements to avoid the anticipated excessive delays. The types of improvement includes shortening of cycle length of the signal, reduce phase numbers and some roadway geometric modifications like left and right turn movements in order to obtain less delay at the study area.

4.1.4 Level of Service (LOS) for do nothing.

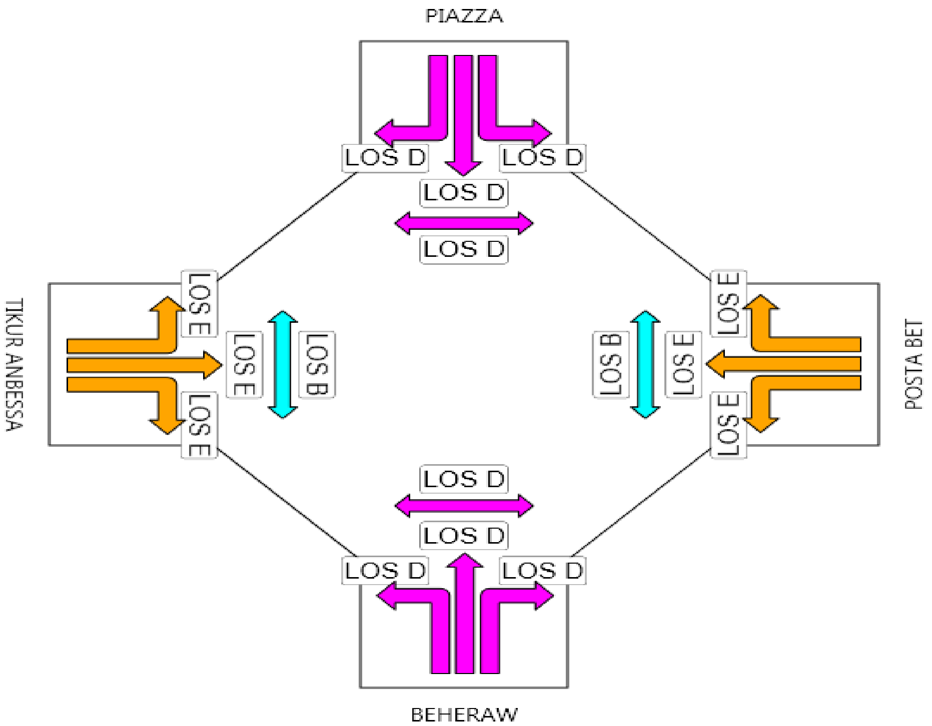

Figure 4.4 The Level of service of Hager Astedader signalized intersection

Figure 4.4 represents the measure of highway's operating conditions or quality of service under a given demand. This information was used for the researcher to evaluate or visualize and understand the traffic flow conditions at intersection of the study area. The level of services at Hager Astedader signalized intersection was E from the software output as shown in the following table 4.5. This implies that the flow condition was unstable or worst because of bad signal allocation as per Victoria Transport Police Institute.

Table 4.5 Summary of level of service for "do nothing"

	South	East	North	West	Intersection
LOS	D	E	D	E	E

Colour code based on Level of Service

LOS A LOS B LOS C LOS D LOS E LOS F Continuous

4.1.5 Delay (average) for "do nothing"

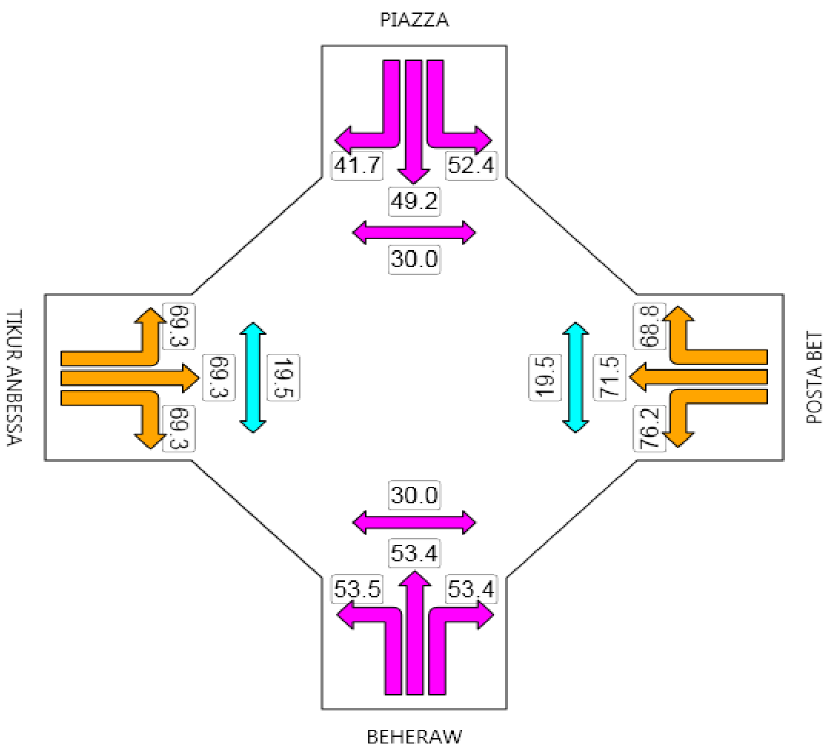

Figure 4.5 Average control delay per vehicle and average pedestrian delay (seconds).

The stopped time per vehicle at the intersection due to traffic signal controller (in seconds per vehicle), usually calculated separately for each lane groups. Each numerical values in front of the arrows represent average delay times for each lane groups. Delay for single approach was obtained by an average of individual lanes but delay for an intersection has been noted below as an average of each approaches or legs that was computed by the software itself. The average control delay per vehicle of the study area from the software output is displayed in the following table 4.6.

Table 4.6 Summarized average delay of the existing intersection and approaches

Entering	South	East	North	West	Intersection
Average delay (sec/veh.)	54.4	72.5	48.8	69.3	58.7
LOS	D	E	D	E	E

From table 4.6 relatively high delay was observed on the Eastern (Posta Bet) approach which was 72.5 sec/veh (1.2 minutes per vehicles) which represents the level of service E. But, the existing average delay for the intersection obtained from the software output was 58.7 seconds per vehicles as summarized in the above table 4.6.

4.1.6 Movement Timing

Phase times determined by the program

Phase	**A**	**B**	**C**	**D**
Green Time (sec)	40	40	30	30
Yellow Time (sec)	3	3	3	3
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	45	45	35	35
Phase Split	28 %	28 %	22 %	22 %

Sequence: Four Phasing

Input Sequence: A, B, C, D

Output Sequence: A, B, C, D

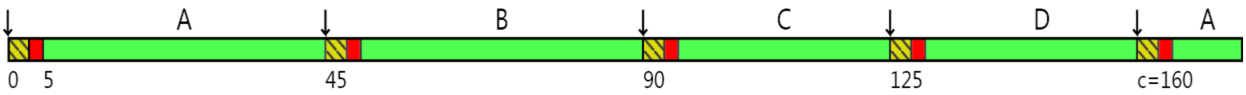

Figure 4.6 Diagram of signal phasing of the existing intersection

The number shows the timing of the actual existing signals for each four phases, for example;

45 seconds = yellow time for phase A + all-red time for phase A + green time of phase A

= 3 seconds + 2 seconds

90 seconds = 45 seconds of phase A+ yellow time of B + green time of phase B

= 45+ 40 + 3 + 2 = 90 seconds.

The rest diagrams and tables of the output are arranged under the appendixes.

4.2 Re-analyzed intersection by adjusting the cycle time and phase number.

As software results shown in this chapter, the intersection performance was not good, because moderately congested with the level of service E. Therefore; in order to decrease the delay experienced and increase the performance of the intersection, at least to attain the intersection with the level of service C rather than the existing level of service E, adjustment of cycle time and phase number has been done with the optimum cycle time (C_O = 96 seconds) calculated above. Thus, by adjusting those parameters it could be increased the performance or decrease the current average delay of the study intersection. Based on trial and error of adjusting these parameters or using optimum cycle, suitable performance of the study area intersection was obtained at cycle time of 96 seconds with the corresponding three phases rather than the existing cycle time of 160 seconds and four phasing.

Table 4.7 Adjusted cycle time of the study intersection with two phasing

Signals	Phase A (From North)	Phase B (From south)	Phase C (West-East)
Green	24	24	33
Yellow	3	3	3
All-Red	2	2	2
Cycle time	96 seconds		

4.2.1 Movement Timing

Phase times determined by the program
Sequence: 3- Phasing
Input Sequence: A, B, C
Output Sequence: A, B, C

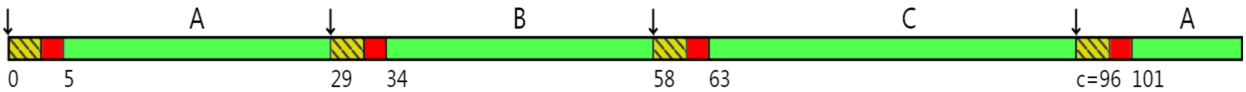

Figure 4.7 Diagram of adjusted signal timing of the existing intersection

The number represents sum of the given signal timing and finally gives cycle time (C= 96 sec.). The reason for allocating larger green time for phase C, it comprised the number of traffic volume comes from two directions (West-East direction).

