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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

CAPACITY EVALUATION OF ROUNDABOUT AND SIGNALIZED JUNCTIONS IN ADDIS ABABA

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The thesis Submitted to School of Graduate Studies of Addis Ababa University in partial fulfillment of the requirement for the Degree of Masters of Science in Civil Engineering (Road and Transport Engineering Stream)

Advisor

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MSc. Thesis on

CAPACITY EVALUATION OF ROUNDABOUT AND SIGNALIZED JUNCTIONS

IN

ADDIS ABABA

By Temesgen Alemayehu

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DECLARATION

I certify that this research work titled "**Capacity Evaluation of Roundabout and Signalized Junctions in Addis Ababa**" is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where material has been used from other sources it has been properly acknowledged/ referred.

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ABSTRACT

Intersections are usually considered as the critical points within the urban road network and the evaluation of their performance provides valuable understanding and useful indication about the performance of the city road network system. The capacity of signalized and roundabout intersection is of more significant because such intersections often control the ability of the city streets to accommodate traffic.

In Addis Ababa, most of the intersections are congested and operate in poor LOS. During peak hours, it is common to see congestion, long queues & delay at junctions. Accordingly, the objective of the study is to evaluate the capacity and LOS of roundabout and signalized intersections in Addis Ababa, and based on the results of the analysis to draw conclusions and recommendations for possible future considerations during junction design in the city.

To achieve this objective, four roundabout and two signalized intersections in the city from major road corridors were selected.

The required data was collected from selected junctions manually by the researcher using the necessary survey equipment suitable for the study purpose. The capacity analysis throughout the study was performed based on analytical method with some geometric elements using SIDRA Software program package, in order to identify the level of service for the studied intersections.

The result of this study indicates that the studied intersections Gerji-Imperial, Bole-Mdihanialem, Teklehaymanot, Ayer-Tena, Legehar and Post-Office in the city currently serving in poor condition of Level of Service (LOS) of "F" except Post-Office intersection which has LOS of "E". Thus, after carrying out additional detail investigation and taking into consideration future traffic growth appropriate improvement should be made by concerned bodies as soon as possible.

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tc	Critical gap
tf	Follow-up time
Vc	Conflicting volume
Cn	Capacity of non-priority stream
vp	Priority flow rate
Csh	Capacity of shared-lane
vl	Volume of left-turn movement in shared-lane
vr	Volume of right-turn movement in shared-lane
tc,x	Critical gap for movement x
tc,	base Base critical gap
G	Percent grade divided by 100
Q95	95th-percentile queue
T	Analysis time period
D	Control delay

Acronyms

U.S. HCM	United States Highway Capacity Manual
TRB	Transportation Research Board
LOS	Level of service
pcph	Passenger car per hour
vph	Vehicle per hour
PHF	Peak-hour factor
veh	Vehicle
FHWA	Federal Highway Agency
AACRA	Addis Ababa City Roads Authority
CBD	Central Business District
USA	United State of America
SIDRA	Signalized (un-Signalized) Intersection Design and Research Aid
TDM	Travel Demand Management
PCU	Passenger Car Unit
TRRL	Transport Research Record Laboratory

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1. INTRODUCTION

1.1 Background

Junctions are usually considered as the critical points within the urban road network and the evaluation of their performance provides valuable understanding and useful indication about the performance of the network. As many studies show, the failure of junction directly reduces the roadway corridor capacity and its improvement can be a very cost effective means of increasing a corridor's traffic capacity. Hence, studies of signalized and roundabout junctions are significant because such junctions often control the capacity of the city streets to accommodate traffic.

In Addis Ababa, most of the junctions are congested and their capacities or level of services is not well identified. During peak hours, it is common to see congestion, long queues and delay at junctions. Hence, evaluation of the capacities of junctions is very important since it is directly related to delays, accidents, high operation costs, and environmental degradation. The study is also important for urban planners, traffic engineers and other concerned bodies who aim to mitigate the problems.

1.2 Problem statement

Urban traffic congestion is currently severe in most cities in the world due to economic and social development. Prevalently, Ethiopia is one of the countries that are in rapid economic development. This influences the travel pattern of the community from their origin to any destination. Generally, transportation system is affected by economic development that results in increase of vehicles on roads. This and other phenomenon increases road congestion, especially during peak hours. Traffic congestion is also influenced by road network. In a road network, intersection is a major cause of bottlenecks thus contributing to congestion.

Despite the intensive road network expansion in Addis Ababa, traffic congestion, long queues and excessive delay during peak hours at junctions have been major problems in the city. However; the problem being recognized by all road users and transport professionals, little attention has been paid to junctions and their capacities. There is only a single attempt made by Tewodros G. (2007) to assess the capacity of roundabout junctions in Addis Ababa on his thesis research. He considered only roundabouts but not intersections. Hence, questions on current level of services on both signalized and roundabout junctions are still not well investigated and answered.

Therefore, quantitative researches based on the engineering parameters of capacity evaluation on signalized and roundabout junctions should be conducted to answer at least the following questions:

- What is the existing level of service of study junctions in Addis Ababa?
- What parameters affect the capacities of intersections in Addis Ababa?
- What improvement measures should be taken to mitigate the problems?

1.3 Objectives

The main objectives of the present study are:

- To evaluate the level of service of Roundabouts and Signalized Intersections in Addis Ababa.
- To select appropriate methodology for evaluating the level of service of Roundabouts and Signalized Intersections in Addis Ababa.
- To draw conclusions and recommendations based on the results of the analysis for possible future considerations during junction design in Addis Ababa.

1.4 Thesis Organization

This thesis consists of seven chapters. The first chapter is introduction, which discusses the problem and objectives of the research work. Chapter 2 deals with literature review, which discusses history and definition of roundabouts and signalized intersections, and the methodologies of roundabout and pre-time signalized intersection capacity analysis. Chapter 3 research approach and software calibration. Chapter 4 discuses data collection and method of analyses. Chapter 5 deals with the results and discussions. Chapter 6 presents conclusions and recommendations and chapter 7 proposes future research area.

2. LITERATURE REVIEW

Literature review will detail the necessary information to ensure the variables affecting the capacity of roundabouts and signalized junctions are understood and accounted for. This information will provide the structure for the variable inputs into the computer software modeling to determine the capacity of signalized and roundabout Junctions.

This section will first provide general information about roundabouts and signalized Junctions and their key features. It will then look into the important features of signalization and traffic flow characteristics that will impact on determining the capacity of roundabout and signalized Junctions. Finally it will describe the features and limitations of the computer software modeling used in determining the research objectives.

2.1 Roundabout

2.1.1 General

'Traffic circles have been part of the transportation system in the United States since 1905, when the Columbus Circle designed by William Phelps Eno opened in New York City' (FHWA, 2000). These traffic circles were unlike modern roundabouts today as they gave entering traffic the right of way, thus causing the circulating traffic to give way. This developed numerous problems with roundabouts which involved locking up of traffic around the central Island, aiding high speed entry and the merging and weaving of vehicles leading to severe crashes.

After numerous traffic mishaps within these traffic circle intersections in the United States, the Americans decided to abandon the traffic circle designed intersections. However the British decided to continue to develop and refine the design of these traffic circles and came up with the mandatory give way rule that allowed the development of modern roundabouts to continue to become safe and effective intersections.

'In 1966, the United Kingdom adopted mandatory "give-way" rule at circular intersections, which required entering traffic to give way, or yield, to circulating traffic' (FHWA, 2000). By adopting this rule, roundabouts became free flowing as it did not allow vehicles to enter the roundabout until there was a sufficient gap in the circulating traffic.

The differences of modern roundabouts from the traditional traffic circles include (FHWA, 2000):

- Roundabouts require entering drivers to give way to all traffic within the roundabout. Roundabouts allow the inner lane of a multi lane roundabout to exit.
- Deflection on entry is used to maintain low speed operation in roundabouts.
- Pedestrians are permitted from the central island of a roundabout.
- Modern roundabouts are much smaller in diameter than traffic circles.

The United States of America (USA) finally adopted the design of the modern roundabout in 1990 in Summerlin, Las Vegas. Since then USA have adopted the modern roundabout and as of December 2009, the number of modern roundabouts in the USA was approximately 2,300(FHWA, 2000).

In 1984, the French government adopted the mandatory give- way rule to circulating traffic and as of mid-1997 there are about 15,000 modern roundabouts in France (Jacquemart, 1998). In addition to their popularity in Great Britain and France, roundabouts are very common in Germany, Switzerland, Spain and Portugal. 'Outside of Europe the modern roundabout is a standard feature in Australia and it is becoming more common in New Zealand, South Africa and Israel' (Jacquemart, 1998). According to Federal Highway Administration 2000, roundabouts can be classed into six main categories as shown in Table-1 below (FHWA2000).

Table 2.1 – Design characteristics for each six roundabout categories

Design Element	Mini-Roundabout	Urban Compact	Urban Single-Lane	Urban Double-Lane	Rural Single-Lane	Rural Double-Lane
Recommended maximum entry design speed	25 km/h (15 mph)	25 km/h (15 mph)	35 km/h (20 mph)	40 km/h (25 mph)	40 km/h (25 mph)	50 km/h (30 mph)
Maximum number of entering lanes per approach	1	1	1	2	1	2
Typical inscribed circle diameter	13m to 25m (45ft to 80ft)	25m to 30m (80ft to 100ft)	30m to 40m (100ft to 130ft)	45m to 55m (150ft to 180ft)	35m to 40m (115ft to 130ft)	55m to 60m (180ft to 200ft)
Splitter island treatment	Raised if possible, crosswalk cut if raised	Raised, with crosswalk cut	Raised, with crosswalk cut	Raised, with crosswalk cut	Raised and extended, with crosswalk cut	Raised and extended, with crosswalk cut
Typical daily service volumes on 4-leg roundabout (veh/day)	10,000	15,000	20,000	40,000 to 50,000	20,000	40,000 to 50,000

Source: Federal Highway Administration, 2000.

This research focuses on double lane Roundabouts and pre time Signalized Intersections. The key geometric elements of Roundabouts are shown in Figure 2.1.

Roundabouts introduce an entry curve to slow entering traffic down to give-way to circulating traffic. The entry and exit curves are separated by a raised median called a splitter island, which is designed to deflect and slow entering traffic in conjunction with the entry curve. The vehicles then enter the roundabout when a sufficient gap is presented, then travel within the circulating carriageway until they reach their desired exit.

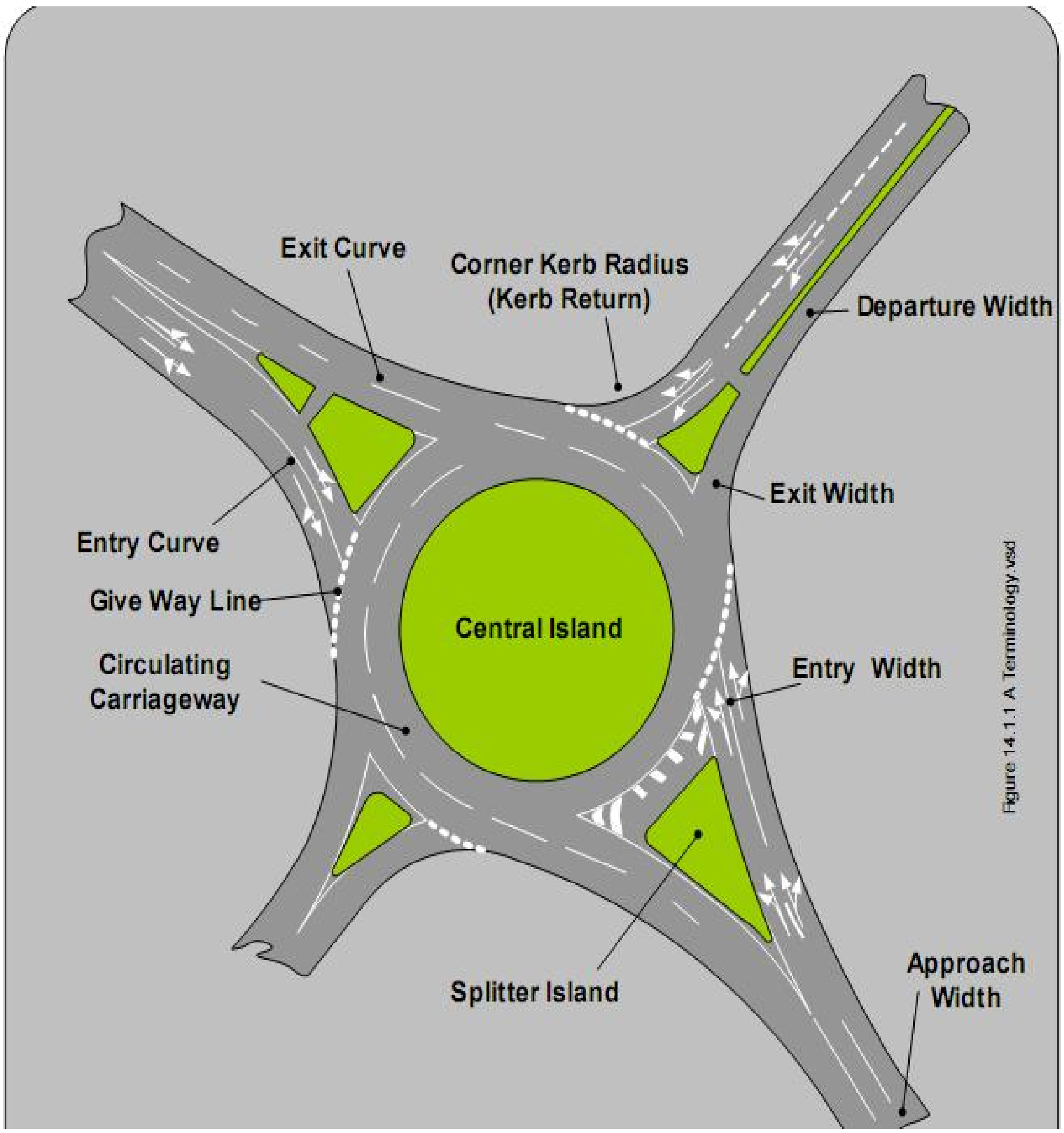


Figure 2.1-GeometricEelements of a Roundabout. (AACRA Manual, 2004)

Table 2.2-Description of Key Roundabout Features.

Feature	Description
Approach Curve	Approach Curve is used to slow down the operating speed of vehicles coming from a high speed environment
Entry Curve	Entry Curve is used to deflect and slow entering vehicles to an appropriate speed to safely circulate the roundabout.
Entry Width	Entry Width is the width of the entry where it meets the circulating carriageway.
Holding Line	Holding Line is pavement marking that defines where the vehicles have to give-way to the circulating traffic. It is generally marked along the inscribed circle.
Circulating Carriageway	Circulating Carriageway is a curved path used by vehicles to travel around the central island. This is defined by line marking.
Circulating Carriageway Width	Circulating Carriageway Width defines the roadway width for vehicle circulation around central island. The Circulating Carriageway Width has to be wide enough to accommodate the largest design vehicles turning path.
Exit Width	Exit Width is the width of the exit where it meets the circulating carriageway.
Exit Curve	Exit Curve is generally bigger/flatter than the entry curve to allow vehicles to exit at faster speed to improve traffic capacity and flow.

2.1.2 Capacity of Roundabout

The Highway Capacity Manual (HCM, 2010) defines the capacity of a facility as 'the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions. The capacity of a roundabout depends on two major principles:

- The effect of traffic flow and driver behavior; and
- The effect of roundabout geometry.

2.1.2.1 Effect of traffic flow and driver behavior

Different driver behaviors are experienced around the world. Some drivers in certain countries may approach a roundabout at a higher speed or accept a smaller gap upon entry into the roundabout, which has an impact on the capacity of the roundabout.

There are several effects of driver behavior that are consistent across the world that have an impact on the capacity of a roundabout. The effects from driver behavior on traffic flow are:

- Effect of exiting vehicles – The effect of exiting vehicles may have an impact on when the entering vehicle feels comfortable to enter the circulating carriageway. This effect is similar to a vehicle wishing to turn right into the lane a vehicle is exiting; the driver may not feel comfortable to exit until the vehicle is in the motion of turning even if the vehicle has indicated on turning left.
- Changes in effective priority – When the roundabout is under saturated conditions driver behavior become more aggressive. Instead of entering traffic providing the required gap as to not disrupt the circulating traffic, the vehicles are more likely to forcefully enter requiring the circulating traffic to give way to the entering traffic.
- Origin to destination patterns – This has an impact if there is a heavy through or left turn movement from one leg. If there is continual traffic flow that is unimpeded from a downstream leg it will not provide sufficient gaps for entering traffic causing long delays and traffic queues from upstream legs.

The effects of driver behavior can be so variable that it is difficult to model for capacities accurately based on computer software. Inputs for driver behavior within computer soft ware modeling should be based on extensive field testing on real life conditions with similar geographic conditions.

2.1.2.2 Effect of Roundabout Geometry

The geometry of a roundabout can have an impact on the capacity of a roundabout in the following areas:

- 'It affects the speed of vehicles through the intersection, thus influencing their travel time by virtue of geometry alone (geometric delay)' (FHWA, 2000).
- The larger the diameter of the roundabout provides more capacity within the circulating carriageway.
- The width of the circulating carriageway, entry widths and exit widths have impact on the capacity and can govern the speed at which drivers feel comfortable to enter and navigate around the roundabout.
- 'It can affect the degree to which flow in a given lane is facilitated or constrained. For example, the angle at which a vehicle enters affects the speed of that vehicle, with entries that are more perpendicular requiring slower speeds and thus longer headways. Likewise, the

geometry of multilane entries may influence the degree to which drivers are comfortable entering next to one another' (FHWA, 2000).

- 'It may affect the driver's perception of how to navigate the roundabout and their corresponding lane choice approaching the entry' (FHWA, 2000).

The capacity of a roundabout is mainly dependent on the amount of approaching lanes and circulating lanes. The capacity is also affected more subtly by entry curves, entry width and lane widths. There has been extensive research done into the capacity of two lane roundabouts across the world. Generally it is found that the capacity of a two-lane roundabout is expected to be between 40,000 to 50,000 vehicles per day (FHWA, 2000).

Figure 2 shows the result of a research conducted by the Federal Highway Administration into the capacity of a two lane roundabout. The capacity forecast is based on simplified UK empirical regression methods that differ from Australia and USA methods of gap acceptance theory.

It identifies that the maximum entry flow reaches a maximum of 2400 veh/hr when there is no circulatory flow. On the contrary it shows that when the circulatory flow reaches approximately 3400 vehicle per hour, no vehicles are able to enter into the roundabout above the capacity.

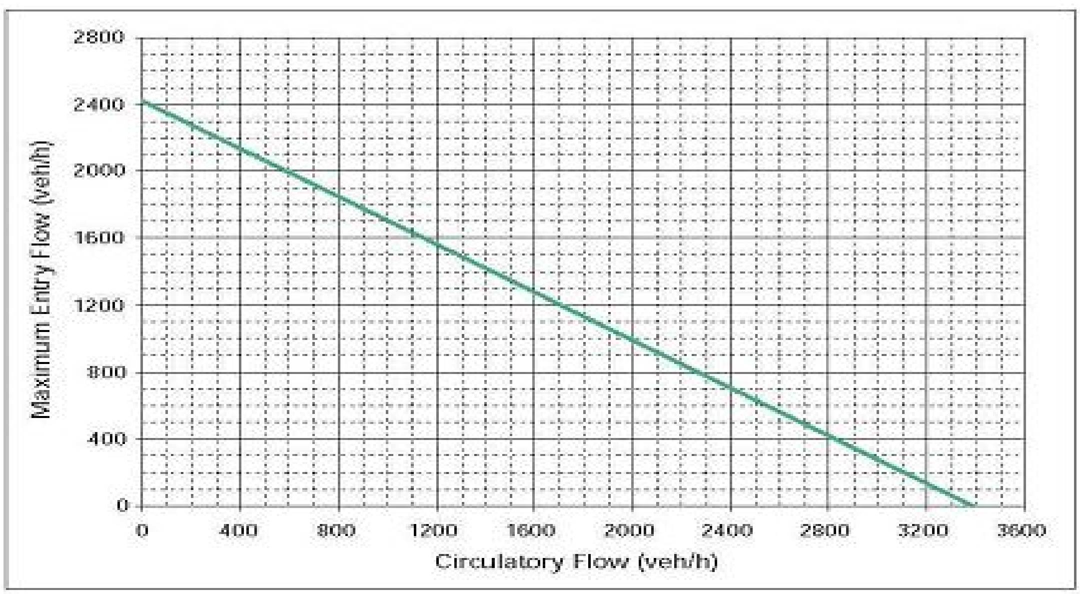


Figure 2.2 – Approach Capacity of a Two-Lane Roundabout (FHWA, 2000)

The HCM (2010) defines the capacity of two lane roundabouts with two circulating flows as:

$$C_{e,R,pce} = 1{,}130e^{(-0.0007)V_{c,pce}} \qquad 2.1$$

$$C_{e,L,pce} = 1{,}130e^{(-0.0007)V_{c,pce}} \qquad 2.2$$

Where:

$C_{e,R,pce}$ = Capacity of the right entry lane, adujested for heavey vehicles (pc/hr),

$C_{e,L,pce}$ = Capacity of the left entry lane, adjusted for heavey vehicles (pc/hr), and

$V_{c,pce}$ = Conflicting flow rate (total of both lanes) pc/hr.

Figure 2.3 has been developed based on these equations to produce the capacity estimates of single-lane and multilane entry capacities.

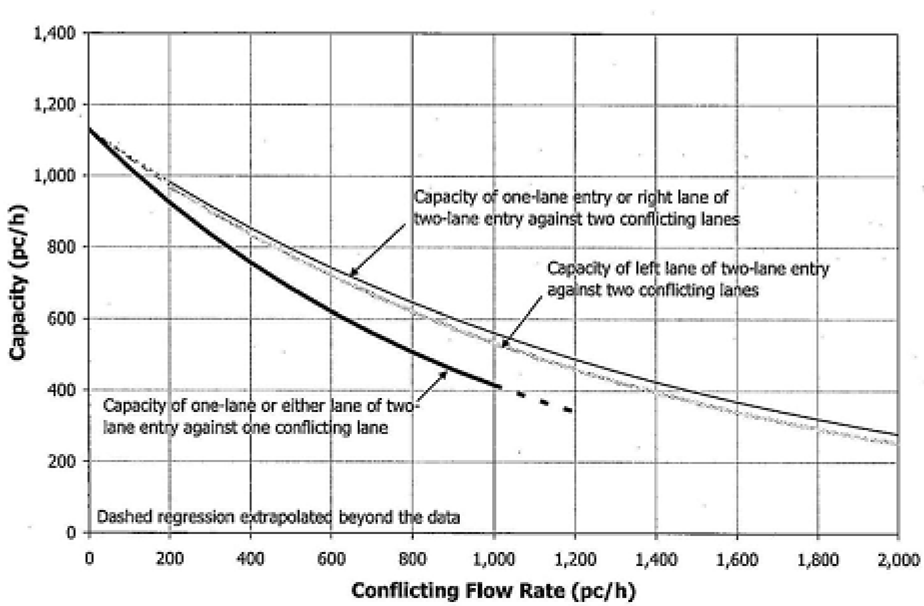


Figure 2.3-Capacity of Single-Lane and Multilane Entries (HCM, 2010)

Pedestrians can reduce the entry capacity of a roundabout if they assert right-of-way on vehicles entering the roundabout. Worldwide, there are different rules and regulations regarding the right-of-way of pedestrians in regards to roundabouts. In Australia vehicles are not obliged to give way to pedestrians upon exiting the roundabout according to the National Transport Commission.

A research by (Brilon, Stuwe, and Drews ,1993) determined a reduction factor for pedestrians on the capacity for roundabout which is represented in Figure 2.4.

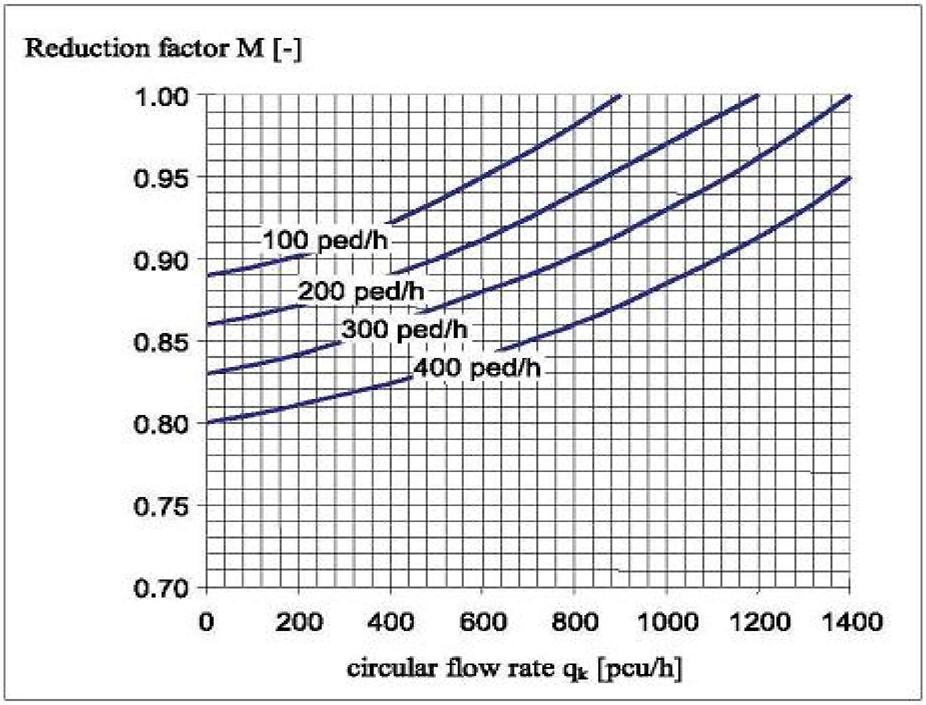


Figure 2.4 – Capacity reduction factor for a two lane roundabout assuming pedestrian priority.

The above factors have to be considered when determining the capacity of a roundabout. Each specific roundabout will have its own unique parameters due to geometry, driver behavior and traffic fleet. To determine the most accurate capacities for a roundabout, all the inputs of these parameters should be conducted on research to find the capacity of roundabout.

2.1.3 Methods for Estimating the Capacity of Roundabouts

Capacity is the main determinant of the performance measures such as delay, queue length and stop rate. The relationship between a given performance measure and capacity is often expressed in terms of degree of saturation (demand volume- capacity ratio). Capacity is the maximum sustainable flow rate that can be achieved during a specific time period under prevailing road, traffic and control conditions. The prevailing condition is important since capacity is not a constant value, but varies as a function of traffic flow levels. Capacity represents the service rate (queue clearance rate) in the performance (delay, queue length, stop rate) functions, and therefore is relevant to both under saturated and over saturated conditions. Conceptually, this is different from the maximum volume that the intersection can handle which is the practical capacity (based on the a target degree of saturation) under increased demand volumes, not the capacity under prevailing conditions (Akcelik, 2005).

There are two distinct theories or methodologies to assess the capacity of the roundabouts. These theories are:

(i) The empirical model; and

(ii) The analytical or gap acceptance based model.

2.1.3.1 Empirical (statistical) model

This model correlates geometric features and performance measures, such as capacity, average delay and queue length, through regression of field data. In this way, a relationship (generally linear or exponential) between the entering flow of an approach and the circulating flow in front of it (RODEGERDTS L. et al.2004) is generated. This model is better than analytical ones but requires an oversaturated condition for calibration and may have poor transferability to other countries.

2.1.3.2 Analytical (Gap-acceptance) model

This model can be developed from uncongested sites; the driver on the approach (entering flow) needs to select an acceptable gap in the circulating stream, to carry out the entering maneuver. The gap is the headway between two consecutive vehicles on the circulating flow; so, the "critical gap" (t_c) is the minimum headway accepted by a driver in the entering stream. If the gap accepted is larger than minimum, then more than one driver can enter the roundabout; the headway between two consecutive vehicles in the entering flow, which utilizes the same gap, is defined as "follow-up time" (t_f). So, the analytical model calculates the roundabout capacity as a function of the critical gap, the follow-up time and the circulating flow. However for capacity evaluation, the following are some assumptions due to the nature of geometry, turning movement, vehicle types and approach grade:

- Constant values for "t_c" and "t_f";
- Exponential distribution for the gaps into the circulating flow;
- Constant traffic volumes for each traffic flow

These specific assumptions make the use of these models difficult in practice. Furthermore, there are other limitations, such as:

- The estimation of the critical gap is not easy;
- The geometric factors are not directly taken into account;

- The inconsistent gaps are not accounted for in theory (forced right of way when traffic is congested, circulating drivers give up right of way, different gap accepted by different vehicles, the rejection of large gap before accepting a smaller one, etc.)

2.1.3.3 Empirical Regression (Geometric) Vs Analytical (Gap Acceptance)

Kimber in his initial laboratory report (1980) states that the dependence of entry capacity on circulating flow depends on the roundabout geometry. Kimber defines five geometric parameters which have an effect on the capacity. These are entry width and flare, the inscribed circle diameter (a line that bisects the center island and the circulating lane twice), and the angle and radius of the entry.

Similarly, Kimber in his paper (1989) states that gap acceptance is not a good estimator of capacity in the United Kingdom. He further states that, single-lane entries are the basis for the simplest case for gap acceptance models; while, empirical models apply also to multilane entries. Kimber reasons that gap acceptance models do not increase capacity correctly when additional entry lanes are added. Perhaps Kimber's reasoning in this publication was due to its creation date. Many new ideas have been put forth on how additional lanes affect capacity in a gap acceptance model.

Kimber makes two interesting comments in his paper (1989), the first being that many circumstances exist where driver response to yield signs conforms to gap acceptance assumptions. However, he questions whether or not gap acceptance is a sufficient description of this interaction. The main flaw of the gap acceptance theory is that it poorly evaluates capacity for at-capacity roundabouts. Flannery et al (1998) comment that congested roundabouts are very scarce in the United States. Therefore, the empirical regression model might be difficult to use since it requires a saturated facility to be calibrated.

The second comment by Kimber is that because of driver behavior and geometric variation, it is not safe to transfer theories from one country to another. Fisk, in a 1991 article, agreed that regression models should not be transferred from region to region, or between roundabouts of different geometrical configurations. Fisk writes that because a regression model requires a great deal of data for calibration, it may work well at a specific facility, but cannot be universal. Further, Fisk feels that gap acceptance models demonstrated reliable predictions for both capacity and delay of New Zealand roundabouts. Fisk believed that by changing vehicle class parameters or providing a range of critical gap values, gap acceptance modeling could be used in other locations.

Akcelik (ARR 321, 1998) contends that while Kimber objects to the "simple gap acceptance method", the model presented for use in the SIDRA software package goes beyond the simple approach. One main addition to Akcelik's gap acceptance approach is the modeling of the roundabout based on approach lane use. Furthermore, Akcelik writes that the method presented in his report improves capacity prediction during heavy flow conditions and especially for multilane roundabouts with uneven approach demands.

Many of the additional elements used in SIDRA are parameters used to enhance its basic gap acceptance theory. The parameters that deal with the entering traffic stream include the inscribed diameter, average entry lane width, the number of circulating and entry lanes, the entry capacity (based on the circulating flow rate), and the ratio of the entry flow to the circulating flow.

These additional model elements demonstrate the detailed nature of the SIDRA model (AACRA also recommend SIDRA for capacity evaluation). Another important component of Akcelik's formulation is the identification of the dominant and sub-dominant entry lanes based on their flows. The dominant lane has the highest flow rate, and all others are subdominants. The purpose of this component is that dominant and sub-dominant entry lanes can have different critical gap and follow up times. The distinction between dominant and sub-dominant lanes appears to be quite important because vehicles using the leftmost entry lane must find a gap in both circulating lanes, as opposed to the right entry lane, which must only deal with traffic in the outer most circulating lane.

SIDRA also includes a passenger car equivalent (pce) factor for heavy vehicles. In this regard, Akcelik (1997) recommended that pce per hour be used in place of vehicles per hour when the proportion of heavy vehicles surpassed 5percent. Many other authors concurred Akcelik's recommendation. In their Roundabout Design Workshop (2001), Ray and Rodegertds explain that heavy vehicles primarily affect roundabout capacity due to their size, not because of their slower acceleration and speed. The U.S. DOT's Roundabout Guide (2000) suggests typical PCE conversion factors for adjusting entering and circulating volumes. These include a 1.5 factor for recreational vehicles and buses, and a 2.0 factor for tractor-trailers.

For the purpose of this thesis, the gap acceptance theory appears to be the most appropriate basis for the capacity evaluation. Because the empirical formulation has some drawbacks, for example, data has to be collected at over-saturated flow (or at capacity) level. It is a painstaking task to collect enough data to ensure reliability of results, and this method is sometimes inflexible under unfamiliar circumstances, for example, when the value is far out of the range of regressed data. Consequently, researchers looked for other reliable methods of determining roundabout capacity. Many researchers agree that a gap acceptance theory (Analytical Method) is a more appropriate tool. An advantage of this method is that the gap acceptance technique offers a logical basis for the evaluation of capacity. Secondly, it is easy to appreciate the meaning of the parameters used and to make adjustments for unusual conditions. Moreover, gap acceptance conceptually relates traffic interactions at roundabouts with the availability of gap in the traffic streams (Thaweesak, 1998).

Further, investigation into which theory is more appropriate shows that the gap acceptance model is felt to be more transferable from country to country and location to location than is the empirical regression model. List et al (1994) investigated multilane roundabouts in New York State using gap acceptance based models. They commented that it is possible to transfer capacity equations from overseas.

2.2 Capacity Models Developed in Different countries

2.2.1 United Kingdom

Kimber (1980) conducted studies in the United Kingdom (UK) and developed an empirical linear regression equation based on large number of observations at roundabouts operating at-capacity. This equation directly relates capacity to roundabout geometry.

$$Qe = k(F - fcQc) \tag{2.3}$$

Where

Qe ; entry capacity, vph;

Qc ; circulating flow, pce/h; and

k, F, f_c; constants derived from the geometry of the roundabout.

2.2.2 Australia

Troutbeck (1993) conducted studies for the Australian Road Research Board and developed an analytical equation based on gap acceptance characteristics observed and measured at roundabouts operating below capacity. Critical gap and follow-up times are related to roundabout geometry and capacity is then determined using the following equation.

$$\mathbf{Qe} = \frac{\propto \mathbf{Qce}^{-\lambda(\mathbf{tc}-\mathbf{tm})}}{\mathbf{1}-\mathbf{e}^{-\lambda \mathbf{tf}}} \tag{2.4}$$

Where

Q_e; entry capacity, vph;

Q_c ; circulating flow, vph;

α ; proportion of non-bunched (free) vehicles in the circulating streams;

λ ; model parameter;

t_c ; critical gap, s;

t_f; minimum headway in circulating streams, s; and

t_m ; follow-up time, s.

2.2.3 Germany

Stuwe (1992) studied eleven German roundabouts and developed the following exponential regression equation to estimate roundabout entry capacity.

$$C = Ae^{-(\frac{BVc}{10000})} \tag{2.5}$$

Where

C ; entry capacity, vph;

V_c ; circulating flow, vph; and

A, B ; parameters dependent on the number of circulating and entry lanes.

2.2.4 Switzerland

Bovy (1991) developed the following linear model based on studies of roundabouts in Switzerland.

$$C=1500-(8/9)V_g \tag{2.6}$$

Where

C ; entry capacity, vph; and

V_g ; impeding flow vph, determined with the equation below.

V_g; βV_c+αV_s (2.7)

Where

V_c ; circulating flow, vph;

Vs ; exiting flow, vph; and

α, β ; parameters depending on geometry and number of circulating lanes.

