Addis Ababa University

Addis Ababa Institute of Technology University

School of Civil and Environmental Engineering

Optimal Traffic Signal Timing Allocation for Urban Congested Roundabouts

Case Study at Gerji Emperial, Bole Mikael and Saris Abo Roundabouts

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The Thesis Submitted To School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirement for the Degree of Master of Science in Civil Engineering (Road and Transport Engineering Stream)

Advisor

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MSc. Thesis on

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Declaration

I certify that this research work titled “Optimal Traffic Signal Timing Allocation for Urban Congested Roundabouts” is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where material has been used from other sources it has been properly acknowledged/ referred.

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Fourthly my gratitude goes to traffic program management office (TPMO) and traffic safety management division in providing me PTV VISTRO software and also supports me technically. Finally my thank goes to Addis Ababa Institute of Technology and Ethiopian Road Authority for this opportunity.

Abstract

Traffic congestion is a critical problem in Addis Ababa Ethiopia due to the fast growing economy, car ownership and population in the country. The road improvement practices also contribute to the reduction in roadway capacity which in turn to aggravates traffic congestion and delay on the study areas. Roundabouts are areas, where traffic congestion is severe.

The potential cause of traffic delay in these locations includes poor parking spaces and practice, compromised geometric design, absence of traffic signs and signals and pavement markings, absence of traffic police personnel and existence of aggressive driver's behavior. This research evaluates roundabout sections performance.

Study area roundabout are Gerji Emperial, Bole Mikael and Saris Abo. Those study areas were prone to traffic congestion due to excessive demand and poor roadway capacity. The study utilized spot speed study, traffic and pedestrian volume study and roundabout geometry study. Signal timing design and approaching saturation flow rate were computed by using HCM 2000. Excel spreadsheet was used to analyze the collected data. Three alternative mitigation measures were evaluated by using VISSIM and VISTRO simulation tools. Those alternatives are, Alternative 1- roundabout as it is, alternative 2 - changing roundabout into signalized intersection and alternative 3 - roundabout signalization.

Roundabouts have in excess of control delay and higher travel time relatively than those two alternative solutions. Alternative 2 and 3 reduce control delay by 34% and 47% from the previous under base and 52% and 69% under forecasted condition for peak hour traffic flow respectively. Thus the proposed engineering solutions enhance and improve selected roundabouts sections performance. Therefore these study findings states that study area roundabouts should be supported by traffic signal timing (roundabout signalization) or demolish roundabouts and changed to signalized intersection.

Keywords: traffic congestion, dead lock, signal timing, control delay, queue length.

Acronyms

AACRA	Addis Ababa City Roads Authority
CBD	Central Business District
ERA	Ethiopian Road Authority
FDT	Florida Department of Transportation
FHWA	Federal Highway Administration
HCM	Highway Capacity Manual
LOS	Level of service
MUTCD	Manual on Uniform Traffic Control Devices
TRB	Transportation Research Board
PHF	Peak-hour factor
PCU	Passenger Car Unit
TRRL	Transport Research Record Laboratory
VISSIM	"Verkehr In Städten - SIMulationsmodell" (German for "Traffic in cities - simulation model")
VISTRO	Traffic Engineering Tool

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Chapter 1: Introduction

1.1. Background

Traffic congestion is a critical problem in (Addis Ababa) Ethiopia due to the fast growing economy, car owner ship and population in the country. The road improvement practices also contribute to the reduction in road way capacity which in turns to aggravates traffic congestion and delay. Traffic congestion location in Addis Ababa includes straight and level road sections, roundabout sections, signalized intersections and un-signalized intersection. Therefore Evaluation of junction capacity is very important since it is directly related to delay, level of service, accident, operation cost and environmental issues.

Roundabout is an alternative form of intersection traffic control. Roundabouts are generally circular in shape, characterized by yield sign on entry and circulation around a central island. Roundabouts are appropriate for many intersections including locations experiencing high number of crashes, long traffic delays and approaches with relatively balanced traffic flows. Roundabouts have the potential to resolve various traffic flow problems. Traffic volume on one approach is significantly higher; it prevents vehicles at any other approach from entering.

But now a day in Ethiopia, Addis Ababa roundabouts are failed to entertain traffic volumes and even it is difficult to imagine driving on it during morning and afternoon peak hours without traffic-police regulating the traffic flow. Thus having this problem in mind, this research proposes and evaluates three alternative solutions for both base and future condition. Those alternatives are Roundabout as it is (alternative 1), changing roundabout in to pre-timed signalized intersection (alternative 2) and Roundabout signalization (alternative 3). Evaluation tool used for this task was VISTRO for alternative 1 and 2 and VISSIM simulation modeling for alternative 3.

Roundabout operates on the principle of giving priority, but during peak hour time, roundabout operation will be affected. Therefore this study accentuates in improving the performance of roundabout during peak hour time by allocating optimal traffic signal timing. During off-peak hour time and in a case of traffic signal timing mal-functioning the normal operation of roundabout continues.

1.2. Problem Statement

Most people know what congestion is and having their own definition on this phenomenon. However, when people need precise definitions of congestion, rapidly give way to descriptive terms (e.g. "stopped traffic") and causal explanations (e.g. "too much traffic"). In a broader sense of defining congestion it may be defined and measured as in terms of delay time experienced on the given road section or the speed you are obliged to use on that street due to dense amount of vehicles or the time duration you wait to stop-and-go movement or queue length and so on (ECMT, 2007).

In Addis Ababa, due to the intensive road network expansion (railway on other street which leads to diverted traffic), poor parking habits and lack of space (road capacity reduction), infrastructures, poor road way capacities (due to asphalt failure), mixed traffic volume (pedestrian and animals with vehicles along the road) and street venders leads traffic congestion to become the major threat in the cities economic growth by restraining the commutes mobility especially at peak hour**.** In addition, waiting time for the public transportation, both vehicles owner and public transport users are forced to delay within the congested traffic lane**.** This clearly leads to late arrival to working areas. Let arrival to working areas also its own adverse effect on the society comfort and economic development.

Apart from the existing problem in Addis Ababa and its adverse effect to the surrounding environment, There is still no universally accepted definition of what exactly "congestion" is. This situation is further complicated by the fact that congestion is as much a physical phenomenon that can be quantitatively described as a subjectively experienced situation that varies from person to person and from place to place (ECMT, 2007).

Study area Gerji Emperial, Bole Mikael and Saris Abo roundabout encounter enormous amount of traffic congestion during peak hour time. From field visit and site surveying data those roundabout didn't work properly during peak hour time. To solve traffic congestion along the selected roundabout, traffic police personals choose two basic techniques which are interventions and flow sequence arrangement. Problems encountered during peak hour time on selected roundabout areas are

Approaching leg congestion - longer queue traffic jam and deadlock (circle jam) - there is lengthy and prolonged traffic congestion on the approach legs beyoned there lane capacity (i.e. aggressively congested) and Deadlock traffic congestion**:** during peak hour time if there is no traffic polices around the study sections.

Alternative route selections (for right turn vehicles) - During peak hour time right turn vehicles find a way not to join roundabout flow, Instead they find an alternative route to escape this congestion.

Improved roundabout flow pattern shows one of the techniques adopted by traffic police to operate roundabout flow during peak hour time for the improvement of sections performance. This flow pattern minimizes delay, dissipate lengthy queue and dissipation speed is relatively higher, Even if the flow pattern is illegal.

Interventions - Bole Mikael roundabout experiences severe traffic congestion during peak period time. An intervention technique was applied on this study area. This intervention technique blocks six roundabout movements among all 16 possible movements.

Crashes - Most frequent accident type occurred on the study areas due to excessive traffic jam during peak hour is head-and-rear collision due to frequent stop-and-go traffic flow on the approaching and circulating lane. In terms of accident severity it is not bad (light crash), but as traffic flow it blocks traffic flow and creates bottle neck scenarios until it resolved by traffic police. Consequently it will create excessive platoon on respective approaches, and at this time traffic congestion has an additional causes.

Pedestrian impedance on roundabout capacity and level of service reduction- Significant pedestrian volume on roundabout crossings has noticeable influence on sections capacity and LOS. This situation was intensified at Saris Abo roundabout. Pedestrian volume count as per this study during morning and evening time indicates that pedestrian safety is in danger because of at grade crossing with vehicle.

Discouraged U-turn movement - According to traffic count survey of those three roundabout sections U-turn movement was discouraged due to the excessive traffic congestion in the area even if roundabout entertains U-turn movement. During peak hour time an average of 2.51% are U-turn vehicles out of 4289 veh/h. Most of the time vehicle from minor road prefers approaching U-turn movement; vehicle from major road prefers illegal U-turn movement (U-turn movement near Refugee Island).

1.3. Study Objective

The objective of this study is to provide practical guidelines for allocating optimal traffic signal timing to enhance roundabout efficiency under peak hour traffic flow condition.

1.3.1 Specific Objective

Specific objectives of this study are

- To compute roundabout saturation flow rate for each approaching legs based on HCM base saturation flow rate with adjustment factors.
- To allocate optimal traffic signal timing for study area roundabouts sections
- To simulate and evaluate alternatives performance (Control delay, Queue length and LOS) under both base and future condition and to draw conclusion and recommend best alternatives.

1.4. Study Limitations

Study limitations are directly affecting the output of this study. Those study limitations are as follow

- ✓ Pavement ideal condition - simulation delay result was minimum as compared to real condition due to the effect that simulation modeling assume ideal pavement condition good ridability no single distress at all.
- ✓ Difficulty in non-lane based simulation modeling - driving trend in Ethiopia is non-lane based changing lanes abruptly even in the area of no overtaking and changing lane is allowed (circulating lane and approaching legs).
- ✓ Impact of traffic accident - Head and rear crash accident type was common in the study area which interns to exaggerate delays and worst roundabout performance.
- ✓ Pedestrian impedance is not considered in VISSIM simulation – pedestrian crossings on approaching legs have an effect on roundabout performance.

1.5. Thesis Organization

This study consists of six chapters. The 1st chapter is an introduction, which includes background of the study, problem statement and objectives of the research. The 2nd chapter is about literature review, which states; roundabout and roundabout signalization and different categories of traffic simulation models and signal cycle components. The 3rd chapter is all about research methodology which includes defining of study area, way and methods of data collection and analysis. The 4th chapter deals with data analyses. 5th chapter includes result and discussion. Finally this study incorporates conclusion, study recommendation and future study.

Chapter 2: Literature Review

Traffic congestion is a critical problem in (Addis Ababa) Ethiopia due to the fast growing economy, car ownership and population in the country. The road improvement practices also contribute to the reduction in roadway capacity which in turn aggravates traffic congestion and delay. Locations of traffic congestion in Addis Ababa may include straight and level road sections, approaches, signalized intersections and un-signalized intersection

Congestion problem on roundabout drastically increases due to massive amount of approaching traffic volumes, the size of roundabouts and capacity of approaching road streets. This incurs too much delay and environment pollution, which exaggerates the discomfort of people leaving in this city. This intension comes in to postgraduate students day and night till they found an engineering solution to it. Therefore there were researches done previously on the problem of congestion on roundabouts, signalized and un-signalized intersection. Among them here are the basic reviews which relates with this studies.

2.1. Overview

2.1.1. Roundabout Definition

Roundabout is a circular intersection with a central island, yield control for entering traffic, channelized approaches, one way continuous flow within a circulatory road way and an appropriate geometric curvature to keep circulating speed low (FHWA,2000).

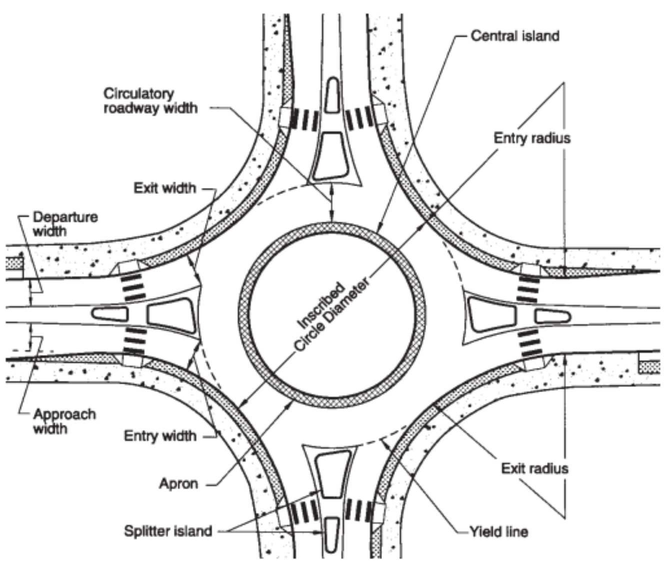


Figure 1 Basic Geometric Element of a Roundabout (FHWA, 2000)

1. Inscribed Circle Diameter

The inscribed circle diameter is the distance across the circle inscribed by the outer curb (or edge) of the circulatory roadway. As stated on Table 1Recommended Inscribed Circle Diameter Ranges (FHWA 2000. P 146)Table 1it is the sum of the central island diameter (which includes the apron, if present) and twice the circulatory roadway.

Table 1Recommended Inscribed Circle Diameter Ranges (FHWA 2000. P 146)

Site category	Typical design vehicle	Inscribed circle diameter
Mini roundabout	Single unit truck	13-25 m (45-80 ft)
Urban roundabout	Single-unit truck/bus	25-30 m (80-100 ft)
Urban single lane	WB-15 (WB-50)	30-40 m (100-130 ft)
Urban double lane	WB-15 (WB-50)	45-55 m (150-180 ft)
Rural single lane	WB-20 (WB-67)	35-40 m (115-130 ft)
Rural double lane	WB-20 (WB-67)	55-60 m (180-200 ft)

* Assume 90-degree angle between entries and no more than four legs

2. Truck Apron

Truck apron is located at the circumference of central island of the given roundabout which is used to give a riding way for large vehicles rear wheel when a vehicle try to turn around the circle.

3. Exit Radius (Exit Curve)

The horizontal curve of the departure carriageway which leads vehicles out of the circulating carriageway (AACRA, 2003)

4. Entry Radius (Entry Curve)

The horizontal curve of the approach carriage way which leads vehicles into the circulating carriageway (AACRA, 2003)

5. Yield Line (Broken Line)

A broken line, marked across the approach carriageway where it meets the circulating carriageway and at which vehicles should wait if necessary for an acceptable gap in traffic to enter the circulating carriageway. (AACRA, 2003)

6. Circulatory Road Width

The required width of the circulatory roadway is determined from the width of the entries and the turning requirements of the design vehicle. In general, it should always be at least as wide as the maximum entry width (up to 120 percent of the maximum entry width) and should remain constant throughout the roundabout (Tan J, 2001).

7. Departure Width

The one-way width of the carriageway on the departure from the roundabout. (AACRA, 2003)

8. Approach Width

The one-way width of the carriageway on the approach to the roundabout. (AACRA, 2003)

9. Entry Width

As shown in fig 2.1 entry widths is measured from the point where the yield line intersects the left edge of the traveled-way to the right edge of the traveled-way, along a line perpendicular to the right curb line. The width of each entry is dictated by the needs of the entering traffic stream. It is based on design traffic volumes and can be determined in terms of the number of entry lanes. The circulatory roadway must be at least as wide as the widest entry and must maintain a constant width throughout (Entry width is the largest determinant of a roundabout's capacity)

10. Exit Width

The exit width is the perpendicular distance from the right curb line of the exit to the intersection of the left edge line and the inscribed circle.

11. Splitter Islands

Splitter islands also called separator islands or median islands should be provided on all roundabouts, except those with very small diameters at which the splitter island would obstruct the visibility of the central island. Their purpose is to provide shelter for pedestrians (including wheelchairs, bicycles, and baby strollers), assist in controlling speeds, guide traffic into the roundabout, physically separate entering and exiting traffic streams, and deter wrong-way movements. Additionally, splitter islands can be used as a place for mounting signs

2.1.2. Roundabout Signalization

The conflicting traffic streams in a roundabout are separated in time by giving priority rules (i.e. the entry vehicles should give ways to circulating vehicles). Signalized junction, on the other hand, is a junction with traffic lights where the conflicts are separated in time by the traffic lights (Tan J, 2001). Roundabouts have considerable advantages in respect of their safety record, specifically for 4wheel vehicles, handling considerable volumes of turning traffic and even dealing well with U-turn traffic and providing minimum off-peak delay. Roundabouts improve vehicle safety by reducing the number of possible conflict points as illustrated Figure 2, reducing speed differentials at intersections and forcing drivers to decrease speeds as they proceed. Accident data have confirmed this statement and shown a reduction in severe crashes (FHWA, 2000)

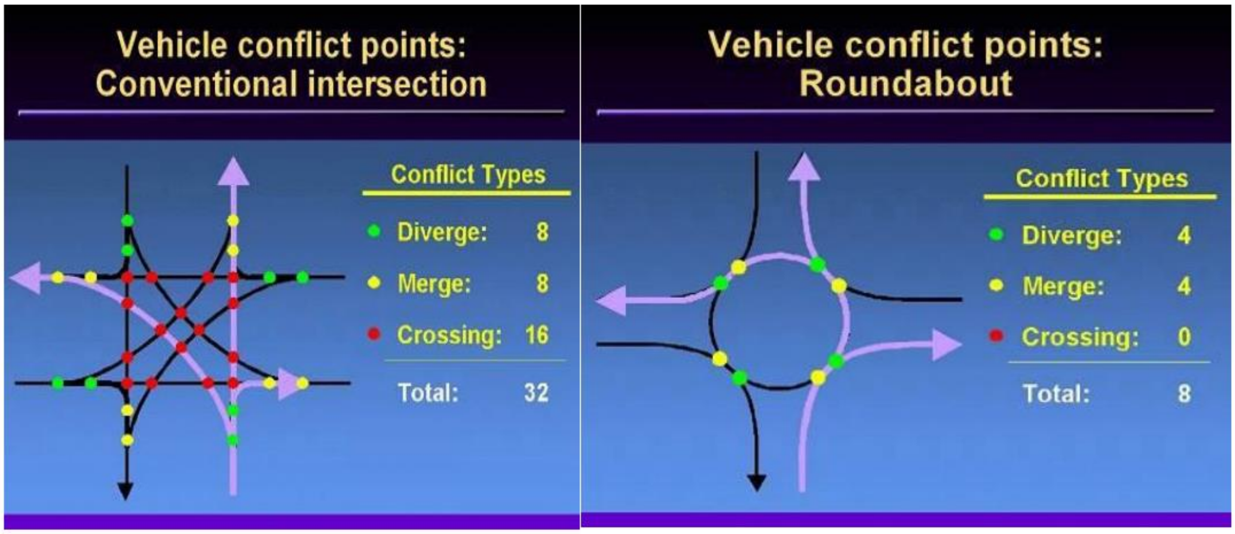


Figure 2 Vehicles Conflict Point (Mallikarjuna, 2014)

Roundabouts are particularly suitable for handling turning traffic. This is because of the considerable space available for the turning traffic and because all traffic is constrained to run parallel and can therefore weave instead of cutting across when making right turns. There is only limited delay to travel through a roundabout, particularly if it is not too large. As opposed to going straight across and there is no need to wait unless there is a preceding traffic with the right of way (Huddart, K, 1983). Roundabouts can efficiently handle particular intersections with decreased delay and greater efficiency than traffic signals. This is especially true where traffic volumes entering the roundabout are roughly similar and where there are a high number of left turning vehicles (FDT 2008). On the contrary, signalization inevitably delays low traffic flows arriving at the junction when signals are at red. Also, the need to provide a minimum green time to each movement in every cycle creates time intervals in which no vehicles are entering the intersection is a main delay cause. However, this can be greatly improved today by taking advantage of actuated control. Moreover, the lost time associated with start-up and termination of a green phase detracts further from the amount of time that is available for moving traffic is also another reason for time delay in signalized intersections (FDT, 2008).