4.2.2 Revised Capacity

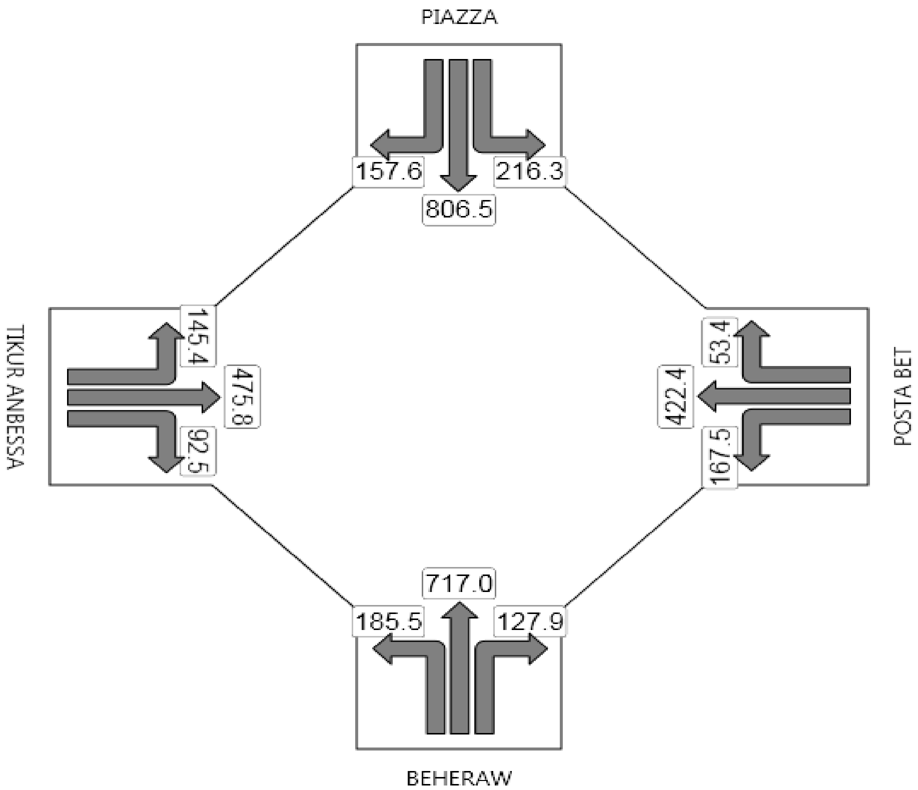

Figure 4.8 Adjusted Capacity of the existing signalized intersection

Figure 4.8 represents improved capacity of the signalized intersection for each lane groups compared to non-adjusted (existing signalized intersection). The "do nothing"" capacity of the intersection was 788.1 pcu/h as mentioned in section 4.1, while for the adjusted one became 806.5 pcu/h which is the maximum capacity of the intersection as observed from Piazza approach. This implies that as the number of vehicles passing the intersection become increase, the average delay at the intersection will decreases based on their green signal time. Not only the capacity of the intersection increased but also capacity of each lane groups was increased. The adjusted capacity of the intersection is summarized in the following table 4.7.

Table 4.8 Adjusted capacity of the existing intersection and approaches

Entering	South	East	North	West	Intersection
Capacity(pcu/h)	717	422.4	806.5	475.8	806.5

4.2.3 Revised traffic Queue

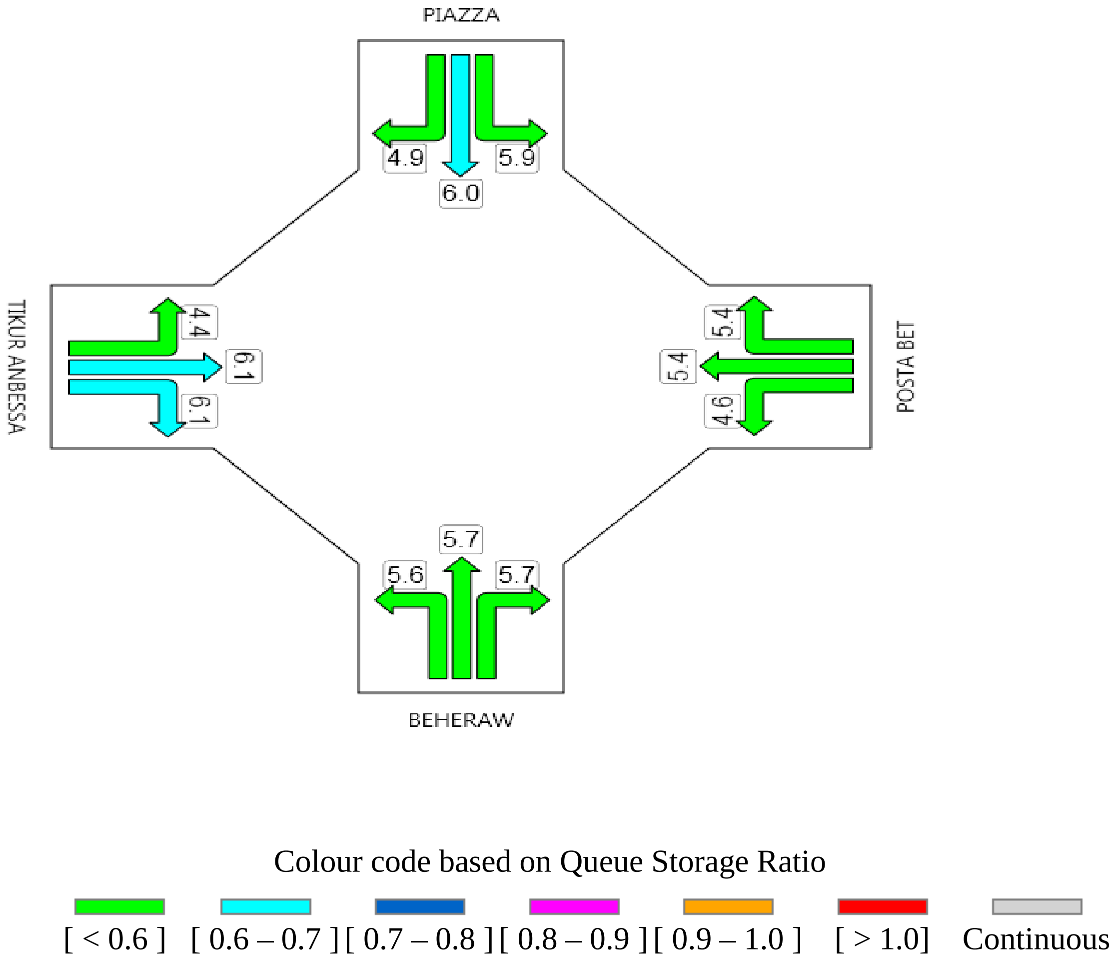

Figure 4.9 Queue adjusted for existing intersection of each lane groups

The numbers in front of the arrows in the above figure 4.9 represents the number of vehicles stopped beyond the stopping lines of each approaches. These numbers are decreased when compared with the non-adjusted queue in the previous figure 4.2. The queue number of the study intersection based on analysis was 10.4 vehicles before adjustment, but after adjustment have made it become reduced to 6.1 vehicles as shown in the following table 3.8. This implies that the intersection flow condition is have less delay. The queue storage ratio which is the ratio of the queue length to the available queue storage distance values become below 0.6 which indicates the flow conditions in each lane groups are stable and small delay. The queue for each approaches are summarized in the following table 4.9.

Table 4.9 Adjusted queue of the existing intersection and approaches

Entering	South	East	North	West	Intersection
Queue (veh.)	5.7	5.4	6.0	6.1	6.1

The table indicates that the queue or the numbers of vehicles waiting for green time to dissipate or discharge from their stopping places to different destinations. The higher the queue value, the more delay experienced relatively and it can be taken as the queue value of the study intersection. From the four approaches, the Eastern or Posta Bet approach has the lowest queue or low numbers of vehicles waiting for green time while the Western or Tikur Anbessa approach has relatively the highest queue value that considered as queue of the signalized intersection.

4.2.4 Revised Degree of Saturation

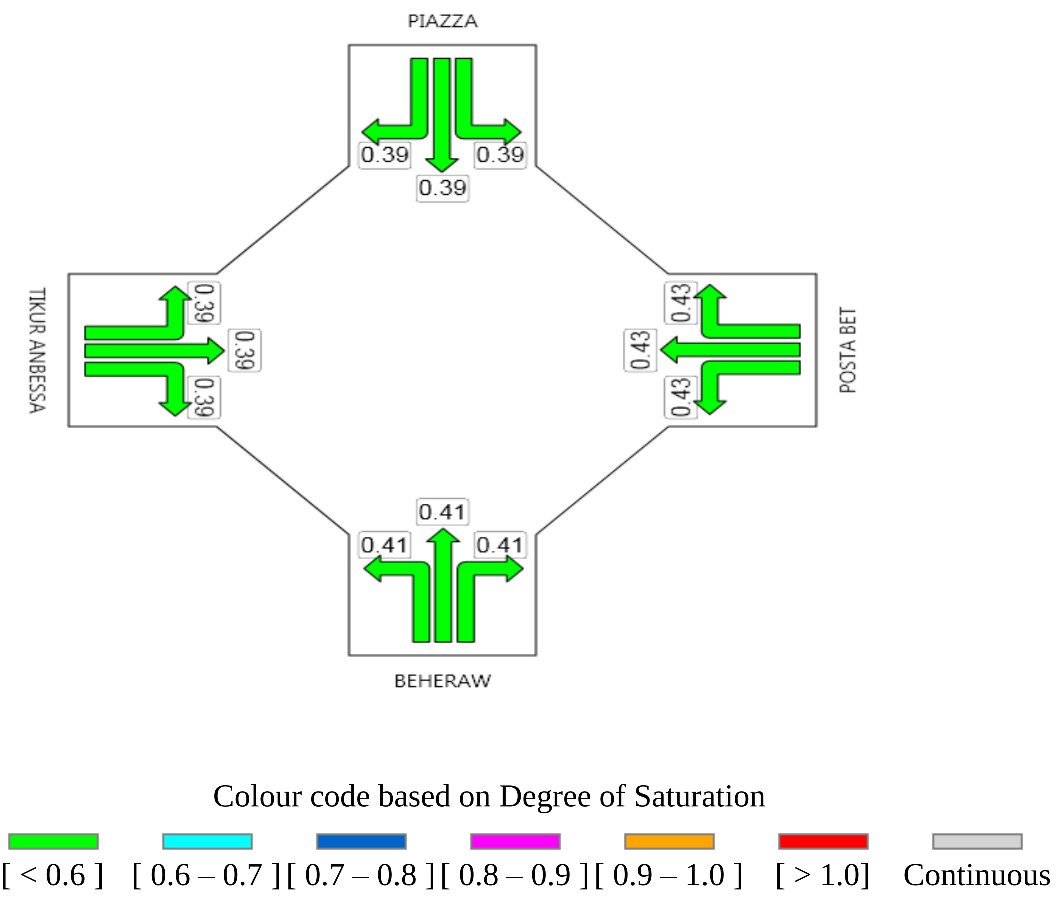

Figure 4.10 Adjusted degree of saturation at Hager Astedader signalized intersection**.**

The values in the above figure 4.10 represents the adjusted volume to capacity ratio of the intersection became small, which implies that capacity of the adjusted existing intersection is much larger than the demand volume of the intersection and their ratio became less than 0.5 as indicated by the color code. Based on Victoria Transport Policy Institute, when the values of volume to capacity ratio is less or equal to 0.5, the intersection is operating under capacity and excessive delays are not experienced. Thus, when the v/c ratio of "non-adjusted" and "adjusted" intersection was compared, the adjusted one had shown better flow condition. The maximum volume to capacity ratio on all approaches will give the degree of saturation of the intersection and summarized in the following table 4.10.