2.2.5 France

In 1988 the French government organization Centre d'Etudes des Transports Urbains (CETUR), now known as CERTU, developed the following linear model for urban roundabout capacity.

$$C=1500-(5/6)V_g \quad (2.8)$$

Where C ; entry capacity, vph; and

V_g ; impeding flow, vph, determined with the equation below.

$$Vg=Vc+\alpha Vs \quad (2.9)$$

Where

V_c; circulating flow, vph;

V_s ; exiting flow, vph; and

α ; parameter dependent on splitter island width.

Similarly SETRA, the French national design organization for rural highways, developed a linear equation for the capacity of rural roundabouts in 1987 as follows:

$$C=(1330-0.7V_g)(1+0.1[l_e-3.5]) \quad (2.10)$$

Where C; entry capacity, vph;

Vg ; impeding flow, vph, dependent on circulating and exiting flows and geometry;

l_e ; entry width, m.

2.2.6 United State U.S.

The Highway Capacity Manual 2010 (HCM) presents a methodology for estimating roundabout capacity based on gap acceptance. The capacity model is applicable only to single-lane roundabouts if the circulating volume is less than 1,200 vph. Insufficient experience in the U.S. precludes the HCM from containing guidelines for multiple-lane roundabouts. The

HCM assumes that the gap acceptance characteristics of drivers entering a roundabout to be similar to those of drivers making right turns at two-way stop-controlled intersections. The following model was developed in the U.S.

$$Ca = \frac{Vc\, e^{-\frac{Vctc}{3600}}}{1-e^{-\frac{Vctt}{3600}}} \tag{2.11}$$

Where

c_a ; approach capacity, vph;

v_c ; conflicting circulating traffic, vph;

t_c ; critical gap, s; and

t_t ; follow-up time, s.

The HCM (2010) recommends ranges for critical gap and follow-up times, which are presented in Table 2.3.

Table 2.3- HCM Critical Gap and Follow-up Times (HCM 2010, Exhibit 17-37)

Bound	Critical gap(s)	Follow up time(s)
Upper bound	4.1	2.6
Lower bound	4.6	3.1

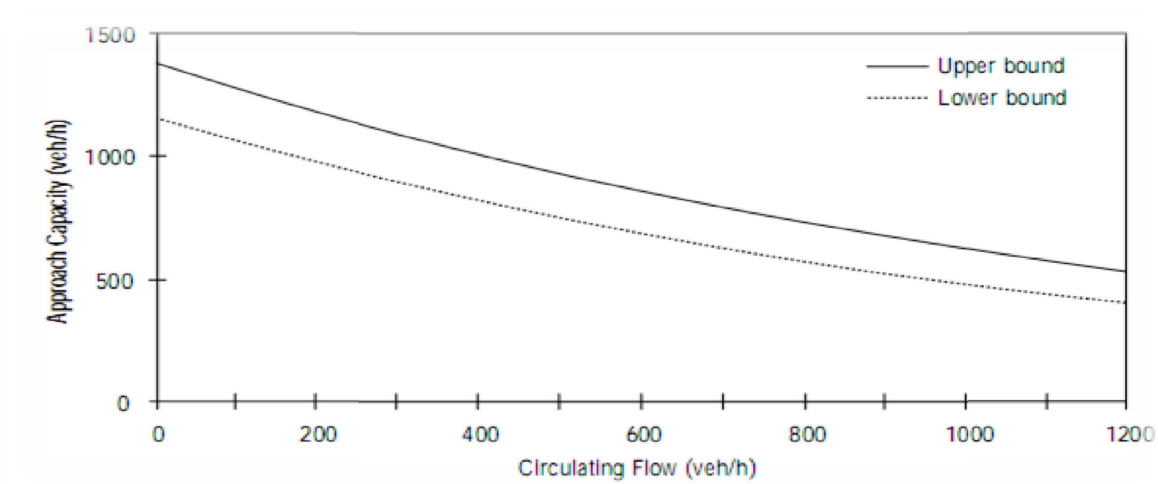


Figure 2.5- Roundabout Approach Capacities (HCM 2010, Exhibit 17-38)

The U.S. Federal Highway Administration (FHWA) publication, *Roundabouts: an Informational Guide* (2000) presents a more comprehensive discussion of roundabout performance analysis than the HCM (2010). This document differentiates roundabouts based upon size and environment and provides capacity models for urban compact roundabouts, typical single-lane roundabouts, and typical double-lane roundabouts.

Urban compact roundabouts have nearly perpendicular single-lane approach legs and inscribed circle diameters in the range of 25 to 30 m (82 to 98.5 ft). The capacity model for urban compact roundabouts is based on the capacity curves developed by Brilon, Wu, and Bondzio (1997) for German roundabouts with single-lane entries and a single-lane circulatory roadway as follows:

$$Q_e = 1218 - 0.74 Q_c \qquad (2\text{-}12)$$

Where: Q_e : entry capacity, vph; and

Qc : circulating flow, vph.

The UK equation developed by Kimber (1980) form the basis for the capacity models derived for typical single-lane and double-lane roundabouts. The indicated assumptions of geometric parameters were chosen so as to simplify the equations as follows:

Single-lane roundabouts:

$$Q = Min\{(1212 - 0.5447Qc), (1800 - Qc)\} \qquad (2\text{-}13)$$

Assuming: D = 40 m, r = 20 m, φ = 30°, v = 4 m, e = 4 m, l' = 40 m

Double-lane roundabouts:

$$Q_e = 2424 - 0.7159 Q_c \qquad (2\text{-}14)$$

Where: Assuming: D = 55 m, r = 20 m, φ = 30°, v = 8 m, e = 8 m, l' = 40 m.

Q_e : entry capacity, vph;

Q_c : circulating flow, vph;

D : inscribed circle diameter, m;

r : entry radius, m;

φ : entry angle, degrees;

v : approach half width, m;

e : entry width, m; and

l' : effective flare length, m.

When capacity requirements are met at double-lane roundabouts with the gradual widening of the approaches (flaring), the capacity of each entry lane, q_{max}, is estimated with the equation below. Equation 2.15 is based on Wu's (1997) studies on the effect of short lanes on entry capacity for two-lane roundabouts

$$q_{max} = \frac{2q}{x^{n+1}\sqrt{2}} = \frac{q_{max2}}{^{n+1}\sqrt{2}} \quad (2\text{-}15)$$

Where: q : flow in each lane, pce/h (assumed to be equal in both lanes);

x : degree of saturation;

n : length of queue space, vehicles; and

$q_{max\,2}$: capacity of entry at a double-lane roundabout, pce/h.

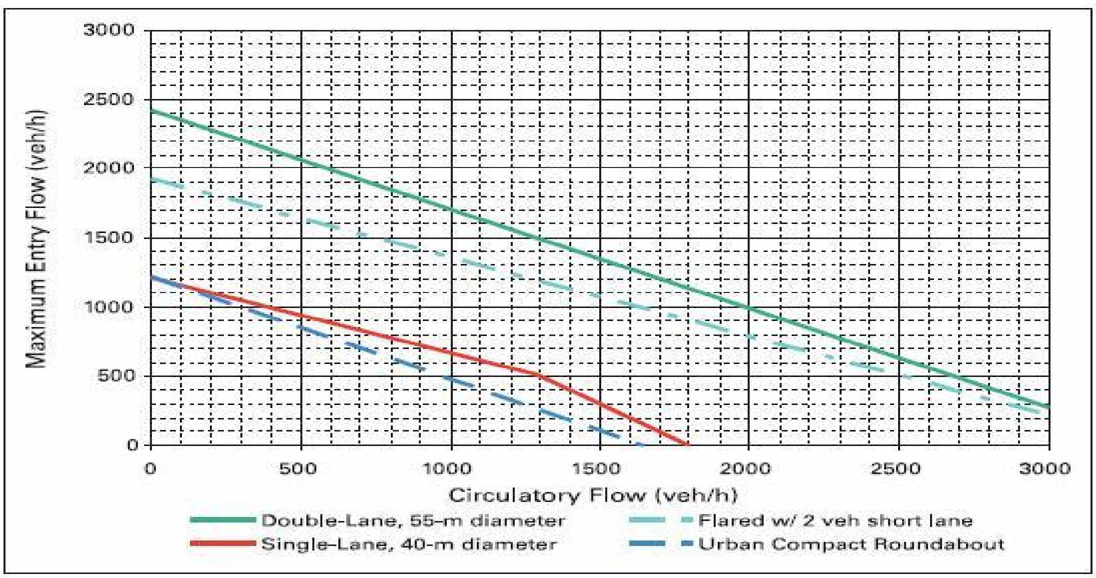


Figure 2.6-comparison of entry capacities based on the equation provided by the FHWA.

Among all above mentioned models Australian and united state U.S models are developed based on gap acceptance method and can be applicable for Addis Ababa.

2.3 Signalized Intersection

2.3.1 General

Signalized intersections typically form the capacity bottlenecks in urban road networks. Signal timing plans are developed in order to segregate potentially conflicting movements at a signalized intersection. Traffic signal controls are implemented for reducing or eliminating conflicts at intersections. Signals accomplish this by allocating green times among the various users at the intersections. Signal controls vary from simple methods, which determine the timing settings on a time-of-day/day-of-week basis, to complex algorithms, which calculate the green time allocation in real time based on traffic volumes. There are several basic timing parameters introduced before discussing current signal control systems. A cycle is the time required for one complete sequence of signal indications. A phase is the portion of a signal cycle allocated to any combination of one or more traffic movements simultaneously receiving the right of way. Each phase is divided into a number of discretely timed intervals, which is a portion of the signal cycle during which all the signal indications remain unchanged, such as green, yellow change, and all red clearance. The split is the percentage of a cycle length allocated to each phase in a signal sequence (Kell and Fullerton, 1991).The geometric layout of signalized junction is shown in Figure 2.7.

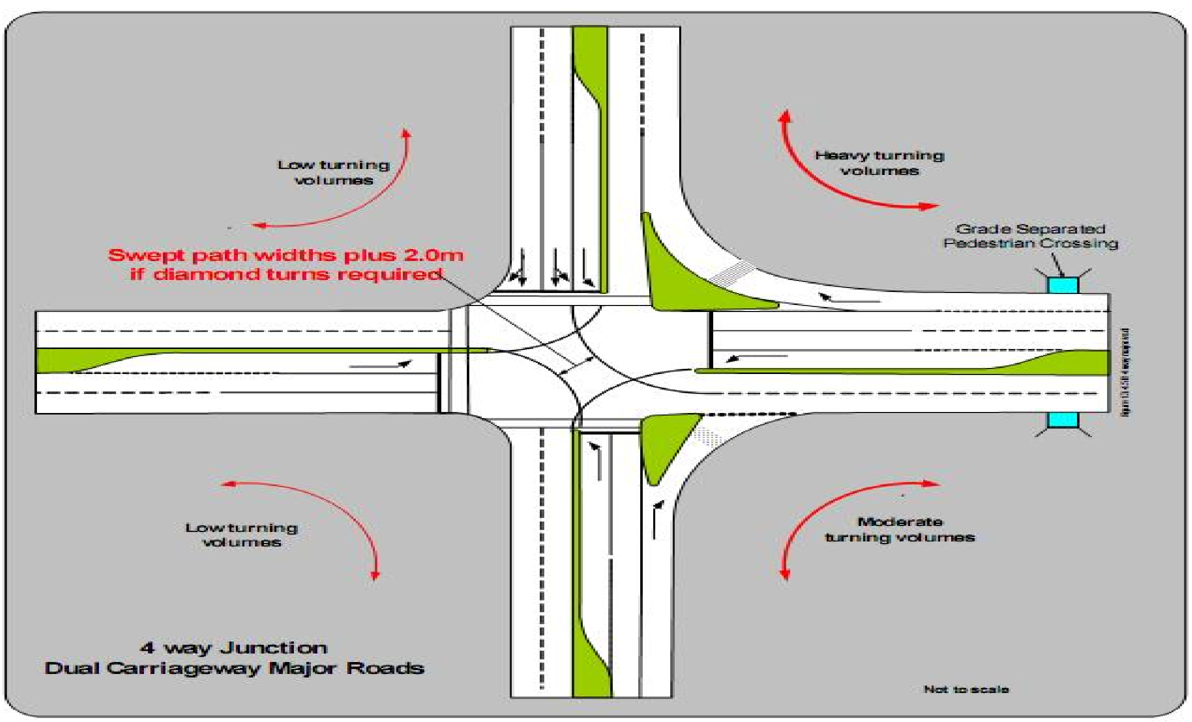


Figure 2.7-Geometric Layout of Signalised Intersection(AACRA Geometric Manual, 2004)

2.3.2 Pre-Timed Controls

The Pre-timed control, which has fixed cycle lengths and preset phase intervals, operates according to a predetermined schedule. The Pre-timed controllers are best suited for locations with predictable volumes and traffic patterns such as downtown areas. Timing plans are usually selected on a time-of-day-of-week basis by means of time clocks. Although pre-timed controllers have a degree of flexibility in varying timing plan, they can cause excessive delay to vehicles where there exists a high degree of variability in the traffic flows because pre-timed control does not recognize or accommodate short-term fluctuations in traffic demand and uses timing plans determined from historical demands.

Pre-timed signals assign the right of way to different traffic streams in accordance with a preset timing plan. The Webster method is used to determine the optimum cycle lengths. Since the actuated signals act as a fixed signal when all approaches are saturated, this method can be used to compute the cycle lengths and the green times for actuated traffic signals when the actuated controller operates as a Pre-timed signal (HCM, 2010).

2.3.3 Actuated Controls

An actuated signal operates with variable vehicular timing and phasing intervals that depend on traffic volumes. The signals are actuated by vehicular detectors placed in the roadways. The cycle lengths and green times of actuated control may vary from cycle to cycle in response to demands. Actuated controllers include semi-actuated, fully actuated, and density controllers (HCM, 2010).

2.3.4 Methods for Estimating Capacity of Signalized Intersection

Methods to analyze the performance of a given signal timing plan, and to develop optimal plans, have been developed since the 1950s and are now embedded in design manuals such as the Highway Capacity Manual (HCM) and Canadian Capacity Guide (Teply et al., 1995). Most of these methods are based on the pioneering work by Webster (1958) who developed an expression for average delay that captured delay from two sources, namely:

(1) Delay assuming deterministic arrival and deterministic service rate; and

(2) Delay assuming random (i.e. Poisson) arrivals and deterministic service rate.

Webster's original formulation was modified to permit application to oversaturated conditions and more recent modifications (e.g. HCM 2010) include a term to account for initial queues. In all of these delay expressions, the only source of randomness is in the distribution of headways in the arrival stream. The mean arrival rate is assumed to be constant over the analysis period.

2.3.5 Operational Measures of Effectiveness

Three measures of effectiveness are commonly used to evaluate signalized intersection operations and these:

- Capacity and volume-to-capacity ratio,
- Delay
- Queue

2.3.5.1 Capacity and Volume-to-Capacity Ratio

Capacity is represented by the maximum rate at which vehicles can pass through a given point in an hour under prevailing conditions; it is often estimated based on assumed values for saturation flow. Capacity accounts for roadway conditions such as the number and width of lanes, grades, and lane use allocations, as well as signalization conditions. Under the HCM 2010 procedure, intersection capacity is measured for critical lane groups (those lane groups that require the most amount of green time). Intersection volume-to-capacity ratios are based on critical lane groups; noncritical lane groups do not constrain the operations of a traffic signal. The v/c ratio, also referred to as degree of saturation, represents the sufficiency of an intersection to accommodate the vehicular demand. A v/c ratio less than 0.85 generally indicate that adequate capacity is available and vehicles are not expected to experience significant queues and delays. As the v/c ratio approaches 1.0, traffic flow may become unstable, and delay and queuing conditions may occur. Once the demand exceeds the capacity (a v/c ratio greater than 1.0), traffic flow is unstable and excessive delay and queuing is expected. Under these conditions, vehicles may require more than one signal cycle to pass through the intersection known as a cycle failure (Source FHWA publication, Signalized Intersections Information Guide, 2000).

2.3.5.2 Delay and Level of Service

Delay is defined in *HCM (2010)* as "the additional travel time experienced by a driver, passenger, or pedestrian." The signalized intersection chapter (chapter 16) of the HCM provides equations for calculating control delay as shown below, the delay a motorist experiences that is attributable to the presence of the traffic signal and conflicting traffic.

The control delay equation comprises three elements: uniform delay, incremental delay, and initial queue delay. The primary factors that affect control delay are lane group volume, lane group capacity, cycle length, and effective green time. Factors are provided that account for various conditions and elements, including signal controller type, upstream metering, and delay and queue effects from oversaturated conditions.

Intersection control delay is generally computed as a weighted average of the average control delay for all lane groups based on the amount of volume within each lane group and it is used as the basis for determining LOS. Caution should be exercised when evaluating an intersection based on a single value of control delay because this is likely to over- or under-represent operations for individual lane groups. Delay thresholds for the various LOS are given in Table 2.4.

Table 2.4-Motor Vehicle LOS Threshold Values at Signalized Intersections (HCM, 2010).

LOS	Control Delay per Vehicle (sec/ vehicle)
A	$d \leq 10$
B	$10 < d \leq 20$
C	$20 < d \leq 35$
D	$35 < d \leq 55$
E	$55 < d \leq 80$
F	$d > 80$

The delay expression, incorporated within the HCM (TRB, 2000), and used within this study to estimate delay, is given by;

$$d = d_{1*}(PF) + d_2 + d_3 \tag{2-16}$$

$$PF = ((1-P)*f_{PA})/(1-(g/C)) \tag{2-17}$$

$$d_1 = (0.5C\,(1-(g/C))^2)/(1-\min(1,X)(g/C)) \tag{2-18}$$

$$d_2 = 900T\ ((X-1) + \sqrt{((X-1)^2 + ((8klX)/(cT)))} \tag{2-19}$$

$d_3 = (1800Q_b(1+u)t)/(cT)$ (2-20)

Where:

c: Lane group capacity (vehicle per hour (vph)),

C: Cycle length (seconds),

d: Control delay per vehicle (seconds/veh),

d_1: Uniform control delay assuming uniform arrivals (seconds/veh),

d_2: Incremental delay to account for randomness (seconds/veh),

d_3: Initial queue delay (seconds/veh),

f_{PA}: Supplemental adjustment factor for platoon arriving during green,

g: Duration of green interval (seconds),

g/C: Proportion of green time available,

k: Incremental delay based on controller settings,

l: Upstream filtering / metering adjustment factor, and

PF: Progression adjustment factor

P: Proportion of vehicles arriving on green

T: Analysis period (hour),

X: Lane group v/c ratio or degree of saturation

Q_b: Initial queue at the start of period T (veh),

U: Delay parameter,

t: Duration of unmet demand in T (h).

As is evident from Equations 2 through 5, delay in the HCM is primarily a function of volume and capacity. Volume is typically the hourly flow rate associated with the peak 15-minutes (i.e. volume = peak hour volume/*PHF*). Capacity is a function of the signal timing (i.e. *g*/*C* ratio) and the saturation flow rate.

The model is adjusted for traffic-actuated control with factor k depending on unit extension and degree of saturation. For isolated Pre-timed signals k = 0.5 and I = 1.0. Control delay is used as the basis for determining LOS.

2.3.5.2 Vehicle Queue

Vehicle queuing is an important measure of effectiveness that should be evaluated as part of all analyses of signalized intersections. Estimates of vehicle queues are needed to determine the amount of storage required for turn lanes and to determine whether spillover occurs at upstream facilities (driveways, un-signalized intersections, signalized intersections, etc.). Approaches that experience extensive queues also are likely to experience an over-representation of rear-end collisions. Vehicle queues for design purposes are typically estimated based on the 95^{th} percentile queue that is expected during the design period.

The main objectives of the traffic engineer are to optimize the operation of the existing traffic systems, and solve traffic problems at such intersections. It is important to improve the effectiveness of the traffic control parameters in order to reduce congestion and to alleviate the problems that impede the traffic flow along any traffic facility. Therefore, an improvement to the different traffic elements must be considered to increase traffic efficiency and performance .These elements include phase sequences, geometric design elements, parking control, and travel demand management (TDM) actions.

2.3.6 Traffic Operations Elements

Signalized intersection operations are a function of three elements described in the following sections along with a discussion on their effects on operations.

- Traffic volume characteristics.
- Roadway geometry.
- Signal timing.

2.3.6.1 Traffic Volume Characteristics

The traffic characteristics used in an analysis can play a critical role in determining intersection treatments. Over conservative judgment may result in economic inefficiencies due to the construction of unnecessary treatments, while the failure to account for certain conditions (such as a peak recreational season) may result in facilities that are inadequate and experience failing conditions during certain periods of the year. An important element of developing an appropriate traffic profile is distinguishing between traffic demand and traffic volume. For an intersection, traffic demand represents the arrival pattern of vehicles, while traffic volume is generally measured based on vehicles' departure rate. For the case of overcapacity or constrained situations, the traffic volume may not reflect the true demand on an intersection. In these cases, the user should develop a demand profile. This can be achieved by measuring vehicle arrivals upstream of the over-capacity or constrained approach. The difference between arrivals and departures represents the vehicle demand that does not get served by the traffic signal. This volume should be accounted for in the traffic operations analysis.

Traffic volume at an intersection may also be less than the traffic demand due to an overcapacity condition at an upstream or downstream signal. When this occurs, the upstream or downstream facilities "starve" demand at the subject intersection. This effect is often best accounted for using a micro-simulation analysis tool. (Source FHWA publication, Signalized Intersections Information Guide, 2000).

2.3.6.2 Intersection Geometry

The geometric features of an intersection influence the service volume or amount of traffic that an intersection can process. A key measure used to establish the supply of an intersection is saturation flow, which is similar to capacity in that it represents the number of vehicles that traverse a point per hour. However, saturation flow is reported assuming the traffic signal is green during the entire hour. By knowing the saturation flow and signal timing for an intersection, one can calculate the capacity (capacity = saturation flow multiplied with the ratio of green time to cycle length). Saturation headway is determined by measuring the average time headway between successive vehicles that discharged from a stop queue at the start of green signal time, beginning with the fourth vehicle. Saturation headway is expressed in time (seconds) per vehicle. Saturation flow rate is simply determined by dividing the average saturation headway into the number of seconds in an hour, 3,600 seconds, to yield units of vehicles per hour. The HCM (2010*)* uses a default ideal saturation flow rate of 1,900 vehicles per hour. Ideal saturation flow assumes 3.6-m (12-ft)-wide travel lanes, through movements only, and no curbside impedances, pedestrians/bicyclists, grades, or central business district influences. The HCM (2010) provides adjustment factors for non-ideal conditions to estimate the prevailing saturation flow rate.

Saturation flow rate can vary in time and location. Saturation flow rates have been observed to range between 1,500 and 2,000 passenger cars per hour per lane. Given the variation that exists in saturation flow rates, local data should be collected where possible to increase the accuracy of the analysis.

Existing or planned intersection geometry should be evaluated to determine features that may impact operations and that require special consideration.

2.3.6.3 Signal Timing

The signal timing of an intersection also plays an important role in its operational performance. In this regard, key factors include:

Effective green time - Effective green time represents the amount of usable time available to serve vehicular movements during a phase of a cycle. It is equal to the displayed green time minus startup loss time plus end gain. The effective green time for each phase is generally determined based on the proportion of volume in the critical lane for that phase relative to the total critical volume of the intersection. If not enough green time is provided, vehicle queues will not be able to clear the intersection, and cycle failures will occur. If too much green time is provided, portions of the cycle will be unused resulting in inefficient operations and frustration for drivers on the adjacent approaches.

Clearance interval - The clearance interval represents the amount of time needed for vehicles to safely clear the intersection and includes the yellow change and red clearance intervals. The capacity effect of the clearance interval is dependent upon the loss time.

Loss time - Loss time represents the unused portion of a vehicle phase. Loss time occurs twice during a phase: at the beginning when vehicles are accelerating from a stopped position and at the end when vehicles decelerate in anticipation of the red indication. Longer loss times reduce the amount of effective green time available and thus reduce the capacity of the intersection. Wide intersections and intersections with skewed approaches or unusual geometrics typically experience greater loss times than conventional intersections.

Cycle length - Cycle length determines how frequently during the hour each movement is served. It is a direct input, in the case of pre-timed or coordinated signal systems running a common cycle length, or an output of vehicle actuations, minimum and maximum green settings, and clearance intervals. Cycle lengths that are too short do not provide adequate green time for all phases and result in cycle failures. Longer cycle lengths result in increased delay and queues for all users.

Progression - Progression is the movement of vehicle platoons from one signalized intersection to the next. A well-progressed or well-coordinated system moves platoons of vehicles so that they arrive during the green phase of the downstream intersection. When this occurs, fewer vehicles arrive on red, and vehicle delay and queues are minimized. A poorly coordinated system moves platoons such that vehicles arrive on red, which increases the delay and queues for those movements beyond what would be experienced if random arrivals occurred. (Source FHWA publication, Signalized Intersections Information Guide, 2000).

2.4 Traffic Composition

Traffic composition needs to be considered when determining capacities for both roundabout and signalized intersections. The increase of heavy vehicles will reduce the capacities of the intersection due to their slow follow-up headways and increased size. Akçelik (1997) recommended that passenger car equivalents (pce) per hour be used instead of vehicles per hour when the proportion of heavy vehicles is greater than 5%. Passenger car equivalents allow heavy vehicles to resemble a standard passenger vehicle to better represent the capacity of an intersection.

Typically, the passenger car equivalent of a heavy vehicle is taken as 2.0 and vehicles like bus, track and trailer were considered as heavy vehicles. The Transport Research Board HCM suggests that the conversion factors for passenger car equivalents shown in Table 2.5 be used:

Table 2.5 – Conversion factors for passenger car equivalents (pce) (HCM 2010,)

Vehicle Type	**Passenger Car Equivalent (PCE)**
Car	1.0
Heavy Vehicle	2.0
Bicycle	0.5

2.5 Summary of Literature Review

- Based on the literature reviewed, different countries have their own methods of Capacity Analysis for roundabouts, as forwarded by different researchers. However, we can categorize them into two: (i) Empirical method that correlates geometric features and performance measures, such as capacity, average delay and queue length, through the regression of field data; (ii) Analytical *method* that calculates the roundabout capacity as a function of critical gap, the follow-up time and the circulating flow. The Sidra Intersection software does not apply purely analytical method to analyze the roundabout capacity. It is semi- Empirical-Analytical approach that uses some geometric elements for the analyses.

  Since the Empirical method totally depends on the geometric elements of roundabouts, it is sometimes difficult to find the necessary geometric features (elements) on Addis Ababa roundabouts because most of the roundabouts were constructed many years ago and do not fulfill modern roundabout geometry , which may be a problem during evaluation. Hence, for Addis Ababa the Analytical Method is preferable to Empirical Method since it includes the traffic environment besides geometry of roundabout. Therefore, the Analytic Method is preferred for this research using the Sidra Intersection software version 5.1 with some geometric elements. In fact, AACRA also recommend Sidra Intersection software for capacity analysis, which is developed by using Analytic Method with some geometric elements.
- Depending on driving (Traffic) rules and Geometric features, it is possible to distinguish the rotary and traffic circles from modern roundabouts. The driving or traffic rules for modern roundabouts are priority to circulating vehicles; no pedestrian access, no parking and one direction circulation. The geometric features are yield line, approach flare, deflection and splitter island (not for small roundabouts).
- Evaluation of signalized intersection operations is commonly measured by Volume-to-capacity Ratio, Delay and Queue length.
- The level of service (LOS) criteria for signalized intersection was according to HCM-2010 and determined using the widely used Sidra Intersection software version 5.1.

3. METHODOLOGY

3.1 Research Approach

The research approach in this thesis involves quantitative data and analysis using the SIDRA Intersection software model to determine the level of service of intersections. To do this, primary data were collected directly through field surveys of the selected junctions and subsequent analysis of the data. This helps to generate inductive conclusion on the Level of Service (LOS) of considered junctions in Addis Ababa. Since it is impossible to assess the Level of Service of all intersections in the City due to the limitation of budget and time, representative samples were taken at different locations of the City to derive a generalized conclusion. The LOS criterion was according to HCM-2010 and determined using the widely used Sidra Intersection software version 5.1.The flow chart in Figure 3.1 shows the overall activities carried out during the determination of the level of service of the junctions.

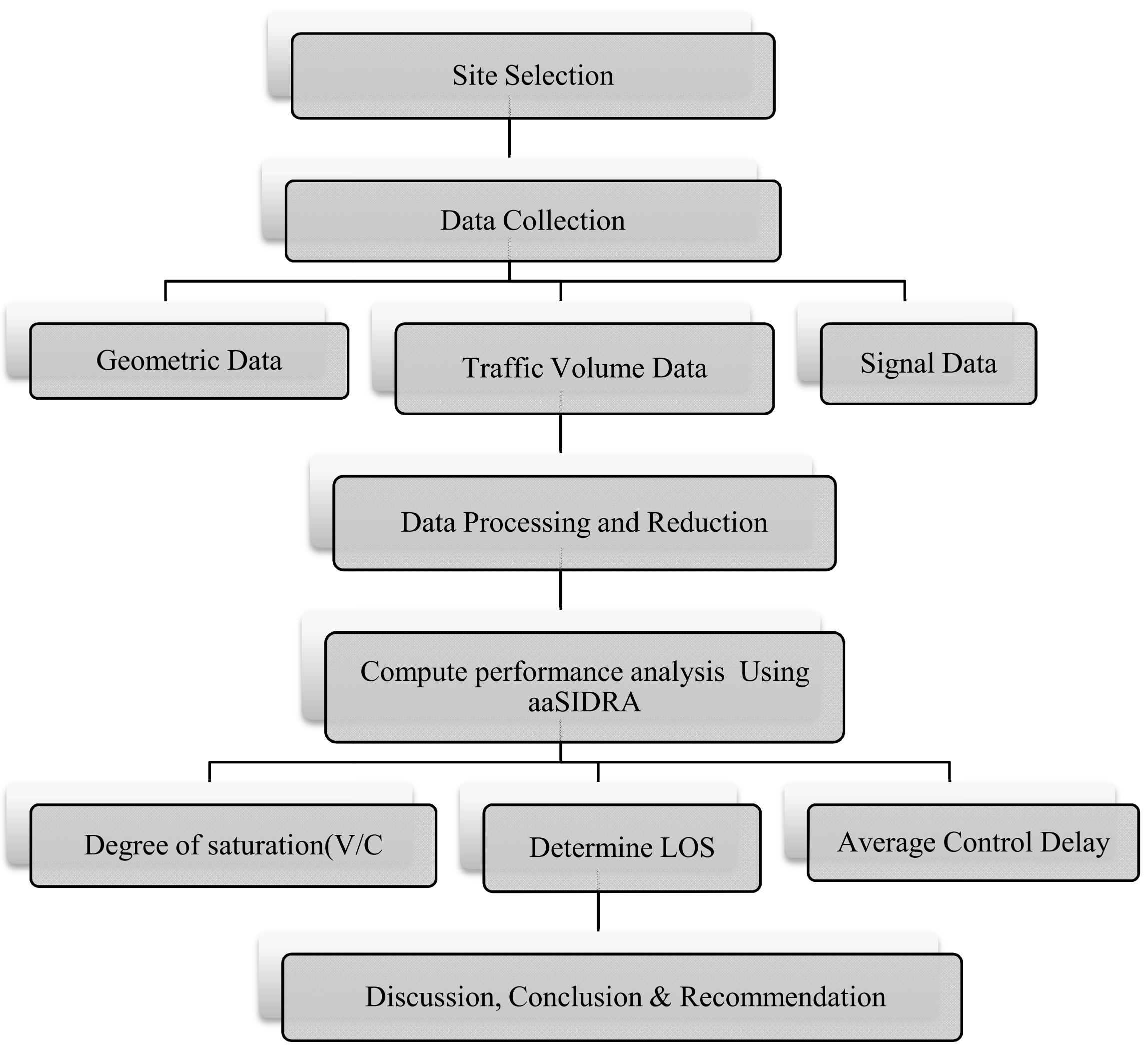


Figure 3.1- Study Flow Chart

3.2 Calibration of Sidra Intersection Software

Since the Sidra Intersection was developed for Australian conditions as default; it requires calibration for other countries. Calibration of the Sidra Intersection is performed by changing values of the parameters affecting capacity. This can be done in a few different ways, either by changing the value of the critical gap and the follow-up headway directly or by using the calibration parameters; environment factor and entry/circulating flow adjustment. The environment factor can be seen as a collection factor that includes everything at the Junction environment e.g. design type, visibility, grade, speed, driver response time and aggressiveness, amount of heavy vehicles and pedestrians and parking near the Junction. On the one hand, factors in the environment with positive effects on traffic are for example; good visibility, small volumes of pedestrians, short driver response times, and low levels of heavy vehicles and parking on the approaches. In cases like that, environment factor should be lower which leads to higher capacity. On the other hand, situations such as bad visibility, large volumes of pedestrians, long driver response times and large volumes of heavy vehicles have negative effects on capacity. Environment factor should therefore be higher which will lead to lower capacity.

The default environment factor is set to 1, which is also the same in Australia. According to Myre (2010) studies in Norway have shown that 1.1 is a good value of environment factor for Norwegian conditions. Similarly, the Anna-Karin Ekman studies in Sewden have shown that 1.1 is also a good value of environmental factor for Swedish condition and he also suggested that the range of interesting values should be within 1.0 ± 0.2. The HCM version of the SIDRA Intersection model uses 1.2 as environment factor.

However, calibration with environment factor can be used to make general conclusions about the model for an area (state, country etc.); there can be problems with using one specific value for a whole country; for example, regional difference in driver behavior and weather conditions. Hence, the calibration process would be interesting to do more extensive research in to the subject for the specific case of Addis Ababa. However; in the case of this research an environment factor of 1.1 and entry/circulating flow adjustment Medium ware applied considering practices in different and drivers' behavior in Addis Ababa. This is because, most of the drivers in Addis Ababa are aggressive and do not obey modern traffic rules and some of their behaviors are manifested through parking on turnings, neglect traffic light at red indication, wrong signing etc. Table 3.1 shows.

Table-3.1 Default Values used in this research

Parameters	Default Values
Critical Gap	4.1 sec
Follow up headway	2.6 sec
Basic Saturation Flow	1950 ve/hr
Lane utilization	100%

4. DATA COLLECTION AND ANALYSIS

4.1 Data Collection

In order to assess the performance of a selected study area, field observations including traffic volumes, geometric data and other conditions have to be collected. Accordingly the measurements were taken manually on workdays, in which the highest congestion and inefficient use of transportation system occur at peak hours.

4.2 Study Area

Addis Ababa is the capital City of Ethiopia, which is located within the horn of Africa with geographical coordinates of $9^o1'48''$ North and $38^o44'24''$ East and with an average elevation of 2355m above sea level. The City has a total area of about 530.14 km^2 and a population of 2,738, 248 according to 2007 census. The City is divided into 10 administrative sub-cities and 99 Kebeles and it is the most important business and commercial center of the country. The rapid increase of the Addis Ababa population (according to 2007 Census Report the annual growth rate for 2007 was 2.1% and as estimated the population will be about 5 million by 2020) is the main cause of the increasing demand for transportation and mobility. This may create major operational problems, especially during the peak periods.

In order to evaluate the capacity of roundabouts and signalized intersections in Addis Ababa, four roundabouts and two signalized intersections are selected as illustrated in Table 6, the intersections are selected from major road corridors which represent significant traffic activities in Addis Ababa. According to the final report of urban transport study for Addis Ababa City, four major corridors are defined within the road net work of the City. These are; the East-west Axis or corridor, the North-South Axis or corridor, the Ring road corridor and the (CBD) orbit. Therefore, the selected intersections for this research connect major road corridors, the ring road and the Central Business District (CBD) of the city road net work. There are many location of attraction close to the study area, such as; market centers, cinemas, churches and residential areas.

