Jimma University

School of Graduate Studies

Jimma Institute of Technology

Faculty of Civil and Environmental Engineering

Highway Engineering Stream

Evaluation of Capacity and Performance of Selected Roundabouts: A Case Study in Addis Ababa City

A Thesis Submitted to the School of Postgraduate Studies of Jimma University, Jimma Institute of Technology in Partial Fulfillment of the Requirement for Masters of Science Degree in Highway Engineering.

By

Bekele Taso

February 2018

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Declaration

I, the undersigned, declare that this thesis entitled: "Evaluation of Capacity and Performance of selected Roundabouts: A Case Study in Addis Ababa City." is my original work, and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for theses have been duly acknowledged.

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As Master's Research Advisors, we hereby certify that we have read and evaluated this MSc research prepared under our guidance, by Bekele Taso entitled: "Evaluation of Capacity and Performance of Selected Roundabouts: A Case Study in Addis Ababa City."

We recommend that it can be submitted as fulfilling the MSc Thesis requirements.

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Finally, my special thanks and sincere gratitude goes to my friends those who shared their kind help and kind support in working my research.

ABSTRACT

The growth and development of metropolitan cities imply a pragmatic increase in all dimensions, causing many challenges especially in transportation systems. Among these, the problems of traffic congestions, long time delay and inappropriate level of services are experiencing by the travelling public every day. In Ethiopia, these problems have been a significant issue which has a great impact on human and economy of the country. To investigate and identify these problems, this research has evaluated the capacity and performance of selected roundabouts in Addis Ababa city. The methodology that has been used for the study was quantitative and qualitative research design method. The collected data for the survey was road geometry data, peak hour traffic volume including pedestrian, while the secondary data was taken from different sources. Traffic volumes data were collected at peak hour within 15minutes intervals starting from 8:00 am - 9:00 am and 5:30 pm - 6:30 pm for five consecutive working days. From these, the three successive working days except Monday and Friday traffic data used in the analysis. Vehicles were classified during traffic collection and converted to the same vehicles category using passengers' care unit (PCU) of each vehicle class. The analysis has been conducted using both manual calculation based on HCM 2010 and SIDRAINTERSECTION software version 5.1. The results obtained from both methods were compared based on the performance measures such as delay, LOS, a degree of saturation and queue length. The results indicated that the analysis using software had shown over-saturated traffic flows, long time delay and the high degree of saturation than using manual calculation method. The reason was due to the calibrated of environmental factors, vehicles and population growth rate embedded as default in the SIDRA INTERSECTION Software. From the analyzed results the existing capacity and performance of Abune Petros and German(Mekanisa) roundabouts were over saturated, long time delay, long queue length and operates beyond their capacity(LOS F.) Hence, it is concluded that factors affecting capacity and performance of roundabouts are high traffic flow, unbalanced heavy vehicles proportion, island diameter, and number of lanes, lane width and a number of circulatory lanes. To improve the capacity and performance level; Abune Petros roundabout would be upgraded by increasing island diameter from 27.8m to 80m, lane width to 6m and number of circulating lane from 2 to 3, whereas German roundabout was recommended to be changed to signalized intersection.

Keywords*:* Capacity, Performance, Manual calculation, SIDRA, Level of services, Delay, Degree of saturation, Queue length.

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ACRONYMS

A.A	Addis Ababa
AASHTO	American Association of State Highway and Transportation Officials
AACRA	Addis Ababa City Roads Authority
AU	Africa Union
ERA	Ethiopian Roads Authority
JiT	Jimma Institute of Technology
LOS	Level of Services
NCHRP	National Cooperative Highway Research Program
HCM	Highway Capacity Manual
TRB	Transport Research Board
FHWA	Federal Highway Administration
PCE	Passengers Car Equivalence
PCPH	Passengers Carper Hour
PCU	Passengers Car Unit
PHF	Peak Hour Factor
PHV	Peak Hour Volume
RA	Roundabout
SFR	Saturation Flow Rate
USA	United State of America
USDOT	United State Department of Transportation

CHAPTER ONE

INTRODUCTION

1.1 Background

A Circular intersection is an alternative form of intersection traffic control. In early 1900's, Circular intersections type's rotaries and neighborhood traffic circles were standard throughout Europe and America before the roundabout was discovered. A roundabout intersection is one of the options for an intersection traffic control which is circular, characterized by the yield on entry and circulation around a central island. They are appropriate for many intersections including locations experiencing a high number of crashes, long traffic delays, and approaches with relatively balanced traffic flows, but they fell out of favor when traffic congestion and intersection crashes increased in the 1950's. Due to this, most circular Intersections were replaced by traffic signals [1].

Americas' affinity for a technical solution may explain the shift to signalized intersections; however, many American highway engineers are now coming "full circle," and are beginning to use roundabouts to reduce crashes and increase capacity [2].

Roundabouts 'like rotaries and traffic circles can serve to replace standard signalized and un-signalized intersections with circular intersections. Modern roundabout was designed or developed in the United Kingdom in the 1960 and operates different from first circular intersections, and as such, are associated with various advantages and disadvantages than are other types of circular junctions [3].

Roundabouts intersection consists of two or more roads that are made up of one way-circulating roadway that has priority over approaching traffic [4]. The Vehicles on a circulatory roadway are circulating counterclockwise in the one way circulating road that has priority approaches and passing in front of circulatory flow entry occupying the inside or outside lane.

The necessity of the roundabout is that the traffic is required to slow down for negotiating the curve around the central island. In most cases, modern roundabouts have been found to be much safer than other intersections. The reduction of points of conflict from 32 to 8 lessens the chance of crashes, and when combined with reducing speed, crashes probability is further reduced.

It also improves the safety of intersections by eliminating or by lowering speed differentials at intersections, and by forcing drives to decrease speeds as proceed into and through the intersection [5].

Roundabouts intersection comprise of two or more roads that are made up of one way-circulating roadway that has priority over approaching traffic [4].

The HCM states that roundabouts share the same primary control delay formation as two-way and all-way stop-controlled intersections. Un-signalized intersections can be uncontrolled, yield-sign controlled, stop-sign controlled, or roundabouts [6].

According to the FHWA Roundabout Guide1, Roundabout reduces vehicle speeds, minimize vehicle weaving, automatically establish right-of-way, and reduce conflict point's from32 to 8 [7].

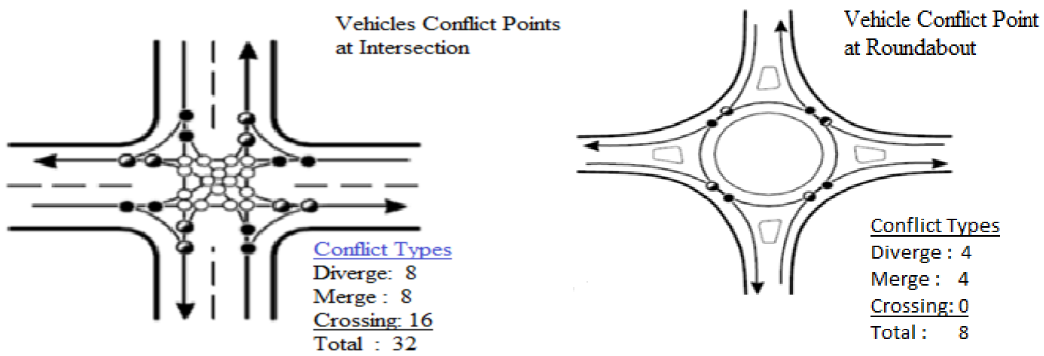


Figure 1.1: The comparison reduces conflict points from 32 to 8 between Roundabout and Junction intersection [7].

Roundabouts were considered as an alternative intersection design and had been used in several countries including Ethiopia. It works on yield at entry principle which causes slow or reduces the speed of entering traffic speed instead of stopping traffic thus, reducing the delay caused at an intersection and providing the much safer environment for pedestrians and bicycle users.

1.2 Statement of the problem

Nowadays traffic congestion is considered as severe problems in most cities in the world due to urban development and economic growth.

Currently, Ethiopia is one of the countries that are in rapid economic growth and social development that resulting from the increase of vehicles in cities causing traffic congestion on roads especially at a roundabout and other intersections.

According to the study conducted, it is common to see traffic congestion at junctions in Addis Ababa at peak hours in morning and afternoon. Hence, traffic enforcer needs to intervene in the situation to regulate traffic flow by overriding the control devices of the intersection. Otherwise, it would be impractical to have regular traffic flow especially at a roundabout junction, which is more dependent on driver behavior and balanced traffic flow between the approaches [8].

Hence, the rapid growth of metropolitan cities such as Addis Ababa caused many challenges to a transportation system. Among these, traffic congestion has been a severe issue that has been created waste of time and energy, thereby produced pollution and noises. To minimize the problems, Addis Ababa City Roads Authority focused its policy to remove all roundabouts and change to signal intersection to reduce delay due to an increment of traffic growth. However, eliminating all roundabouts and changing to indicate intersection has not the only solution, because missing roundabouts in the city have negative impacts such as missing safety, reducing the beauty of downtown and traffic police need to intervene in the situation to regulate traffic flow when there is no electric power in the city. Therefore, to minimize such problem, this study focused on two roundabouts namely Abune Petros and German (Mekanisa) roundabouts.

The Significance of the surrounding area of the roundabouts was due to the existence of many governments, educational and health institution. Especially at Abune Petros roundabout, there is an open market in Ethiopia which covers several square meters of commercial areas and there also vehicles from big bus station of country and different segment come to each other at this roundabout, and as well as at the German roundabout due the various governmental and non-government offices, company, school and the direction of residential areas from which massive traffic flows come to this roundabout.

For this reason, these roundabouts have become a point of high traffic congestion, and this problem has been observed increasing rapidly. These factors have caused the problems of inconvenience to drivers, leads to lost time from job, sources for traffic crashes, pedestrian injury and fatalities are observed at these roundabouts.

So, these factors have the impact on human and economy of the country. Therefore this study has evaluated these problems and has implemented the remedial measures.

1.3 Objectives of the Study

1.3.1 General Objective

The main objective of the study is to evaluate the existing capacity and performance of selected roundabouts in Addis Ababa City to implement the remedial measures.

1.3.2 Specific Objectives

- ✓ To quantify traffic volume and other identified significant factors tend to affect performance level of the roundabout.
- ✓ To analyze the capacity and performance of existing roundabout using both Manual calculation using HCM, 2010 and SIDRA INTERSECTION Software.
- ✓ To identify the significant factors affecting the Capacity and Performance level of roundabout
- ✓ To investigate its problems in order to suggest the improvement measures

1.4 Research Questions

The research questions are related to the specific objectives:

1. How traffic volume was quantified, and other significant factors tend to affect performance level of the roundabout was identified?
2. How the existing capacity and performance of roundabout was analyzed?
3. What were the significant factors affecting capacity and Performance Level of the roundabout?
4. What were the remedial measures that could be suggested to improve the intersection?

1.5 Significance of the Study

This research will be benefited the AACRA to a new alternative way which can be proposed to solve the problem at a roundabout. It is significant to Addis Ababa city administration for its social and economic policy formulation about the whole of the city based on the outcome of the research. Also, it will be added to the existing academic knowledge and enable to understand the subject matter for further investigation on related topics.

Finally, it accelerates the national development through the provision of problem-solving research output to the policy and decision makers.

1.6 Scope of the study

The coverage of the study is to assess the percent capacity and performances of the roundabout by considering queue length, Level of services, degree of saturation and delay based upon Highway Capacity Manual using manual calculation based up on HCM, 2010 and SIDRA INTERSECTION software methods and to identify the problem due to incapability of roundabout in Addis Ababa City, specifically at Abune Petros square and German roundabouts. In this research the rash hour traffic volume was not considered as well as Motorcycles, Bicycles and Bajaj were not considered due to their number were insignificant during traffic data collection.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

This section discusses the current literature concerning the definition of Roundabout, methods of roundabout capacity evaluation, factors that affect the capacity of roundabout, level of service and performance of roundabout based on queue length, Level of Service, delay, degree of saturation and it concludes with an overview of impacts due to inadequacy of roundabout capacity and possible remedial measures.

All information has gathered from different reference Books, Journals and internet browsers.

2.2 Basic Concept of roundabouts

Before roundabout was developed circular intersections or rotaries and neighborhood traffic circles were common throughout Europe and America in the early 1900's. But, a modern roundabout was first installed in England in early 1960's and is becoming popular in most countries including Ethiopia. These facilities were initially introduced to solve the problems of existing rotaries and traffic circles. As Roundabouts USA Web site, 2004 described that, the difference between modern roundabout and rotaries are listed as the following:

a. Modern roundabouts are a circular intersection where drivers travel counterclockwise around a center island. There are no traffic signals or stop signs in a modern roundabout. Drivers yield at the entry to traffic in the roundabout, and then enter the intersection and exit at their desired street and are designed to accommodate vehicles of all sizes, including emergency vehicles, buses, and trucks and trailer combinations and also main features of the modern roundabout is a raised central island. Differences between Modern roundabout and other traffic circles or rotaries

Figure 2.1: Modern Roundabouts [9].

b. Traffic circles or Rotaries are much larger than modern roundabouts. Traffic circles often have stop signs or traffic signals within the circular intersection. Drivers enter a traffic circle in a straight line and do not have to yield to traffic already in the circle. Traffic circles typically become congested if many vehicles enter at the same time. In detail it described on figure 2.2 below

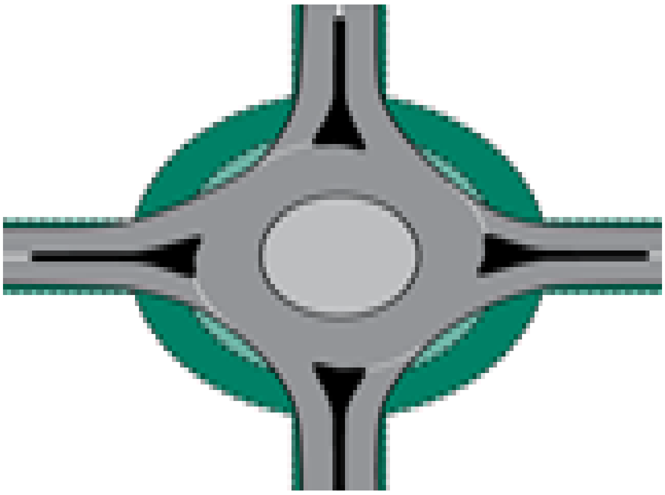


Figure 2.2: Traffic circles or Rotaries [9].

b. Neighborhood traffic calming circles are much smaller than modern roundabouts and replace stop signs at four-way intersections. They are typically used in the residential neighborhood to slow traffic speeds and reduce accidents but are not designed to accommodate larger vehicles. Many drivers often turn left in front of the circles rather than revolving around them. The following figure 2.3 is describing neighborhood traffic calming circle.

Figure 2.3: Neighborhood Traffic Calming circles [9]

2.3 Types of Modern Roundabouts

2.3.1 Mini-Roundabout

Mini-roundabouts are small single-lane roundabouts used in 25 mph or less urban/suburban environments. Because of this, mini-roundabouts are typically not suitable for use on higher-volume (greater than 6,000 AADT) state routes. A 2-inch mountable curb for the splitter islands and the central island is desirable because larger vehicles might be required to cross over it.

A standard application is to replace a stop-controlled or uncontrolled intersection with a mini-roundabout to reduce delay and increase capacity. With mini roundabouts, the existing curb and sidewalk at the intersection can sometimes be left in place.

Figure 2.4: Mini-roundabouts [9].

2.3.2 Single-Lane Roundabouts

As HCM stated, that Single-lane roundabouts have single-lane entries at all legs and one circulating lane. They typically have mountable raised splitter islands, a mountable truck apron, and a central island, which is usually landscaped.

Figure 2.5: Single-Lane Roundabouts [9].

In the single-lane roundabout, a capacity of a single entry lane conflicted by one circulating lane (e.g., a Single-lane roundabout is based on the different flow). The equation for estimating the capacity is given as follows

$$Ce, pce = 1{,}130e^{(-1.0*10)^{-3}\text{vc,place}}$$

Where

Ce, pc = Lane capacity, adjusted for heavy vehicles (pc/h)

Vc, pce = conflicting flow rate (pc/h)

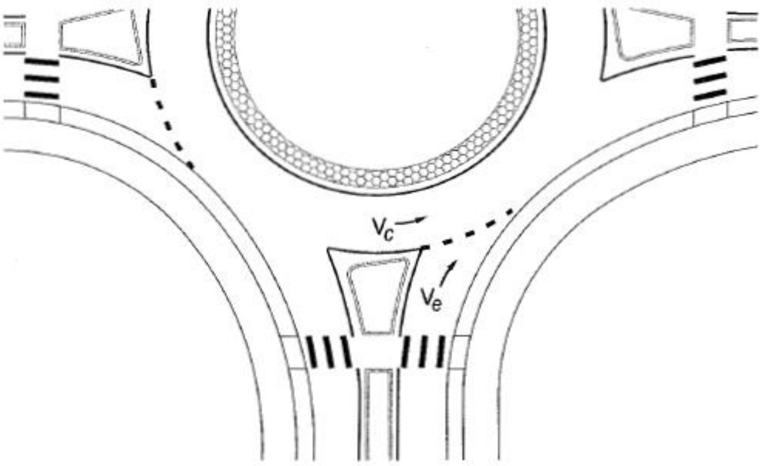


Figure 2.6: Single lane Roundabouts source [10].

2.3.3 Multilane Roundabouts

Multilane roundabouts have at least one entry or exit with two or more lanes and more than one circulating lane. The operational practice for trucks negotiating roundabouts is to encroach on adjacent lanes.

Figure 2.7: Multilane Roundabouts [10].

The number of entry, circulating, and exiting lanes may vary throughout the roundabout. Because of the many possible variations, the computational complexity is higher than for single-lane roundabouts. The definition of headways and gaps for multilane facilities is more complicated than for single-lane facilities. If the current roadway indeed functions as a multilane facility, then motorists at the entry perceive differences in both the inside and outside lanes in some integrated fashion. Some drivers who choose to enter the roundabout via the right entry lane will yield all traffic in the circulatory roadway due to their uncertainty about the path of the moving vehicles. The number of entry, circulating and exiting lanes may vary asunder here.

I. Capacity for Two-Lane Entries Conflicted by One Circulating Lane

The capacity of each entry lane conflicted by one circulating lane as follows:

$$Ce, pce = 1{,}130e^{(-1.0*10)^{-3}\text{vc,place}}$$

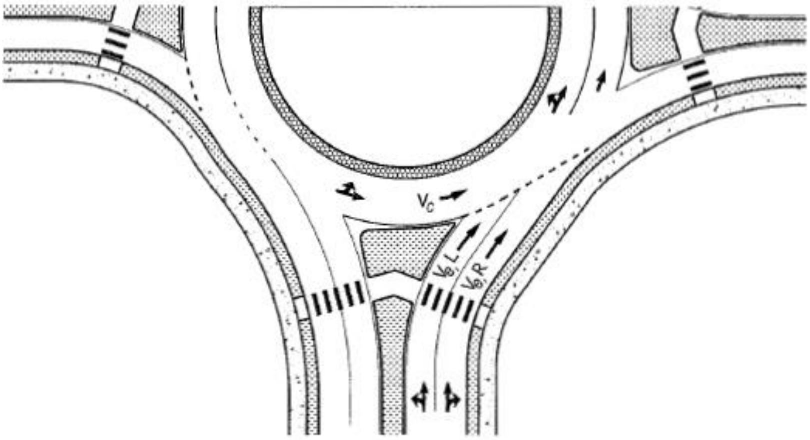


Figure 2.8: Capacities for two-lane entries conflicted by one circulating lane [10].

II. Capacity for One-Lane Entries Conflicted by Two Circulating Lanes

The capacity of a one-lane roundabout entry conflicted by two circulating lanes given as follows

$$Ce, pce = 1{,}130e^{(-0.7*10)^{-3}\text{vc,place}}$$

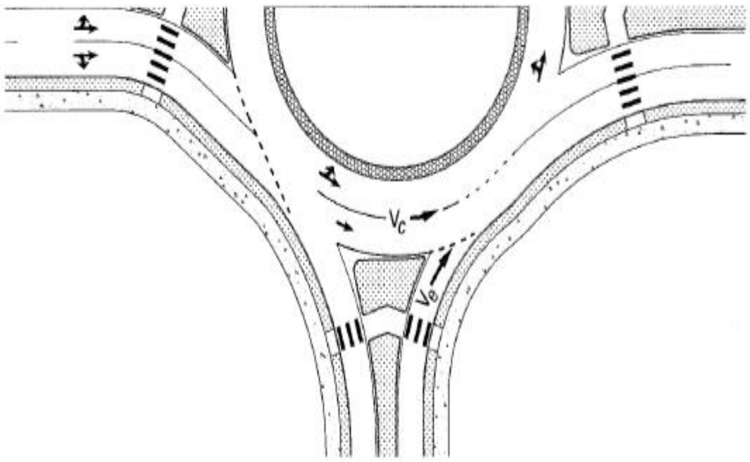


Figure 2.9: Capacities for one-lane entries conflicted by two circulating lane [10].

III. Capacity for Two-Lane Entries Conflicted by Two Circulating Lanes

Equation bellows give the Capacity of the right and left lanes, respectively, of a two-lane roundabout entry conflicted by two circulating lanes

$$Ce, R, pce = 1{,}130e^{(-0.7*10)^{-3}\text{vc,pce}}$$

$$Ce, L, pce = 1{,}130e^{(-0.75*10)^{-3}\text{vc,pce}}$$

Where

Ce, R, pce = Capacity of the right entry lane, adjusted for heavy vehicles (pc/h)

Ce, R, pce = Capacity of the left entry lane, adjusted for heavy vehicles (pc/h)

Vc, pce = Conflicting flow rate (total of both lanes)(pc/h)

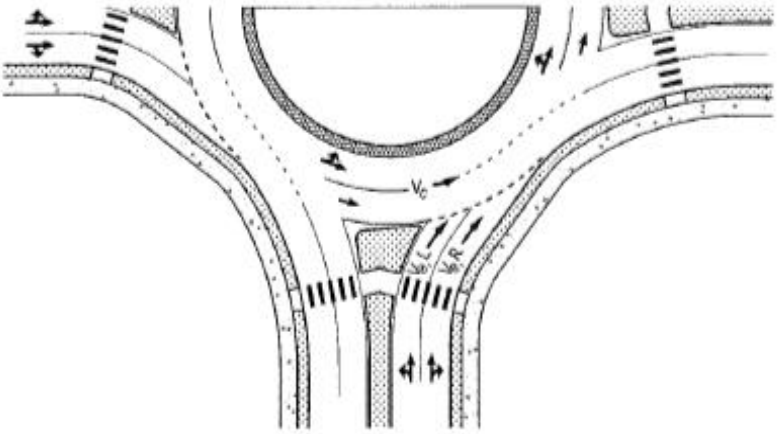


Figure 2.10: Capacity for Two-Lane Entries Conflicted by Two Circulating Lanes **[10]**.

2.3.4 Teardrop Roundabout

Teardrops are usually associated with ramp terminals at interchanges: typically, at diamond interchanges. Teardrop roundabouts allow the "wide node, narrow link" concept. Unlike round roundabouts, teardrops do not allow for continuous 360° travel. This design offers some advantages at interchanges. Traffic traveling on the crossroad (link) between ramp terminal intersections (nodes) does not encounter a yield as it enters the teardrop intersections. Because this improves traffic throughput on the crossroad between the ramps, it reduces the need for additional lane capacity, thus keeping the cross-section between the ramp terminals as narrow as possible [11].

Figure 2.11: Teardrop Roundabout [11].

2.4 Capacity

Capacity has been defined as the maximum sustainable number of vehicles to Travers a location within a given period under prevailing condition. For roundabouts, this means that each approach has a capacity for entering vehicles traversing the yield line. The capability is dynamic due to continually varying traffic composition (heavy cars, motorcycles, and bicycles), a proportion of turning vehicles, driver population characteristics, and weather conditions. For example, a roundabout that services nearly all heavy cars at one time of the day could be expected to have different capacity during a time when only passenger cars are repaired. Varying conditions are the reason that capacity must be thought of regarding what flow rates can repeatedly be observed during peak periods and no the maximum flow ever observed [10].

Capacity is the primary determinant of the performance measures such as delay, queue length and stops rate while performance is expressed regarding a degree of saturation (demand volume-capacity ratio).

2.4.1 The Major Factors affecting Roundabout capacity

2.4.1.1 Geometric Configuration

The geometric elements of the roundabout also affect the rate of entry flow. The essential geometric feature is the width of the entry and circulatory roadways, or the number of lanes at the entrance and on the roundabout. Two entry lanes permit nearly twice the rate of entry flow as does one lane. More full circulatory roadways allow vehicles to travel alongside, or follow, each other in tighter bunches and so provide longer gaps between groups of cars. The flare length also affects the capacity. The inscribed circle diameter and the entry angle have minor effects on size.

The draft Highway Capacity Manual 2000 is limited to the capacity analysis of single-lane roundabouts and is insensitive to the full range of geometric configurations possible with roundabouts [10].

2.4.1.1.1 Number of Lanes and Lane width

The roundabout is characterized as single lane where having a single lane entry at all legs and one circulatory path. Single lane roundabouts have 25,000 vehicles per day and Reduce in total Crashes by 35 percent. While double-lane roundabouts have 45,000 cars per day and Reducing Injury Crashes by 76 percent [12].

Figure 2.12 below shows a comparison of the expected capacity for both the single-lane and double-lane roundabouts. Again, it is evident that the number of lanes, or the size of the entry and circulating roadways, has a significant effect on the entry capacity [13].

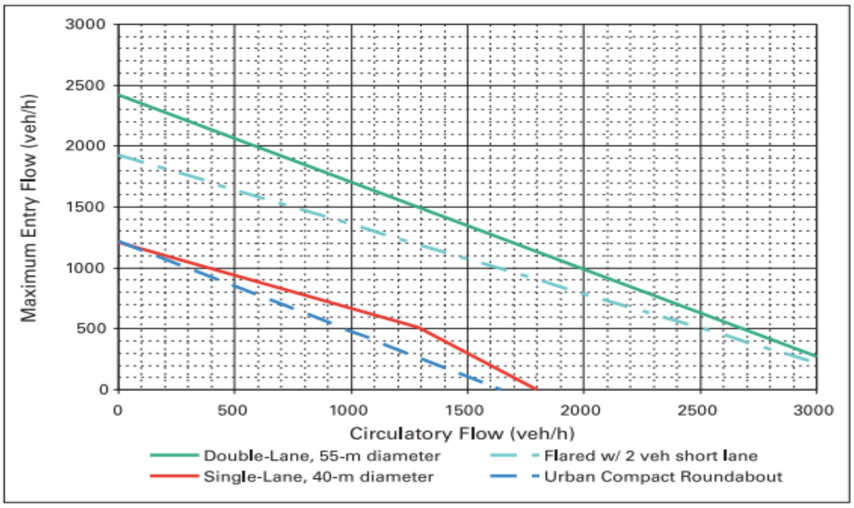


Figure 2.12: Comparisons of single-lane and double-lane roundabouts [13].

2.4.1.2 Traffic volume and Flow rate

Volume and flow rata are two measures that quantify the amount of traffic passing a point on a lane or roadway during a given time interval. These terms are defined as flows:

Volume:-the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval.

Flow rate:-the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than 1hr, usually 15min.Volume and Flow rate variables that quantify demand, that is, the number of vehicles occupants or drivers.

2.4.1.3 Drivers and Vehicles Characteristics

As NCHRP Report 572 Confirmed that roundabout geometry alone is not sufficient for modeling capacity of roundabout and Driver behavior is the most significant variable affecting roundabout performance [14].

The Highway capacity manual recommended that "because driver behavior appears to be the most significant variable affecting roundabout performance, calibration of the models to account for local driver behavior and changes in driver experience over time is highly recommended to produce accurate capacity estimates [6].

When the circulating flow is low, drivers at the entry can enter the roundabout without significant delay. The more significant gaps in the current flow are more useful to the entering vehicles, and more than one vehicle can enter each gap. As the current flow increases, the size of the gap in the moving flow decrease, and the rate at which cars can enter also decreases. Note that when computing the capacity of a particular leg, the actual circulating flow to use may be less than demand flows, if the entry capacity of one leg contributing to the rotating flow is less than demand on that leg [15].

2.4.1.4 Pedestrian effects on entry capacity

Pedestrians crossing at a marked crosswalk that gives them priority over entering motor vehicles can have a significant impact on the entry capacity. In such cases, if the pedestrian crossing volume and circulating volume are known, the vehicular size should be factored (multiply by M) according to the relationship shown in FHWA for single-lane and double-lane roundabouts, respectively [15].

Note that the pedestrian impedance decreases as the different vehicle flow increases. They provide additional guidance on the capacity of pedestrian crossings and should be consulted if the size of the crosswalk itself is an issue [16].

Table 2.1: Equation to the approximate pedestrian effect on one entry capacity [10].

One- Lane Entry Capacity Adjustment Factor for Pedestrians

Case, If $\mathbf{v_{c,pce}>881}$ $\mathbf{f_{ped} = 1}$ ……………….2.1

Else if

$\mathbf{f_{ped} \leq 1}$ $\mathbf{f_{ped}} = 1\text{-}.00013\mathbf{n_{ped}}$ …………2.2

Else

$$\mathbf{f_{ped}} = \frac{\mathbf{1{,}119.5-0.715fv_{c,pce}-0.644fn_{ped}+0.00073fv_{c,pce}n_{ped}}}{\mathbf{1{,}119.6-0.654fv_{c,pce}}} \quad \text{……………2.3}$$

Where

f_{ped} $= entry\ capacity\ adjustment\ factor\ for\ pedestrian$

n_{ped} $= number\ of\ conflicting\ pedestrians\ per\ hour$(P/h)

$v_{c,pce}$ $= conflictung\ vehicular\ flow\ rate\ in\ the\ circulatory\ road\ pc/h$

For two-lane entries, the model shown in Table 2.2 below can be used to approximate this effect. These equations are illustrated and share the assumption as before that pedestrians have absolute priority

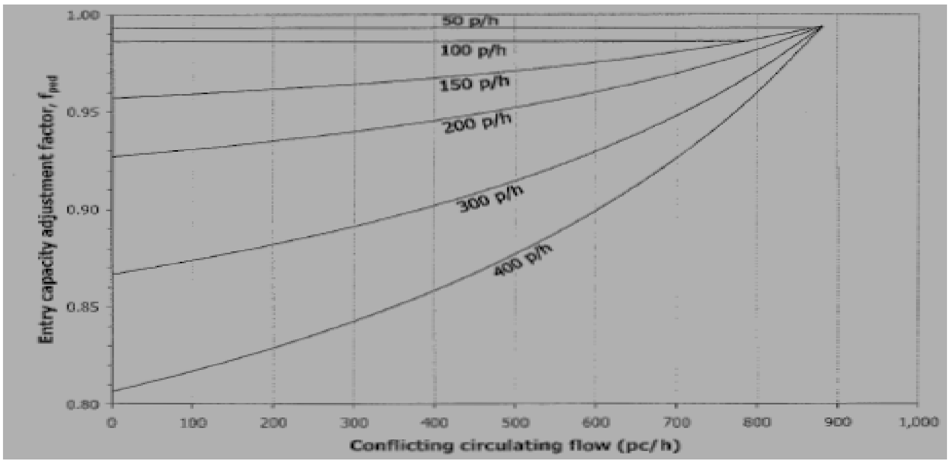


Figure 2.13: Pedestrian effect factors at one entry capacity [16].