When there is significantly higher traffic volume, roundabout operations creates more delay, higher frequency of head and rear collision, longer travel time and extremely slow travel speed, irregular (weaving) flow. Therefore this study uses the combined effect of those two important intersection flow sciences during peak hour time.

(I) Components of a Signal Cycle

The following terms describe portions and sub portions of a signal cycle. The most fundamental unit in signal design and timing is the cycle, as defined below (Roger p. et al, 2004).

1) **Cycle.** A signal cycle is one complete rotation through all of the indications provided. In general. Every legal vehicular movement receives a "**green**" indication during each cycle, although there are some exceptions to this rule.
2) **Cycle Length** the cycle length is the time (in seconds) that it takes to complete one full cycle of indications. It is given the symbol "**C**."
3) **Interval**. The interval is a period of time during which no signal indication changes. It is the smallest unit of time described within a signal cycle. There are several types of intervals with in a signal cycle: (refer Roger p. et al, 2004, p 515)

(II) Advantages of Traffic Signal Control

The MUTCD lists the following advantages of traffic control signals that are "properly designed, located, operated and maintained" (MUTCD. 2001). These advantages include: They provide for the orderly movement of traffic, They increase the traffic handling capacity of the intersection if proper physical layouts and control measures are used and if the signal timing is reviewed and updated on a regular basis (every two years) to ensure that it satisfies the current traffic demands, They reduce the frequency and severity of certain types of crashes, especially right-angle collisions, They are coordinated to provide for continuous or nearly continuous movement at a definite speed along a given route under favorable conditions, They are used to interrupt heavy traffic at intervals to permit other traffic, vehicular or pedestrian, to cross. These specific advantages address the primary reasons why a traffic signal would be installed: to increase capacity (thereby improving level of service), to improve safety, and to provide for orderly movement through a complex situation. Coordination of signals provides other benefits, but not all signals are necessarily coordinated and Signal timing can reduce head and rear collision do to stop and go movement during peak hour.

(III) Disadvantages of Improperly Design Traffic Signal Control

The description of the second advantage in the foregoing list indicates that capacity is increased by a well-designed signal at a well-designed intersection. Poor design of either the signalization or the geometry of the intersection can significantly reduce the benefits achieved or negate them entirely. Improperly design & traffic signals, or the placement of a signal where it is not justified, can lead to some of the following disadvantages (MUTCD. 2001). Those disadvantages are Excessive delay, Excessive disobedience of the signal indications, Increased use of less adequate routes as road users attempt to avoid the traffic control signal and Significant increases in the frequency of collisions (especially rear-end collisions)

2.2. Performance Measures

There are four measures which are used to describe the performance of two-way stop-controlled intersections. These are

2.2.1. Delay

A critical performance measure on interrupted flow facilities is delay. There are several types of delay, but this study uses control delay as the principal service measure in evaluating level of service at signalized and un-signalized intersections. Although the definition of control delay is consistent among signalized and un-signalized intersections, its application, including LOS threshold values, differs for these facilities.

Control Delay involves movements at slower speeds and stops on intersection approaches, as vehicles move up in the queue or slow down upstream of an intersection Drivers frequently reduce speed when a downstream signal is red or a queue is present at the downstream intersection approach. Control delay requires the determination of a realistic average speed for each roadway segment. It is implied in the estimates of the average travel speed on urban streets. (HCM, 2000)

Roundabouts can increase delays in locations where traffic would otherwise often not be required to stop. For example, at the junction of a high-volume and a low-volume road, traffic on the busier road would stop only when cross traffic was present, otherwise not having to slow for the roundabout. When the volumes on the roadways are relatively equal, a roundabout can reduce delays, because half of the time a full stop would be required. Dedicated left turn signals (in countries where traffic drives on the right) further reduce throughput.

2.2.2. Level of Service (LOS)

Level of service is a qualitative measure used to relate the quality of traffic service. LOS is used to analyze highway by categorizing traffic flow and assigning quality levels of traffic based on performance measure.

The HCM defines LOS for signalized and un-signalized intersections as a function of the average vehicle control delay is stated on Table 2. LOS may be calculated per movement or per approach for any intersection configuration but LOS for the intersection as a whole is only defined for signalized and all way stop configuration.

Table 2 Level of Service Criteria (HCM, 2010)

LOS	Signalized intersection	Un signalized intersection
A	≤ 10 sec	≤ 10 sec
B	10-20 sec	10-25 sec
C	20-35 sec	15-25 sec
D	35-55 sec	25-35 sec
E	55-80 sec	35-50 sec
F	>80 sec	>50 sec

Roundabouts can reduce delays for pedestrians compared to traffic signals, because pedestrians are able to cross during any safe gap rather than waiting for a signal. During peak flows when large gaps are infrequent, the slower speed of traffic entering and exiting can still allow crossing, despite the smaller gaps.

The primary measure which is used to provide an estimate of level of service (LOS) is control delay. This measure can be estimated for any movement on the minor (i.e., the stop-controlled) street. By summing delay estimates for individual movements, a delay estimate for each minor street movement and approach can be achieved. (HCM, 2010)

2.2.3. Travel Time

Travel time is the time taken by a vehicle to traverse a given section of a highway. A travel time study determines the amount of time required to travel from one point to another on a given route. In conducting such a study, information may also be collected on the locations, durations, and causes of delays. When this is done, the study is known as a travel time and delay study. Data obtained from travel time and delay studies give a good indication of the level of service on the study section. These data also aid the traffic engineer in identifying problem locations, which may require special attention in order to improve the overall flow of traffic on the route (Nicholas J. Garber and Lester A. Hoel 2009).

2.2.4. Queue

When demand exceeds capacity for a period of time or arrival time headway is less than the service time at a specific location, a queue is formed. A queue is also formed when arrivals wait at a service area for service. This service can be the arrival of an accepted gap in a major street traffic stream, the collection of tolls at a tollbooth, the payment of parking fees at a parking garage and so forth.

The term back of queue refers to the number of vehicles that are queued at an approach of a signalized intersection depending on the arrival patterns of vehicles during the red phase and vehicles that do not clear the intersection during a given green phase. To mathematically predict the characteristics of a queuing system, it is necessary to specify the following system characteristics and parameters (Tan J, 2001).

- ✓ Arrival pattern characteristics including average rate of arrival and the statistical distribution of time between arrivals,
- ✓ Service facility characteristics including service time average rates and distribution and the number of customers that can be served simultaneously or number of channels available and
- ✓ Queue discipline characteristics, such as the means by which the next customer to be served is selected.

Oversaturated queues are those in which the arrival rate is higher than the service rate and under saturated queues are those in which the arrival rate is less than the service rate. The length of an under saturated queue may vary but will reach a steady state with the arrival of vehicles. The length of an oversaturated queue will, however, never reach a steady state but will continue to increase with the arrival of vehicles.

2.2.5. Roundabout Capacity

The capacity of a roundabout varies based on entry angle, lane width and the number of entry and circulating lanes. As with other types of junctions, operational performance depends heavily on the flow volumes from various approaches. A single-lane roundabout can handle approximately 20,000–26,000 vehicles per day, while a two-lane design supports 40,000 to 50,000. (FHWA, 2000).

Under many traffic conditions, a roundabout operates with less delay than signalized or all-way stop approaches. Roundabouts do not stop all entering vehicles, reducing both individual and queuing delays. Throughput further improves because drivers proceed when traffic is clear without waiting for a signal to change.

Roundabouts can increase delays in locations where traffic would otherwise often not be required to stop. For example, at the junction of a high-volume and a low-volume road, traffic on the busier road would stop only when cross traffic was present, otherwise not having to slow for the roundabout. When the volumes on the roadways are relatively equal, a roundabout can reduce delays, because half of the time a full stop would be required. Dedicated left turn signals (in countries where traffic drives on the right) further reduce throughput.

Roundabouts can reduce delays for pedestrians compared to traffic signals, because pedestrians are able to cross during any safe gap rather than waiting for a signal. During peak flows when large gaps are infrequent, the slower speed of traffic entering and exiting can still allow crossing, despite the smaller gaps.

Studies of roundabouts that replaced stop signs and/or traffic signals found that vehicle delays were reduced 13% –89 % and the proportion of vehicles that stopped was reduced 14 % – 56% percent. Delays on major approaches increased as vehicles slowed to enter the roundabouts (IIHSHLDI, 2016).

Roundabouts have been found to reduce carbon monoxide emissions by 15% up to 45% percent, nitrous oxide emissions by 21% up to 44%, carbon dioxide emissions by 23% up to 37% percent and hydrocarbon emissions by 0% – 42% percent. Fuel consumption was reduced by an estimated 23% up to 34% percent (IIHSHLDI, 2016).

2.3. Saturation Flow Rate

Saturation flow rate is defined as the flow rate per lane at which vehicles can pass though. The saturation flow rate represents the number of vehicles per hour per lane that can pass through a signalized intersection if the green signal were available for the full hour, the flow of vehicles were never halted, and no large headways occurred at signalized intersection (HCM 2000).

2.3.1 Base Saturation Flow Rate

Base saturation flow rate is affected by approaching speed. Approaches with lower approaching speed (less than 50km/h) use 1800pc/h/ln and approaches with higher approaching speed use (greater than 80km/h) use higher than 1900pc/h/ln. (HCM, 2000)

Saturation flow rate for signalized intersection is computed using head difference and but for roundabout is very complicated to do an experiment (head difference due to stop and go movement). No single study in Ethiopia was conducted to have roundabout signalization and to determine base saturation flow rate.

Computations begin with the selection of a base saturation flow rate, usually 1,800 passenger cars per hour per lane (pc/h/ln) and adjust of base saturation flow rate were done for variety of existing conditions. Those adjustments are Number of lanes in the lane group, Adjustment factor for lane width (3.6m is base condition), Adjustment factor for heavy vehicles in the traffic stream, Adjustment factor for approach grade, Adjustment factor for the existence of parking lane and parking activity adjacent to the lane group, Adjustment factor for blocking effect of local buses that stop within the intersection area, Adjustment factor for area type, Adjustment factor for lane utilization, Adjustment factor for left-turn in the lane group, Adjustment factor for right-turn in the lane group, Pedestrian adjustment factor for left-turn movements and Pedestrian/bicycle adjustment factor for right-turn movements.

2.4. Spot Speed Studies

Speed is an important transportation consideration because it relates to safety, time, comfort, convenience and economics. Spot speed studies are used to determine the speed distribution of a traffic stream at a specific location. The data gathered in spot speed studies are used to determine vehicle speed percentiles, which are useful in making many speed related decisions. Spot speed data have a number of safety applications, including the following (Robertson, 1994):

1. Determining existing traffic operations and evaluation of traffic control devices
   a) Determining the 15th, 50^{th,} and 85th speed percentiles

b) Evaluating and determining proper advisory speeds

c) Determining the proper placements of traffic control signs and markings

d) Setting appropriate traffic signal timing

2. Assessing roadway safety questions

a) Evaluating and verifying speeding problems

b) Assessing speed as a contributor to vehicle crashes

3. Monitoring and studying traffic speed trends by systematic ongoing speed studies

4. Measuring effectiveness of traffic control devices or traffic programs, including signs and markings, traffic operational changes and speed enforcement programs

2.4.1. Traffic Spot Speed Study Sample Size

For a spot speed study at a selected location, a sample size of at least 50 and preferably 100 vehicles is usually obtained (Ewing, 1999). Traffic counts during a Monday morning or a Friday peak period may show exceptionally high volumes and are not normally used in the analysis; therefore, counts are usually conducted on a Tuesday, Wednesday and Thursday. Spot speed data are gathered using one of three methods:

(1) Stopwatch method

(2) Radar meter method

(3) Pneumatic road tube method.

2.4.2. Speed Percentiles

Speed percentiles are tools used to determine effective and adequate speed limits. The three speed percentiles used on this study are 15th, 50th and the 85th percentiles.

The 15th percentile is the speed at which 15% of the observed vehicles are traveling at or below. This percentile is used in calculating all red clearance intervals (ar).

The 50th percentile is the median speed of the observed data set. This percentile represents the speed at which half of the observed vehicles are below and half of the observed vehicles are above. The 50th percentile of speed represents the average speed of the traffic stream.

The 85th percentile is the speed at which 85% of the observed vehicles are traveling at or below. This percentile is used in calculating yellow interval and evaluating/recommending posted speed limits based on the assumption that 85% of the drivers are traveling at a speed they perceive to be safe (Homburger W.S et al. 1996). In other words, the 85th percentile of speed is normally assumed to be the highest safe speed for a roadway section.

2.4.3. Stopwatch Method

The stopwatch method can be used to successfully complete a spot speed study using a small sample size taken over a relatively short period of time.

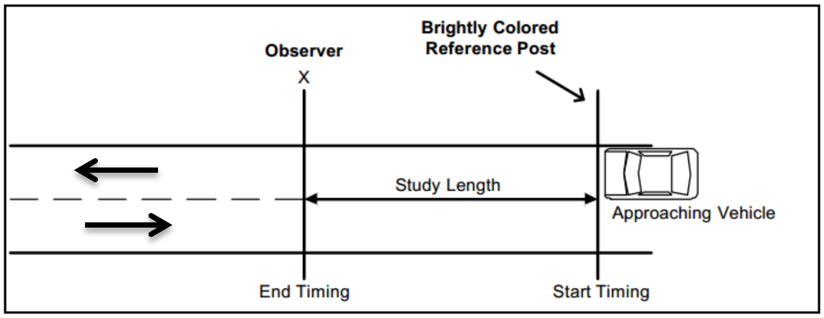


Figure 3 Spot Speed Study Length

2.5. Traffic Volume Studies

Traffic volume study is conducted to determine the number, movements and classifications of roadway vehicles at a given location.

These data can help

- To identify critical flow time periods (peak hour),
- To determine the influence of large vehicles or pedestrians on vehicular traffic flow, or
- To document traffic volume trends.

Two methods are available for conducting traffic volume counts:

1) Manual Counting Method

Manual counts are typically used to collect data for determination of vehicle classification, turning movements, direction of travel, pedestrian movements, or vehicle occupancy. Manual count are typically used for period of less than a day, normal intervals for a manual counts are 5, 10 and 15 minutes. Traffic count during Monday morning rush hour and Friday evening rush hour may show exceptionally high volumes and are not normally used in analysis. Therefore count are usually conducted on Tuesday, Wednesday or Thursday (traffic study, 2002).

2) Automatic Counting Method.

Automatic counts are typically used to gather data for determination of vehicle hourly patterns, daily or seasonal variations and growth trends, or annual traffic estimates. The selection of study method should be determined using the count period. The count period should be representative of the time of day, day of month, and month of year for the study area. The count period should avoid special event or compromising weather conditions (Sharma S, 1994). Count periods may range from 5 minutes to 1 year.

Typical count periods are 15 minutes or 2 hours for peak periods, 4 hours for morning and afternoon peaks, 6 hours for morning, midday (4hr), and afternoon peaks (2hr), and 12 hours for day time periods (Robertson H, 1994).

The Pedestrian Volume Count Study is used to determine the volume of pedestrians crossing the streets at signalized or non-signalized intersections. Pedestrian count data are usually or frequently used in planning applications.

Pedestrian count data are used to evaluate

2.6. Traffic Simulation Models

Simulation model descriptors are Stochastic and Deterministic Models; Event-Based and Time-Based Models; Micro-, Macro-, and Mesoscopic Models; Static Flow and Time-Varying Flow Models; Descriptive and Normative Models and Off-Line and Real-Time Models. Among those descriptors this study uses microscopic simulation model called VISSIM, traffic simulation in city model. (HCM, 2000)

2.6.1) Micro-, Macro- and Mesoscopic Models

Modelers often try to describe simulation models as being microscopic, mesoscopic or macroscopic. The difference pertains mainly to the level at which the traffic flow phenomena are being represented.

Microscopic models capture the movement of every vehicle. Individual vehicles can be traced through the network and their time-space trajectories can be plotted. The model contains processing logic that describes how the vehicles behave. This behavior includes acceleration, deceleration, lane changes, passing maneuvers, turning movement execution and gap acceptance.

Macroscopic Models are at the other end of the spectrum. They tend to employ flow rate variables and other general descriptors of how the traffic is moving. The flow rate within one segment of the freeway is related to upstream and downstream flow rates through conservation of flow equations and other equations that ensure that boundary conditions are met at the interface between system segments.