Table 4.1- Selected Study Junctions

Intersection Type	Intersection Name	No. of approaches	Location
Roundabout	Gerji-Imperial	4	On Eastern Ring road
	Bole-Medanialem	5	In CBD Orbit
	Teklehaymanot	4	In CBD Orbit
	Ayer-Tena	4	Along East-West Axis
Signalized	Legehar	4	Along East-West Axis
	Post-Office	4	In CBD Orbit

The chosen intersection names were adopted from the locality or publically declared names by the Government.

The locations of the junctions contribute to high traffic volume and congestion due to the limited flow capacity. Hence, the intersections were selected as the subject of this research study. The Google Earth Image of these intersections, taken from Google Earth, can be seen below from Figure 4.1 to Figure 4.6

Figure 4.1- Gerji Imperial Roundabout

Figure 4.2- Bole-Medihanialem Roundabout

Figure 4.3- Teklehaymanot Roundabout

Figure 4.4- Ayer-Tena Roundabout

Figure 4.5- Post-office Signalized Intersection

Figure 4.6- Legahar Signalized Intersection

4.3 Traffic Data

The traffic count surveys were carried out at all selected junctions for four hours; in the morning 7:00 am-9:00 am and in the afternoon 4:00 pm-6:00 pm at 15 minute intervals during working days by setting up video cameras. The peak hour was determined by finding the four consecutive 15-minute periods with the highest total volume. The highest traffic volume in each direction was recorded for use in the analysis of this research. Traffic volumes can vary greatly throughout the day, by day of week, by time of year, and even at 5-minute intervals during the peak hour. Traffic volumes can also experience additional fluctuations due to accidents, special incidents, or weather and will also change over time depending on the growth dynamism of the City. However; normally, traffic surveys are performed using the average weekday peak hour traffic counts. On normal commuter routes, there are morning and afternoon peak hours.

High pedestrian volume also has a significant effect on capacity. Because of this numbers of pedestrian were recorded at peak hours along the direction of their movements. The number of counted vehicles and pedestrians are shown in Table 4.2. For detailed information on the vehicles and pedestrian counts, please see Appendix A. The surveys were carried out during the month of April, 2013.

4.3.1 Video With Manual Transcription Method

Video recording and manual transcription were used to collect traffic and pedestrian flow data. This method of data collection relies on video cameras to collect or capture the traffic and pedestrian flows in the field and also data recorders in other. According to travel time collection handbook; though costly, Video capturing techniques is preferred to manual collection. This was because of the following reasons:

- ✓ It provides a permanent, easily-review record and shows traffic conditions at any time;
- ✓ It permits reading of required parameters in a controlled environment in which plate characters can be closely examined;
- ✓ It provides additional information about traffic flow characteristics such as traffic volume and vehicle headways; and
- ✓ It provides time for accurate determination of arrival times and has better accuracy than manual counts.

Therefore, in order to utilize the above advantages and due to its convenience, video camera was arranged at convenient height where maximum view could be captured and visibility was also maximized. The locations of video capturing were on roofs & floors of high-rising buildings at the vicinity of the study locations as shown in Figure 4.7 below.

Figure 4.7- Image During Survey Time by The Researcher

The traffic volume data at the Junctions was summarized for all vehicles types as shown in Table 4.2 below.

Table 4.2- Hourly Traffic Volumes (PCU) at the Junctions during the time period of survey

Time	Gerji-Imperial	Bole-Medihanialem	Teklehay-manot	Ayertena	Post-Office	Lege-har
7:00-8:00 AM	4800	2494	3793	4205	2768	3561
Left	1200	624	948	1051	692	890
Through	2880	1496	2276	2523	1661	2137
Right	720	374	569	631	415	534
8:00-9:00 AM	5067	2787	4005	4593	2930	4608
Left	1267	697	1001	1148	733	1152
Through	3040	1672	2403	2756	1758	2765
Right	760	418	601	489	439	691
4:00-5:00 PM	4912	2601	3793	4191	2363	4188
Left	1228	650	948	1047	591	1047
Through	2947	1561	2276	2515	1418	2513
Right	737	390	569	629	354	628
5:00-6:00 PM	5333	3467	4507	5088	4028	4855
Left	1333	867	1127	1272	1008	1213
Through	3200	2080	2704	3053	2416	2912
Right	800	520	676	763	604	730

The traffic counts shown in Table 4.2 help to specify the peak hour period. The peak hour at all intersection were found to be between 5:00-6:00p.m and Figures 4.8 show the peak hour during the time period of survey at each intersection (for detail see appendix –A).

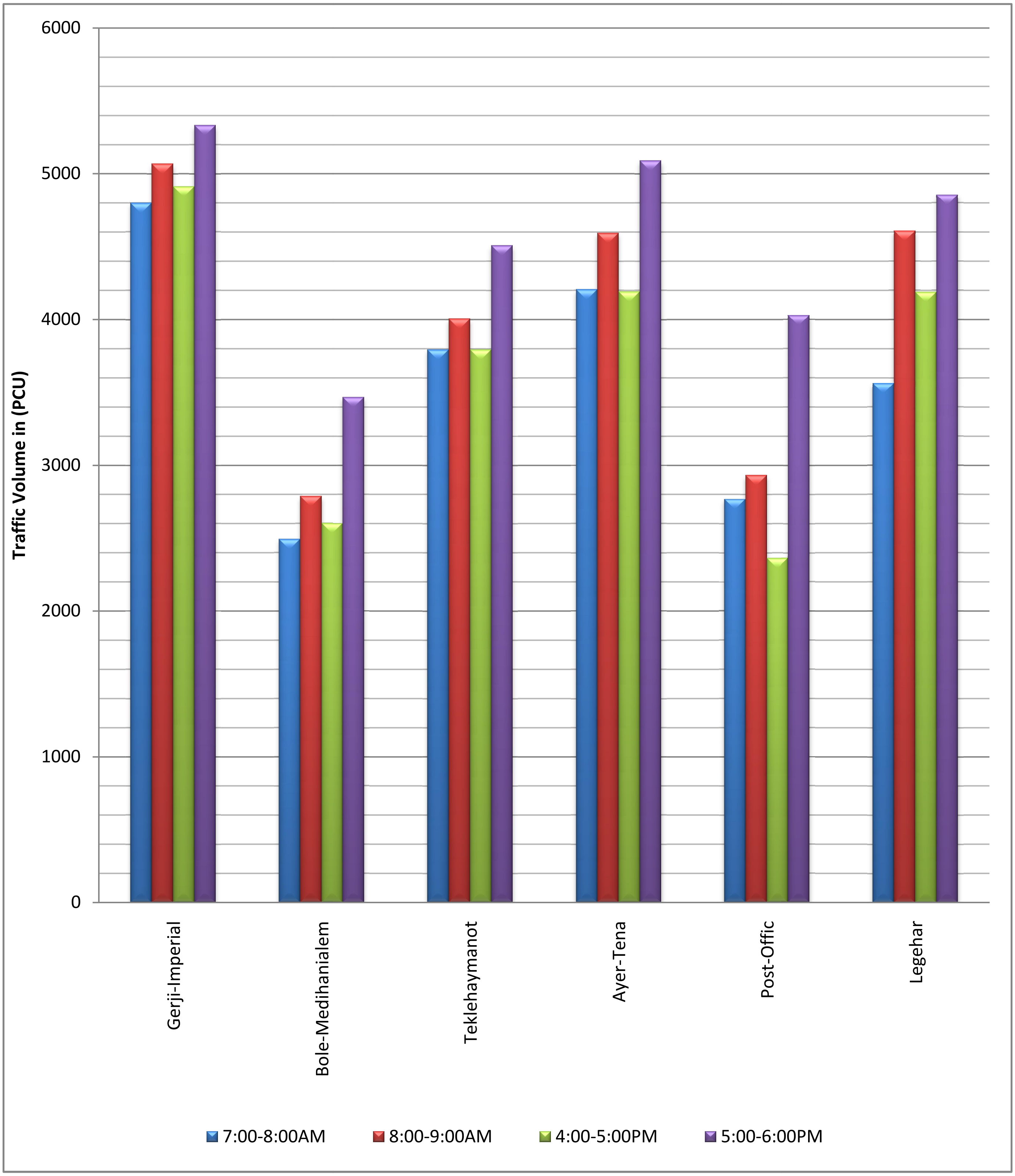


Figure 4.8- Hourly traffic volumes (PCU) during the period of survey at each intersection.

Table 4.3- Vehicles and pedestrians volume at intersections at peak hour (5:00 to 6:00 pm).

S.N	Junction Name	Heavy Vehicles			Light Vehicles	Total No. of Vehicles	Total Traffic (PCU)	% of Heavy Vehicles	Number of Pedestrians
		Bus & Dump Truck	Truck & Trailer	Total					
1	Gerji-Imperial	548	40	588	4370	4745	5333	5.5	2466
2	Bole-Medihanalem	92	0	92	2908	2954	3467	1.3	1413
3	Teklehymanot	352	14	366	3656	4141	4507	4.4	2488
4	Ayer-Tena	832	60	892	3640	5008	5088	8.3	2490
5	Post-Office	187	0	187	2715	3840	4028	3	1310
6	Legehar	248	16	264	4623	4589	4855	2	2532

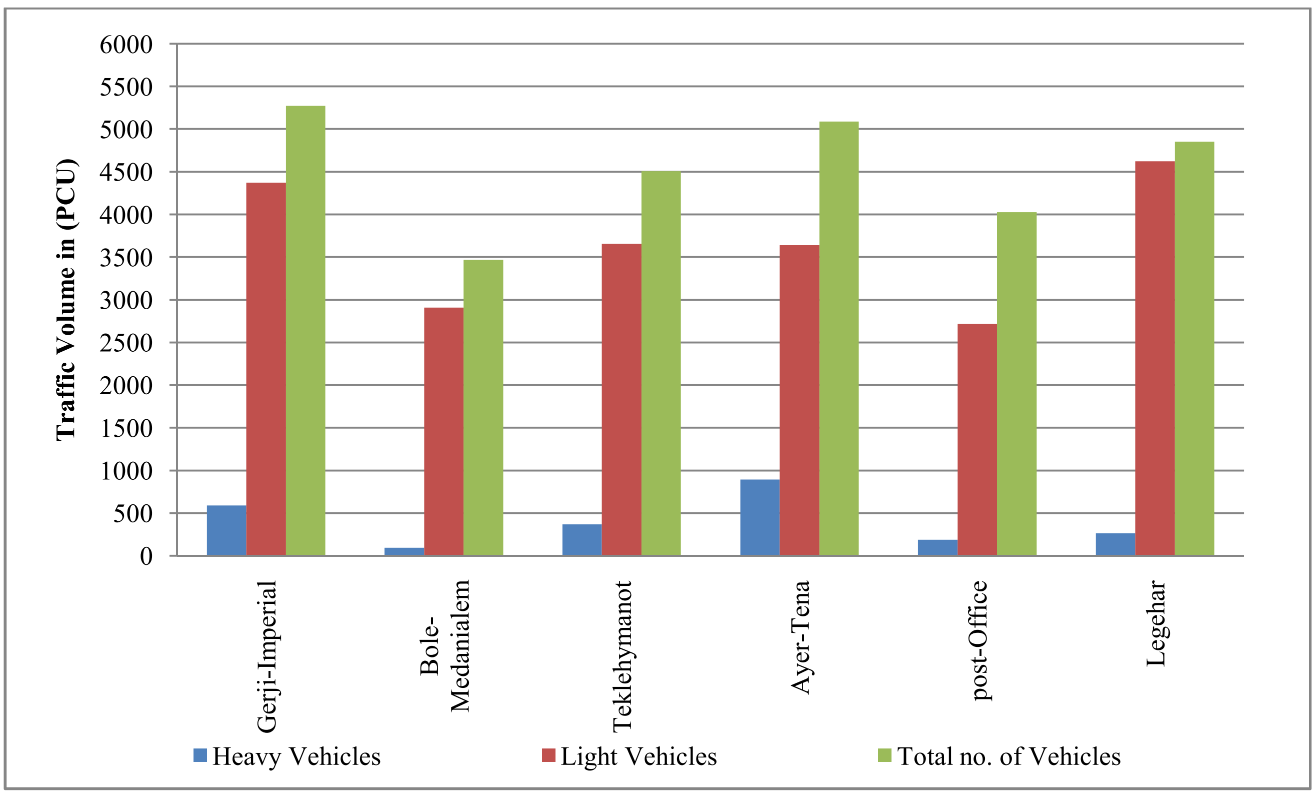


Figure 4.9- Maximum Peak Hour Vehicle Volumes Distribution at Intersections

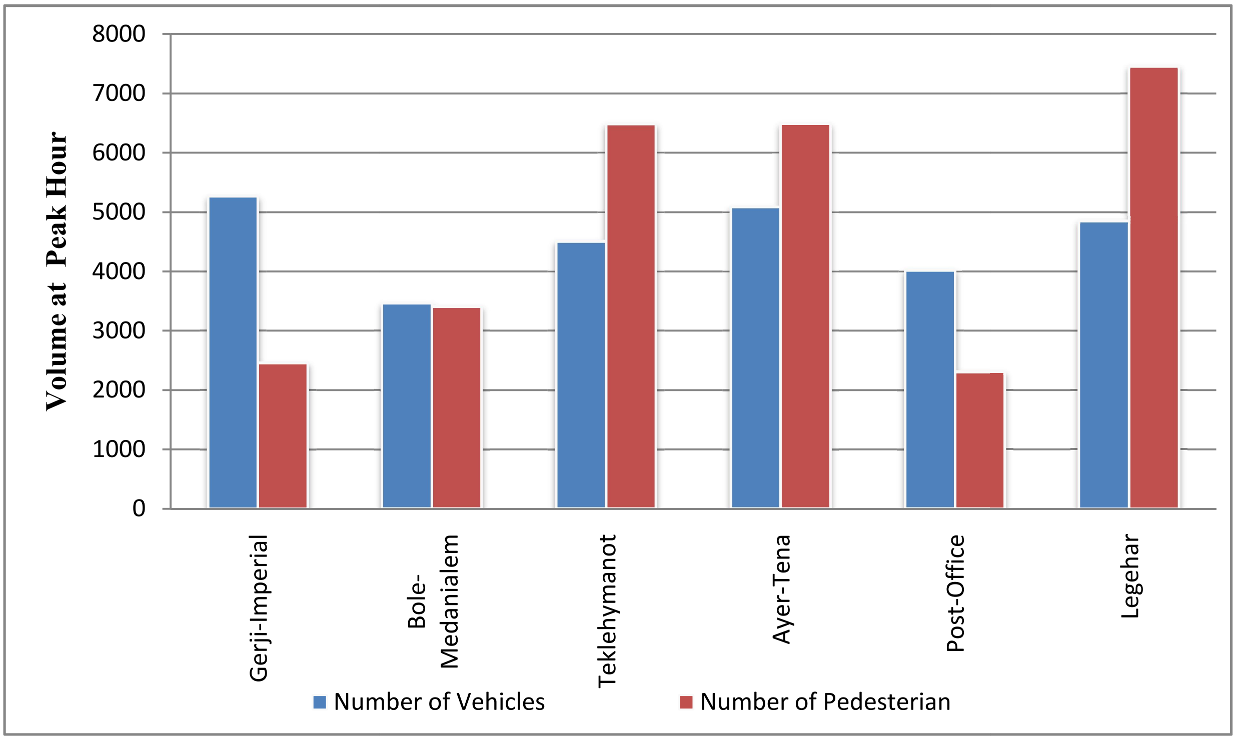


Figure 4.10-Maximum Peak Hour Vehicle Volumes vs Pedestrian Numbers at Junctions.

Figure 4.9 and Figure 4.10 clearly show the maximum and minimum numbers of vehicle and pedestrian traffic at surveyed junctions. For the most part when there is increased traffic volume, there are more pedestrians. The reason for this can mostly be attributed to land use. The maximum numbers of vehicles and pedestrians traffic exist at Ayertena and Teklehaymanot that are located at the major road corridor, central business district and residential part of Addis Ababa. The minimum numbers of vehicle and pedestrian traffic were counted at Post-Office signalized junction where there are less residential areas and commercial activities.

The percentages of heavy vehicles at the majority of the intersections do not exceed 10% as shown in Table 4.3. This is because; heavy vehicles are not allowed to inter the city at the data collection times of the day. However; Ayer-Tena and Gergi-Imperial roundabouts have 8% and 5.5% of heavy vehicles respectively, out of the total number of vehicles counted at those junctions. These junctions are a link between ring road and a collector road; mostly, heavy vehicles travel along the ring road.

Table 4.4 Entry Peak Hour Traffic Flow & Percentage Share on Each Leg.

No	Junction Name	Approach Leg	Entry Traffic on Leg (PCU)	% of Traffic Share
1	Gerji-Imperial	Bole-Airport	1932	34.9
		Gerji-Imperial	897	17.2
		Megenagna	1836	35.1
		Hayahulet	668	12.8
2	Bole-Medihanialem	Kazanches	1016	28.5
		Friendship	276	9.1
		Bole Airport	882	27.3
		Gipson Academy	395	12.1
		Hayahulet	899	23.0
3	Teklehaymanot	Merkato	1675	32.3
		Abinet	352	16.5
		Black Lion	1673	31.9
		Piazza	808	19.4
4	Ayertena	Alemgena	1444	28.2
		Jomo	1148	25.6
		Mexico	1557	28.0
		Alembank	940	18.1
5	Legehar	Mexico	1775	36.6
		Meskel Sq.	1907	39.2
		Cherkos	620	12.7
		Piazza	551	11.4
6	Post-office(near Immigration)	Black Lion	524	18.5
		Legehar	882	31.1
		Filwuha	475	16.7
		Piazza	958	33.7

4.4 Geometric Data

As per the requirement of the SIDRA Intersection Version 5.1, the collected geometric data include; island diameter, circulatory width, number of circulatory lanes, entry lane number and average lane width at entry for roundabout junction; and number of lanes, lane width, configurations of lanes, grade, width of median for signalized junction. These data are measured with a tape meter and the collected geometric data are summarized as shown in Table 4.5. The geometry features of considered junctions are also shown in Appendix-B

Table 4.5- Summary of Existing Roundabouts & Signalized Junctions Geometry Data

S.N	Junction Name	Approach Leg	Entry Lane	Number of Circulatory Lane	Island Diameter (m)	Average lane width	Circulatory Road width (m)
1	Gerji-Imperial	Bole-Airport	3	2	21	3.2	12
		Gerji	2	2	21	3.6	12
		Megenagna	3	2	21	3.2	12
		Hayahulet	2	2	21	3.6	12
2	Bole-Medihanialem	Kazanches	2	2	36	3.3	11
		Friendship	1	2	36	3.2	11
		Bole-Airport	3	2	36	3.3	11
		Gipeson Acadamy	1	2	36	3.2	11
		Hayahulet	2	2	36	3.2	11
3	Teklehymanot	Merkato	2	2	65	3.2	10
		Abinet	2	2	65	3.6	10
		Black Lion	2	2	65	3.6	10
		Piazza	2	2	65	3.2	10
4	Ayer-Tena	Alemgena	2	2	50	3.2	10
		Jomo	2	2	60	3.2	10
		Mexico	2	2	50	3.2	10
		Alembank	2	2	60	3.6	10

Signalized Intersection Geometric Data					
	Junction Name	Approach name	No.of Entry Lane	No. of Exit Lane	Median width
1	Legehar	Mexico	3	3	1.5
		Meskel Square	3	3	NA
		Cherkos	3	3	1.5
		Piazza	3	3	1.5
2	Post-office(near Immigration)	Filwuha	3	2	NA
		Piazza	3	3	1.5
		Legahar	3	3	1.5
		Black Lion	2	2	NA

From the summarized geometric data Table 4.5 we can see that the island diameter of the roundabouts ranges from 15m to 75m.When we add their circulatory width, the range becomes 26m to 99m,which can be categorized from mini-roundabouts to urban multilane-roundabouts according to Roundabout Information Guide 2000.

4.5 Methods of Analysis

The methodology employed in the research work was the critical aspect for ensuring the proper result which aligns with the objective. Hence, this part of the Thesis discusses the methodology applied to address the research problem and software programs that are available to analyze traffic operations at the roundabouts and signalized intersections.

4.5.1 Methods of Analysis for Roundabout Junction

As noted in the Highway Capacity Manual (Transportation Research Board, 2000), intersection analysis models can be classified into two types: empirical and analytical models. Empirical models use observations at many different intersections under all types of conditions to develop regression equations that match intersection characteristics with intersection capacity. Analytical models estimate capacity based on gap-acceptance relationships that do not require observations under congested conditions. Since the Empirical Method totally depends on the geometric elements of the roundabout, it is sometimes difficult to find the necessary geometric features (elements) on Addis Ababa roundabouts. In this regard, the Analytical Method is more realistic than Empirical Method since it considers the traffic environment. Therefore, the Analytic Method is utilized in this research. The basis of determining the capacity of a roundabout can be seen from the HCM (2010) shown in Figure 4.11.

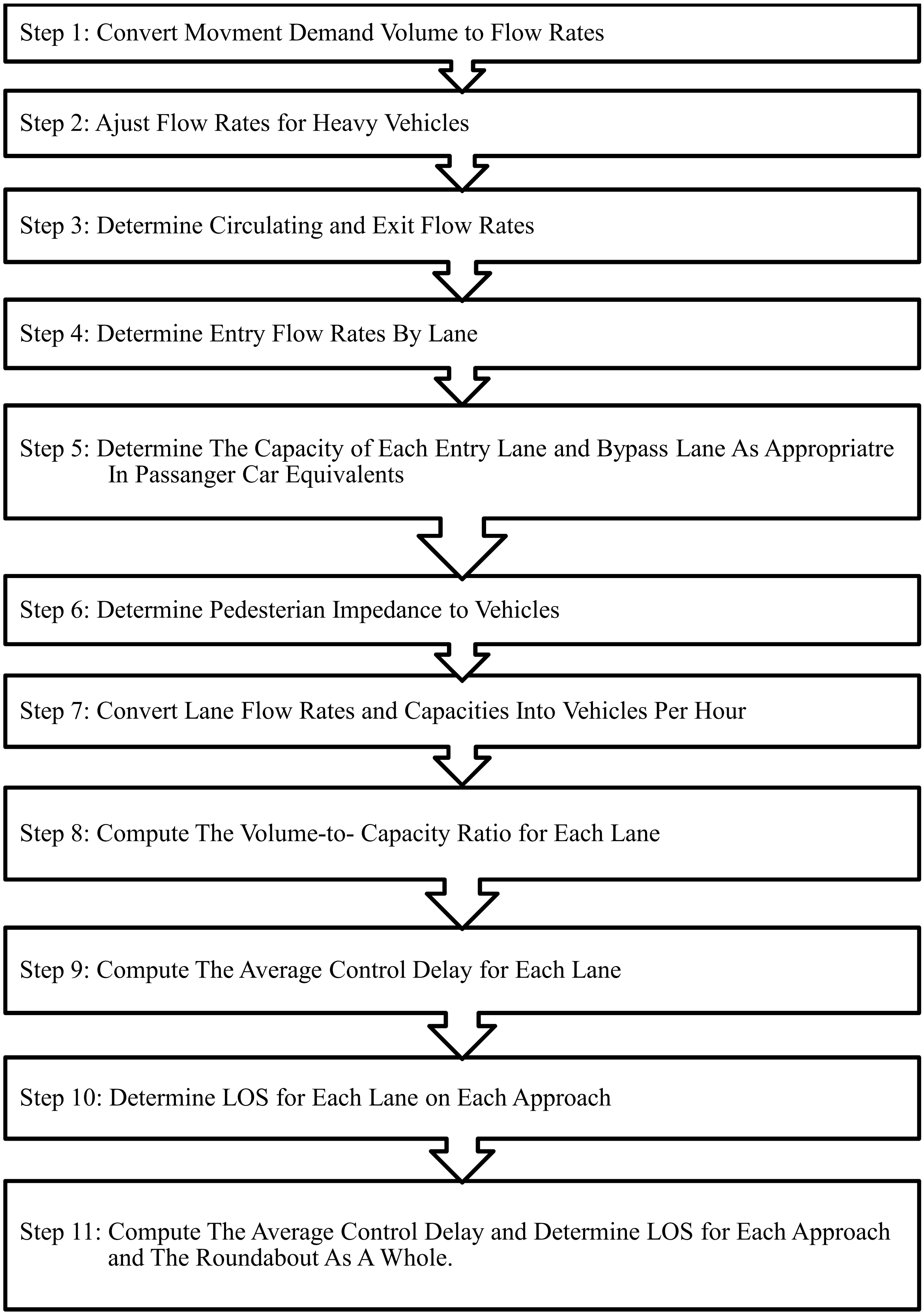


Figure 4.11 – Roundabout Analysis Methodology (HCM 2010)

4.5.2 Methods of Analysis for Signalized Intersection

Highway Capacity Manual (HCM 2010) is the most widely adopted method for analysis of signalized intersections. The HCM defines signalized intersection performance in terms of average vehicle delay (seconds per vehicle) and then maps this delay against predefined boundaries to define intersection performance in terms of six levels of service (i.e. LOS A through LOS F). Figure 4.12 represents the methodology determined from the HCM (2010) of calculating level of service for a signalized intersection.

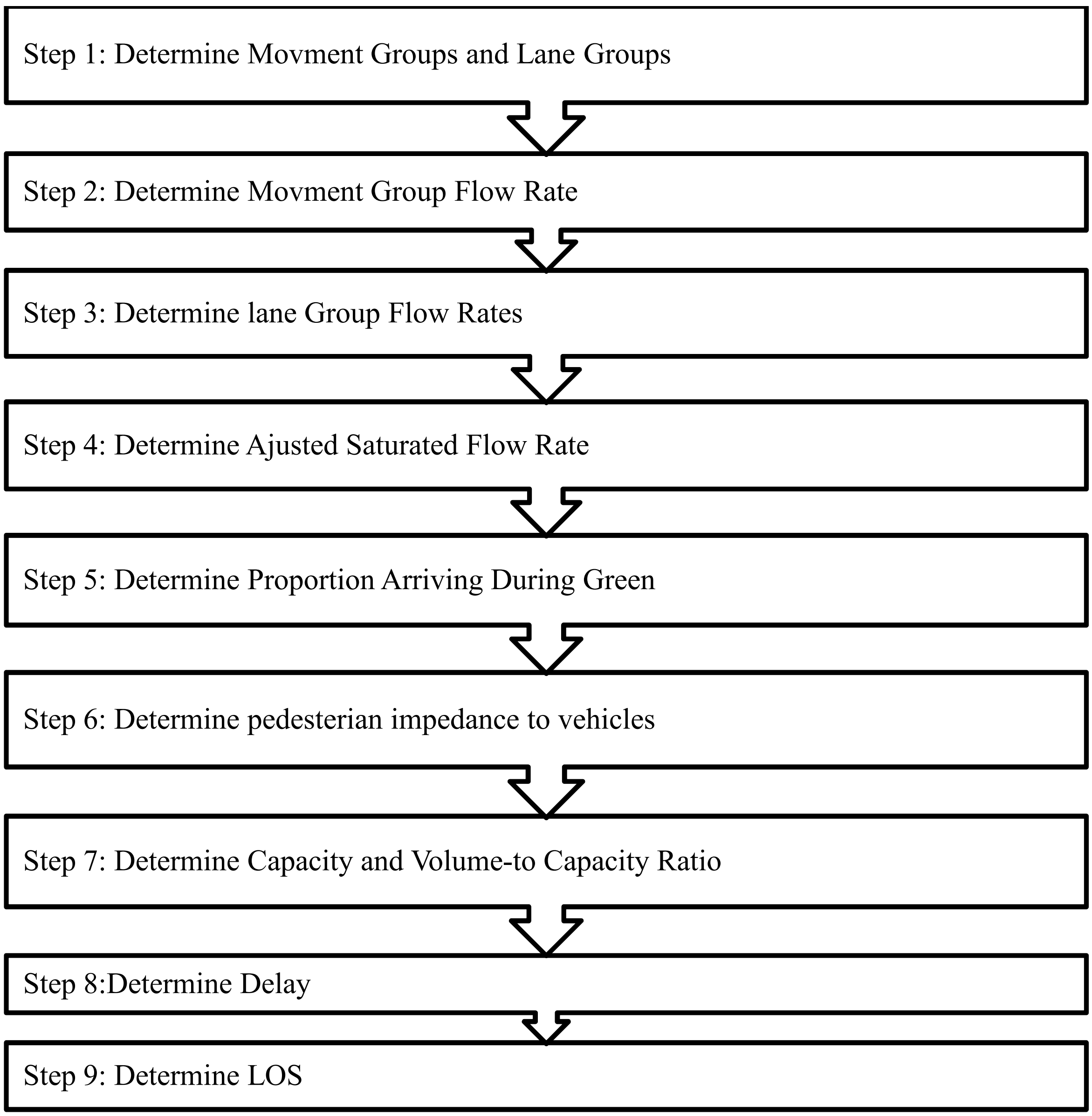


Figure 4.12 – Pretimed Signalized Intersection Analysis Methodology (HCM 2010)

4.5.3 Analysis Software

The FHWA Guideline lists available computer Software programs that analyze traffic operations at roundabouts and signalized intersections. The Software can be divided into two types: macroscopic and microscopic models. The Macroscopic Models use traffic volume flows to model intersections as isolated locations. On the other hand, the Microscopic Models simulate the movement of individual vehicles thereby allowing a network-wide analysis. For this study, one of the macroscopic models (SIDRA) software program was applied to analyze traffic operations at roundabouts and signalized intersections. In fact, AACRA also recommends Sidra Intersection software for capacity analysis, which is developed by using analytic method with some geometric elements.

The SIDRA Intersection software is preferred for capacity analysis in this research due to the following reasons:

- ✓ It is commercially available tool to offer full geometric and gap acceptance modeling capability within a single product;
- ✓ It has employed a combined (hybrid) geometry and gap-acceptance modeling approach in order to take into account the effect of roundabout geometry on driver behavior directly through gap-acceptance modeling; and
- ✓ It can be calibrated for local conditions and it is highly flexible.

5. RESULTS AND DISCUSSIONS

The results of analysis were achieved by using all input data and following analysis procedures described above with the aid of software models (SIDRA version 5.1) for all selected roundabouts and signalized intersections described below.

5.1 Roundabout Junctions

5.1.1 Gerji Imperial Roundabout

Geometric Data

1) Number of approaches or legs - 4
2) Number of circulating lane - 2
3) Inscribed circle diameter - 33m
4) Central island diameter -21m
5) Truck apron width - 0

Table 5.1-Geometric data for Gerji Imperial Roundabout

	Bole- Airport Leg	Gerji Leg	Megenagna Leg	Hayahulet Leg
Number of entry lane	3	2	3	2
Average Lane width (m)	3.2	3.6	3.2	3.6

Table 5.2-Traffic volume data for Gerji-Imperial Roundabout

Junction Name	**Approach Leg**	**Entry Traffic on leg (PCU)**	**% of traffic share**
Gerji-Imperial	Bole	1930	36.2
	Gerji	899	16.8
	Megenagna	1835	34.4
	Hayahulet	669	12.6
	Total	5333	

An excel program is used to analyze the traffic count shown in Table 4.2 to specify the peak hour. From site investigation and traffic count, the following were observed:

a) The peak hour at Gerji roundabout is found to be between 5:00-6:00 p.m. The total traffic volume during this hour was 5333. Figures 5.1 shows the peak hour traffic during the time of the survey at the Junction.

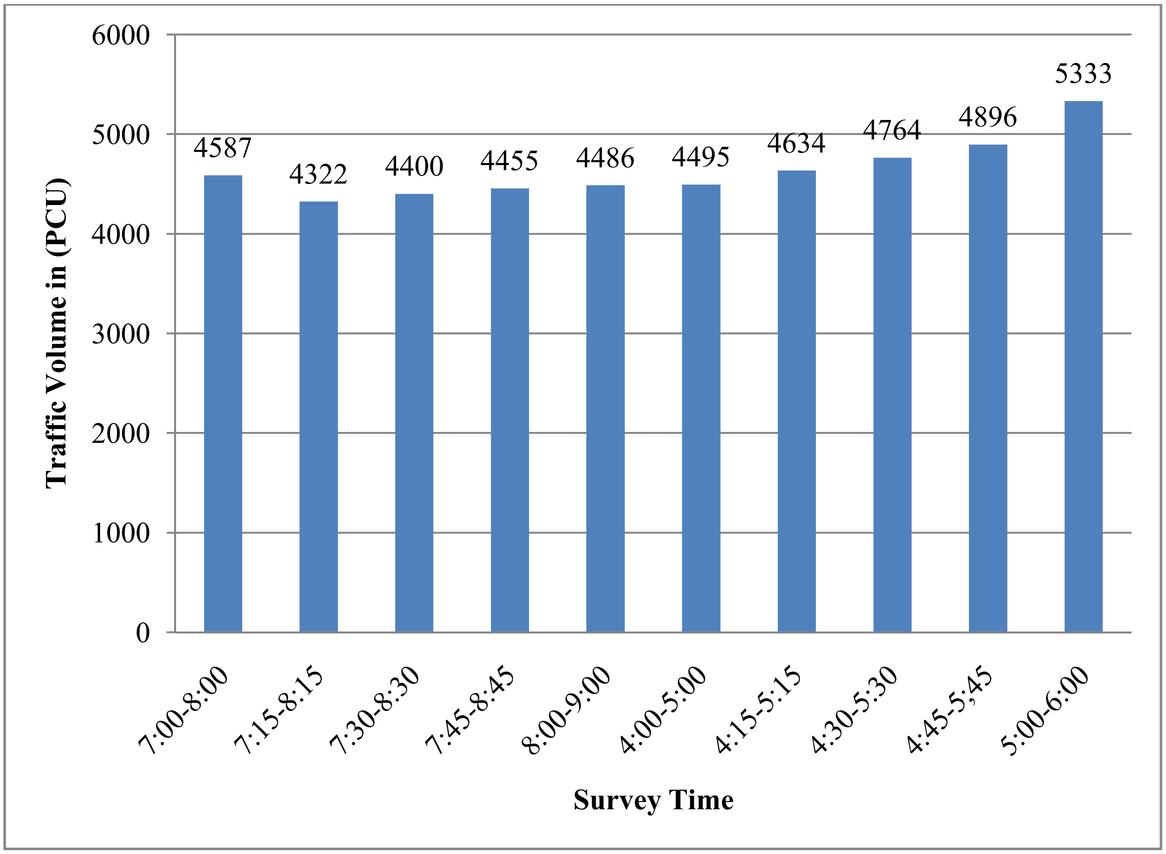


Figure 5.1- Traffic Volume at Gerji Imperial Roundabout counted within 15 minutes Time Interval

b) The percentage of heavy vehicles at Gerji Imperial roundabout was concentrated on the approaches that come from Bole-Airport and Megenagna, as shown in Table 5.3.

Table 5.3- Percentage of Heavy Vehicle for All Approaches at Gerji Imperial Roundabout

Approach	Percentage of Heavy Vehicle at Peak Hour
Bole-Airport	5.6
Gerji	1.8
Megenagna	4.1
Hayahulet	2.8

c) The PHF is defined as the ratio of total volume to the maximum 15 min rate of flow within the hour. Table 5.4 depicts PHF values for all approaches at Gerji-Imperial Junction.