Table 2.2: The equation to the approximate pedestrian effect on two entry capacity [10].

Case	Two- Lane Entry Capacity Adjustment Factor for Pedestrians
If	$\mathbf{n_{ped} < 100}$
	$$\mathbf{f_{ped} = \min\left[1 - \frac{n_{ped}}{100}\left(1 - \frac{260.6 - 0.329 v_{c,pce} - 0.381 * 100}{1{,}380 - 0.5 v_{c,pce}}\right), 1\right]} \ldots\ldots 2.4$$
	Else
	$$\mathbf{f_{ped} = \left[\frac{1{,}260.6 - 0.329 v_{c,pce} - 0.381 n_{ped}}{1{,}380 - 0.5 v_{c,pce}}, 1\right]} \ldots\ldots 2.5$$

Where

f_{ped} $= entry\ capacity\ adjustment\ factor\ for\ pedestrian$

n_{ped} $= number\ of\ conflicting\ pedestrians\ per\ hour$(P/h)

$v_{c,pce} =$ $conflicting\ vehicular\ flow\ rate\ in\ the\ circulatory\ roadway$

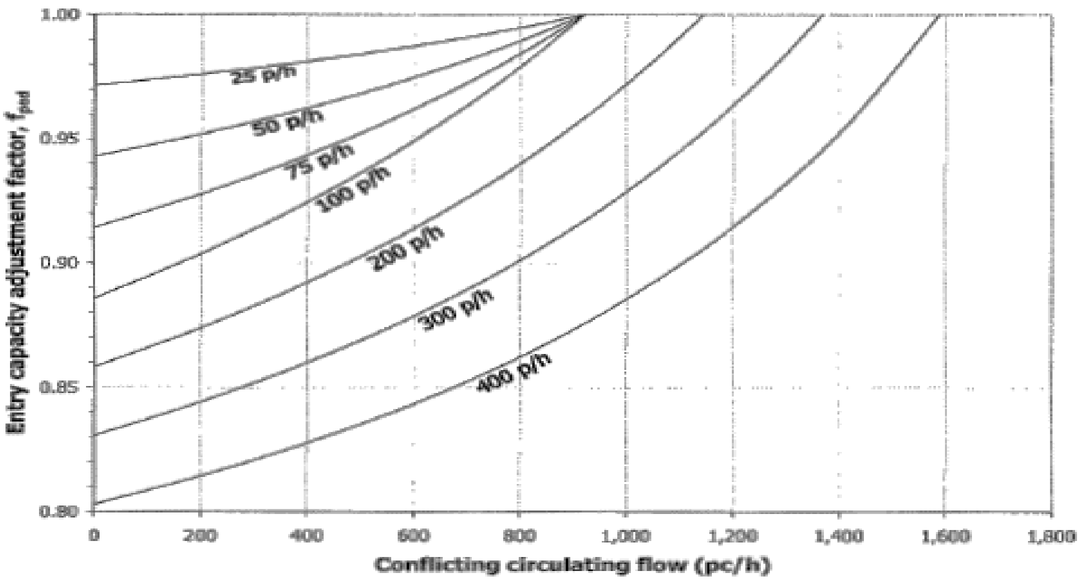


Figure2.14: Pedestrian effect factors at two entry capacity [16]

2.5 Performance measures

Three performance measures are typically utilized to estimate the performance of a given roundabout design: degree of saturation, delay, and queue length. In all cases, a capacity estimate must be obtained for entry to the roundabout before a specific performance measure can be computed.

2.5.1 Degree of Saturation

A degree of saturation is the ratio of the demand at the roundabout entry lanes to the capacity of the entry. It provides a direct assessment of the sufficiency of a given design. While there are no absolute standards for a degree of saturation, some sources suggest that the degree of saturation for an entry lane should be less than 0.85 for satisfactory operation. Once the degree of saturation exceeds the range, the process of the roundabout will likely deteriorate rapidly, particularly over short periods of time. Queues may form and delay to increase exponentially [17].

2.5.2 Delay

Delay is a standard parameter used to measure the performance of an intersection. The Highway Capacity Manual, Transportation Research Board Special Report 209 identifies delay as the primary measure of effectiveness for both signalized and un-signalized intersections, with a level of service determined from the delay estimate (18). Currently, however, the Highway Capacity Manual only includes control delay, the delay attributable to the control device. As 1994 HCM, based on Akçelic and Trout beck (1991) define that, Control delay is the time that a driver spends queuing and then waiting for a gap acceptance in the circulating movement while at the front of the queue.

2.5.3 Queue Length

Queue length is essential when assessing the adequacy of the geometric design of the roundabout approaches [19].

2.5.4 Level of Service (LOS)

It is the qualitative measurement considering operational condition with the traffic stream such as time, travel, speed, freedom to maneuver, traffic interruption, comfort, and convenience. It measures the traffic quality service. As defined in Highway Capacity Manual, the level of service (LOS) criteria for automobiles in roundabouts is given in the following table.

For assessment LOS at the approach and intersection levels, LOS is based solely on control delay. The thresholds are based on the considered judgment of the Transportation Research Board Committee on [10].

Table 2.3: Level of services thresholds [10].

Control Delay $\frac{s}{veh}$	LOS by Volume –to-Capacity Ratio $\frac{v}{c} \leq 1.0$	$\frac{v}{c} > 1.0$
0 - 10	A	F
>10 - 15	B	F
>15 – 25	C	F
>25 – 35	D	F
>35 – 50	E	F
>50	F	F

2.6 Roundabout Capacity Evaluation Models

There are several models to evaluate the capacity and performance of roundabout. From this three categories of models are reprinted in this context

I. Gap acceptance model: predict capacity as a function of critical gap and follow –up headway driver behavior parameters

II. . Geometric capacity models show that size is correlated to roundabout geometry such as entry width and inscribed circle diameter.

III. Hybrid models combine elements of both methods to predict capacity (4).

In general, this research has gone into developing the mathematical model and along with the associated software package used in this study by using Highway Capacity Manual Method and SIDRA intersection Software Package. Because, it used semi Empirical –Analytical approach since it uses some geometric elements, Traffics, Driver behavior and Environmental factors for the analysis.

2.6.1 Highway Capacity Manual Model

Some capacity models use traffic flow theory related to gap acceptance, of which two main parameters area critical gap (t_c) and follow-up headway (t_f). Some research has also used the term critical headway and follow-up time to represent the same parameters, but definitions are consistent throughout the literature. Gap acceptance models have been used for determining capacity at other un-signalized intersections, such as two-way stop-controlled or yield-controlled junctions [10].

Roundabouts and these other un-signalized intersection types share a familiar traffic flow theory concept of a priority, or significant, traffic stream, conflict a minor, or entering, traffic stream. A capacity of the entering stream is then a function of how time gaps between considerable stream vehicles are distributed and how well the minor stream utilizes these gaps. The following definitions of a critical gap and follow-up headway further clarify the idea. Follow-up headway is the amount of time between entering vehicles that are utilizing the same gap in circulating traffic. Unlike critical gap, follow-up headway can be directly measured in the field by taking a sample average and standard deviation [6].

As Highway Capacity Manual define that the capacity of a roundabout is directly influenced by flow patterns. The three flows of interest that affect the functions are, the entering flow, the current flow, and the exiting flow. The capacity of an approach decreases as the conflicting flow increases. In general, the primary contradictory flow is the current flow that passes directly in front of the subject entry. While the present flow directly conflicts with the entry flow, the exiting flow may also affect a driver's decision to enter the roundabout. This situation is similar to the effect of the right-turning flow approaching from the left side of a two-way STOP-controlled intersection. Until the drivers completely exit maneuvering or right turn, there can be some uncertainty in the decision of the driver at the yield or stop line about the intentions of the existing or turning vehicles [6].

The methodology does not necessarily apply to other types of circular intersections such as rotaries, neighborhood traffic circles, or signalized traffic circles because these types of circular intersections usually have geometric or traffic control elements that deviate from those used in roundabouts.

The required data input in this methods are: Number and configuration of lanes on each approach, Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak 15 min, or Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak hour, and a peak hour factor for the hour, Percentage of heavy vehicles, Volume distribution across lanes for multilane entries; and Length of analysis period, generally a peak 15-min period within the peak hour. Any 15-min period can be analyzed [6].

2.6.1.1 Entering flow

Entry flows depend on the number of entry lanes. For single-lane entries, the entry flow rate is the sum of all movement flow rates using that entrance. For multilane entries or entries with bypass lanes, or both, the following procedure may be used to assign flows to each lane:

1. If a right-turn bypass lane is provided, the flow using the bypass lane is removed from the calculation of the roundabout entry flows.
2. If only one traffic lane is available for a given movement, the flow for that change is assigned only to that lane.
3. The remaining flows are assumed to be distributed across all lanes, subject to the constraints imposed by any designated or de facto lane assignments and any observed or estimated lane utilization balances. Five generalized multilane cases may be analyzed with this procedure. For examples in which a movement may use more than one lane, a check should first be made to determine what the assumed lane configuration may be. This may differ from the designated lane assignment based on the specific turning movement patterns being analyzed. These assumed lane assignments are given in [10].

2.6.1.2 Circulating flow

The current flow opposing a given entry is defined as the flow conflicting with the entry flow (i.e., the flow passing in front of the splitter island next to the subject entrance). The circulating flow rate calculation for the northbound circulating flow rate is illustrated and numerically exhibited in Equation 2.6 below

All flows are in passenger car equivalents.

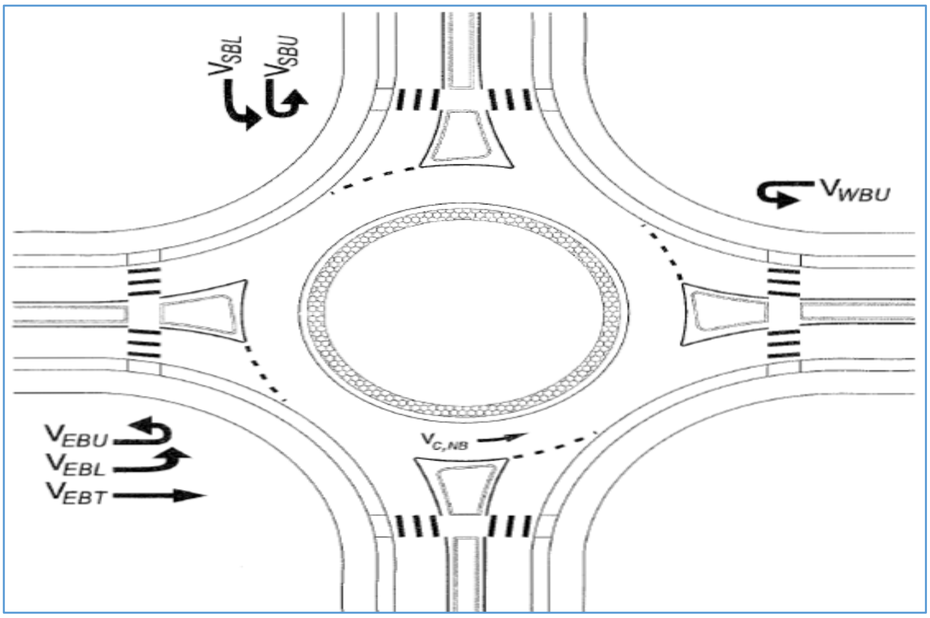


Figure 2.15: Circulating flow [10].

$v_{c,NB,pc} = v_{c,WBU,pc} + v_{c,SBL,pc} + v_{c,SBU,pc} + v_{c,EBT,pc} + v_{c,EBL,pc} + v_{c,EBU,pc}$……2.6

For other bound similar calculation is conducted (10)

2.6.1.3 Exiting flow

The dissipating flow rate for a particular leg is used mainly in the calculation of different flow for right-turn bypass lanes. The exiting flow calculation for the southbound exit is illustrated in Equation 2.15 below

If a bypass lane is present on the immediate upstream entry, the right-turning flow using the bypass lane is deducted from the existing flow. All flows are in passenger car equivalents [6].

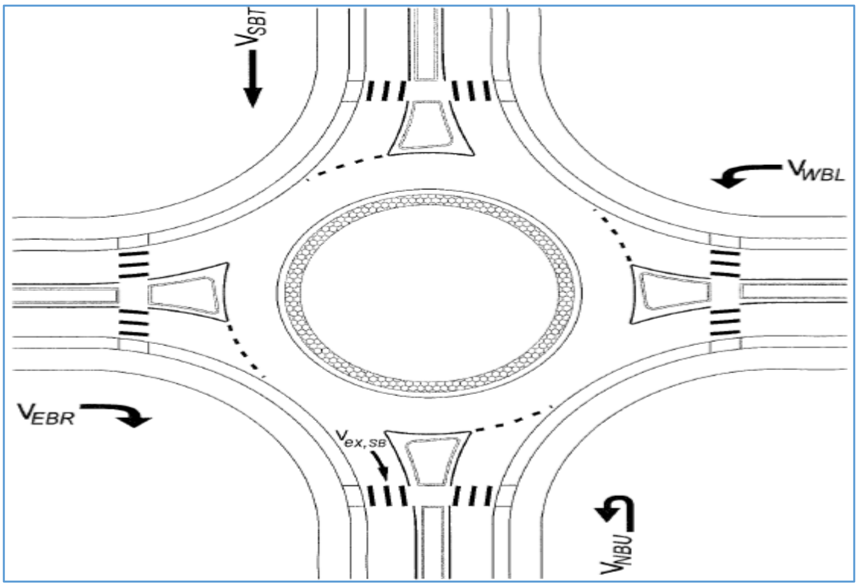


Figure 2.16: Exiting Flows

$$v_{ex,NB,pce} = v_{NBU,pce} + v_{WBL,pce} + v_{SBT,pce} + v_{EBR,pce} - v_{c,EBR,pcE,bypass} \text{ ……… .2.7}$$

Similar to south, west and eastbound [6].

2.6.1.4 Right-Turn Bypass Lanes

Two common types of right-turn bypass lanes are used at both single-lane and multilane roundabouts.

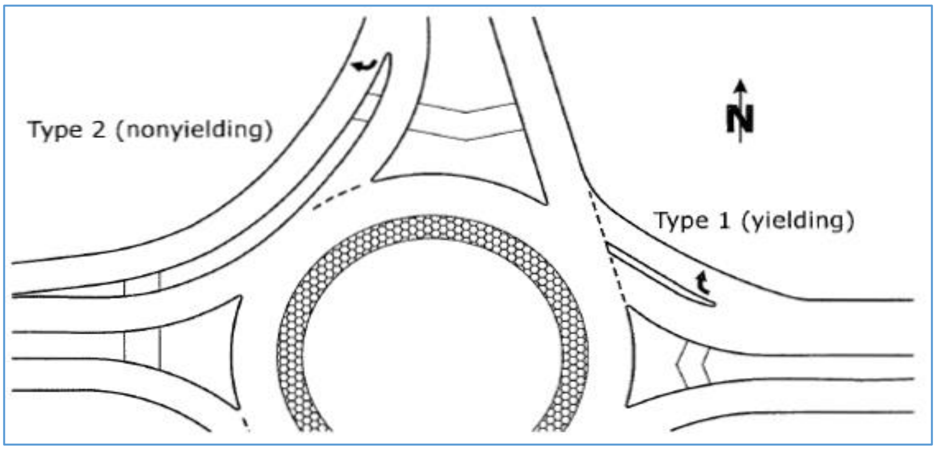


Figure 2.17: Right turn bypass lane [10].

2.6.1.4.1Type 1(Yielding Bypass Lane)

A Type 1 bypass lane terminates at a high angle, with right-turning vehicle yielding to dissipating flows. Right-turn bypasses lanes not included in the recent national research.

However, the capacity of a yield bypass lane may be approximated by using one of the capacity formulas given previously by treating the exiting flow from the roundabout as the circulatory flow and treating the current in the right-turn bypass lane as the entry flow [6].

The capacity for a bypass lane opposed by one exiting lane can be approximated by using Equation 2.8 and 2.9below

$$C_{bypass,pce} = 1{,}130e^{(-1.0*10)^{-3})}v_{ex,place} \quad \ldots\ldots 2.8$$

The capacity for a bypass lane opposed by two exiting lanes can be approximated by using Equation

$$C_{bypass,pce} = 1{,}130e^{(-0.7*10)^{-3}}v_{ex,place} \quad \ldots\ldots 2.9$$

Where

$C_{bypass,place}$ = Capacity of the bypass lane, adjusted for heavy vehicles (pc/)

$v_{ex,place}$ = Conflicting existing flow rate (pc/h)

2.6.1.4.2 Type 2 (Non yielding Bypass Lane)

A Type 2 bypass lane merges at a low angle with existing traffic or forms a new lane adjacent to existing traffic. The capacity of a merging bypass lane has not been assessed in the United States. Its capacity is expected to be relatively high due to a merging operation between two traffic streams at similar speeds [6].

2.7 SIDRA INTERSECTION Software Package

The software has evolved over 30 years of research in signalized and un-signalized intersections under the guidance of the Australian Research Board and Akcelic & Associates [20]. Version 5.1 of SIDRA was used for this evaluation of capacity and performance of roundabout. Regarding capacity analysis, some parameters are considered based on recent research had been used in SIDRA, and are not shown in the material source used during this investigation.

For roundabouts, gap-acceptance parameters (especially follow up headway and critical gap) are key parameters to represent driver behavior. The gap-acceptance parameters, as well as the overall approach and circulating road lane use, are affected by roundabout geometry as well as the overall demand flow levels and patterns, which in turn affects capacity and performance significantly [20].

2.8 Problems on human and economy of the country

2.5.1 Traffic congestion

Congestion involves slower speeds, queuing and travel times increased, which apply costs on the economy and generate several impacts on urban areas and their inhabitants. Congestion also has a variety of indirect implications including the marginal environmental and resource congestion impacts. It also includes effects on quality of life, stress, and safety as well as implications for non-vehicular road space occupants such as the users of sidewalk and frontage of properties. Policy-makers should assure that cost-benefit assessment or other policy evaluation tools include identifying these impacts as well as considering a broader consideration such as the type of cities people want. Also, long travel time to reach a destination that affects business user's productivity time, increasing fuel consumption are the primary impacts of road traffic congestion which still prevail [21].

CHAPTER THREE

RESEARCH METHODOLOGY

The methodology followed to achieve the objective of the study was described as follows. The areas of study were visited and selected based upon those assumed to have more problems and appropriate for research and aimed to solve the problem of traffic congestion in Addis Ababa City. Secondly, after the sites were identified two roundabouts were selected, intensive review of literature associated with the title of the research was collected and geometric data and traffic data of these roundabouts were collected. Thirdly the received data was prepared as appropriate for input of software. The general methodologies are described in detail as the following.

3.1 Study Area

The study area was conducted in Addis Ababa City, a Capital City of Ethiopia, and geographically exists in the center of the country. Due to its economic growth and urban development, the Road utilization in this City is not fit with traffic demand.

For the goals of this research, several sites were visited for the inclusion of site selection and data collection. Unfortunately, some were not attended and selected due to distance, time and funding limitations. Some roundabouts were observed but uncomfortable to install the video recorder for data collection. Also, some roundabouts were dropped from the study due to AACRA's information that such intersections would be demolished and to be changed to a signalized intersection. However, from those roundabouts, two roundabouts were selected namely Abune Petros and German (Mekanisa) roundabouts to cover physical features, traffic volumes, and traffic condition in Addis Ababa City. Primary observations showed that these Roundabouts have potential high traffic congestion and the principle of the possible representative of the target population of the roundabout.

3.1.1 Abune Petros Roundabout

This site was located in Arada Sub City around Georges at Abune Petros square. Arada is one of the ten Addis Ababa sub Cities which is geographically located in the Northern area of the City nearby Centre. It borders with the districts of Gullale, Yeka, Kirkos, Lidata and Addis Ketama Sub cities. The study area is precisely located just west of Addis Ababa's impressive City Hall, down the hill on the main road to the Merkato district at Abune Petros square.

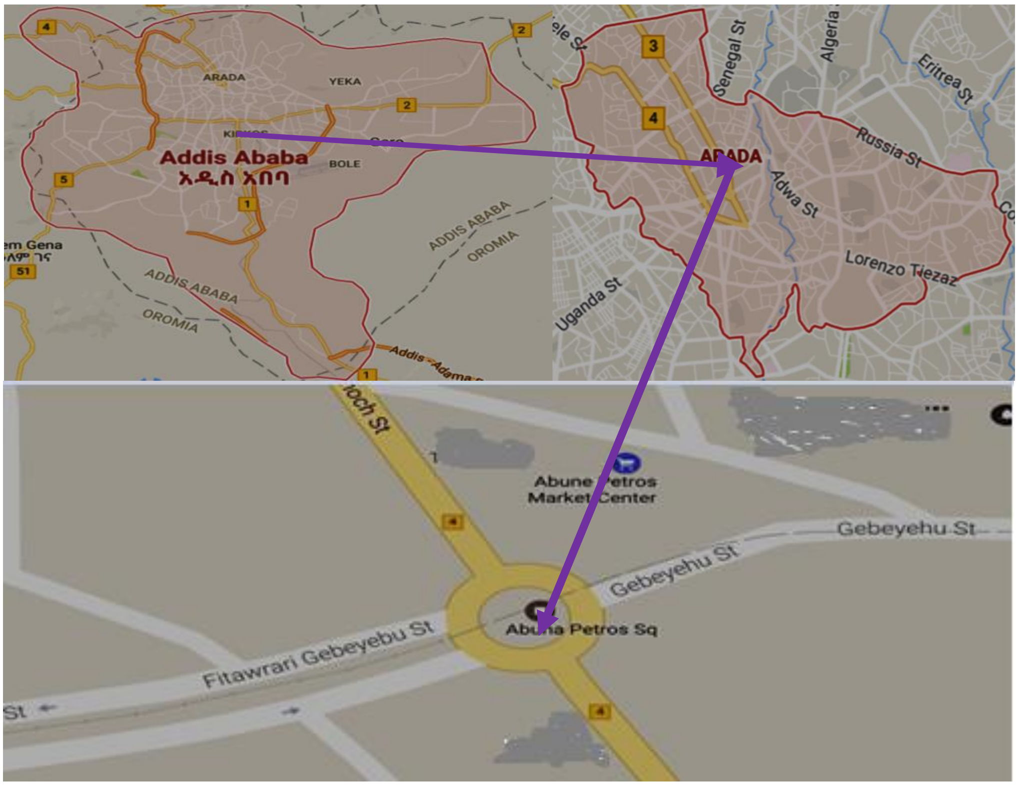


Figure 3.1: Location of Abune Petros Roundabout [Source: Google Map]

3.1.2 German (Mekanisa) Roundabout

This study area is located in Nifas Silk Sub City, and it connects the roads come from Sarbet, Gofa, Hana, and Jemo Michael direction. In this area, different government and non-government offices, as well as the residential areas exist. Due to this, the proportion of Car, minibus, and mid-bus were very high. The following Photo was captured from German roundabouts during the traffic survey on June 28/2017 at time 8:00 am.

Figure 3.2: Photo captured from German roundabouts during the traffic survey

(Source: Survey, June 28/2017 at time 8:00 am)

3.2 Study Design

For this study, a research type used both qualitative and quantitative approach to maintain and remains unbiased as possible, and also analytical and descriptive research method has been conducted. Since Analytical models are based on traffic flow theory measures and formulation of the relationship between those field and performance measures.

Knowledge of mathematical method and SIDRA Software package were kept in mind to ensure analyses and comparisons of both plans were based on scientific evidence and engineering judgment. The purposive data collection was conducted for this research. The input data for both analyses were the overall roundabout geometry (configuration of approach roads, number of approaches and circulating road lanes, and allocation of lanes to movement) measured from the site, the traffic data gathered and converted to passenger car equivalence, and environmental factors (effects of grade, weather condition, roadway condition and other natural and manmade factors on traffic flow) were calibrated to analyze this the capacity and performance. Finally, results and discussions were organized. Figure 3.3 shows a high-level overview of the methodology used for evaluation illustrated.

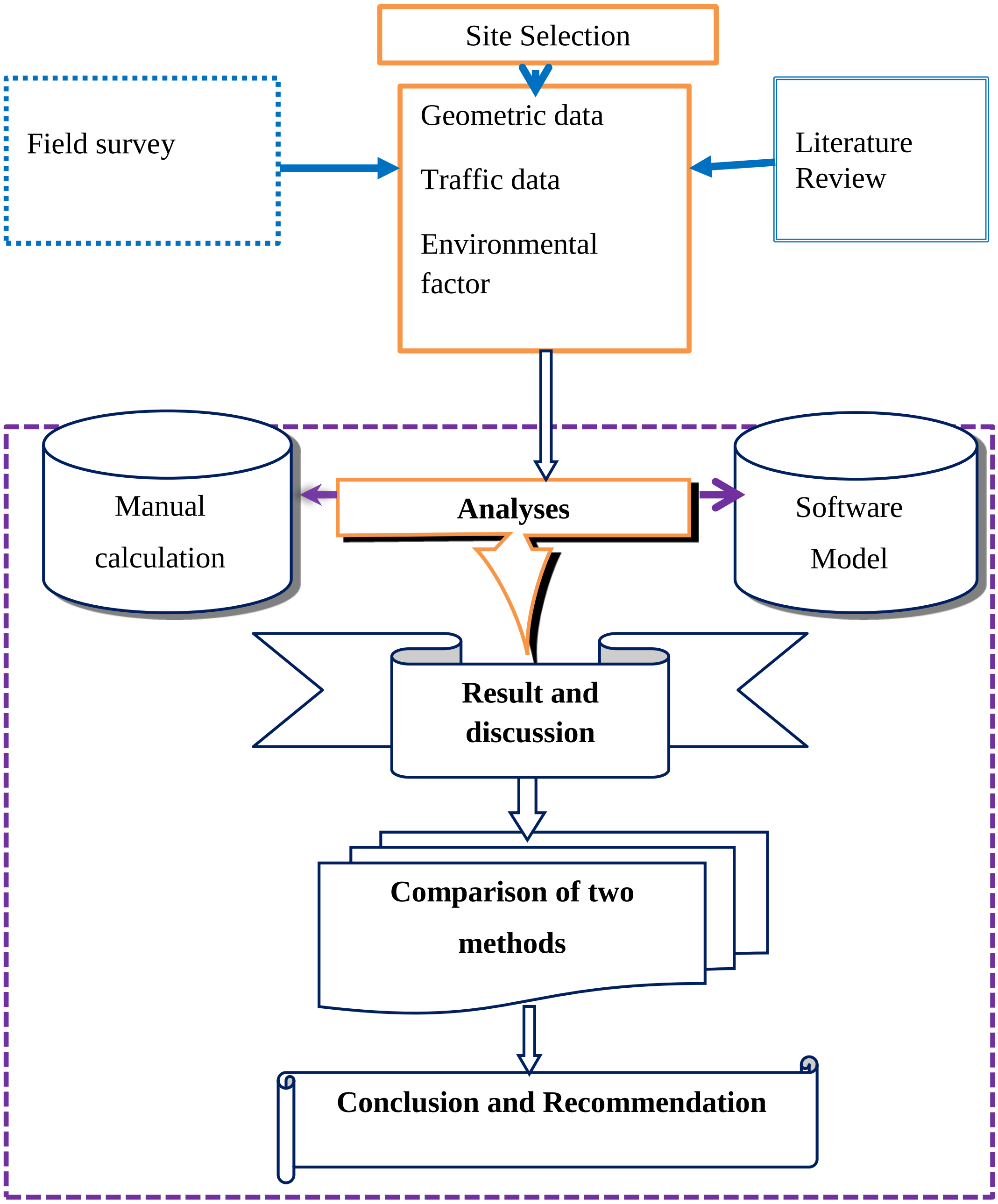


Figure 3.3: Study design flow chart

3.3 Study Variables and Descriptions

3.3.1 Dependent Variables:

Level of Service (LOS) of Roundabout

3.3.2 Independent Variables:

Traffic volume, Saturation flow rate, Travel time & delay

Road geometry: Number of lanes, width, radius, island diameter

3.3.2.1 Traffic Volume

Traffic volume is the total number of vehicles or pedestrians that pass over a given point or section of a lane, or roadway during a given time interval. It can vary considerably with time; it means the variation of traffic volume can be further identified within minutes, hourly, daily, and weekly and seasons.

Volume or traffic flow is a parameter common to both uninterrupted and interrupted flow characteristics, but speed and density apply primarily to continuous flow. Some parameters related to flow rates, such as spacing and headway, also are used for both types of flow characteristics: others parameters, such as saturation flow or gap, are specific to interrupted flow [22].

3.3.2.1.1 Peak Hour Volume

It is the hourly highest traffic volume in the given direction, lane or lane group of traffic volume can be further identified within minutes, hourly, daily, weekly and seasons. The Peak hour volume would be the sum of four peak 15minutes volume of passenger car units in given hour.

3.3.2.1.2 Peak Hour Factor

Peak hour factor (PHF) is used in the capacity analysis by the Highway Capacity Manual, which selected 15minute flow rates as the basis for most of its procedures. It is the average volume during the peak 60 minute period V60av divided by four times the average size during the peak 15minutes's period V15av [23].

$$PHF = \frac{V}{(4*Vp15minute)} \qquad \text{3.1}$$

Where

PHF = Peak hour factor

V = Hourly Volume (Veh/hr.)

V_p15min = Peak volume during the 15minute within the hour (Veh/15min)

The peak hour factors values can vary based upon traffic flow conditions which indicate that the smaller amount shows, the greater variability of flow or low traffic volume while the higher value indicates the little variability flow value or high amounts [24].

3.3.2.2 Saturation Flow Rate

Saturation flow rate for a roundabout is the number of vehicles that would pass over the point of intersection. Critical gap and Follow-up Headway can affect the saturation flow rate. On the other hand, if the approaches have very narrow lanes, traffic will naturally provide longer gaps between vehicles, which will reduce the saturation flow rate. If there are large numbers of turning movements or large numbers of trucks and busses, the saturation flow rate will be reduced [25].