Mesoscopic Models fall between microscopic and macroscopic models. They typically model the movement of clusters or platoons of vehicles and incorporate equations that indicate how these clusters of vehicles interact. Simulation models of signalized networks are often designed this way, since the vehicles tend to move in platoons that interact with other platoons and exhibit predictable changes in character over time and distance, as with platoon dispersion.

VISSIM is the leading microscopic simulation program for modeling multimodal transport operations and belongs to the Vision Traffic Suite software. Realistic and accurate in every detail, VISSIM creates the best conditions to test different traffic scenarios before their realization. (PTV-VISSIM manual, 2015)

PTV VISTRO Calculate Intersection Level of Service for signals, two-way stops, all-way stops, and roundabouts using industry standard methodologies, including HCM 2010, HCM 2000, Intersection Capacity Utilization (ICU) and Kimber methods.

Optimize Signal Timing for individual intersections, routes and networks using robust optimization techniques within user defined timing parameters

Test Mitigation Options for failing intersections and compare the various options to each other and the base network

Visualize Results on your network or as graphical output for various volume levels, volume balancing results, LOS, and optimization

Obtain Standardized Report-Ready Tables and Figures in one easy step to insert directly into any report to meet agency requirements

Analyze Queues and Spillbacks via quick export to VISSIM for micro simulation and Evaluate Need for a New Traffic Signal through the built-in MUTCD Signal Warrants Analysis.

2.7. Gap and Gap Acceptance

Important aspect of traffic flow is the interaction of vehicles as they join, leave or cross a traffic stream includes ramp vehicles merging onto an expressway stream, freeway vehicles leaving the freeway onto frontage roads and the changing of lanes by vehicles on a multilane highway (Garber and Hoel, 2009)

The most important factor a driver considers in making any one of these maneuvers is the availability of a gap between two vehicles that, in the driver's judgment, is adequate for him or her to complete the maneuver. The evaluation of available gaps and the decision to carry out a specific maneuver within a particular gap are inherent in the concept of gap acceptance. Following are the important measures that involve the concept of gap acceptance:

1. Merging is the process by which a vehicle in one traffic stream joins another traffic stream moving in the same direction, such as a ramp vehicle joining a freeway stream.

2. Diverging is the process by which a vehicle in a traffic stream leaves that traffic stream, such as a vehicle leaving the outside lane of an expressway.

3. **Weaving** is the process by which a vehicle first merges into a stream of traffic, obliquely crosses that stream, and then merges into a second stream moving in the same direction; for example, the maneuver required for a ramp vehicle to join the far side stream of flow on an expressway.

4. **Gap** is the headway in a major stream, which is evaluated by a vehicle driver in a minor stream who wishes to merge into the major stream. It is expressed either in units of time (time gap) or in units of distance (space gap).

5. **Time Lag** is the difference between the time a vehicle that merges into a main traffic stream reaches a point on the highway in the area of merge and the time a vehicle in the main stream reaches the same point.

6. **Space Lag** is the difference, at an instant of time, between the distances a merging vehicle is away from a reference point in the area of merge and the distance a vehicle in the main stream is away from the same point. Fig 2.4 depicts the time-distance relationships for a vehicle at a stop sign waiting to merge and for vehicles on the near lane of the main traffic stream.

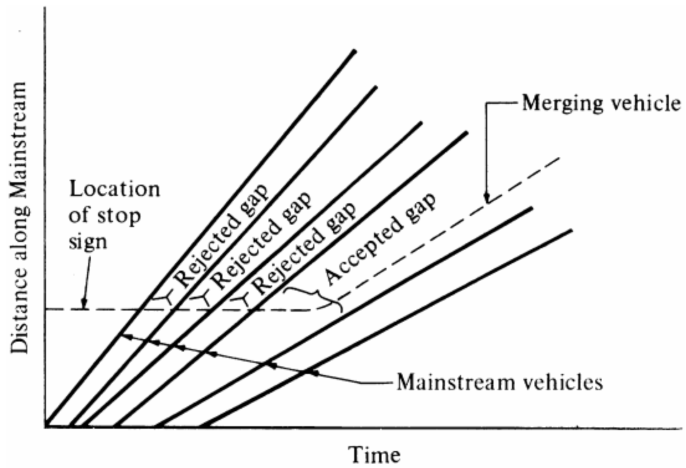


Figure 4 Time-Space Diagrams for Vehicles in the Vicinity of a Stop Sign
(Garber and Hoel, 2009)

2.8. Literature Summary

According to HCM 2000 simulation model descriptors are Stochastic and Deterministic Models; Event-Based and Time-Based Models; Micro-, Macro-, and Mesoscopic Models; Static Flow and Time-Varying Flow Models; Descriptive and Normative Models and Off-Line and Real-Time Models. Among those descriptors this study uses microscopic simulation model called VISSIM, traffic simulation in city model.

Microscopic models capture the movement of every vehicle. Individual vehicles can be traced through the network and their time-space trajectories can be plotted. The model contains processing logic that describes how the vehicles behave. This behavior includes acceleration, deceleration, lane changes, passing maneuvers, turning movement execution and gap acceptance.

Default value of saturation flow rate may increases or decreases depend on local field measurements. Approaches with less than 50km/h approaching speed uses 1800 pc/h/ln greater than uses 1900pc/h/ln.

Roundabouts have been found to reduce carbon monoxide emissions by 15% up to 45%, nitrous oxide emissions by 21% up to 44%, carbon dioxide emissions by 23% up to 37% percent and hydrocarbon emissions by 0% up to 42% percent. Fuel consumption was reduced by an estimated 23% up to 34% percent.

Chapter 3: Methodology

This topic states methodology detailing includes defining study area; way of data collection and types of data collected and methods used for data analysis and evaluation.

3.1. Study Area

This research study area is located at Addis Ababa which is the capital city of Ethiopia. Since Addis Ababa is the capital city of Ethiopia the population growth rate and socio-economic development is highest relative to other regions. This growth rate has direct effect on traffic volume and it is major way cause of traffic congestion on different intersections, Specially on roundabouts. This study evaluates three roundabout areas which expose to excessive amount of traffic volume and lengthy queue. Those study areas are Gerji Emperial roundabout, Bole Mikael roundabout and Saris Abo roundabout.

3.1.1. Gerji Emperial Roundabout

Gerji Emperial roundabout is located at Bole sub city around Gerji Emperial hotel. This study section has 4 legs, two of it legs are major roads directly linked to ring road and the other 2 legs are minor roads. Four approaching legs are indicated in Figure 5, which are Bole leg, Megenagna leg, 24 street leg and Gerji leg. This roundabout has 3 circulating lanes, 22m circular island diameter, 3 lanes for each major road and 2 lanes for each minor road.

Figure 5 Gerji Emperial Roundabout

3.1.2. Bole Mikael Roundabout

Bole Mikael roundabout is located at Bole sub city around Bole Mikael church. This study section has 4 legs, two of it legs are major roads directly linked to ring road and the other 2 legs are minor roads. Four approaching legs are indicated in Figure 6, which are Bole leg, Kadisco leg, Bulbula leg and Rwanda leg. This roundabout has 3 circulating lanes, 22m circular island diameter, 3 lanes for each major road and 1 lane for each minor road.

Figure 6 Bole Mikael Roundabout

3.1.3. Saris Abo Roundabout

Saris Abo roundabout is located at Nifas-Silk-Lafto sub city around Saris Abo church. This study section has 4 legs, two of it legs are major roads directly linked to ring road and the other 2 legs are minor roads. Four approaching legs are indicated in Figure 7, which are Kadisco leg, Maseltegna leg, Saris leg and Abo Church leg. This roundabout has 3 circulating lanes, 22m circular island diameter, 3 lanes for each major road and 2 lanes for each minor road.

Figure 7 Saris Abo Roundabout

3.2. Data Collection

Data collected for this study was primary data and secondary data. Spot speed study, vehicle traffic volume and pedestrian volume, existing roundabout geometry study and an areal photography were collected. The necessary field visit and data collections were made on study area roundabout sections. Those collected data are:-

3.2.1. Spot Speed Study on Approaching Legs

Spot speed study was conducted on approaching four legs of those three roundabouts. This study was conducted by using stop watch techniques. Study length determination was according to speed range category as indicated in Table 3. According to hand book of traffic studies, sample size of studying spot speed study was 50 up to 100 vehicles. To have accurate result, sample size should be larger. Therefore number of vehicles used for this study was 100 vehicles per each approaching legs.

Table 3 Recommended Spot Speed Study Length (Traffic Studies, 2002)

No	Traffic Stream Average Speed			Recommended Study Length	
	mph	ft/s	km/h	Feet	Meter
1	< 25	< 37	< 40	88	27
2	25-40	37 - 60	40 – 64	176	54
3	> 40	> 60	> 64	264	80

3.2.2. Traffic Volume Study

Traffic volume count was conducted for three consecutive days (Tuesday, Wednesday and Thursday) during peak hour and off peak hour time for the selected study areas from 8:00 am to 6:00 pm. Five vehicle classifications were used as indicted in Table 4, for traffic volume study and analysis. Fifteen minute traffic count was conducted to represent one hour traffic volume count. This study utilizes passenger car unit as indicated in Table 5 to covert each vehicle type count in to passenger car.

Table 4 Vehicle Classification (ERA, 2013)

Class	Type	Grouping	Axles	Description
1	Car	PC	2	Passenger car and taxis
2	Pick- up/ 4- wheel drive		2	Pick-up, minibus, land Rovers, land cruisers
3	Small bus		2	≤ 27 seats
4	Bus/coach	BUS	2	> 27 seats
5	Small truck	S&MT	2	≤ 3.5 tons
6	Medium truck		2 or 3	3.5 - 7.5 tons
7	Large 2 –axles truck	LT	2	> 7.5 tons
8	3 -axles truck		3	> 7.5 tons
9	4 -axles truck		4	*
10	5 -axles truck		5	*
11	6 -axles truck		6	*
12	2 -axles trailer	TT	2	*
13	3 -axles trailer		3	*

Table 5 Passenger Car Equivalency (HCM 2000)

Passenger Car Equivalency (PCE)					
Vehicle category	PC	BUS	TRUCK	TRUCK TRAILER	MOTOR & BICYCLE
PCU	1.0	2.0	2.0	2.0	0.5

Roundabout traffic volume study on respective of each study areas are majorly classified based on their turning movement. Those turning movements are U- turn movement, Left turn movement, Right turn movement and through vehicle movement. Therefore traffic volume count study was categorized according to turning movement direction and vehicle classification.

3.2.3. Pedestrian Volume Study

Pedestrian volume count was conducted during morning rush hour (AM peak hour) and afternoon rush hour (PM peak hour) for each fifteen minutes. This study incorporates pedestrian volume crosses each roundabout legs. Pedestrian volume count method used for this study was manual counting techniques.

3.2.4. Roundabouts Geometry Study

Roundabout geometry study is vital for computation of saturation flow rate, for allocatin of traffic signal timing and for roundabout simulation. Those collected data were Approaching road gradient (road profile), approaching Lane width (carriage way), number of approaching lanes and legs, number of circulating lanes, inscribed circle diameter, exit and entry radius. Methods used to collect those data were google-earth map and manual tape.

3.3. Saturation Flow Rate

A saturation flow rate for each lane group is computed according to Equation 1. Saturation flow rate is the flow in vehicles per hour that could be accommodated by the lane group assuming that the green phase were displayed 100 percent of the time g/C= 1, in which “g” is vehicular green time and “C” is cycle length.

Table 6 is formula source for saturation flow rate factors. Those factors are as a function of roundabout geometry, traffic volume and study area. Therefore this table is useful for computation of roundabout approaching saturation flow rate.

$$S= S_O * N * f_W * f_{HV} * f_g * f_p * f_{bb} * f_a * f_{LU} * f_{LT} * f_{RT} * f_{Lpb} * f_{Rpb} \quad (1)$$

Where

S	=	Saturation flow rate for the subject lane group, expressed as a total for all lanes in the lane group (veh/h)
S_O	=	Base saturation flow rate per lane (1900pc/h/ln)
N	=	Number of lanes in the lane group
f_W	=	Adjustment factor for lane width (3.6m is base condition)
f_{HV}	=	Adjustment factor for heavy vehicles in the traffic stream
f_g	=	Adjustment factor for approach grade
f_p	=	Adjustment factor for the existence of parking lane and parking activity

adjustment to the lane group.

f_{bb} = Adjustment factor for the blocking effect of local buses that stop within the intersection area

f_a = Adjustment factor for area type

f_{LU} = Adjustment factor for lane utilization

f_{LT} = Adjustment factor for left-turn in the lane group

f_{RT} = Adjustment factor for right-turn in the lane group

f_{Lpb} = Pedestrian adjustment factor for left-turn movements, and

f_{Rpb} = Pedestrian/bicycle adjustment factor for right-turn movements

Table 6 Adjustment Factor Formulas for Saturation Flow Rate Calculation (HCM 2000)

Factor	Formula	Definition of variables	Notes
f_W	$f_W = 1 + \frac{(W - 3.6)}{9}$	W = lane width (m)	W ≥2.4 If W > 4.8 two lane analysis may be considered
f_{HV}	$f_{HV} = \frac{100}{100 + \%HV(E_T - 1)}$	%HV= % heavy vehicle for lane group volume	
f_g	$f_g = 1 - \frac{\%G}{200}$	%G= % grade on a lane group approach	$-6 \leq \%G \leq +10$
f_p	$f_p = \frac{N - 0.1 - \frac{18N_m}{3600}}{N}$	N = number of lane in lane group N_m = number of parking maneuvers per hour	$0 \leq N_m \leq 180$ $f_p \geq 0.05$ $f_p = 1$ for no parking
f_{bb}	$f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N}$	N_B = number of bus stopping per hour	$0 \leq N_b \leq 250$ $f_{bb} \geq 0.55$
f_a	$f_a = 0.9$ in CBD f_a =1 in all other areas	V_g = un adjusted demand flow rate for the lane group veh/h Vg_L= un adjusted demand flow rate on the single lane in the lane group with the highest volume and N = number of lanes in the lane group	
f_{LU}	$f_{LU} = \frac{v_g}{v_{gL} - N}$		

<table>
<tr><th>Factor</th><th>Formula</th><th>Definition of variables</th><th>Notes</th></tr>
<tr><td>f_{LT}</td><td>Protected phasing
Exclusive lane
$f_{LT} = 0.95$
Shared lane
$f_{LT} = \frac{1}{1 + 0.05P_{LT}}$</td><td rowspan="3">$P_{LT}$ = proportion of LT in lane group

P_{RT} = proportion of RT in lane group

P_{LT}= proportion of LT in lane group
A_{pbT}= permitted phase adjustment
PLTA= proportion of LT protected green over total LT green
PRT= proportion of RT in lane group
P_{RTA}= proportion of RT protected green over total RT green</td><td></td></tr>
<tr><td>f_{RT}</td><td>Exclusive lane
$f_{LT} = 0.85$
Shared lane
$f_{RT} = 1 - (0.15)p_{RT}$
Single lane
$f_{RT} = 0.9 - (0.135)p_{RT}$</td><td></td></tr>
<tr><td>f_{LT}</td><td>Left turn adjustment
$f_{Lpb} = 1 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$
Right turn adjustment
$f_{RTA} = 1 - P_{RT}(1 - A_{pbT})(1 - P_{LTA})$</td><td></td></tr>
</table>

3.4. Signal Timing (HCM, 2000)

Traffic signal timing design was executed on this study to have traffic control on roundabout sections during peak hour time. To allocate traffic signal timing vehicular green time, pedestrian green time, clearance interval (all red interval), transition interval (yellow interval) are essential. In designing traffic signal timing for vehicle it is critical to check pedestrian green time requirement. Formulas used for traffic signal timing designed are detailed as follow.

1) **Equivalent Factor for Left and Right Turn Vehicles :** To estimate an appropriate cycle length and to split the cycle length into appropriate green times for each phase, it is necessary to find the critical-lane volume for each discrete phase. Simple volumes cannot be simply compared, Trucks require more time than passenger cars, left and right-turns require more time than through vehicles, vehicles on a downgrade approach require less time than vehicles on a level or upgrade approach. Thus, intensity of demand is not

measured accurately by simple volume. Where phase plans involve over lapping elements, the ring diagram must be carefully examined to determine which flows constitute critical lane volume flows. Ideally, demand volumes would be converted to equivalents based on all of the traffic and roadway factors that might affect intensity. Left and right turn vehicles converted to equivalent through vehicle unit (tvus) by using factors as shown in Table 7and Table 8 respectively.

Table 7 Through Vehicle Equivalents for Left-Turning Vehicles, E_{LT} (HCM, 2000)

Opposing flow V_O (veh/h)	Number of opposing lanes N_O		
	1	2	3
0	1.1	1.1	1.1
200	2.5	2.0	1.8
400	5.0	3.0	2.5
600	10.0*	5.0	4.0
800	13.0*	8.0	6.0
1000	15.0*	13.0*	10.0*
≥ 1200	15.0*	15.0*	15.0*
E_{LT} for all protected left turns = 1.05			

*Indicates that the LT capacity is only available through "sneakers."

Table 8 Through Vehicle Equivalents for Right-Turning Vehicles, E_{RT} (HCM, 2000)

Pedestrian volume in conflicting cross walk peds/h	Equivalent
None (0)	1.18
Low (50)	1.21
Moderate (200)	1.32
High (400)	1.52
Extreme (800)	2.14

2) **Clearance Interval (all red interval; ar)** : Assuming that a vehicle has just entered the intersection legally on yellow, the all red must provide sufficent time for the vehicle to cross the intersection clear it back bumper past the far cross walk line before conflicting vehicles are given green.

✓ For cases in which there is no pedestrian traffic

$$ar = \frac{w + L}{1.47 * S_{15}} \qquad (2a)$$

✓ For cases in which significant pedestrian traffic exists

$$ar = \frac{P + L}{1.47 * S_{15}} \tag{2b}$$

✓ For cases in which some pedestrian traffic exists

$$ar = \max\left[\left(\frac{w + L}{1.47 * S_{15}}\right), \left(\frac{P}{1.47 * S_{15}}\right)\right] \tag{2c}$$

Where

ar = Length of all red phase, s

w = Distance from the departure STOP line to the far side of the farthest conflicting traffic lane, ft

p = Distance from the departure STOP line to the far side of the farthest conflicting crosswalk, ft

L = Length of a standard vehicle, usually taken to be 18-20 ft.