Table 5.4 PHF Values for All Approaches at Gerji Imperial Roundabout

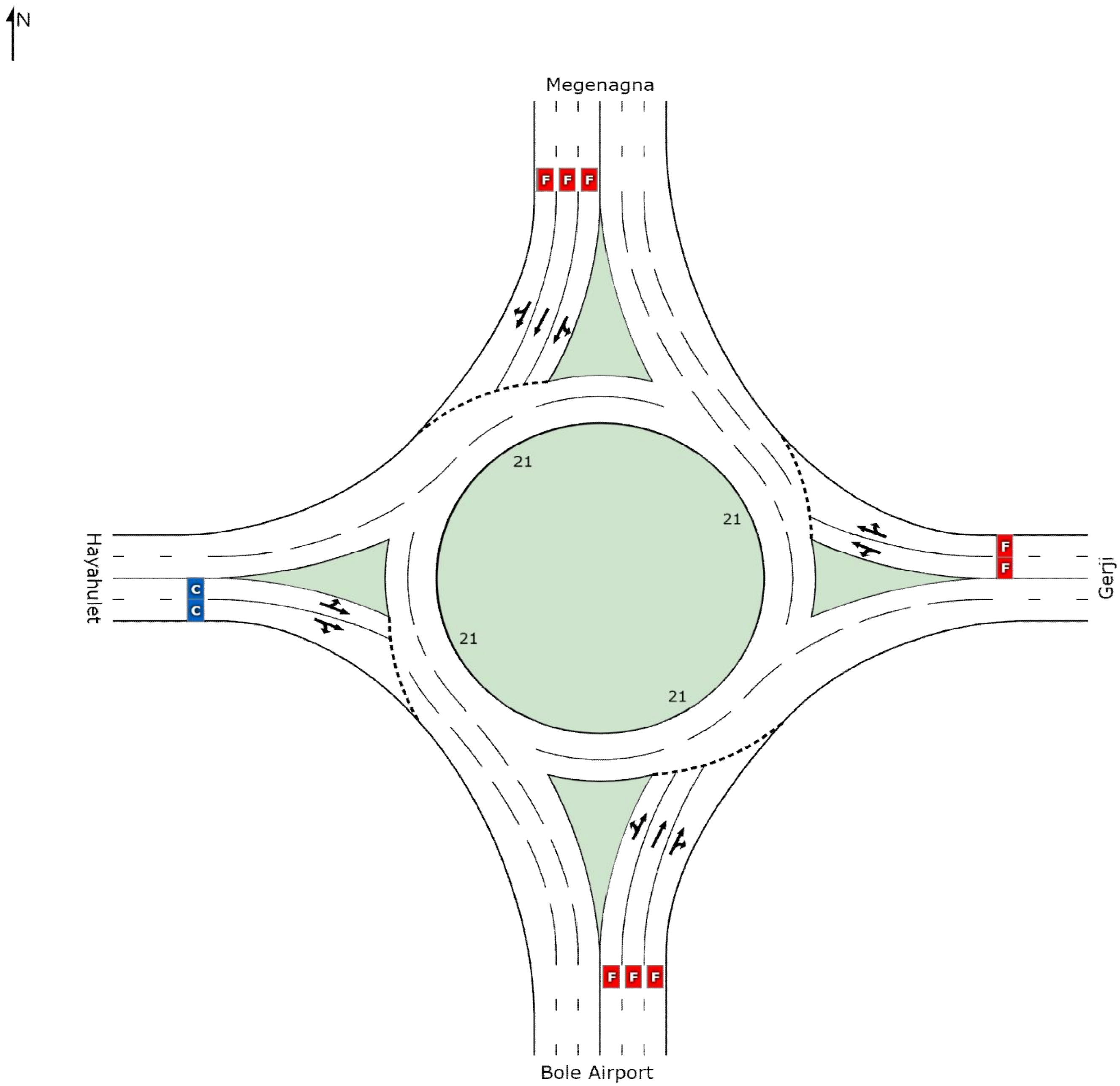
Approach	Movement	PHF
Bole-Airport	Left	0.96
	Through	0.98
	Right	0.96
Gerji	Left	0.94
	Through	0.95
	Right	0.94
Megenagna	Left	0.97
	Through	0.97
	Right	0.97
Hayahulet	Left	0.97
	Through	0.97
	Right	0.99

Gerji Imperial Roundabout Analysis Results in SIDRA Intersection Output

To evaluate the existing LOS, the SIDRA Software Program was used and lane LOS values were based on degree of saturation per lane and Intersection & Approach LOS values were based on worst degree of saturation for any lane. Table 5.5 below shows the detail.

Table 5.5-Output Summary of Gerji Imperial Roundabout

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: Bole Airport											
1	L	350	4.0	1.874	406.6	LOS F	136.5	1005.5	1.00	12.99	4.0
2	T	966	7.5	1.874	406.1	LOS F	150.3	1099.1	1.00	13.26	3.9
3	R	481	3.0	1.874	405.5	LOS F	150.3	1099.1	1.00	13.77	3.7
Approach		1798	5.6	1.874	406.0	LOS F	150.3	1099.1	1.00	13.35	3.9
East: Gerji											
4	L	334	2.0	1.152	89.5	LOS F	32.8	235.0	1.00	4.93	10.6
5	T	226	3.7	1.152	89.3	LOS F	35.8	253.3	1.00	4.95	10.4
6	R	563	1.0	1.152	87.9	LOS F	35.8	253.3	1.00	5.14	10.3
Approach		1123	1.8	1.152	88.6	LOS F	35.8	253.3	1.00	5.04	10.4
North: Megenagna											
7	L	395	3.0	1.501	241.3	LOS F	93.0	673.5	1.00	9.77	5.9
8	T	918	5.2	1.501	240.8	LOS F	99.1	718.4	1.00	9.98	5.8
9	R	274	2.0	1.501	240.5	LOS F	99.1	718.4	1.00	10.15	5.6
Approach		1586	4.1	1.501	240.8	LOS F	99.1	718.4	1.00	9.96	5.8
West: Hayahulet											
10	L	167	2.0	0.847	19.5	LOS C	7.3	52.4	0.94	1.64	16.8
11	T	400	4.0	0.847	18.4	LOS C	7.9	56.3	0.95	1.65	16.8
12	R	199	1.0	0.847	17.5	LOS C	7.9	56.3	0.96	1.66	16.8
Approach		766	2.8	0.847	18.4	LOS C	7.9	56.3	0.95	1.65	16.8
All Vehicles		**5272**	**3.9**	**1.874**	**232.4**	**LOS F**	**150.3**	**1099.1**	**0.99**	**8.86**	**5.9**



	South	East	North	West	Intersection
LOS	F	F	F	C	F

Figure 5.2- Level of Service (LOS) at Gerji Imperial Roundabout

The results of capacity analysis shows that the level of service (LOS) of the Gerji-Imperial Roundabout is “F” and it is noticed that all approaches except Hayahulet approach have level of service (LOS) of “F”; but Hayahulet approach has a level of service of “E”. The capacity of Gerji Imperial Roundabout is inadequate to control traffic flow at peak hour time due to the following problems observed;

- Inadequacy of inscribed circle diameter;
- Unbalanced traffic flow;
- Percentage of heavy traffic; and
- Pedestrian volume.

This Roundabout has an inscribed circle diameter of 33m; however, according to FHWA, 2,000 double lane urban roundabout that has a typical inscribed circle diameter should be in the range of 45 to 55m.This shows that the inscribed circular diameter of Gerji Imperial Roundabout is below the standard and it is one of the problems that has lower the capacity. From Table 4.3 it is observed that there is unbalanced traffic flow at legs or approaches at this Roundabout. According FHWA it is not recommended to build roundabouts as a mechanism of traffic control devices when there is unbalanced traffic on the legs (FHWA-RD-00-067). Hence, this is also one of the problems that have great impact on the capacity and other factors that have impact on Level of Service at Gerji Imperial Roundabout since the junction connects high speed primary Bole to Megenagna road and Gerji to Hayahulet access road. Furthermore, the percentage of heavy vehicle which is greater than 5% and the pedestrian number is high and separate pedestrian cross is not provided; as a result, the pedestrian conflict with vehicle at approaches reduce speed and increases delay at junction lowering the Level of Service of the roundabout. Therefore currently at peak hour the Gerji-Imperial Roundabout is not functioning well and so traffic police man regulates traffic flow.

5.1.2 Bole Medihanialem Roundabout

Geometric Data

1) Number of approaches or legs - 5
2) Number of circulating lane - 2
3) Inscribed circle diameter - 58m
4) Central island diameter -36m
5) Truck apron width - 0

Table 5.6- Geometric data for Bole Medihanialem Roundabout

	Bole Airport Leg	Kazanches Leg	Friendship Leg	Gipson Leg	Hayahulet Leg
Number of Entry Lane	2	2	1	1	2
Average Lane Width (m)	3.6	3.6	3.3	3.3	3.2

Table 5.7-Traffic Volume Data for Bole-Medihanialem Roundabout

Junction Name	**Approach Leg**	**Entry Traffic on Leg (PCU)**	**% of Traffic Share**
Bole-Medihanialem	Kazanches	1016	28.5
	Friendship	276	9.1
	Bole Airport	882	27.3
	Gipson Academy	395	12.1
	Hayahulet	899	23.0

An excel program was used to analyze the traffic count shown in Table 4.2 to specify the peak hour. From site investigation and traffic count, the following were observed:

a) The peak hour at Bole Medihanialem was found to be between 5:00-6:00 p.m. The total traffic volume during this hour was 3467. Figures 5.3 show the peak hour traffic (pcu) during the survey time at Junction.

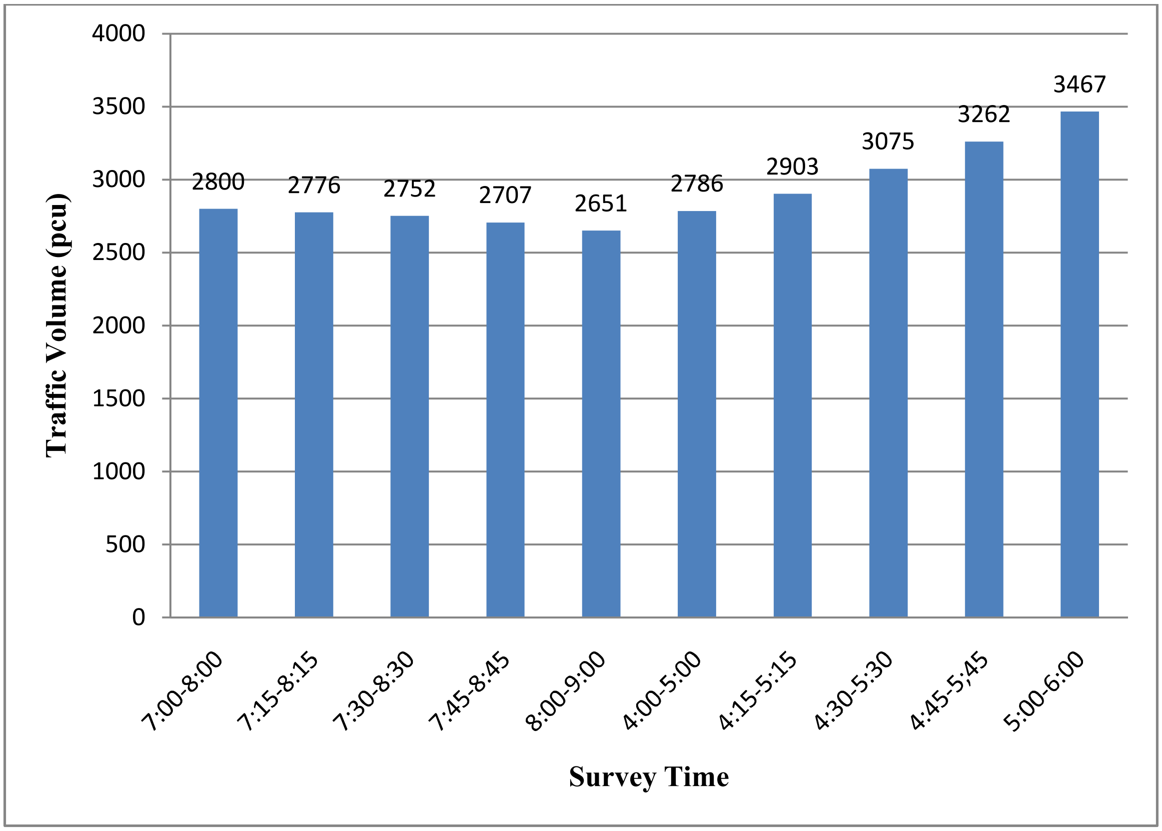


Figure 5.3- Traffic Volume at Bole Medihanialem Roundabout counted within 15 minutes Time Interval

b) The percentage of heavy vehicles at Bole-Medihanialem Roundabout was not significant at all approaches as shown in Table 5.8

Table 5.8 -Percentage of Heavy Vehicle for All Approaches at Bole-Medihanialem Roundabout

Approach	Percentage of Heavy Vehicle at Peak Hour
Kazanches	1.3
Friendship	0
Bole-Airport	1.4
Gipson -Academy	2.6
Hayahulet	1.7

c) The PHF is defined as the ratio of total volume to the maximum 15 min rate of flow with-in the hour. Table 5.9 depicts PHF values for all approaches at Bole-Medihanialem Junction.

Table 5.9-PHF Values for All Approaches at Bole-Medihanialem Junction

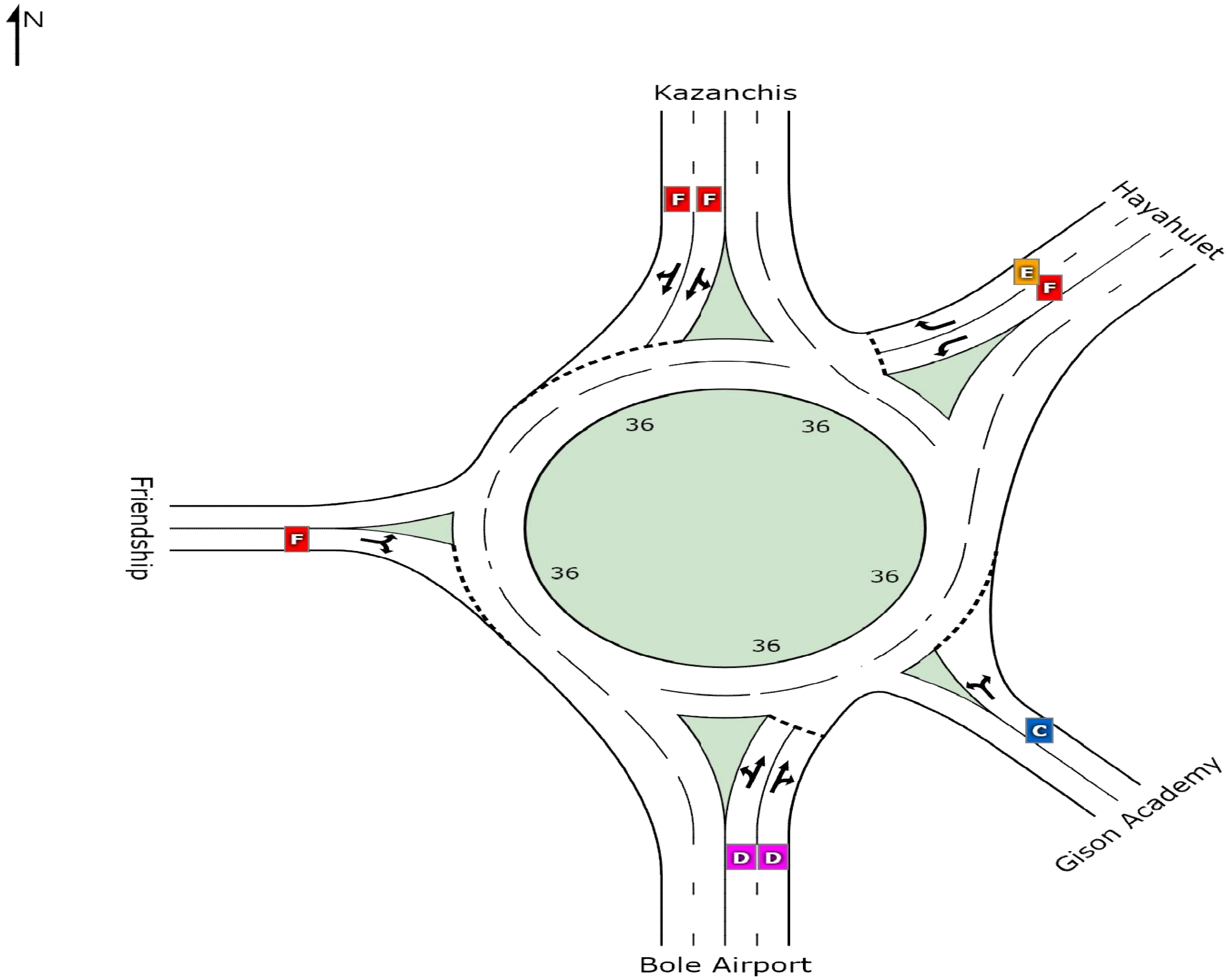
Approach	Movement	PHF
Kazanchis	Left	0.87
	Through	0.95
	Right	0.91
Friendship	Left	0.83
	Through	0.89
	Right	0.77
Bole Airport	Left	0.83
	Through	0.92
	Right	0.92
Gibson Academy	Left	0.94
	Through	0.94
	Right	0.85
Hayahulet	Left	0.9
	Right	0.88

<u>Results of Bole-Medihanialem Roundabout Analysis from the SIDRA Intersection Output</u>

To evaluate the existing LOS, the SIDRA Software Program was used and lane LOS values were based on degree of saturation per lane. Intersection & Approach LOS values are based on worst degree of saturation for any lane. Table 5.10 below shows the detail.

Table 5.10- Summary of Outputs at Bole-Medihabialem Roundabout

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: Bole Airport											
1	L	152	1.0	0.866	21.9	LOS D	8.9	63.5	0.89	1.49	21.0
2	T	429	2.0	0.866	18.4	LOS D	8.9	63.5	0.89	1.42	20.3
3	R	311	0.8	0.866	18.6	LOS D	8.9	63.2	0.89	1.42	19.9
Approach		892	1.4	0.866	19.1	LOS D	8.9	63.5	0.89	1.43	20.3
South East: Gibson Academy											
21	L	221	3.3	0.793	21.5	LOS C	6.4	45.6	0.92	1.29	20.8
23	R	98	1.0	0.793	18.1	LOS C	6.4	45.6	0.92	1.25	19.5
Approach		319	2.6	0.793	20.5	LOS C	6.4	45.6	0.92	1.28	20.4
North East: Hayahulet											
24	L	557	1.5	1.236	131.7	LOS F	44.4	315.0	1.00	4.20	6.5
26	R	401	1.0	0.998	43.2	LOS E	14.0	98.5	0.93	2.06	12.6
Approach		958	1.3	1.236	94.7	LOS F	44.4	315.0	0.97	3.30	7.9
North: Kazanches											
7	L	376	1.0	1.005	42.2	LOS F	15.1	107.0	1.00	2.13	15.1
8	T	457	2.0	1.005	38.2	LOS F	15.5	109.5	1.00	2.14	14.2
9	R	125	0.0	1.005	38.1	LOS F	15.5	109.5	1.00	2.15	13.6
Approach		958	1.3	1.005	39.7	LOS F	15.5	109.5	1.00	2.14	14.5
West: Friendship											
10	L	178	0.0	1.008	60.3	LOS F	15.2	106.4	1.00	2.01	11.8
12	R	161	0.0	1.008	57.1	LOS F	15.2	106.4	1.00	2.01	10.3
Approach		339	0.0	1.008	58.8	LOS F	15.2	106.4	1.00	2.01	11.1
All Vehicles		**3467**	**1.3**	**1.236**	**49.7**	**LOS F**	**44.4**	**315.0**	**0.96**	**2.19**	**12.5**



	South	Southeast	Northeast	North	West	Intersection
LOS	D	C	F	F	F	F

Figure 5.4- Level of Service (LOS) at Bole-Medihanialem Roundabout

The result of capacity analysis shows the level of service (LOS) on Bole Airport approach is "D", Friendship approach is "F", Gibson Academy approach is "C", Kazanches and Hayahulet approaches have "F". However the overall level of service at Bole Medihanialem Roundabout is "F". The problem in this junction is observed at Kazanches, Friendship and Hayahulet approaches because of inadequacy of number of lanes. If improvements were made on Kazanches and Hayahulet approaches, the capacity of the Junction as a whole will be improved.

5.1.3 Teklehaymanot Roundabout

Geometric Data

1) Number of approaches or legs - 4
2) Number of circulating lane - 2
3) Inscribed circle diameter – 65/85m
4) Central island diameter- 40/65m
5) Truck apron width - 2

Table 5.11- Geometric Data for Teklehaymanot Roundabout

	Merkato Leg	Abinet Leg	Black-Lion Leg	Piazza Leg
Number of Entry Lane	2	2	2	2
Average Lane width(m)	3.2	3.6	3.6	3.2

Table 5.12- Traffic Volume Data at Teklehaymanot Roundabout

Junction Name	**Approach Leg**	**Entry Traffic on leg (PCU)**	**% of Traffic Share**
Teklehaymanot	Merkato	1675	32.2
	Abinet	352	16.5
	Black Lion	1673	31.9
	Piaza	808	19.4

An excel program was used to analyze the traffic data shown in Table 4.2 to specify the peak hour. From site investigation and traffic count, the following were observed:

a) The peak hour at Teklehaymanot is found to be between 4:45-5:45 p.m. The total traffic volume during this hour was 4507. Figures 5.5 show the peak hour during the time period of survey at Junction

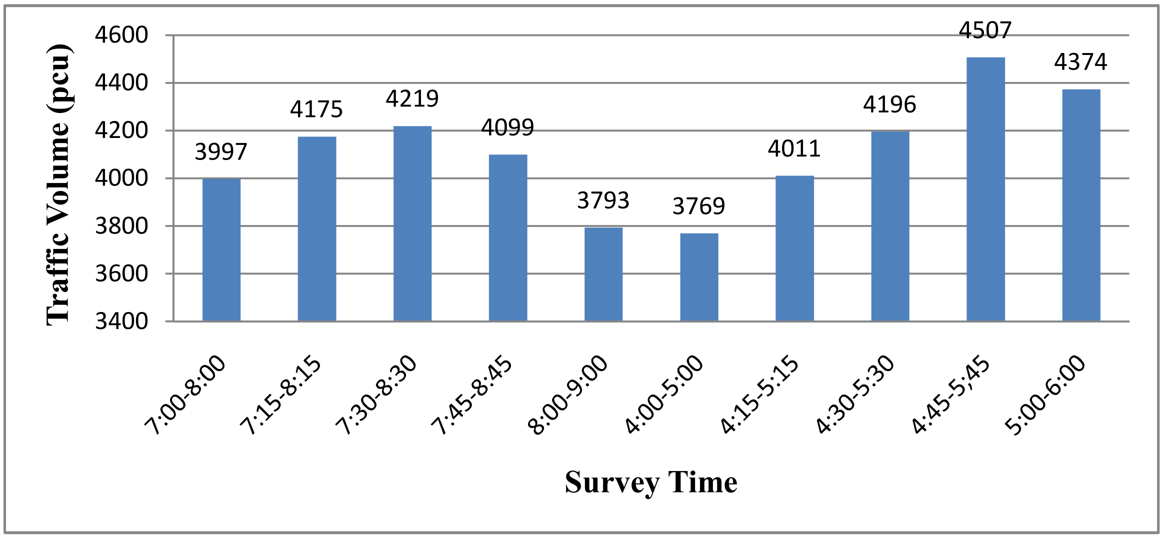


Figure 5.5- Traffic volumes at Teklehaymanot Roundabout counted within 15 minutes Interval

b) The percentage of heavy vehicles at Teklehaymanot Roundabout was not significant on all approaches as shown in Table 5.13

Table 5.13- Percentages of Heavy Vehicle on all approaches at Teklehaymanot Roundabout

Approach	Percentage of Heavy Vehicle at Peak Hour
Merkato	5.3
Abinet	2.5
Black Lion	4.6
Piaza	4.3

c) The PHF is defined as the ratio of total volume to the maximum 15 min rate of flow within the hour. Table 5.14 depicts PHF values on all approaches at Teklehaymanot Roundabout.

Table 5.14- PHF Values for all approaches at Teklehaymanot Rounabout

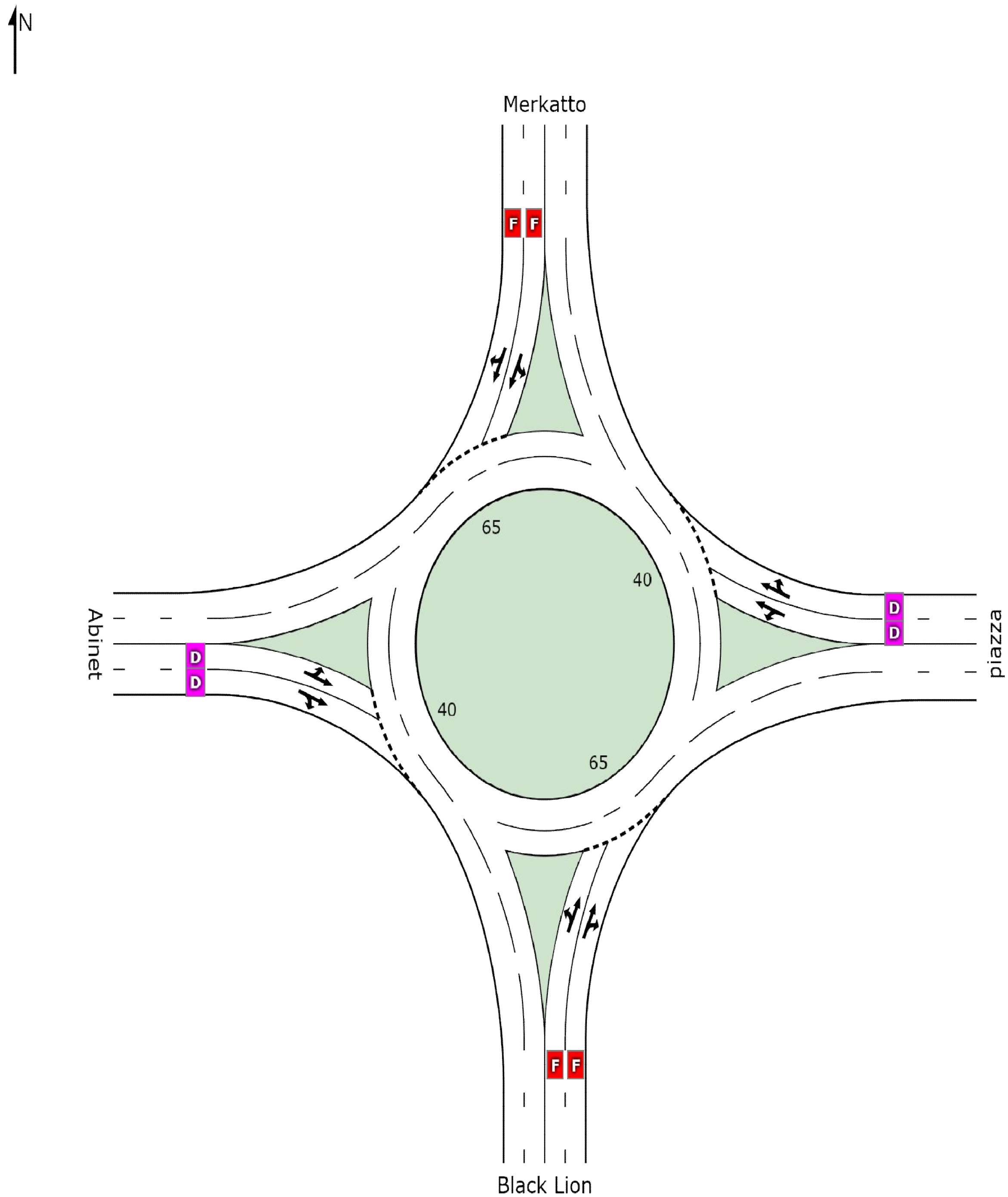
Approach	Movement	PHF
Merkato	L	0.93
	TH	0.97
	R	0.98
Abinet	L	0.91
	TH	0.95
	R	0.88
Black-Lion	L	0.93
	TH	0.92
	R	0.88
Piazza	L	0.93
	TH	0.89
	R	0.82

Results of Teklehaymanot Roundabout Analysis based on SIDRA Intersection Output

To evaluate the existing LOS, SIDRA Software Program was used and lane LOS values are based on degree of saturation per lane; Intersection & Approach LOS values are based on worst degree of saturation for any lane. Table 5.15 below shows the detail.

Table 5.15- Summary of Output at Teklehaymanot Roundabout

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: Black Lion											
1	L	418	3.0	1.164	96.7	LOS F	45.4	329.3	1.00	4.03	13.1
2	T	942	6.0	1.164	94.2	LOS F	55.3	403.8	1.00	4.33	12.6
3	R	167	1.0	1.164	93.2	LOS F	55.3	403.8	1.00	4.46	12.2
Approach		1527	4.6	1.164	94.8	LOS F	55.3	403.8	1.00	4.26	12.7
East: piazza											
4	L	146	3.0	0.919	50.1	LOS D	10.3	74.5	1.00	1.77	17.8
5	T	397	5.3	0.919	44.7	LOS D	12.5	91.2	1.00	1.85	18.1
6	R	87	2.0	0.919	42.5	LOS D	12.5	91.2	1.00	1.88	17.9
Approach		630	4.3	0.919	45.7	LOS D	12.5	91.2	1.00	1.84	18.0
North: Merkatto											
7	L	249	2.0	1.171	97.1	LOS F	42.8	312.5	1.00	4.28	13.1
8	T	828	7.0	1.171	95.3	LOS F	53.1	389.3	1.00	4.51	12.5
9	R	415	4.0	1.171	93.6	LOS F	53.1	389.3	1.00	4.78	12.2
Approach		1492	5.3	1.171	95.2	LOS F	53.1	389.3	1.00	4.55	12.5
West: Abinet											
10	L	464	2.0	0.908	19.8	LOS D	10.5	74.8	0.98	1.66	23.6
11	T	153	1.0	0.908	22.0	LOS D	10.5	74.8	0.97	1.64	22.7
12	R	298	4.0	0.908	22.9	LOS D	9.4	67.8	0.96	1.63	22.0
Approach		915	2.5	0.908	21.2	LOS D	10.5	74.8	0.97	1.65	23.0
All Vehicles		4507	4.4	1.171	73.4	LOS F	55.3	403.8	0.99	3.50	14.5



	South	East	North	West	Intersection
LOS	F	D	F	D	F

Figure 5.6- Level of Service (LOS) at Teklehaymanot Roundabout

As the results of capacity analysis shows, the level of service (LOS) of the Black-Lion approach is “F”, Merkatto approach is “F”, Piazza approach is “D” and Abinet approach is “D”. However, the overall level of service of Teklehaymanot Roundabout is “D”. When looking into the degree of saturation (V/C) at each approach, the Merkato approach has 1.17, Black-Lion approach has 1.16, Piazza approach has 0.92 and Abinet approach has 0.91. Hence, both Merkato and Black-Lion approaches were performing poorly and in worst conditions affecting the overall capacity of the Roundabout. The problems at Teklehaymanot Roundabout were related to:

- Inadequate number of entry and exit lanes at Merkato and Black-Lion approaches;
- Pavement condition; and
- Pedestrian volume.

This Roundabout is located near the biggest market center of the City Merkato, and in front Teklehaymano church. Consequently, the number of pedestrian at the approaches of this roundabout is very high, at the peak hour of 4:45-5:45 pm, the number of pedestrian was counted as 2,488 and this has great impact on lowering the capacity of the Roundabout. In addition, the numbers of entry and exit lanes particularly at Merkato and Black-Lion approaches are two with a width of 3.2m which are not sufficient. Even though, there was no separate parking lane at approaches, vehicle parking was allowed as result only one lane serving as entry and exit lane and this phenomena reduces the serviceability of the junction. Potholes, rutting and corrugation were major pavement distress observed on the approaches and circulatory lane, these pavement distress conditions reduce vehicle speed and contributes to delay as well as lowers the capacity of the roundabout. During the time of survey, Teklehaymanot roundabout was not performing well and traffic police-man regulates the traffic flow. Therefore, in addition to aforementioned problems, further detailed investigations are recommended to propose best solutions that alleviate the existing problem.

5.1.4 Ayertena Roundabout

<u>*Geometric Data*</u>

1) Number of approaches or legs - 4
2) Number of circulating lane - 2
3) Inscribed circle diameter – 70/80m
4) Central island diameter-50/60m
5) Truck apron width - 0

Table 5.16 and 5.17 show geometric and traffic volume data respectively.

Table 5.16- Geometric Data for Ayertena Roundabout

	Jomo Leg	Mexico Leg	Alembank Leg	Alemgena Leg
Number of entry lanes	2	2	2	2
Average Lane width (m)	3.2	3.2	3.6	3.2

Table 5.17- Traffic Volume Data for Ayertena Roundabout

Junction Name	**Approach Leg**	**Entry Traffic on leg (PCU)**	**% of Traffic share**
Ayer-Tena	Alemgena	1444	28.2
	Jomo	1148	25.6
	mexico	1557	28.0
	Alembank	940	18.1

An excel program was used to analyze the traffic count data as shown in Table 4.2 to specify the peak hour. From site investigation and traffic count, the following were observed:

a) The peak hour at Ayertena Roundabout was found to be between 5:00-6:00 p.m. The total traffic volume during this hour was, 5088. Figures 5.7 show the peak hour during the time of survey at the Roundabout.

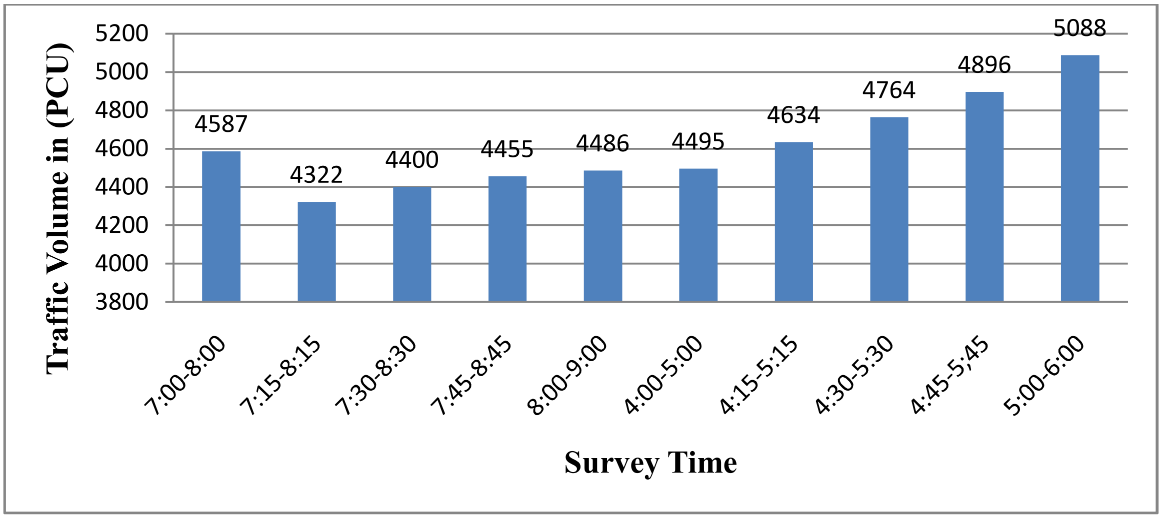


Figure 5.7-Traffic volumes at Ayertena Roundabout counted within 15 minutes time Interval

b) The percentage of heavy vehicles at Ayertena Roundabout was very significant at all approaches as shown in Table 5.18.

Table 5.18-Percentage of Heavy Vehicle for all approaches at Ayertena Roundabout

Approach	Percentage of Heavy Vehicle
Alemgena	9.7
Jomo	8.9
Mexico	8.3
Alembank	5.1

c) The PHF is defined as the ratio of total volume to the maximum 15 min rate of flow within the hour. Table 5.19 depicts PHF values for all approaches at Ayertena roundabout junction.