3.3.2.2.1 Critical gap and Follow-up Headway

As SIDRA INTERSECTION version 5.1 defines that, critical gap is the minimum time (headway) between successive vehicles in the opposing (significant) traffic stream that is acceptable for entry by opposed (minor) stream vehicles, and While, Follow-up Headway is the average headway between successive opposed (small) stream vehicles entering a gap available in the opposing (significant) traffic stream. The follow-up Headway (second) is saturation (queue discharge) headway, and the corresponding saturation flow rate (vehicles per hour) in a gap-acceptance analysis is 3600/Follow-up Headway. This opposing flow rate is reduced according to the proportion of time when acceptable gaps. For the roundabout, SIDRA INTERSECTION estimates the critical gap and follow-up headway parameters as the function of the island diameter, circulating flow rate, and other factors when the checkboxes for a crucial gap and follow-up headway unchecked.

Departure headway is the elapsed time between the front vehicles of the first and that of the second cars over the stop line at an intersection. For interrupted flow, headway represents the time between the passage of front axle of one vehicle and the front axle of the next car over a given cross-section of the progress [22].

$$S = \frac{3600}{\mathrm{t}f} \quad \ldots\ldots 3.3$$

Where

tf = follow-up headway (second)

S = saturation flow rate (Veh/h).

3.4 Data Collection

Data collection preceded by selecting locations for study, gathering operational field data, and lastly reducing data.

Obtaining operational data was one part of a larger and comprehensive evaluation of roundabouts, which formed the basis for the other studies [26]. This section describes the data collection procedure which is relevant to the roundabout analyses. The type of data gathered as described in the following.

3.4.1 Geometric Data Collection

For the requirement of SIDRA software and Manual calculation for capacity and performance analyses based on Highway Capacity Manual. The geometric data were collected including the number of a circulatory lane, island diameter, circulatory roadway width, inscribed circles diameter, number of entry lane, average lane width at entry, entry angle and entry radius. These data were collected from Addis Ababa City from Abune Petros and German Roundabouts. The collected geometric data from these roundabouts were summarized in table 3.1 and 3.2

Table 3.1: Summary of geometric data from Abune Petros roundabout

s/no	Roundabout name	Number of Legs	Number of circulatory lanes (m)	Island diameter(m)	Circulatory roadway width(m)	Inscribed circle diameter(m)
1	Abune Petros Roundabout	4	2	27.8	12	51.8
s/no	Roundabout name	Legs Name	Number of entry lane	Average lane width(m)	Entry angle	Entry radius(m)
1	Abune Petros Roundabout	Seb./ Babur	2	6	26	36
		Churchill	2	6	64	36
		Minilik	2	6	25	40
		Merkato	2	6	65	45

Table 3.2: Summarized geometric data from German Roundabout

s/no	Roundabout name	Number of Legs	Number of circulatory lanes (m)	Island diameter(m)	Circulatory roadway width(m)	Inscribed circle diameter(m)
1	German	4	3	50.8	16	76
s/no	Roundabout name	Legs Name	No. of entry lane	Average lane width(m)	Entry angle	Entry radius(m)
1	Jerman	Sarbet	2	5	45	50
		Hana	3	4.3	33	60
		Gofa	2	5	30	43
		Jemo Michael	3	4.3	45	40

3.4.2 Traffic Data collection

The traffics data which can influence the roundabout capacity and performance includes vehicles, pedestrian, motorcycles, bicycles, and animals. From these types of traffics, this study was focused on cars and pedestrian. Due to the number of Motorcycles and cycles are insignificant in their number during data collection at a field, both of them were omitted from traffic analysis. The movements of traffics and their volumes are essential parameters in capacity and performance analysis by using Manual Calculation and SIDRA software model the selection of study method should be determined using the count period. The count period should be represented the time of day, the day of the month, and month of the year for the study area. The count period should avoid special event, or compromise weather conditions [27].

Typical count periods are 15minutes or 2 hours for peak periods, 4 hours for morning and afternoon peaks, 6 hours for the morning, midday, and afternoon peaks, and 12 hours for daytime periods. The weather condition during the data collection was sunny weather. Before traffic data was collected, vehicles were categorized based on AACRA Classification system [28].

According to Nurhussien Hassan, 2015 stated in his study of Performance Evaluation of Selected Intersections in Bahir Dar City, the peak hour period was one hour [29].
Similar to this the traffics data was counted at two peak hours in the morning at 8:00 am to 9:00 am when the worker go to their workplace and in the afternoon at 5:30 pm to 6:30 pm when they back to their home from the work. The traffic volumes were collected by manual and video camera for each separated lanes. To get the accurate result, it is essential to count at least for seven days, but due to economic and time constraint, the count was done only for five days. From the five working days data, the three consecutive working days except Monday and Friday has been conducted in the analysis. The reason for the analysis was made for three working days, due to a Monday morning rush in the hour and a Friday evening rush out the hour. This implies that all workers flow to their workplace from their rest Sunday on Monday morning while on Friday afternoon all workers flow from their workplace to their home for rest Saturday and Sunday. These shows exceptionally high traffic volumes and has shown a significant volume variation relative to the three working days. In detail, the variety of traffic volume was exhibited in the following figure 3.4 and 3.9.
Due to this, the two days were not used in the analysis. Therefore, the capacity and performance were conducted based on three successive days that means Tuesday, Wednesday, and Thursday. From the selected sites, German Roundabout data was collected by video, while, traffic volume data at Abune Petros roundabout was collected manually by twelve people, because, it was difficult to install the video and capture the data. The registered vehicles and collected traffic data from each roundabout are summarized separately hereunder.

3.4.2.1 Vehicles Classification

Traffic counts were classified as given in Table 3.3 These rankings have been specified in the AACRA manual, Table 6.1 vehicle classification System.

Table 3.3: Vehicles classification system

Category	Includes
Car	Car, Utility, Minibus, 4WD
Light	Bus, 1Axle Truck
Medium	2Rear Axle Truck
Heavy	4Axle Truck
Articulated	Large Truck

Addis Ababa City Roads Authority (AACRA) Design Vehicle grouping and Traffic manual with annual vehicles population growth rate of 1.6%, mixed vehicles classifications are summarized as follows:

P. Cars = standard Car +Utility +Minibus + 4WD +bus with 12 seats up to 18 seats Light vehicles (Lv)

Heavy Vehicles (HV) = Medium (2Rear Axle Truck) + Heavy (4Axle Truck) + Articulated

The same to this, for roundabout analysis, SIDRA INTERSECTION defines a Heavy Vehicles with more than two axles or with dual tires on the rear axles, and consider other vehicles other than this as passengers' cars.

For roundabout analysis, SIDRA INTERSECTION defines the vehicles as the flowing vehicles classification system as input in software. Before using the Volumes dialog, select the Volume Data Method (HV Option) parameter in the drop-down list in the Options group of the ribbon to select the HV option you want to use, i.e., Separate LV & HV, Total Vehicles & HV (%) or Total Vehicles & HV (Veh). The volume data fields will be displayed according to the method chosen.

The three options used for specifying and displaying the vehicle's data in using the SIDRA INTERSECTION software are:

I. Separate LV and HV: different volumes for Light cars (LVs) and Heavy Vehicles (VHS) would be specified, E.g., LVs 900veh/h and HVs 100veh/h

II. Total Vehicles and HVs (%): Total Volume and precent of Heavy vehicles would be specified, e.g., whole 1000veh/hr. and ten precent HV, and

III**.** Total Vehicles and HV (Veh): Total volume and Heavy Vehicle Volume would be specified, e.g., 1000veh/hr. And 100 HV Veh /hr. (SIDRA INTERSECTION version 5.1)

Therefore, this research work used the third option.

3.4.2.2 Concept of Passenger Car Unit (Equivalents)

A Passenger Car Unit is a measure of the impact that a mode of transport has on traffic variables (such as headway, speed, density) compared to a single standard passenger car. This is also known as passenger car equivalent. For example, typical values of PCU (or PCE) are: Highway capacity is measured in PCU/hour daily [23].

The heavy-vehicle adjustment factor is based on the concept of passenger car equivalents or Passengers Car Unit (PCU). In the other hand, a passenger Car Unit is the number of passenger cars displaced by one truck and bus in the given traffic stream under prevailing conditions. Therefore, the passenger car unit (PCU) is the universally adopted unit of measures for traffic volume or capacity. The traffic flow with any vehicular composition can be expressed regarding its equivalent Passengers Car Unit. In the HCM 2000, the definition of PCE is given as "the number of passenger cars displaced by a single massive vehicle of a particular type under a specified roadway. Traffic and control condition [22].
In the HCM 2010, the PCU value of passenger cars is 1.0 because of maneuverability in any directions. Each vehicle type was given a single PCU equivalent to represent its relative disturbance to the flow under the prevailing traffic condition. Sometimes a set of PCU values is assigned to a particular type of vehicle to describe the various troubles in its presence in different traffic situation [30]. The PCU values of bus, Bicycle, and trucks are summarized in the table below.

Table 3.4: passenger cars equivalent factors

Movement Type	Left		Through		Right	
Vehicles types	Buses	Trucks	Buses	Trucks	Buses	Trucks
PUC ranges	1.34 - 2.4	2.4– 3.38	1.28-1.77	1.83–2.82	1.68– 2.27	2.252.83
Aveg. Pcu	1.87	2.89	1.5	2.3	1.98	2.68

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

Vehicles which are categorized under heavy vehicle were converted to total passenger car units (PCU) using the above factors for each 15min intervals in the peak hour traffic condition that is used as input for software. As it is shown in the table above the equivalent for passenger car unit is varies with a shared lane.

3.4.2.3 Time Period for Traffic Data Collections

The period for traffic data collection was selected based upon collecting traffic operations representative of average condition. Counting periods vary from low counts at spot points to consecutive numbers at permanent stations. Hourly counts are significant in all engineering design, while daily and annual traffic is essential in economic calculations, road system classification and investment programs.

Continuous numbers are made to establish national and local highway use, trends of usage and behavior and for estimating purposes. Some of the more commonly used intervals are [31].

Typical Weekday peak operation: traffic was collected from the week continuously five days (Monday, Tuesday, Wednesday, Thursday, and Friday), at the two peak hours (8:00-9: 00 am and 5:30-6: 30 pm) at the interval of 15minutes.

3.4.3 Vehicles Data from Abune Petros Roundabout

This roundabout is geographically located at the congestive place near to marketplace, and a big country's Bus station and also vehicles from different segments can connect to each other at this roundabout. These factors can be affected by the demand volumes to this study area. Under this traffic count, the vehicles were classified as given in Table 3.3 above.

Total 15minutes PCU at Abune Petros Roundabout

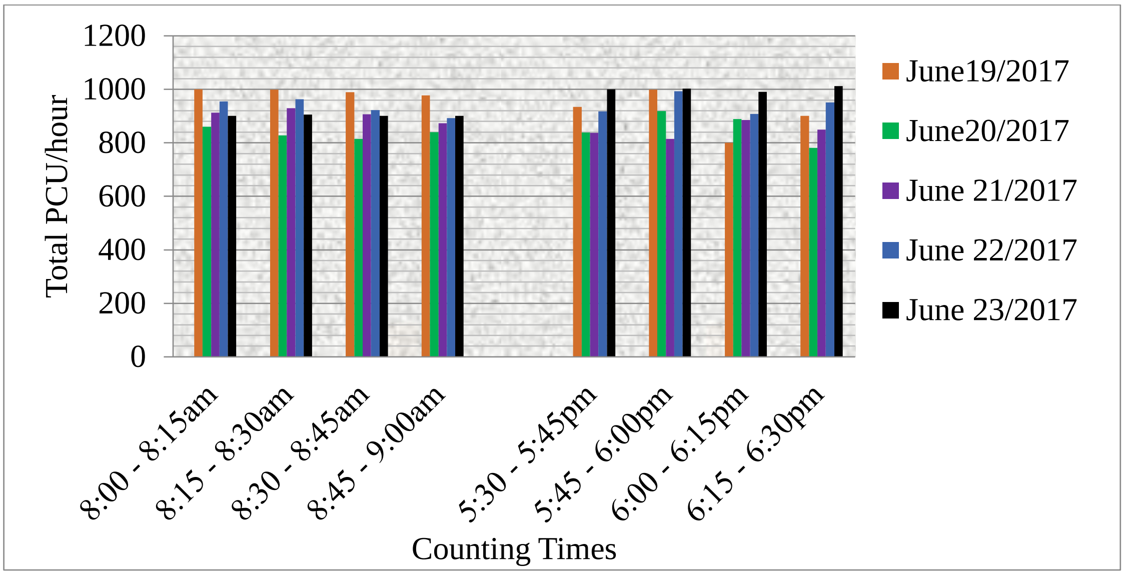


Figure 3.4: Comparison of total counted 15minutes PCU in counting times and day at Abune Petros Roundabout

As described in above figure 3.4, the observation was taken for five consecutive working days. The highest passenger car unit per 15minutes at Abune Petros Roundabout is observed on Monday, June 19/ 2017 at 08:00 am – 08:15 am and 8:15 am to 8:30 am in the morning. Because this time was rush in the hour or this was a time at which all workers go to their work from their stayed rest. The same as on Friday, June 23/2017 at 5:30 pm - 5:45 pm, 5:45 pm - 6:00 pm, 6:15 pm - 6:30 pm the traffic volume was very high and this time was rush out the hour. This time was all workers go from their work to their home. These factors can show the traffic flow variation at this roundabout.

For this reason, the analysis of this study was based on traffic data collected in three consecutive working days except for Monday and Friday.

To identify the capacity of existing roundabout and to give remedial measures, this research work was analyzed manual calculation based on HCM, 2010 and SIDRA software method. As it used as input for software, the counted average three days of traffic movements on each approach of the roundabouts with their movement directions are summarized as follow.

Date 20-22/07 /2017

Time 8:00 - 9:00am and 5:30 - 6:30pm

Table 3.5: Summarized three days average of traffic volume per hour on each approach with movements' direction

Roundabout Legs	Left		Through		Right		U-Turn		Total PCU//hr.
	Total Veh	HV	Total Veh	HV	Total Veh	HV	Total Veh	HV	
Merkato	331	36	352	38	336	16	14	4	1033
Minilik	275	12	339	31	277	13	5	2	896
Churcher	347	22	308	29	347	22	22	3	1023
Sebara Babur	262	21	257	12	275	22	5	5	799
Total									3750

The value described in table 3.5 above is the final factored average of three days traffic data used as input in the software. As it was observed in the table, there are four legs and four directions of movements at this roundabout. Among these, Merkato and Churchill approach had relatively higher traffic volume (1033pcu/hr., 1023pcu/hr.) while the lower traffic volume had observed at Sebara Babur approach (799pcu/hr.) respectively. The reasons why traffic volumes high are at these plans are as described above, due to the marketplace and bus station is in this direction. In detail, it is described in the following Figure 3.5 below clearly.

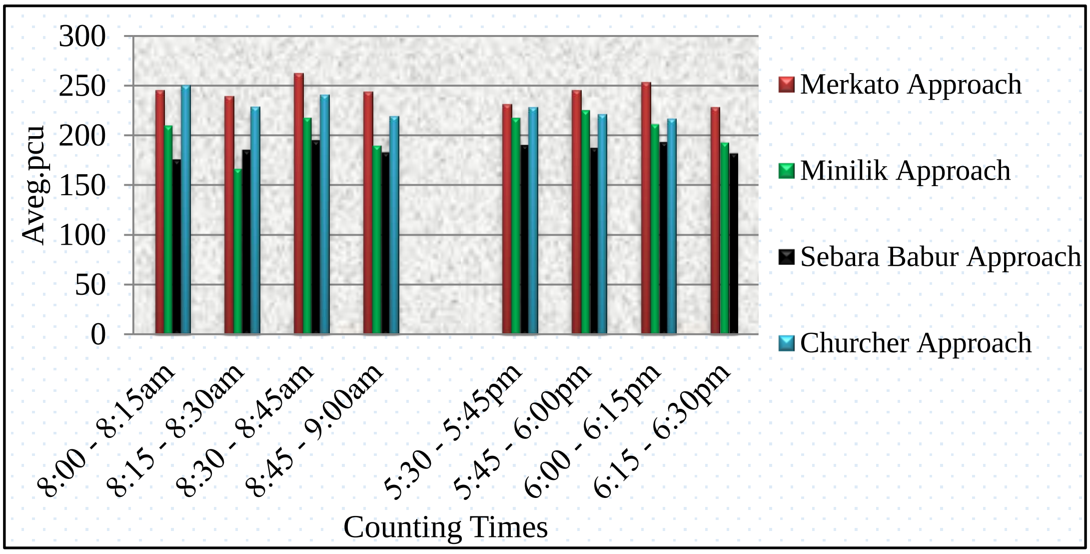


Figure 3.5: Comparison of average three days PCU/15minute on each approach

All approaches have different peak 15minutes volumes (V15min), peak hour volumes (PHV), peak hour factors (PHF), and design flow rate (FR). The peak 15minutes volume

(Vp15min) is the highest volume in minutes within the hourly amounts, and the peak hour volume (PHV) of each approach was obtained by the summing up the most significant four 15minute volume within the peak hours. The peak hour factor (PHF) is obtained by dividing the peak hourly capacity by four times peak 15minute mass within the peak hour volume. Actual (design flow rate) is obtained dividing the hourly amount by peak hour factors or by multiplying the peak 15-minute volumes by four. To describe this concept graphically and mathematically, as sample Merkato approach movement direction is selected and described as follows

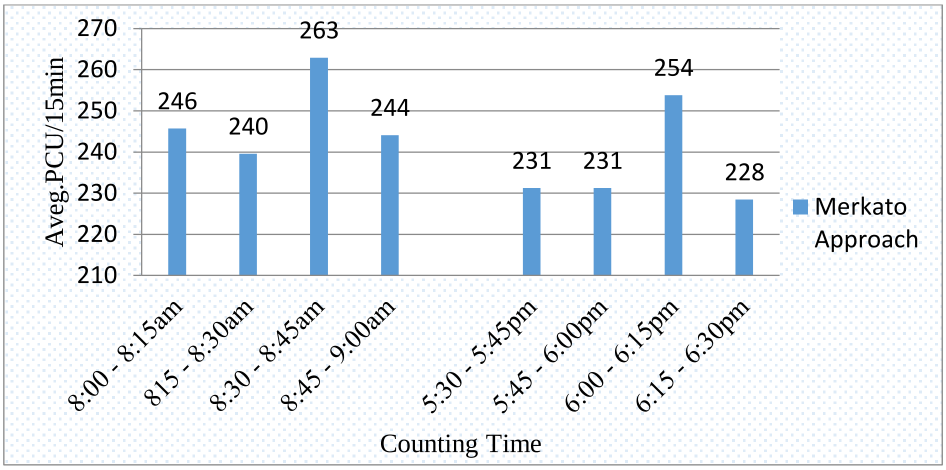


Figure 3.6: Exhibit peak 15-minute volumes (Vp15min) of three days of average passenger cars unit of Merkato

From the figure 3.6 above the highest peak 15minute volume (Vp15min) is 263 at the time of 08:3am – 08:45am , and the highest four peak of 15minute interval volume data are 263,254, 246 and 244 at the time of 08:30 -08:45am, 06:00pm – 6:15pm,08:00am – 08:15am, and 08:45am – 9:00am respectively.

Therefore sample calculation was conducted based on the above graph.

Peak Hour volume (PHV) = 263+254+246+244 = 1007pcu/hr.

Peak 15minute volume (Vp15min) = 263pcu/15min

Peak Hour Factor ($PHF) = \frac{V}{(4*Vp15minute\ )}$ = $\frac{1007}{(4*263\ )}$ =0.96

Flow Rate (FR) = $\frac{PHV}{PHF} = 4 * Vp15min = \frac{1007}{0.96}$ = $4 * 263$ = 1052pcu/hr.

Similar to the rest approaches, and summarized in table 3.9 below

Table 3.6: The summarized flow conditions for each method used as input in software

Roundabout Legs	PHV (Veh/hr.)	Total HV	%HV	f_{HV}	V15min (pcu)	PHF	Flow rate (PCU)
Merkato	1007	98	9.70	0.91	263	0.96	1052
Minilik	925	65	7.06	0.93	254	0.91	1015
Sebara Babur	816	55	6.78	0.94	208	0.98	834
Churchill	948	74	7.84	0.93	250	0.95	1002

3.4.4. Pedestrian Volume Data

Pedestrians are the type of traffic that can influence the capacity and performance of roundabout. To determine the capacity and performance of roundabout in Addis Ababa City, the pedestrian count method was the same as vehicles count method during the two peak hours. But, the process of data capturing is not by video, the count method is manually by a tally sheet. The purpose of the pedestrian count was used for capacity and performance analysis at a roundabout. Volume counts of the pedestrians were made at locations such as subway station, midblock, and crosswalk. The safe and efficient accommodation of pedestrian at an intersection is equally essential as the provisions prepared for vehicles. Pedestrian movements should be given and their locations controlled to maximize safety and minimize conflicts with other traffic flows. Often, pedestrians are a secondary consideration in the design of roadways, particularly at intersections in suburban areas [22].

As defined on the HWCM the pedestrian movement data can affect the movement of the vehicle at the intersection when the speeds of elder and (or young pedestrians' will be high or small below the threshold value of walking speed (1.2m/s) [10].

3.4.4.1 Pedestrian Volume from Abune Petros Roundabout

The counted pedestrian data are summarized in table 3.7 below.

Table 3.7: Average of three days peak 15minutes Pedestrians Volume data

Counting Times	Merkato Approach	Churchill Approach	Minilik Approach	Sebara Babur Approach
8:00 - 8:15am	78	89	75	86
8:15am -8:30am	85	84	71	81
8:30am - 8:45am	75	82	68	79
8:45am -9:00am	71	82	75	69
5:30 - 5:45pm	76	88	72	77
5:45pm -6:00pm	86	88	72	80
6:00 - 6:15pm	76	74	62	61
6:15 - 30pm	70	76	81	69
PHVped/hr.	326	349	302	325

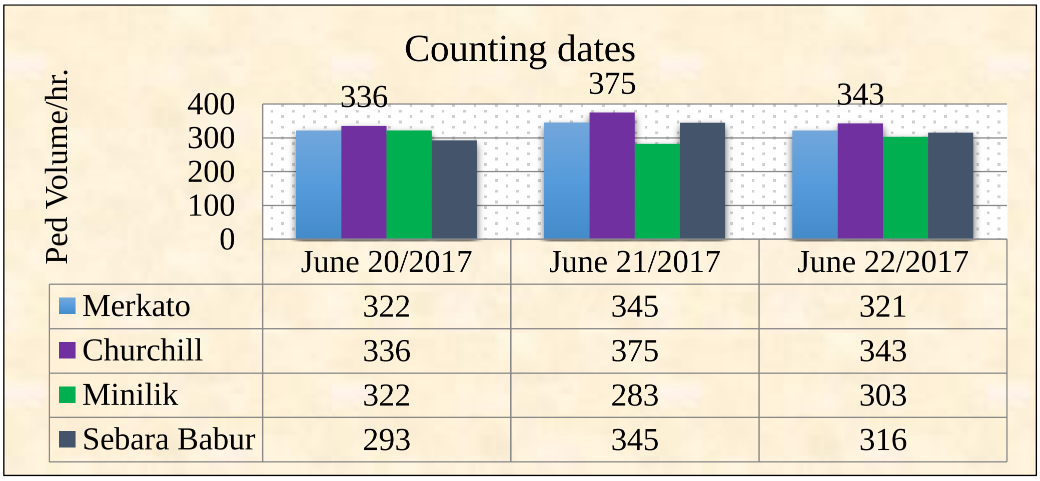


Figure 3.7: Exhibits difference ped volume per hour in collection date

From above Figure 3.7, the Pedestrians Volume for each approach varies from times to time. The Pedestrians Volume data was collected for three working days of two peak hours. From these three days, Wednesday (June 21/2017) in the afternoon is very high Ped volume at Churchill direction than the other direction. Because of this day and

Thursday was market day and, due to this it shows $_{\text{Ped}}$ very high and can affect the capacity of a study area.

The ped volume also varies with minutes hour intervals. In detail**,** the average pedestrian per 15minutes of each approach was summarized in the following figure**.**

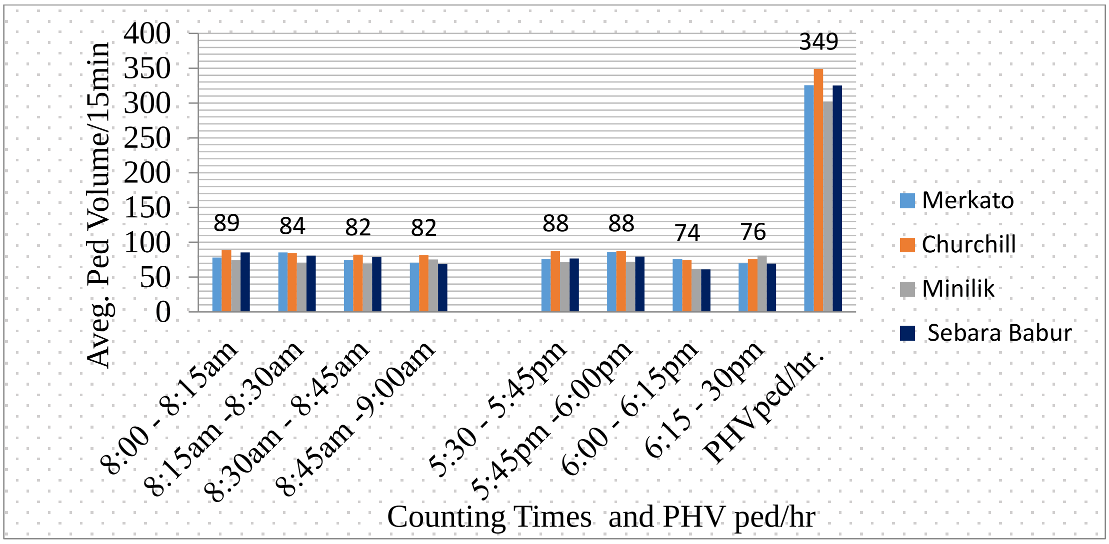


Figure 3.8: The average of 15minutes and peak hour Pedestrians Volume for each approach

From above Figure 3.8, the average 15minutes Pedestrians Volume for each procedure varies from times to time. The average 15minutes Pedestrians Volume data was collected for three working days of two peak hours (8:00- 9:00 am) at Morning and (5:30 - 6:30 pm) at evening.

The most significant values (88 ped/15minutes) was counted at Churchill direction in morning peak hour at time of 8:00 am - 8:15 am), and the hourly peak volume observed at the course of Churchill and Merkato in the morning and evening time at 8:00 am - 8:15 am, 5:30 pm - 3:45 pm,5:45pm-6: 00 pm, and 6:15 pm – 6:30pm.The dominance count of the overall volume is Churchill direction, and while least volume data observed at Sebara Babur direction. The reason for Churchill direction pedestrian volume data was very high, as the country; there is a significant marketplace which so-called Merkato at this direction. Due to this many pedestrians can flow to this course, and this also can affect the traffic flow conditions in this study area. As it is observed on the figure3.8 above, the peak hourly volume is the summation of the four peak V15minutes within the peak hour (PHV_{ped}=89+88+88+86=349ped/hr.) as it labeled at Churchill direction. The peak V15minutes is 89ped/15min. this was used for input in SIDRA software. Based on above-summarized pedestrian volume data, sample calculation and prepared Pedestrian data were listed in the table 3.11 below. For sample calculations, Churchill direction was taken as the example.

Sample calculation of Churchill approach

Peak hour volume (PHV_{ped}=89+88+88+75=349ped/hr.).

Peak V15minutes = 89ped/15min.

Peak Hour Factor ($PHF = \frac{PHV}{4*VP15minutes} = \frac{349}{4*89} = 0.98$

Flow rate $FR = \frac{PHV}{PHF}$ or = 4*Vp15minutes= 4*132= 528ped/hour

For each approach the calculation is similar and summarized in table 3.11 below.

Table 3.8: Summary of pedestrians flow value used as input for software

Leg Name	PHV(ped)	Vp15min	PHF	FR (ped. /hr.)
Merkato approach	326	86	0.943	345
Churchill approach	349	89	0.984	355
Minilik approach	302	81	0.937	323
Sebara Babur approach	325	86	0.949	343

3.4.5 Vehicles Data from German (Mekanisa) Roundabout

This roundabout has connected the road from Gofa, Sarbet, Hana Merriam and Jemo Michael to each other. This study area is a center for different government and non-governmental offices, separate industrial and factories area and also, traffic from different regions of settlement come to this roundabout. These factors can be effects on the demand volumes to this study area. Under this traffic count, the vehicles were classified as given in Table 3.5 above. The traffics were counted at two peak hours in the morning at 8:00 am to 9:00 am when the worker go to their workplace and in the afternoon at 5:30 pm to 6:30 pm when they back to their home from work.

The following figures 3.9 shows summarized factored vehicles traffic data collected from the study area and with the comparison of volumes in counting times and days.

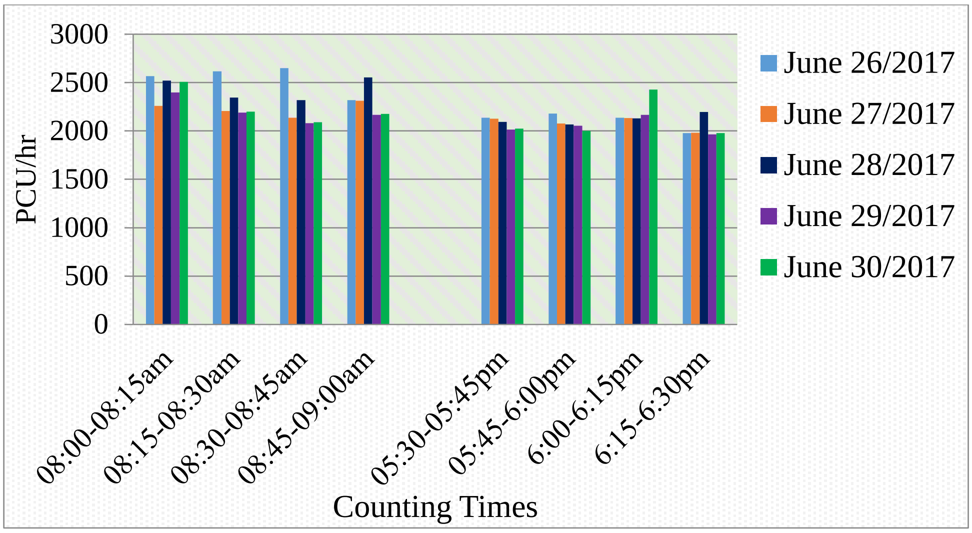


Figure 3.9: Comparison of total 15minutes PCU within the counting times and day at study area

As described in above figure 3.9, the observation was taken for five consecutive working days. The highest passenger car unit per 15minutes at German Roundabout is observed on Monday, June 26, 2017, at 08:15 am – 08:30 am and 8:30 pm-8: 45 pm in the morning; on Friday, June 30/2017 at 6:00pm - 6:15 pm. Hence due to the high volume variation in the study area, these two days have been omitted from the analysis and the three consecutive working days data has been used for analysis. To identify the capacity of existing roundabout and to give remedial measures, this research work was analyzed by SIDRA software using the method of the total Vehicles and heavy vehicles volume data method. As it used as input for software, the counted average three days of traffic movements on each approach of the roundabouts with their movement directions are summarized as follows.