S_{15} = 15th percentile speed of approaching traffic or speed limit, as appropriate, mi/h

The difference between the three equation (2a, 2b and 2c) involves pedestrian activity levels and the decision to clear vehicles beyond the line of potential conflicting vehicles path and/or conflicting pedestrian paths before releasing the conflicting flows. Equation 2c, which adresses the most frequently occuring situations, some but not significant pedestrian flow is a compromise.

To provide optimal safety, the equation for yellow and all red intervals use different speeds, 85th and 15th percentile respectively. Because speed appears in the numerator of the all yellow determination and in the denominator of the all red determination, accommodating the majority of motorist safety requirement the use fo different percentiles.

3) **Change Interval (Yellow Interval, y)** : this interval allows a vehicle that one is safe stopping distance away from the stop line when the green is withdrawn to continue at the approach speed and enter the intersection legally on yellow.

$$y = t + \frac{1.47 * S_{85}}{2a + (2g * 0.01G)} \tag{3}$$

Where

y = Length of the yellow interval, s

t = Drivers reaction time, s

S_{85} = 85^{th} percentile speed of approaching traffic or speed limit, as appropriate, mi/h

A = Deceleration rate of vehicles, ft/s^2

G = Grade of approaches, %

G = Acceleration rate due to gravity, which is $32.2 ft/s^2$

4) **Cycle Length (C_{des}) :** equation 4 describing the max sum of critical lane volume that coud be handeled by a signal was manipulated to find a desirable cycle length . this equation is used to find the desirable cycle length based on tvu volumes.

$$C_{des} = \frac{L}{1 - [\frac{V_C}{S * PHF * (V/_C)}]} \quad (4)$$

Where

C_{des} = Desirable cycle length, s

L = Total lost time per cycle, s/cycle

PHF = Peak hour factor

v/c =Target v/c ratio for the critical movements in the intersection

S = Saturation flow rate veh/h/ln

5) **Effective Green Time (g_i) :** the available effective green time in the cycle must be divided signal phases. Total available green time in the cycle is found by deducting the lost time per cycle from cycle length as equation (5b).

$$g_i = g_{TOT} * (\frac{V_{ci}}{V_c}) \quad (5a)$$

$$g_{TOT} = C - L \quad (5b)$$

Where

g_i = Effective green time for phase i, s

g_{TOT} = Total effective green time in the cycle, s

Vci = Critical lane volume for phase or sub-phase i, (veh/h)

Vc = Sum of the critical lane volumes, (veh/h)

C = Cycle length, s

L = Lost time per cycle, s

6) **Pedestrian Green Time (Gp) :** The signal design has considered vehicular green time requirements, however pedestrian must be accomodated by the signal timing. Equation 6a and 6b was used to allocate minimum green time requirement for pedestrian.

$$G_P = 3.2 + \left(\frac{L}{s_p}\right) + \left(2.7 * \frac{N_{Ped}}{W_E}\right) \tag{6a}$$

For $W_E > 10$ ft

$$G_P = 3.2 + \left(\frac{L}{s_p}\right) + \left(0.27 * N_{ped}\right) \tag{6b}$$

For $W_E \leq 10$ ft

Where

G_P = Minimum pedestrian crossing time, s

L = Length of the cross walk, ft

S_P = Average Walking speed of pedestrians, ft/s

N_{ped} = Number of pedestrians crossing per phase on a single crosswalk, peds

W_E = Width of cross walk, ft

3.5. VISSIM Simulation Modeling

VISSIM is microscopic simulation program for modeling multimodal transport operations. It captures the movement of every vehicle. Individual vehicles can be traced through the network. This software is used to model roundabout with signalization and extract result (control delay, level of service and queue length).

This tool has capability to model multilane roundabout (three lane approaches) and intersections with and without traffic signal timing and provide a result. Developing road network, node and link development, route decision, signal timing and phase diagram sequencing was used to model roundabout by this tool. Thus major five steps implemented on VISSIM modeling are indicated in Figure 8

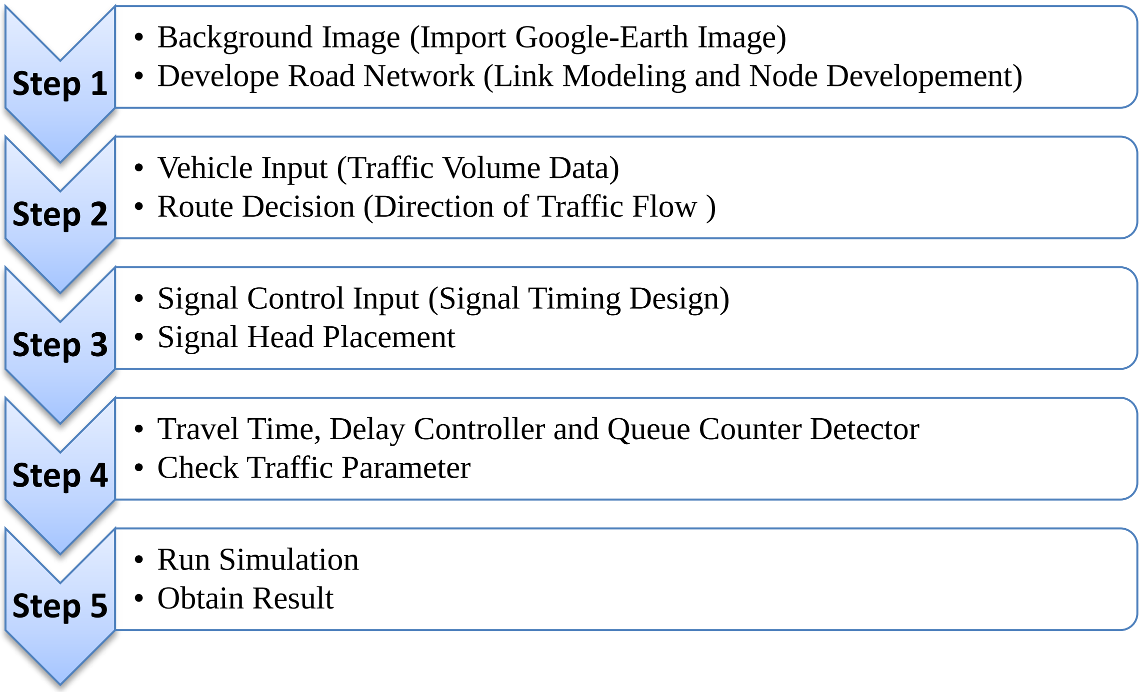


Figure 8 VISSIM Simulation Modeling Steps

3.6. VISTRO

With PTV VISTRO this study can evaluate, optimize and re time traffic signal, evaluate intersection LOS. This makes it a useful tool for alternative 2 and 3 evaluation. VISTRO considers two useful parameters as an option to optimize traffic signal timing as an option, Minimize critical movement delay or Balance v/c ratio. This study evaluates three alternatives for study area as indicated in Table 9.

Table 9 Study Alternatives

Existing Condition	Alternatives	Mitigation Alternatives	Tools
Roundabout	Alternatives One	Evaluation of roundabout as it is	VISTRO
	Alternatives Two	Evaluation of changing Roundabout in to Signalized Intersection	
	Alternatives Three	Evaluation of Roundabout Signalization	VISSIM

3.7. Summary

This research utilizes methods for data collection, analysis and evaluation. Methods used for data collection are video record and stop watch techniques for traffic volume survey and spot speed study respectively.

Traffic volume collected data considers roundabout turning movements (right turn, left turn, U-turn and through vehicle movement with 5 vehicle classification categories according to ERA 2013 manual. Roundabout geometric study is conducted both by using field measurement and google earth map.

Data analysis were done by using excel spread-sheet. Formulas are used for Saturation flow rate and signal timing design computation from HCM 2010. For performance measurement of the three alternatives VISSIM and VISTRO simulation modeling was used to have a comparative study on those three alternatives under base and future condition.

Chapter 4: Data Analysis

Data analysis includes spot speed study analysis, traffic volume study analysis, computation of roundabout approaching saturation flow rate and traffic signal timing design..

4.1. Spot Speed Study and Analysis

The collected data in spot speed study taken only from a sample of vehicles using the section of study area in which the study is conducted, but those speed data were used to determine the speed characteristics of the whole population of vehicles traveling on the study site. It is therefore necessary to use statistical methods in analyzing these data.

First step in the preparation of frequency distribution table is selection of the number of classes that is, the number of speed ranges in which the data are to be fitted. The number of classes chosen is usually between 8 and 20, depending on the data collected. Technique used to determine the number of classes is, first determine the range for a class size of 8 and then for a class size of 20. Finding the difference between the maximum and minimum speeds in the data and dividing this number first by 8 and then by 20 gives the maximum and minimum ranges in each class.

A convenient range for each class is then selected. Frequency distribution, average speed, 15^{th} and 85^{th} percentile spot speed study analysis and result are as follow, stated in Figure 9 and Figure 10 for the rest of study areas refer appendix A and B.

Table 10, in which the speed classes are listed in column-2 and the speed mid value are in column-3. The number of observation for each class is listed in column-5; the cumulative percentage of all observations is listed in column-8.

Figure 9 shows the frequency histogram for the data shown in Table 10. The value in column-3 and 5 of Table 10 are used to plot the frequency histogram where abscissa represents the approaching speed and the ordinate represents vehicle frequency in each class.

Figure 10, shows the cumulative frequency distribution curve for the data given. In this case the cumulative percentage in column-8 of Table 10 is plotted again corresponding speed class. This curve gives the percentage of vehicles that are traveling at or below a given speed class.

Table 10 Spot Speed Study Analysis at Gerji Emperial Roundabout, 24 Street Approach

1	2	3	4	5	6	7	8	9
Class	Speed class km/hr	Class mid value, Ui (km/h)	Class mid value, U_i (mil/h)	Class frequency fi (number of observation)	fi*Ui	Percentage of observations in the class	Cumulative percentage of observations	$f*(u_i - \hat{U})^2$
1	16-18.99	17.5	11.9	3	52.5	3%	3%	943.059
2	19-21.99	20.5	13.9	3	61.5	3%	6%	650.919
3	22-24.99	23.5	16.0	5	117.5	5%	11%	687.965
4	25-27.99	26.5	18.0	12	318.0	12%	23%	914.555
5	28-30.99	29.5	20.1	15	442.50	15%	38%	492.493
6	31-33.99	32.5	22.1	12	390.0	12%	50%	89.435
7	34-36.99	35.5	24.1	9	319.5	9%	59%	0.656
8	37-39.99	38.5	26.2	8	308.0	8%	67%	85.543
9	40-42.99	41.5	28.2	10	415.0	10%	77%	393.129
10	43-45.99	44.5	30.3	9	400.5	9%	86%	773.396
11	46-48.99	47.5	32.3	6	285.0	6%	92%	903.317
12	49-51.99	50.5	34.4	6	303.0	6%	98%	1399.037
13	52-54.99	53.5	36.4	1	53.5	1%	99%	333.793
14	55-57.99	56.5	38.4	1	56.5	1%	100%	452.413
	Total	518	352	100	3523	100%		8119.710

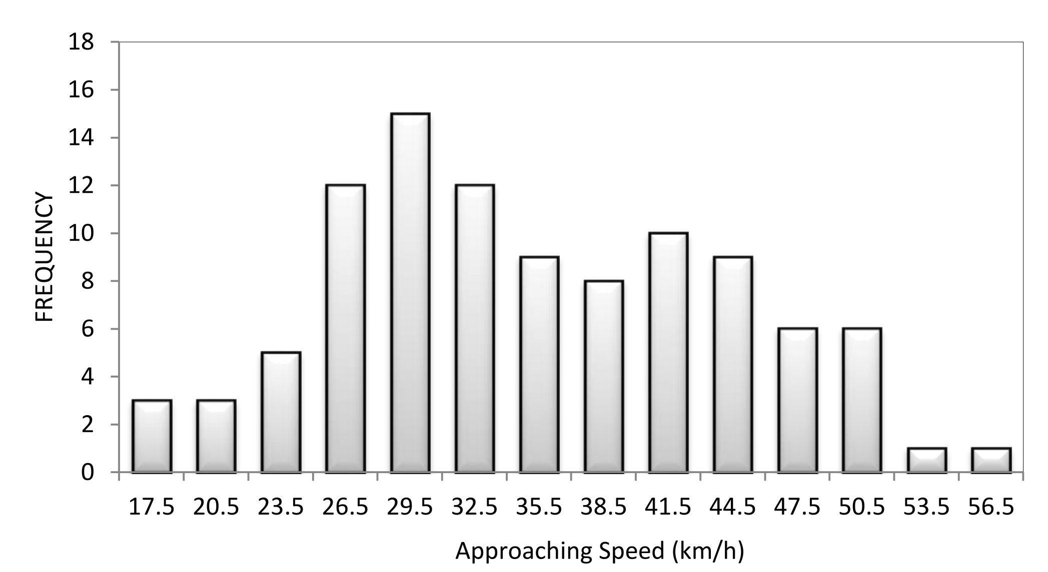


Figure 9 Spot Speed Study Frequency Distribution at Gerji Emperial Roundabout, 24 Street Approach

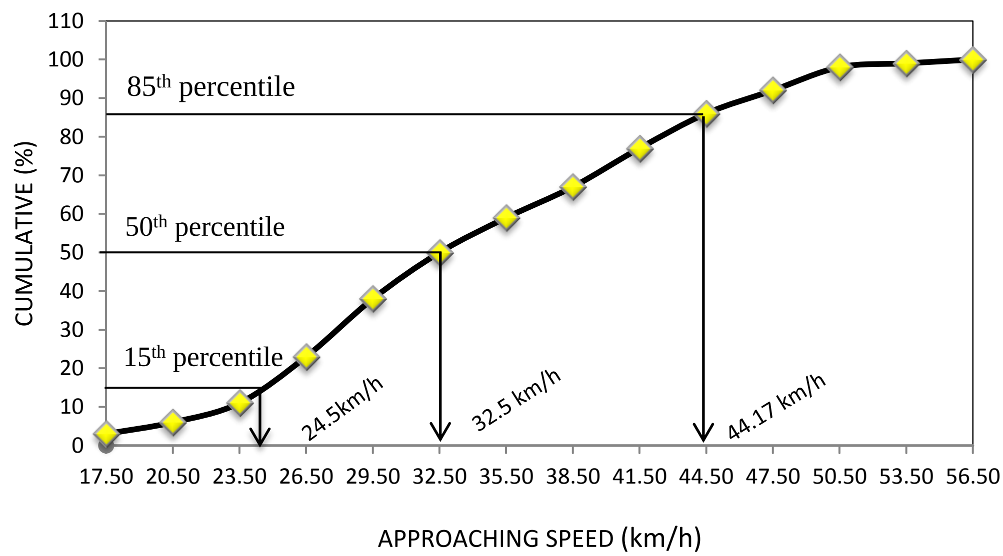


Figure 10 15th, 50th and 85th Percentile Spot Speed at Gerji Emperial Roundabout, 24 Street Approach

4.2. Traffic Volume Study

Figure 11, Figure 12 and Figure 13 indicates traffic count study on respective study areas for three days from 8:00 am up to 7:00 pm. During the time of study conducted at Gerji Emperial roundabout the indicated figure shows peak volume stays for longer time relative to the other roundabouts and it has AM and PM peak traffic volume and the rest stays off peak as shown in Figure 12.

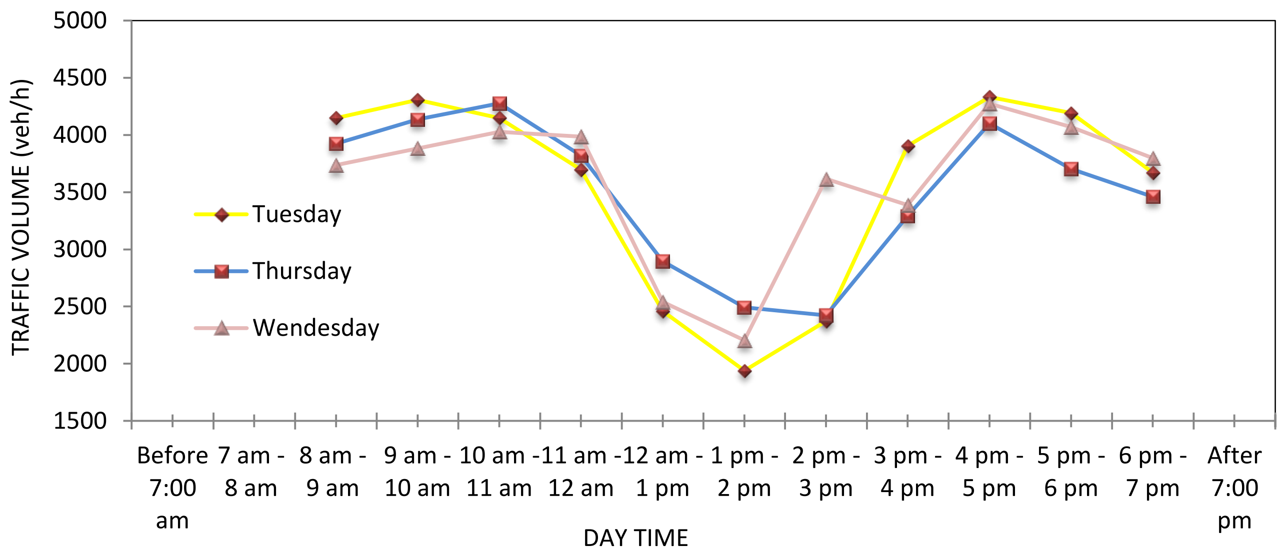


Figure 11 Three Days Traffic Count Summary at Gerji Emperial Roundabout

Figure 12 summarizes traffic volume survey at Bole Mikael roundabout. By the time of this study it show that three peak periods were observed (AM, PM and lunch time peak period). Small duration peak period were observed relative to Gerji Emperial roundabout peak period. Figure 12 also indicates that the peak period was not captured, because the data collection starts from 8:00 am in the morning. The analysis didn’t show before 8:00 am whether It falls down or increases. But the traffic volume before 8:00 am in the morning is not expected to be greater than the peak traffic volume.