Table 5.19-PHF Values for all approaches at Ayertena Roundabout

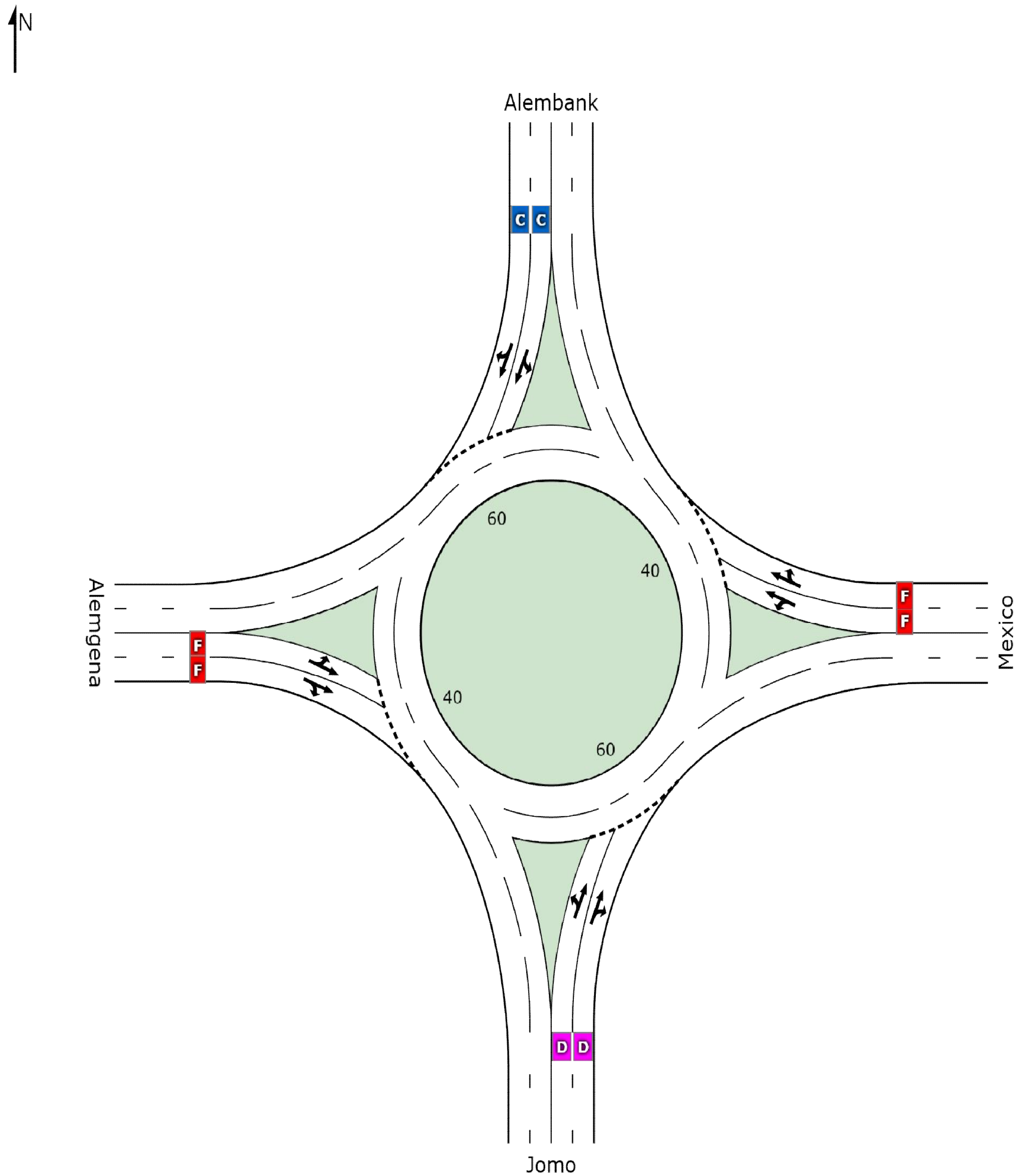
Approach	Movement	PHF
Merkato	Left	0.96
	Through	0.98
	Right	0.96
Abinet	Left	0.94
	Through	0.95
	Right	0.94
Black Lion	Left	0.975
	Through	0.975
	Right	0.97
Piazza	Left	0.97
	Through	0.97
	Right	0.99

Ayertena Roundabout Analysis Results in SIDRA Intersection Output

To evaluate the existing LOS SIDRA software program was used and lane LOS values are based on degree of saturation per lane and Intersection & Approach LOS values are based on worst degree of saturation for any lane and Table 5.20 below shows the detail.

Table 5.20-Output Summary of Ayertena Roundabout

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: Jomo											
1	L	287	5.0	0.890	20.1	LOS D	10.4	77.8	1.00	1.61	23.6
2	T	688	12.0	0.890	17.0	LOS D	12.5	94.4	1.00	1.62	24.0
3	R	173	3.0	0.890	15.8	LOS D	12.5	94.4	1.00	1.65	24.0
Approach		1148	8.9	0.890	17.6	LOS D	12.5	94.4	1.00	1.62	23.9
East: Mexico											
4	L	389	5.0	1.420	205.1	LOS F	73.4	546.3	1.00	5.61	7.9
5	T	934	11.0	1.420	202.5	LOS F	96.8	729.5	1.00	6.28	7.5
6	R	234	3.0	1.420	201.6	LOS F	96.8	729.5	1.00	6.71	7.2
Approach		1557	8.3	1.420	203.0	LOS F	96.8	729.5	1.00	6.17	7.6
North: Alembank											
7	L	235	0.0	0.755	14.4	LOS C	6.9	49.7	0.98	1.32	25.1
8	T	564	8.0	0.755	11.7	LOS C	8.2	60.7	0.99	1.34	25.6
9	R	141	2.0	0.755	10.7	LOS C	8.2	60.7	1.00	1.35	25.7
Approach		940	5.1	0.755	12.2	LOS C	8.2	60.7	0.99	1.34	25.5
West: Alemgena											
10	L	361	6.0	1.294	152.4	LOS F	57.0	429.8	1.00	4.84	9.7
11	T	866	13.0	1.294	149.4	LOS F	73.1	557.2	1.00	5.31	9.4
12	R	217	3.0	1.294	148.2	LOS F	73.1	557.2	1.00	5.67	9.0
Approach		1444	9.7	1.294	150.0	LOS F	73.1	557.2	1.00	5.24	9.4
All Vehicles		**5088**	**8.3**	**1.420**	**110.9**	**LOS F**	**96.8**	**729.5**	**1.00**	**3.99**	**11.5**



	South	East	North	West	Intersection
LOS	D	F	C	F	F

Figure 5.8-Level of Service Summary of Ayertena Roundabout

As the result of capacity analysis shows, the level of service (LOS) of the Ayertena Roundabout is “F” and it is noticed that the Mexico and Alemgena approaches have level of service (LOS) of “F” and Jemo and Alembank approaches have “D” and “C” respectively. When looking into the degree of saturation (V/C) at each approach, Alemgena approach has 1.29, Jomo approach has 8.9, Mexico approach has 1.42 and Alembank approach has 0.79. This shows that Mexico and Alemgena approaches were within the degree of saturation greater than 1 and they were at worst conditions than the rest of the approaches. The problems at Ayertena roundabout were related to:

- Inadequate number of entry and exit lane at Mexico and Alemgena approaches.
- Pavement condition,
- Pedestrian volume; and
- Percentage of heavy vehicle.

The Ayertena Roundabout is located along the East –West corridor of Addis Ababa and close to resident areas. Consequently, the number of traffic, number of pedestrian and percentages of heavy vehicles at the approaches of this Roundabout were very high, at the peak hour of 5:00-6:00 pm and these have great impact on lowering the capacity of the roundabout. In addition, the number of entry and exit lanes particularly on Alemgena and Mexico approaches, were two with width of 3.2m each and which were not adequate for existing traffic volume. Even though, there was no separate parking lane at approaches, city-buses and taxis were allowed to load and unload passengers at approaches, thus one lane was serving as entry and exit and this practices reduces the serviceability of the junction. Pavement condition in respect of potholes, rutting and corrugations were the major pavement distresses as observed on the approaches and circulatory lane. These pavement distress conditions reduce vehicle speed and contribute to delay as well as lower the capacity of the roundabout. During the time of the survey, Ayertena Roundabout was not performing well and so traffic police men regulate the traffic flow. Therefore, in addition to aforementioned problems, further detail investigations are recommended to propose best solutions that will alleviate the prevailing problem.

5.2 Signalized Junction

5.2.1 Post-Office Signalize Junction

Table 5.21-Geometric data for Post-Office Signalized Junction

Junction Name	Approach Name	No.of Entry-Lane	No. of Exit Lane	Lane Width (m)	Median Width (m)
Post-office	Filwuha	2	2	3.3	1.0
	Piazza	3	3	3.3	1.5
	Legehar	3	3	3.3	1.5
	Black Lion	2	2	3.3	1.0

Table 5.22-Traffic Volume Data for Post-Office Signalized Junction

Junction Name	**Approach Leg**	**Entry Traffic on Leg (PCU)**	**% of Traffic Share**
Post-office	Black Lion	637	15.8
	Legehar	1230	30.5
	Filwuha	583	14.5
	Piazza	1578	39.2

Table 5.23-Signal Time Data for Post-Office Signalized Junction

Phase	**A**	**B**	**C**	**D**
Green Time (sec)	30	30	20	20
Yellow Time (sec)	2	2	2	2
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	34	34	24	24
Phase Split	29 %	29 %	21 %	21 %
Cycle-Time	116 sec			

An excel program was used to analyze the traffic volume count shown in Table 4.2 to specify the peak hour. From site investigation and traffic account, the following were observed:

a) The peak hour at Post office signalized intersection is found to be between 5:00-6:00 p.m. The total traffic volumes during this hour at intersection was 4028 pc/h. Figure 5.9 show the peak hour during the time period of survey at Junction.

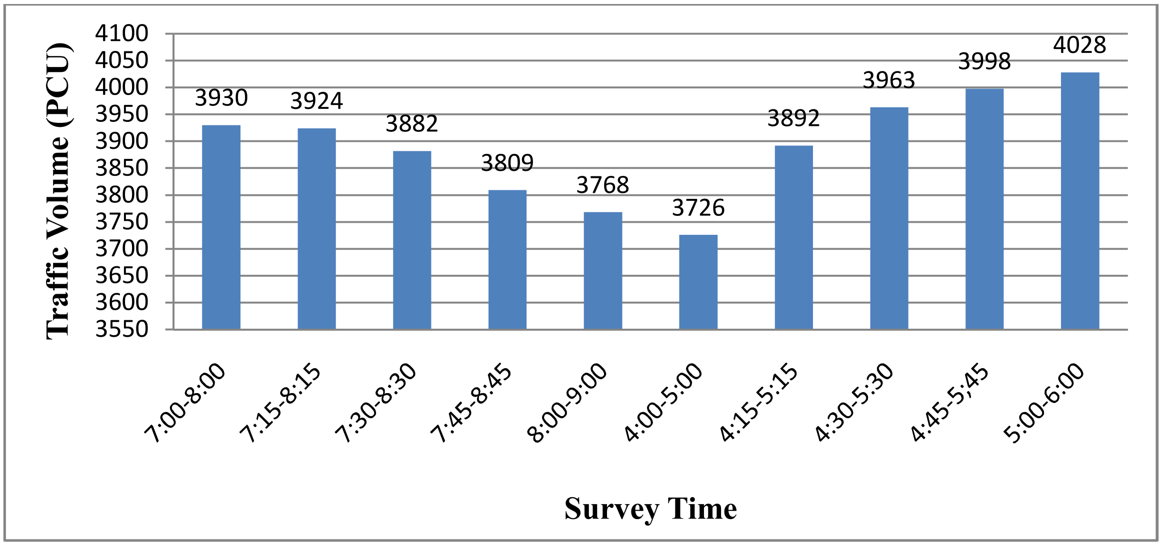


Figure 5.9-Total traffic volumes at Post-Office intersection counted within 15 minutes Time interval

c) The percentages of heavy vehicles at Post-office Signalized Junction were concentrated on Black Lion and Legehar approaches as shown in Table 5.24.

Table 5.24-Percentages of Heavy Vehicles for all approach at the Post-Office Signalized Junction

Approach	Percentage of Heavy Vehicle
Filwuha	1.1
Piazza	3.2
Legehar	3.6
Black -Lion	3.7

c) The PHF is defined as the ratio of total volume to the maximum 15 min rate of flow with-in the hour. Table 5.25 depicts PHF values for all approaches at Post-Office Junction.

Table 5.25 - PHF Values for All Approaches at Post-Office Signalized Junction.

Approach	Movement	PHF
Filwuha	Left	0.95
	Through	0.96
	Right	0.95
Piazza	Left	0.94
	Through	0.95
	Right	0.95
Legehar	Left	0.95
	Through	0.96
	Right	0.95
Black Lion	Left	0.94
	Through	0.95
	Right	0.94

Post-Office Signalized Intersection Analysis Results in SIDRA Intersection Output

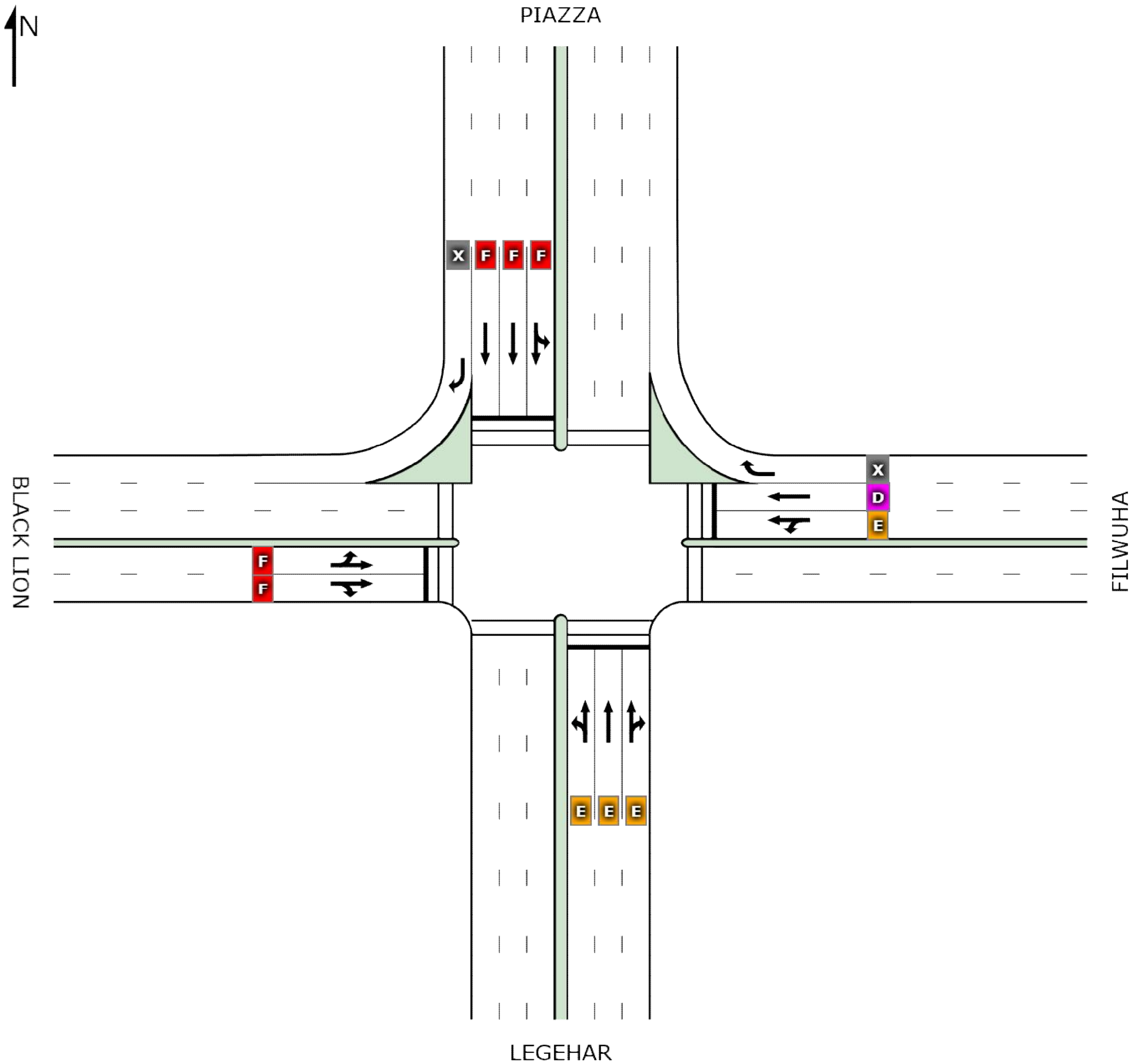
To evaluate the existing LOS SIDRA software program was used and lane LOS values are based on average delay per lane and Intersection and Approach LOS values are based on average delay for all lanes and Table 5.26 below shows the detail.

Table 5.26-Output Summary of Post-Office Signalized Intersection

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: Legehar											
1	L	146	2.0	0.962	81.8	LOS F	22.6	162.5	1.00	1.15	17.1
2	T	1019	4.0	0.962	70.9	LOS E	33.5	242.5	1.00	1.14	17.9
3	R	65	1.0	0.962	76.8	LOS E	32.0	231.2	1.00	1.13	18.0
Approach		1230	3.6	0.962	72.5	LOS E	33.5	242.5	1.00	1.14	17.8
East: Filwuha											
4	L	150	2.0	0.756	58.7	LOS E	13.8	98.0	1.00	0.89	21.0
5	T	345	1.0	0.756	52.4	LOS D	14.3	101.2	1.00	0.90	21.3
6	R	88	0.0	0.047	5.6	X	X	X	X	0.53	44.1
Approach		583	1.1	0.756	46.9	LOS D	14.3	101.2	0.85	0.84	23.0
North: Piazza											
7	L	244	0.0	1.017	91.7	LOS F	38.6	275.1	1.00	1.19	15.8
8	T	1229	4.0	1.017	85.7	LOS F	39.0	282.6	1.00	1.23	15.9
9	R	105	1.0	0.057	5.6	X	X	X	X	0.53	44.1
Approach		1578	3.2	1.017	81.3	LOS F	39.0	282.6	0.93	1.18	16.6
West: Black Lion											
10	L	118	3.0	1.119	133.6	LOS F	28.1	202.9	1.00	1.37	12.1
11	T	428	4.0	1.119	125.6	LOS F	29.0	209.7	1.00	1.36	12.1
12	R	91	3.0	1.119	130.4	LOS F	29.0	209.7	1.00	1.36	12.2
Approach		637	3.7	1.119	127.7	LOS F	29.0	209.7	1.00	1.36	12.1
All Vehicles		4028	3.1	1.119	81.0	LOS F	39.0	282.6	0.95	1.15	16.6

Table 5.27- Output Summary of Pedestrian at Post-Office Signalized Junction.

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
2P	Across S approach	429	45.7	LOS E	1.3	1.9	0.89	0.89
8P	Across E approach	524	37.3	LOS D	1.4	2.1	0.80	0.80
6P	Across N approach	667	45.7	LOS E	2.0	2.9	0.89	0.89
4P	Across W approach	762	37.3	LOS D	2.0	3.0	0.80	0.80
All Pedestrians		2382	41.2	LOS E			0.84	0.84



	South	East	North	West	Intersection
LOS	E	D	F	F	F

X: Not applicable for Continuous lane.

Figure 5.10-Level of Service Summary of Post-Office Signalized Intersection

As the results of capacity analysis shows, the level of service (LOS) of Legehar approach is "E", Filiwuha approach is "D", Piazza approach is "F", Black Lion approach "F". Overall level of service at the Post-Office Signalized Intersection is "F". Looking into the delays developed at each approach, at Legehar approach it is 72.5 sec, at Filwuha approach it is 46.9 sec, at Piazza approach it is 81.3 sec and at Black-Lion approach it is 127.7 sec. Thus, Black-Lion and Piazza approaches were performing at poor level of service. The problems at this intersection were related to:

- ✓ Inadequate number of entry and exit lanes;
- ✓ Inadequate phase time;
- ✓ Pedestrian volume; and
- ✓ Pavement distress.

The number of entry lanes at Black-Lion approaches was not sufficient for peak hour traffic volume and allocated phase time for Black Lion and Piazza approaches were not enough to discharge traffic in the queue and these have great impact on lowering the capacity of the Intersection. In addition, the number of pedestrians was high since the junction is located near schools, hospital and commercial centers. Pavement condition of the intersection approaches were not good to have smooth and fast driving, especially, in Black-Lion and Filwuha approaches. Pavement distress like, potholes, rutting and corrugations were observed. Therefore, the pavement distress conditions reduce vehicle speed and contribute to delays as well as lower the capacity of the Intersection. During the peak time of the survey, the Post-Office Signalized intersection was not performing well by the traffic signal and the traffic flows were regulated by traffic police-men. However, after the survey time of this research, Addis Ababa City Authority had made improvement regarding phase time allocation and cycle-time for Post-Office signalized intersection. Table 5.28 below illustrates the new signal data.

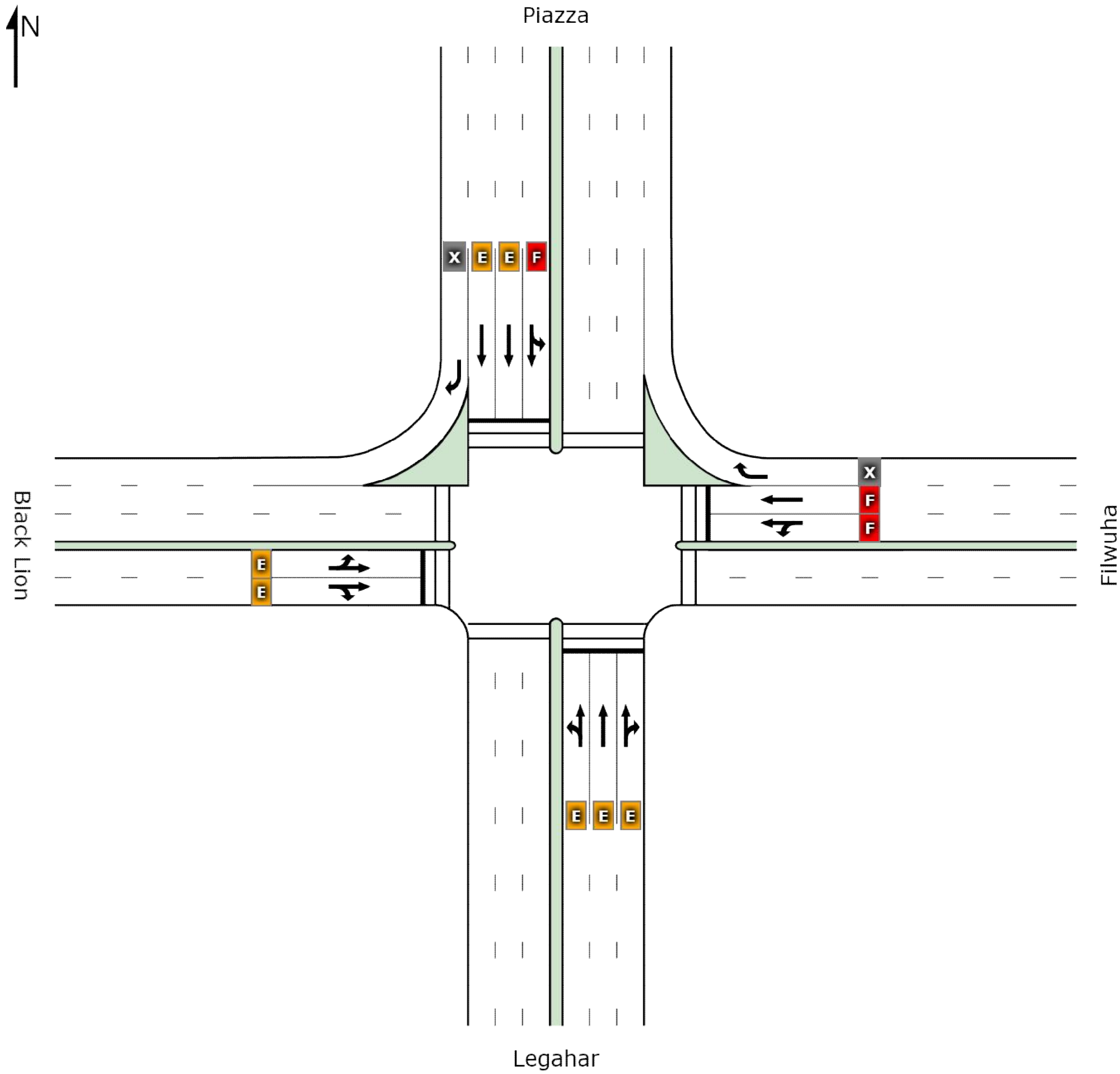
Table 5.28-New Signal Time data for Post-Office Signalized Junction

Phase	**A**	**B**	**C**	**D**
Green Time (sec)	30	18	39	39
Yellow Time (sec)	2	2	2	2
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	34	22	43	43
Phase Split	24 %	16 %	30 %	30 %
Cycle-Time	142 sec			

The level of service (LOS) of this junction was evaluated considering the newly collected signal data. The result showed that the rest geometric as well as traffic data remained the same as the previous one and so the same analysis procedure and software model was adopted and the following results were obtained.

Table 5.29-Output Summary of Post-Office Signalized Intersection for The New signal Data

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: Legehar											
1	L	146	2.0	0.903	78.1	LOS E	24.2	173.9	1.00	1.04	17.6
2	T	1019	4.0	0.903	67.2	LOS E	35.8	258.9	1.00	1.02	18.5
3	R	65	1.0	0.903	73.2	LOS E	34.6	249.7	1.00	1.01	18.5
Approach		1230	3.6	0.903	68.8	LOS E	35.8	258.9	1.00	1.02	18.4
East: Filwuha											
4	L	150	2.0	1.029	114.4	LOS F	22.1	157.0	1.00	1.14	13.4
5	T	345	1.0	1.029	107.9	LOS F	23.0	162.3	1.00	1.16	13.5
6	R	88	0.0	0.047	5.6	X	X	X	X	0.53	44.1
Approach		583	1.1	1.029	94.1	LOS F	23.0	162.3	0.85	1.06	15.1
North: Piazza											
7	L	244	0.0	0.957	83.8	LOS F	41.0	291.8	1.00	1.06	16.8
8	T	1229	4.0	0.957	77.7	LOS E	41.4	299.7	1.00	1.09	16.9
9	R	105	1.0	0.057	5.6	X	X	X	X	0.53	44.1
Approach		1578	3.2	0.957	73.9	LOS E	41.4	299.7	0.93	1.04	17.6
West: Black Lion											
10	L	118	3.0	0.897	78.4	LOS E	24.7	178.6	1.00	1.01	17.7
11	T	428	4.0	0.897	70.3	LOS E	24.7	178.6	1.00	1.01	17.9
12	R	91	3.0	0.897	74.8	LOS E	23.6	170.8	1.00	1.01	18.2
Approach		637	3.7	0.897	72.4	LOS E	24.7	178.6	1.00	1.01	17.9
All Vehicles		**4028**	**3.1**	**1.029**	**75.0**	**LOS E**	**41.4**	**299.7**	**0.95**	**1.03**	**17.4**



	South	East	North	West	Intersection
LOS	E	F	E	E	E

X: Not applicable for Continuous lane.

Figure 5.11- LOS of Post-Office Signalized Intersection for The New Signal Data

The current capacity of Post-Office Signalized Intersection, as observed in Table 5.30 the level of service (LOS) of Legehar approach is “E”, Filiwuha approach is “F”, Piazza approach is “E”, Black Lion approach “E”, however the overall LOS of Post-Office Signalized intersection is “E”. When looking into the delay developed at each approach, at Legehar approach it is 68 sec, at Filwuha approach it is 94.1 sec, at Piazza approach it is 73.9 sec, at Black-Lion approach it is 72 sec and overall delays is 75 sec. However, Filwuha approach is performing in poor level of service.

Comparison of Capacity for Current Signal Time with Previous Signal Time

As the current capacity was compared with previous one, the overall level of service of the junction had improved from LOS of “F” to LOS of “E”. When looking to each approach level of service, the detail shown in Table 5.30 and the delay comparison is also shown in Figure 5.12.

Table 5.30-LOS Comparison at Post-Office Junction in Respect to Current &Previous Signal Time

Approach Name	Previous Level of Service (LOS)	Current Level of Service (LOS)	Remark
Leghar	E	E	Same
Piazza	F	E	Improved
Black Lion	F	E	Improved
Filwuha	D	F	Worsened
Intersection	F	E	Improved

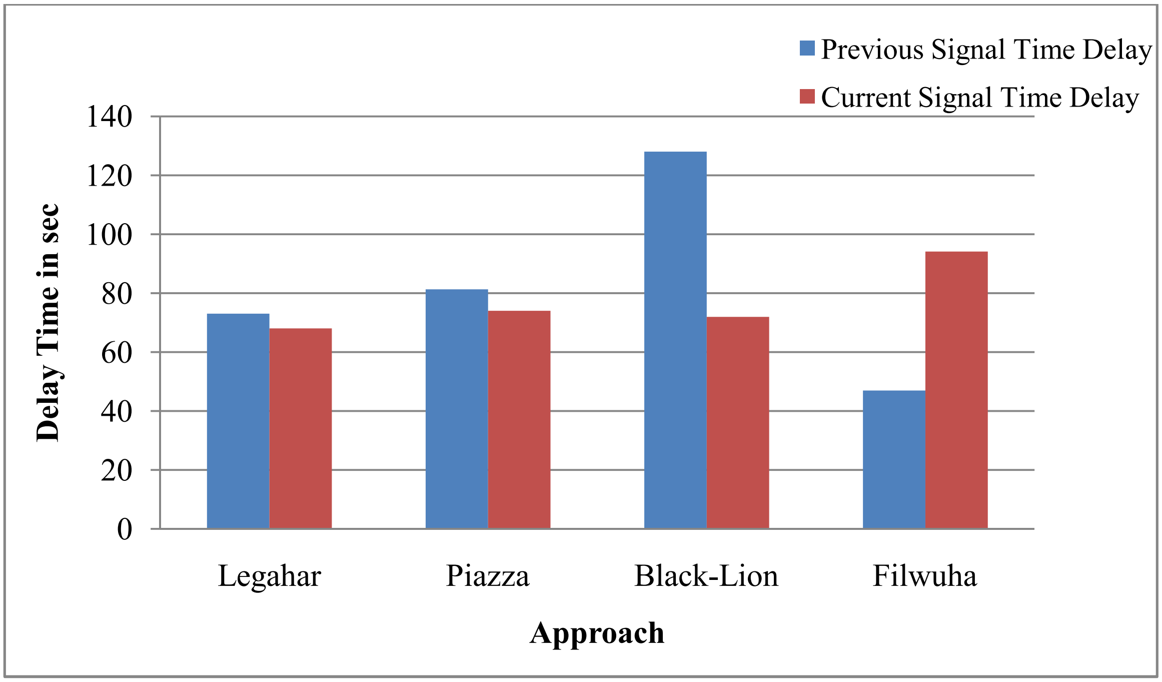


Figure 5.12-Delay Comparison at Post-Office Junction for Current & Previous Signal Time

As shown in Figure 5.12, signal time has improved, similarly the level of service from “F” to “E”. However, the improvement is not sufficient, even the Filwuha approach experienced great delay than the previous one. Hence, the problems of this intersection is not only inadequacy of signal time but also inadequate number of entry and exit lane, Pavement distress and pedestrian volume as explained above in the discussions under the previous capacity of the junction.

Therefore, to alleviate the problem entirely and to improve level of service, the above mentioned problems should be investigated intensively and should be addressed.

5.2.2 Legahar Signalized Junction

Table 5.31-Geometric Data for Legahar Signalized Junction

Junction Name	Approach name	No.of entry Lane	No. of exit Lane	Lane width	Median width
Legehar	Mexico	3	3	3.2	1.5
	Meskel Square	3	3	3.2	NA
	Cherkos	3	3	2.6	1.5
	Piazza	3	3	2.6	1.5

Table 5.32-Traffic Volume Data for Legahar Signalized Junction

Junction Name	Approach Leg	Entry Traffic on leg (PCU)	% of traffic share
Legehar	Mexico	1775	36.6
	Meskel Square	1907	39.3
	Cherkos	620	12.7
	Piazza	551	11.4

Table 5.33-Signal Time Data for Legahar Signalized Junction

Phase	A	B	C	D
Green Time (sec)	25	21	36	38
Yellow Time (sec)	3	3	3	3
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	30	26	41	43
Phase Split	21 %	19 %	29 %	31 %

An excel program was used to analyze the traffic volume count shown in Table 4.2 to specify the peak hour. From site investigation and traffic account, the following were observed:

a) The peak hour at Legahar Signalized Intersection is found to be between 5:00-6:00 p.m. The total traffic volumes during this hour at intersection was 4855 pc/h. Figure 5.13 show the peak hour during the time of survey at the Intersection.

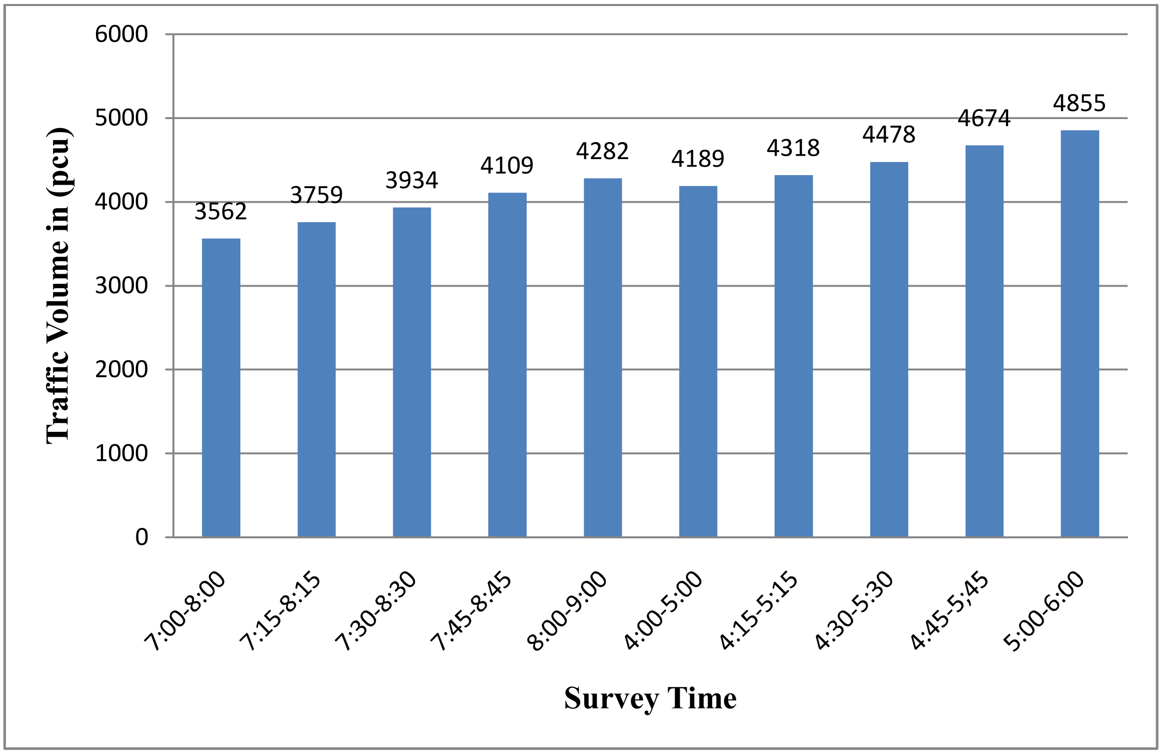


Figure 5.13-Total traffic volumes at Legahar intersection each one hour of 15 minute Interval

b) The percentage of heavy vehicles at Legahar signalized junction was concentrated on the approach that comes from Cherkos and Mexico approaches as depicted in Table 5.34.

Table 5.34-Percentage of Heavy Vehicle at Legahar Signalized Intersection for All Approach

Approach	Percentage of Heavy Vehicle
Mexico	1.9
Meskel Square	1.8
Cherkos	2.3
Piazza	0.4

c) The PHF is defined as the ratio of total volume to the maximum 15 min rate of flow within the hour. Table 5.35 depicts PHF values for all approaches at Legahar signalized Junction.