Three days average Peak hourly Volume data with their direction of movement were summarized in table 3.12 below.

Date 27-92/07 /2017

Time 8:00 - 9:00am and 5:30 - 6:30pm

Table 3.9: Summarized three days average traffic volume per hour on each approach with movements' direction

Roundabout Legs	Left		Through		Right		U-Turn		Summation PCU/hr.
	Total Veh	HV	Total Veh	HV	Total Veh	HV	Total Veh	HV	
Hana Direction	1087	90	1229	99	722	58	121	17	3159
Gofa Direction	376	34	485	42	555	44	373	32	1789
Sarbet Direction	484	46	728	54	541	38	339	38	2092
Michael Direction	824	55	520	46	969	74	283	30	2596
Total									9635

The value described in table 3.9 above the final factored average of three-day traffic data used as input in the software. As it was observed in the table, there are four legs and four directions of movements at this roundabout. Among these, Hana and Michael's approach had relatively higher traffic volume (3159pcu/hr., 2596 pcu/hr.) while the lower traffic volume had observed at Gofa and Sarbet approach (1789pcu/hr., 2358pcu/hr.) respectively. Due to the residential and industrial areas at the Hana and Michael approaches, there was high traffic flows in this direction. In detail, it is described in the following figure 3.10 below.

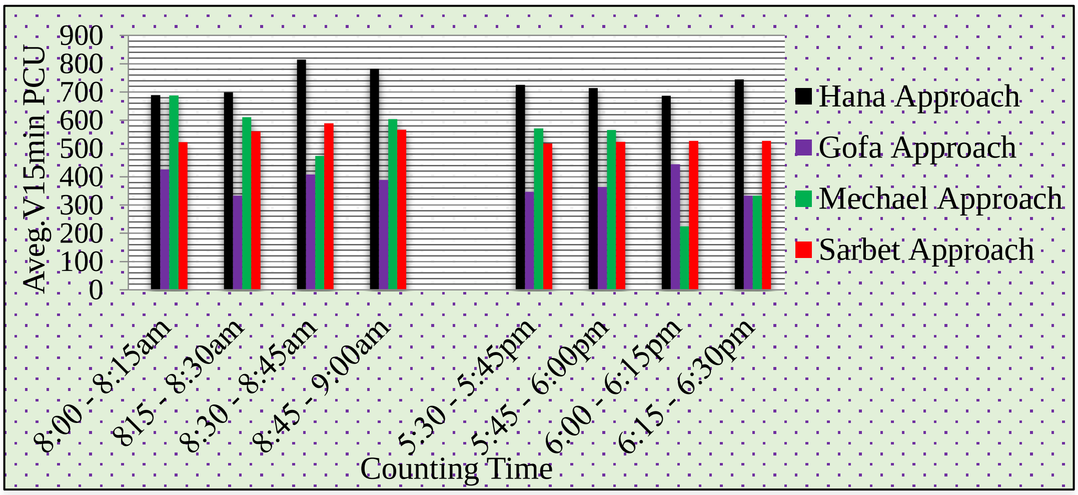


Figure 3.10: Average of traffic volume distribution at each approach within counting times

All methods have different peak 15minutes volumes (V15min), peak hour volumes (PHV), peak hour factors (PHF), and design flow rate (FR). The peak 15minutes volume (Vp15min) is the highest volume in minutes within the hourly amounts, and the peak hour volume (PHV) of each approach was obtained by the summing up the most significant four 15minute volume within the peak hours. The peak hour factor (PHF) is obtained by dividing the peak hourly capacity by four times peak 15minute mass within the peak hour volume. Actual (design flow rate) is obtained dividing the hourly amount by peak hour factors or by multiplying the peak 15-minute volumes by four. To describe this concept graphically and mathematically, as sample Hana approach movement direction is selected and described as follows

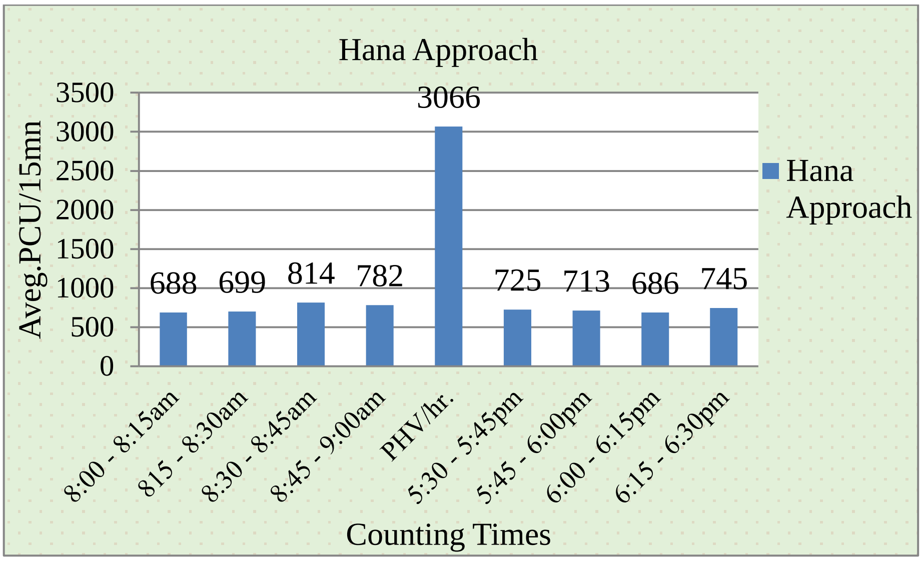


Figure 3.11 15 minutes three days average for passenger cars unit of Hana approach

From the above graph, the highest peak 15minute volume (Vp15min) is 814 at the time of 08:30 am – 08:45 am, and the highest four peak of 15minute interval volume data are 814,782, 725 and 745 at the time of 08:30 am – 08:45 am, 8:45 am – 9:00 am, 05:30 pm – 05:45 pm, and 06:15 pm – 6:30 pm respectively.

Therefore, sample calculation was conducted based on the above figure 3.11

Peak Hour volume (PHV) = 814+782+745+725 = 3066pcu/hr.

Peak 15minute volume (Vp15min) = 814pcu/15min

Peak Hour Factor (PHF) $= \frac{V}{(4*Vp15minute\,)} = \frac{3066}{(4*814\,)} =$ 0.94

Flow Rate (FR) $= \frac{PHV}{PHF} = 4 * Vp15min = \frac{3066}{0.94} = 4 * 814$ = 3257pcu/hr.

Similar to the rest approaches, and summarized in the table 3.14 below

Table 3.10: Summarized traffic Flow conditions for each approaches used as input in software

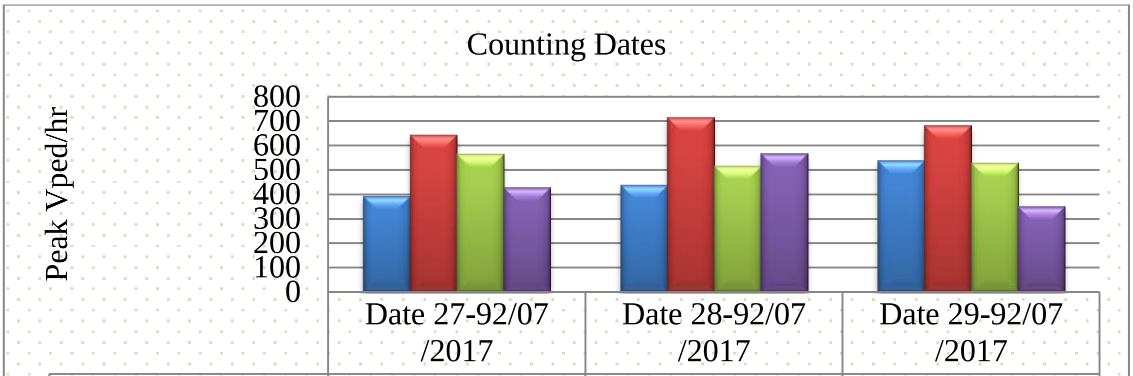
Roundabout Legs	PHV (Veh/hr.)	Total HV	%HV	f_{HV}	V15min (pcu)	PHF	Flow rate
Hana Approach	3066	256	8.34	0.92	814	0.94	3257
Gofa Approach	1703	144	8.47	0.92	444	0.96	1776
Michael Approach	2472	206	8.32	0.92	688	0.90	2752
Sarbet Approach	2276	214	9.40	0.91	589	0.97	2354

3.4.6 Pedestrian Volume from German (Mekanisa) Roundabout

The counted pedestrian data are summarized in Table 3.12 below.

Table 3.11: The average of three days peak 15minutes Pedestrians Volume data.

Counting Times	Hana Approach	Gofa Approach	Sarbet Approach	Michael Approach
02:00 - 02:15am	94	151	116	100
02:15 - 02:30 am	88	138	124	96
02:30 - 02:45 am	89	143	106	79
02:45 - 03:00 am	88	129	95	64
5:30- 5:45 pm	111	157	124	100
5:45- 6:00 pm	110	144	129	96
6:00- 6:15 pm	119	152	127	79
5:15- 5:30 pm	124	125	117	64
PHVped/hr.	464	604	504	393
Peak V15minutes	124	157	129	100



	Date 27-92/07 /2017	Date 28-92/07 /2017	Date 29-92/07 /2017
Hana Approach	394	439	539
Gofa Approach	644	715	683
Sarbet Approach	566	517	530
Michael Approach	429	568	351

Figure 3.12: Peak 15minutes Pedestrians Volume of each day

From above Figure 3.12, the Pedestrians Volume for each approach varies from times to time. The Pedestrians Volume data was collected for three working days of two peak hours. From these three days, Gofa approach is very high Ped volume than the other methods.

The ped volume also varies with minutes hour intervals. In detail**,** the average pedestrian per 15 minutes of each approach was summarized in the following figure**.**

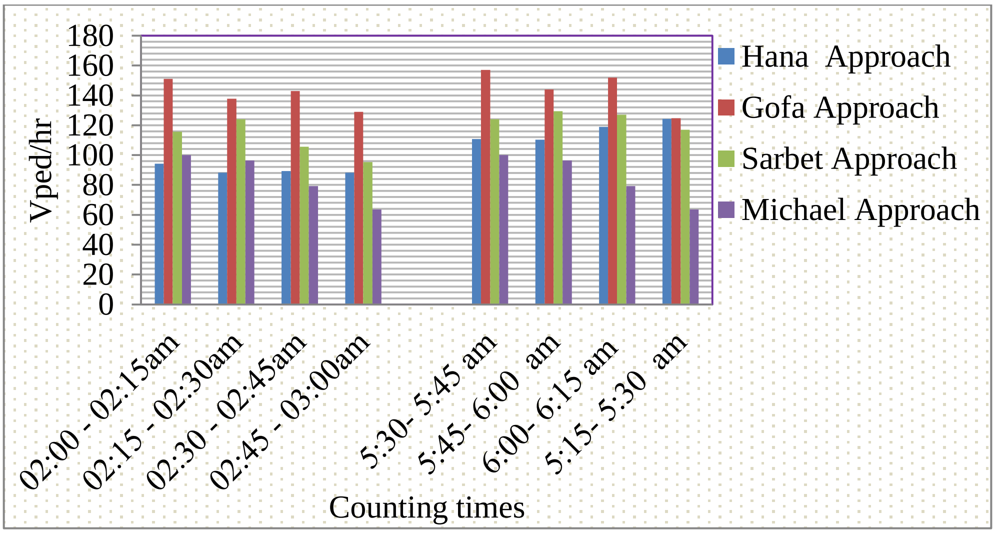


Figure 3.13: The average 15minutes Pedestrians Volume for each approach

From above Figure 3.13, the average 15minutes Pedestrians Volume for each path varies from times to time. The average 15minutes Pedestrians Volume data was collected for three working days of two peak hours (8:00- 9:00 am) at Morning and (5:30 - 6:30 pm) at evening except for Monday and Friday.

The largest values (157 ped/15minutes) was counted at Gofa direction in evening peak hour at a time of 5:30 pm - 5:45 pm), and the hourly peak volume for direction was observed in morning and evening time at 8:00 am - 8:15 am, 5:30 pm - 5:45 pm, 5:45 pm –6:00pm, 6:00 pm– 6:15 pm. The dominance count of the overall volume is Gofa direction, and while least volume data observed at Jemo Michael direction. The reason for Gofa direction pedestrian volume data was very high; workers are distributed to a different direction at peak hours when they go to their workplace and come back from work to their home. Due to this many pedestrians can flow to this direction, and this also can affect the traffic flow conditions in this study area. As it is observed in the graph above, the peak hourly volume is the summation of the four peak V15minutes within the peak hour. (PHV_{ped}=157+152+151+144=604ped/hr.). As it labeled at Gofa direction, the peak V15minutes is 157ped/15min. Which was used for input in SIDRA software? Based upon above-summarized pedestrian volume data, sample calculation and prepared Pedestrian data were listed in the table below. For sample calculations, Merkato direction was taken as the example.

Sample calculation of Gofa Approach

Peak hour volume (PHV_{ped}) =157+152+151+144=604ped/hr.

Peak V15minutes = 157ped/15min.

Peak Hour Factor ($PHF = \frac{PHV}{4*VP15minutes} = \frac{604}{4*157}$ = 0.962

Flow rate $FR = \frac{PHV}{PHF}$ or = 4*Vp15minutes= 4*157= 628ped/hour

For each approach the calculation is similar and summarized in table 3.13 below.

Table 3.12: Summary of pedestrians flow value used as input for software

Leg Name	PHV(ped)	Vp15min	PHF	FR (ped. /hr.)
Hana Approach	464	124	0.93	497
Gofa Approach	604	157	0.96	628
Sarbet Approach	504	129	0.97	517
Michael Approach	393	100	0.98	400

3.5 Other needed data used for analysis

3.5.1 Environment Factor

Environment Factor in SIDRA model allows calibration of the capacity model for less restricted (higher capacity) and more restricted (lower position) environments. The Environment Factor represents the general roundabout environment in terms of roundabout design type, visibility, significant grades, operating speeds, size of light and heavy vehicles, driver aggressiveness and alertness, pedestrians, heavy vehicle activity, parking maneuvers, etc. which affect the vehicle movements on approach and exit sides as well as at circulating road as relevant [10]. Environment Factors value ranges between 0.5 and 2 representing less restricted to more controlled conditions. The default value for Environment factor set in SIDRA is 1.20 but, in this research, the value environmental factors calibrated in the SIDRA intersection was 1.0

E. Grade factor

Grade affects the capacity and performance of roundabout by changing critical gaps (headways). The value for degree of each approach was calculated by vertical increase Over horizontal increase of elevation difference collected at the site by Global Positioning System (GPS) instrument. The data was collected on July 2017 at the time of 10:00 am Abune Petros site and 5:00 pm at the German site. The elevation difference and distance between two points were collected in all approach as the following table.

3.5.1 Coordinate Data Collected at Abune Petros Roundabout

Table 3.13: Summary of Coordinates Measured at Abune Petros Roundabout

Roundabout Approaches	Easting(X)	Northing(Y)	Elevation
Merkato direction	472475	998342	2462
Churchill Direction	472491	998369	2398
Minilik square direction	472520	998884	2460
Sebara Babur direction	472457	998834	2457

3.5.2 Coordinate Data Collected at German (Mekanisa) Roundabouts

Table 3.14: Summary of Coordinates measured at German (Mekanisa) Roundabout

Roundabout Approaches	Easting(X)	Northing(Y)	Elevation
Hana	470610.00	990814.00	2107
Gofa	470610.00	990540.00	2216
Sarbet	470620.00	990996.00	2241
Michael	470627.00	990999.00	2225

The grade factor in percent was obtained by formula = $\frac{\Delta y}{\Delta x} * 100$ … … … … … … … … … .3.1

The sample calculation of the grade is given as the following with the summary of all approach legs

Sample Calculated at Abune Petros Roundabout

Grade of Merkato direction = $\frac{\Delta y}{\Delta x} * 10 = \frac{472475-472520}{998342-998884} * 100$ = 8.30%

Similar calculation has conducted, for both roundabouts approaches and the values of the exits and approach grade are summarized in the following table

Table 3.15: Grade factor for each approach at Abune Petros Roundabout

Approach Name	Approach grade	Exit grade
Merkato	8.30	-8.30
Churchill	7.31	-7.31
Minilik	8.31	-8.31
Sebara	-7.31	7.31

Table 3.16: Grade factor for each approach at German (Mekanisa) Roundabout

Approach Name	Approach grade	Exit grade
Hana Mariam	5.49	-5.49
Gofa	9.12	-9.12
Sarbet	-5.49	5.49
Jemo Michael	8.21	8.21

3.6 Basic Saturation Flow rates of Vehicles and Pedestrian

According to Tarekegn Kumela described that in his research, there was no related research done for standard saturation flow rate in Ethiopia. Therefore, this research work used the following saturation flow rate question developed in South Africa in 2007, by Bester, C.J. and Meyers, W.L.

The equation developed in South Africa in 2007, by Bester, C., J., and W.L described that the saturation flow rate majorly depends on the following factors.

I. Speed limits –urban traffic intersection speed limits of 30km/hr. For gradient and 50km/hr. For the flat area was used.
II. Gradient intersection on different gradient was observed for traffic flow uphill and traffic flow downhill. Thus
III. The number of through lanes [32].

To estimate the saturation flow rate at study area is use the following equation

SFR= 990+288TL+8.5SL-26.8G

Where SFR = saturation flow rate, TL = number of through lane

SL = speed limit G = gradient in percent

Therefore, using the collected Data from the study area, the saturation flow rate and corresponding saturation headway are listed in table below

Table 3.17: Gradient, Saturation flow rate, Speed Limit and Corresponding Saturation headway at Abune Petros Roundabout

Approaches	Number of Through Lane	Gradient (%)	Speed Limit(Km/h)	Saturation flow(tcu./h)	Headway(sec) H=3600/SFR
Merkato	1	8.30	30	1311	2.7
Churchill	1	7.31	30	1337	2.69
Minilik	1	-8.30	30	1755	2.05
Sebara Babur	1	-7.31	30	1729	2.08

Table 3.18: Gradient, Saturation flow rate, Speed Limit and Corresponding Saturation headway at German (Mekanisa) Roundabout

Approaches	Number of Through Lane	Gradient (%)	Speed Limit(Km/h)	Saturation flow(tcu./h)	Headway(sec)
Hana	1	-5.49	50	1850	1.95
Gofa	1	3.70	50	1603	2.24
Sarbet	1	-5.49	50	1850	1.95
Michael	1	-3.70	50	1802	2

Annual population vehicles growth rate = 1.6 % (33).

Population growth rate of Addis Abba City = 3.8 % (32).

The following additional estimated data was collected from a site visit.

Table 3.19: Additional estimated data for input software

Additional estimated data	Name of Roundabouts	
	Abune Petros Site	German(Mekanisa)site
Queue space for light vehicles(LV)	7	6
Queue space for heavy vehicles	11	13
Vehicle length heavy vehicles	11	11
Vehicle length for light vehicles	5.1	5.1
Pedestrian queue space	1.2	1.2
Car occupancy	12	12

3.7 Extra Bunching

According to Tarekegn Kumela, et al. described that the Extra Bunching parameters are a general parameter applicable to an intersection.

The purpose of extra bunching parameter is to adjust the proportion of free vehicles in the traffic stream according to the proximity of upstream signalized junctions [32].

Table 3.20: Value for extra bunching is provided in the SIDRA INTERSECTION user's Manual as summarized in the following table.

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

[Source: SIDRA user Guide]

As Highway Capacity Manual defines of Critical Gap and Follow-up Headway:

Critical Gap is the minimum time (headway) between successive vehicles in the opposing (significant) traffic stream that is acceptable for entry by opposed (minor) stream vehicles.

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

Gap Acceptance: The process by which an opposed (minor) stream vehicles accepts an open gap in the opposing traffic flows for entering.

The gap acceptance and critical gap used in this research were 2sec. And 5sec. respectively and Follow up Headway was varied as shown in above table 3.17 and 3.18.

3.8 Methods of Analysis using Highway Capacity Manual

In this research, the analysis was made by both Mathematical model based upon HWCM, 2010 and SIDRA INTERSECTION software model

The following diagram shows the outlines of analysis process to be used for analyzing roundabout based upon HWCM ,2010 [6].

step1: Convert movement demand volumes (V, veh/h) to flow rates (v, veh/h).

Step 2: Adjust flow rates for heavy vehicles

Step 3**:** Determine circulating and exiting flow rates

Step 4**:** Determine entry flow rates by lane.

Step 5: Determine the capacity of each entry lane in passenger car equivalents.

Step 6: Determine pedestrian impedance (*f*ped) to vehicles

Step 7: Convert lane flow rates and capacity into vehicles per hour

Step 8**:** Compute the volume-to-capacity ratio for each lane.

Step 9**:** Compute the average control delay and corresponding LOS for each lane.

Step 10**:** Determine Level of Services (LOS) for each lane on each approach and average control delay as whole at intersection

Step 11: Compute 95th-percentile queues for each lane

Figure 3.14: The methodology of analysis roundabout capacity using HCM model

CHAPTER FOUR

ANALYSIS, RESULTS AND DISCUSSIONS

4.1 The quantified traffic volume and other identified significant factors tend to affect performance level of the roundabout

The significant factors affecting capacity and performance level of the roundabout were traffic volumes data, geometric data, environmental factors, grade factors and other input data. In detail, it was described in chapter three.

4.2 Analyzing capacity and performance of existing roundabout using both manual calculation and SIDRA INTERSECTION Software

4.2.1 Analysis based on manual calculation using HCM, 2010

The size and performance of the intersections were analyzed using manual calculation based upon highway Capacity Manual, while on the other hand, the SIDRA Intersection software also used for a separate analysis**.** It was seen that the formula in HCM 2010 for multilane roundabout has possible variation which leads to computational complexity for more than single-lane roundabouts. As described in HCM 2010 for roundabouts with up to two circulating lanes, which is the only type of multilane roundabout, addressed the analytical methodology. This means the entry and exit can be either one or two lanes wide (plus a possible right turn bypass lane). In this study, the selected roundabouts, like Abune Petros has two entrances with two current paths which can be analyzed both using manual and software as the multilane roundabout method. But, German roundabout has three entry lanes with three running lanes in Hana and Michael approaches. This intersection was very complicated wherein SIDRA Software was used, instead of manual calculation to analyze the intersection.

4.2.1.1 Capacity Concepts

HCM define that, the capacity of roundabout approaches directly influenced by flow patterns. These are three flows of interest, the entering flow, the current flow and the existing flow. The capacity of an approach decreases as the conflicting flow increases. The primary contradictory flow is the current flow that passes directly in front of the subject entry. While the fluid flow directly conflicts with the entry flow, the exiting flow also affects a driver's decision to enter the roundabout [6].

The results indicated the peak 15-min period and peak hour volume of both vehicles and pedestrian data measured in the field. Table 4.1 below shows the volume of a 15-min period and peak hour for both car and pedestrian.

Table 4.1: The average 3- days' peak 15-min vehicles volume and heavy vehicle data for each approach.

Time	Merkato Approach		Minilik Approach		Sebara Babur Approach		Churcher Approach	
	PHV(Veh/h)	HV	PHV	HV	PHV	HV	PHV	HV
8:00 - 8:15am	206	29	195	12	157	13	214	24
815 - 8:30 am	204	26	149	13	166	13	205	15
8:30 - 8:45 am	222	29	194	16	175	15	219	16
8:45 - 9:00 am	220	16	176	12	167	9	203	13
5:30 - 5:45 pm	198	23	198	15	167	13	187	17
5:45 - 6:00 pm	206	28	200	19	167	14	202	19
6:00 - 6:15 pm	221	24	189	16	173	14	195	18
6:15 - 6:30 pm	193	25	176	15	162	15	196	17

Table 4.2: Three days Average of pedestrians' volume per 15-min and hour of each approach

Approaches	Merkato	Churchill	Minilik	Sebara Babur
8:00 - 8:15am	78	89	75	86
815 - 8:30 am	85	84	71	81
8:30 - 8:45 am	75	82	68	79
8:45 - 9:00 am	71	82	75	69
5:30 - 5:45 pm	76	88	72	77
5:45 - 6:00 pm	86	88	72	80
6:00 - 6:15 pm	76	74	62	61
6:15 - 6:30 pm	70	76	81	69
PHVped/hr.	326	349	302	325

Based on the above tables 4.1 and 4.2 data, the analysis using manual calculation based on the HCM 2010 equations are presented below the following steps:

Sep 1: Convert Movement demand volume into a flow rates

In this case, as it described on the above methodology, each turning- movement demand volume is converted to a demand flow rate by dividing a peak hour to Peak hour factor (PHF) [6].

$$\mathrm{PHF} = \frac{\mathrm{V}}{(4*\mathrm{Vp15minute})} \ldots\ldots\ldots 4.1$$

Where

PHF = Peak hour factor

V = Hourly Volume (Veh/hr.)

V_p15min = Peak volume during the 15minute within the hour (Veh/15min)

The described sample calculation in this analysis is taken from Merkato approach, and the other approaches were the similar calculation and summarized in the table. From table 4.1

Peak Hour volume for Merkato approach (PHV) = 206+222+220+221 = 869veh/hr.

Peak 15minute volume (Vp15min) = 222veh/15min

Peak Hour Factor (PHF) $= \frac{V}{(4*Vp15minute)} = \frac{869}{(4*222)} = 0.978$

$$\mathrm{v_i} = \frac{\mathrm{Vi}}{\mathrm{PHF}} \ldots\ldots\ldots 4.2$$

Where

v_i = demand flow rate for movement i (Veh/h)

V_i =Demand volume for movement i or hourly volume (Veh/h)

PHF =Peak hour factor

Flow Rate (FR) $= \frac{PHV}{PHF} = 4 * Vp15min = \frac{869}{0.978} = 4 * 222 = 888$veh/hr.

Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement adjusted to account for stream characteristics by factors given in table 4.3 below

Table 4.3: Passenger Car Equivalent Factor adopted using Highway Capacity Manual (6).

Vehicle type	Conversion factor
Trucks	2
P.car	1

$$f_{THV} = \frac{1}{1+P_T\,(E_T-1)} \quad \ldots\ldots 4.4$$

$$v_{i,pce} = \frac{v_i}{f_{HV}} \quad \ldots\ldots 4.3$$

$$P_T = \frac{\text{Vheavy veh}}{\text{PHV}} \quad \ldots\ldots 4.4$$

Where

$v_{i,pce}$ = demand flow rate for movement i (pc/h)

v_i = demand flow rate for movement i (Veh/h)

f_{HV} = heavy-vehicle adjustment factor

P_T = proportion of demand volume that consists of heavy vehicles

E_T = passenger car equivalent for heavy vehicles.

The example of sample calculation of adjustment factor for heavy vehicles is taken from Merkato approach

(P_T) Proportion of demand volume for Merkato approach is calculated as follows

From the table above the peak hour volume is 869veh/hr, and the massive vehicle within the peak hour in Merkato direction is 16+29+24+29=98veh/hr.

$$P_T = \frac{98}{869} * 100 = 11.28\%$$

From table 4.3 above the passenger car equivalent for heavy vehicle is =2

$$f_{THV\text{ markato approach}} = \frac{1}{1+(11.28/100)(2-1)} = 0.898$$

Similar for other approaches, in detail it summarized in table 4.4 below

Adjust the demand flow rate for heavy vehicle

$$v_{i,pce} = \frac{v_i}{f_{HV}} = \frac{888}{0.90} = 986.66 = 986\text{pc/hr.}$$

The similar calculation has been conducted for other approaches.

The converted demand volume to flow rate and adjusted for the heavy vehicle is summarized in the table below.

Table 4.4: The Converted demand value to flow rate, and adjusted for the heavy vehicle.

Roundabout Legs	PHV (Veh/hr.)	Total HV (Veh/hr.)	%HV	f_{HV}	V15min (Veh/hr.)	PHF	Flow rate	Adj. FR hv (pc/hr.)
Merkato	869	98	11.23	0.90	222	0.98	887	986
Minilik	788	51	6.43	0.94	200	0.98	800	851
Sebara Babur	681	57	8.32	0.92	175	0.98	699	757
Churchill	841	68	8.12	0.92	219	0.96	877	949

As described in Table 4.4 above the converted demand volume to flow rate and adjusted for the heavy vehicle is determined for legs of roundabout approach. But, to identify Circulating and exiting flow rate, the fixed flow rate for heavy vehicle for each direction of movement is summarized in Table 4.5 below.