Table 4.10 Revised degree of saturation of the existing intersection and approaches

Entering	South	East	North	West	Intersection
Degree of Saturation	0.41	0.43	0.39	0.39	0.43

As observed in the above table 4.10, degree of saturation for the intersection is in the range of stable traffic conditions implies low or no congestion as per Victoria Transport Policy Institute (VTPI) specified bellow.

V/C Ratio greater than 1.0 = Severe Congestion

V/C Ratio of 0.75 to 1.0 = Heavy Congestion

V/C Ratio of 0.5 to 0.74 = Moderate Congestion

V/C Ratio of less than 0.5 = Low or No Congestion/ less delay.

4.2.5 Revised Level of Service

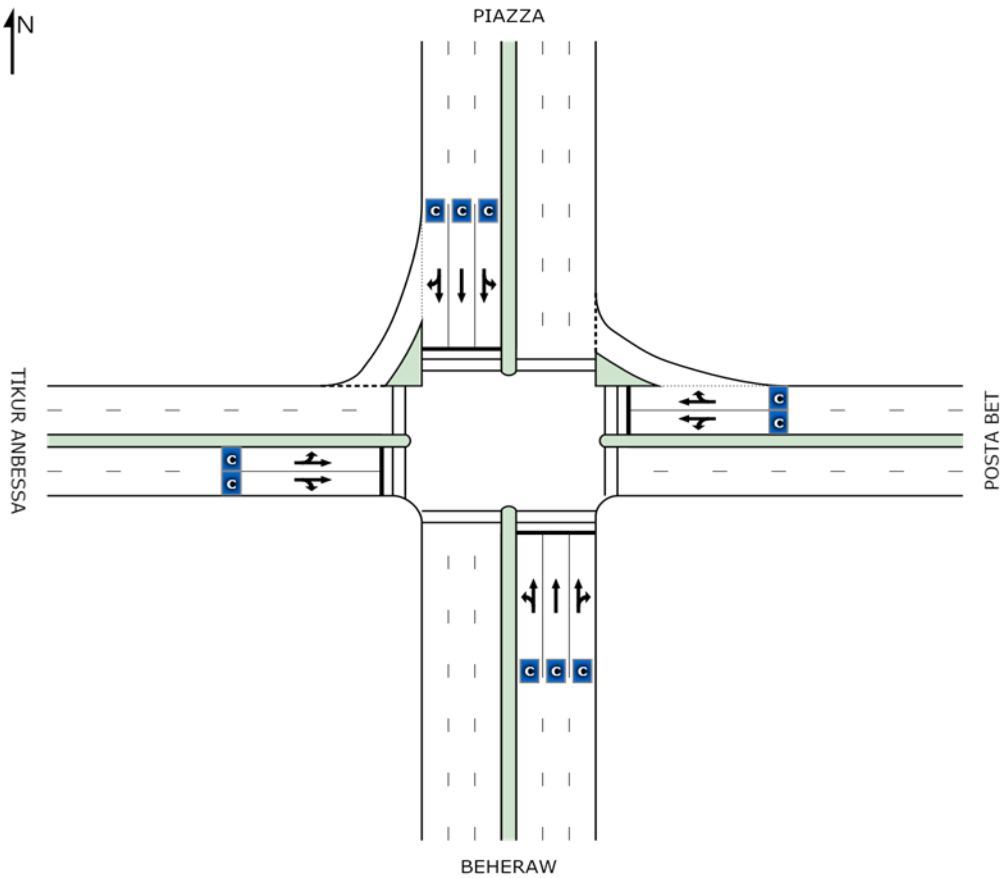

Figure 4.11 Revised Level of service of Hager Astedader signalized intersection

The level of service of the intersection in "do nothing" was E for vehicles, but after adjusting the cycle time and phase number the level of service of the intersection became C as shown in the following table 4.11 which is better than the existing level of service of the intersection.

Table 4.11 Adjusted level of service of the intersection

Entering	South	East	North	West	Intersection
LOS	C	C	C	C	C

The above table 4.11 represents the output of the software that the level of service became improved from E to C after adjustment have made. This implies that the flow condition became fairly stable from near unstable and the intersection have less average delay time rather than considerable delay.

4.2.6 Revised Queue Length

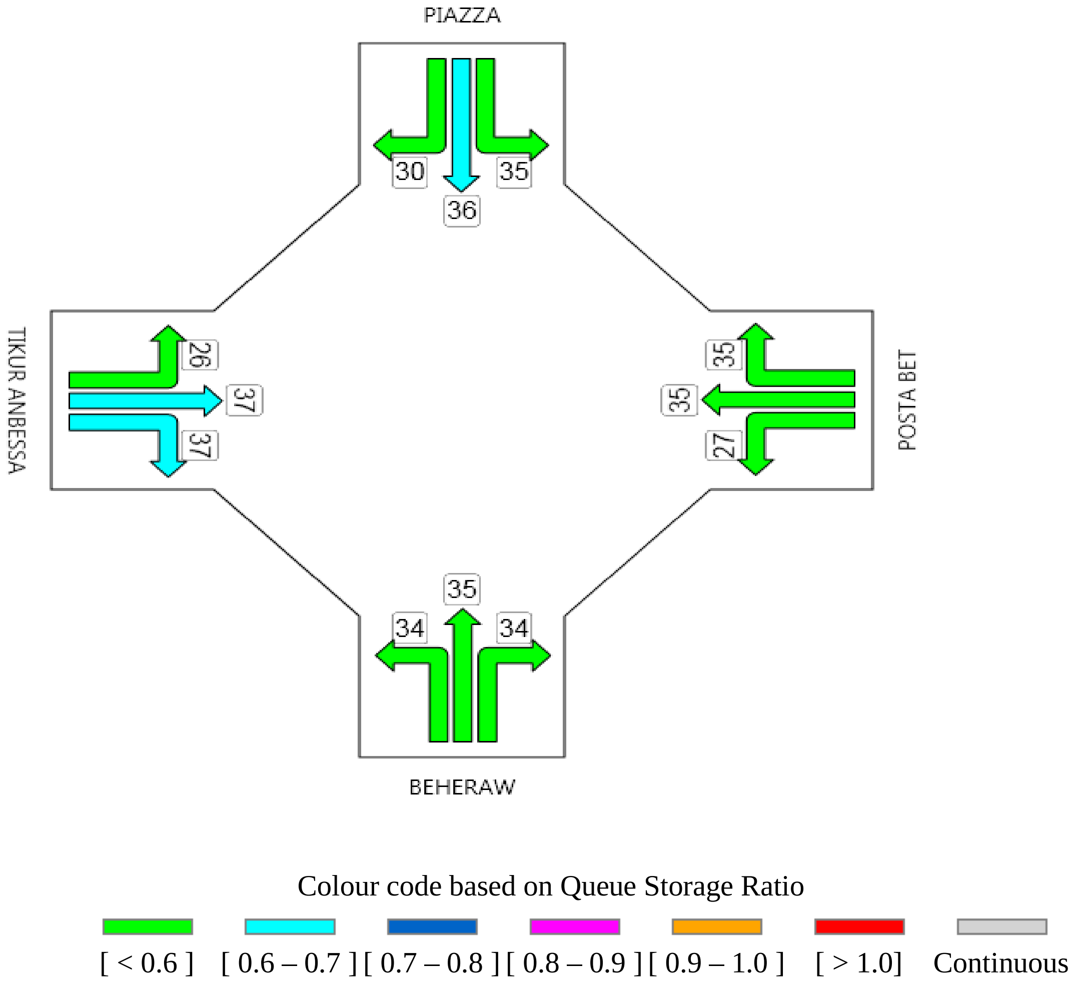

Figure 4.12 Revised Queue length of Hager Astedader signalized intersection

After the intersection was re-analyzed, the approximate possible distance on which the vehicles were queued are shown on the above figure 4.12. The maximum queue length from all approaches represent the queue length of the intersection as summarized in the following table 4.12.