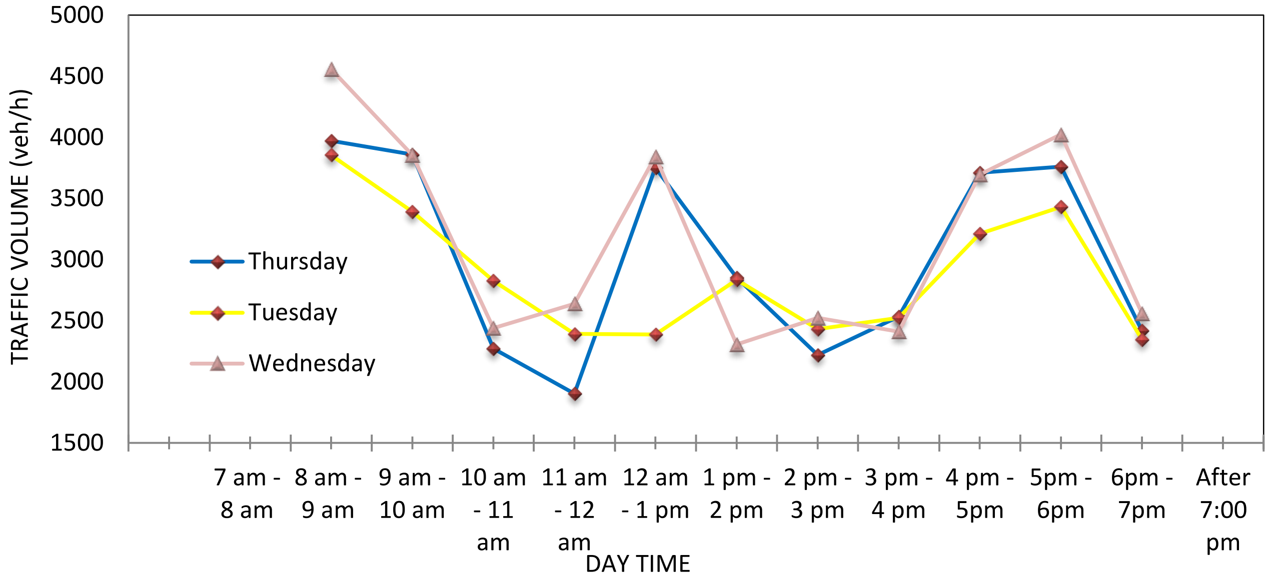


Figure 12 Three Days Traffic Count Summary at Bole Mikael Roundabout

Saris Abo roundabout has relative longer duration of off peak period as compared to other study areas. Peak point in Figure 13 describes peak period corresponds to peak hour traffic volume. Sharp corners and immediate fall down in this figure shows the extent of peak period and time it takes to change peak hour to off peak hour as shown in Figure 13.

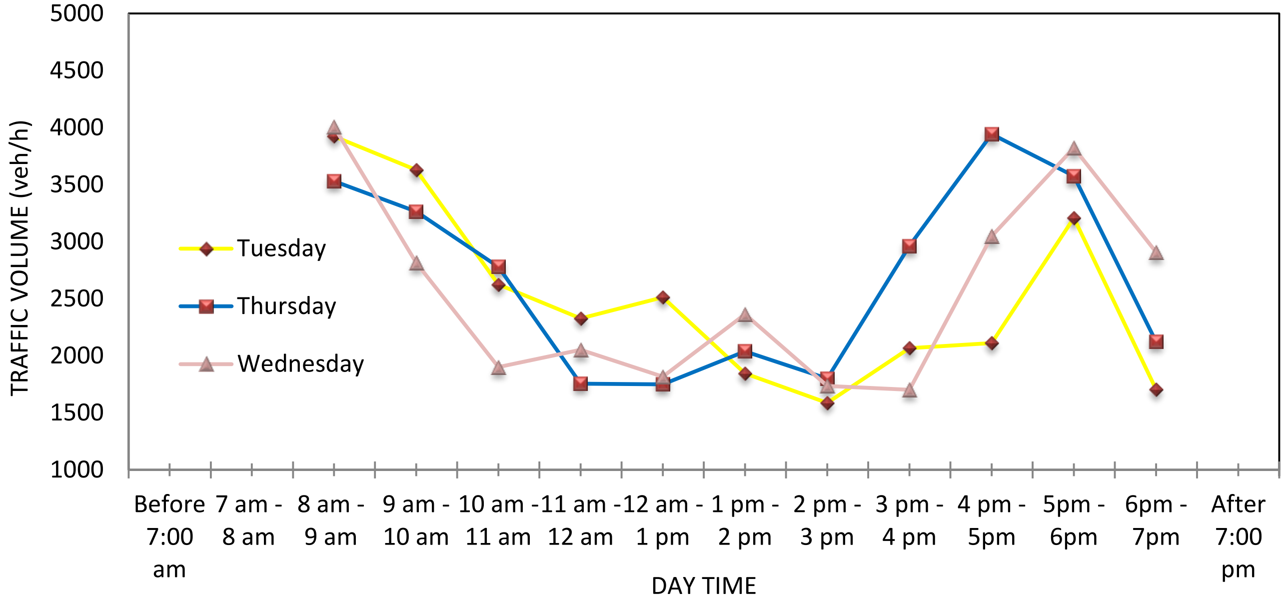


Figure 13 Three Days Traffic Count Summary at Saris Abo Roundabout

4.2.1. Peak Hour Factor (PHF)

Table 11 indicates peak one hour traffic data collection and peak hour factor value for those three roundabouts. The peak hour factor (PHF) is the hourly volume during the maximum-volume hour of the day divided by the peak 15-minute flow rate within the peak hour. It is also a measure of traffic demand fluctuations within the peak hour. When peak hour approximate to one, means one hour traffic volume data variability is for each 15 minute period is minimum, which leads to equal amount of traffic volume was observed for one hour peak period. Demand fluctuation at Saris Abo roundabout, Maseltegna approach is relatively small due to PHF highest value. Use of peak hour factor ensures that the timing is appropriate for the peak 15 minutes of the design hour.

Table 11 Peak Hour Factor computation for (a) Gerji Emperial, (b) Bole Mikael and (c) Saris Abo roundabouts

TUESDAY FROM 9:00 am TO 10:00 am																								
APPROACH		MOVEMENT																				TOTAL		PHF
		THROUGH					LEFT TURN					RIGHT TURN					U-TURN							
	No	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT			
BOLE	1	310	40	9	9	1	11	0	1	3	0	40	6	0	6	0	3	1	1	1	0	442	1557	0.88
	2	304	39	4	1	0	13	0	1	0	0	23	4	0	1	0	2	1	0	1	0	394		
	3	284	30	7	4	1	15	0	1	2	0	29	4	0	1	0	4	1	1	1	0	385		
	4	263	21	8	5	1	9	0	1	1	0	18	1	0	3	0	4	1	0	0	0	336		
MEGEN AGNA	1	286	22	12	2	1	46	3	2	0	0	29	0	0	1	0	1	0	0	0	0	405	1629	0.85
	2	279	26	15	2	1	50	2	1	0	0	14	0	0	0	0	1	0	0	0	0	391		
	3	254	18	10	3	0	32	2	0	3	0	31	0	0	1	0	0	0	0	0	0	354		
	4	314	33	13	0	2	60	3	3	6	0	43	0	0	1	0	1	0	0	0	0	479		
GERJI	1	41	0	0	0	0	66	2	0	2	1	42	4	0	0	0	0	0	0	0	0	158	670	0.89
	2	49	0	0	0	0	60	1	1	3	0	39	2	0	0	0	0	0	0	0	0	155		
	3	56	0	1	1	0	66	0	0	4	0	41	0	0	0	0	0	0	0	0	0	169		
	4	58	0	1	1	0	73	2	1	3	1	45	3	0	0	0	0	0	0	0	0	188		
24 STREET	1	46	0	0	0	0	93	1	2	0	0	52	1	3	3	0	0	0	0	0	0	201	678	0.84
	2	49	0	0	0	0	76	1	2	0	0	38	1	1	2	0	0	0	0	0	0	170		
	3	40	0	0	0	0	80	1	0	0	0	23	1	0	0	0	0	0	0	0	0	145		
	4	57	0	0	0	0	58	1	3	0	0	42	0	1	0	0	0	0	0	0	0	162		

(a)

WEDNESDAY FROM 8:00 AM TO 9:00 AM																										
APPROACH		MOVEMENT																						TOTAL		PHF
		THROUGH					LEFT TURN					RIGHT TURN					U-TURN									
	No	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT			
BOLE	1	192	16	3	3	0	34	2	0	1	0	78	2	3	0	0	28	0	3	0	1	366	1604	0.92
	2	238	24	5	5	0	39	1	1	1	0	83	2	3	0	0	30	0	2	1	1	436		
	3	202	20	4	4	0	31	1	0	1	0	71	2	3	0	0	26	0	2	0	1	368		
	4	247	21	5	5	0	33	0	0	0	0	81	2	4	0	0	35	0	1	0	0	434		
KADISCO	1	286	14	7	9	1	0	0	0	0	0	2	0	0	0	0	0	0	3	0	0	322	1398	0.89
	2	320	22	10	13	2	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	369		
	3	272	19	9	11	2	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	315		
	4	321	22	10	13	2	0	0	0	0	0	12	2	4	0	0	5	0	1	0	0	392		
RWANDA	1	32	1	2	0	0	18	1	3	1	0	51	1	2	0	0	0	0	0	0	0	112	466	0.90
	2	29	1	2	0	0	21	1	4	1	0	67	1	3	0	0	0	0	0	0	0	130		
	3	26	0	3	0	0	20	1	4	1	0	60	1	3	0	0	0	0	0	0	0	119		
	4	23	1	2	0	0	17	1	3	1	0	54	1	2	0	0	0	0	0	0	0	105		
BULBULA	1	0	0	0	0	0	0	0	0	0	0	172	1	4	0	0	0	0	0	0	0	177	670	0.95
	2	0	0	0	0	0	0	0	0	0	0	170	2	4	0	0	0	0	0	0	0	176		
	3	0	0	0	0	0	0	0	0	0	0	154	1	3	0	0	0	0	0	0	0	158		
	4	0	0	0	0	0	0	0	0	0	0	155	2	2	0	0	0	0	0	0	0	159		

(b)

TUESDAY FROM 8:00 AM TO 9:00 AM																								
APPROACH		MOVEMENT																				TOTAL		PHF
		THROUGH					LEFT TURN					RIGHT TURN					U-TURN							
	No	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT	PC	BUS	S&MT	LT	TT			
KADISCO	1	209	23	19	16	2	18	0	0	5	0	37	3	0	1	0	29	7	2	0	0	371	1553	0.94
	2	232	26	21	18	2	20	0	0	5	0	41	3	0	1	0	32	10	2	0	0	413		
	3	230	29	19	17	0	21	0	0	4	0	39	3	0	1	0	33	6	2	0	0	404		
	4	227	21	16	13	4	16	0	0	0	0	36	0	0	0	0	27	5	0	0	0	365		
MASELTEGNA	1	244	13	18	6	4	31	3	2	0	0	29	1	0	4	0	26	2	0	0	0	383	1469	0.96
	2	239	15	11	7	3	29	0	0	0	0	40	0	0	0	0	30	6	0	0	0	380		
	3	236	8	13	1	1	26	0	0	2	0	24	0	0	0	0	21	4	0	0	0	336		
	4	227	21	16	13	4	21	0	0	0	0	36	0	0	0	0	27	5	0	0	0	370		
ABO CHURCH	1	29	2	2	0	0	22	2	3	6	0	71	0	5	1	0	11	1	2	0	1	158	710	0.86
	2	38	2	2	0	0	29	2	4	8	0	95	0	6	1	0	15	1	3	0	1	207		
	3	31	2	2	0	0	24	2	3	7	0	78	0	5	1	0	12	0	2	0	1	170		
	4	33	2	2	0	0	25	2	3	2	0	82	0	5	1	0	13	1	3	0	1	175		
SARIS	1	2	0	0	0	0	21	0	0	0	0	5	2	1	0	0	3	0	0	0	0	34	154	0.88
	2	1	0	0	0	0	28	0	0	0	0	6	3	1	0	0	4	0	0	0	0	43		
	3	3	0	0	0	0	23	0	0	0	0	0	3	1	0	0	3	0	0	0	0	33		
	4	3	0	0	0	0	27	0	0	0	0	6	3	1	0	0	4	0	0	0	0	44		

(c)

Traffic count summary at Gerji Emperial roundabout detailed on Figure 14. On this figure three generalized vehicle classification categories are used, light vehicle (passenger car), heavy vehicle (bus, small and medium truck, large truck and truck trailer) and bicycle & motor cycle. In Figure 14 light vehicle have higher proportions relatively when we compared with heavy vehicle and cycles. For the remaining study section refer Appendix E.

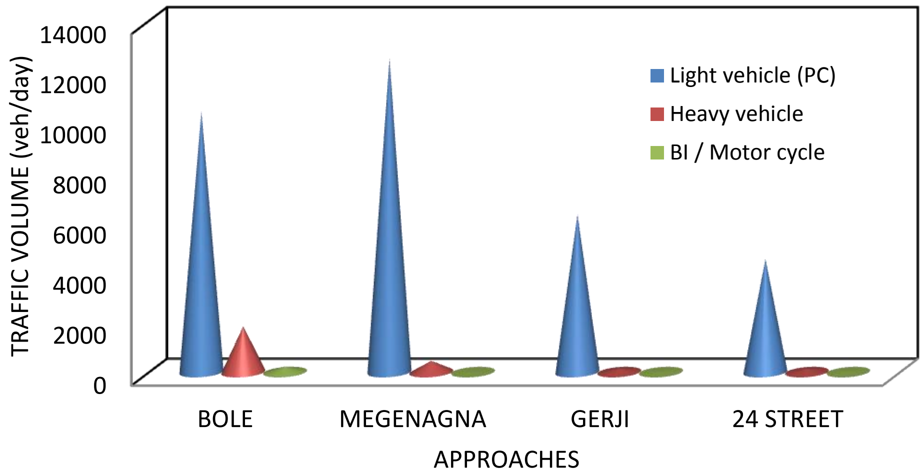


Figure 14 Traffic Count Summary at Gerji Emperial Roundabout; Tuesday

Figure 15 detailed the dominancy of through vehicle movement for major road approaches (more than 80% of traffic flow from the respective approaches is through vehicle) and minor road approaches have equivalent traffic volume data for each movement (through, right turn and left turn movements). For the rest of study areas refer Appendix F.

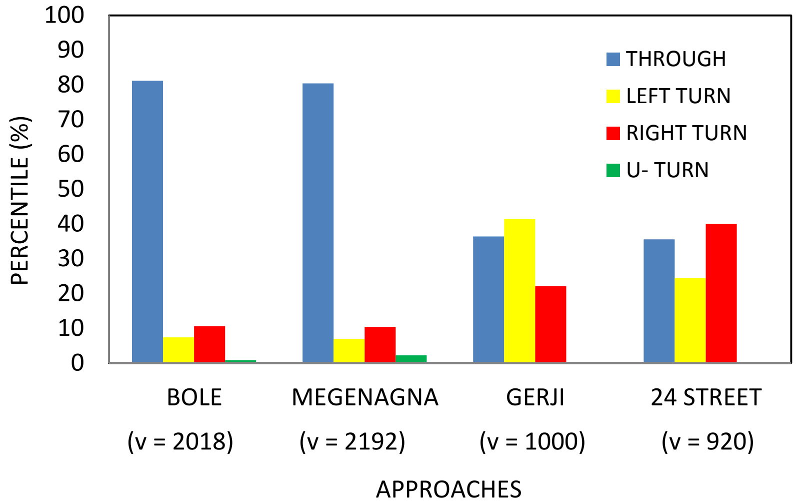


Figure 15 Percentile Proportioning of Each Turning Movement per Hour per Approach at Gerji Emperial Roundabout during peak hour

Alternative 2 (changing roundabout in to signalized intersection) consider U-turn movement as left turn movement for the application of intersection signalization. Thus according to traffic volume study and analysis U-turn movement per phase was insignificant as indicated in Figure16. Consider U-turn movement as left turn movement minimizes flow complexity and minimizes excessive delay due to additional phasing.

Figure16 and Appendix G explains the amount of vehicles expected to join roundabouts as U-turn movement per phase during peak hour time is in significant as compared to other movement type.

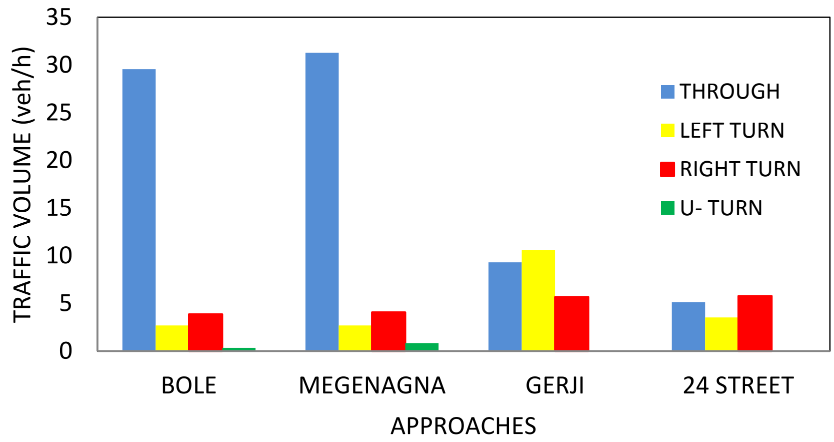


Figure16 Peak hour Turning Movement Study per Phase per Approaching at Gerji Emperial Roundabout

Figure 17 and Figure 18 indicates that there was a considerable significant variability in traffic volume between peak hour (Am and Pm peak hour) and off peak hour data at the time this study was conducted. Analysis result presented in Figure 17, Gerji approach shows highest traffic volume variability which is 60% between AM peak hour and off peak hour data. Figure 18, Megenagna approach shows highest traffic volume variability, which is 56% between PM peak hour and off peak hour data. For the rest of the study areas refer Appendix H.