Table 4.5: Summarized average of adjusted flow rate for heavy vehicle for each direction of movement

Roundabout Legs	Movement direction	PHV (Veh/hr.)	PHF	Flow rate (Veh/hr.)	f_{HV}	Adj.HV (Pc/hr.)
Merkato(EB)	U	22	0.980	22	0.90	25
	L	286	0.980	292	0.90	324
	T	296	0.980	302	0.90	335
	R	292	0.980	298	0.90	332
Minilik(WB)	U	2	0.985	2	0.94	2
	L	249	0.985	253	0.94	269
	T	285	0.985	289	0.94	308
	R	259	0.985	263	0.94	280
Sebara Babur(SB)	U	4	0.975	4	0.92	4
	L	224	0.975	230	0.92	249
	T	228	0.975	234	0.92	254
	R	221	0.975	227	0.92	245
Churchill(NB)	U	15	0.959	16	0.92	17
	L	301	0.959	314	0.92	339
	T	281	0.959	293	0.92	317
	R	247	0.959	257	0.92	278

Step 3: Determine Circulating and Exiting Flow Rates

This roundabout has four legs. Circulating, entering and exiting flow rates were calculated for each roundabout leg using the following method.

a. Circulating Flow Rate

As Highway Capacity Manual determine that, the current flow opposing a given entry is defined as the flow conflicting with the entry flow (i.e., the flow passing in front of the splitter island next to the subject entry). The circulating flow rate was calculated the following method. For sample calculation, eastbound or Merkato approach was selected

$$v_{c,EB,pc}(mrto) = v_{c,NBU,pc} + v_{c,WBL,pc} + v_{c,WBU,pc} + v_{c,SBT,pc} + v_{c,SBL,pc} + v_{c,SBU,pc} \dots\dots 4.5$$

Where

$v_{c,EB,pc}$ = circulating flow eastbound adjusted for heavy vehicle

$v_{c,NBU,pc}$ = circulating flow northbound u-turn adjusted for heavy vehicle

$v_{c,WBL,pc}$ = circulating flow westbound left- turn adjusted for heavy vehicle

$v_{c,WBU,pc}$ = circulating westbound flow u-tern adjusted for heavy vehicle

$v_{c,SBT,pc}$ = circulating flow southbound through adjusted for heavy vehicle

$v_{c,SBL,pc}$ = circulating flow southbound left turn adjusted for heavy vehicle

$v_{c,SBU,pc}$ = circulating flow southbound u-turn adjusted for heavy vehicle

From above table for the west leg (eastbound entry), the circulating flow is calculated as follows

$$v_{c,EB,pc}(merkato) = 17 + 269 + 2 + 254 + 249 + 4 = \mathbf{795} pc/hr.$$

a. Exiting flow rate

$$v_{ex,WB,pc}(mrto) = v_{c,EBU,pc} + v_{c,NBL,pc} + v_{c,WBUT,pc} + v_{c,SBR,pc} - v_{EB,pc\text{ by pass lane}} \dots\dots 4.6$$

Where

$v_{ex,WB,pc}$ = exiting flow west bound adjusted for heavy vehicle

$v_{EBU,pc}$ = flow rate north bound u turn adjusted for heavy vehicle

$v_{NBL,pc}$ = flow rate west bound left- turn adjusted for heavy vehicle

$v_{WBU,pc}$ = flow rate west bound u-tern adjusted for heavy vehicle

$v_{SBR,pc}$ = flow south bound through adjusted for heavy vehicle

$v_{EBU,pc\text{ by pass lane}}$ = circulating flow west bound u-tern adjusted for heavy vehicle

In this study, the roundabout has no bypass lane and the selected sample calculation is worked as under here.

$$v_{ex,EB,pc}(merkato) = 25 + 308 + 245 + 339 - 0 = 918 pc/hr.$$

For other bounds, a similar calculation has conducted. In detail, it is summarized in Table 4.6 below

Step 4: Determine Entry Flow Rates by Lane

Highway capacity manual defines that for single-lane entries, the entry flow rate is the sum of all movement flow rates using that entry.

For multilane entries or entries with bypass lanes, or both, the following procedure may be used to assign flows to each lane: if a right-turn bypass lane is provided, the flow using the bypass lane is removed from the calculation of the roundabout entry flows. 2. If only one lane is available for a given movement, the current for that change is assigned only to that lane. 3. The remaining flows are assumed to be distributed across all lanes, subject to the constraints imposed by any designated or de facto lane assignments and any observed or estimated lane utilization imbalances [6].

The entry flow rate is calculated by summing up the movement flow rates that inter to the roundabout based on the numbers of entry lanes and opposing lane as shown in 4.6 Table Designated Lane Assignment

Table 4.6: Designated Lane Assignment

Designated Lane Assignment	Assumed Lane Assignment
LT, TR	If Vu+VL >VT+VR: L,R (defacto left-lane) If VRE >Vu+VL+VT :LT,R (defacto right turn-lane) Else LT, TR
L, LTR	If VT+VRe>Vu+VL :L, TR(defacto through –right lane Else L, LTR
LTR<R	If Vu+VL+VT >VRe :LT,R(defacto left-through lane Else LTR, R

Source [6].

Where

Notes: V_u, V_L, V_T, and V_{RE} are the U-turn, left-turn, through and none bypass right-turn flow rates using a given entry, respectively

L= left, LT = left-through, TR = through-right, LTR = left-through- right, and R= right.

On the basis of the assumed lane assignment for the entry and the lane utilization effect described above, flow rates can be assigned to each lane by using the formulas given as

%RL is the percentage of entry traffic using the right lane, %LL is the percentage of entry traffic using the left lane, and %LL + %RL = 1.

Based on the above concept the lane assignment to each lane is calculated as under here. Sample calculation was taken from Merkato approach**.**

From table 4.5 above merkato direction has Vu =25pc/hr. , VL = 324pc/hr., VT = 335 and VRe = 332pc/hr. Then when assigned to each lane and lane utilization effect used

Case 1: Vu+VL >VT+VR: L, R: 25pc/hr. +324pc/hr. =349pc/hr. is not greater than 335pc/hr.+332pc/hr.=667pc/hr.

Case2: VRe >Vu+VL+VT: LT, R: 332pc/hr.is not greater than 335pc/hr.+324pc/hr.+25pc/hr.=684pc/hr. Else Volume is distributed to LT, TR. Lane assignment on the bases of %LL+%RL =1

The total volume enter to Merkato approach is 25+324+335+332 = 1016

VL+VRe = 656pc/hr.

%VLL = $\frac{324\text{pc.}}{656\text{pc}} * 100.$ = 49.39%

%VLL+%VRL =1, 1-49% = 51%

Therefore from total volume 49.39%*1016pc/hr. = 501.8024= 502pc/hr. is assigned to LT lane, and 51.61%*1016pc/hr. = 514pc/hr. is assigned to TR lane.

For other approaches lanes, it is similar and generalized in a table and below.

Table 4.7: Summarized circulating, exiting and entry flow rate by lane

Roundabout leg bound	Circulating $exiting flow rate		entry flow rate							
	Vc_{pce}	Exiting Flow Rate	Total entry	(LL +RL)	L entry	R, entry	%LL	%RL	LT	TR
Merkato (v_{cEBpce})	795	918	1016	656	324	332	49	51	503	514
Minilik (v_{cWBpce})	1027	865	859	549	269	280	49	51	421	438
Sebara Babur (v_{cSBpce})	960	926	752	495	249	245	50	50	379	373
Churchill (v_{cNBpce})	940	871	952	617	339	278	55	45	523	428

From above Table 4.7, the current flow greater than exiting flow. This can influence the capacity of a roundabout.

Step 5: Determining Capacity of each entry Lane in the passenger car equivalents

Capacity calculations for each approach are calculated based upon the number of entry lane and number of circulating lane.

As described in literature review capacity of a single entry lane conflicted by one circulating lane and two-lane entries conflicted by one circulating lane estimated by the following formula. (e.g., a Single-lane roundabout is based on the conflicting flow). The equation for estimating the capacity entry lane is $Ce,pce = 1{,}130e^{(-1.0*10)^{-3}\text{vc,place}}$

i. Capacity for one-lane entries conflicted by two circulating lanes

The capacity of a one-lane roundabout entry conflicted by two circulating lanes given as follows

$$Ce,pce = 1{,}130e^{(-0.7*10)^{-3}\text{vc,pce}} \quad \text{4.7}$$

ii. Capacity for two-lane entries conflicted by two circulating lanes

Equation bellows give the Capacity of the right and left lanes, respectively, of a two-lane round of entry conflicted by two circulating lanes.

$$Ce,R,pce = 1{,}130e^{(-0.7*10)^{-3}\text{vc,pce}} \quad \text{4.8}$$

$$Ce,L,pce = 1{,}130e^{(-0.75*10)^{-3}\text{vc,pce}} \quad \text{4.9}$$

Where

Ce, pc = Lane capacity, adjusted for heavy vehicles (pc/h)

Vc, pce = conflicting flow rate (pc/h)

Ce, R,pce = Capacity of the right entry lane, adjusted for heavy vehicles(pc/h)

Coe, R,pce = Capacity of the left entry lane, adjusted for heavy vehicles(pc/h)

In this roundabout due to it has two entry lane and two circulating lanes in all approaches, the method of Two-Lane Entries Conflicted by Two Circulating Lanes capacity estimation is selected

Using the above estimated circulating flow rate, the capacity of each approach is estimated as follows. Sample calculation Merkato (Eastbound) approach was illustrated hereunder.

Circulating Flow Rates (Vcpce) in Merkato approach is = 795pc/hr.

Therefore the capacity of right and left lane is as follows

$$Ce,R,pce\ merkato = 1{,}130e^{(-0.7*10)^{-3}*795)=648\text{pc/hr}}.$$

$$Ce,L,pce\ merkato = 1{,}130e^{(-0.75*10)^{-3}*795=\frac{623\text{pc}}{\text{hr}})}$$

For another approach similar calculation is conducted and summarized in table 4.9 below

Sep 6: Determine Pedestrian impedance to conflicting Vehicles

Pedestrian crossing at a marked crosswalk that gives priority to entering motor vehicles can have a significant effect on the entry capacity. As HCM, 1999draft define that the pedestrian impedance decreases as the different vehicles flow increases [16].

This roundabout has a different pedestrian flow rate (Vn_{ped}/hr at all approaches. Therefore, the pedestrian impedance factor is calculated follows.

Table 4.8: Two lane entry capacity adjustment factor for pedestrian [16].

Case	Two- Lane Entry Capacity Adjustment Factor for Pedestrians
If	$n_{ped} < 100$ $f_{ped} = \min\left[1 - \frac{n_{ped}}{100}\left(1 - \frac{260.6 - 0.329 v_{c,pce} - 0.381 * 100}{1{,}380 - 0.5 v_{c,pce}}\right), 1\right]$ 4.10
Else	$f_{ped} = \min\left[\frac{1{,}260.6 - 0.329 v_{c,pce} - 0.381 n_{ped}}{1{,}380 - 0.5 v_{c,pce}}, 1\right]$ 4.11

Where

f_{ped} = entry capacity adjustment factor for pedestrian

n_{ped} = number of conflicting pedestrians per hour(P/h)

$v_{c,pce}$ = conflicting vehicular flow rate in the circulatory roadway

Sample calculation taken from Merkato direction in Table mmm above

$n_{ped} = 326$p/h and $v_{c,pce} = \frac{795pc}{hr}$.

Since $n_{ped} = 326$p/h >100,

Use $f_{ped} = [\frac{1{,}260.6 - 0.329 v_{c,pce} - 0.381 n_{ped}}{1{,}380 - 0.5 v_{c,pce}}, 1]$

$$f_{ped} = \min\left[\frac{1{,}260.6 - 0.329 * \frac{795pc}{hr}. - 0.381 * \frac{326p}{h}}{1{,}380 - 0.5 * 795}, 1\right]$$

$$= 0.89$$

For other approaches similar calculation and summarized in Table 4.9 below

Step 7: Convert lane flow rate and capacities into Vehicle per hour.

The capacity for a given lane is converted back to vehicle per hour by using pedestrian impedance and large vehicle factors. The following formula is used to estimate this condition.

$$\mathrm{CEB(merkato), R} = \mathrm{Cpce, EB, R} * f_{HV} * f_{ped} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .4.12$$

Where

C_i = capacity for lane i(veh/h)

C_{pce} = capacity for lane i(pc/h)

f_{HV} = heavy vehicle adjustment factor for the lane

f_{ped} = pedestiram impedance factor

For Merkato approach $f_{ped} = 0.89$

$f_{HV} = 0.90$

$\mathrm{Cpce, EB, R} = 648\mathrm{pc/h}$

$\mathrm{CEB(merkato), R} = 648 * 0.90 * 0.89 = 519\mathrm{veh/hr.}$

The similar calculation has conducted for all right and left of all approaches and summarized in table 4.9 below.

Also, the flow rate for a given lane is converted back to vehicles per hour by the formula 4.13 below

$$\mathrm{vi} = v_i * f_{HV} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4.13$$

Where

vi = flow rate for lane i(veh/hr.)

v_{iPCE} = flow rate for lanei(pc/hr.)

f_{HV} = heavy vehicle adjustment factor for the lane

For Merkato approach $f_{HV} = 0.90$

$v_{iPCE} = 514(\mathrm{pc/hr.})$

$V_{EB}\mathrm{merkato}, \mathrm{R} = 514 * 0.90 = 462\mathrm{veh/hr.}$

Similar calculation has conducted for all right and left of all approaches and summarized in table 4.9 below

Step 8: Compute the Degree of saturation for each lane

As it is described in the literature review degree of saturation is the ratio of demand at the roundabout entry to the capacity of entry. It suggested that the degree of saturation for admission for an entry lane should be less than 0.85 for satisfactory operation. When the degree of saturation exceeds this range, the process of the roundabout will likely deteriorate rapidly, particularly over the short period. Queues may form and delay to increase exponentially [27].

$$x_i = \frac{v_i}{C_i} \quad \text{..................} 4.14$$

Where

$x_i =$ volume to capacity ratio for subject lane

$C_i =$ capacity of the subject lane I $(\frac{\text{veh}}{\text{hr}}.)$

v_i = demand volume flow rate of the subject lane I (veh/hr.)

Sample calculation for market approach

$$x_{EB,R} = \frac{462}{519} = 0.89$$

Similar calculation has conducted for all right and left of all approaches and summarized in table 4.9 below

Step 9: Compute an average of control delay for each lane.

Highway Capacity Manual only includes control delay, the delay attributable to the control device. Control delay is the time that a driver spends queuing and then waiting for an acceptable gap in the circulating flow while at the front of the queue

$$d = \frac{3,600}{c} + 900T\left[x - 1 + \frac{\sqrt{(x-1)^2 + (\frac{3,600}{c})x}}{450T}\right] + 5 * min[x, 1] \text{ ... ... } 4.15$$

Where

d = average control delay(s/Veh)

x = volume to capacity ratio of the subject lane

c = capacity of a subject lane (Veh/h)

T = time period (h)(T = 0.25h for a 15-minute analysis

Control delay for Merkato or eastbound approach

From above-computed data

T =15min, or 0.25

$c_{EB,R} = 519\text{veh/h}$

$x_{EB,R0.89} = 0.89$

$$d = \frac{3,600}{519} + 900 * 0.25\left[0.89 - 1 + \frac{\sqrt{\left(0.25 - 1\right)^2 + \left(\frac{3,600}{519}\right) * 0.89}}{450 * 0.25}\right] + 5$$

$$* \, min[0.89,1]$$

d = 39.56sec

For other approaches, the similar calculation has conducted, and in detail, it summarized in table 4.9 below.

Step 10: Determine Level of Services (LOS) for each lane on each approach and average control delay as a whole at the intersection.

The LOS for each approach lane on approach is computed as follows

$$d_{approach} = \frac{d_{LL}v_{LL} + d_{LR}v_{LR}}{v_{LL}+v_{LR}} \quad \ldots\ldots 4.16$$

Where

$d_{approach}$ = control delay for approaches(s/veh)

$d_{LL,\$RL}$ = control delay for Left lane and Right lane (s/veh)

The v_i = flow rate for left and Right lane vehicle I (veh/hr.

$$d_{intersection} = \frac{\sum d_i v_i}{\sum v_i} \quad \ldots\ldots 4.17$$

Where

$d_{intersection}$ = control delay for entire intersection (s/veh)

d_i = control delay for approach (s/veh)

v_i = flow rate for approach vehicle i (veh/hr.

Table 4.9: The general summary of analyzed capacity at Abune Petros Roundabout

Roundabout leg bound	Capacity of entry lane		n_{ped}(p/h)	f_{ped}	f_{HV}	C(ve/h)		v(entry)	
	Cpce, R	Cpce, L				L	R	L	R
Merkato (v_{cEBpce})	648	623	326	0.89	0.90	498	519	452	462
Minilik (v_{cWBpce})	551	523	302	0.932	0.94	458	482	395	412
Sebara Babur (v_{cSBpce})	577	550	325	0.912	0.93	465	488	347	418
Cherchil (v_{cNBpce})	585	558	349	0.899	0.92	464	487	484	396

As defined in Highway Capacity Manual, the level of service (LOS) criteria for automobiles in roundabouts is given in the following table 4.11

For assessment LOS at the approach and intersection levels, LOS is based solely on control delay. The thresholds are based on the considered judgment of the Transportation Research Board Committee on [6].

Table 4.10: Level of services thresholds

Control Delay $\frac{s}{veh}$	LOS by Volume –to-Capacity Ratio $\frac{v}{c} \leq 1.0$	$\frac{v}{c} > 1.0$
0 - 10	A	F
>10 - 15	B	F
>15 – 25	C	F
>25 – 35	D	F
>35 – 50	E	F
>50	F	F

Table 4.11: The Summarized performance measures of Abune Petros roundabout.

Roundabout leg bound	Direction of lane	D/saturation (x= v/c)	control delay	LOS each Lane	Approach control delay	Approach LOS	d inter section	d LOS
Merkato (v_{cEBpce})	L Lane	0.9065	50.32	F	44.089	E	41.66	E
	R Lane	0.891	39.557	E				
Minilik (v_{cWBpce})	L Lane	0.863	42.14	E	37.342	E		
	R Lane	0.853	32.359	E				
Sebara Babur (v_{cSBpce})	L Lane	0.7554	16.55	C	27.945	D		
	R Lane	0.709	33.052	C				
Cherchil (v_{cNBpce})	L Lane	1.0417	81.88	F	48.437	E		
	R Lane	0.814	21.848	C				

The analyzed result indicated that this roundabout a whole operates at LOS E on basis degree of saturation and control delay during a peak 15minutes of the analysis hour.

As Table 4.11 describe that Merkato and Churchill left operates at LOS F with the control delay of 50.32 and 81.88 respectively. Merkato right, Minilik Left and right operates at LOS E, while Sebara Babur Left and Right and Churchill directly operates at LOS C. However, the eastbound (Merkato) approach operates at LOS E with control delay 44.089, Westbound (Minilik) approach operates at LOS E with control delay 37.342, Northbound (Churchill) approach operates at LOS E with control delay 48.437 and southbound (Sebara Babur) approach operates D with control delay 27.945. But the LOS of the intersection was E with Average control delay of 41.66.

Step 11: Compute 95th percentile Queues for each Lane

The queue length at a roundabout is estimated using the following formula.

$$Q_{95,i} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \frac{\left(\frac{3600}{C}\right)(x)}{150T}}\right]\left(\frac{C}{3600}\right) \ldots\ldots\ldots\ldots\ldots..\ldots\ldots\ldots\ldots\ldots.4.17$$

Where

$Q_{95,i}$ = 95th percentile Queues for a given lane(veh)

X = Volume to capacity ratio of the subject lane

C = capacity of the subject lane (Veh/h)

T = time period (h)

Sample calculation for Merkato direction

$x_{EB,R,} = 0.891$

C = 519veh/h

T = 0.25

$$Q_{95,EB,R} = 900 * 0.25\left[0.891 - 1 + \sqrt{(1-0.891)^2 + \frac{\left(\frac{3600}{519}\right)(0.891)}{150 * 0.25}}\right]\left(\frac{519}{3600}\right)$$

= 10.09veh take 10veh

For other approaches, a similar calculation has conducted. In detail, it is summarized in the table 4.12 below.

Table 4.12: Q95th percentile queues for each Lane (Veh)

Roundabout leg bound	Q95th (Veh)		Intersection Q95th (Veh)
	Left	Right	
Merkato(v_{cEBpce})	10	10	15
Minilik(v_{cWBpce})	8	8	
Sebara Babur (v_{cSBpce})	4	8	
Churchill(v_{cNBpce})	15	7	

As described in table 4.12 above the Q95th percentile queues for Churchill and Merkato approach was the most prolonged queue relative to the other approaches with estimated vehicles in the line are 15 and ten cars respectively. As the whole, the Q95th percentile queues for the intersection is 15. In detail, general flow conditions at Abune Petros intersection is described in the following figure 4.1

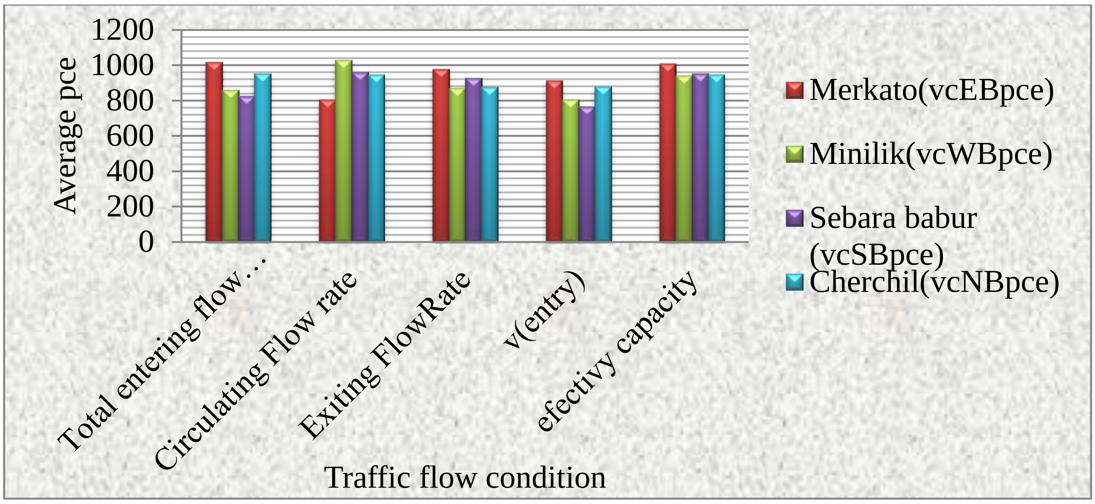


Figure 4.1: Comparison of flow condition at Abune Petros roundabout

From figure 4.1 it is seen that a roundabout has the maximum total entry flow at Merkato direction and the maximum circulating flow at Minilik direction. From total entry flow versus capacity, the maximum entry flow and effective capacity occur at Merkato side, and minimum flow occurs at Sebara Babur and minimum capacity at Minilik approach.

4.2.2 Capacity and performance analysis using SIDRA intersection method

As SIDRA INTERSECTION 5.1 defines that, Capacity is the maximum sustainable flow rate that can be achieved during a specified period under given (prevailing) road, traffic and control conditions. SIDRA INTERSECTION computes the capacity of each approach lane separately and then adds the lane capacities to obtain movement capacities. This method allows for capacity losses due to lane under-utilization and allocates the most significant degree of saturation in any lane to the movement.

Capacity can be measured with the degree of saturation, lane utilization, and the entry and opposed circulatory lanes, model calibration and level of services. The evaluation was made by both lane-by-lane and general evaluation methods using SIDRA software.

4.2.2.1 Evaluation of Existing Capacity and Performance at Abune Petros Roundabout

The total existing Capacity per movement of the intersection is measured by the total fellow /Degree of Saturation (Veh/hr.).The evaluation of Capacity was made lane by lane and general evaluation methods using the existing geometric data specified in table 3.2

above. Therefore, the Capacity of each leg and the whole intersection at each study area is shown in the following figures 4.2

4.2.2.1.1 Approach and Exit Traffic Flows at Abune Petros Roundabout

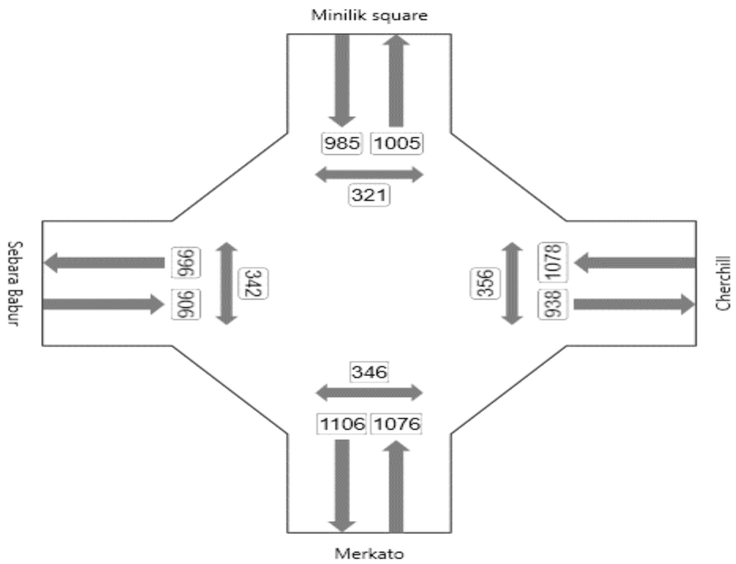


Figure 4.2: Approach and exit flows at Abune Petros Roundabout

As is shown in the above figure 4.2 exiting flow at Churchill approach is less than entering flow. But, in the other approach, it is higher than coming flow.

In traffic flow condition at a roundabout, if the entering flows greater than exit flow, the traffic congestion may occur. Therefore, unless at the side of Churchill approach, the traffic flow condition at all approaches were in the right traffic flow conditions.

4.2.2.1.2 Degree of Saturation (V/C) at Abune Petros Roundabout

The degree of saturation is one of the performance measures when a ratio of Demand Volume to Capacity (v/c). Based on highway Capacity Manual 2010, when V/C ratio is less than 0.85 the intersection service under its capacity and when V/C is between 0.85 – 1.0 the intersection service with high risk and considerable delay be looking at its level of service and also the intersection improvement will be required soon to avoid excessive delay. But, when V/C is 1.0, and above 1.0 the intersection service beyond its capacity and the demand exceeds the available capacity of the intersection, as well as excessive delays and queuing, is anticipated. Figure 4.3 below shows the summary of Degree of Saturation at Abune Petros Roundabout.

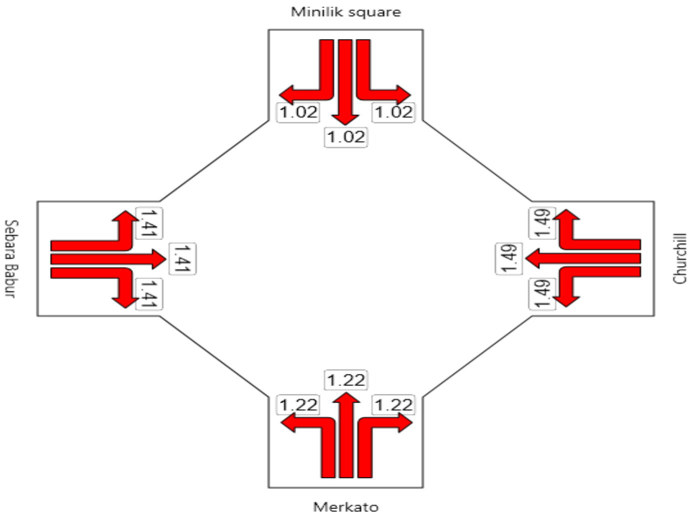


Figure 4.3: Degree of saturation (V/C) of Abune Petros Roundabout at an existing flow condition

As described in Figure 4.3 above the degree of saturation is higher than 1.0. This implies that the intersection service beyond its capacity and the demand exceeds the available capacity of the intersection, as well as excessive delays and queuing, is anticipated.

As it is described in Table 4.13 below the maximum Degree of Saturation of any lane group from all approaches and all movement directions represents the Degree of Saturation of the intersection.

Table 4.13: Summarized queue of existing Abune Petros roundabout

Approaches	South	East	North	West	Intersection
Degree of Saturation	1.22	1.49	1.02	1.41	1.49

From table 4.13 above the demand volume to capacity ratio East (Churchill) approaches higher than other approaches and dominance for the intersection as the whole. This means the demand volume or the traffic flow very exceed than the existing lane capacity at this approaches.

4.2.2.1.3 Delay (Average) at Abune Petros Roundabout

Delay is the standard parameter used to measure the performance of an intersection. Delay for unique approaches was obtained by an average of individual lanes while a delay of an intersection was obtained by an average of each approach or legs. The average control delay per vehicles of the intersection from the software output is described in the following figure 4.4

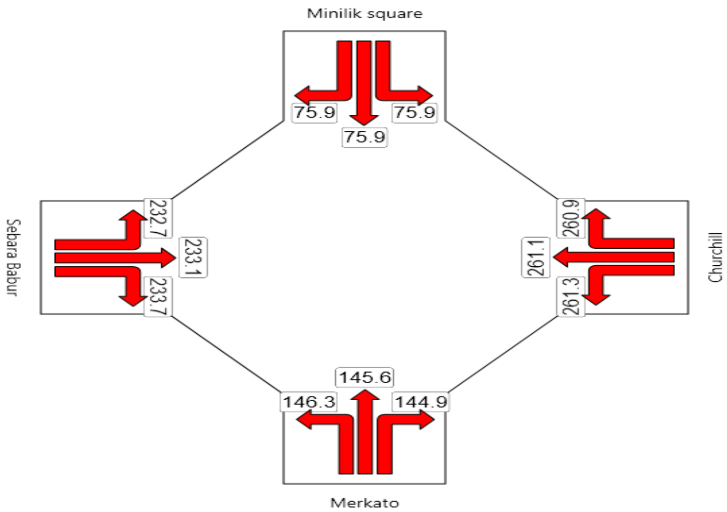


Figure 4.4: Average control delays per vehicle at Abune Petros Roundabout

From above figure 4.4, the control delay for the single approach was obtained by an average of individual lanes, but delay for an intersection was the standard had been described in table 4.14 below.

Table 4.14: Summarized average control delay per vehicle

Approaches	South	East	North	West	Intersection
Delay (Average)	145.6	261.1	75.9	233.2	179.0
LOS	F	F	F	F	F

From Table 4.14 above the highest delay was relatively occurred at East (Churchill) approaches and the Average control delay was 179.0.

4.2.2.1.4 Level of Service at Abune Petros Roundabout

Level of service is the qualitative measurement considering operational condition with the traffic stream such as time, travel, speed, freedom to maneuver, traffic interruption, comfort, convenience and the traffic quality service. The level of service at Abune Petros roundabout is summarized in the following figure 4.5

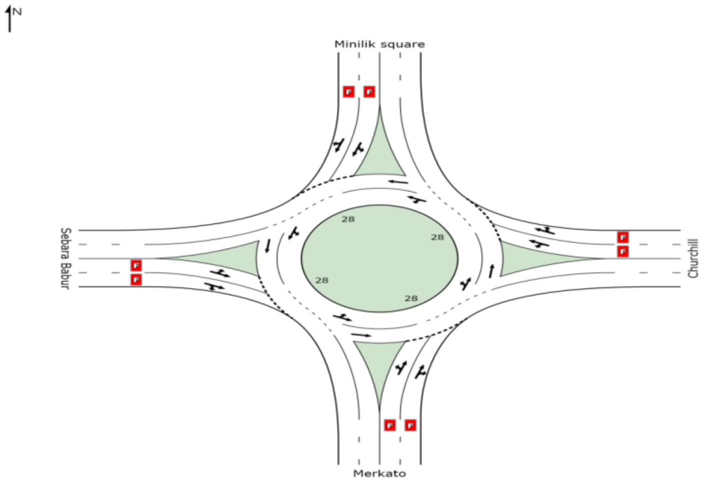


Figure 4.5: The Level of service at Abune Petros Roundabout

From figure 4.5 above the existing Level of Service at all approaches or movement, directions were F. From the Software, output implies that the flow condition was the severe problem and which need to be improved.