Table 4.12 Queue length of the revised intersection and approaches

Entering	South	East	North	West	Intersection
Queue length(m)	35	35	36	37	37

4.2.7 Revised Delay (Average)

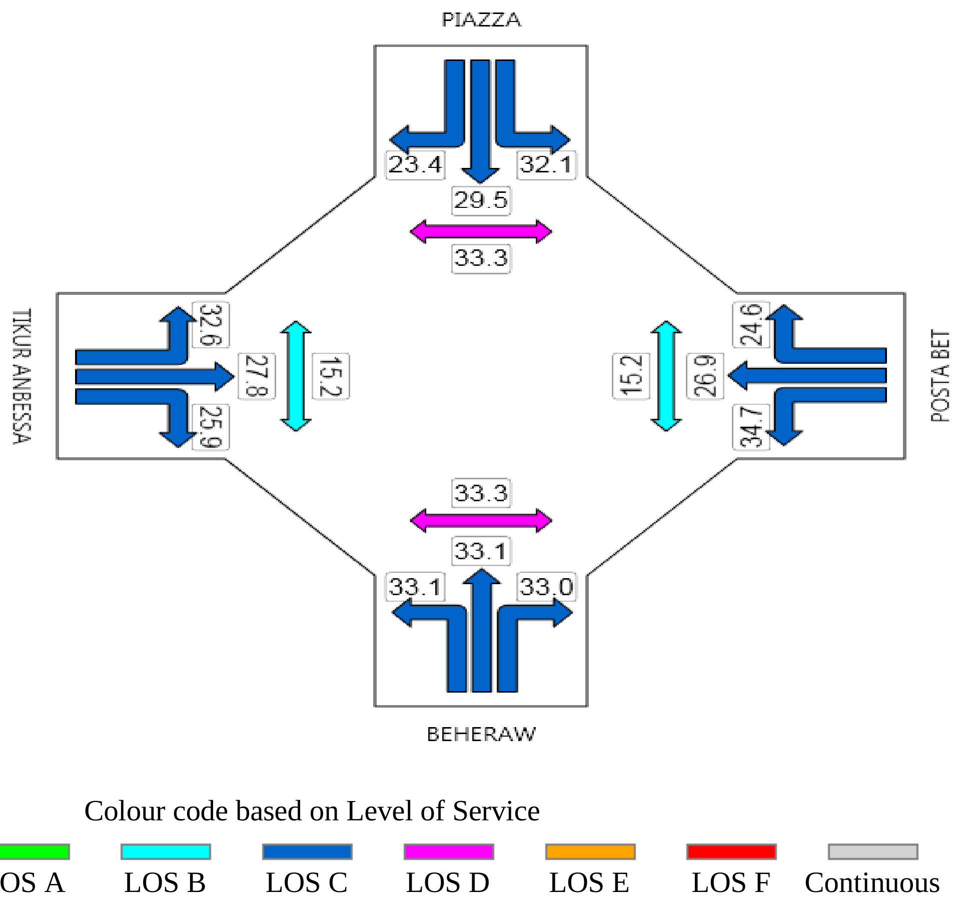

Figure 4.13 Revised average delay per vehicle and average pedestrian delay (seconds).

The average delay experienced at the study signalized intersection became decreased after adjustment of the cycle time and phase number had been made, because of the increase in capacity and the decrease in cycle length of the intersection. This would bring the intersection become uncongested or moderate flow conditions.

In table 4.5, intersection delay was 58.7 second per vehicles in order to cross the stopping lines of the intersection, but after adjustment had been made, it became reduced to 30.1 second per vehicles as observed in the following table 4.13. Not only the decrease in delay for vehicles, but also delay for pedestrians was decreased only by decreasing the cycle time and phase number from the following table 4.13

Table 4.13 Revised average delay per vehicle and per pedestrian for an intersection**.**

	South	East	North	West	Intersection
Delay (sec/veh.)	33.1	28.7	29.1	28.6	30.1
LOS	C	C	C	C	C
Delay(sec/ped)	33.3	15.2	33.3	15.2	24
LOS	D	B	D	B	C

From the above table 4.13, average delay of the intersection for both vehicles and pedestrians fall within the ranges of 20-35seconds per vehicles and 20-30seconds per pedestrians respectively. These ranges represent the level of service C as per the standard level of service criteria of Highway Capacity manual. Thus, the re-analyses of the intersection would have been made better flow condition and average time delay than the “do nothing” at the study intersection.

For pedestrians, the level of service along the South (Beheraw approach) and North (Piazza approach) was LOS D. As recommendation on these approaches, it was better to construct over pass pedestrian crossing in order to minimized conflicts between the pedestrian and vehicles.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Any intersection has to be designed to provide good quality of service as perceived by the users and to avoid conflicting traffic flow movements.

Based on the results of "do nothing" and "Revised parameters" and also field survey that had been performed to observe geometric configuration, traffic flow movements, signal timing and operational performance; the following conclusions are drawn:

- ✓ The "do nothing" or the actual traffic movements of the existing signalized traffic intersection was noted to have long average time delay of 58.7 seconds per vehicles while after adjustment of cycle time and phase number was made, the average time delay of the intersection became reduced to (30.1 seconds per vehicles). Delay in this research work was not due to the intersection geometric problem rather due to long cycle time.
- ✓ The level of service of the intersection in the "do nothing" was found to have LOS E which means, the intersection has been serving near its capacity with low speed, but for the revised it became LOS C which implies relatively stable flow condition at the intersection.
- ✓ The traffic flow conditions of the intersection in the "do nothing" was moderately congested with V/C ratio of 0.67 while in the "Revised" it became low or no congestion because of the degree of saturation (V/C ratio) of the intersection in the "Revised" became less than 0.5, which indicates that arrival and departure volumes are the same.
- ✓ Delay and level of service are an indication of the potential capacity and performance measure of an intersection.
- ✓ Based on this research work, the main contributory factors for moderate traffic congestion at study area were the allocation of long cycle time and four phase numbers at the intersection.
- ✓ Finally, the possible measures would be under taken in reducing average time delay at the study area by checking the allocation of signal cycle time and phase numbers by concerning bodies.

5.2 Recommendations

Based on the field survey, SIDRA output for “do nothing” and “revised signal parameters”; hereunder are the recommendations in order to improve traffic flow conditions at Hager Astedader signalized intersection.

1. For Addis Ababa City Transportation Agency, would be taken a possible treatments in decreasing such types of traffic congestion by checking the allocation of the cycle length time and the number of signal phases in order to reduce the time waiting periods of both motorists and passengers.
2. Traffic regulations principles around the intersection area that may include: restriction for heavy vehicles concerning their weight and length to pass the intersection, restricting parking lot placement along the roadway and limiting direct access to the intersection which can have a positive impact on both traffic operation and safety.
3. Additionally; the researcher identified treatments that are of little or high cost and beneficial to improving congestion reduction and increase operational performance of the intersection. Such treatments may relate to check the time of cycle length and phase number, reapplying the faded pavement markings especially on west-east direction, realignment of the left turn lanes and relocating or adding new traffic signs.
4. Validation of the formula on equation developed by the Author concerning saturation flow rate for future studies relative to the same problems encountered in study area due to some limitations.
5. Finally, the researcher proposed that a sequence of three phase numbers with a cycle time of 96 seconds rather than the existing four phasing and cycle time of 160 seconds to improve the quality service from level of service E to level of service C for the study signalized intersection.

REFERENCES

[1] Joel Marcuson P.E., 2010. Highway Capacity: Manual. Jacobs Engineering. U.S.A.

[2] Andrew W. and Jean-Marc M., 2009. Road Transport: Thematic Research Summary. European Commission DG Energy and Transport.

[3] Yared Haregewoin, 2010. Impact of Vehicle Traffic Congestion in Addis Ababa: (The case of Kolfe sub city).Thesis, Civil Service university.

[4] World Bank, 2002. Cities on the Move: A Worlds Bank Urban Transport Strategy. Review. Washington D.C.

[5] Oluwoye J.O., 2000. An Investigation of the Factors of the Design of Roads. That Affect Planning For Traffic (Pedestrian) Safety in the Third World. South African Transport Conference. South Africa.

[6] Cameron J.W.M., & Krynauw M.N., 2003. Urban Activities, Movement and Parking: A Time for Hard Decisions. Proceedings of the 22nd Southern African Transport Conference (SATC2003). No.0-958 – 460965. TRC Africa (Pty) Ltd. Pretoria-South Africa

[7] Litman T., 2003 Mobility Management: A Sourcebook for Policy-makers in Developing Cities. GTZ Transport and Mobility Group.

[8] Ogundipe O.M, 2007. Appropriate Traffic Congestion Mitigation for an Unplanned Urban City. Department of Civil Engineering. University of Ado-Ekiti, Nigeria.

[9] Malenkovska M. et al., 2009. Transport Problems. vol. 4. Issue 3.Part 1. Ivo Lola Ribar Bitola: Republic of Macedonia.

[10] Zhang N., & Yang X., 2005. The Marginal Decrease of Lane Capacity with the Number of Lanes on Highway. Proceedings of the Eastern Asia Society for Transportation Studies.

[11] Orn H., 2002. Urban Traffic and Transport. Housing development Management Building issues 2. vol. 12. Lund University, Sweden.

[12] Addis Ababa City Road Authority, 2010. Road network coverage of Addis Ababa and Traffic Study Report. Addis Ababa, Ethiopia.

[13] European Conference of Ministers of Transport, 2007. Managing Urban Traffic Congestion: Summary Document. Transport Research Center.

[14] Victoria Transport Policy Institute, 2000. Congestion Reduction Strategies: Identifying and Evaluating Strategies to Reduce Traffic Congestion.

[15] Loop H.V., & Perdok J., 2009 "Ex-Post Evaluation of Policy Measures on Traffic Congestion". Ministry of Transport. The Netherlands.

[16] Mathew V., and Krishna K.V., 2007. *"*Introduction to Transportation Engineering*"*.

[17] A Policy on Geometric Design of Highways and Streets. AASHTO, Washington, D.C, 2001.

[18] John Ringert, et.al. FHWA, 2004. Signalized intersections: informational guide.

[19] Addis Ababa City Roads Authority design manual, 2003; volume 11.

[20] Nicholas J. et al., 2009. Traffic and highway engineering. Fourth edition; University of Virginia

[21] Traffic Signal links from Darcy Bullock, Purdue University; http://ce.ecn.purdue.edu/~darcy/signalequipment.htm viewed 20/9/2015.

[22] Addis Ababa City Roads Authority draft traffic report manual, October 2009; page 28.

[23] Canadian Capacity Guide for Signalized Intersections, 2010, Sec. Ed. published by the Institute for Transportation Engineers (District 7 - Canada)

[24] Highway Performance Monitoring Systems (HPMS) Field Manual. U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., April 2007.

[25] G. F. Qu, et al., 2008, "Determine Traffic Signal Timing Plan with Particle Swarm Optimization," Journal of Computational Information Systems, 4: 2809-2816

[26] D. Mariagrazia, et al., 2006 "A Signal Timing Plan Formulation for Urban Traffic Control," Control Engineering Practice, 14:1297-1311.

[27] M. Nazmi, et al., 2005 "Simulation and Evaluation for Solving the Traffic Flows Problem at Four Intersection in Malaysia," Proceedings for the 24th JSST Conference, Chester.