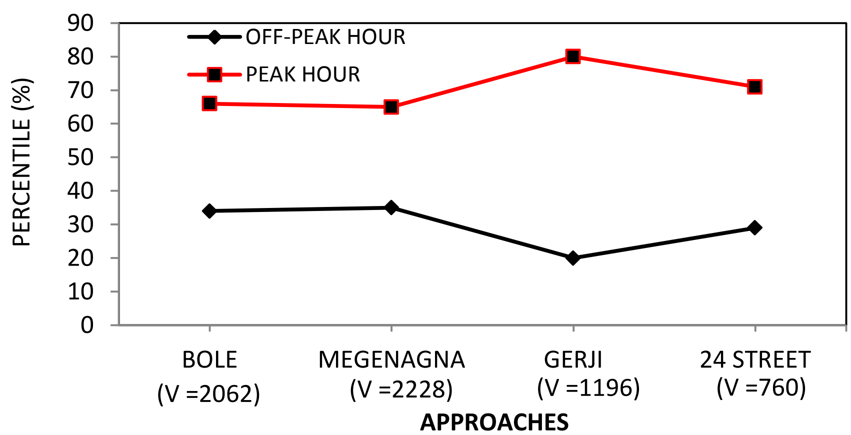


Figure 17 AM peak and off peak Hour Traffic Count Comparison at Gerji Emperial Roundabout

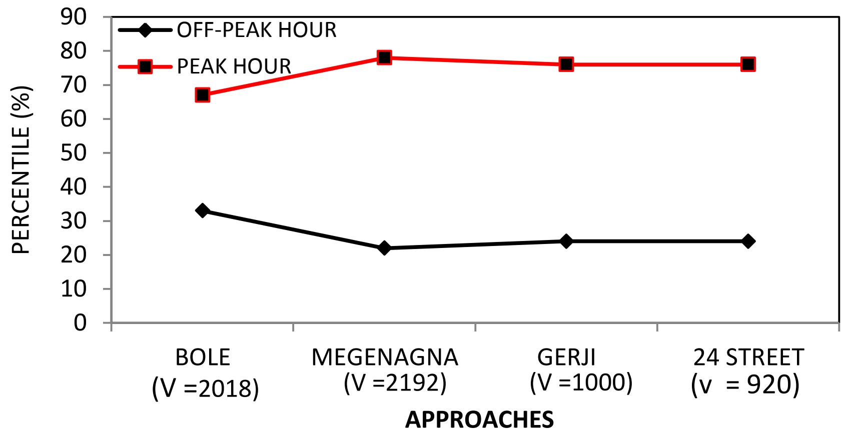


Figure 18 PM peak and off peak Hour Traffic Count Comparison at Gerji Emperial Roundabout

4.3. Saturation Flow Rate

To do an experiment and to have saturation flow rate value for roundabout section is very complicated and very difficult, because of the traffic flow stop and go movement make it difficult to calculate head difference, so base saturation flow rate 1900pc/h/ln with adjustment factors are used in this study. But this default value may increase or decreases depend on local field measurements.

Table 13 presents the result of study areas saturation flow rate. Saturation flow rate is sensitive on lane geometry, lane width, heavy vehicle and approaching grade factors. In this table w there is variation of result as indicated. Minor road and major road saturation flow rate variation is due to the saturation flow rate factor. When we consider base saturation flow rate 1900pc/h/ln we can notice significant variation in flow for roundabout approaches.

According to this study finding saturation flow rate significantly decreases from base saturation flow rate value (1900pc/h/ln), due to existing adjustment factors stated in Table 12. For instance minor road approach have relatively lowest saturation flow rate than major roads due to the existence of road side parking, compromised lane geometry and approaching grades. From minor road approaches, 24 street approach has smallest saturation flow rate value due to the existence of considerable heavy vehicle proportioning.

Table 12 Saturation Flow Rate Adjustment Factor (a) Gerji Emperial (b) Bole Mikael and (c) Saris Abo Roundabouts Approaches

Gerji Emperial Roundabout Approaches				
	Bole	Gerji	24 Street	Megenagna
f_W	0.9889	0.9889	0.9889	0.9889
f_{HV}	0.8410	0.8030	0.7270	0.8280
f_g	0.9999	1.0003	1.0000	1.0001
f_p	1.0000	0.9300	0.9500	1.0000
f_{bb}	1.0000	1.0000	1.0000	1.0000
f_a	1.0000	1.0000	1.0000	1.0000
f_{LU}	0.9100	0.9500	0.9500	0.9100
f_{LT}	1.0000	1.0000	1.0000	1.0000
f_{RT}	1.0000	1.0000	1.0000	1.0000
f_{Lpb}	1.0000	1.0000	1.0000	1.0000
f_{Rpb}	1.0000	1.0000	1.0000	1.0000

(a)

Bole Mikael Roundabout Approaches			
Bole	Kadisco	Rwanda	Bulbula
0.9889	0.9889	0.9333	0.9333
0.8994	0.8652	0.9216	0.9673
1.0000	0.9999	1.0000	0.9999
1.0000	1.0000	1.0000	1.0000
1.0000	1.0000	1.0000	1.0000
1.0000	1.0000	0.9000	1.0000
0.9100	0.9500	0.9100	0.9500
1.0000	0.9963	1.0000	0.9912
1.0000	1.0000	1.0000	0.8927
1.0000	1.0000	1.0000	1.0000
1.0000	1.0000	1.0000	1.0000

(b)

Saris Abo Roundabout Approaches			
Kadisco	Maseltegna	Abo Church	Saris
0.9889	0.9889	0.9889	0.9889
0.9010	0.8720	0.8810	0.926
0.9999	0.9999	0.9997	1.0000
0.9125	1.0000	0.8667	1.0000
1.0000	0.9810	1.0000	1.0000
1.0000	1.0000	1.0000	0.9000
0.9100	0.9100	0.9500	0.9500
1.0000	1.0000	1.0000	1.0000
1.0000	1.0000	1.0000	1.0000
1.0000	1.0000	1.0000	1.0000
1.0000	1.0000	1.0000	1.0000

(c)

Table 13 Saturation Flow Rate (S) Result Summary on Study Area Roundabouts Approaches

Gerji Emperial Roundabout		S
Approaches	Bole	1410 pc/h/ln
	Gerji	1262 pc/h/ln
	24 Street	1144 pc/h/ln
	Megenagna	1389 pc/h/ln
Bole Mikael roundabout		S
Approaches	Bole	1478 pc/h/ln
	Kadisco	1530 pc/h/ln
	Rwanda	1267 pc/h/ln
	Bulbula	1381 pc/h/ln
Saris Abo roundabout		S
Approaches	Kadisco	1376 pc/h/ln
	Maseltegna	1434 pc/h/ln
	Abo church	1243 pc/h/ln
	Saris	1384 pc/h/ln

4.4. Signal Timing Design

Traffic signal timing allocation is used on this study as an alternative mitigation solution to regulate traffic flow with existing roundabouts. In Table 14 results of traffic signal timing is presented which includes, vehicular green time, yellow interval and all red interval.

Vehicular green time interval on major approaches (Bole, Megenagna, Kadisco and Maseltegna) has relatively highest green time because of in excess of traffic volume and higher saturation flow rate as indicated in Figure 14 and Table 13 respectively. Yellow interval for all approaches doesn't show significant variation but it is as a function of 85th percentile speed and approaching grade. Thus change in speed and approaching grade may show variation relatively to the other approaches. Allred interval increment was due to lengthy roundabout geometry to cross from one approach to the other far side to dissipate and 15th percentile speed decrement leads to have highest all red values.

Phase sequencing used for modeling roundabout signalization splitting signal phases sequence indicated in Figure 19. Split phase sequencing is used due to roundabout geometry, which means roundabout as it is didn't allow left turn to be protected or right turn by pass. Therefore this study provides shared lane and split phase sequencing as one input for roundabout signalization modeling, but for signalized intersection indicated in Figure 20.

Table 14 Traffic Signal Timing Result Summary on Study Area Roundabout Approaches

Signal cycle components	Gerji Emperial Roundabout			
	Bole	Gerji	24 Street	Megenagna
Phasing	$Ø_1$	$Ø_4$	$Ø_2$	$Ø_3$
Green time (sec)	39.00	39.00	25.00	47.00
Yellow interval (sec)	3.00	2.00	4.00	4.00
All red interval (sec)	4.00	7.00	5.00	4.00
Signal cycle components	Bole Mikael Roundabout			
	Bole	Kadisco	Rwanda	Bulbula
Phasing	$Ø_1$	$Ø_3$	$Ø_4$	$Ø_2$
Green time (sec)	42.00	47.00	18.00	29.00
Yellow interval (sec)	4.00	3.00	2.00	2.00
All red interval (sec)	3.00	7.00	7.00	8.00
Signal cycle components	Saris Abo Roundabout			
	Kadisco	Maseltegna	Abo Church	Saris
Phasing	$Ø_1$	$Ø_3$	$Ø_2$	$Ø_4$
Green time (sec)	66.00	66.00	50.00	12.00
Yellow interval (sec)	3.00	3.00	3.00	2.00
All red interval (sec)	5.00	7.00	7.00	9.00

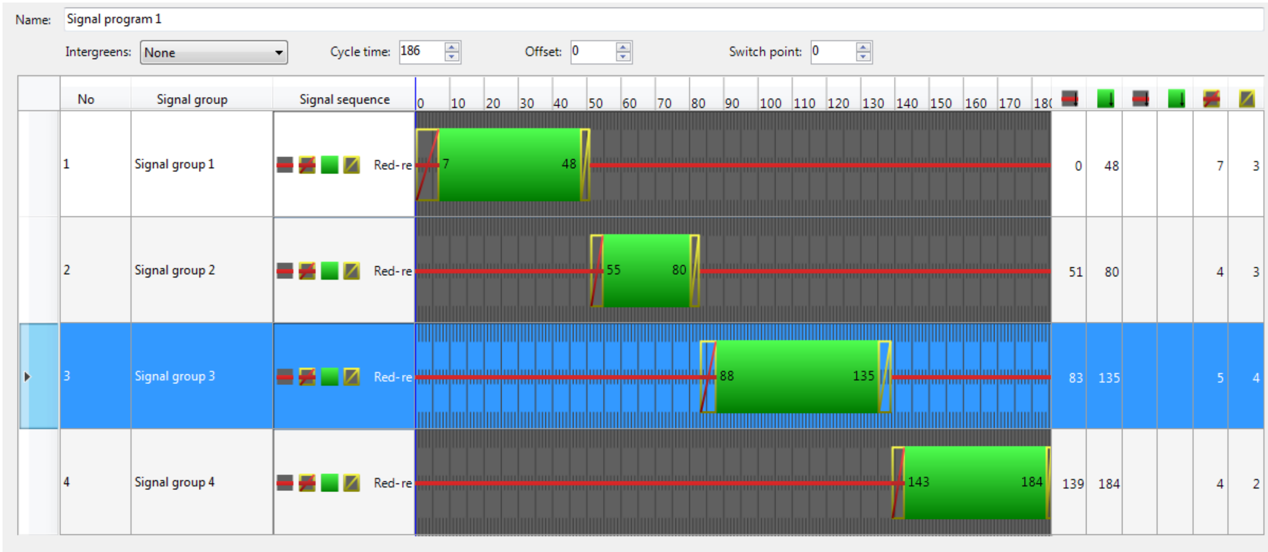
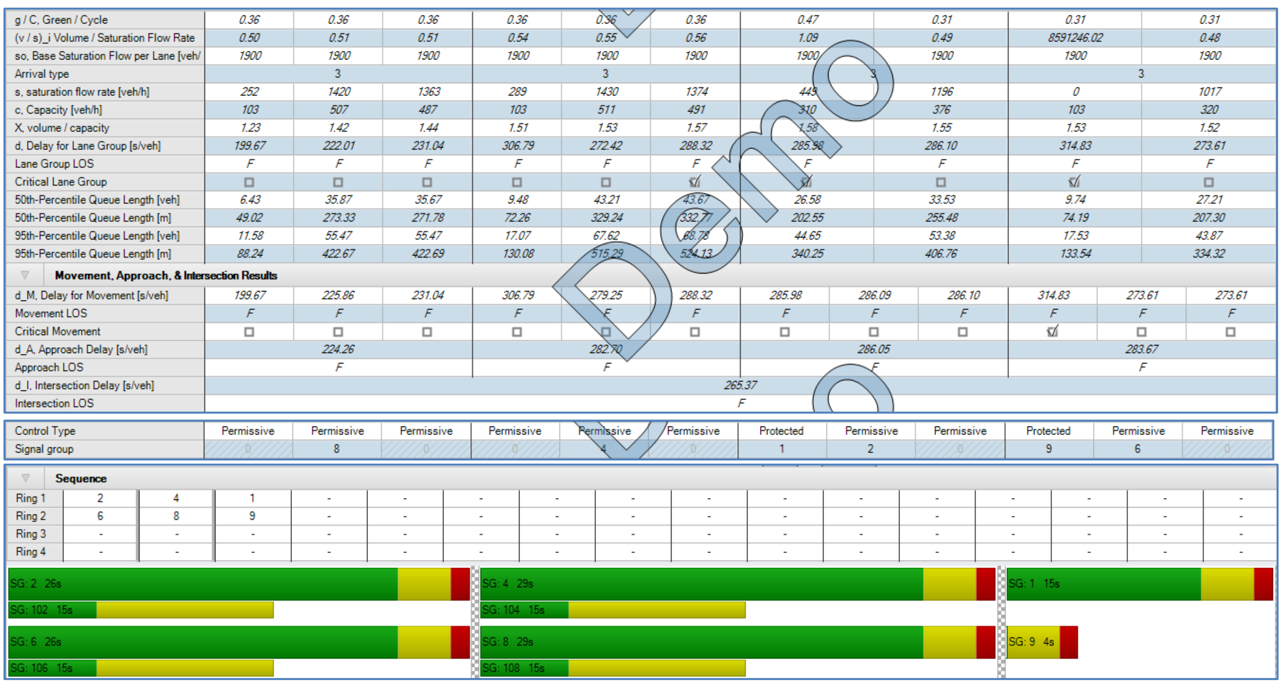


Figure 19 Gerji Emperial signal phasing diagram for roundabout signalization



g / C, Green / Cycle	0.36	0.36	0.36	0.36	0.36	0.36	0.47	0.31	0.31	0.31
(v / s)_i Volume / Saturation Flow Rate	0.50	0.51	0.51	0.54	0.55	0.56	1.09	0.49	8591246.02	0.48
so, Base Saturation Flow per Lane [veh/	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Arrival type		3			3			3		3
s, saturation flow rate [veh/h]	252	1420	1363	289	1430	1374	449	1196	0	1017
c, Capacity [veh/h]	103	507	487	103	511	491	310	376	103	320
X, volume / capacity	1.23	1.42	1.44	1.51	1.53	1.57	1.58	1.55	1.53	1.52
d, Delay for Lane Group [s/veh]	199.67	222.01	231.04	306.79	272.42	288.32	285.98	286.10	314.83	273.61
Lane Group LOS	F	F	F	F	F	F	F	F	F	F
Critical Lane Group	☐	☐	☐	☐	☐	☑	☑	☐	☑	☐
50th-Percentile Queue Length [veh]	6.43	35.87	35.67	9.48	43.21	43.67	26.58	33.53	9.74	27.21
50th-Percentile Queue Length [m]	49.02	273.33	271.78	72.26	329.24	332.77	202.55	255.48	74.19	207.30
95th-Percentile Queue Length [veh]	11.58	55.47	55.47	17.07	67.62	68.78	44.65	53.38	17.53	43.87
95th-Percentile Queue Length [m]	88.24	422.67	422.69	130.08	515.29	524.13	340.25	406.76	133.54	334.32

Movement, Approach, & Intersection Results												
d_M, Delay for Movement [s/veh]	199.67	225.86	231.04	306.79	279.25	288.32	285.98	286.09	286.10	314.83	273.61	273.61
Movement LOS	F	F	F	F	F	F	F	F	F	F	F	F
Critical Movement	☐	☐	☐	☐	☐	☐	☐	☐	☐	☑	☐	☐
d_A, Approach Delay [s/veh]		224.26			282.70			286.05			283.67	
Approach LOS		F			F			F			F	
d_I, Intersection Delay [s/veh]						265.37						
Intersection LOS						F						
Control Type	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Protected	Permissive	Permissive	Protected	Permissive	Permissive
Signal group	0	8	0	0	4	0	1	2	0	9	6	0

Sequence																
Ring 1	2	4	1	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 2	6	8	9	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Figure 20 Gerji Emperial signal phasing diagram for intersection signalization

4.4.1. Pedestrian Green Time Requirement

Designing traffic signal timing requires checking for pedestrian green time requirement. Pedestrian green time requirement from safety and traffic flow point of view, it is crucial. This scenario is fulfilled on Gerji Emperial and bole Mikael roundabout but at Saris Abo roundabout pedestrian green time requirement is not accommodated by vehicle green time (Kadisco and Maseltegna approach), because of pedestrian volume is in excess amount. Applying signal timing to this study area either in the form of roundabout signalization or changing roundabout into signalized intersection creates compromised pedestrian safety or vehicular capacity reduction. Pedestrian green time requirement as compared to vehicular green time is presented on Table 15 for those study areas. In computation of pedestrian green time pedestrian walking speed was taken to be 4 ft/sec (HCM, 2000). But pedestrian volume and roundabout geometry were used from data collection.