Table 5.35- PHF Values for All Approaches at Legahar Signalized Intersection

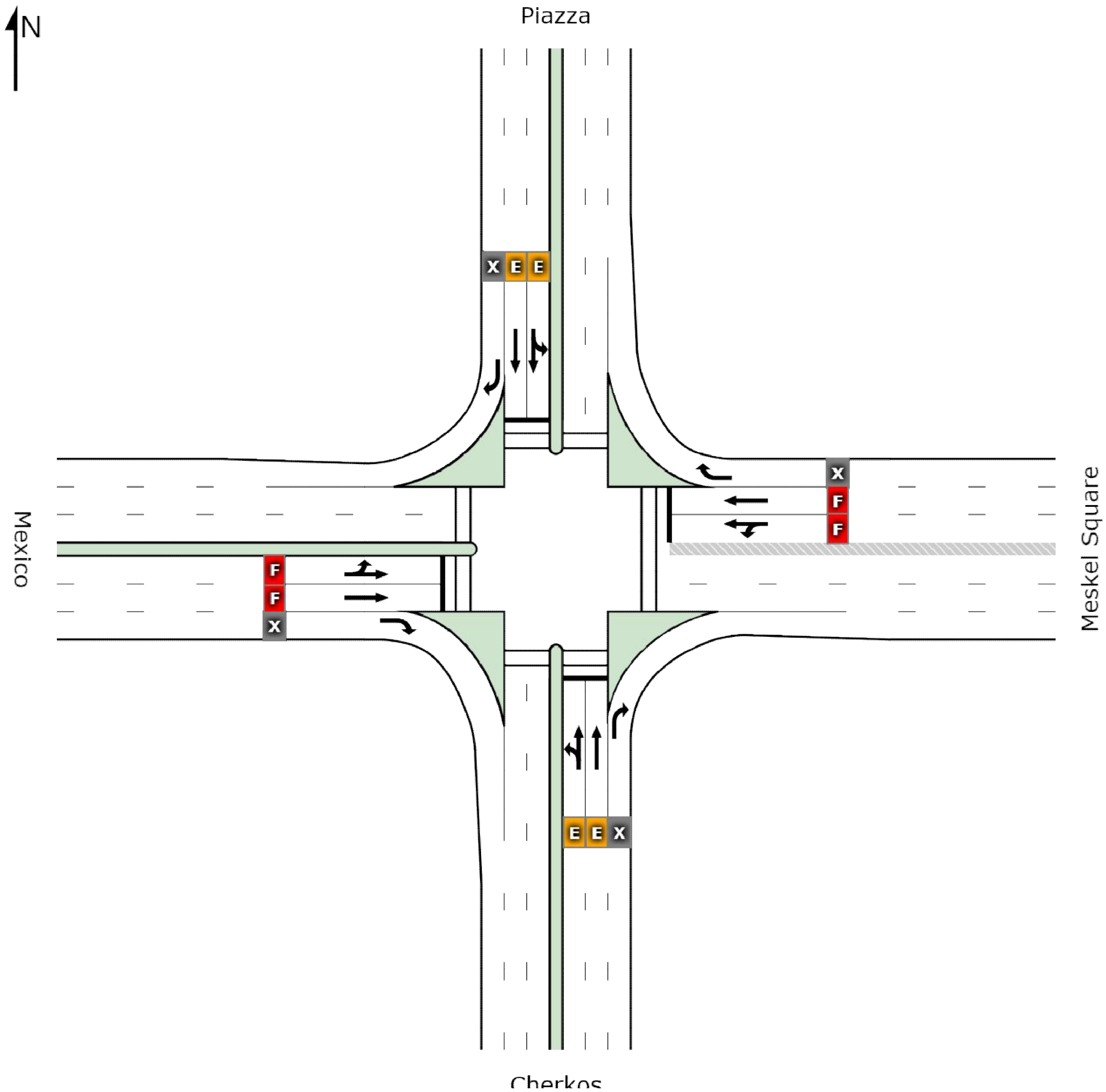
Approach	Movement	PHF
Mexico	Left	0.95
	Through	0.96
	Right	0.95
Meskel Square	Left	0.94
	Through	0.95
	Right	0.95
Cherkos	Left	0.95
	Through	0.96
	Right	0.95
Piazza	Left	0.94
	Through	0.95
	Right	0.94

Legahar Signalized Intersection Analysis Results in SIDRA Intersection Output

To evaluate the existing LOS SIDRA software program was used and lane LOS values are based on average delay per lane and Intersection and approach LOS values are based on average delay for all lanes and Table 5.36 below shows the detail.

Table 5.36-Output Summary of Legahar Signalized Intersection

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Cherkos											
1	L	184	0.0	0.858	75.6	LOS E	17.1	120.2	1.00	0.97	9.3
2	T	283	4.0	0.858	71.4	LOS E	17.3	125.5	1.00	0.99	9.2
3	R	208	2.0	0.118	3.4	X	X	X	X	0.41	34.9
Approach		676	2.3	0.858	51.6	LOS D	17.3	125.5	0.69	0.80	12.1
East: Meskel Square											
4	L	179	0.0	1.391	256.2	LOS F	97.4	690.8	1.00	1.69	3.3
5	T	1261	2.0	1.391	251.9	LOS F	98.3	699.8	1.00	1.72	3.2
6	R	323	2.0	0.177	3.4	X	X	X	X	0.41	34.9
Approach		1763	1.8	1.391	206.8	LOS F	98.3	699.8	0.82	1.47	3.9
North: Piazza											
7	L	236	0.0	0.866	73.8	LOS E	20.5	143.7	1.00	0.97	9.5
8	T	332	1.0	0.866	69.3	LOS E	21.2	149.6	1.00	0.99	9.4
9	R	242	0.0	0.135	3.4	X	X	X	X	0.41	34.9
Approach		809	0.4	0.866	50.9	LOS D	21.2	149.6	0.70	0.81	12.2
West: Mexico											
10	L	242	1.0	1.479	296.4	LOS F	103.3	733.7	1.00	1.77	2.9
11	T	1204	2.0	1.479	292.2	LOS F	104.8	746.3	1.00	1.81	2.8
12	R	174	2.0	0.095	3.4	X	X	X	X	0.41	34.9
Approach		1620	1.9	1.479	261.8	LOS F	104.8	746.3	0.89	1.65	3.1
All Vehicles		4868	1.7	1.479	177.7	LOS F	104.8	746.3	0.81	1.33	4.5



	South	East	North	West	Intersection
LOS	D	F	D	F	F

Figure 5.14- Level of Service Legahar Signalized Intersection

The results of capacity analysis show that the level of service (LOS) of Mexico and Meskel Square approaches were “F” and Piazza and Cherkos approaches were “E” however the overall level of service of the Legahar Signalized Intersection was “F”. Looking into delays at each approach, on Mexico approach it is 261.8 sec, on Meskel Square approach it is 207 sec, on Piazza approach it is 50.9 sec and Cherkos approach it is 51.6 sec. Hence, Mexico and Meskel Square approaches experience delays greater than the threshold values depicted in Table 2.4.

The problems observed at Legahar Signalized Intersection were related to:

- Inadequate Signal time;
- Inadequate entry & exit lane;
- Pedestrian volume; and
- Pavement distress.

Number of entry & exit lanes at Mexico and Meskel Square approaches was not sufficient for peak hour traffic volume and signal times allocated for these approaches were not enough to discharge traffic on the queue and these have great impact on lowering the capacity of the intersection. In addition, the number of pedestrians was high since the approaches were located near commercial centers. Pavement condition; potholes, rutting and corrugations were major pavement distresses observed on the approaches. These pavement distress conditions reduce vehicle speeds and contribute to delays as well as lower the capacity of the intersection. During the survey time, the Legahar Signalized intersection was not performing well and traffic flows were being regulated by traffic police.

5.3 Summary of Results

The results of analysis for the six intersections under this study are summarized in Table 5.37 and Table 5.38. The performances were measured in terms of v/c ratio or degree of saturation, average control delay per vehicle and level of service.LOS was applied using the HCM 2010 Manual. Detailed analysis results of SIDRA Intersection Output are available in Appendix-B.

Table 5.37- Summarized Result of Analyses for Roundabout Junctions.

Roundabout	Total Vehicle Flow	Degree of Saturation (V/C)	LOS
Gerji Imperial	5333	1.87	F
Bole Medihanialem	3467	1.24	F
Teklehaymanot	4508	1.17	F
Ayertena	5088	1.42	F

Table 5.38- Summarized Result of Analyses for Signalized Intersections.

Signalized Intersection	Demand Flow (Veh/h)	Average Delay (sec)	LOS
Legehar	4853	177.7	F
Post office	4027	81.5	F
Post office(for Current Signal Time)	4027	75	E

All roundabouts have higher entry flows and their degrees of saturation (v/c ratio) were also very high; more than 1 and the Level of service is “F”. All signalized intersections have average delay of greater than 80 seconds and level of service of “F”. Therefore, the capacity analysis results of selected representative intersections of the Addis Ababa city indicate that most of the intersections are in worse condition or serving over capacity during the peak period and the degree of saturation of all roundabout junctions is greater than 1; delays occurred at signalized junctions is greater than 80 sec and the level of service is “F”. The major factors affecting the capacity are: inadequate geometric nature, high traffic volume and drivers’ aggressive behavior.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Based on the findings of the analysis in this study, the following points are concluded

1. The result of analysis for selected representative intersections of the Addis Ababa City indicated that most of the intersections are in serious condition and during the peak periods the degree of saturation is almost greater than 2 for most of the intersections and the level of service is "F".
2. It is recognizable to see that at peak hours, the traffic police have to regulate the traffic situation at these intersections since traffic control devices cannot function or regulate the traffic. As the study revealed, the major problems are related to inadequacy of number of entry lanes, number of circulatory lanes, high traffic flow, high volume of pedestrians, inadequate effective green time and unbalanced traffic on the approaches.
3. Most of signalized intersections in the city are not functional, and serving as un-signalized Give- way intersection type since most of the signals were built many years ago and even the functioning signals are inadequate and traffic polices interfere to regulate traffic flow at peak hours.
4. Some Geometric elements of roundabouts used as input parameters for empirical method analysis do not exist at Addis Ababa Roundabouts; thus, analytical method was preferred to carry out analysis using aaSIDRA.
5. High traffic entry flows at Gerji, Teklehaymanot and Ayertena roundabouts were found to be more than 4000 vehicle per hour. This traffic volume is very high to be accommodated by the roundabouts. In addition, there are also high traffic flows at legs at Gerji, Teklehaymanot and Ayertena that show high percentage of traffic volume share, which is not recommended for the roundabouts.
6. Traffic congestion at junctions during the evening peak hour is more than the morning peak hours.

6.2 Recommendations

1. After additional study for the roundabout and signal intersections that have high and unbalanced traffic flow, their replacement with other junction type is recommended. The roundabouts, which are located at the ring road, are not providing the intended services since this device connects high-speed primary road and access road. Therefore, replacement of these roundabouts by other junction type is recommended after careful study.

2. It is better to separate the pedestrians from vehicular traffic at the intersections where high pedestrian flows were observed since they affect normal traffic flows.

3. The existing condition of Roundabout and signalized intersections in Addis Ababa need improvements to reduce their delay time and raise their level of service (LOS).

7. FUTURE RESEARCH PROPOSAL

The current method adopted by Addis Ababa City Road Authority (AACRA) is based on the HCM. However, The HCM is the capacity analysis manual developed in the United States. This manual is also based on the traffic condition of United States and do not consider local traffic conditions as well as travel behavior of road users that are unique in Ethiopia. Therefore, to overcome this problem further study need to be carried out to develop the Ethiopian Highway Capacity Manual (EHCM).

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APPENDIX-A (SUMMERIZED TRAFFIC DATA)

Table 1- Gerji-Imperial Roundabout

Approach Leg	Heavy Vehicle		Light Vehicle	Total Traffic(PCU)	Pedestrian
	Bus/D.Truck	Truck-Trailer			
8:00-9:00 AM	**Gerji-Imperial Roundabout**				
Bole Leg	217	69	1372	1835	495
Gerji Leg	61	7	747	853	353
Megenagna Leg	173	11	1462	1744	311
Hayahulet Leg	43	0	570	635	254
Total				5067	
9:00-10:00AM					
Bole Leg	205	65	1300	1737	777
Gerji Leg	58	7	708	809	666
Megenagna Leg	164	10	1385	1652	555
Hayahulet Leg	41	0	540	602	222
Friendship Leg	0	0	249	249	384
Total				5049	
4:00-5:00 PM					
Bole Leg	210	66	1328	1776	794
Gerji Leg	59	7	724	827	681
Megenagna Leg	168	10	1416	1689	567
Hayahulet Leg	42	2	552	619	227
Total				4911	
5:00-6:00 PM					
Bole Leg	228	72	1444	1930	863
Gerji Leg	65	8	787	899	616
Megenagna Leg	182	11	1539	1835	543
Hayahulet Leg	46	0	600	669	444
`				5333	2466

Table 2- Bole-Medihanialem Roundabout

Approach Leg	Heavy Vehicle		Light Vehicle	Total Traffic(PCU)	Pedestrian
	Bus/D.Truck	Truck-Trailer			
8:00-9:00 AM	**Bole-Medihanialem Roundabout**				
Bole Leg	32	0	704	753	311
Monico Leg	25	0	332	370	286
Hayahulet Leg	22	0	599	632	248
Kazanchis Leg	29	0	740	783	113
Friendship Leg	0	0	249	249	184
Total				2538	
9:00-10:00AM					
Bole Leg	29	0	630	673	225
Monico Leg	23	0	297	331	135
Hayahulet Leg	19	0	536	565	380
Kazanchis Leg	26	0	662	701	238
Friendship Leg	0	0	223	223	222
Total				2494	
4:00-5:00 PM					
Bole Leg	30	2	645	693	242
Monico Leg	23	0	304	339	345
Hayahulet Leg	20	3	549	585	194
Kazanchis Leg	26	1	678	719	153
Friendship Leg	2	0	262	265	114
Total				2336	
5:00-6:00 PM					
Bole Leg	34	0	741	792	395
Monico Leg	27	0	350	390	303
Hayahulet Leg	23	0	631	665	311
Kazanchis Leg	30	0	779	825	252
Friendship Leg	0	0	262	262	152
Total				**3467**	

Table 3- Teklehaymanot Roundabout

Approach Leg	Heavy Vehicle		Light Vehicle	Total Traffic(PCU)	Pedestrian
	Bus/D.Truck	Truck-Trailer			
8:00-9:00 AM	**Teklehaymanot Roundabout**				
Merkato Leg	173	0	1029	1289	972
Diafric Leg	43	8	578	659	109
Black Lion Leg	137	8	1051	1272	726
Piazza Leg	76	18	635	785	356
Total				4005	
9:00-10:00AM					
Merkato Leg	164	0	975	1221	769
Diafric Leg	41	7	547	623	652
Black Lion Leg	130	7	995	1205	535
Piazza Leg	72	17	602	744	285
Total				3793	
4:00-5:00 PM					
Merkato Leg	164	0	975	1221	869
Diafric Leg	41	7	547	623	651
Black Lion Leg	130	7	995	1205	635
Piazza Leg	72	17	602	744	285
Total				3793	
5:00-6:00 PM					
Merkato Leg	182	0	1083	1357	870
Diafric Leg	46	8	608	692	622
Black Lion Leg	144	8	1106	1338	547
Piazza Leg	80	19	669	887	448
Total				4507	2488

Table 4- Ayer-Tena Roundabout

Approach Leg	Heavy Vehicle		Light Vehicle	Total Traffic(PCU)	Pedestrian
	Bus/D.Truck	Truck-Trailer			
8:00-9:00 AM	**Ayer-Tena Roundabout**				
Alemgena Leg	253	29	866	1303	726
Jomo Leg	181	22	722	1037	603
mexico Leg	224	29	1011	1405	850
Alembank Leg	94	29	650	849	863
Total				4593	
9:00-10:00AM					
Alemgena Leg	231	26	793	1193	581
Jomo Leg	165	20	661	949	468
mexico Leg	205	26	926	1286	1694
Alembank Leg	86	26	595	777	790
Total				4205	
4:00-5:00 PM					
Alemgena Leg	231	26	791	1189	545
Jomo Leg	165	20	659	946	434
mexico Leg	204	26	922	1282	655
Alembank Leg	86	26	593	774	772
Total				4191	
5:00-6:00 PM					
Alemgena Leg	266	30	912	1372	872
Jomo Leg	190	23	760	1091	622
mexico Leg	236	30	1064	1478	548
Alembank Leg	99	30	684	893	488
Total				5089	2490

Table 5- Post-office Intersection

	Left Turn	Through	Right Turn	Total Ve-hicles(PCU)	Pedestrian
7:00-8:00 AM	**Post-office Intersection**				
Black Lion Leg	72	403	72	547	465
Legehar Leg	103	745	61	908	374
Filwuha Leg	129	281	80	490	457
Piazza Leg	68	817	99	984	582
Total				2930	
8:00-9:00 AM					
Black Lion Leg	69	383	69	520	302
Legehar Leg	98	715	58	872	395
Filwuha Leg	121	264	75	461	483
Piazza Leg	64	760	92	915	614
Total				2768	
4:00-5:00 PM					
Black Lion Leg	67	371	67	504	280
Legehar Leg	99	715	58	872	382
Filwuha Leg	122	265	75	461	467
Piazza Leg	62	737	89	888	595
Total				2726	
5:00-6:00 PM					
Black Lion Leg	69	787	69	925	481
Legehar Leg	101	730	60	990	327
Filwuha Leg	125	273	77	975	288
Piazza Leg	65	776	94	1137	236
Total				4027	1310

Table 6- Legehar Intersection

	Left Turn	Through	Right Turn	Total Ve-hicles(PCU)	Pedestrian
7:00-8:00 AM	**Legehar Intersection**				
Mexico Leg	163	952	188	1303	813
Meskel sq, Leg	191	990	218	1399	457
Cherkos Leg	124	140	190	455	559
Piazza Leg	159	82	163	404	711
Total				3561	
8:00-9:00 AM					
Mexico Leg	211	1232	243	1686	961
Meskel sq, Leg	248	1282	282	1811	540
Cherkos Leg	161	182	246	589	660
Piazza Leg	206	106	211	523	840
Total				4608	
4:00-5:00 PM					
Mexico Leg	192	1120	221	1533	887
Meskel sq, Leg	225	1165	256	1646	499
Cherkos Leg	146	165	224	535	610
Piazza Leg	187	96	192	475	776
Total				4189	
5:00-6:00 PM					
Mexico Leg	230	1344	265	1840	886
Meskel sq, Leg	270	1398	307	1975	633
Cherkos Leg	175	198	269	842	557
Piazza Leg	224	115	230	770	456
Total				4853	2532

APPENDIX –B (GEOMETRIC FEATURE AND SIDRA OUT PUT)

Roundabout Basic Parameters
Site:GERJI IMPERIAL BOBMARLE ROUNDABOUT

Intersection ID: 1
Roundabout

```
---------------------------------------------------------------------------------
                                            Circulating/Exiting Stream
 Cent  Circ  Insc  Ent Ent Cir Ent Av.Ent -----------------------------------
Island Width Diam. Rad Ang Lan Lan Lane   Flow %HV Adjust. %Exit  Cap.    O-D
 Diam                              Width            Flow   Incl. Constr. Factor
   m     m     m    m  deg           m   veh/h      pcu/h        Effect
---------------------------------------------------------------------------------
South: Bole Airport
  Environment Factor: 1.10   Entry/Circulating Flow Adjustment: Medium
  21    12    45   20  30   2   2   3.70  830  3.3   830     0     Y     0.848
---------------------------------------------------------------------------------
East: Gerji
  Environment Factor: 1.10   Entry/Circulating Flow Adjustment: Medium
  21    12    45   20  30   2   2   3.80  870  5.7   883     0     Y     0.803
---------------------------------------------------------------------------------
North: Megenagna
  Environment Factor: 1.10   Entry/Circulating Flow Adjustment: Medium
  21    12    45   20  30   2   2   3.70  673  3.1   674     0     Y     0.874
---------------------------------------------------------------------------------
West: Hayahulet
  Environment Factor: 1.10   Entry/Circulating Flow Adjustment: Medium
  21    12    45   20  30   2   2   3.80 1164  3.9  1167     0     Y     0.736
---------------------------------------------------------------------------------
  HV Method for Gap Acceptance: Include HV effect if above 5 per cent
  Roundabout Capacity Model: SIDRA Standard
```

Intersection ID: 1
Roundabout

Turn	Lane No.	Lane Type	Circulating/Exiting Stream Flow Rate pcu/h	Aver Speed km/h	Aver Dist m	In-Bnch Headway sec	Prop Bunched	Critical Gap Hdwy sec	Critical Gap Dist m	Foll-up Headway sec
South: Bole Airport										
Environment Factor: 1.10			Entry/Circulating Flow Adjustment: Medium							
Left	1	Subdominant	830	20.0	24.1	1.44	0.523	3.88	21.6	2.53
Thru	1	Subdominant	830	20.0	24.1	1.44	0.523	3.98	22.1	2.59
	2	Dominant	830	20.0	24.1	1.44	0.523	3.72	20.7	2.42
Right	2	Dominant	830	20.0	24.1	1.44	0.523	3.63	20.2	2.36
East: Gerji										
Environment Factor: 1.10			Entry/Circulating Flow Adjustment: Medium							
Left	1	Subdominant	883	20.0	22.7	1.36	0.525	3.75	20.8	2.51
Thru	1	Subdominant	883	20.0	22.7	1.36	0.525	3.75	20.8	2.52
	2	Dominant	883	20.0	22.7	1.36	0.525	3.49	19.4	2.34
Right	2	Dominant	883	20.0	22.7	1.36	0.525	3.49	19.4	2.34
North: Megenagna										
Environment Factor: 1.10			Entry/Circulating Flow Adjustment: Medium							
Left	1	Subdominant	674	20.0	29.7	1.78	0.523	4.00	22.2	2.55
Thru	1	Subdominant	674	20.0	29.7	1.78	0.523	4.02	22.3	2.56
	2	Dominant	674	20.0	29.7	1.78	0.523	3.82	21.2	2.43
Right	2	Dominant	674	20.0	29.7	1.78	0.523	3.80	21.1	2.42
West: Hayahulet										
Environment Factor: 1.10			Entry/Circulating Flow Adjustment: Medium							
Left	1	Subdominant	1167	20.0	17.1	1.34	0.629	3.56	19.8	2.47
Thru	1	Subdominant	1167	20.0	17.1	1.34	0.629	3.56	19.8	2.47
	2	Dominant	1167	20.0	17.1	1.34	0.629	3.23	17.9	2.24
Right	2	Dominant	1167	20.0	17.1	1.34	0.629	3.22	17.9	2.24

Roundabout Capacity Model: SIDRA Standard

Priority sharing is implied for some movements (Follow-up Headway plus Intra-bunch Headway is larger than the Critical Gap). The O-D Factor (Roundabout Basic Parameters table) allows for priority sharing and priority emphasis.

Dist (Distance): Spacing, i.e. distance between the front ends of two successive vehicles across all lanes in the circulating or exiting stream

MOVEMENT SUMMARY

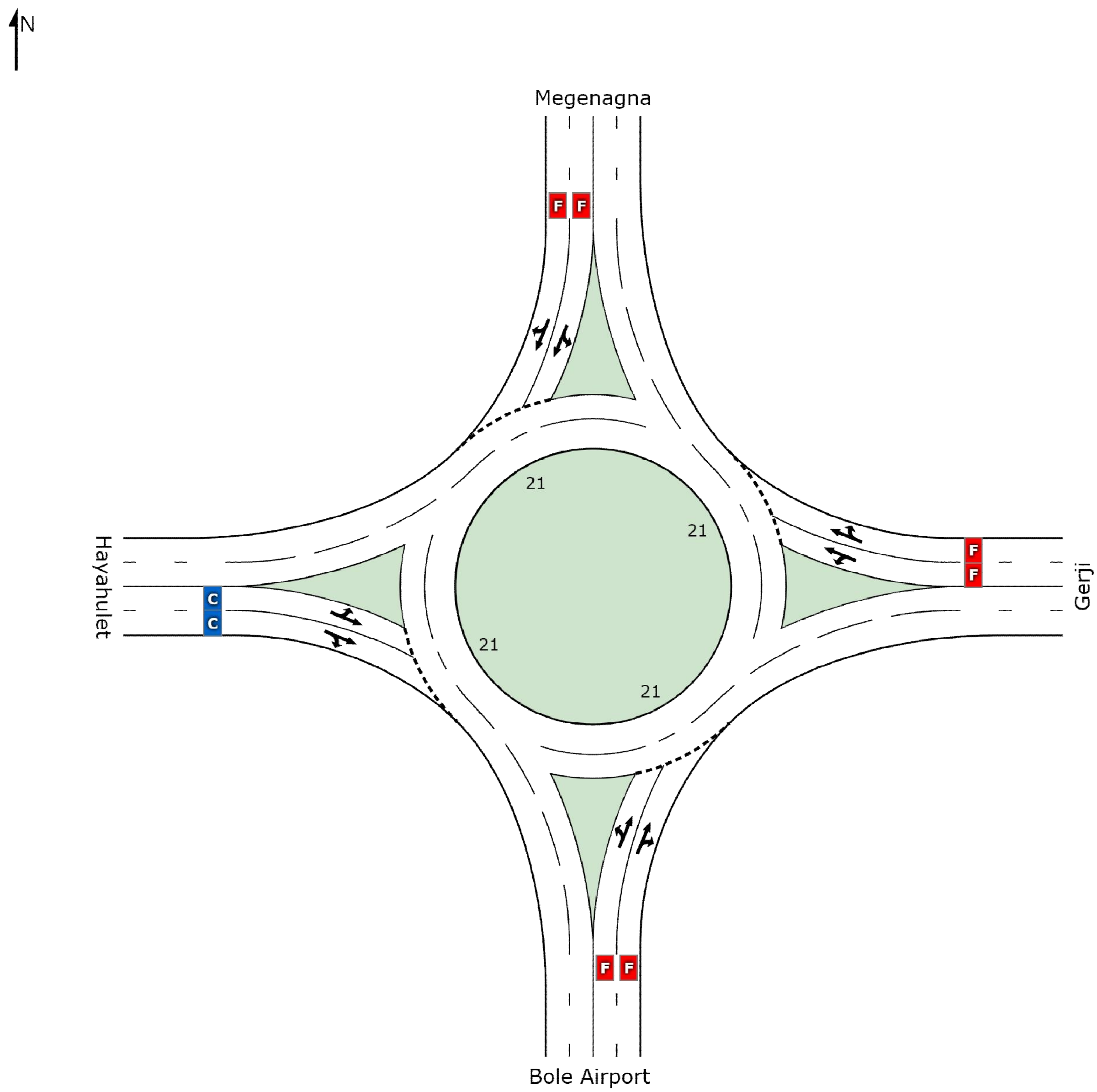
Site: GERJI IMPERIAL BOBMARLE ROUNDABOUT

GERJI IMPERIAL BOBMARLE ROUNDABOUT

Roundabout

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: Bole Airport											
1	L	350	4.0	1.874	406.6	LOS F	136.5	1005.5	1.00	12.99	4.0
2	T	966	7.5	1.874	406.1	LOS F	150.3	1099.1	1.00	13.26	3.9
3	R	481	3.0	1.874	405.5	LOS F	150.3	1099.1	1.00	13.77	3.7
Approach		1798	5.6	1.874	406.0	LOS F	150.3	1099.1	1.00	13.35	3.9
East: Gerji											
4	L	334	2.0	1.152	89.5	LOS F	32.8	235.0	1.00	4.93	10.6
5	T	226	3.7	1.152	89.3	LOS F	35.8	253.3	1.00	4.95	10.4
6	R	563	1.0	1.152	87.9	LOS F	35.8	253.3	1.00	5.14	10.3
Approach		1123	1.8	1.152	88.6	LOS F	35.8	253.3	1.00	5.04	10.4
North: Megenagna											
7	L	395	3.0	1.501	241.3	LOS F	93.0	673.5	1.00	9.77	5.9
8	T	918	5.2	1.501	240.8	LOS F	99.1	718.4	1.00	9.98	5.8
9	R	274	2.0	1.501	240.5	LOS F	99.1	718.4	1.00	10.15	5.6
Approach		1586	4.1	1.501	240.8	LOS F	99.1	718.4	1.00	9.96	5.8
West: Hayahulet											
10	L	167	2.0	0.847	19.5	LOS C	7.3	52.4	0.94	1.64	16.8
11	T	400	4.0	0.847	18.4	LOS C	7.9	56.3	0.95	1.65	16.8
12	R	199	1.0	0.847	17.5	LOS C	7.9	56.3	0.96	1.66	16.8
Approach		766	2.8	0.847	18.4	LOS C	7.9	56.3	0.95	1.65	16.8
All Vehicles		5273	3.9	1.874	232.4	LOS F	150.3	1099.1	0.99	8.86	5.9

Level of Service (LOS) Method: Degree of Saturation (SIDRA METHOD).
Roundabout LOS Method: SIDRA Roundabout LOS.
Vehicle movement LOS values are based on degree of saturation per movement
Intersection and Approach LOS values are based on worst degree of saturation for any vehicle movement.
Roundabout Capacity Model: SIDRA Standard.
SIDRA Standard Delay Model used.



	South	East	North	West	Intersection
LOS	F	F	F	C	F

Level of Service (LOS) Method: Degree of Saturation (SIDRA METHOD).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on degree of saturation per lane.
Intersection and Approach LOS values are based on worst degree of saturation for any lane.

Intersection ID: 2
Roundabout

Cent Island Diam m	Circ Width m	Insc Diam. m	Ent Rad m	Ent Ang deg	Cir Lan	Ent Lan	Av.Ent Lane Width m	Circulating/Exiting Stream Flow veh/h	%HV	Adjust. Flow pcu/h	%Exit Incl.	Cap. Constr. Effect	O-D Factor
South: Bole Airport													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
36	11	58	20	30	3	2	3.30	706	0.5	706	0	Y	0.880
SouthEast: Gipesen Acadami													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
36	11	58	20	30	3	1	3.20	1229	1.2	1229	0	Y	0.771
NorthEast: Hayahulet													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
36	11	58	20	30	3	2	3.10	932	1.9	933	0	Y	0.845
North: Kazanchis													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
36	11	58	20	30	3	2	3.65	964	1.8	964	0	Y	0.822
West: Freindship													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
36	11	58	20	30	3	1	3.20	1361	1.6	1361	0	Y	0.741

HV Method for Gap Acceptance: Include HV effect if above 5 per cent
Roundabout Capacity Model: SIDRA Standard

Roundabout Gap Acceptance Parameters
Site:BOLE MEDIHANIALEM ROUNDABOUT

Intersection ID: 2
Roundabout

Turn	Lane No.	Lane Type	Circulating/Exiting Stream: Flow Rate pcu/h	Aver Speed km/h	Aver Dist m	In-Bnch Headway sec	Prop Bunched	Critical Gap: Hdwy sec	Critical Gap: Dist m	Foll-up Headway sec
South: Bole Airport										
Environment Factor: 1.10 Entry/Circulating Flow Adjustment: Medium										
Left	1	Subdominant	706	27.9	39.4	2.00	0.587	3.74	28.9	2.63D
Thru	1	Subdominant	706	27.9	39.4	2.00	0.587	3.74	28.9	2.63D
	2	Dominant	706	27.9	39.4	2.00	0.587	3.74	28.9	2.63
Right	2	Dominant	706	27.9	39.4	2.00	0.587	3.74	28.9	2.63
SouthEast: Gipesen Acadami										
Environment Factor: 1.10 Entry/Circulating Flow Adjustment: Medium										
Left	1	Dominant	1229	31.6	25.7	1.32	0.644	3.86	33.9	2.84
Right	1	Dominant	1229	31.6	25.7	1.32	0.644	3.86	33.9	2.84
NorthEast: Hayahulet										
Environment Factor: 1.10 Entry/Circulating Flow Adjustment: Medium										
Left	1	Dominant	933	31.4	33.7	1.61	0.612	3.66	31.9	2.55
Right	2	Subdominant	933	31.4	33.7	1.61	0.612	3.96	34.5	2.75
North: Kazanchis										
Environment Factor: 1.10 Entry/Circulating Flow Adjustment: Medium										
Left	1	Subdominant	964	28.5	29.6	1.46	0.586	3.23	25.5	2.59
Thru	1	Subdominant	964	28.5	29.6	1.46	0.586	3.23	25.6	2.59
	2	Dominant	964	28.5	29.6	1.46	0.586	3.16	25.0	2.54
Right	2	Dominant	964	28.5	29.6	1.46	0.586	3.16	25.0	2.54
West: Freindship										
Environment Factor: 1.10 Entry/Circulating Flow Adjustment: Medium										
Left	1	Dominant	1361	29.9	22.0	1.45	0.728	3.74	31.1	2.79
Right	1	Dominant	1361	29.9	22.0	1.45	0.728	3.74	31.1	2.79

Roundabout Capacity Model: SIDRA Standard
Priority sharing is implied for some movements (Follow-up Headway plus Intra-bunch Headway is larger than the Critical Gap). The O-D Factor (Roundabout Basic Parameters table) allows for priority sharing and priority e

D Subdominant lane follow-up headway was calculated as less than the dominant lane value and was set to the dominant lane value

Dist (Distance): Spacing, i.e. distance between the front ends of two successive vehicles across all lanes in the circulating or exiting stream

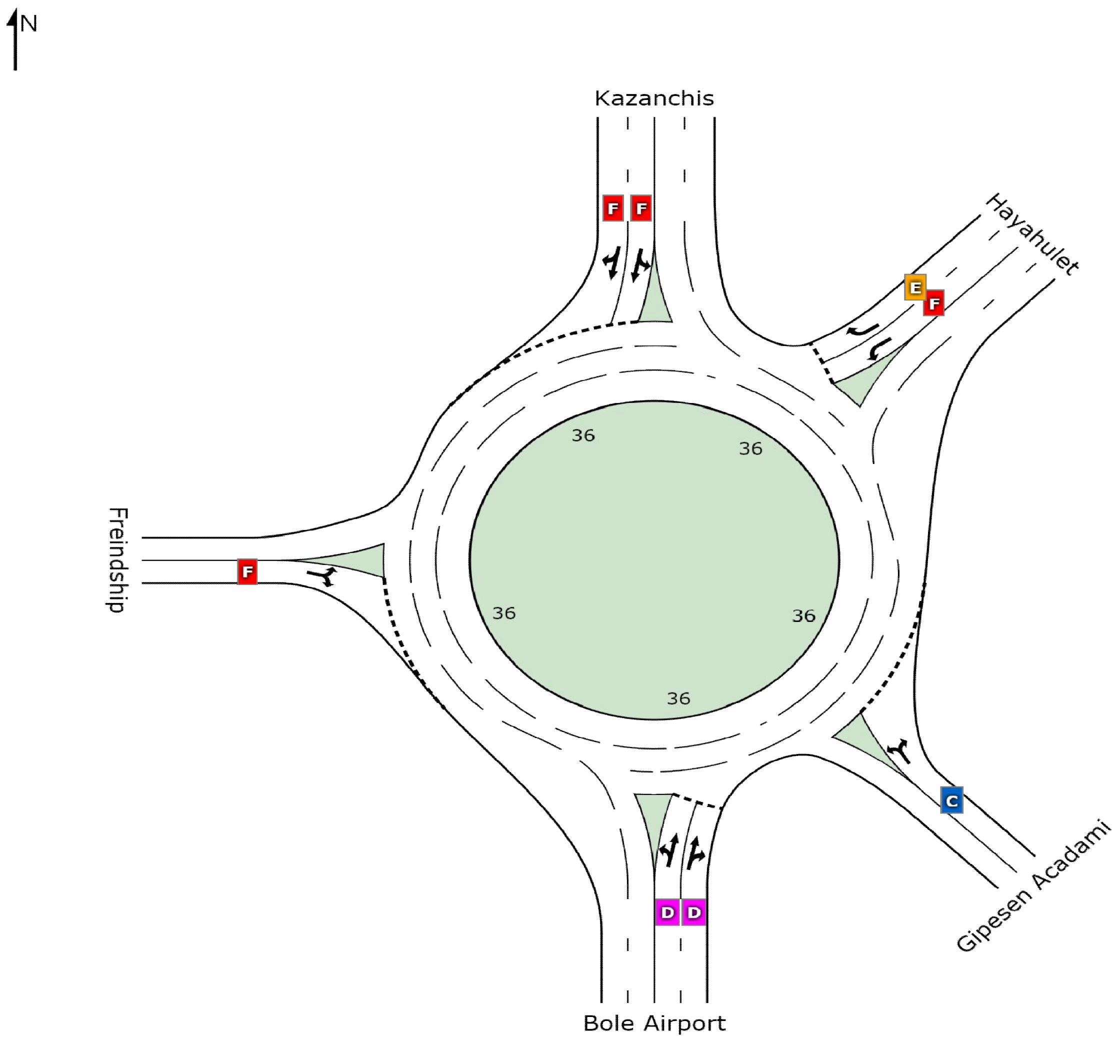
Movement Performance - Vehicles												
Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue		Prop. Queued	Effec- tive Stop Rate	Average Speed
							Vehicles	Distance			
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Bole Airport											
1	L	152	1.0	0.866	21.9	LOS D	8.9	63.5	0.89	1.49	21.0
2	T	429	2.0	0.866	18.4	LOS D	8.9	63.5	0.89	1.42	20.3
3	R	311	0.8	0.866	18.6	LOS D	8.9	63.2	0.89	1.42	19.9
Approach		892	1.4	0.866	19.1	LOS D	8.9	63.5	0.89	1.43	20.3
South East: Gipesen Acadami											
21	L	221	3.3	0.793	21.5	LOS C	6.4	45.6	0.92	1.29	20.8
23	R	98	1.0	0.793	18.1	LOS C	6.4	45.6	0.92	1.25	19.5
Approach		319	2.6	0.793	20.5	LOS C	6.4	45.6	0.92	1.28	20.4
North East: Hayahulet											
24	L	557	1.5	1.236	131.7	LOS F	44.4	315.0	1.00	4.20	6.5
26	R	401	1.0	0.998	43.2	LOS E	14.0	98.5	0.93	2.06	12.6
Approach		958	1.3	1.236	94.7	LOS F	44.4	315.0	0.97	3.30	7.9
North: Kazanchis											
7	L	376	1.0	1.005	42.2	LOS F	15.1	107.0	1.00	2.13	15.1
8	T	457	2.0	1.005	38.2	LOS F	15.5	109.5	1.00	2.14	14.2
9	R	125	0.0	1.005	38.1	LOS F	15.5	109.5	1.00	2.15	13.6
Approach		958	1.3	1.005	39.7	LOS F	15.5	109.5	1.00	2.14	14.5
West: Freindship											
10	L	178	0.0	1.008	60.3	LOS F	15.2	106.4	1.00	2.01	11.8
12	R	161	0.0	1.008	57.1	LOS F	15.2	106.4	1.00	2.01	10.3
Approach		339	0.0	1.008	58.8	LOS F	15.2	106.4	1.00	2.01	11.1
All Vehicles		3465	1.3	1.236	49.7	LOS F	44.4	315.0	0.96	2.19	12.5

Level of Service (LOS) Method: Degree of Saturation (SIDRA METHOD).
Roundabout LOS Method: SIDRA Roundabout LOS.
Vehicle movement LOS values are based on degree of saturation per movement
Intersection and Approach LOS values are based on worst degree of saturation for any vehicle movement.
Roundabout Capacity Model: SIDRA Standard.