[28] Chitturi, M.V. and Benekohal, R.F., 2008."Passenger Car Equivalents for heavy vehicles in work zones" *87th TRB Annual Meeting and Publication* in TRR.

[29] Webster, N., and Elefteriadou, L., 1999. A simulation study of truck passenger car Equivalents (PCE) on basic freeway sections. *Journal of Transportation Research, Part B.*

[30] Dey, P. P., Chandra, and Gangopadhyay, S. 2008. “Simulation of Mixed Traffic Flow on Two-Lane Roads”, *Journal of Transportation Engineering*, ASCE, 134(9), 361-369

[31] Al.Kaisy, A. F., et al., 2005. “Developing Passenger Car Equivalency Factors for Heavy Vehicles during Congestion”. *Journal of Transportation Engineering*, ASCE, 131:7, 514-523.

[32] J.W.Zwarteveen.2011,’’ Urban Traffic Flow Modeling in Addis Ababa’’.

[33] Chandra, S., and Sikdar, P. K. 2000. “Factors affecting PCU in mixed traffic situations on urban roads.” Road Transp. Res., 9:3, 40–50.

[34] Benekohal, R. F., & Zhao, W. 2000. Delay-based passenger car equivalents for trucks at Signalized intersections. Transportation Research Part A: Policy and Practice, 34:6, 437- 457.

[35] Al-Kaisy, A., Jung, Y., & Rakha, H. 2005. Developing passenger car equivalency factors for heavy vehicles during congestion. Journal of transportation engineering, 131:7, 514-523.

[36] Sharma, H. K., Swami, M. 2012. “Effect of Turning Lane at Busy Signalized At-Grade Intersection under Mixed Traffic in India”. *European Transport/ Transport European*, 52, 1–14.

[37] Manual of Traffic Signal Design, 2008; 2nd Edition, published by the Institute of Transportation Engineers.

[38] A Policy on Geometric Design of Highways and Streets. AASHTO, Washington, D.C, 2001

[39] Tom V. Mathew, 2014; Signalized Intersection Delay Models. NPTEL

[40] R Akcelik. 1991. The HCM; delay formulas for signalized intersection-ite journal,

[41] Dion, F. *et al.*, 2004. Comparison of Delay Estimates at Under-Saturated and Over-Saturated Pre-Timed Signalized Intersections, Transportation Research Part B.

[42] Federal Highway Administration by Kittelson and Associates. Traffic Signal Timing Manual**,** Publication Number: FHWA-HOP-08-024**.** 2008, USA.

APPENDIXES

Appendix-1 Counted average traffic raw data

Name of Junction = Hager Astedader

Dates **20 -- 23/7/2015**

Time **7:30-7:45 am**

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**PCU**
Piazza	24	3	2	125	13	2	11	1	0	196
Posta Bet	13	1	0	44	6	1	6	1	0	78
Beheraw	26	3	1	116	10	5	13	1	0	189
T. Anbesssa	14	1	1	63	5	1	14	2	1	108
										571

Time **7:45-8:00 am**

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**PCU**
Piazza	27	3	2	126	11	3	11	2	0	203
Posta Bet	16	1	0	51	7	1	7	1	0	91
Beheraw	29	3	2	117	11	3	13	0	0	193
T. Anbesssa	19	1	1	60	6	3	9	3	1	113
										600

Time **8:00-8:15 am**

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**PCU**
Piazza	27	3	1	125	13	3	13	1	0	202
Posta Bet	13	1	1	68	8	2	6	0	0	108
Beheraw	25	2	2	119	13	2	18	2	0	197
T. Anbesssa	16	1	0	65	8	2	21	0	0	119
										626

Time **8:15-8:30 am**

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**P.car**	**Buses**	**Tracks**	**PCU**
Piazza	24	3	1	130	12	3	14	2	0	204
Posta Bet	16	1	0	82	4	1	7	1	0	115
Beheraw	27	3	0	122	14	3	18	1	0	201
T. Anbesssa	16	1	1	79	8	2	18	2	0	135
										656

Dates 20 -- 23/7/2015
Time 8:30-8:45 am

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	29	2	1	130	14	4	13	2	0	212
Posta Bet	14	1	1	75	6	1	8	1	0	112
Beheraw	27	2	2	117	11	4	17	2	1	200
T. Anbesssa	15	1	1	77	3	1	19	2	0	122
										646

Time 8:45-9:00 am

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	32	2	1	124	14	4	14	2	0	207
Posta Bet	15	1	0	77	4	1	8	1	0	111
Beheraw	28	4	0	118	14	5	15	1	0	200
T. Anbesssa	15	1	1	77	5	3	17	1	0	129
										647

Time 9:00-9:15 am

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	31	3	1	124	14	3	15	2	0	207
Posta Bet	13	0	1	79	8	1	7	1	0	116
Beheraw	27	3	2	118	12	3	19	3	1	203
T. Anbesssa	14	1	1	70	5	1	16	3	0	119
										645

Time 9:15-9:30 am

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	30	3	2	122	12	5	12	1	0	206
Posta Bet	15	0	0	72	5	1	6	0	0	102
Beheraw	28	3	1	118	12	5	19	1	1	203
T. Anbesssa	17	1	1	70	2	1	22	2	0	120
										631

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Dates 20 -- 23/7/2015

Time 5:30-5:45pm

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	28	3	2	126	13	3	13	2	0	208
Posta Bet	17	1	1	71	3	0	7	0	0	102
Beheraw	24	2	1	117	14	4	17	1	0	197
T. Anbesssa	14	1	2	82	5	1	22	2	0	135
										642

Time 5:45-6:00 pm

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	27	2	0	126	13	4	11	2	0	197
Posta Bet	17	0	0	65	4	1	7	1	0	97
Beheraw	28	3	0	117	14	5	26	3	0	214
T. Anbesssa	17	1	1	76	4	1	23	1	0	130
										639

Time 6:00-6:15 pm

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	29	3	0	129	12	3	12	1	0	201
Posta Bet	17	1	0	68	3	0	7	0	0	98
Beheraw	29	3	1	117	11	4	21	1	1	200
T. Anbesssa	17	1	2	73	7	1	24	1	0	134
										633

Time 6:15-6:30 pm

Leg Name	Left traffic			Through Traffic			Right Traffic			TOTAL
	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	25	2	0	127	13	2	15	2	0	198
Posta Bet	17	1	0	47	1	0	7	1	0	75
Beheraw	28	3	0	117	13	5	23	2	1	210
T. Anbesssa	16	1	0	69	7	2	20	2	0	123
										607

Dates 20 -- 23/7/2015
Time 6:30-6:45 pm

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	30	3	1	122	10	2	12	2	0	192
Posta Bet	14	1	0	66	3	1	7	1	0	94
Beheraw	25	3	1	116	13	5	27	2	1	214
T. Anbesssa	15	1	0	56	6	2	18	1	0	106
										605

Time 6:45-7:00 pm

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	28	2	0	126	13	1	11	1	0	193
Posta Bet	17	1	0	54	7	0	6	1	0	90
Beheraw	29	4	1	116	12	4	24	1	0	209
T. Anbesssa	15	1	1	73	6	1	25	1	0	130
										621

Time 7:00-7:15 pm

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	27	1	0	125	7	1	12	2	0	183
Posta Bet	15	1	0	53	3	0	7	1	0	83
Beheraw	24	2	1	110	9	3	28	2	1	192
T. Anbesssa	16	1	1	64	3	1	21	1	0	114
										572