Table 4.15 Summary of Level of Service at Abune Petros roundabout

Approaches	South	East	North	West	Intersection
LOS	F	F	F	F	F

From the Table 4.16 above the Level of Service of the intersection is F

4.2.2.1.5 Traffic Queue at Abune Petros roundabout

The most significant 95% Back of Queue for lane used by movement (vehicles) are shown in the following figure 4.6

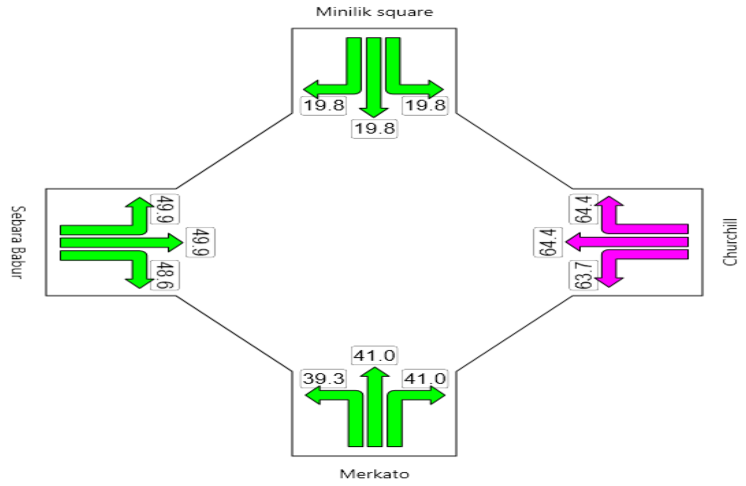


Figure 4.6: Queue for each lane groups occurred by Movement (vehicles)

As described in figure 4.6 above the closely spaced collection of cars at the roundabout approaches was formed when demand flow exceeds a capacity of the existing roundabout. From the above figure, the large number of vehicles queued at East (Churchill) approaches during the peak hour period. Therefore, queue form Churchill approaches were governing one which was considered as the queue of existing intersection. It is summarized in Table 4.16 below

Table 4.16: Summarized queue of existing at Abune Petros roundabout

Approaches	South	East	North	West	Intersection
Queue	41.0	64	19	49	64

4.2.2.1.6 Capacity of existing at Abune Petros roundabout

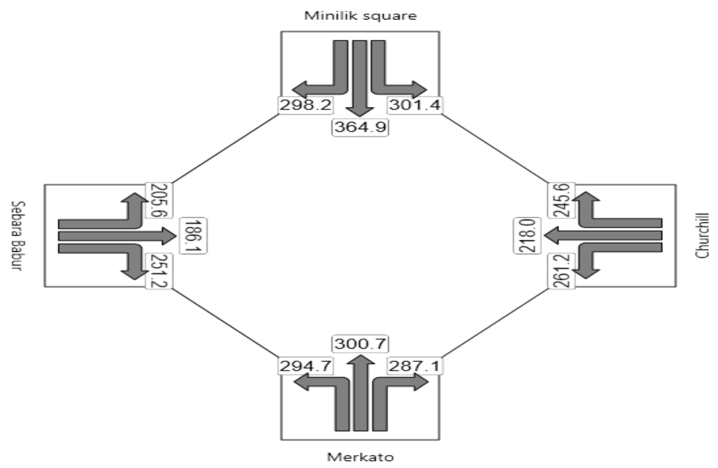


Figure 4.7: Existing capacity at Abune Petros roundabout

As described in the above figure 4.7 the maximum capacity of the intersection was determined from the maximum capacity of each movement direction of each leg or each lane group. The maximum numerical value from the legs represents the capacity of the intersection. It is exhibited in Table 4.17 below.

Table 4.17: Summarized existing capacity at Abune Petros roundabout

Entrance	Merkato	Churchill.	Minilik	Sebara Babur	Intersection
Capacity (pcu/hr.)	261.2	364.9	251.2	300.7	364.9

As it is shown in Table 4.17 above the capacity of existing roundabout was 364.9pcu/hr. This demonstrated that the capacity of a given intersection is determined by demand flow (Veh/hr.)/ A degree of Saturation. In detail, the general performance of study area in its design life is summarized in the following graph.

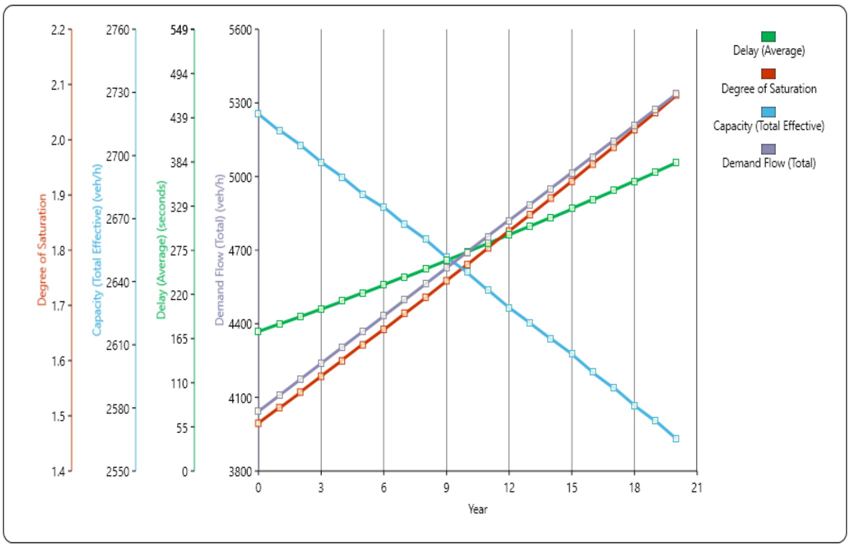


Figure 4.8 Performance graph of Abune Petros roundabout in its design life

As shown in figure 4.8 the average control delay is greater than other performance measures. The existing capacity was decreasing while demand flow increase in design life and the degree of saturation less than demand flow which shows that the existing roundabout was severe congestion its design life.

4.2.2.2 Evaluation of Existing Capacity at German Roundabout

Similarly, the evaluation of capacity and performance of German Roundabout was made lane by lane and general evaluation methods. Therefore, the Capacity and performance of each leg and the whole intersection at the study area is shown in the following figures.

4.2.2.2.1 Entry and Exit Traffic Flows at German (Mekanisa) Roundabout

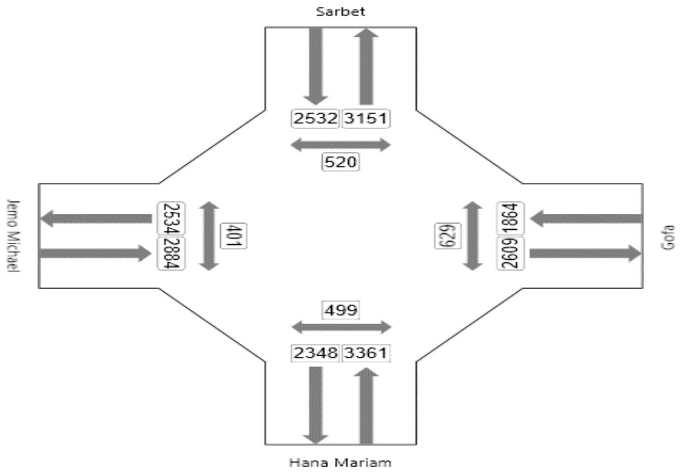


Figure 4.9: Approach and Exit flows at German Roundabout

As is shown in the above figure 4.9 exiting flow at Jemo Michael and Hana Mariam approaches were less than entering flow. But, in the other approaches were greater than entering flow. Therefore, unless at the side of both approaches, the traffic flow condition at different approaches were in the proper traffic flow conditions

4.2.2.2.2 Degree of Saturation (V/C) of German (Mekanisa) Roundabout

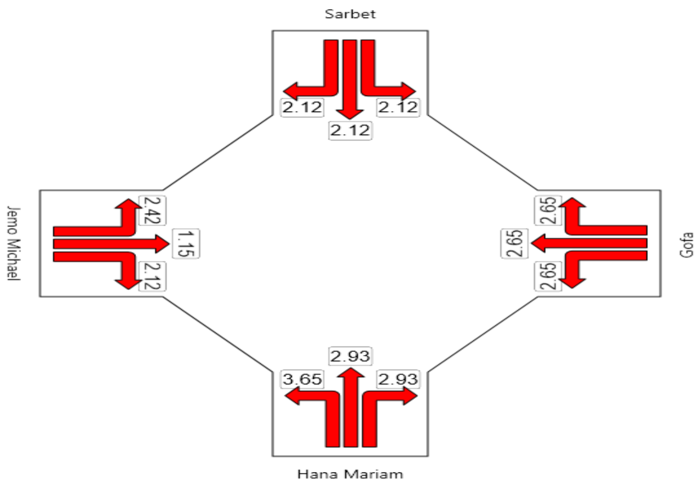


Figure 4.10: Degree of saturation (V/C) of German Roundabout at an existing flow condition

As described in Figure 4.10 above, the degree of saturation (V/C) at all movement direction was above 1.0. This implies that the intersection serves beyond its capacity and the demand exceeds the available capacity of the intersection as well as excessive delays, and severe congestion was formed.

As is described in Table 4.18 below the maximum Degree of Saturation of any lane group from all approaches and all movement directions represents the Degree of Saturation of the intersection.

Table 4.18: Summarized existing Degree of Saturation at German roundabout.

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
Degree of Saturation	3.65	2.65	2.12	2.42	3.65

From table 4.18 above the demand volume to capacity ratio of South (Hana Mariam) approach was higher than other approaches and dominance for the intersection as the whole. This means the demand volume or the traffic flow very exceed than the existing lane capacity at this approaches.

4.2.2.2.3 Average Control Delay at German (Mekanisa) Roundabout

The average control delay per vehicle of the intersection from the software output is described in the following 4.11

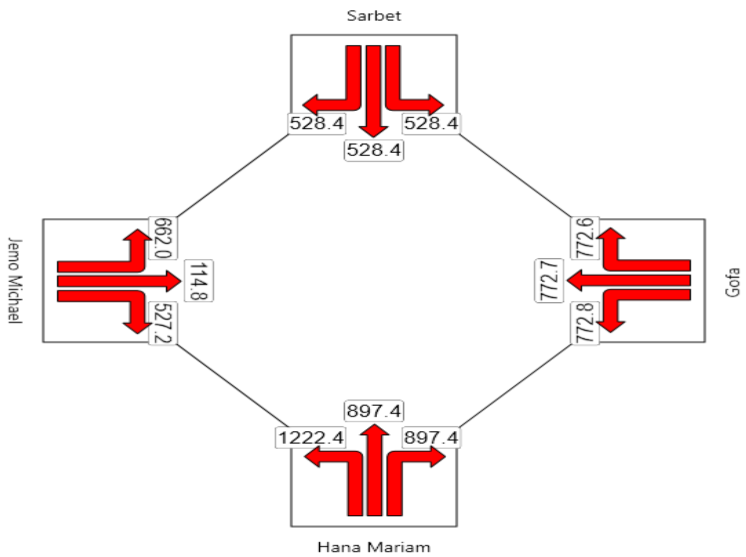


Figure 4.11: Average delay of German Roundabout

As described in the figure 4.11 above the overall approaches were under severe congestion. The average control delay at this study area was very high relative to the other study area.

Table 4.19: Summarized average control delay per vehicle at German Roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
Delay (Average)	1021.7	772.7	528.4	502.1	719.8
LOS	F	F	F	F	F

From Table 4.19 above the highest delay was relatively occurred at South (Hana Mariam) approaches and the overall intersection was serving under LOS F

4.2.2.2.4 Level of Service at German (Mekanisa) Roundabout

The level of service at German roundabout is summarized in the following figure 4.12

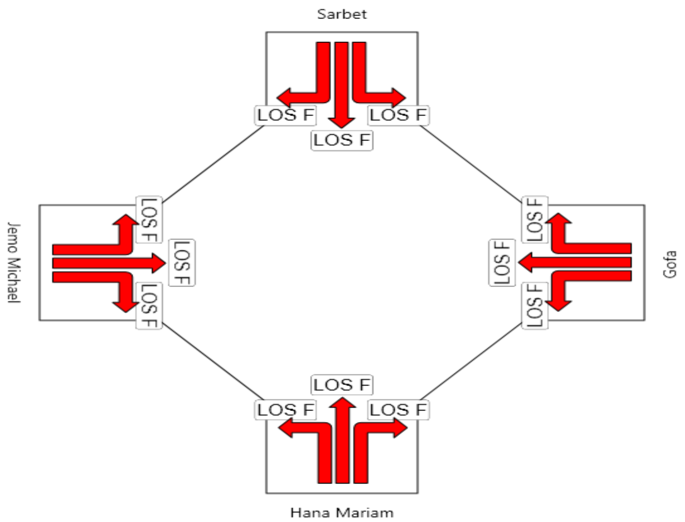


Figure 4.12: Level of service at German Roundabout

From figure 4.12 above the existing Level of Service at all side of movement directions were F. From the Software, output implies that, the flow condition was the severe problem and which needs to be solved to improve the critical congestion.

Table 4.20: Summary of Level of Service at German roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
LOS	F	F	F	F	F

From the Table 4.20 above the Level of Service of the intersection is F

4.2.2.2.5 Traffic Queue at German (Mekanisa) roundabout

The most significant 95% Back of Queue for lane used by movement (vehicles) are shown in the following figure 4.13

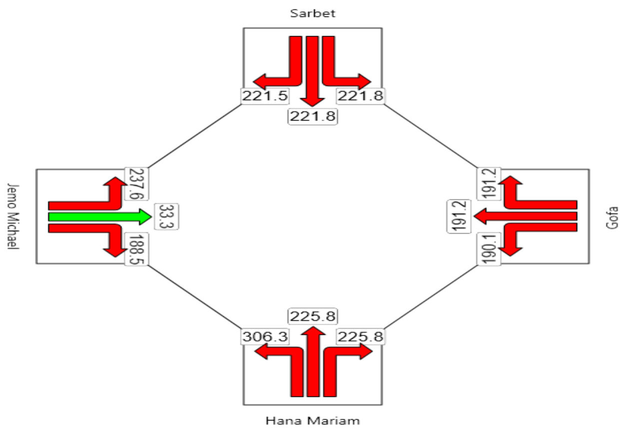


Figure 4.13: Queue for each lane groups occurred by Movement (vehicles)

From the above figure 4.13, a large number of cars queued at South (Hana Mariam) approaches. Therefore, the queue from Hana Mariam approaches was governing one which was considered as the queue of existing intersection. It is summarized in Table 4.21 below

Table 4.21: Summarized queue of existing German roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
Queue	306.3	191.2	221.8	237.6	306.3

4.2.2.2.6 Capacity of existing at German (Mekanisa) roundabout

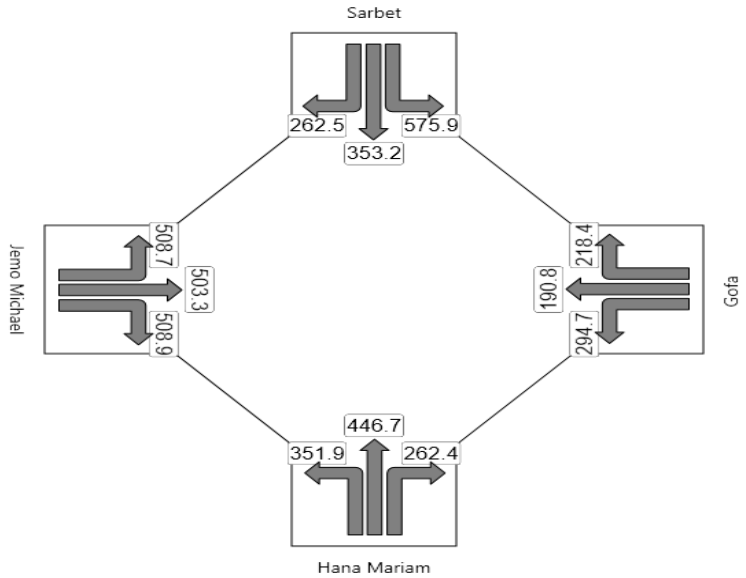


Figure 4.14: Capacity of existing German roundabout

As described in the above figure 4.14 the maximum capacity of the intersection was exist at North (Sarbet).The maximum numerical value from the legs represents the capacity of the intersection. It is exhibited in Table 4.22 below.

Table 4.22: Summarized Capacity of existing German roundabout

Entrance	South(Hana)	East(Gofa)	North(Sarbet)	West(Jemo)	Intersection
Capacity (pcu/hr.)	446.7	294.7	575.9	508.9	575.9

As it is shown in Table 4.22 above the capacity of existing roundabout was 575.9pcu/hr. This demonstrated that the capacity of a given intersection is determined by demand flow (Veh/hr.)/ A degree of Saturation.

4.2.3. Re-Analyzed of Capacity and Performance at the study area

Since the existing capacity of the study areas was over saturated and exists in a severe problem and the study areas, have re-analyzed and exhibited asunder here.

4.2.3. 1 Re-Analyzed Capacity and Performance at Abune Petros Roundabout

Abune Petros roundabout was re-analyzed by increasing circulating lane from 2m to 3m; island diameter from 27.8m to 80m, lane width was the same as existing 6m. The results are described asunder here.

4.2.3. 1.1 Revised Degree of Saturation at Abune Petros Roundabout

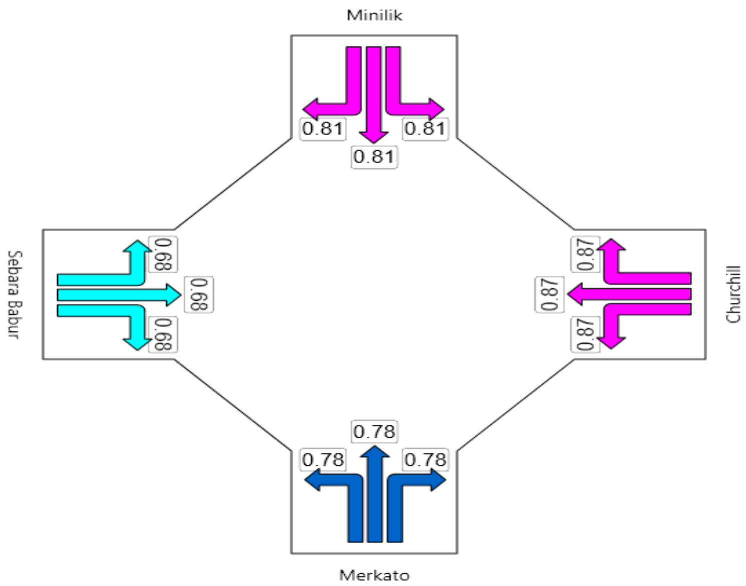


Figure 4.15: Revised degree of saturation at Abune Petros Roundabout

As shown in the figure 5.15 above the adjusted volume to capacity ratio of the roundabout (0.87) is less than to the existing value to capacity ratio (1.49). This implies that the capacity of a re-analyzed roundabout is higher than the existing one and there is no more congested and operates under normal condition relative to the former one**.** The V/C of South, West, and North were less than 0.85 which means, these approaches had shown better flow condition. But, the east path has demonstrated heavy and Traffic congestion. These approaches still need improvement to minimize the V/C ratio and increase the capacity of the roundabout. In detail, it has been noted in table 4.23 below.

Table 4.23: Summary of Revised Degree of Saturation at Abune Petros Roundabout

Approaches	Merkato	Churchill	Minilik	Sebara Babur	Intersection
Degree of Saturation	0.78	0.87	0.81	0.68	0.87

4.2.3.1.2 Revised Delay at Abune Petros Roundabout

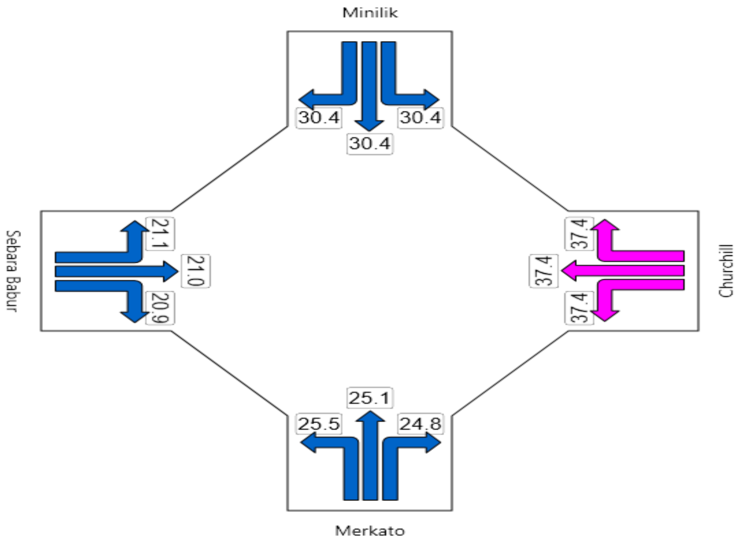


Figure 4.16: Revised delays at Abune Petros Roundabout

In table 4.4 the intersection delay was 179.0. But, as shown in figure 4.16 above no more delay was formed after re-analysis has been conducted it was decreased to 28.9

Table 4.24: Summary of Revised Delays at Abune Petros Roundabout

Approaches	Merkato	Churchill	Minilik	Sebara Babur	Intersection
Delay (Average)	25.1	37.4	30.4	21.0	28.9
LOS	C	D	C	C	C

As described in table 4.14 above the LOS at each approach is F, but in table 4.24 above the LOS at each approach are C except East (Churchill) direction after re-analyzed. The LOS D at the direction of Gofa is due to the traffic congested circulating at this side.

4.2.3.1.3 Revised Level of Service at Abune Petros Roundabout

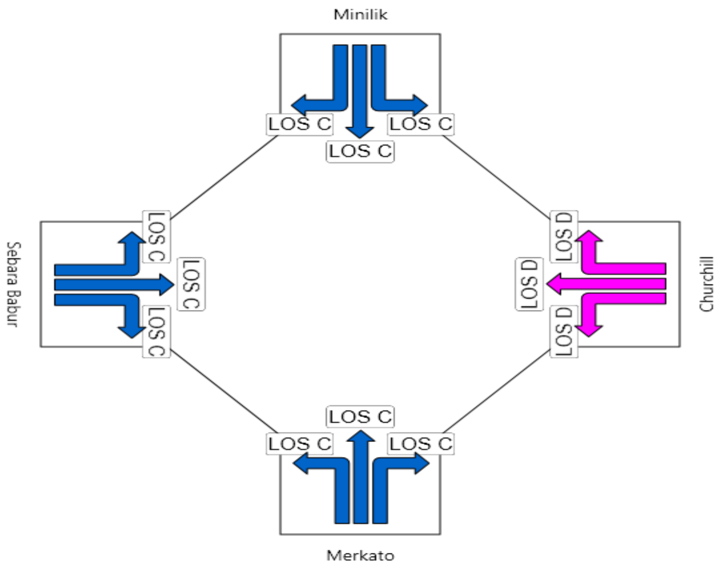


Figure 4.17: Revised level of service at Abune Petros Roundabout

From figure 4.5 above the existing Level of Service at all approaches or movement directions was F. But, as result on figure 4.17 shown that, the level of service at three approaches are C while at Churchill approach is D. This implies that, after the re-analyzed was made and the increments of island diameter, number of lane and lane width have been conducted, the level of service of the intersection is operating under normal traffic flow conditions. In detail exhibited in table 5.25 below.

Table 4.25: Summarized Level of Service at Abune Petros Roundabout

Approaches	Merkato	Churchill	Minilik	Sebara Babur	Intersection
LOS	C	D	C	C	C

4.2.3.1.4 Revised Capacity at Abune Petros Roundabout

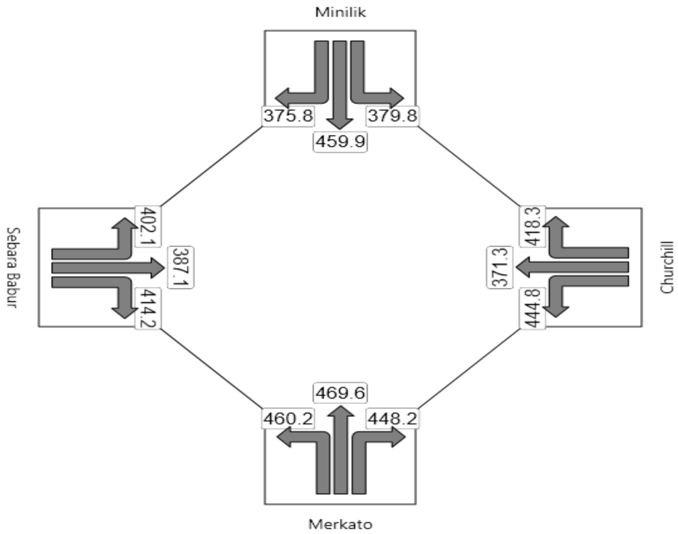


Figure 4.18: Capacity of existing at Abune Petros roundabout

As described in the above figure 4.18 the maximum capacity of the intersection was determined from the maximum capacity of each movement direction of each leg or each lane group. The maximum numerical value from the legs represents the capacity of the intersection. It is exhibited in Table 4.26 below.

Table 4.26: Summarized Capacity of existing Abune Petros roundabout

Entrance	Merkato	Churchill	Minilik	Sebara Babur	Intersection
Capacity (pcu/hr.)	469.6	444.8	459.9	414.2	469.6

As is shown in Table 4.26 above the capacity of a revised roundabout was 469.6pcu/hr. This is greater than existing capacity (364.9). This showed that the capacity of a given intersection depends on island diameter, lane width and the number of lane including traffic flow.

4.2.3.1.5 Revised Traffic Queue at Abune Petros Roundabout

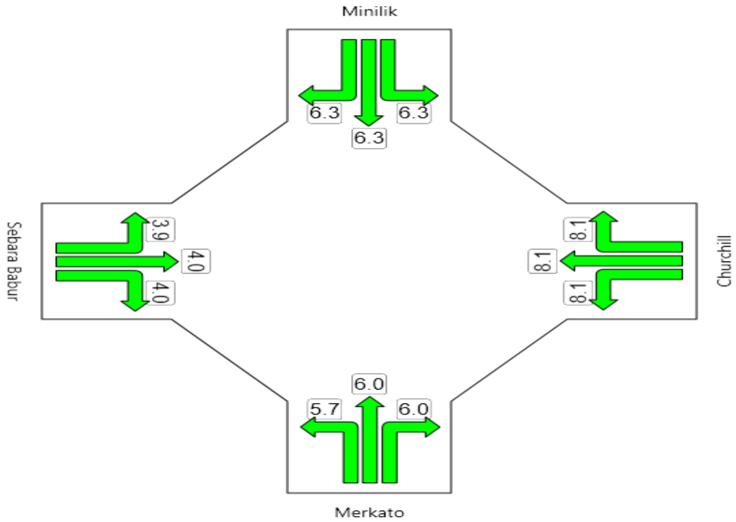


Figure 4.19: Queue adjusted for existing roundabout of each lane at Abune Petros Site

The queue number of at Abune Petros Site before adjustment was 64. But, after a change has made, it was decreased to 8 vehicles as shown in the Table 4.27 below. It implies that the roundabout flow condition has less delay, while the flow condition at each lane group is also stable.

Table 4.27: Revised Queue for existing roundabout of Abune Petros Site

Approaches	Merkato	Churchill	Minilik	Sebara Babur	Intersection
Queue	6.0	8.1	6.3	4.0	8

4.2.3.1.6 Revised Queue Length at Abune Petros Site

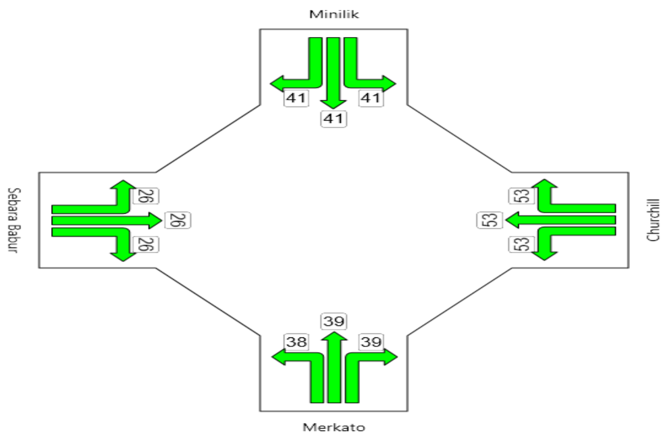


Figure 4.19: Revised queue lengths at Abune Petros Site

Table 4.28: Summarized Queue Length at Abune Petros Site.

Approaches	South	East	North	West	Intersection
Queue Distance	39	53	41	26	53

As described in table 4.28 above the maximum queue length is 53m. This implies that there is no more queue at Abune Petros site.

4.2.3.1.7 Queue storage ratio for revised Abune Petros roundabout

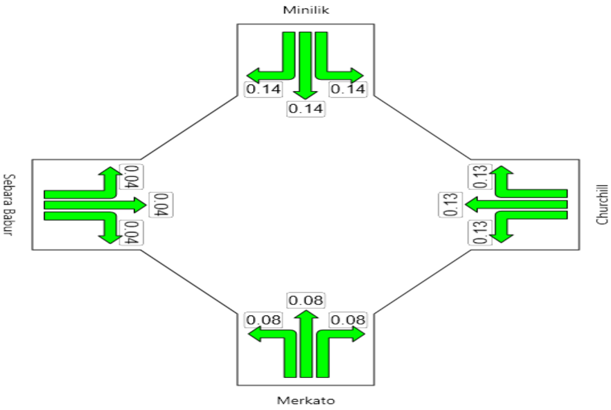


Figure 4.20: Queue storage ratios for revised Abune Petros roundabout

As shown in figure 4.20, the queue storage ratio is the ratio of the queue length to the available queue length which is less than 1. This indicated that the flow condition in each lane groups is the stable and small delay.

4.2.3.1.8 Performance graph of Abune Petros roundabout in its design life

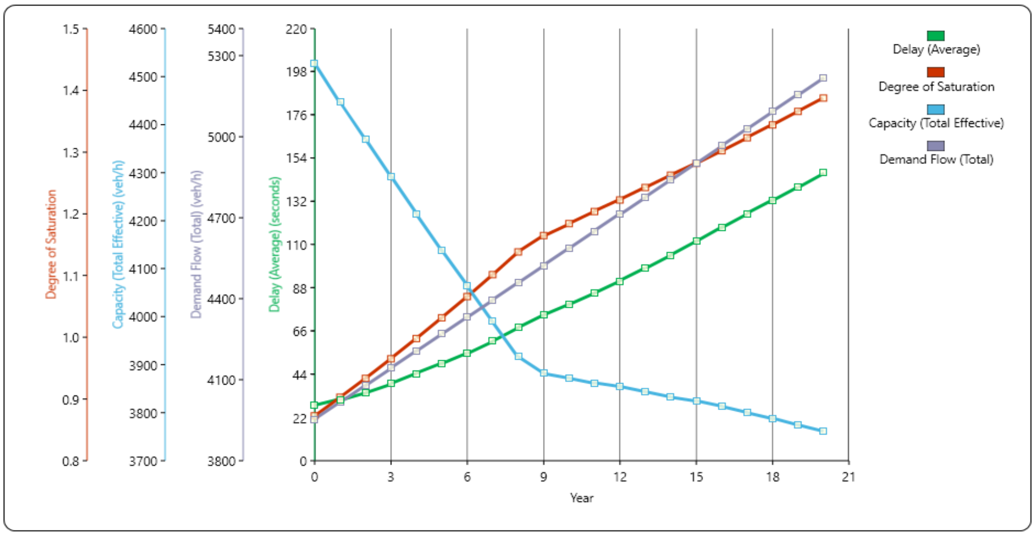


Figure 4.21: Performance graph of revised Abune Petros roundabout in its design life

As shown in figure 4.21, the average control delay is less than other performance measures. The improved capacity will be less than a degree of saturation after six years of its design life. Up to this, the capacity will be in good traffic flow condition, and there is no congestion and delay. After six years the roundabout will be severe congestion due to as demand traffic flow is increasing while the capacity will decreasing in design life.