Table 15 Pedestrian Green Time Requirement

Vehicle green time (sec)		Pedestrian green time (sec)		Formula	Check Requirement
Gerji Emperial Roundabout					
G_{VB}	48	G_{PG}	27	$G_{PG} \leq G_{VB} + Y_{VB}$	Okay
G_{VG}	50	G_{PM}	30	$G_{PM} \leq G_{VG} + Y_{VG}$	Okay
G_{V24}	36	G_{PB}	33	$G_{PB} \leq G_{V24} + Y_{V24}$	Okay
G_{VM}	57	G_{P24}	23	$G_{P24} \leq G_{VM} + Y_{VM}$	Okay
Bole Mikael Roundabout					
G_{VB}	51	G_{PR}	25	$G_{PR} \leq G_{VB} + Y_{VB}$	Okay
G_{VK}	55	G_{PBU}	31	$G_{PBU} \leq G_{VK} + Y_{VK}$	Okay
G_{VR}	30	G_{PK}	30	$G_{PK} \leq G_{VR} + Y_{VR}$	Okay
G_{VBU}	41	G_{PB}	29	$G_{PB} \leq G_{VBU} + Y_{VBU}$	Okay
Saris Abo Roundabout					
G_{VK}	76	G_{PS}	43	$G_{PS} \leq G_{VK} + Y_{VK}$	Okay
G_{VMA}	78	G_{PAC}	54	$G_{PAC} \leq G_{VM} + Y_{VM}$	Okay
G_{VAC}	62	G_{PK}	107	$G_{PK} \leq G_{VAC} + Y_{VAC}$	Not Okay
G_{VS}	25	G_{PMA}	90	$G_{PM} \leq G_{VS} + Y_{VS}$	Not Okay

4.5. Pedestrian Traffic Volume

Pedestrian count is conducted for 4 hours per day for one day during AM and PM peak hour period for 15 min manually. This study was conducted from 8:00 AM to 10:00 AM in the morning and from 4:00PM to 6:00 PM in the afternoon.

Table 16 is pedestrian volume count summary result at Gerji Emperial roundabout. Refer Appendix D for the rest of study areas.

Table 16 Pedestrian Volume Count Study at Gerji Emperial Roundabout

<table>
<tr><td rowspan="18">Gerji Emperial Roundabout</td><td colspan="5">Pedestrian volume study 8:00 AM – 10:00 AM</td></tr>
<tr><td rowspan="2">Approaches</td><td colspan="2">8:00 AM – 9:00 AM</td><td colspan="2">9:00 AM – 10:00 AM</td></tr>
<tr><td>15 minutes</td><td>Ped/h</td><td>15 minutes</td><td>Ped/h</td></tr>
<tr><td>Bole</td><td>161</td><td>644</td><td>192</td><td>768</td></tr>
<tr><td>Megenagna</td><td>103</td><td>412</td><td>111</td><td>444</td></tr>
<tr><td>Gerji</td><td>71</td><td>284</td><td>76</td><td>304</td></tr>
<tr><td>24 Street</td><td>24</td><td>96</td><td>35</td><td>140</td></tr>
<tr><td colspan="2">Total</td><td>1436</td><td></td><td>1656</td></tr>
<tr><td colspan="5">Pedestrian volume study 4:00 PM – 6:00 PM</td></tr>
<tr><td rowspan="2">Approaches</td><td colspan="2">4:00 PM – 5:00 PM</td><td colspan="2">5:00 PM – 6:00 PM</td></tr>
<tr><td>15 minutes</td><td>Ped/h</td><td>15 minutes</td><td>Ped/h</td></tr>
<tr><td>Bole</td><td>238</td><td>952</td><td>207</td><td>828</td></tr>
<tr><td>Megenagna</td><td>103</td><td>412</td><td>154</td><td>616</td></tr>
<tr><td>Gerji</td><td>70</td><td>280</td><td>92</td><td>368</td></tr>
<tr><td>24 Street</td><td>24</td><td>96</td><td>20</td><td>80</td></tr>
<tr><td colspan="2">Total</td><td>1740</td><td></td><td>1892</td></tr>
</table>

Chapter 5: Result and Discussion

After compiling and analyzing the collected data in the study, VISTRO and VISSIM tools result is presented as follow with a detail discussion, figurative and tabular illustrations. Study results were relaying on comparing three possible alternatives under base and future condition. Those alternatives are roundabout as it is, changing roundabout into pre-timed signalized intersection and supporting roundabout by traffic signal during peak hour time which called roundabout signalization. Future condition is predicted for 5 year on average constant growth rate of 4.5 percent.

Roundabout as it is and pre timed signalized intersection for base and future condition is evaluated using VISTRO tool and roundabout with traffic signal at peak hour is evaluated using VISSIM simulation modeling tool. Figure 21 indicates, roundabout signalization modeling at Gerji Emperial roundabout. All the output data is on appendix D and result summaries are presented in figure and table as follow on their respective headings.

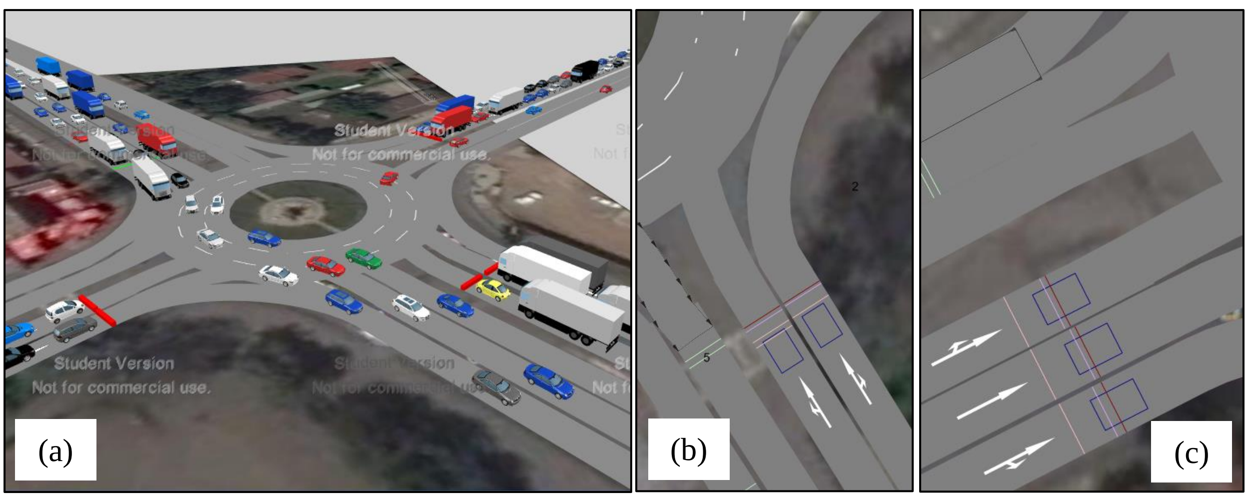


Figure 21 (a) Signalized Roundabout Simulation at Gerji Emperial Roundabout, (b) Shared Lane Configuration on Minor Approaches and (c) Shared Lane Configuration on Major Approaches

5.1. Gerji Emperial Roundabout

Gerji Emperial roundabout performance evaluation, approach delay, queue length and LOS results are stated in, Figure 22, Figure 23 and Table 17 for three alternatives under base and future condition respectively. Approaching delay show significant change, but LOS doesn't. Therefore according to this study finding at Gerji Emperial roundabout during peak hour time alternative 2 and 3 reduces sections control delay by 34%, 47% under base and 52%, 69% future condition respectively.

Alternative 1 (Roundabout as it is)

If Gerji Emperial roundabout stays as it is without any interventions techniques including traffic police for the next five year its control delay become 913.49 sec/veh during peak hour which means delay increases by 125.88% from base condition and 153.23 m queue length increment is observed from base condition as compared to future condition.

Alternative 2 (As pre-timed signalized intersection)

If Gerji Emperial roundabout changed to pre-timed signalized intersection stays for the next five year its control delay become 437.56 sec/veh during peak hour which means delay increases by 64.89% from base condition.

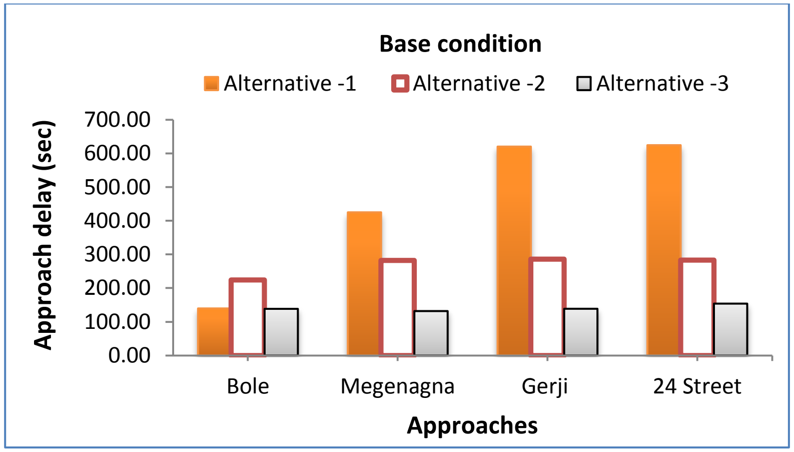
Considering roundabout queue length as reference this alternative increases sections average queue length. On average alternative 2 has 91m and 132.17 m queue length increment is observed as compared to roundabout as it is for base and future condition respectively.

Alternative 3 (Roundabout signalization)

If Gerji Emperial roundabout changed in to roundabout signalization and stays for the next five year its control delay become137.77 sec/veh during peak hour which means delay decreases 2.09% from base condition.

Delay improvement on this regard indicates three things. First one is signal timing is not sensitive in terms of control delay for next five year and secondly current signal timing design can accommodate future traffic volume and thirdly pedestrian impedance is not considered as simulation part.

Considering roundabout queue length as reference this alternative decreases sections average queue length. On average alternative 3 has 145.29m and 234.64 m queue length decrement is observed as compared to roundabout as it is for base and future condition respectively. Detail queue length comparison is illustrated on

(a) 

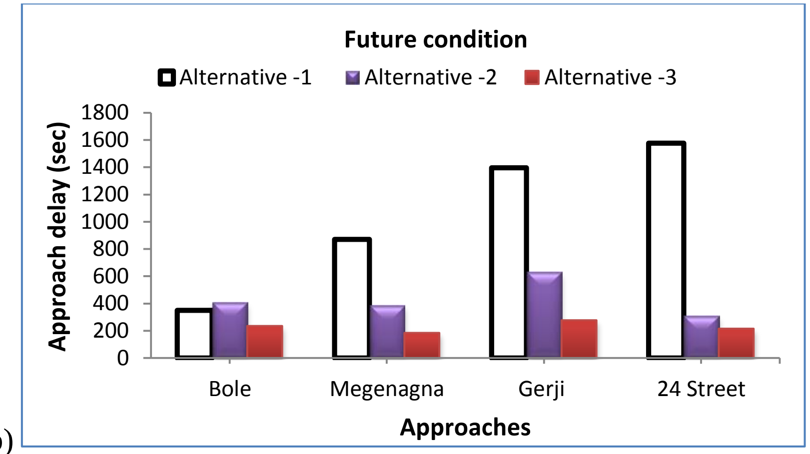
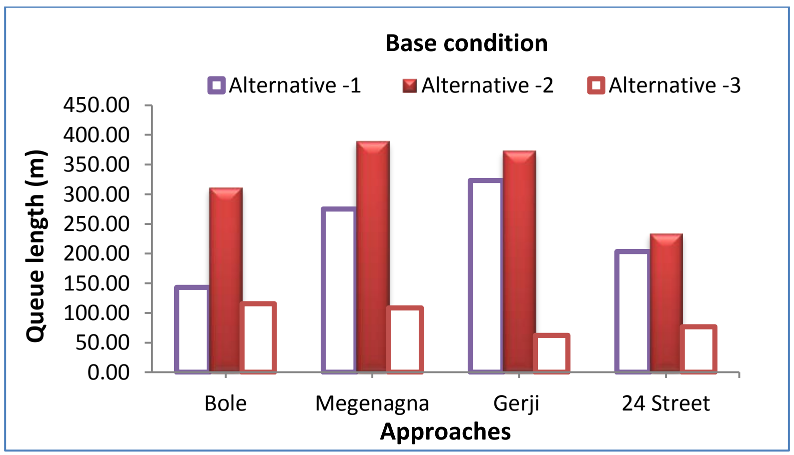
(b) 

Figure 22 Approaching Control Delay under (a) Base and (b) Future Condition at Gerji Emperial Roundabout

(a) 

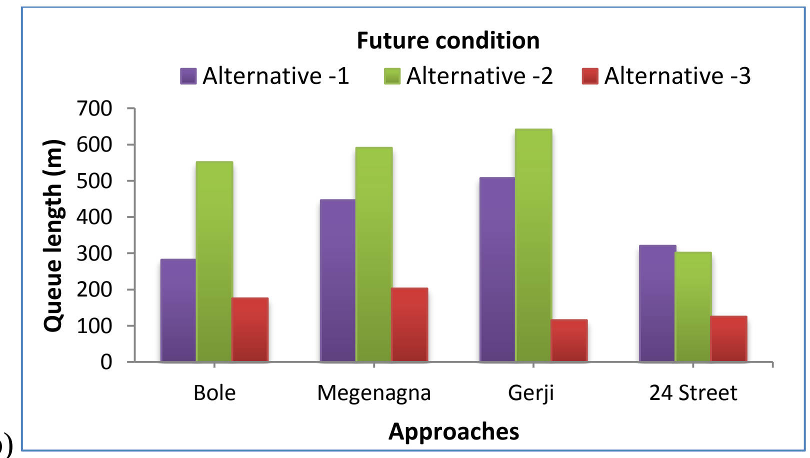
(b) 

Figure 23 Approaching Queue Length under (a) Base and (b) Future Condition at Gerji Emperial Roundabout

Table 17 Approaching LOS Evaluation Result, (a) Alternative 1, (b) Alternative 2 and (c) Alternative 3 at Gerji Emperial Roundabout

Alternative 1	Base Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	A	F	F	C	F	F	F		F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	B	F	F	D	F	F	F		F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(a)

Alternative 2	Base Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F	F	F	F	F	F	F		F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F	F	F	F	F	F	F		F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(b)

Alternative 3	Base Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F	F	F	F	F	F	F		F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F	F	F	F	F	F	F		F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(c)

5.2. Bole Mikael Roundabout

Bole Mikael roundabout performance evaluation (approach delay, queue length and LOS) results are stated in Figure 24, Figure 25 and Table 18, for three alternatives under base and future condition respectively. Approaching delay show significant change, but LOS doesn’t. Therefore according to this study finding at Bole Mikael roundabout during peak hour time alternative 2 and 3 reduces sections control delay by 54%, 20% under base and 58%, 57% future condition respectively.

Alternative 1 (Roundabout as it is)

If Bole Mikael roundabout stays as it is without any interventions techniques including traffic police for the next five year its control delay become 707.67 sec/veh during peak hour which means delay increases by 101.51% from base condition and 183.20 m queue length increment is observed from bas condition as compared to future condition.

Alternative 2 (As pre-timed signalized intersection)

If Bole Mikael roundabout changed to pre-timed signalized intersection stays for the next five year its control delay become 300.34 sec/veh during peak hour which means delay increases by 86.19% from base condition.

Considering roundabout queue length as reference this alternative increases sections average queue length. On average alternative 2 has 24.51m and 86.70m queue length increment is observed as compared to roundabout as it is for base and future condition respectively.

Alternative 3 (Roundabout signalization)

If Gerji Emperial roundabout changed in to roundabout signalization and stays for the next five year its control delay become 129.18sec/veh during peak hour which means delay increases 0.12% from base condition.

Delay improvement on this regard indicates three things. First one is signal timing is not sensitive in terms of control delay for next five year and secondly current signal timing design can accommodate future traffic volume and thirdly pedestrian impedance is not considered as simulation part.

Considering roundabout queue length as reference this alternative decreases sections average queue length. On average alternative 3 has 208.79m and 391.58 m queue length decrement is observed as compared to roundabout as it is for base and future condition respectively. Detail queue length comparison is illustrated on

.