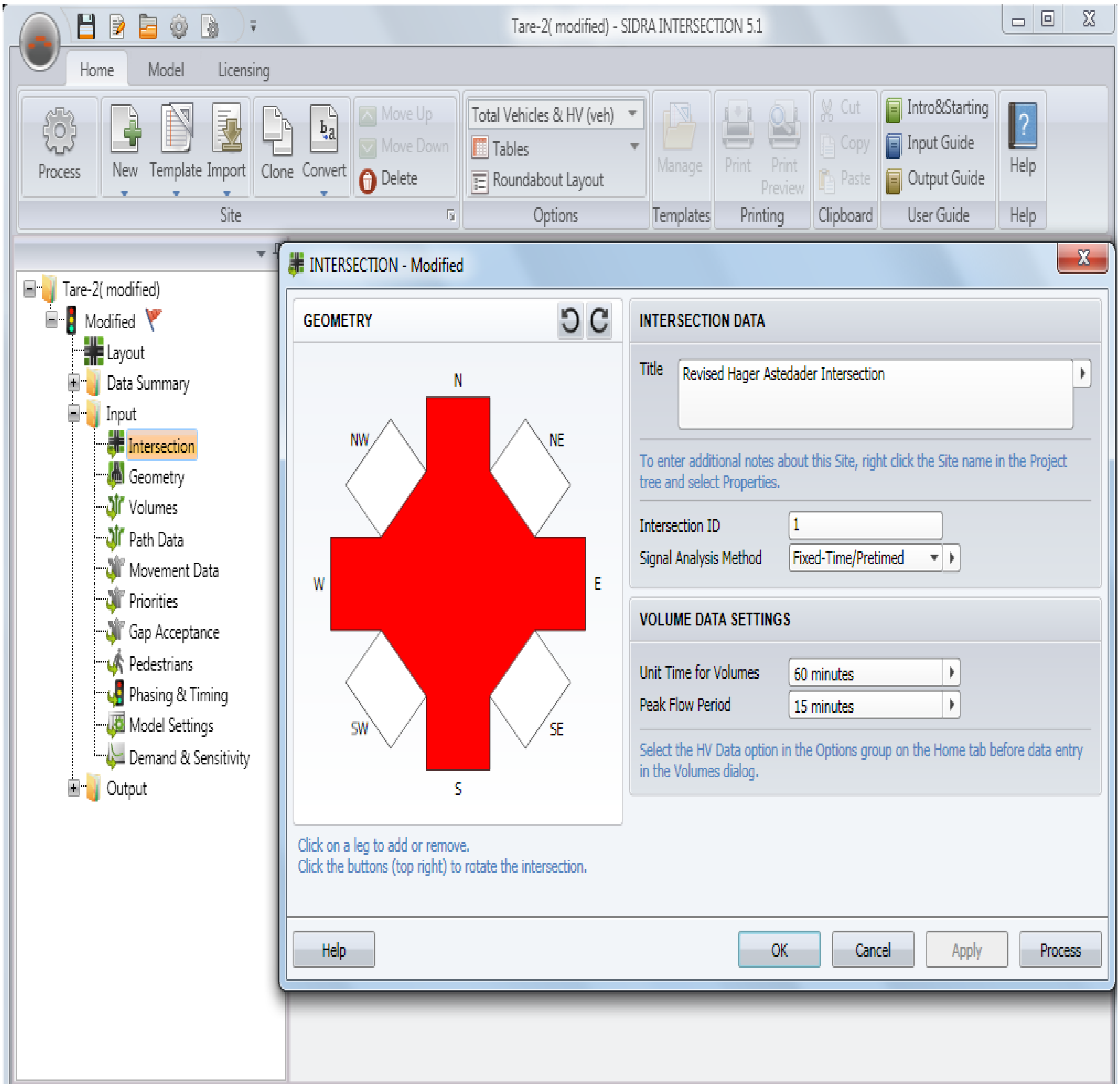
Time 7:15-7:30 pm

	Left traffic			Through Traffic			Right Traffic			TOTAL
Leg Name	P.car	Buses	Tracks	P.car	Buses	Tracks	P.car	Buses	Tracks	PCU
Piazza	29	2	1	121	8	1	10	2	0	183
Posta Bet	15	1	0	51	3	0	5	1	0	78
Beheraw	27	2	1	112	8	3	23	2	0	190
T. Anbesssa	12	1	0	58	4	1	17	1	0	98
										549

Appendix-2 Four day's average pedestrian counted raw data

Time	Piazza	Posta Bet	Biheraw	T. Anbesssa
7:30-7:45am	32	25	65	71
7:45-8:00am	50	38	92	83
8:00-8:15am	47	42	80	87
8:15-8:30am	41	51	102	92
sum	**170**	**156**	**339**	**333**
8:30-8:45am	72	58	94	99
8:45-9:00am	93	55	86	102
9:00-9:15am	60	61	73	95
9:15-9:30am	52	46	82	73
sum	**277**	**220**	**335**	**369**
5:30-5:45pm	68	37	61	80
5:45-6:00pm	63	42	70	70
6:00-6:15pm	59	58	63	62
6:15-6:30pm	62	46	84	76
sum	**252**	**183**	**278**	**288**
6:30-6:45pm	52	41	56	74
6:45-7:00pm	49	50	40	65
7:00-7:15pm	46	36	42	72
7:15-7:30pm	28	23	35	54
sum	**174**	**150**	**173**	**265**
aveg.ped/h	218	177	282	314

Appendix-3 Representation of Software dialog



Appendix -4 Movement summary for “do nothing” analysis

HAGER ASTEDADER INTERSECTION ANALYSIS
Signals - Fixed Time Cycle Time = 160 seconds (Practical Cycle Time)
Design Life Analysis (Practical Capacity): Results for 0 years

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue		Prop. Queued	Effective Stop Rate	Average Speed
							Vehicles	Distance			
		veh/h	%	v/c			veh	m		per veh	km/h
South: BEHERAW											
1	L	77	13.5	0.421	53.5	LOS D	9.2	55.8	0.88	0.79	7.9
2	T	297	17.5	0.421	53.4	LOS D	9.4	58.2	0.88	0.73	9.9
3	R	53	7.8	0.421	53.4	LOS D	9.4	55.9	0.88	0.81	7.8
Approach		426	15.6	0.421	53.4	LOS D	9.4	58.2	0.88	0.75	9.3
East: POSTA BET											
4	L	72	8.7	0.674	76.2	LOS E	10.4	62.8	0.98	0.85	6.0
5	T	182	21.8	0.674	71.5	LOS E	10.4	64.5	0.98	0.85	8.0
6	R	23	22.7	0.674	68.8	LOS E	9.9	64.5	0.98	0.87	6.6
Approach		278	18.5	0.674	72.5	LOS E	10.4	64.5	0.98	0.85	7.3
North: PIAZZA											
7	L	84	9.9	0.397	52.4	LOS D	9.8	57.3	0.88	0.79	8.0
8	T	313	15.2	0.397	49.2	LOS D	9.9	60.1	0.87	0.73	10.6
9	R	61	20.3	0.397	41.7	LOS D	8.6	53.4	0.87	0.81	9.5
Approach		458	14.9	0.397	48.8	LOS D	9.9	60.1	0.87	0.75	10.0
West: TIKUR ANBESSA											
10	L	57	14.5	0.593	69.3	LOS E	10.2	60.9	0.96	0.82	6.5
11	T	187	13.9	0.593	69.3	LOS E	10.2	61.8	0.96	0.80	8.1
12	R	36	20.0	0.593	69.3	LOS E	10.2	61.8	0.96	0.82	6.4
Approach		280	14.8	0.593	69.3	LOS E	10.2	61.8	0.96	0.80	7.6
All Vehicles		1442	15.8	0.674	58.7	LOS E	10.4	64.5	0.91	0.78	8.7

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement

LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).

Appendix-5 Pedestrian movement performance.

Movement Performance - Pedestrians								
Mov ID	Description	Demand Flow	Average Delay	Level of Service	Average Back of Queue		Prop. Queued	Effective Stop Rate
					Pedestrian	Distance		
		ped/h	sec		ped	m		per ped
P3	Across S approach	342	30.0	LOS D	1.0	1.0	0.61	0.61
P2	Across E approach	94	19.5	LOS B	0.2	0.2	0.49	0.49
P1	Across N approach	245	30.0	LOS D	0.7	0.7	0.61	0.61
P4	Across W approach	368	19.5	LOS B	0.8	0.8	0.49	0.49
All Pedestrians		1049	25.4	LOS C			0.56	0.56

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)

Pedestrian movement LOS values are based on average delay per pedestrian movement.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

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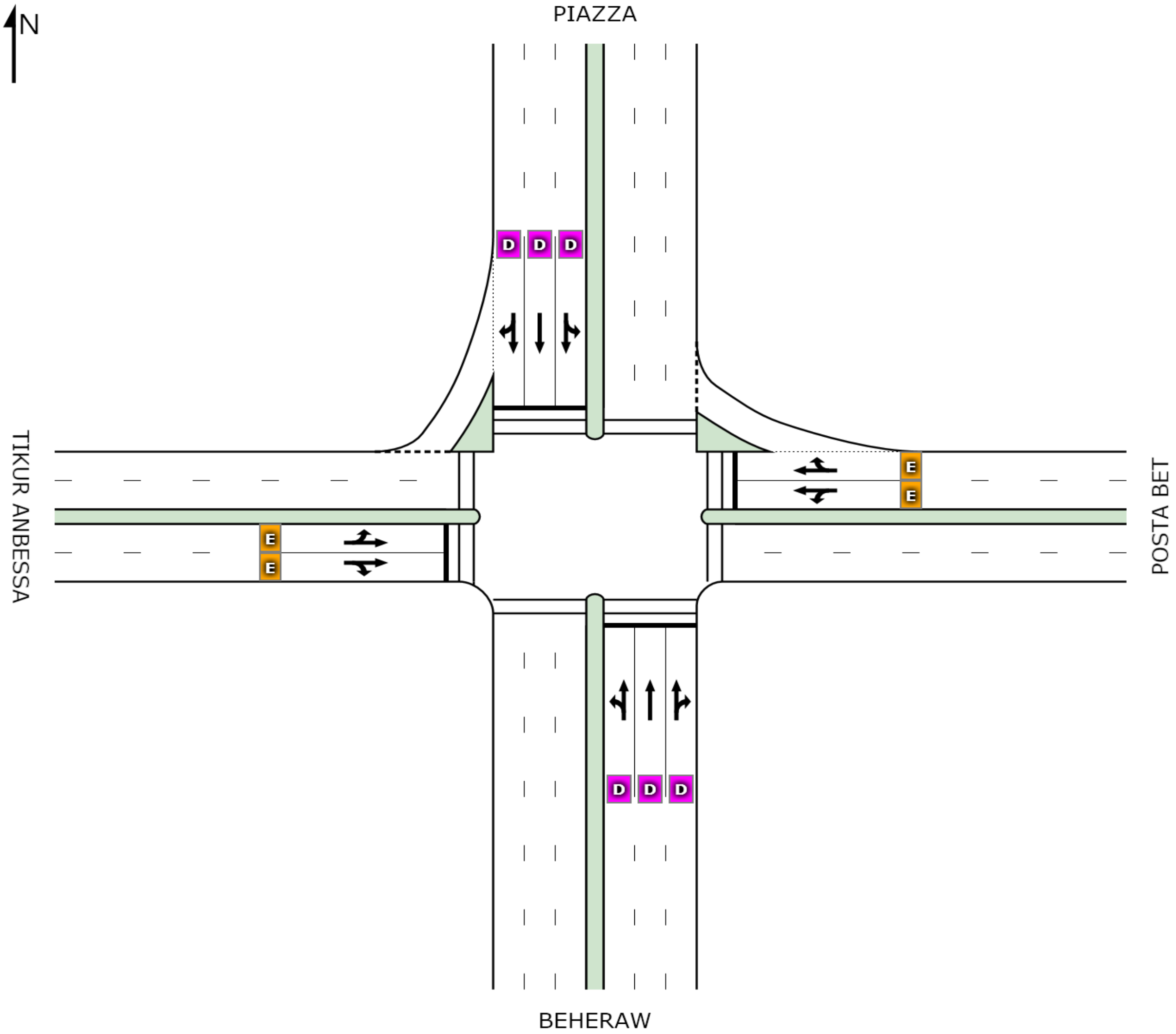
Appendix -6 Lane summary for “do nothing” analysis