LEVEL OF SERVICE SUMMARY

Site: BOLE MEDIHANIALEM ROUNDABOUT

Roundabout with 5 legs, and 2-lane approaches & circulating road
Needs attention to turn designations and matching lane disciplines
Roundabout



	South	Southeast	Northeast	North	West	Intersection
LOS	D	C	F	F	F	F

Level of Service (LOS) Method: Degree of Saturation (SIDRA METHOD).

Roundabout LOS Method: SIDRA Roundabout LOS.

Lane LOS values are based on degree of saturation per lane.

Intersection and Approach LOS values are based on worst degree of saturation for any lane.

SIDRA Standard Delay Model used.

Roundabouts

Roundabout Basic Parameters
Site:TEKLEHAYMANOT RA

Intersection ID: 1
Roundabout

Cent Island Diam m	Circ Width m	Insc Diam. m	Ent Rad m	Ent Ang deg	Cir Lan	Ent Lan	Av.Ent Lane Width m	Circulating/Exiting Stream Flow veh/h	%HV	Adjust. Flow pcu/h	%Exit Incl.	Cap. Constr. Effect	O-D Factor
South: Black Lion													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
65	10	85	20	30	2	2	3.20	829	1.8	830	0	Y	0.833
East: piazza													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
40	10	60	20	30	2	2	3.00	1633	4.2	1642	0	Y	0.661
North: Merkatto													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
65	10	85	20	30	2	2	3.60	902	4.0	904	0	Y	0.837
West: Abinet													
Environment Factor: 1.10							Entry/Circulating Flow Adjustment: Medium						
40	10	60	20	30	2	2	3.60	1065	5.5	1080	0	Y	0.735

HV Method for Gap Acceptance: Include HV effect if above 5 per cent
Roundabout Capacity Model: SIDRA Standard

Roundabout Gap Acceptance Parameters
Site:TEKLEHAYMANOT RA

Intersection ID: 1
Roundabout

Turn	Lane No.	Lane Type	Circulating/Exiting Stream: Flow Rate pcu/h	Aver Speed km/h	Aver Dist m	In-Bnch Headway sec	Prop Bunched	Critical Gap: Hdwy sec	Critical Gap: Dist m	Foll-up Headway sec
South: Black Lion										
Environment Factor: 1.10		Entry/Circulating Flow Adjustment: Medium								
Left	1	Subdominant	830	29.9	36.1	1.59	0.561	3.96	32.9	2.32
Thru	1	Subdominant	830	29.9	36.1	1.59	0.561	4.00	33.3	2.35
	2	Dominant	830	29.9	36.1	1.59	0.561	3.39	28.2	1.99
Right	2	Dominant	830	29.9	36.1	1.59	0.561	3.35	27.9	1.96
East: piazza										
Environment Factor: 1.10		Entry/Circulating Flow Adjustment: Medium								
Left	1	Subdominant	1642	30.0	18.2	1.29	0.758	3.79	31.5	2.27
Thru	1	Subdominant	1642	30.0	18.2	1.29	0.758	3.81	31.7	2.28
	2	Dominant	1642	30.0	18.2	1.29	0.758	3.14	26.1	1.88
Right	2	Dominant	1642	30.0	18.2	1.29	0.758	3.12	26.0	1.87
North: Merkatto										
Environment Factor: 1.10		Entry/Circulating Flow Adjustment: Medium								
Left	1	Subdominant	904	30.0	33.2	1.41	0.547	3.58	29.8	2.30
Thru	1	Subdominant	904	30.0	33.2	1.41	0.547	3.66	30.4	2.35
	2	Dominant	904	30.0	33.2	1.41	0.547	3.07	25.6	1.98
Right	2	Dominant	904	30.0	33.2	1.41	0.547	3.01	25.1	1.94
West: Abinet										
Environment Factor: 1.10		Entry/Circulating Flow Adjustment: Medium								
Left	1	Dominant	1080	30.0	27.8	1.27	0.575	3.17	26.4	2.09
Thru	1	Dominant	1080	30.0	27.8	1.27	0.575	3.17	26.4	2.08
	2	Subdominant	1080	30.0	27.8	1.27	0.575	3.64	30.3	2.39
Right	2	Subdominant	1080	30.0	27.8	1.27	0.575	3.64	30.4	2.39

Roundabout Capacity Model: SIDRA Standard
Priority sharing is implied for some movements (Follow-up Headway plus Intra-bunch Headway is larger than the Critical Gap). The O-D Factor (Roundabout Basic Parameters table) allows for priority sharing and priority emphasis.

Dist (Distance): Spacing, i.e. distance between the front ends of two successive vehicles across all lanes in the circulating or exiting stream

MOVEMENT SUMMARY

Site: TEKLEHAYMANOT RA

Roundabout with 2-lane approaches and circulating road
MUTCD (FHWA 2009) example number: 3C-6
Roundabout Guide (TRB 2010) example number: A-7
Roundabout

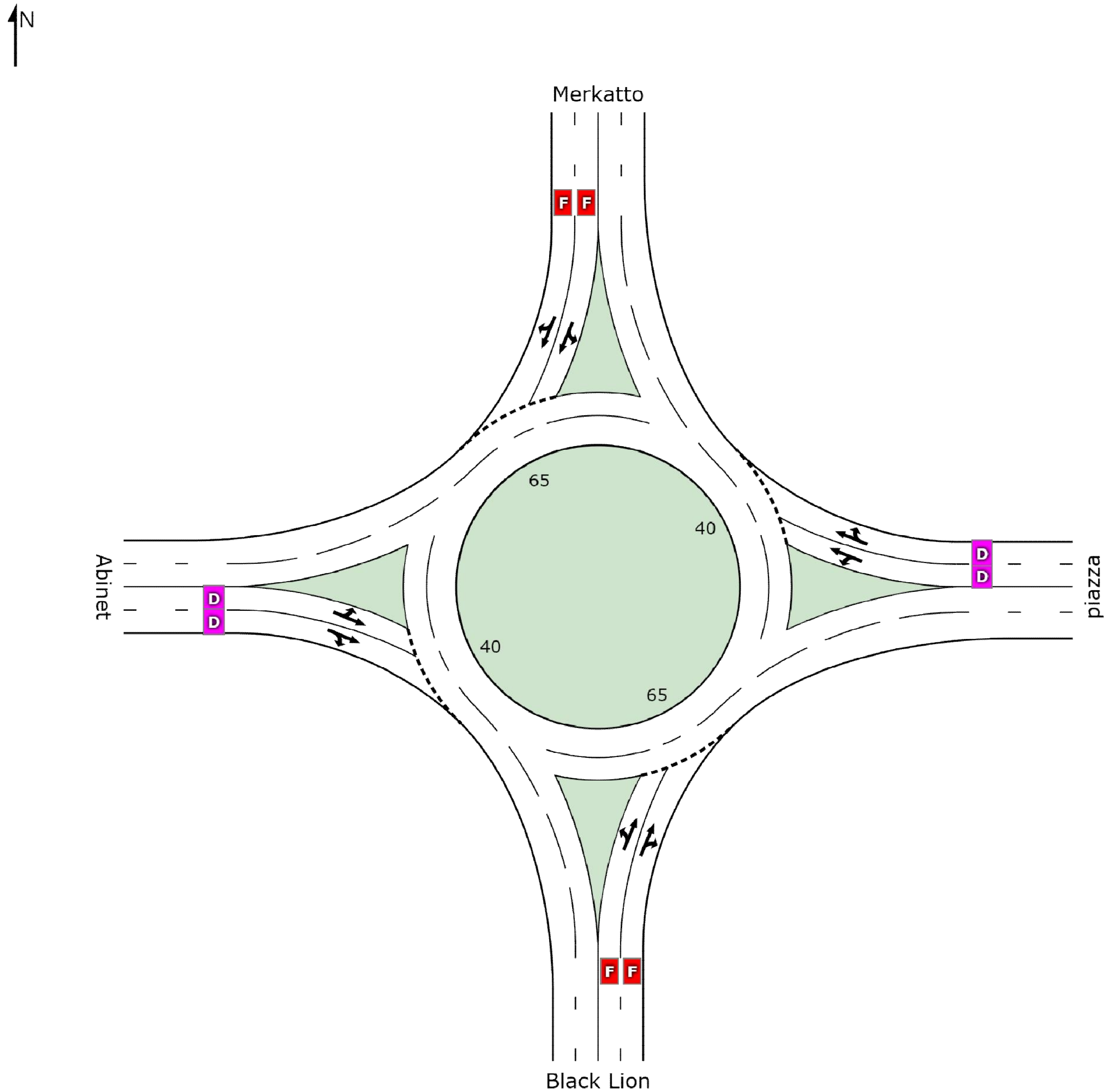
Movement Performance - Vehicles											
Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue		Prop. Queued	Effec-tive Stop Rate	Average Speed
							Vehicles	Dis-tance			
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: Black Lion											
1	L	418	3.0	1.164	96.7	LOS F	45.4	329.3	1.00	4.03	13.1
2	T	942	6.0	1.164	94.2	LOS F	55.3	403.8	1.00	4.33	12.6
3	R	167	1.0	1.164	93.2	LOS F	55.3	403.8	1.00	4.46	12.2
Approach		1527	4.6	1.164	94.8	LOS F	55.3	403.8	1.00	4.26	12.7
East: piazza											
4	L	146	3.0	0.919	50.1	LOS D	10.3	74.5	1.00	1.77	17.8
5	T	397	5.3	0.919	44.7	LOS D	12.5	91.2	1.00	1.85	18.1
6	R	87	2.0	0.919	42.5	LOS D	12.5	91.2	1.00	1.88	17.9
Approach		630	4.3	0.919	45.7	LOS D	12.5	91.2	1.00	1.84	18.0
North: Merkatto											
7	L	249	2.0	1.171	97.1	LOS F	42.8	312.5	1.00	4.28	13.1
8	T	828	7.0	1.171	95.3	LOS F	53.1	389.3	1.00	4.51	12.5
9	R	415	4.0	1.171	93.6	LOS F	53.1	389.3	1.00	4.78	12.2
Approach		1492	5.3	1.171	95.2	LOS F	53.1	389.3	1.00	4.55	12.5
West: Abinet											
10	L	464	2.0	0.908	19.8	LOS D	10.5	74.8	0.98	1.66	23.6
11	T	153	1.0	0.908	22.0	LOS D	10.5	74.8	0.97	1.64	22.7
12	R	298	4.0	0.908	22.9	LOS D	9.4	67.8	0.96	1.63	22.0
Approach		915	2.5	0.908	21.2	LOS D	10.5	74.8	0.97	1.65	23.0
All Vehicles		4564	4.4	1.171	73.4	LOS F	55.3	403.8	0.99	3.50	14.5

Level of Service (LOS) Method: Degree of Saturation (SIDRA METHOD).
Roundabout LOS Method: SIDRA Roundabout LOS.
Vehicle movement LOS values are based on degree of saturation per movement
Intersection and Approach LOS values are based on worst degree of saturation for any vehicle movement.
Roundabout Capacity Model: SIDRA Standard.

SIDRA Standard Delay Model used.

LEVEL OF SERVICE SUMMARY

Site: TEKLEHAYMANOT RA



	South	East	North	West	Intersection
LOS	F	D	F	D	F

Level of Service (LOS) Method: Degree of Saturation (SIDRA METHOD).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on degree of saturation per lane.
Intersection and Approach LOS values are based on worst degree of saturation for any lane.

SIDRA Standard Delay Model used.

Roundabouts

Roundabout Basic Parameters
Site:AYERTENA ROUNDABOUT

Intersection ID: 1
Roundabout

								Circulating/Exiting Stream					
Cent Island Diam m	Circ Width m	Insc Diam. m	Ent Rad m	Ent Ang deg	Cir Lan	Ent Lan	Av.Ent Lane Width m	Flow veh/h	%HV	Adjust. Flow pcu/h	%Exit Incl.	Cap. Constr. Effect	O-D Factor
South: JOMO													
Environment Factor: 1.00							Entry/Circulating Flow Adjustment: Medium						
60	10	80	20	30	2	2	3.60	1183	8.8	1240	0	Y	0.703
East: MEXICO													
Environment Factor: 1.00							Entry/Circulating Flow Adjustment: Medium						
40	10	60	20	30	2	2	3.60	1255	9.1	1308	0	Y	0.694
North: ALEMBANK													
Environment Factor: 1.00							Entry/Circulating Flow Adjustment: Medium						
60	10	80	20	30	2	2	3.20	1219	8.2	1261	0	Y	0.708
West: ALEMGENA													
Environment Factor: 1.00							Entry/Circulating Flow Adjustment: Medium						
40	10	60	20	30	2	2	3.00	1073	5.5	1091	0	Y	0.759

HV Method for Gap Acceptance: Include HV effect if above 5 per cent
Roundabout Capacity Model: SIDRA Standard

Roundabout Gap Acceptance Parameters
Site:AYERTENA ROUNDABOUT

Intersection ID: 1
Roundabout

Turn	Lane No.	Lane Type	Circulating/Exiting Stream: Flow Rate pcu/h	Aver Speed km/h	Aver Dist m	In-Bnch Headway sec	Prop Bunched	Critical Gap: Hdwy sec	Critical Gap: Dist m	Foll-up Headway sec
South: JOMO										
Environment Factor: 1.00		Entry/Circulating Flow Adjustment: Medium								
Left	1	Subdominant	1240	29.9	24.1	1.19	0.606	3.16	26.2	2.11
Thru	1	Subdominant	1240	29.9	24.1	1.19	0.606	3.37	28.0	2.25
	2	Dominant	1240	29.9	24.1	1.19	0.606	2.62	21.8	1.75
Right	2	Dominant	1240	29.9	24.1	1.19	0.606	2.45	20.3	1.63
East: MEXICO										
Environment Factor: 1.00		Entry/Circulating Flow Adjustment: Medium								
Left	1	Subdominant	1308	29.9	22.9	1.24	0.644	3.29	27.3	2.21
Thru	1	Subdominant	1308	29.9	22.9	1.24	0.644	3.48	28.9	2.33
	2	Dominant	1308	29.9	22.9	1.24	0.644	2.80	23.3	1.88
Right	2	Dominant	1308	29.9	22.9	1.24	0.644	2.64	22.0	1.77
North: ALEMBANK										
Environment Factor: 1.00		Entry/Circulating Flow Adjustment: Medium								
Left	1	Subdominant	1261	29.9	23.7	1.24	0.628	3.42	28.4	2.09
Thru	1	Subdominant	1261	29.9	23.7	1.24	0.628	3.52	29.2	2.16
	2	Dominant	1261	29.9	23.7	1.24	0.628	2.73	22.7	1.68
Right	2	Dominant	1261	29.9	23.7	1.24	0.628	2.65	22.0	1.63
West: ALEMGENA										
Environment Factor: 1.00		Entry/Circulating Flow Adjustment: Medium								
Left	1	Subdominant	1091	29.9	27.4	1.28	0.583	3.93	32.6	2.28
Thru	1	Subdominant	1091	29.9	27.4	1.28	0.583	4.20	34.8	2.43
	2	Dominant	1091	29.9	27.4	1.28	0.583	3.45	28.7	2.00
Right	2	Dominant	1091	29.9	27.4	1.28	0.583	3.20	26.6	1.85

Roundabout Capacity Model: SIDRA Standard
Priority sharing is implied for some movements (Follow-up Headway plus Intra-bunch Headway is larger than the Critical Gap). The O-D Factor (Roundabout Basic Parameters table) allows for priority sharing and priority emphasis.

Dist (Distance): Spacing, i.e. distance between the front ends of two successive vehicles across all lanes in the circulating or exiting stream

MOVEMENT SUMMARY

Site: AYERTENA ROUNDABOUT

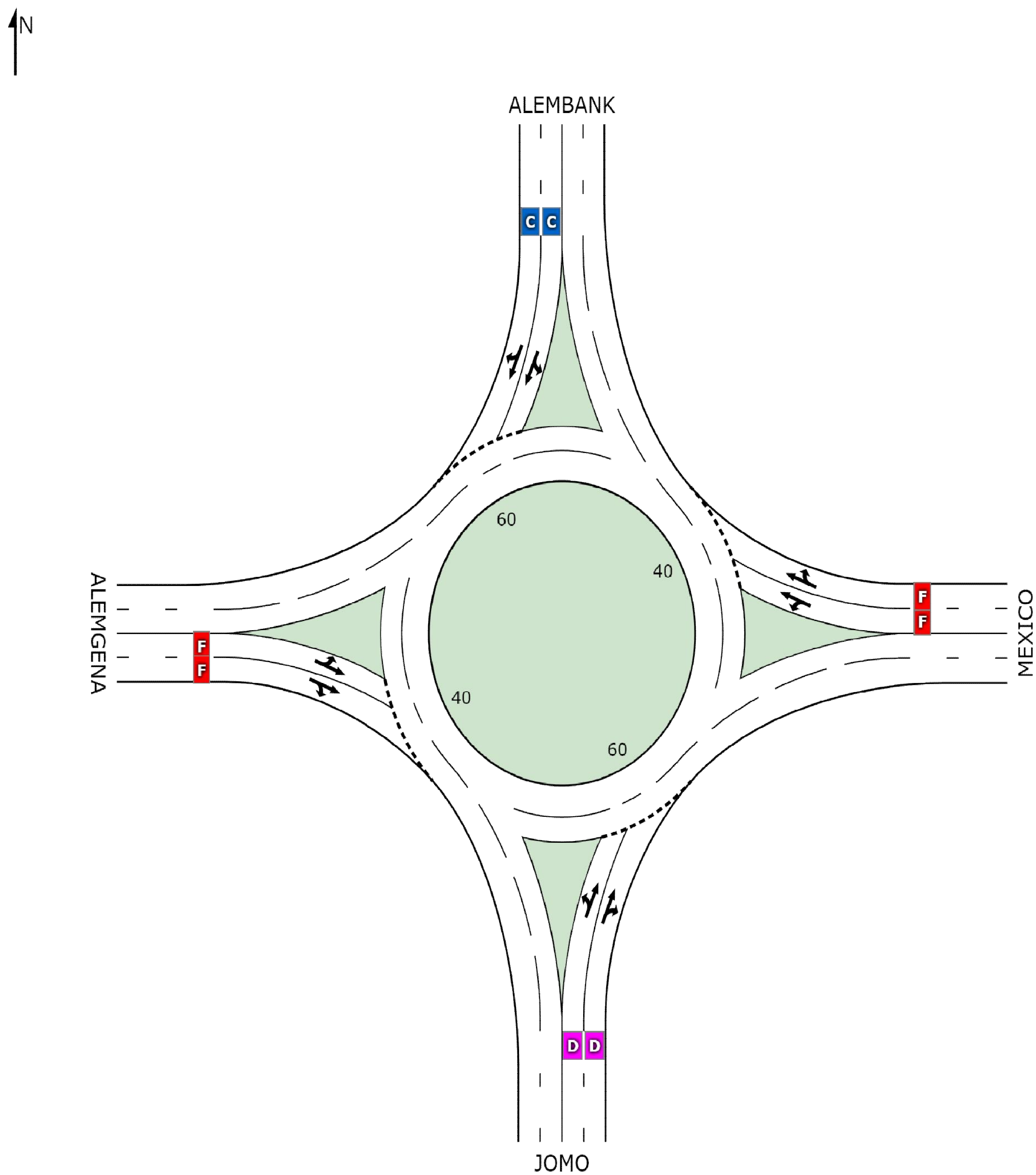
Movement Performance - Vehicles											
Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue		Prop. Queued	Effective Stop Rate	Average Speed
							Vehicles	Distance			
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: JOMO											
1	L	287	5.0	0.890	20.1	LOS D	10.4	77.8	1.00	1.61	23.6
2	T	688	12.0	0.890	17.0	LOS D	12.5	94.4	1.00	1.62	24.0
3	R	173	3.0	0.890	15.8	LOS D	12.5	94.4	1.00	1.65	24.0
Approach		1148	8.9	0.890	17.6	LOS D	12.5	94.4	1.00	1.62	23.9
East: MEXICO											
4	L	389	5.0	1.420	205.1	LOS F	73.4	546.3	1.00	5.61	7.9
5	T	934	11.0	1.420	202.5	LOS F	96.8	729.5	1.00	6.28	7.5
6	R	234	3.0	1.420	201.6	LOS F	96.8	729.5	1.00	6.71	7.2
Approach		1557	8.3	1.420	203.0	LOS F	96.8	729.5	1.00	6.17	7.6
North: ALEMBANK											
7	L	235	0.0	0.755	14.4	LOS C	6.9	49.7	0.98	1.32	25.1
8	T	564	8.0	0.755	11.7	LOS C	8.2	60.7	0.99	1.34	25.6
9	R	141	2.0	0.755	10.7	LOS C	8.2	60.7	1.00	1.35	25.7
Approach		940	5.1	0.755	12.2	LOS C	8.2	60.7	0.99	1.34	25.5
West: ALEMGENA											
10	L	361	6.0	1.294	152.4	LOS F	57.0	429.8	1.00	4.84	9.7
11	T	866	13.0	1.294	149.4	LOS F	73.1	557.2	1.00	5.31	9.4
12	R	217	3.0	1.294	148.2	LOS F	73.1	557.2	1.00	5.67	9.0
Approach		1444	9.7	1.294	150.0	LOS F	73.1	557.2	1.00	5.24	9.4
All Vehicles		5089	8.3	1.420	110.9	LOS F	96.8	729.5	1.00	3.99	11.5

Level of Service (LOS) Method: Degree of Saturation (SIDRA METHOD).
Roundabout LOS Method: SIDRA Roundabout LOS.
Vehicle movement LOS values are based on degree of saturation per movement
Intersection and Approach LOS values are based on worst degree of saturation for any vehicle movement.
Roundabout Capacity Model: SIDRA Standard.

SIDRA Standard Delay Model used.

LEVEL OF SERVICE SUMMARY

Site: AYERTENA ROUNDA-BOUT



MOVEMENT SUMMARY

Site: POST OFFICE SIGNA-

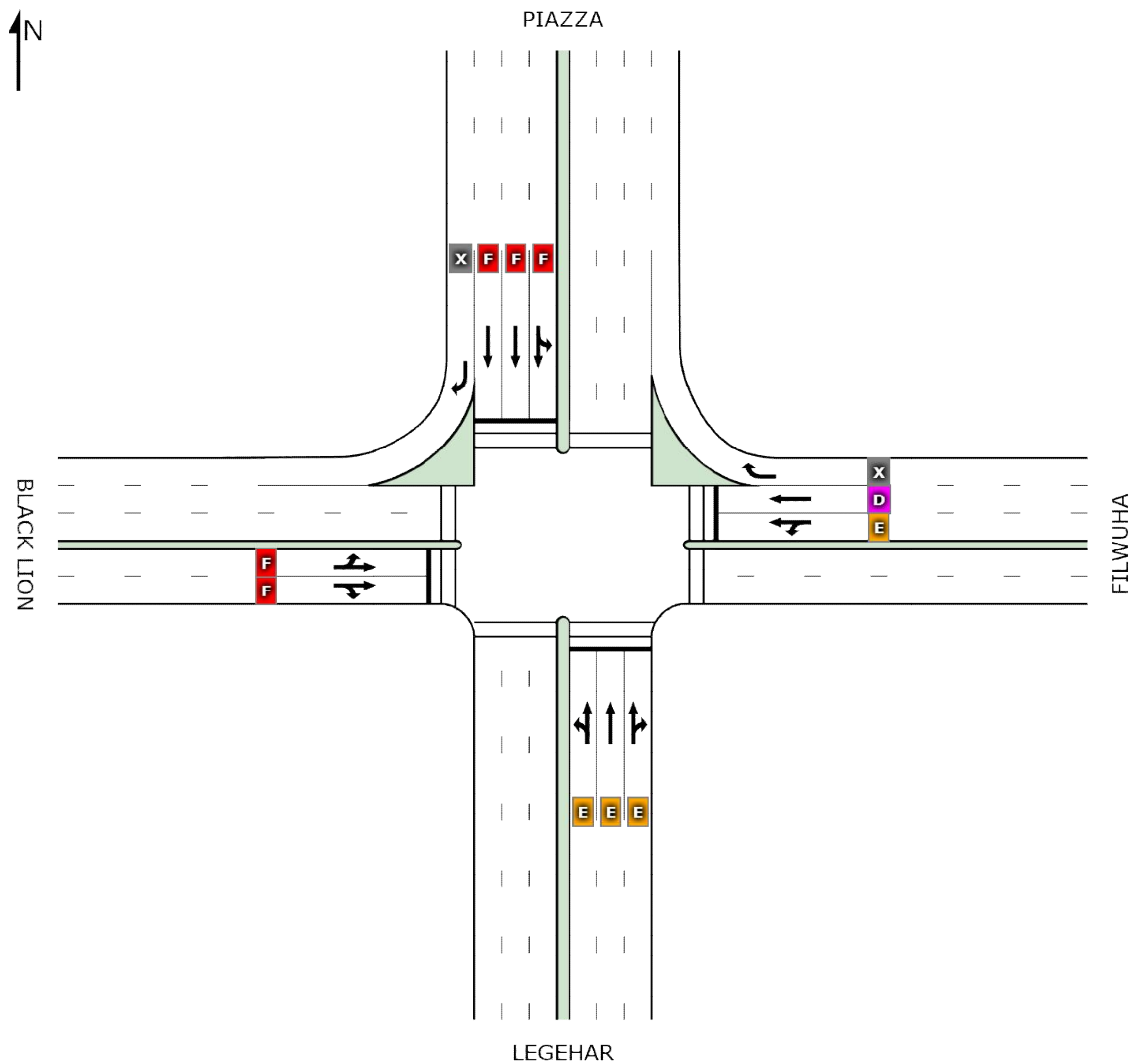
Four-way intersection with 2-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 116 seconds (User-Given Phase Times)

Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: LEGEHAR											
1	L	146	2.0	0.962	81.8	LOS F	22.6	162.5	1.00	1.15	17.1
2	T	1019	4.0	0.962	70.9	LOS E	33.5	242.5	1.00	1.14	17.9
3	R	65	1.0	0.962	76.8	LOS E	32.0	231.2	1.00	1.13	18.0
Approach		1230	3.6	0.962	72.5	LOS E	33.5	242.5	1.00	1.14	17.8
East: FILWUHA											
4	L	150	2.0	0.756	58.7	LOS E	13.8	98.0	1.00	0.89	21.0
5	T	345	1.0	0.756	52.4	LOS D	14.3	101.2	1.00	0.90	21.3
6	R	88	0.0	0.047	5.6	X	X	X	X	0.53	44.1
Approach		583	1.1	0.756	46.9	LOS D	14.3	101.2	0.85	0.84	23.0
North: PIAZZA											
7	L	244	0.0	1.017	91.7	LOS F	38.6	275.1	1.00	1.19	15.8
8	T	1249	4.0	1.017	85.7	LOS F	39.0	282.6	1.00	1.23	15.9
9	R	105	1.0	0.057	5.6	X	X	X	X	0.53	44.1
Approach		1599	3.2	1.017	81.3	LOS F	39.0	282.6	0.93	1.18	16.6
West: BLACK LION											
10	L	118	3.0	1.119	133.6	LOS F	28.1	202.9	1.00	1.37	12.1
11	T	428	4.0	1.119	125.6	LOS F	29.0	209.7	1.00	1.36	12.1
12	R	91	3.0	1.119	130.4	LOS F	29.0	209.7	1.00	1.36	12.2
Approach		637	3.7	1.119	127.7	LOS F	29.0	209.7	1.00	1.36	12.1
All Vehicles		4050	3.1	1.119	81.0	LOS F	39.0	282.6	0.95	1.15	16.6

X: Not applicable for Continuous movement.

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).
Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).



	South	East	North	West	Intersection
LOS	E	D	F	F	F

X: Not applicable for Continuous lane.

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).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).
SIDRA Standard Delay Model used.