4.2.3.2 Re-Analyzed Capacity and Performance at German (Mekanisa) Roundabout

Re-analyzed capacity at German (Mekanisa) Roundabout was conducted by increasing number of lanes from 2 to 3, lane width 4.33m to 5.0m., circulating path from 3 to 4, island diameter from 50.8m to 90m. The results are described in detail as under here.

4.2.3.2.1 Revised Degree of Saturation at German (Mekanisa) Roundabout

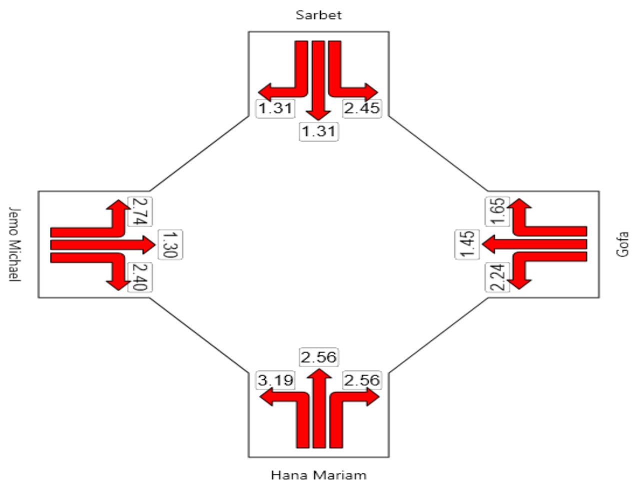


Figure 4.22: Revised Degree of Saturation at German (Mekanisa) Roundabout

As shown in the figure 4.22 above the adjusted degree of saturation is less than to the existing degree of saturation. This implies that the capacity of a re-analyzed roundabout is higher than the existing one. However**,** at this roundabout, the traffic flows were still over saturated. The degree of saturation at this study area is higher than one. This implies that at this study area the traffic flows cannot be improved after increased the island diameter 50.8m to 90m, the number of lanes 3 to 4 and lane width 4.33m to 5.0m. Therefore, it is better if the roundabouts changed to signal interaction.

Table 4.29: Summary of Revised Degree of Saturation at German (Mekanisa) Roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
Degree of Saturation	3.19	2.24	2.45	2.74	3.19

As described table 4.29 above the intersection degree of saturation was 3.65, but after revised it decreased to 3.19 still not be less than 1. It shows that the traffic flow even over saturated.

4.2.3.2.2 Revised Delay at German (Mekanisa) Roundabout

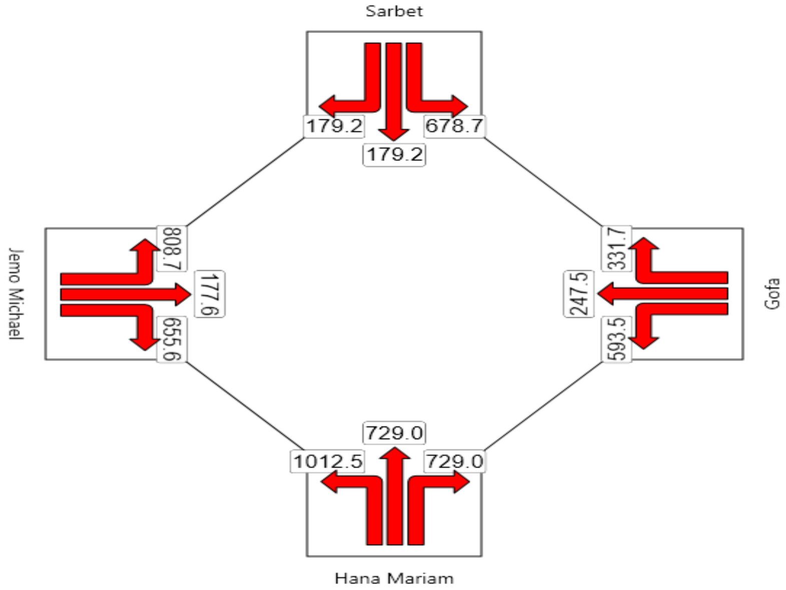


Figure 4.23: Revised Delays at German (Mekanisa) Roundabout

As shown in figure 4.23 above even if the number of delays is minimized, the suspension was formed after re-analyzed has been conducted

Table 4.30: Summary of Revised delay German (Mekanisa) Roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
Delay (Average)	837.4	418.5	420.6	625.1	605.8
LOS	F	F	F	F	F

As noted on Table 4.19 above the existing delay was 719.8 with LOS F. But, after re-analyzed, it was decreased to 605.8. However, LOS of the intersection was F.

4.2.3.2.3 Revised Level of Service at German (Mekanisa) Roundabout

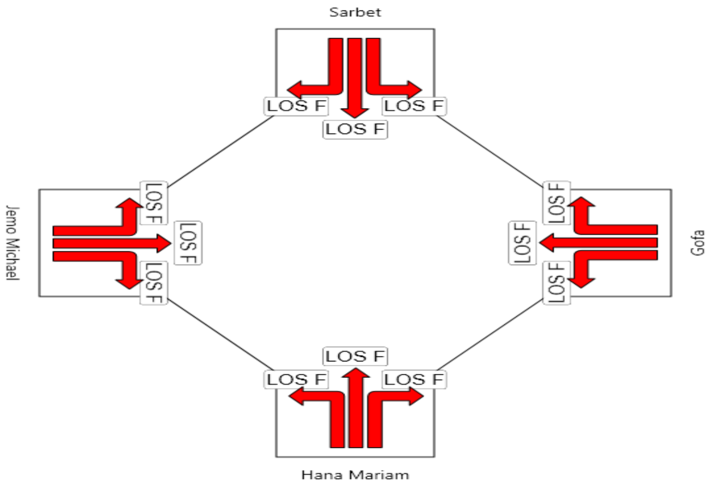


Figure 4.24: Revised Level of Service at German (Mekanisa) Roundabout

As result on figure 4.25 shown that, the level of service at all approaches are F. This implies that, after the re-analyzed was made and the increments of island diameter, number of lane and lane width have been conducted, the level of service of the intersection is operating in LOS F. In detail exhibited in table 4.30 below.

Table 4.31: Summarized Level of Service at German (Mekanisa) Roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
LOS	F	F	F	F	F

4.2.3.2.4 Revised Capacity at German (Mekanisa) Roundabout

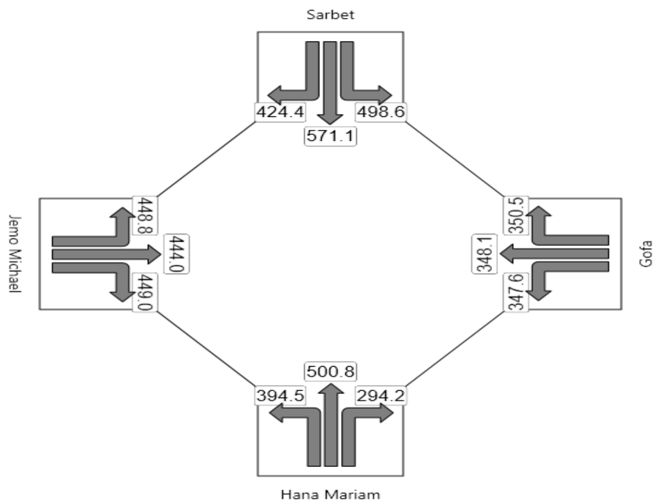


Figure 4.25: Capacity of existing at German (Mekanisa) Roundabout

As described in the above figure 4.22 the maximum capacity of the existing intersection were 575.9, but after revised it decreased to 571.1. There is no more change due to increasing the island diameter, number of lane and lane width.

4.2.3.2.5 Revised Traffic Queue at German (Mekanisa) Roundabout

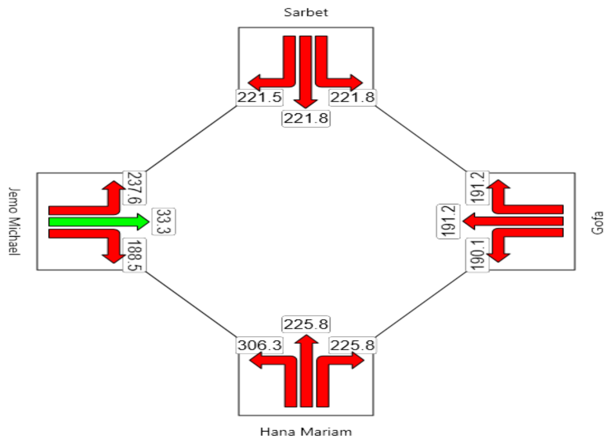


Figure 4.26: Queue adjusted for of each lane at German (Mekanisa) Roundabout

The queue number at German (Mekanisa) Roundabout Site before adjustment was 306. But, after a change has made, it was decreased to 283 vehicles as shown in the Table 4.31 below. This implies that the roundabout flow condition has less delay and the flow condition at each lane group is also not stable and forms a suspension.

Table 4.32 Revised Queue for existing roundabout at German (Mekanisa) Roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
Queue	283.4	143.5	238.6	256.9	283.4

4.2.3.2.6 Revised Queue Length at German (Mekanisa) Roundabout

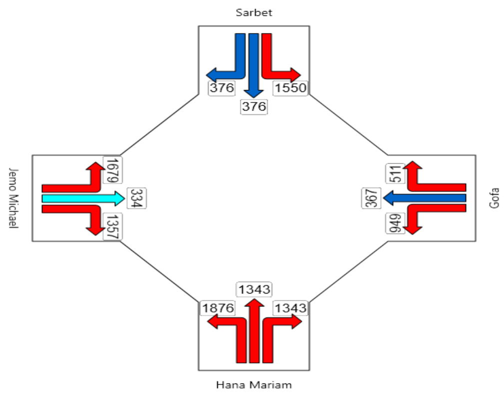


Figure 4.27: Revised queue lengths at German (Mekanisa) Roundabout

Table 4.33 Revised Queue Lengths at German (Mekanisa) Roundabout

Approaches	Hana Mariam	Gofa	Sarbet	Jemo Michael	Intersection
Queue Distance	1876	949	1550	1679	1876

As shown in table 4.32 the queue length after revised 1876 and this is a severe problem for this roundabout.

4.2.3.2.7 Queue storage ratio for revised at German (Mekanisa) Roundabout

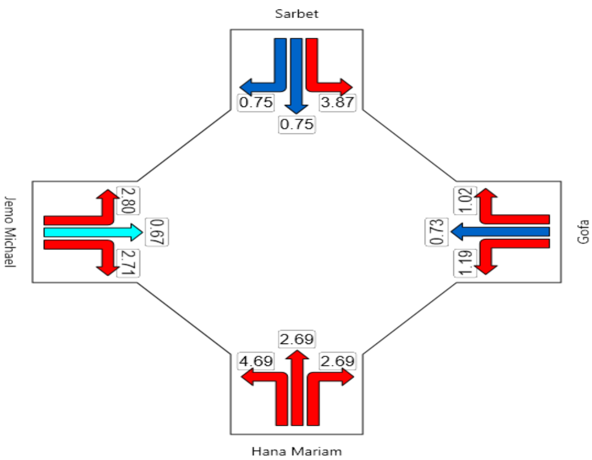


Figure 4.28: Queue storage ratios for revised at German (Mekanisa) Roundabout

As shown in figure 4. 28, the queue storage ratio is higher than 1. This indicated that the flow condition in each lane groups is still not stable and a high delay occurred.

4.2.3.2.8 Performance graph of German (Mekanisa) Roundabout in its design life

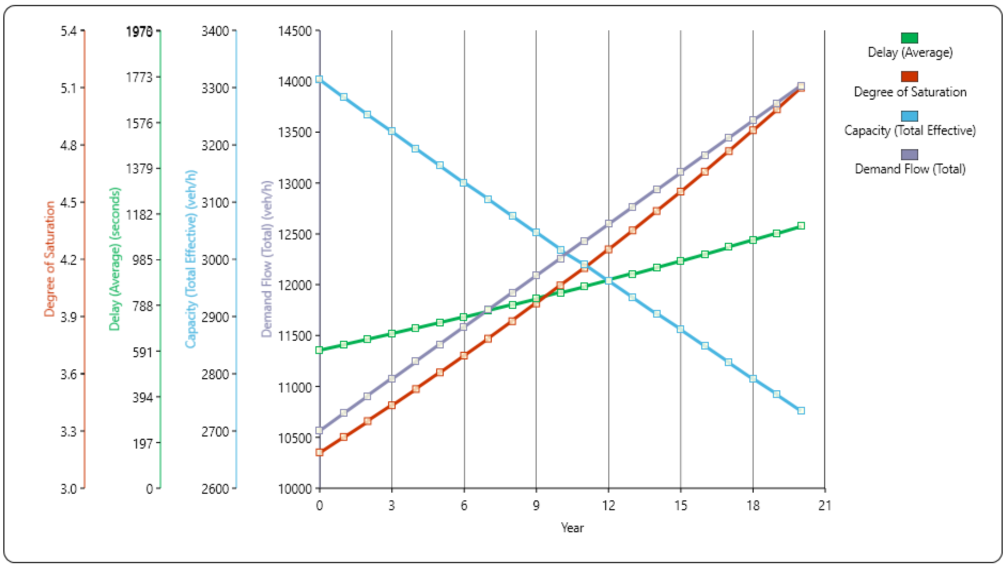


Figure 4.27: Performance graph of German (Mekanisa) Roundabout in its design life

As shown in figure 4.29 the capacity of a roundabout decrease from zero years to always in all its design life. The degree of saturation is also less traffic demand flow. This implies that the existing capacity of the revised roundabout is less than demand flow. Due to this the congestion still formed at the study area.

The summary of results and comparisons of between two methods are described in on tables of appendix-19, 20 and 21 in detail.

4.3 The Identified significant factors affecting the Capacity and Performance level of roundabouts

The study showed that the significant factors contributing to Capacity and Performance Level of roundabout were related to the number of entry lane, some circulating lanes, Island diameter, and high traffic flow, the proportion of heavy vehicles in the traffic stream, unbalanced traffic and movement condition on the approaches. In detail, it is listed as under here.

4.3.2 High traffic flow

The study showed that the collected traffic data at peak hour on the roundabout approaches have high unbalanced traffic flow and especially German roundabout is found to be oversaturated of traffic flow. When it compared to the existing roundabout geometric configuration, the number of lanes, island diameter, lane width with the traffic flow condition, it is unbalanced and exists in severe problems.

4.3.3 The proportion of heavy vehicles

The percentage of heavy vehicles can affect the capacity and performance of roundabout. Especially Abune Petros roundabout is located near to the marketplace at which the proportion of freight vehicles is very high due to this the percentage of heavy vehicles in the traffic stream is very high. It is known that one heavy vehicle can displace two or greater than two passenger cars unit. Therefore if a heavy vehicle is increased in the entry vehicles, the capacity of roundabout decreased. The following figures show relationship

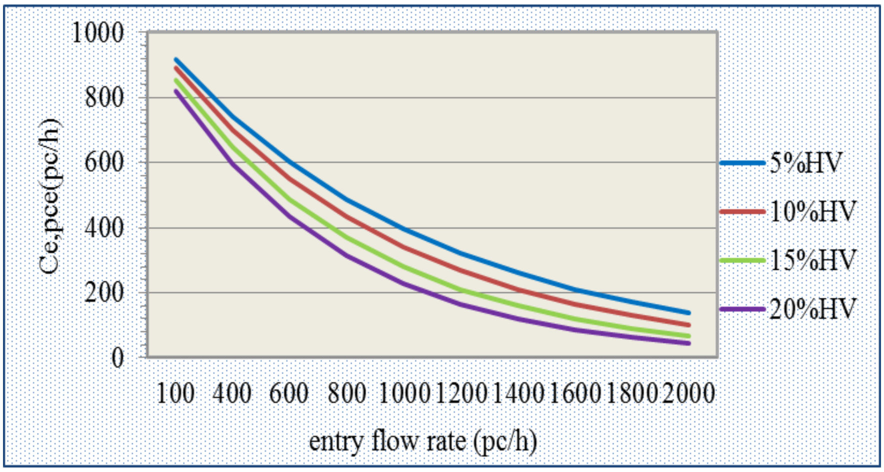


Figure 4.30: Effects of heavy vehicles proportion in the capacity of a roundabout.

As it is described in the above figure 4.30, if the volume of transit or freight vehicles is increased in the traffic stream, the capacity of a roundabout is decreased based on the proportion of heavy vehicles, the island diameter and the width of circulatory lanes. Because of the significant trucks or transit vehicles require the use of the broader lane to rapid maneuver or freedom to rotate to the roundabout within the given gap. From the above figure 5%heavy vehicle less effect on capacity than the 10%, 15% and 20% heavy vehicles.

4.3.4 Geometric condition

Geometric data such as Lane width, Number of the lane, Number of circulating lane, island diameter, entry angle and radius are the other factors affected the capacity and performance of selected roundabout. As the analyzed result showed that, the existing geometric condition was not handled the current demand flows. Due to this the over saturation and traffic congestion occurred in the selected study area.

4.3.5 Environmental and grade factors

The Environment Factors represents concerning classes, operating speeds, heavy vehicles, pedestrians facilities, heavy vehicle activity, and parking maneuvers are not considered during roundabout design which affects the vehicle movements on approach and exit sides as well as at circulating road as relevant.

4.4 The investigated problems and suggested remedial measures of existing roundabout

4.4.1 Comparison of analysis Manual using HCM, 2010 and SIDRA method

The analyzed capacity and Performance of roundabout in this research is using both manual calculations based on HCM,2010 and SIDRA INTERSECTION software model. HCM, 2010 is applicable for roundabout which has not more than two entry and two circulating lanes. Due to this Abune Petros roundabout was analyzed using both manual HCM, 2010 and SIDRA software method while German(Mekanisa) roundabouts were analyzed using only SIDRA software. As the study result shown that the two selected Roundabouts in Addis Ababa City were low levels of service, congestion, and extreme delays. Due to the continuing increase in commercial and residential development, the Roundabouts would not be able to accommodate the increasing traffic demand. As the result Capacity, delay, level of service, speed, and queuing lengths have occurred from Manual and SIDRA INTERSECTION 5.1 results.

The analysis based SIDRA method indicated that Abune Petros Roundabout as a whole operates at LOS F by the degree of saturation and control delay during a peak 15minutes of the analysis hour. Table 4.14 indicates that Abune Petros roundabout operates at LOS F with the control delay of 179.0. The reason why Using SIDRA INTERSECTION software result differs from using the manual calculation in Highway Capacity Manual 2010, multilane roundabout capacity models are limited to two-by-two lanes that are two entry lanes and two circulating lanes. These capacity models are not valid for three-by three-lane roundabouts (three entry lanes and three flowing lanes) due to traffic flow characteristics and features. And also, SIDRA software uses system calibration such as environmental factors, grade factors, and other factors.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

- ⊗ The identified significant factors affecting capacity and performance of roundabouts are included high unbalanced traffic flow at peak hour period, the unbalanced proportion of heavy vehicles, a geometric condition such as the number of lanes, lane width, island diameter, number of circulating lanes, environmental and grade factors.
- ⊗ Analyzing the capacity and performance using the manual calculation based upon highway Capacity Manual is not the same as using by SIDRA INTERSECTION software. Analyzing using HWCM 2010 is applicable only for multilane which has not more than two entries and circulating lanes and environmental factors not calibrated to it.
- ⊗ As a result of SIDRA software indicated that the selected roundabouts were oversaturated, long time delay, long queue length and operate beyond their capacity (LOS F). Because, the parameters such as environmental factors, grade factors, Vehicles and Population growth rate are calibrated to the software which results affect the capacity and performance of roundabout.
- ⊗ As a mitigation measures, Abune Petros Roundabout could be improved by entry metering, increasing island diameter, number of circulating lanes, lane width. While German (Mekanisa) roundabout recommended improving by changing roundabout to signal intersection.

5.2 Recommendations

- ⊗ During the research study, there were many challenges confronted the researcher and data collector; the problems were there was no systematic traffic counter machine for traffic data collection. Due to this, there is missing traffic data while counting manually from the site or video recorder. Therefore, the researcher recommends that the University and government industries for the fulfillment of the research.
- ⊗ In this paper, the study was focused only the evaluation of capacity and performance of selected roundabout using a manual calculation based upon HCM,2010 and software model with some proposed improving method, due to financial and time limit the cost analysis and safety not considered. So, it is better if other researchers study based on cost evaluation and security for the next time to improve roundabouts in Addis Ababa City.
- ⊗ As analyzed results from both methods indicated that, using manual method based upon HCM, 2010 was not the same as using SIDRA method. This was due to there is no calibration factors which affects capacity and performance of roundabout in using manual calculation based upon HCM, 2010. Therefore, to analysis capacity and performance of roundabouts it is better if based upon SIDRA INTERSECTION software.
- ⊗ A study showed that when island diameter and lane width increased the size and performance of roundabout are increased. Therefore, to improve the capacity and performance of roundabout at Abune Petros Roundabout, the island diameter, the number of entry, exit lanes, and lane width shall be increased to the specified in the above chapter four.
- ⊗ Because of heavy traffic flow at entry and circulating of the Roundabouts, it will not function efficiently due to there are insufficient acceptable gaps in the flowing traffic stream at both sites. Therefore, it is better if entry metering installed at each sites to add time gap between traffics.
- ⊗ Since Abune Petros roundabout is located around the market area, the flow of freight vehicles and other heavy vehicles are experienced at this intersection. Therefore, it is better if heavy and freight vehicles are diverted from this roundabout.

- ⊗ As it was shown on the result, at German (Mekanisa) roundabout, the traffic flow conditions were over saturated, long time delay, long traffic queue existed. Even after the re-analyzed by increasing island diameter from 50.8m to 90m, some lanes from two to three, lane width from 4.33m to 6m and circulating lanes from 3 to 4. Still, the capacity of the re-analyzed roundabout was in unstable traffic condition, and LOS was F, long time delay and oversaturation existed. Therefore, to improve the capacity and performance of roundabout, it is better if, the roundabout change to signal intersection
- ⊗ Grade Separation is means minimizing time delay especially on heavily traveled roads. The traffic flows from Hana Mariam and Jemo Michael to Sarbet were very massive and oversaturated. To improve such problems, it is better if Rapid Bus Transport incorporating grade separated changes can offer significant benefits at German roundabout.

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APPENDIXES

Appendix A: Traffic Data collection

Appendix -1 Vehicle Volume data at Abune Petros Site

Date: June 20-22/2017

Time: 8:00am - 8:15am

L.Name	Left			Through			Right			U-Turn			Total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	Aveg .pcu
Merkato	54	3	6	67	3	8	54	3	6	3	0	1	246
Minilik	61	2	1	61	1	5	61	1	1	0	0	0	210
Sebara Babur	48	2	2	47	1	2	57	1	6	0	0	0	184
Churcher	74	1	8	58	2	5	55	1	7	3	0	0	250
Total													890

Date: June 20-22/2017

Time: 8: 15am - 8:30am

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	pcu
Merkato	61	2	4	64	4	5	52	1	7	1	0	3	240
Minilik	62	1	3	62	1	5	12	0	1	0	0	0	167
Sebara Babur	49	2	5	56	1	2	65	0	3	0	0	0	202
Churcher	67	2	4	70	0	3	53	0	6	1	0	0	229
Total													837

Date: June 20-22/2017

Time: 8: 30am - 8:45am

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	pcu
Merkato	57	4	5	63	1	4	70	3	8	3	1	3	263
Minilik	62	0	3	55	1	4	61	2	6	0	0	0	218
Sebara Babur	49	1	5	55	2	3	55	2	2	0	0	0	195
Churcher	67	0	4	70	2	5	60	1	2	6	1	0	241
													917

Date: June 20-22/2017

Time: 8:45am - 9:00am

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	t	pcu
Merkato	75	0	5	66	1	2	62	3	6	2	0	0	244
Minilik	51	1	3	53	5	0	59	1	3	0	0	0	190
Sebara Babur	47	0	5	59	0	0	66	1	3	2	0	0	200
Churcher	73	1	0	61	3	3	56	0	6	0	0	0	220
													853

Date: June 20-22/2017

Time: 5:30 - 5:45pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	pcu
Merkato	58	1	7	61	3	6	52		6	2	0	0	231
Minilik	59	1	1	64	2	6	60	1	3	0	0	1	218
Sebara Babur	56	2	3	55	1	2	62	1	3	1	0	0	204
Churcher	65	2	4	54	2	4	50	2	4	1	0	0	211
Total													864

Date: June 20-22/2017

Time: 5:45- 6:00pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	t	pcu
Merkato	56	2	8	61	3	6	60	3	7	1	0	0	245
Minilik	61	0	2	64	2	9	56	1	4	0	0	0	225
Sebara Babur	49	3	3	54	0	2	63	1	6	1	0	0	201
Churcher	62	1	5	59	3	5	60	1	4	2	0	1	228
Total													900

Date: June 20-22/2017

Time: 6:00 -6:15pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	truck	P.Car	Bus	truck	P.Car	Bus	truck	P.Car	Bus	truck	pcu
Merkato	64	1	8	66	4	5	66	2	4	1	0	0	254
Minilik	54	1	1	59	1	7	60	1	4	0	0	1	211
Sebara Babur	51	1	3	56	0	2	57	3	4	1	0	0	199
Churchill	66	1	6	56	1	4	54	1	5	1	0	0	221
Total													886

Date: June 20-22/2017

Time: 6:15pm - 6:30pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	t	pcu
Merkato	53	1	9	56	3	6	55	3	3	4	0	0	228
Minilik	58	1	2	56	3	6	47	1	2	0	0	0	193
Sebara Babur	49	1	3	51	1	2	74	3	4	0	0	0	208
Churcher	63	2	2	57	3	5	59	1	3	0	0	1	217
Total													847

Appendix -2 Vehicle Volume data at German (Mekanisa)

Date: June 20-22/2017

Time: 8:00am - 8:15am

L.Name	Left			Through			Right			U-Turn			Total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	Aveg. pcu
Hana Direction	136	3	21	200	13	18	136	3	21	18	1	2	688
Gofa Direction	53	3	7	92	5	10	114	5	7	57	2	7	425
Michael Direction	224	8	11	113	2	5	162	6	16	53	1	7	688
Sarbet Direction	72	2	6	117	8	7	120	4	6	110	8	4	522
Total													2324

Date: June 20-22/2017

Time: 8:15am - 8:30am

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	pcu
Hana Direction	186	6	15	240	11	8	130	2	7	19	0	3	699
Gofa Direction	71	1	5	84	5	10	17	3	8	65	2	6	333
Michael Direction	166	7	5	104	3	9	167	5	16	48	3	5	610
Sarbet Direction	97	2	12	151	8	9	110	4	5	79	8	6	561
Total													2202

Date: June 20-22/2017

Time: 8:30am - 8:45am

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	pcu
Hana Direction	156	9	8	177	8	15	192	13	18	107	3	6	814
Gofa Direction	65	2	6	64	2	5	84	2	10	110	2	5	408
Michael Direction	48	3	5	192	7	7	98	2	6	48	3	5	474
Sarbet Direction	88	9	7	94	2	9	101	8	5	154	2	21	589
Total													2284

Date: June 20-22/2017

Time: 8:45am - 9:00am

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	t	pcu
Hana Direction	229	6	15	258	13	9	132	4	9	23	1	4	782
Gofa Direction	63	1	4	83	4	0	122	2	7	58	1	6	388
Michael Direction	175	4	12	104	2	10	131	1	17	56	1	6	603
Sarbet Direction	100	3	7	151	11	8	111	2	8	87	8	6	566
Total													2339

Date: June 20-22/2017

Time: 5:30pm - 5:45pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	pcu
Hana Direction	188	4	18	236	12	14	133		10	18	2	3	725
Gofa Direction	56	3	3	80	5	4	86	3	3	61	3	3	347
Michael Direction	197	7	6	100	1	4	131	6	12	40	3	4	571
Sarbet Direction	74	3	4	131	9	4	88	4	4	144	7	1	514
Total													2156

Date: June 20-22/2017

Time: 5:45pm - 6:00pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	t	pcu
Hana Direction	183	6	19	232	9	9	140	6	8	19	1	1	713
Gofa Direction	72	1	5	56	3	4	96	3	5	74	0	6	364
Michael Direction	219	6	7	96	2	5	128	2	12	28	1	4	565
Sarbet Direction	90	1	4	116	2	2	103	7	3	127	15	4	518
Total													2160

Date: June 20-22/2017

Time: 6:00pm - 6:15pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	truck	P.Car	Bus	truck	pcu
Hana Direction	190	8	13	231	8	10	126	6	7	17	1	0	686
Gofa Direction	92	1	8	83	5	4	98	3	6	80	2	8	444
Michael Direction	3	0	37	6	0	6	4	0	10	39	8	3	224
Sarbet Direction	85	2	6	141	5	7	106	3	2	113	7	4	523
Total													1877

Date: June 20-22/2017

Time: 6:15pm - 6:30pm

L.Name	Left			Through			Right			U-Turn			total
	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	P.Car	Bus	Truck	pcu
Hana Direction	264	8	12	224	7	10	121	4	9	19	0	1	745
Gofa Direction	59	4	7	55	1	2	94	4	7	41	4	5	333
Michael Direction	159	9	8	103	0	6	105	0	12	34	1	2	495
Sarbet Direction	70	2	7	146	7	5	108	3	2	130	4	3	527
													2100

Appendix -3 Pedestrian volume data at Abune Petros Site

Date: June 20/2017

Peak 15minutes Pedestrians Volume of 1st day

Time	Merkato	Churchill	Minilik	Sebara Babur
8:00 - 8:15am	78	79	78	70
8:15am -8:30 am	87	77	87	65
8:30am - 8:45 am	69	91	79	79
8:45am -9:00 am	88	89	78	69
Sum	322	336	322	283
5:30 - 5:45 pm	84	76	60	76
5:45pm -6:00 pm	89	88	67	79
6:00 - 6:15 pm	78	75	59	65
6:15 – 30Ppm	59	89	87	73
Sum	310	328	273	293

Date: June 21/2017

Peak 15minutes Pedestrians Volume of 2nd day

Time	Merkato	Churcher	Minilik	Sebara Babur
8:00 - 8:15 am	88	98	78	97
8:15am -8:30 am	89	89	67	89
8:30 - 8:45 am	88	67	59	90
8:45am -9:00 am	80	78	79	69
5:30 - 5:45 pm	78	111	88	78
5:45pm -6:00 pm	80	89	70	80
6:00 - 6:15 pm	66	79	60	58
6:15 – 30 pm	77	96	65	56
Sum	301	375	283	272

Date: June 22/2017

Peak 15minutes Pedestrians Volume of 3rdr day

Time	Merkato	Churcher	Minilik	Sebara Babur
8:00 - 8:15 am	68	89	68	90
8:15am -8:30 am	80	87	59	89
8:30am - 8:45 am	67	89	67	68
8:45am -9:00 am	45	78	69	69
5:30 - 5:45 pm	79	77	67	76
5:45pm -6:00 pm	90	87	79	80
6:00 - 6:15 pm	79	69	67	61
6:00 - 6:15 pm	73	43	90	79
Sum	321	276	303	296

Date: June 20-22/2017

Average pedestrians per hour of each approach at Abune Petros Roundabout

Approaches	Merkato	Churcher	Minilik	Sebara Babur
8:00 - 8:15 am	78	89	75	86
8:15am -8:30 am	85	84	71	81
8:30am - 8:45 am	75	82	68	79
8:45am -9:00 am	71	82	75	69
5:30 - 5:45 pm	76	88	72	77
5:45pm -6:00 pm	86	88	72	80
6:00 - 6:15 pm	76	74	62	61
6:15 – 30 pm	70	76	81	69
PHVped/hr.	326	349	302	325

Abune Petros Roundabout Pedestrian movement condition

Table summary of pedestrians flow value used as input for software				
Leg Name	PHV(ped)	Vp15min	PHF	Flow rate (ped. /hr.)
Merkato	326	86	0.943	345
Churcher	349	89	0.984	355
Minilik	302	81	0.937	323
Sebara Babur	325	86	0.949	343

Appendix -4 Pedestrian volume data at German (Mekanisa) Site.