Lane Use and Performance

	Demand Flows				HV	Cap.	Deg. Satn	Lane Util.	Average Delay	Level of Service	95% Back of Queue		Lane Length	SL Type	Cap. Adj.	Prob. Block.
	L	T	R	Total							Vehicles	Distance				
	veh/h	veh/h	veh/h	veh/h	%	veh/h	v/c	%	sec		veh	m	m		%	%
South: BEHERAW																
Lane 1	77	64	0	140	15.3	333	0.421	100	53.5	LOS D	9.2	55.8	60	–	0.0	0.0
Lane 2	0	143	0	143	17.5	339	0.421	100	53.4	LOS D	9.3	58.2	60	–	0.0	2.2
Lane 3	0	90	53	143	13.9	340	0.421	100	53.4	LOS D	9.4	55.9	60	–	0.0	0.0
Approach	77	297	53	426	15.6		0.421		53.4	LOS D	9.4	58.2				
East: POSTA BET																
Lane 1	72	65	0	138	14.9	204	0.674	100	76.2	LOS E	10.4	62.8	60	–	0.0	9.1
Lane 2	0	117	23	140	22.0	208	0.674	100	68.8	LOS E	9.9	64.5	60	–	0.0	11.6
Approach	72	182	23	278	18.5		0.674		72.5	LOS E	10.4	64.5				
North: PIAZZA																
Lane 1	84	67	0	151	12.3	380	0.397	100	52.4	LOS D	9.8	57.3	60	–	0.0	0.8
Lane 2	0	153	0	153	15.2	385	0.397	100	52.3	LOS D	9.9	60.1	60	–	0.0	5.1
Lane 3	0	93	61	154	17.3	388	0.397	100	41.7	LOS D	8.6	53.4	60	–	0.0	0.0
Approach	84	313	61	458	14.9		0.397		48.8	LOS D	9.9	60.1				
West: TIKUR ANBESSA																
Lane 1	57	83	0	140	14.2	236	0.593	100	69.3	LOS E	10.2	60.9	60	–	0.0	6.4
Lane 2	0	104	36	140	15.5	236	0.593	100	69.3	LOS E	10.2	61.8	60	–	0.0	7.7
Approach	57	187	36	280	14.8		0.593		69.3	LOS E	10.2	61.8				
Intersection				1442	15.8		0.674		58.7	LOS E	10.4	64.5				

Appendix -7 Level of Service Summary



Entering	South	East	North	West	Intersection
LOS	D	E	D	E	E

Level of Service (LOS) Method: Delay & v/c (HCM 2010). g
Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.
LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).

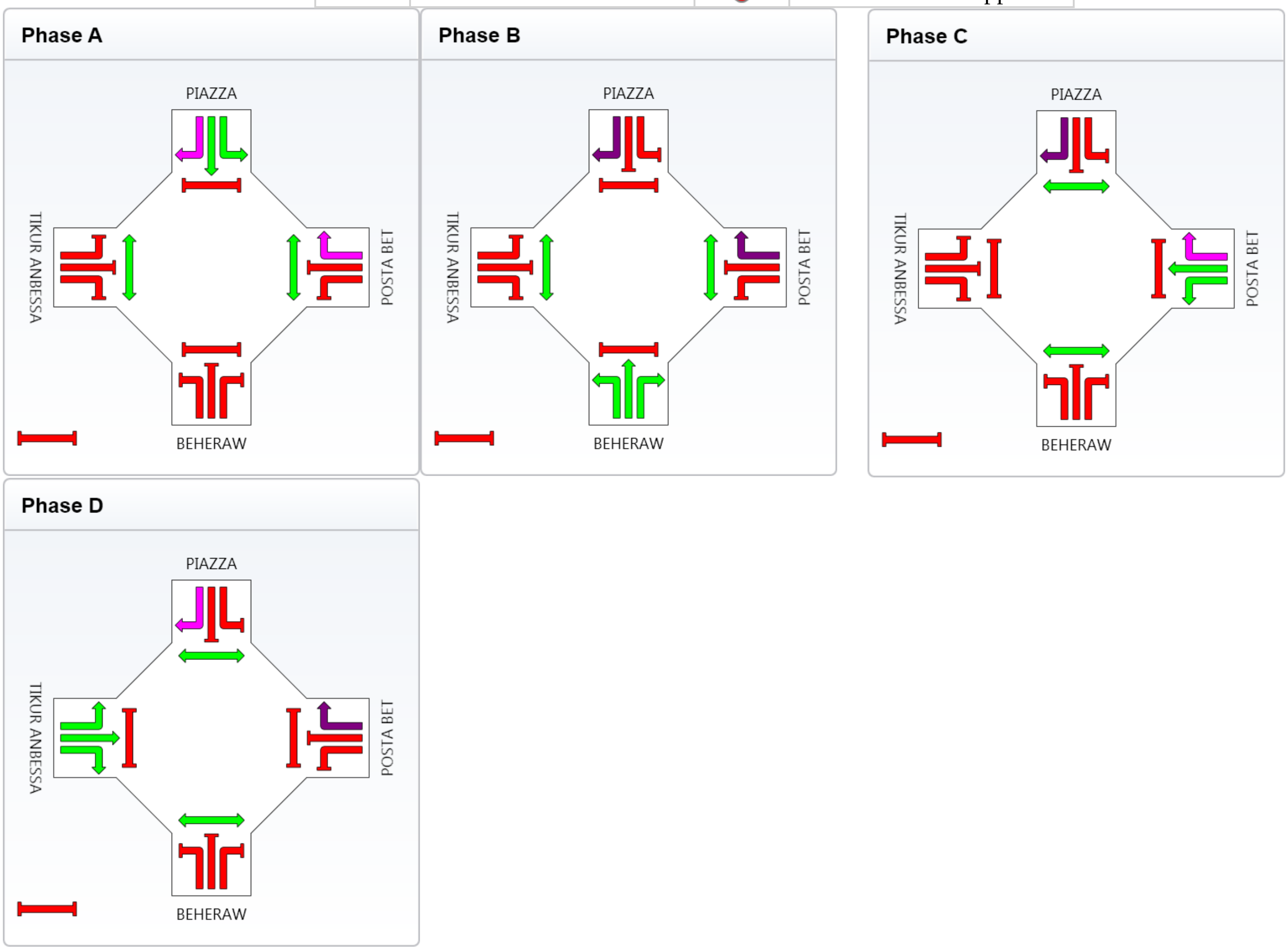
Appendix- 8 Phase summary for do nothing analysis

HAGER ASTEDADER INTERSECTION ANALYSIS
Signals - Fixed Time Cycle Time = 160 seconds (Practical Cycle Time)
Design Life Analysis (Practical Capacity): Results for 0 years

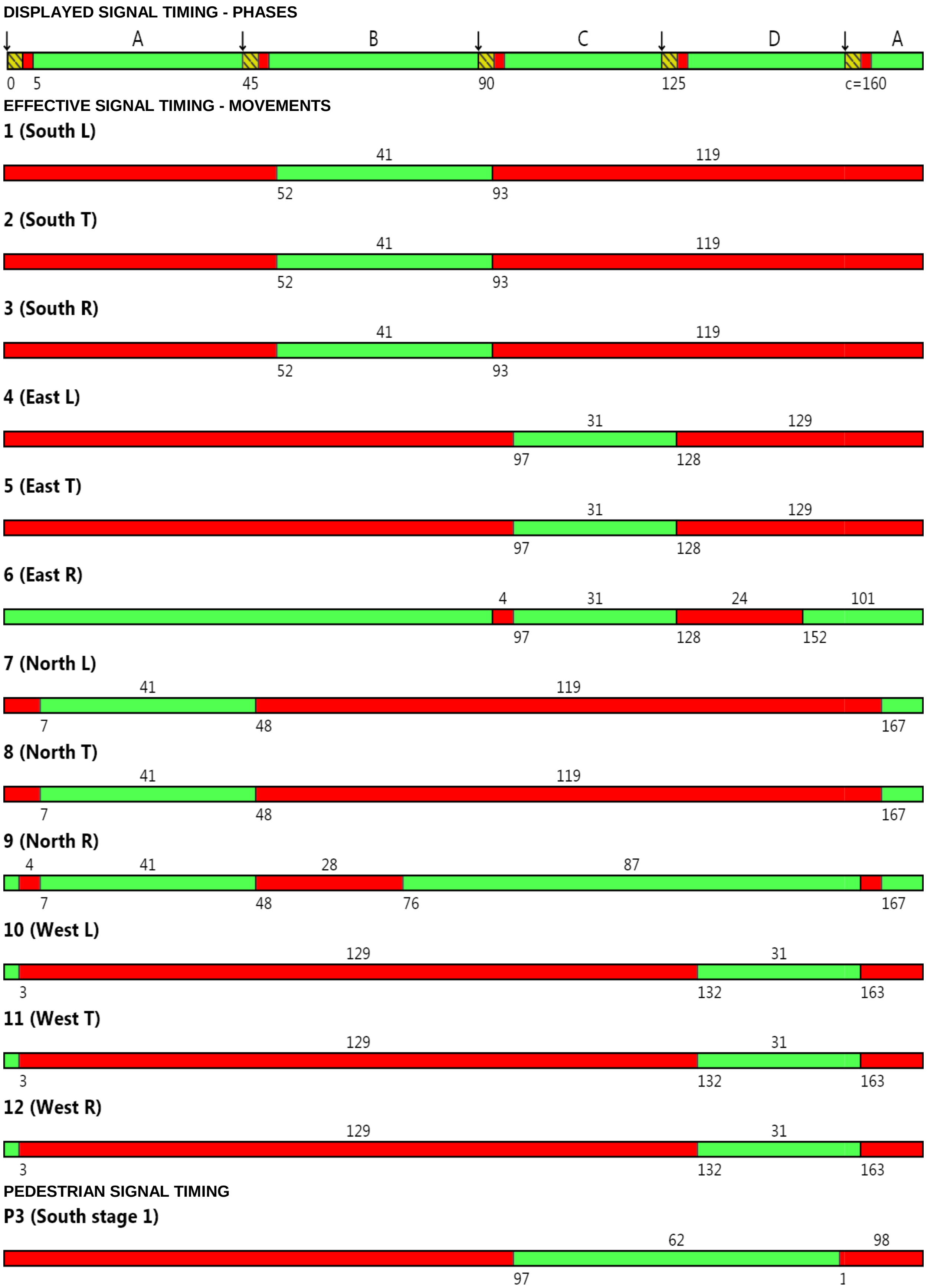
Phase times determined by the program
Sequence: 4- Phasing
Input Sequence: A, B, C, D
Output Sequence: A, B, C, D