PHASING SUMMARY

Site: POST OFFICE SIGNALIZED

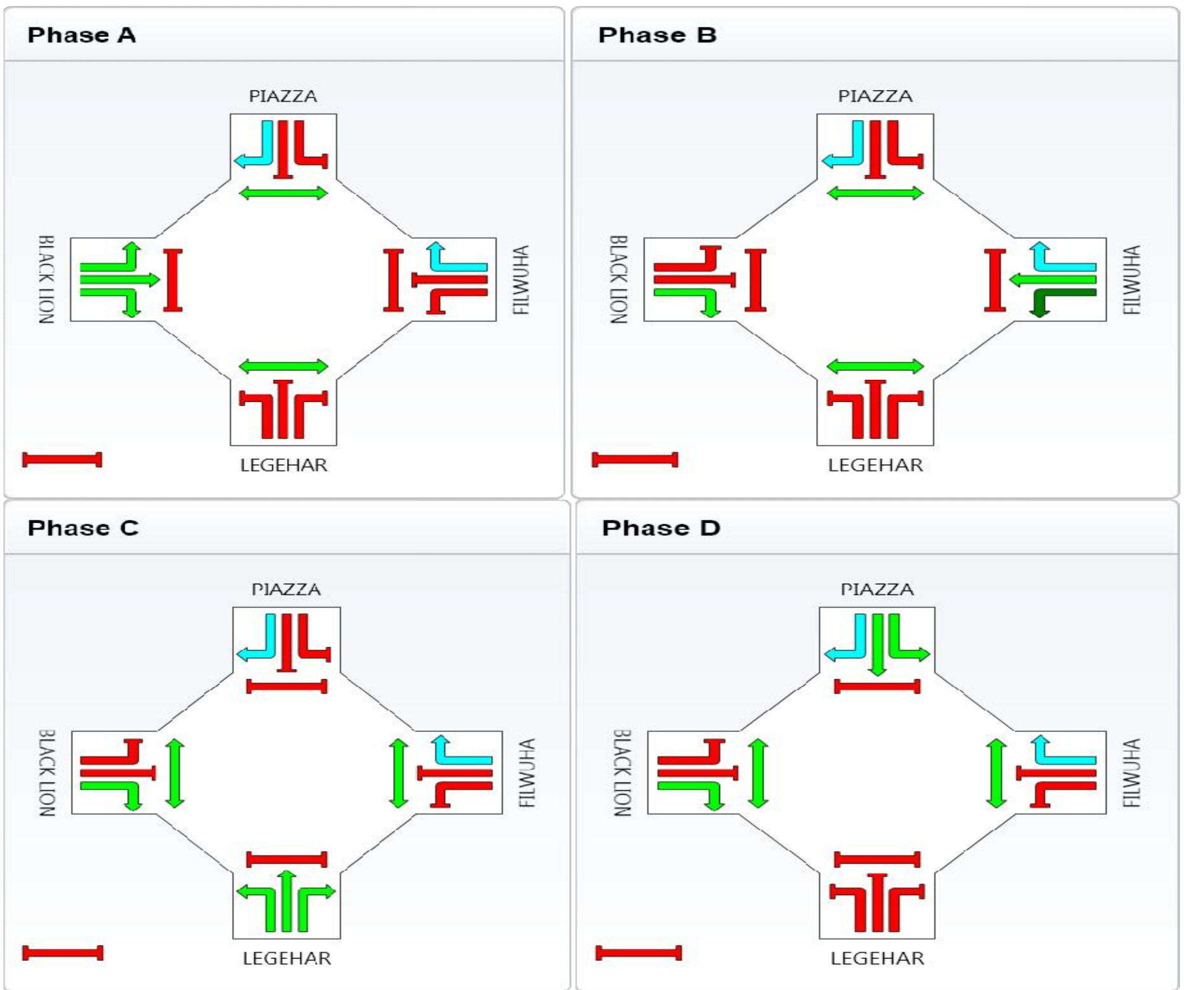
Four-way intersection with 2-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 116 seconds (User-Given Phase Times)

Phase times specified by the user
Sequence: Split Phasing
Input Sequence: A, B, C, D
Output Sequence: A, B, C, D

Phase Timing Results

Phase	A	B	C	D
Green Time (sec)	20	20	30	30
Yellow Time (sec)	2	2	2	2
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	24	24	34	34
Phase Split	21 %	21 %	29 %	29 %

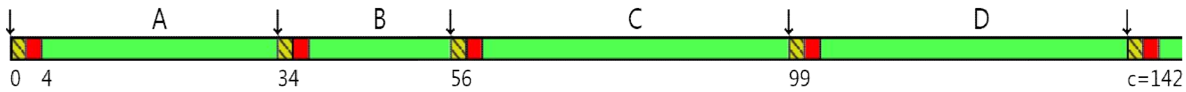


Normal Movement	Permitted/Opposed
Slip-Lane Movement	Opposed Slip-Lane
Stopped Movement	Continuous Movement
Turn On Red	Undetected Movement

Four-way intersection with 2-lane approaches (Signals)

Signals - Fixed Time Cycle Time = 142 seconds (User-Given Phase Times)

DISPLAYED SIGNAL TIMING - PHASES

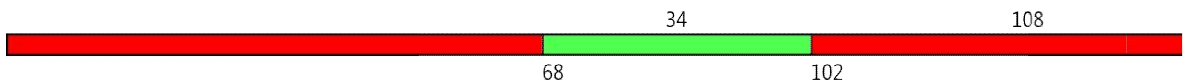


EFFECTIVE SIGNAL TIMING - MOVEMENTS

1 (South L)

2 (South T)

3 (South R)

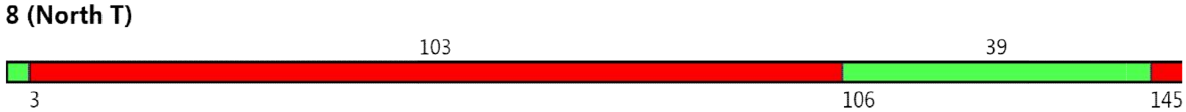


4 (East L)

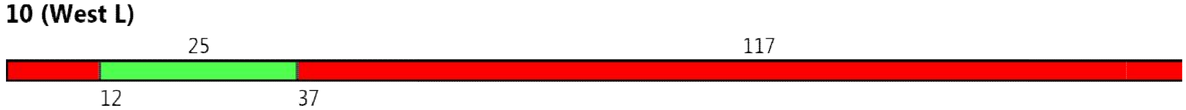
5 (East T)

7 (North L)

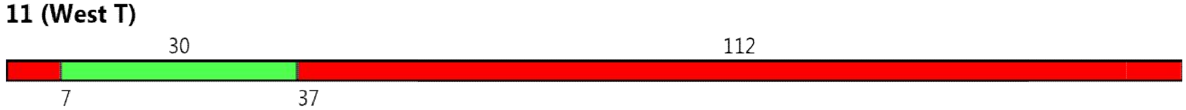
8 (North T)



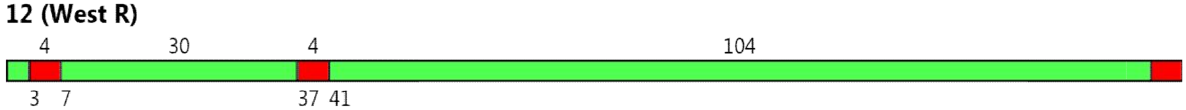
10 (West L)



11 (West T)



12 (West R)



Movement Performance
Site:POST OFFICE SIGNALIZED

Intersection ID: 1
Fixed-Time Signals, Cycle Time = 142 sec (Sum of User-given Phase Times)

Mov ID		Total Delay (veh-h/h)	Total Delay (pers-h/h)	Aver. Delay (sec)	Eff. Stop Rate	Total Stops	Perf. Index	Tot.Trav. Distance (veh-km/h)	Tot.Trav. Time (veh-h/h)	Aver. Speed (km/h)
South: LEGEHAR										
1	L	3.17	3.80	78.1	1.04	151.5	12.85	84.7	4.8	17.6
2	T	19.01	22.81	67.2	1.02	1037.6	84.24	584.3	31.7	18.5
3	R	1.32	1.58	73.2	1.01	65.7	5.46	37.4	2.0	18.5
East: FILWUHA										
4	L	4.77	5.72	114.4	1.14	170.8	15.84	86.8	6.5	13.4
5	T	10.34	12.40	107.9	1.16	400.9	35.77	197.8	14.6	13.5
6	R	0.14	0.16	5.6	0.53	46.6	1.43	51.6	1.2	44.1
North: PIAZZA										
7	L	5.68	6.82	83.8	1.06	259.3	22.37	142.2	8.4	16.8
8	T	26.98	32.38	77.7	1.09	1356.0	112.27	716.5	42.4	16.9
9	R	0.16	0.20	5.6	0.53	55.7	1.71	61.7	1.4	44.1
West: BLACK LION										
10	L	2.56	3.07	78.4	1.01	118.6	10.12	68.5	3.9	17.7
11	T	8.36	10.04	70.3	1.01	432.4	35.60	245.7	13.7	17.9
12	R	1.90	2.27	74.8	1.01	92.2	7.66	52.5	2.9	18.2
Pedestrian Movements										
P1		4.90	4.90	40.3	0.75	330.0	10.79	19.0	9.0	2.1
P3		2.64	2.64	17.8	0.50	267.5	8.26	19.4	6.8	2.9
P5		8.06	8.06	42.6	0.77	527.5	17.77	31.7	14.8	2.1
P7		4.17	4.17	19.3	0.52	405.4	12.99	30.7	10.7	2.9

MOVEMENT SUMMARY

Site: POST OFFICE SIGNA-LIZED

Four-way intersection with 2-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 142 seconds (User-Given Phase Times)

Movement Performance - Vehicles											
Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effec-tive Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: LEGEHAR											
1	L	146	2.0	0.903	78.1	LOS E	24.2	173.9	1.00	1.04	17.6
2	T	1019	4.0	0.903	67.2	LOS E	35.8	258.9	1.00	1.02	18.5
3	R	65	1.0	0.903	73.2	LOS E	34.6	249.7	1.00	1.01	18.5
Approach		1230	3.6	0.903	68.8	LOS E	35.8	258.9	1.00	1.02	18.4
East: FILWUHA											
4	L	150	2.0	1.029	114.4	LOS F	22.1	157.0	1.00	1.14	13.4
5	T	345	1.0	1.029	107.9	LOS F	23.0	162.3	1.00	1.16	13.5
6	R	88	0.0	0.047	5.6	X	X	X	X	0.53	44.1
Approach		583	1.1	1.029	94.1	LOS F	23.0	162.3	0.85	1.06	15.1
North: PIAZZA											
7	L	244	0.0	0.957	83.8	LOS F	41.0	291.8	1.00	1.06	16.8
8	T	1249	4.0	0.957	77.7	LOS E	41.4	299.7	1.00	1.09	16.9
9	R	105	1.0	0.057	5.6	X	X	X	X	0.53	44.1
Approach		1599	3.2	0.957	73.9	LOS E	41.4	299.7	0.93	1.04	17.6
West: BLACK LION											
10	L	118	3.0	0.897	78.4	LOS E	24.7	178.6	1.00	1.01	17.7
11	T	428	4.0	0.897	70.3	LOS E	24.7	178.6	1.00	1.01	17.9
12	R	91	3.0	0.897	74.8	LOS E	23.6	170.8	1.00	1.01	18.2
Approach		637	3.7	0.897	72.4	LOS E	24.7	178.6	1.00	1.01	17.9
All Vehicles		4050	3.1	1.029	75.0	LOS E	41.4	299.7	0.95	1.03	17.4

X: Not applicable for Continuous movement.

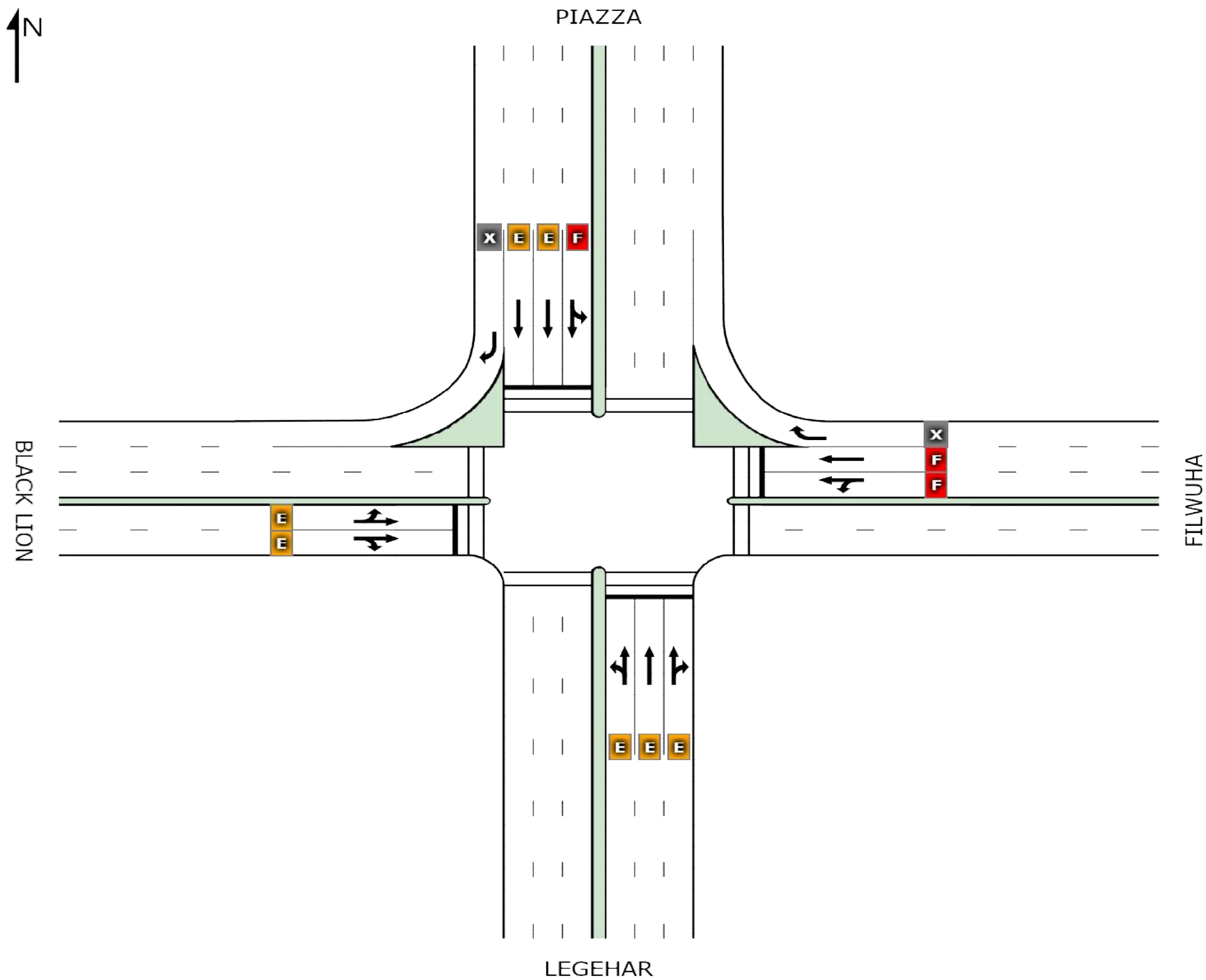
Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement
LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).

LEVEL OF SERVICE SUMMARY

Site: POST OFFICE SIGNALIZED

Four-way intersection with 2-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 142 seconds (User-Given Phase Times)



	South	East	North	West	Intersection
LOS	E	F	E	E	E

X: Not applicable for Continuous lane.

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).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).

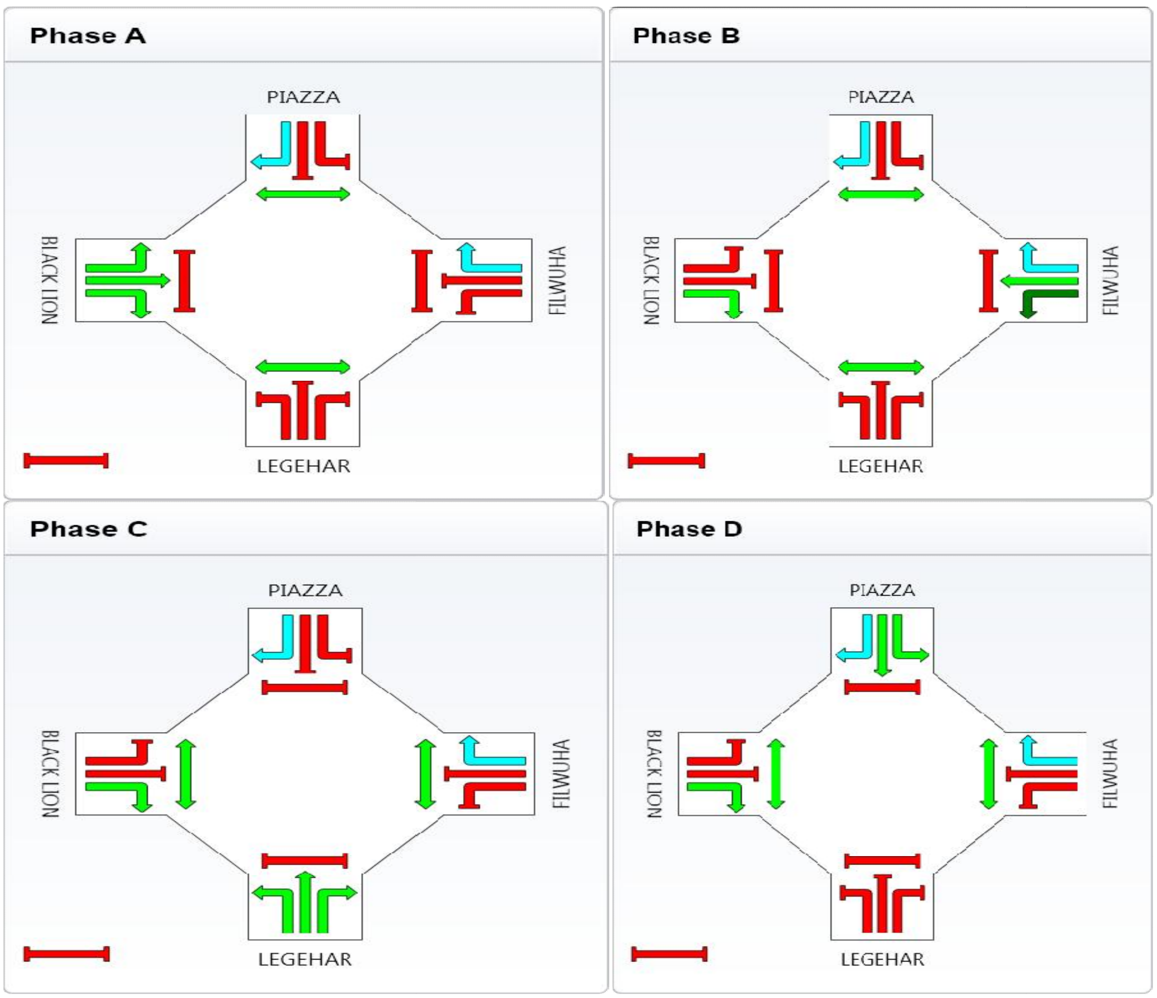
PHASING SUMMARY **Site: POST OFFICE SIGNALIZED**

Four-way intersection with 2-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 142 seconds (User-Given Phase Times)

Phase Timing Results

Phase	A	B	C	D
Green Time (sec)	30	18	39	39
Yellow Time (sec)	2	2	2	2
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	34	22	43	43
Phase Split	24 %	15 %	30 %	30 %



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

Movement Performance
Site:LEGEHAR SIGNALIZED INTERSECTION

Intersection ID: 2
Fixed-Time Signals, Cycle Time = 140 sec (Sum of User-given Phase Times)

Mov ID		Total Delay (veh-h/h)	Total Delay (pers-h/h)	Aver. Delay (sec)	Eff. Stop Rate	Total Stops	Perf. Index	Tot.Trav. Distance (veh-km/h)	Tot.Trav. Time (veh-h/h)	Aver. Speed (km/h)
South: RAILWAY STATION										
1	L	3.87	4.64	75.6	0.97	178.0	14.34	46.9	5.0	9.3
2	T	5.61	6.74	71.4	0.99	279.7	21.67	69.9	7.6	9.2
3	R	0.20	0.24	3.4	0.41	84.6	2.03	54.3	1.6	34.9
East: MESKEL SQUARE										
4	L	12.74	15.28	256.2	1.69	302.4	30.46	45.4	13.8	3.3
5	T	88.25	105.90	251.9	1.72	2164.7	213.09	311.3	96.9	3.2
6	R	0.31	0.37	3.4	0.41	131.1	3.14	84.2	2.4	34.9
North: PIAZZA										
7	L	4.83	5.80	73.8	0.97	228.1	18.23	59.9	6.3	9.5
8	T	6.38	7.66	69.3	0.99	328.1	25.16	81.8	8.7	9.4
9	R	0.23	0.28	3.4	0.41	98.6	2.35	63.1	1.8	34.9
West: MEXICO										
10	L	19.93	23.92	296.4	1.77	428.2	45.21	61.6	21.4	2.9
11	T	97.73	117.27	292.2	1.81	2178.4	223.46	297.3	106.0	2.8
12	R	0.17	0.20	3.4	0.41	70.5	1.69	45.2	1.3	34.9
Pedestrian Movements										
P1		9.81	9.81	47.2	0.82	614.4	19.06	27.3	15.6	1.7
P3		6.20	6.20	36.4	0.72	442.2	13.64	23.3	11.2	2.1
P5		12.53	12.53	47.2	0.82	784.5	24.34	34.9	20.0	1.7
P7		11.45	11.45	37.9	0.74	800.5	25.08	43.0	20.6	2.1

MOVEMENT SUMMARY

Site: LEGEHAR SIGNALIZED INTERSECTION

Legehar Four-way intersection with 2 & 3-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 140 seconds (User-Given Phase Times)

Movement Performance - Vehicles												
Mov ID	Turn	Demand Flow	HV	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed
		veh/h	%	v/c	sec		veh	m		per veh	km/h
South: RAILWAY STATION											
1	L	184	0.0	0.858	75.6	LOS E	17.1	120.2	1.00	0.97	9.3
2	T	283	4.0	0.858	71.4	LOS E	17.3	125.5	1.00	0.99	9.2
3	R	208	2.0	0.118	3.4	X	X	X	X	0.41	34.9
Approach		676	2.3	0.858	51.6	LOS D	17.3	125.5	0.69	0.80	12.1
East: MESKEL SQUARE											
4	L	179	0.0	1.391	256.2	LOS F	97.4	690.8	1.00	1.69	3.3
5	T	1261	2.0	1.391	251.9	LOS F	98.3	699.8	1.00	1.72	3.2
6	R	323	2.0	0.177	3.4	X	X	X	X	0.41	34.9
Approach		1763	1.8	1.391	206.8	LOS F	98.3	699.8	0.82	1.47	3.9
North: PIAZZA											
7	L	236	0.0	0.866	73.8	LOS E	20.5	143.7	1.00	0.97	9.5
8	T	332	1.0	0.866	69.3	LOS E	21.2	149.6	1.00	0.99	9.4
9	R	242	0.0	0.135	3.4	X	X	X	X	0.41	34.9
Approach		809	0.4	0.866	50.9	LOS D	21.2	149.6	0.70	0.81	12.2
West: MEXICO											
10	L	242	1.0	1.479	296.4	LOS F	103.3	733.7	1.00	1.77	2.9
11	T	1204	2.0	1.479	292.2	LOS F	104.8	746.3	1.00	1.81	2.8
12	R	174	2.0	0.095	3.4	X	X	X	X	0.41	34.9
Approach		1620	1.9	1.479	261.8	LOS F	104.8	746.3	0.89	1.65	3.1
All Vehicles		4868	1.7	1.479	177.7	LOS F	104.8	746.3	0.81	1.33	4.5

X: Not applicable for Continuous movement.

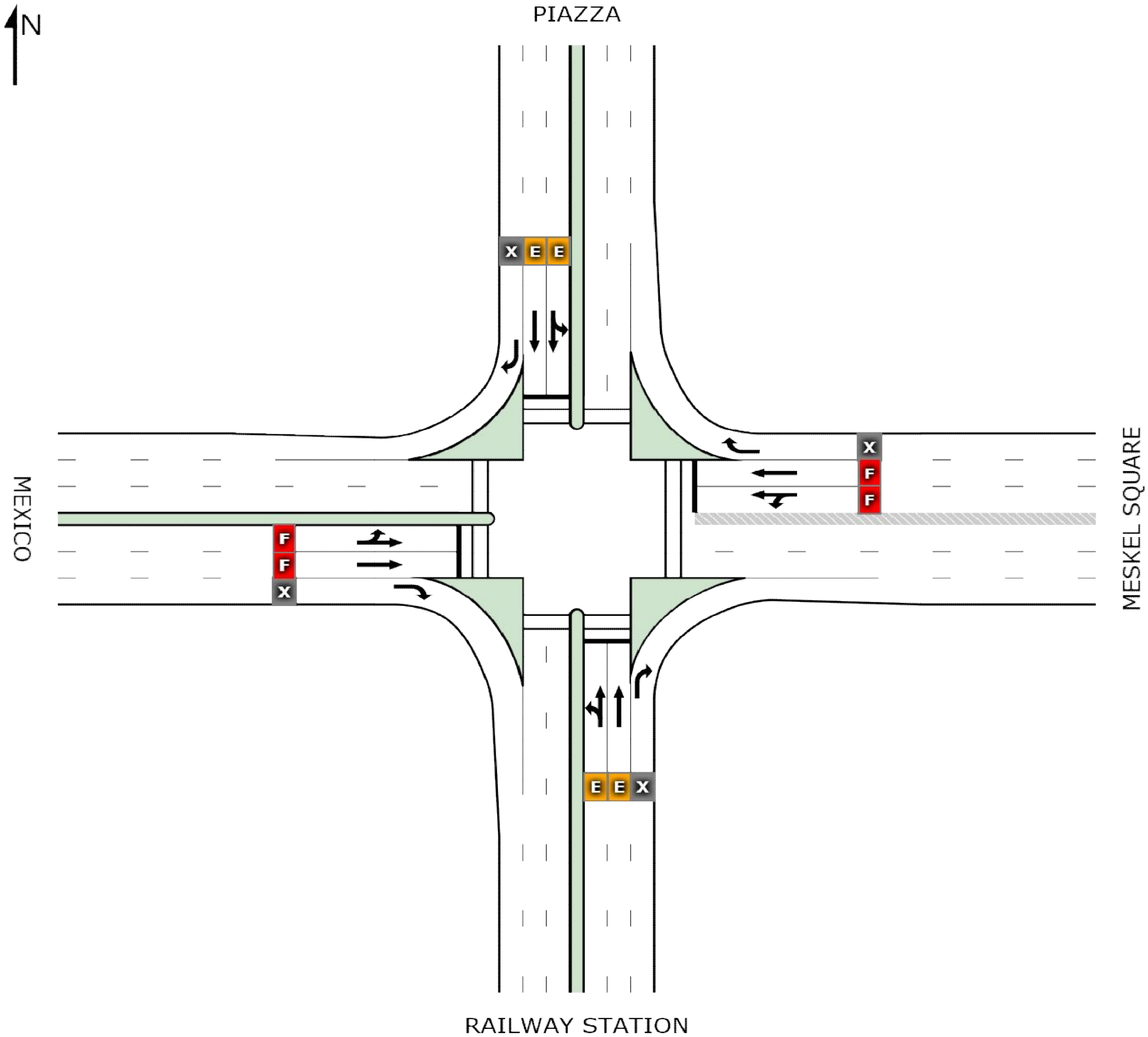
Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement
LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).
SIDRA Standard Delay Model used.

LEVEL OF SERVICE SUMMARY

Site: LEGEHAR SIGNALIZED INTERSECTION

Legehar Four-way intersection with 2 & 3-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 140 seconds (User-Given Phase Times)



	South	East	North	West	Intersection
LOS	D	F	D	F	F

X: Not applicable for Continuous lane.

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).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).

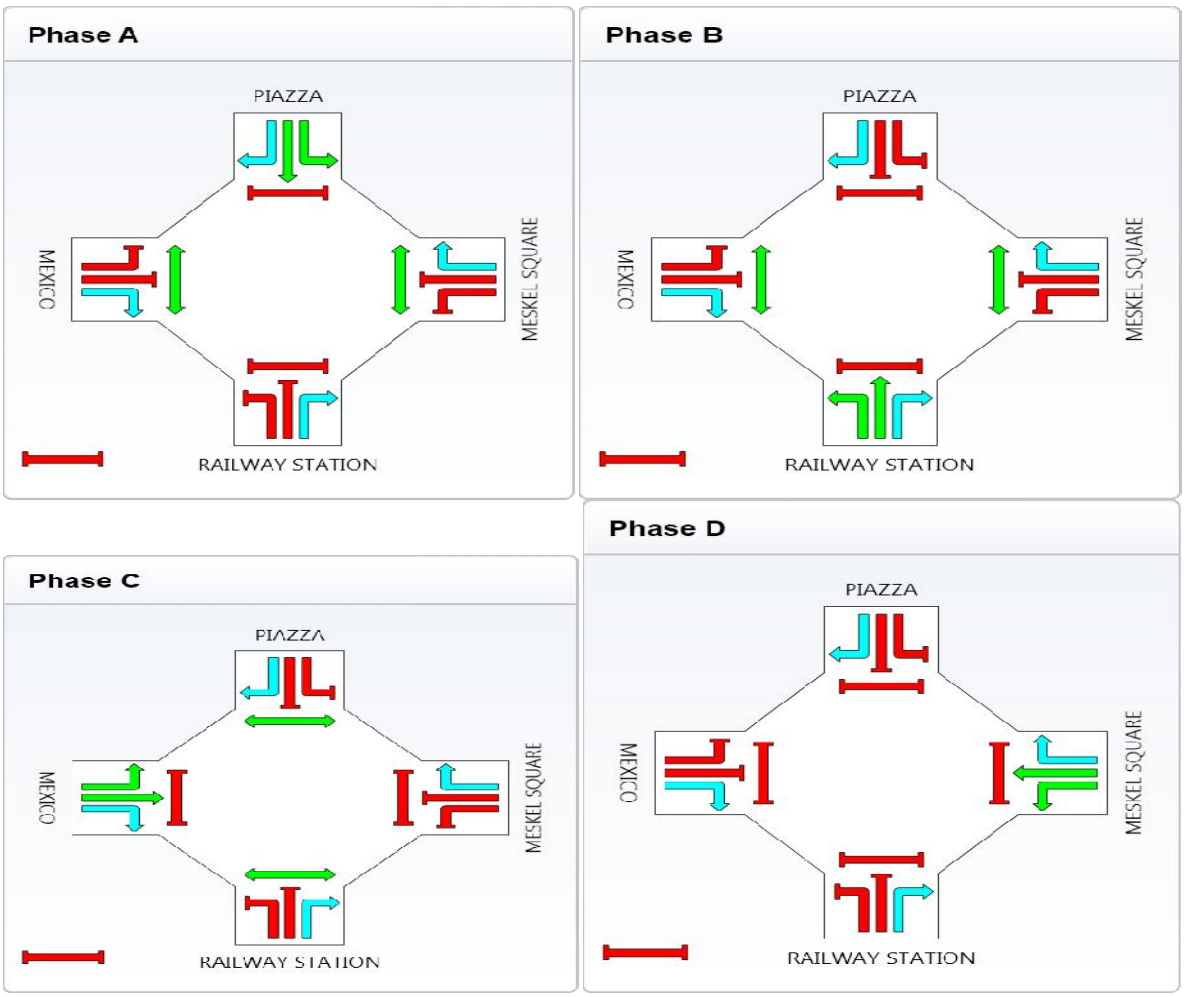
PHASING SUMMARY Site: LEGEHAR SIGNALIZED INTERSECTION

Legehar Four-way intersection with 2 & 3-lane approaches (Signals)

Signals - Fixed Time   Cycle Time = 140 seconds (User-Given Phase Times)

Phase Timing Results

Phase	**A**	**B**	**C**	**D**
Green Time (sec)	25	21	36	38
Yellow Time (sec)	3	3	3	3
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	30	26	41	43
Phase Split	21 %	19 %	29 %	31 %

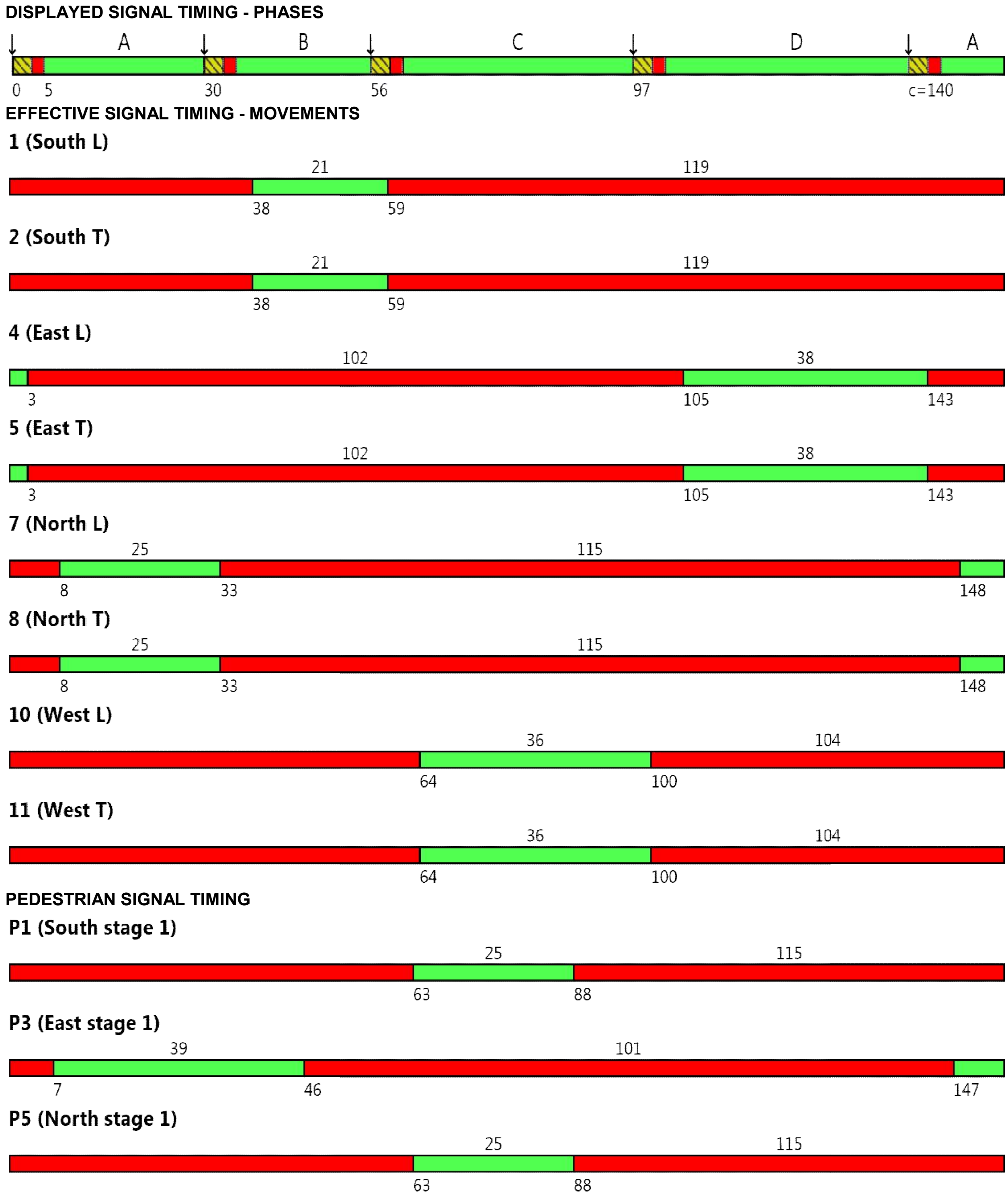


Normal Movement	Permitted/Opposed
Slip-Lane Movement	Opposed Slip-Lane
Stopped Movement	Continuous Movement
Turn On Red	Undetected Movement

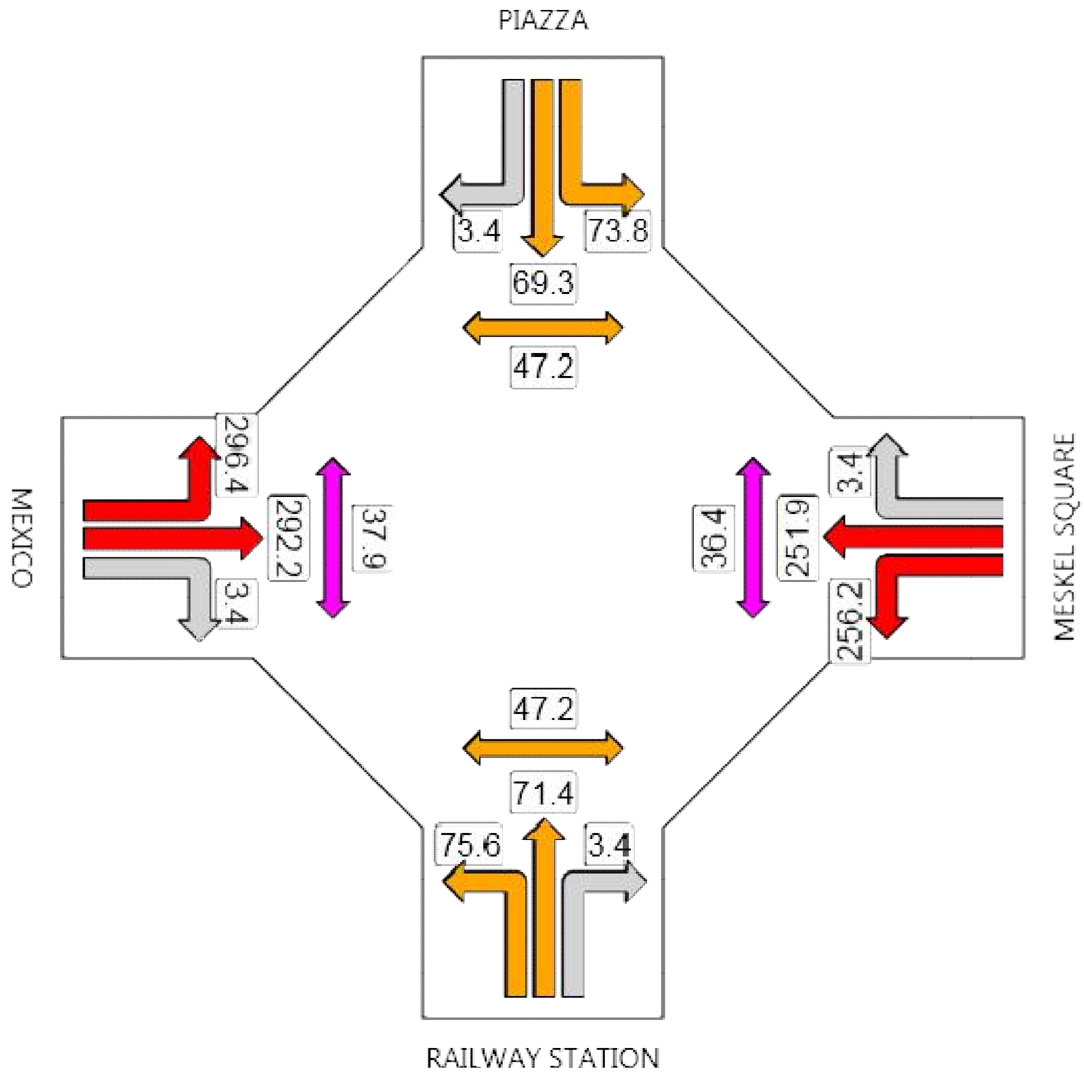
Phase Transition Applied

MOVEMENT TIMING Site: LEGEHAR SIGNALIZED INTERSECTION

Signals - Fixed Time   Cycle Time = 140 seconds (User-Given Phase Times)



P7 (West stage 1)



	South	East	North	West	Intersection
Delay (Average)	51.6	206.8	50.9	261.8	177.7
LOS	D	F	D	F	F

Colour code based on Level of Service

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