Date 27-92/07 /2017

Peak 15minutes Pedestrians Volume of 1st day

	Hana A	Gofa A	Sarbet A	Michael A
02:00 - 02:15am	99	130	150	110
02:15 - 02:30am	98	160	168	120
02:30 - 02:45am	78	157	100	98
02:45 - 03:00am	89	114	97	79
5:30- 5:45 pm	99	160	119	89
5:45- 6:00 pm	79	167	119	67
6:00- 6:15 pm	120	111	129	99
5:15- 5:30 pm	117	98	114	100
HPVped	394	644	566	429

Date 28-92/07 /2017

Peak 15minutes Pedestrians Volume of 1st day

	Hana A	Gofa A	Sarbet A	Michael A
02:00 - 02:15am	85	180	99	112
02:15 - 02:30am	89	89	111	100
02:30 - 02:45am	78	150	120	81
02:45 - 03:00am	80	178	100	45
5:30- 5:45 pm	90	155	123	156
5:45- 6:00 pm	120	167	129	187
6:00- 6:15 pm	116	190	120	113
5:15- 5:30 pm	113	110	145	99
HPVped	439	715	517	568

Date 29-92/07 /2017

Peak 15minutes Pedestrians Volume of 1st day

Counting Times	Hana A	Gofa A	Sarbet A	Michael A
02:00 - 02:15 am	99	143	98	78
02:15 - 02:30 am	78	164	93	69
02:30 - 02:45 am	112	160	97	59
02:45 - 03:00 am	96	154	89	67
5:30- 5:45 pm	143	156	130	99
5:45- 6:00 pm	132	157	140	98
6:00- 6:15 pm	121	155	132	56
5:15- 5:30 pm	143	166	128	76
HPVped	539	635	530	351

Average of three days 15min ped volume at German(Mekanisa)

Counting Times	Hana Approach	Gofa Approach	Sarbet Approach	Michael Approach
02:00 - 02:15am	94	151	116	100
02:15 - 02:30am	88	138	124	96
02:30 - 02:45am	89	143	106	79
02:45 - 03:00am	88	129	95	64
5:30- 5:45 pm	111	157	124	100
5:45- 6:00 pm	110	144	129	96
6:00- 6:15 pm	119	152	127	79
5:15- 5:30 pm	124	125	117	64
PHVped/hr.	464	604	504	393
Peak V15minutes	124	157	129	100

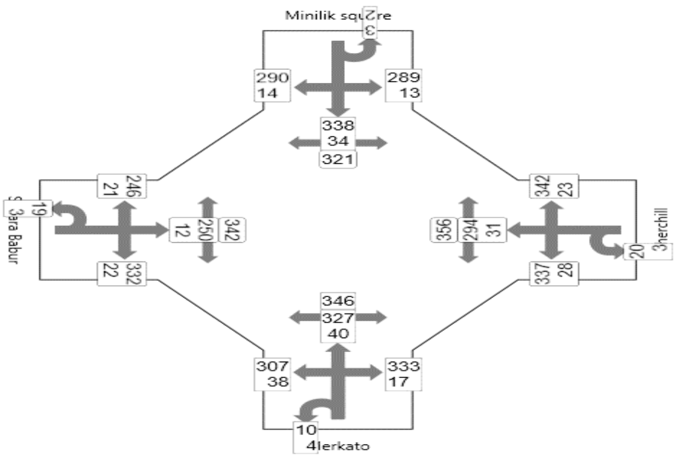
German Roundabout Pedestrian movement condition used for software

Leg Name	PHV(ped)	Vp15min	PHF	Flow rate (ped. /hr.)
Hana Approach	464	124	0.93	497
Gofa Approach	604	157	0.96	628
Sarbet Approach	504	129	0.97	517
Michael Approach	393	100	0.98	400

APPENDIX -B

SIDRA INTERSECTION OUTPUT

Appendix -5 Existing Light, Heavy vehicles and Pedestrian flow at Abune Petros Roundabout



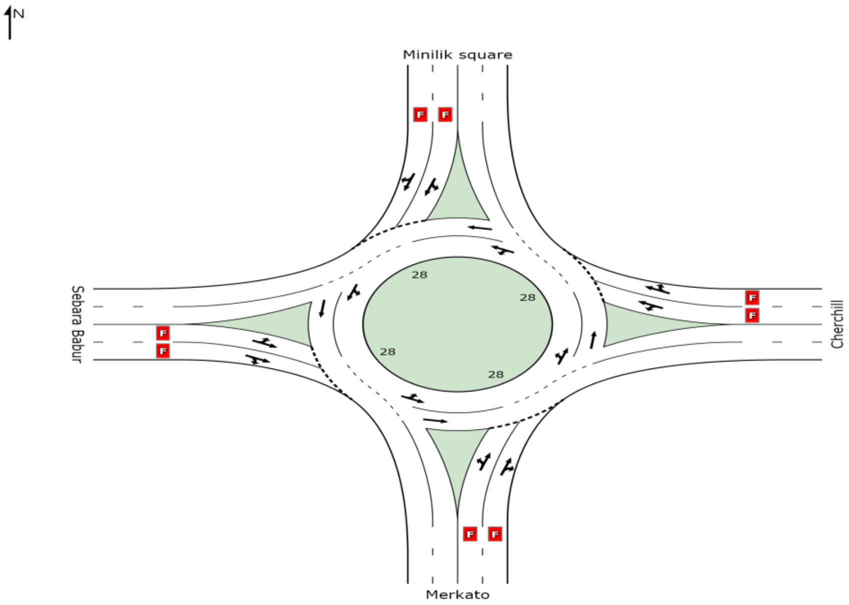
Appendix -6 Existing Movement Summary at Abune Petros Roundabout

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: Merkato											
1	L	359	11.6	1.219	146.3	LOS F	39.3	266.9	1.00	4.45	13.4
2	T	367	10.8	1.219	145.6	LOS F	41.0	266.9	1.00	4.42	13.4
3	R	350	4.8	1.219	144.9	LOS F	41.0	265.6	1.00	4.39	13.5
Approach		1076	9.1	1.219	145.6	LOS F	41.0	266.9	1.00	4.42	13.4
East: Churchill											
4	L	388	8.1	1.487	261.3	LOS F	63.7	420.0	1.00	5.93	9.4
5	T	324	9.4	1.487	261.1	LOS F	64.4	420.0	1.00	5.91	9.4
6	R	365	6.3	1.487	260.9	LOS F	64.4	419.4	1.00	5.90	9.4
Approach		1078	7.9	1.487	261.1	LOS F	64.4	420.0	1.00	5.91	9.4
North: Minilik square											
7	L	308	5.0	1.021	75.9	LOS F	19.8	127.6	1.00	2.59	18.2
8	T	373	9.1	1.021	75.9	LOS F	19.8	127.6	1.00	2.59	18.2
9	R	304	4.7	1.021	75.9	LOS F	19.8	127.6	1.00	2.59	18.2
Approach		985	6.5	1.021	75.9	LOS F	19.8	127.6	1.00	2.59	18.2
West: Sebara Babur											
10	L	290	8.5	1.409	232.7	LOS F	49.9	324.1	1.00	4.86	10.1
11	T	262	4.7	1.409	233.1	LOS F	49.9	324.1	1.00	5.34	10.1
12	R	354	6.3	1.409	233.7	LOS F	48.6	311.8	1.00	4.85	10.1
Approach		906	6.5	1.409	233.2	LOS F	49.9	324.1	1.00	4.99	10.1
All Vehicles		4045	7.6	1.487	179.0	LOS F	64.4	420.0	1.00	4.50	11.9

Appendix -7 Existing Lane Summary at Abune Petros Roundabout

	Demand Flows L	T	R	Total	HV	Cap.	Deg. Satn	Lane Util.	Average Delay	Level of Service	95% Back of Queue Vehicles	Distance	Lane Length
	veh/h	veh/h	veh/h	veh/h	%	veh/h	v/c	%	sec		veh	m	m
South: Merkato													
Lane 1	359	168	0	527	11.3	432	1.219	100	146.3	LOS F	39.3	266.9	600
Lane 2	0	199	350	549	6.9	450	1.219	100	144.9	LOS F	41.0	265.6	500
Approach	359	367	350	1076	9.1		1.219		145.6	LOS F	41.0	266.9	
East: Churchill													
Lane 1	388	148	0	536	8.5	361	1.487	100	261.3	LOS F	63.7	420.0	500
Lane 2	0	177	365	542	7.3	364	1.487	100	260.9	LOS F	64.4	419.4	500
Approach	388	324	365	1078	7.9		1.487		261.1	LOS F	64.4	420.0	
North: Minilik square													
Lane 1	308	184	0	492	6.6	482	1.021	100	75.9	LOS F	19.8	127.6	400
Lane 2	0	188	304	493	6.4	483	1.021	100	75.9	LOS F	19.8	127.6	400
Approach	308	373	304	985	6.5		1.021		75.9	LOS F	19.8	127.6	
West: Sebara Babur													
Lane 1	290	169	0	459	7.1	325	1.409	100	232.7	LOS F	49.9	324.1	800
Lane 2	0	93	354	448	6.0	318	1.409	100	233.7	LOS F	48.6	311.8	800
Approach	290	262	354	906	6.5		1.409		233.2	LOS F	49.9	324.1	
Intersection				4045	7.6		1.487		179.0	LOS F	64.4	420.0	

Appendix -8 Existing Level of Service Summary at Abune Petros Roundabout



Approaches	South	East	North	West	Intersection
LOS	F	F	F	F	F

Appendix -9 Movement Summary at Abune Petros Roundabout after re-analyzed

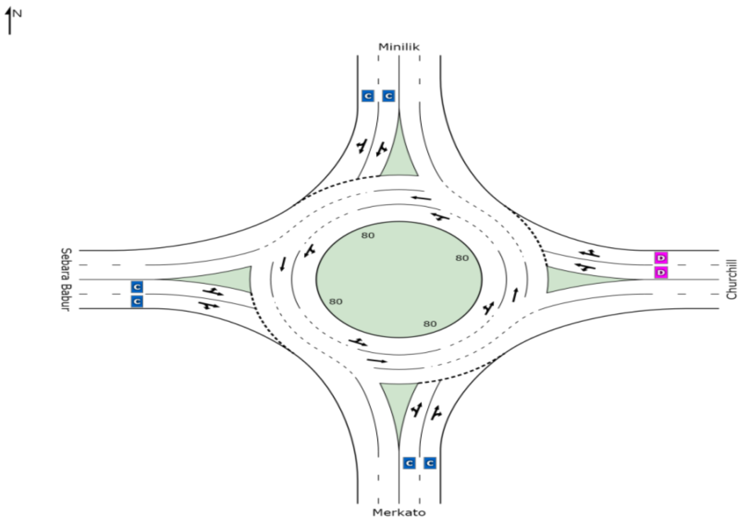
Movement Performance – Vehicles

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: Merkato											
1	L	359	11.6	0.781	25.5	LOS C	5.7	38.4	0.84	1.24	27.7
2	T	367	10.8	0.781	25.1	LOS C	6.0	38.8	0.85	1.19	26.5
3	R	350	4.8	0.781	24.8	LOS C	6.0	38.8	0.86	1.19	26.3
Approach		1076	9.1	0.781	25.1	LOS C	6.0	38.8	0.85	1.20	26.9
East: Churchill											
4	L	388	6.8	0.873	37.4	LOS D	8.1	52.7	0.90	1.50	24.7
5	T	324	9.4	0.873	37.4	LOS D	8.1	52.7	0.90	1.46	23.4
6	R	365	6.3	0.873	37.4	LOS D	8.1	52.7	0.90	1.46	23.5
Approach		1078	7.4	0.873	37.4	LOS D	8.1	52.7	0.90	1.48	24.0
North: Minilik											
7	L	308	5.0	0.810	30.4	LOS C	6.3	40.7	0.88	1.33	26.4
8	T	373	9.1	0.810	30.4	LOS C	6.3	40.7	0.88	1.29	25.4
9	R	304	4.7	0.810	30.4	LOS C	6.3	40.7	0.88	1.29	25.6
Approach		985	6.5	0.810	30.4	LOS C	6.3	40.7	0.88	1.30	25.8
West: Sebara Babur											
10	L	272	9.7	0.677	21.1	LOS C	3.9	25.8	0.83	1.12	29.2
11	T	262	4.7	0.677	21.0	LOS C	4.0	25.9	0.83	1.06	28.6
12	R	281	8.0	0.677	20.9	LOS C	4.0	25.9	0.83	1.06	28.9
Approach		815	7.5	0.677	21.0	LOS C	4.0	25.9	0.83	1.08	28.9
All Vehicles		3954	7.7	0.873	28.9	LOS C	8.1	52.7	0.87	1.28	26.1

Appendix -10 Lane Summary at Abune Petros Roundabout after re-analyzed

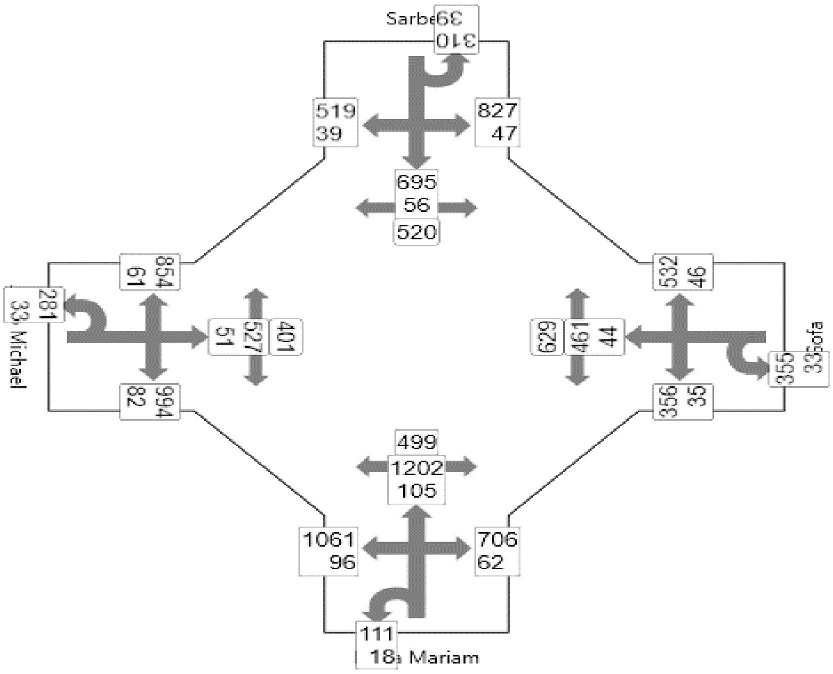
	Demand Flows L	Demand Flows T	Demand Flows R	Total	HV	Cap.	Deg. Satn	Lane Util.	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Lane Length
	veh/h	veh/h	veh/h	veh/h	%	veh/h	v/c	%	sec		veh	m	m
South: Merkato													
Lane 1	359	168	0	527	11.3	675	0.781	100	25.5	LOS C	5.7	38.4	500
Lane 2	0	199	350	549	6.9	703	0.781	100	24.8	LOS C	6.0	38.8	500
Approach	359	367	350	1076	9.1		0.781		25.1	LOS C	6.0	38.8	
East: Churchill													
Lane 1	388	150	0	539	7.5	617	0.873	100	37.4	LOS D	8.1	52.7	400
Lane 2	0	174	365	539	7.3	618	0.873	100	37.4	LOS D	8.1	52.7	400
Approach	388	324	365	1078	7.4		0.873		37.4	LOS D	8.1	52.7	
North: Minilik													
Lane 1	308	184	0	492	6.6	607	0.810	100	30.4	LOS C	6.3	40.7	300
Lane 2	0	188	304	493	6.4	608	0.810	100	30.4	LOS C	6.3	40.7	300
Approach	308	373	304	985	6.5		0.810		30.4	LOS C	6.3	40.7	
West: Sebara Babur													
Lane 1	272	133	0	406	8.1	599	0.677	100	21.1	LOS C	3.9	25.8	600
Lane 2	0	129	281	410	7.0	605	0.677	100	20.9	LOS C	4.0	25.9	600
Approach	272	262	281	815	7.5		0.677		21.0	LOS C	4.0	25.9	
Intersection				3954	7.7		0.873		28.9	LOS C	8.1	52.7	

Appendix -11 Level of Service Summary at Abune Petros Roundabout after re-analyzed



Approaches	South	East	North	West	Intersection
LOS	C	D	C	C	C

Appendix -12 Light, Heavy vehicles and Pedestrian flow at German (Mekanisa) Roundabout



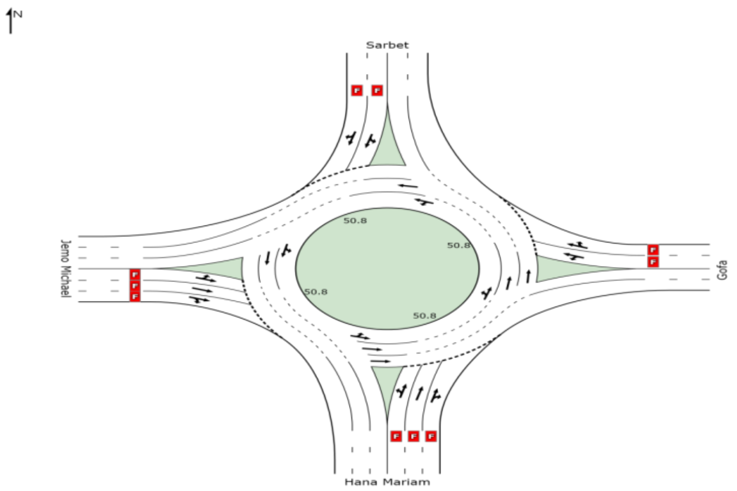
Appendix -13 Existing Movement Summary at German (Mekanisa) Roundabout

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: Hana Mariam											
1	L	1285	8.9	3.652	1222.4	LOS F	306.3	2027.7	1.00	8.81	2.5
2	T	1307	8.1	2.927	897.4	LOS F	225.8	1481.8	1.00	8.07	1.9
3	R	768	8.0	2.927	897.4	LOS F	225.8	1481.8	1.00	8.07	1.8
Approach		3361	8.4	3.652	1021.7	LOS F	306.3	2027.7	1.00	8.36	2.2
East: Gofa											
4	L	780	8.8	2.648	772.8	LOS F	190.1	1257.9	1.00	8.78	3.6
5	T	505	8.7	2.648	772.7	LOS F	191.2	1257.9	1.00	8.77	2.6
6	R	578	7.9	2.648	772.6	LOS F	191.2	1256.9	1.00	8.76	2.5
Approach		1864	8.5	2.648	772.7	LOS F	191.2	1257.9	1.00	8.77	3.0
North: Sarbet											
7	L	1224	7.1	2.125	528.4	LOS F	221.8	1441.1	1.00	9.92	5.8
8	T	751	7.4	2.125	528.4	LOS F	221.8	1441.4	1.00	9.92	5.3
9	R	558	7.0	2.125	528.4	LOS F	221.5	1441.4	1.00	9.92	7.4
Approach		2532	7.2	2.125	528.4	LOS F	221.8	1441.4	1.00	9.92	6.0
West: Jemo Michael											
10	L	1230	7.7	2.418	662.0	LOS F	237.6	1553.2	1.00	9.50	5.3
11	T	578	8.8	1.148	114.8	LOS F	33.3	220.6	1.00	3.36	14.6
12	R	1077	7.6	2.116	527.2	LOS F	188.5	1231.9	1.00	8.67	4.2
Approach		2884	7.9	2.418	502.1	LOS F	237.6	1553.2	1.00	7.96	5.4
All Vehicles		10641	8.0	3.652	719.8	LOS F	306.3	2027.7	1.00	8.69	3.7

Appendix -14 Existing Lane Summary at German (Mekanisa) Roundabout

	Demand Flows				HV	Cap.	Deg. Satn	Lane Util.	Average Delay	Level of Service	95% Back of Queue		Lane Length
	L	T	R	Total							Vehicles	Distance	
	veh/h	veh/h	veh/h	veh/h	%	veh/h	v/c	%	sec		veh	m	m
South: Hana Mariam													
Lane 1	1285	0	0	1285	8.9	352	3.652	100	1222.4	LOS F	306.3	2027.7	400
Lane 2	0	1038	0	1038	8.1	355	2.927	805	897.4	LOS F	225.8	1481.8	400
Lane 3	0	270	768	1038	8.0	355	2.927	805	897.4	LOS F	225.8	1481.8	400
Approach	1285	1307	768	3361	8.4		3.652		1021.7	LOS F	306.3	2027.7	
East: Gofa													
Lane 1	780	149	0	929	8.8	351	2.648	100	772.8	LOS F	190.1	1257.9	800
Lane 2	0	356	578	934	8.2	353	2.648	100	772.6	LOS F	191.2	1256.9	500
Approach	780	505	578	1864	8.5		2.648		772.7	LOS F	191.2	1257.9	
North: Sarbet													
Lane 1	1224	43	0	1267	7.1	596	2.125	100	528.4	LOS F	221.8	1441.1	400
Lane 2	0	707	558	1265	7.2	595	2.125	100	528.4	LOS F	221.5	1441.4	500
Approach	1224	751	558	2532	7.2		2.125		528.4	LOS F	221.8	1441.4	
West: Jemo Michael													
Lane 1	1230	0	0	1230	7.7	509	2.418	100	662.0	LOS F	237.6	1553.2	600
Lane 2	0	578	0	578	8.8	503	1.148	475	114.8	LOS F	33.3	220.6	500
Lane 3	0	0	1077	1077	7.6	509	2.116	875	527.2	LOS F	188.5	1231.9	500
Approach	1230	578	1077	2884	7.9		2.418		502.1	LOS F	237.6	1553.2	
Intersection				10641	8.0		3.652		719.8	LOS F	306.3	2027.7	

Appendix -15 Existing Level of Service Summary at German (Mekanisa) Roundabout



Approaches	South	East	North	West	Intersection
LOS	F	F	F	F	F

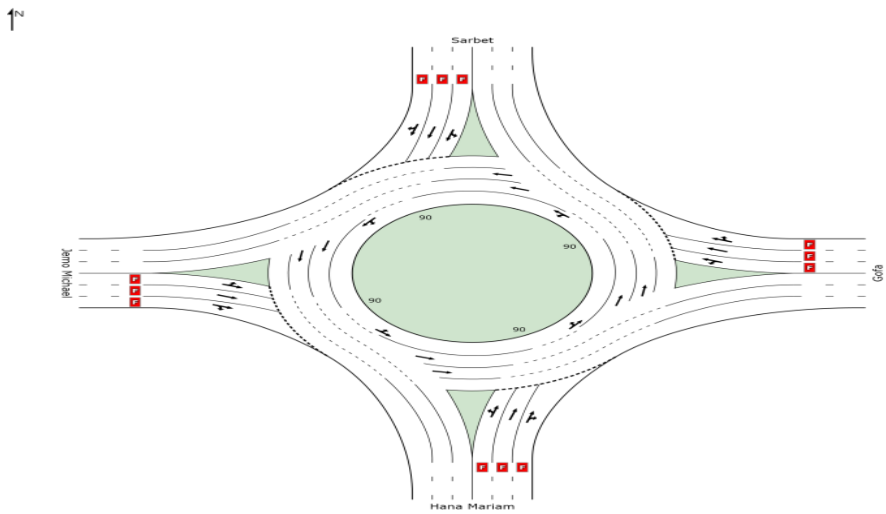
Appendix -16 Movement Summary at German (Mekanisa) Roundabout after re-analyzed

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: Hana Mariam											
1	L	1258	8.9	3.189	1012.5	LOS F	283.4	1875.8	1.00	9.29	3.0
2	T	1280	8.1	2.556	729.0	LOS F	204.7	1343.2	1.00	8.35	4.0
3	R	752	8.0	2.556	729.0	LOS F	204.7	1343.2	1.00	8.35	4.0
Approach		3291	8.4	3.189	837.4	LOS F	283.4	1875.8	1.00	8.71	3.6
East: Gofa											
4	L	780	8.8	2.244	593.5	LOS F	143.5	949.3	1.00	7.90	4.9
5	T	505	8.7	1.451	247.5	LOS F	55.6	367.1	1.00	5.01	10.1
6	R	578	7.9	1.650	331.7	LOS F	78.0	511.3	1.00	5.96	8.0
Approach		1864	8.5	2.244	418.5	LOS F	143.5	949.3	1.00	6.51	6.6
North: Sarbet											
7	L	1224	7.1	2.454	678.7	LOS F	238.6	1549.5	1.00	9.81	4.3
8	T	751	7.4	1.314	179.2	LOS F	57.9	376.1	1.00	4.82	12.8
8	R	558	7.0	1.314	179.2	LOS F	57.9	375.9	1.00	4.81	12.8
Approach		2532	7.2	2.454	420.6	LOS F	238.6	1549.5	1.00	7.23	6.6
West: Jemo Michael											
9	L	1230	7.7	2.740	808.7	LOS F	256.9	1679.5	1.00	9.16	3.7
10	T	578	8.8	1.301	177.6	LOS F	50.4	333.7	1.00	4.26	12.9
12	R	1077	7.6	2.398	655.6	LOS F	207.7	1357.2	1.00	8.51	4.4
Approach		2884	7.9	2.740	625.1	LOS F	256.9	1679.5	1.00	7.93	4.6
All Vehicles		10571	8.0	3.189	605.8	LOS F	283.4	1875.8	1.00	7.76	4.8

Appendix -17 Lane Summary at German (Mekanisa) Roundabout after re-analyzed

	Demand Flows				HV	Cap.	Deg. Satn	Lane Util.	Average Delay	Level of Service	95% Back of Queue		Lane Length
	L	T	R	Total							Vehicles	Distance	
	veh/h	veh/h	veh/h	veh/h	%	veh/h	v/c	%	sec		veh	m	m
South: Hana Mariam													
Lane 1	1258	0	0	1258	8.9	395	3.189	100	1012.5	LOS F	283.4	1875.8	400
Lane 2	0	1016	0	1016	8.1	397	2.556	805	729.0	LOS F	204.6	1343.2	500
Lane 3	0	264	752	1016	8.0	398	2.556	805	729.0	LOS F	204.7	1343.2	500
Approach	1258	1280	752	3291	8.4		3.189		837.4	LOS F	283.4	1875.8	
East: Gofa													
Lane 1	780	0	0	780	8.8	348	2.244	100	593.5	LOS F	143.5	949.3	800
Lane 2	0	505	0	505	8.7	348	1.451	655	247.5	LOS F	55.6	367.1	500
Lane 3	0	0	578	578	7.9	350	1.650	735	331.7	LOS F	78.0	511.3	500
Approach	780	505	578	1864	8.5		2.244		418.5	LOS F	143.5	949.3	
North: Sarbet													
Lane 1	1224	0	0	1224	7.1	499	2.454	100	678.7	LOS F	238.6	1549.5	400
Lane 2	0	653	0	653	7.4	497	1.314	545	179.3	LOS F	57.7	376.1	500
Lane 3	0	97	558	655	7.1	499	1.314	545	179.2	LOS F	57.9	375.9	500
Approach	1224	751	558	2532	7.2		2.454		420.6	LOS F	238.6	1549.5	
West: Jemo Michael													
Lane 1	1230	0	0	1230	7.7	449	2.740	100	808.7	LOS F	256.9	1679.5	600
Lane 2	0	578	0	578	8.8	444	1.301	475	177.6	LOS F	50.4	333.7	500
Lane 3	0	0	1077	1077	7.6	449	2.398	885	655.6	LOS F	207.7	1357.2	500
Approach	1230	578	1077	2884	7.9		2.740		625.1	LOS F	256.9	1679.5	
Intersection				10571	8.0		3.189		605.8	LOS F	283.4	1875.8	

Appendix -18 Level of Service Summary at German (Mekanisa) Roundabout after re-analyzed



Approaches	South	East	North	West	Intersection
LOS	F	F	F	F	F

Apendix-19 Comparison of two methods at Abune Petros Roundabout

Description	Manual method	SIDRA method
Degree of saturation	1.0417	1.49
Average control Delay	41.66	179.0
IntersectionQ95th (Veh)	15	64
LOS	E	F
Capacity	585 veh/hr	364 veh/hr.

Apendix-20: Comparison of existing and re-analyzed capacity & performance measures

Description	Existing	After Re-analyzed
Degree of Saturation	1.49	0.87
Avg. Delay (Sec/Veh.)	179	28
LOS	F	C
95Queue	64	8
Capacity (Veh.hr.)	364	469
Queue Storage	0.84	0.14

Apendix-21 comparison of existing and Re-analyzed at German site

Description	Existing	Re-analyzed
Degree of Saturation	3.65	3.19
Avg. Delay (Sec/Veh.)	719	605
LOS	F	F
95Queue	306	283
Capacity (Veh.hr.)	575	571