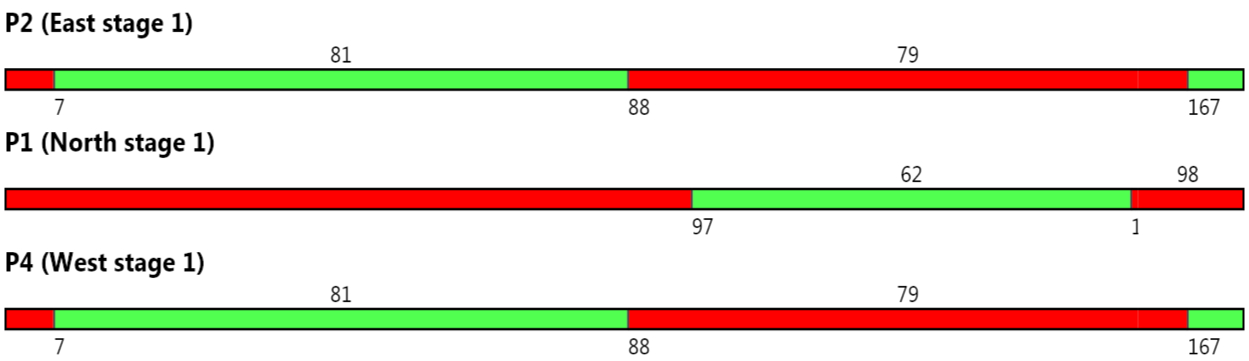
Phase	**A**	**B**	**C**	**D**
Green Time (sec)	40	40	30	30
Yellow Time (sec)	3	3	3	3
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	45	45	35	35
Phase Split	28 %	28 %	22 %	22 %

	Normal Movement		Permitted/Opposed
	Slip-Lane Movement		Opposed Slip-Lane
	Stopped Movement		Continuous Movement
	Turn On Red		Undetected Movement
			Phase Transition Applied



Appendix-9 Movement timing summary for do nothing





Appendix-10 Movement summary for revised parameters

Movement Performance - Vehicles											
Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue		Prop. Queued	Effective Stop Rate	Average Speed
							Vehicles	Distance			
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: BEHERAW											
1	L	77	13.5	0.414	33.1	LOS C	5.6	34.0	0.88	0.78	11.1
2	T	297	17.5	0.414	33.1	LOS C	5.7	35.4	0.88	0.72	13.7
3	R	53	7.8	0.414	33.0	LOS C	5.7	34.1	0.88	0.80	10.9
Approach		426	15.6	0.414	33.1	LOS C	5.7	35.4	0.88	0.74	12.9
East: POSTA BET											
4	L	72	8.7	0.431	34.7	LOS C	4.6	27.1	0.88	0.78	10.7
5	T	182	21.8	0.431	26.9	LOS C	5.4	35.5	0.81	0.73	15.6
6	R	23	22.7	0.431	24.6	LOS C	5.4	35.5	0.79	0.84	13.5
Approach		278	18.5	0.431	28.7	LOS C	5.4	35.5	0.83	0.75	14.1
North: PIAZZA											
7	L	84	9.9	0.388	32.1	LOS C	5.9	34.6	0.87	0.78	11.3
8	T	313	15.2	0.388	29.5	LOS C	6.0	36.3	0.86	0.72	14.7
9	R	61	20.3	0.388	23.4	LOS C	4.9	30.3	0.85	0.80	13.8
Approach		458	14.9	0.388	29.1	LOS C	6.0	36.3	0.86	0.74	14.0
West: TIKUR ANBESSA											
10	L	57	14.5	0.392	32.6	LOS C	4.4	26.2	0.86	0.78	11.2
11	T	187	13.9	0.392	27.8	LOS C	6.1	36.9	0.81	0.68	15.2
12	R	36	20.0	0.392	25.9	LOS C	6.1	36.9	0.80	0.80	12.8
Approach		280	14.8	0.392	28.6	LOS C	6.1	36.9	0.82	0.71	14.1
All Vehicles		1442	15.8	0.431	30.1	LOS C	6.1	36.9	0.85	0.74	13.7

Appendix-11 Lane summary for the revised

Lane Use and Performance

	Demand Flows				HV	Cap.	Deg. Satn	Lane Util.	Average Delay	Level of Service	95% Back of Queue		Lane Length	SL Type	Cap. Adj.	Prob. Block.
	L	T	R	Total							Vehicles	Distance				
	veh/h	veh/h	veh/h	veh/h	%	veh/h	v/c	%	sec		veh	m	m		%	%
South: BEHERAW																
Lane 1	77	64	0	140	15.3	339	0.414	100	33.1	LOS C	5.6	34.0	60	–	0.0	0.0
Lane 2	0	143	0	143	17.5	345	0.414	100	33.0	LOS C	5.7	35.4	60	–	0.0	0.0
Lane 3	0	90	53	143	13.9	346	0.414	100	33.0	LOS C	5.7	34.1	60	–	0.0	0.0
Approach	77	297	53	426	15.6		0.414		33.1	LOS C	5.7	35.4				
East: POSTA BET																
Lane 1	72	40	0	112	13.4	260	0.431	100	34.7	LOS C	4.6	27.1	60	–	0.0	0.0
Lane 2	0	142	23	165	22.0	383	0.431	100	24.6	LOS C	5.4	35.5	60	–	0.0	0.0
Approach	72	182	23	278	18.5		0.431		28.7	LOS C	5.4	35.5				
North: PIAZZA																
Lane 1	84	66	0	150	12.2	387	0.388	100	32.1	LOS C	5.9	34.6	60	–	0.0	0.0
Lane 2	0	152	0	152	15.2	392	0.388	100	32.1	LOS C	6.0	36.3	60	–	0.0	0.0
Lane 3	0	95	61	156	17.2	402	0.388	100	23.4	LOS C	4.9	30.3	60	–	0.0	0.0
Approach	84	313	61	458	14.9		0.388		29.1	LOS C	6.0	36.3				
West: TIKUR ANBESSA																
Lane 1	57	53	0	110	14.2	281	0.392	100	32.6	LOS C	4.4	26.2	60	–	0.0	0.0
Lane 2	0	134	36	170	15.2	433	0.392	100	25.9	LOS C	6.1	36.9	60	–	0.0	0.0
Approach	57	187	36	280	14.8		0.392		28.6	LOS C	6.1	36.9				
Intersection				1442	15.8		0.431		30.1	LOS C	6.1	36.9				

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.

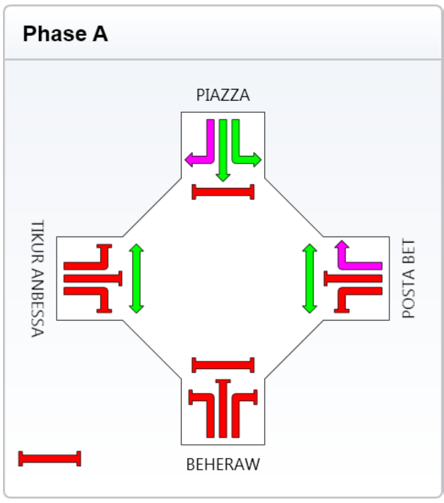
LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).

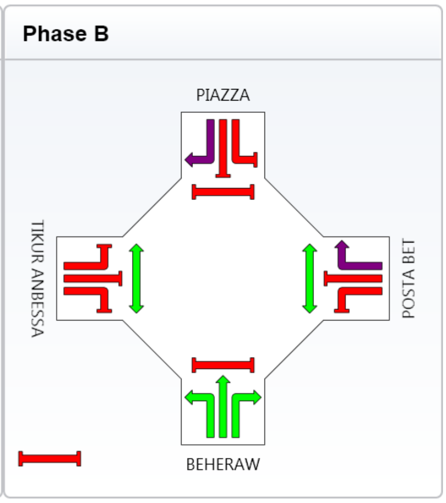
Appendix 12 Phasing summary for the revised

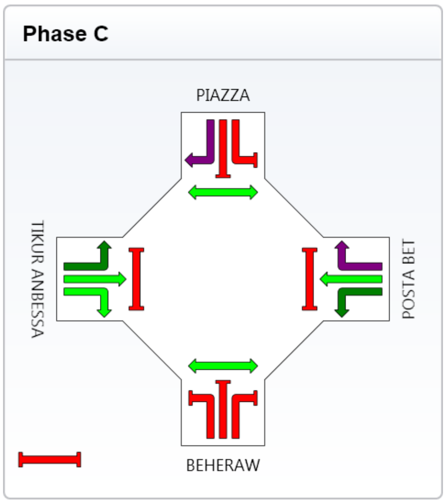
Phase times determined by the program
Sequence: 3- Phasing
Input Sequence: A, B, C
Output Sequence: A, B, C

Phase Timing Results

Phase	A	B	C
Green Time (sec)	24	24	33
Yellow Time (sec)	3	3	3
All-Red Time (sec)	2	2	2
Phase Time (sec)	29	29	38
Phase Split	30 %	30 %	40 %







	Normal Movement		Permitted/Opposed
	Slip-Lane Movement		Opposed Slip-Lane
	Stopped Movement		Continuous Movement
	Turn On Red		Undetected Movement
			Phase Transition Applied

Appendix 13 Movement timing summary

Revised Hager Astedader Intersection
Signals - Fixed Time   Cycle Time = 96 seconds (User-Given Cycle Time)
Design Life Analysis (Practical Capacity): Results for 1 years
Sequence: 3- Phasing
Input Sequence: A, B, C
Output Sequence: A, B, C

12 (West R)

PEDESTRIAN SIGNAL TIMING

P3 (South stage 1)

P2 (East stage 1)

P1 (North stage 1)

P4 (West stage 1)

Appendix-14 graphical representation of performance measures