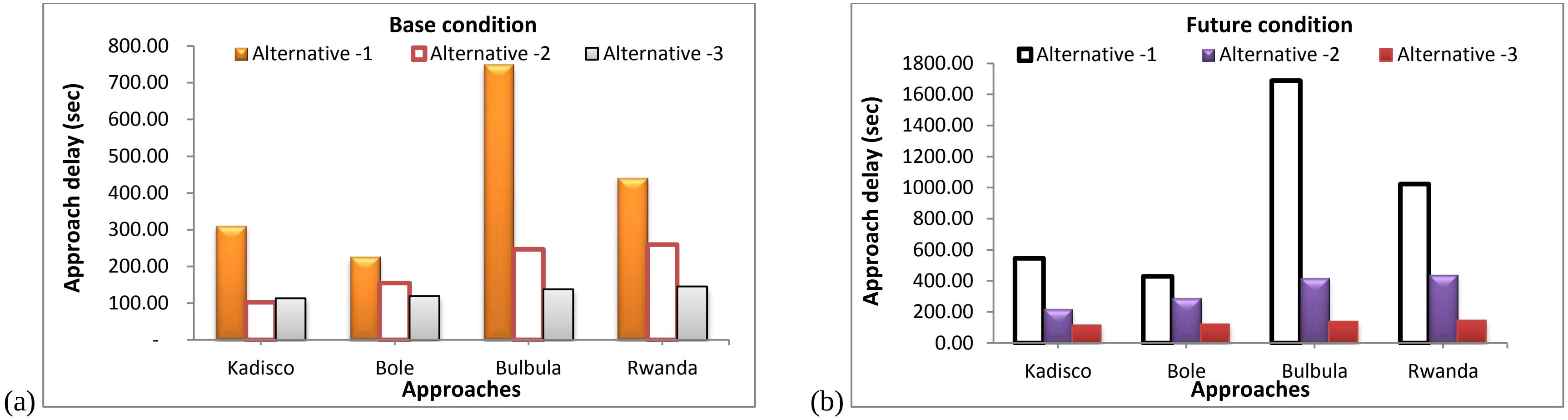


Figure 24 Approach Control Delay under (a) Base and (b) Future Condition at Bole Mikael Roundabout

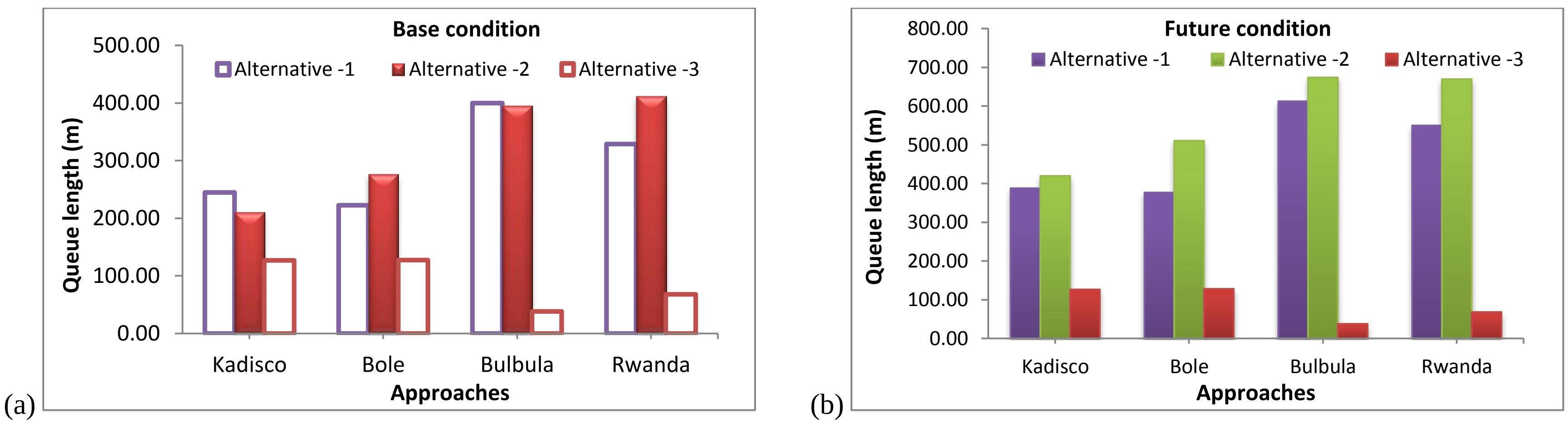


Figure 25 Approach Queue Length under (a) Base and (b) Future Condition

Table 18 Approaching LOS Evaluation Result, (a) Alternative 1, (b) Alternative 2 and (c) Alternative 3 at Bole Mikael Roundabout

Alternative 1	Base Condition												
	Approaches	Kadisco			Bole			Bulbula			Rwanda		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	B	F	F	A	F	F	F			F		
	Approach LOS	F			F			F			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	B	F	F	D	F	F	F		F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(a)

Alternative 2	Base Condition												
	Approaches	Kadisco			Bole			Bulbula			Rwanda		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F	F	F	F	F	F	F			F		
	Approach LOS	F			F			F			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	B	F	F	D	F	F	F			F		
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(b)

Alternative 3	Base Condition												
	Approaches	Kadisco			Bole			Bulbula			Rwanda		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F	F	F	F	F	F	F			F		
	Approach LOS	F			F			F			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	B	F	F	D	F	F	F			F		
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(c)

5.3. Saris Abo Roundabout

Saris Abo roundabout performance evaluation (approach delay, queue length and LOS) results are stated in Figure 26, Figure 27 and Table 19 Three alternatives under base and future condition respectively. Approaching delay show significant change, but LOS doesn't. Therefore according to this study finding at Saris Abo roundabout during peak hour time alternative 2 and 3 reduces sections control delay by 40%, 13% under base and 17%, 67% future condition respectively.

Alternative 1 (Roundabout as it is)

If Saris Abo roundabout stays as it is without any interventions techniques including traffic police for the next five year its control delay become 635.14 sec/veh during peak hour which means delay increases by 100.56% from base condition and 128.71 m queue length increment is observed from bas condition as compared to future condition.

Alternative 2 (As pre-timed signalized intersection)

If Saris Abo roundabout changed to pre-timed signalized intersection stays for the next five year its control delay become 529.23 sec/veh during peak hour which means delay increases by 180.02% from base condition.

Considering roundabout queue length as reference this alternative increases sections average queue length. On average alternative 2 has 48.88 m and 212.88 m queue length increment is observed as compared to roundabout as it is for base and future condition respectively.

Alternative 3 (Roundabout signalization)

If Saris Abo roundabout changed in to roundabout signalization and stays for the next five year its control delay become176.36 sec/veh during peak hour which means delay increases 7.85% from base condition.

Delay improvement on this regard indicates two things. First one is signal timing is not sensitive in terms of control for next five year and secondly current signal timing design can accommodate future traffic volume.

Considering roundabout queue length as reference this alternative decreases sections average queue length. On average alternative 3 has 79.85 m and 206.70 m queue length decrement is observed as compared to roundabout as it is for base and future condition respectively. Detail queue length comparison is illustrated in Figure 27.

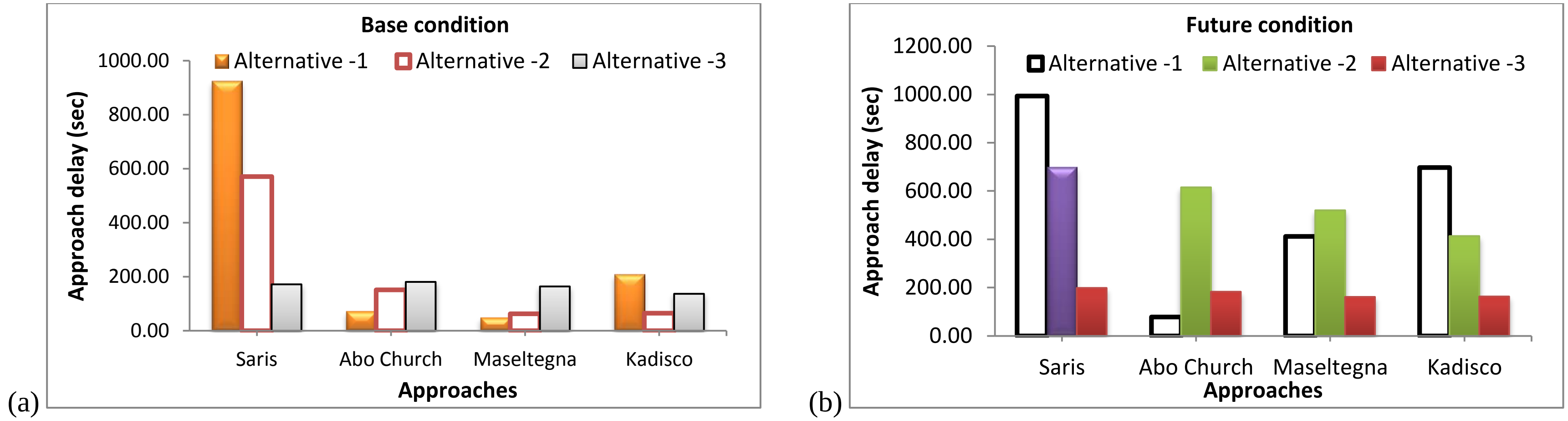


Figure 26 Approach Control Delay under (a) Base and (b) Future Condition at Saris Abo Roundabout

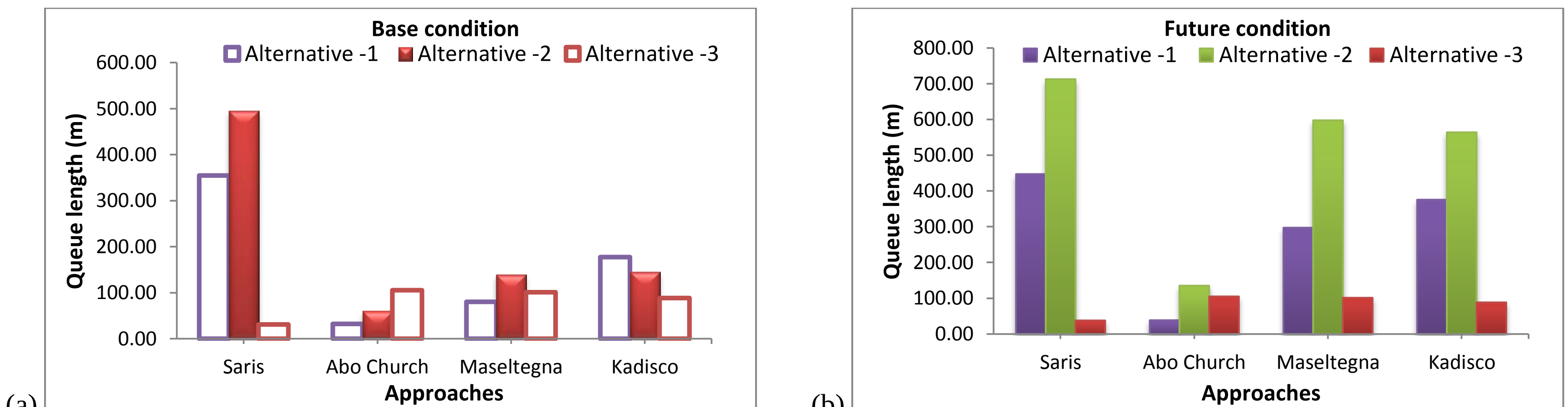


Figure 27 Approaching Queue Length under (a) Base and (b) Future Condition

Table 19 Approaching LOS Evaluation Result, (a) Alternative 1, (b) Alternative 2 and (c) Alternative 3 at Saris Abo Roundabout

Alternative 1													
	Base Condition												
	Approaches	Saris			Abo Church			Maseltegna			kadisco		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F		F	F		E	A	F	F	B	F	F
	Approach LOS	F			F			E			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F		F	F		E	F	F	B	F	F	E
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(a)

Alternative 2													
	Base Condition												
	Approaches	Saris			Abo Church			Maseltegna			kadisco		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F		F	F		F	B	B	F	C	F	F
	Approach LOS	F			F			E			E		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F		F	F		C	F	F	C	F	F	D
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(b)

Alternative 3	Base Condition												
	Approaches	Saris			Abo Church			Maseltegna			kadisco		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F		F	F		F	F	F	F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											
	Future Condition												
	Approaches	Bole			Megenagna			Gerji			24 street		
		Northeast bound			Southwest bound			Northwest bound			Southeast bound		
	Movement	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT	L&UT	T	RT
	Lane LOS	F		F	F		F	F	F	F	F		F
	Approach LOS	F			F			F			F		
	Intersection LOS	F											

(c)

5.4. Summary and Discussion

Roundabouts have in excess of control delay relatively than those two alternative solutions. Changing roundabout in to signalized intersection and roundabout signalization in terms of control delay and queue length for both base and future conditions have significant reduction but LOS doesn’t. LOS standard is too high and it need well improved situation to make change. Here are result summary and discussions on each alternative in general.

Alternative 1

Control delay is in excess under future condition, which indicates that roundabout is sensitive for traffic volume increment. From its operation base roundabout has a rule of giving priority but higher amount of traffic volume causes minor roads to stay on their approach for longer time. Therefore on this regard drivers patience is in danger leads to aggressively join the intersection.

Now on control delay increment is not the only treat for this roundabout but also traffic flow management, which means due to stop and go movement and excessive delay giving priority rule doesn’t work out properly therefore frequently dead lock (circle jam) traffic situation is created. This situation is not modeled but from field visit this kind of traffic jam didn’t dissipate by itself it needs interventions. Therefore this alternative is the least solution among the alternative solutions.

Alternative 2

Changing roundabout in to pre-timed intersection signalization has positive result in terms of control delay reduction but it increases sections queue length. This alternative has two limitation first, during off-pick hour and signal malfunctioning (due to luck of sustainable energy in terms of power or other reason) this area may experience excessive control delay and prolonged queue length. Secondly traffic flow safety is compromised which means accident is worst relatively to other alternatives. Therefore this alternative is the second choice for base and future condition.

Alternative 3

Roundabout signalization is has two alternatives; during off-peak hour roundabout operates as usual and during peak hour roundabout signalizations take a step to regulate traffic flow. Results show that roundabout signalization has a very positive result as compared to the above two alternatives.

During roundabout signalization all lanes except the middle lanes are shared lane movement from through vehicle movement as indicated on Figure 21. Shared lane configuration reduces delay in general and balance queue length on each approach. But specifically left turn and right turn have increase their delay and queue length relatively but decrease for trough vehicle movement. Figure 15 clearly shows that through vehicle dominancy was in excess for major road approaches on the other side roundabout geometry make it difficult to make right turn always flows (RTOR) or by pass because of it needs extra infrastructure and isolated lane grouping and if we give one lane for right turn vehicles especially minor roads suffer from driving lane in adequacy and also it is impossible for bole Mikael roundabout, since minor roads have one lane. Therefore this alternative uses split signal phasing sequence as best optimization techniques for roundabout signalization during peak hour so as to make roundabout flow compatible with the allocated signal timing.

Therefore Alternative 2 limitations can be accommodated by roundabout by itself. Roundabout signalization is the first alternative choice as compared to signalized intersection during peak hour time for both base and future condition when we focus only on control delay and also roundabout signalization is important in terms of off peak hour traffic flow condition.

Chapter 6: Conclusion, Recommendation and Future Study

6.1. Conclusion

Study area roundabouts data collection and analysis results indicate most of the legs of roundabouts are in serious problems of congestion up to deadlock situations. Based on observed actual field conditions it is common to see that at peak hours, the traffic police need to regulate traffic flow on those roundabouts as a traffic control devices.

Apart from traffic volume survey at the study area, roundabouts were built more than 10 years ago and it is clearly observed that current conditions were difficult to entertain by those roundabouts.

All the collected data were analyzed by excel spreadsheet and traffic simulation tools (VISSIM and VISTRO) using HCM 2010 as a reference.

High traffic entry flows at Gerji Emperial, Bole Mikael and Saris Abo roundabout was 4332 veh/h, 4560 veh/h and 4008 veh/h respectively. This traffic is very high to be accommodated by the roundabout. In addition there was high percentage of heavy vehicle at rush hour time from major road approaches which is greater than 10%. Higher traffic volume and heavy vehicle proportioning leads to LOS F and control delay more than 250 sec on study areas on base condition. Therefore the study areas are in critical condition. If those study areas continue being roundabout without any intervention techniques and traffic police for next five year, at the end five year, during peak hour time under ideal (no crash and smooth pavement condition) condition the study area will experience an average control delay of 991.86 sec/veh.

Roundabouts have in excess of control delay and higher travel time relatively than those two alternative solutions. Changing roundabout in to signalized intersection and roundabout signalization reduces its delay by 34% and 47% from the previous under base and 52% and 69% under forecasted condition for peak hour traffic flow respectively. Therefore the proposed engineering solutions enhance and improve selected roundabouts sections.

According to this study analysis and finding signalization of roundabout worth for the effective flow of traffic on study areas at peak hour time and during off peak hour time roundabout by itself will regulate as usual without signal timing. Roundabout signalization will give relatively minimized control delay at rush hour time and safety during off peak hour time. Higher variability of traffic volume during peak and off peak hour is one problem for demolishing roundabout.

6.2. Recommendation

- This study supports provision of signalized roundabout but have limitation of VISSIM modeling doesn't incorporated real flow situations (pedestrian impedance, pavement ideal condition, aggressive drivers' behavior, and non-lane based flows). Therefore temporary signal timing will be provided and counter check modeling scenarios feet with real situation so that we can adopt permanent signal timing for study locations.
- Saris Abo Roundabout experience excess amount of pedestrian volume which leads to high level of conflict between vehicles and pedestrian at the approach crossing roundabout areas. This study shows how much it is difficulty to entertain it by vehicular green time. So care should be considered in treating pedestrian volume on the roundabout crossing section. Therefore this study will recommend grade separated pedestrian crossing on roundabout approaches
- This study has a two possible alternative solution. Those solutions really based on ideal pavement condition and in the absence of aggressive drivers behavior. Therefore pavement marking and rehabilitation should be provided beside the provision of signal timing so as to have simulation result in real condition.
- For consistent and accurate data analysis and result representation, there should be frequent study on the selected study areas to minimize risk of traffic volume variability in a decisions making process.
- In most of the approaches heavy vehicle percent proportioning is more than 10%. This value has a great role in capacity reduction of study areas to incur excessive delay. Therefore heavy vehicles should be allocated to drive in relation to off-peak hour.

6.3. Future Study

- VISSIM simulation modeling calibration so that we do have a mode of non-lane based simulation, which exactly represents developing countries driving trend.
- Developing HCM and MUTCD manuals for Ethiopia which exactly represent Ethiopian condition so as to make it stop comparing result and take reference from them. For instance

control delay for signalized intersection greater than 80 sec gives LOS F for USA but it may not be F in Ethiopia condition.

- Grade separated road intersection around congested roundabout sections, simulation modeling and its best advantage apart from its huge infrastructure and land use pattern.
- Modeling and evaluation of temporary interventions on congested roundabouts so as to use it whenever it is necessary

Reference

Addis Ababa City Roads Authority (AACRA) Geometric Design Manual, January 2003.

Amudapuram Mohan Rao and Kalaga Ramachandra Rao "Measuring Urban Traffic Congestion, International Journal for Traffic and Transport Engineering", 2012, 2(4): 286 – 305. Central Road Research Institute, Mathura Road, PO CRRI, New Delhi - 110 020, India. DOI: http://dx.doi.org/10.7708/ije.2012.2 (4).01

Duane Smith, Jeff McIntyre, Mark Anderson-Wilk and Sarah Moreau "Handbook of Simplified Practice for Traffic Studies" (November 2002) Center for Transportation Research and Education Iowa State University

European Conference of Ministers of Transport (ECMT), 2007 "Managing Urban Traffic Congestion", European Conference of Ministers of Transport. ISBN 978-92-821-0128-5.

Ewing, R. 1999. Traffic Calming Impacts. In Traffic Calming: State and Practice. Washington, D.C.: Institute of Transportation Engineers,

FHWA. 2001. Manual on Uniform Traffic Control Devices: Millennium Edition. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation.

Federal Highway Administration (FHWA)"Modern Roundabouts, an Informational Guide". Research and Technology. Retrieved 15 September 2015.

"Florida Roundabout Guide". Florida Department of Transportation, Available (FDT) from: http://www.dot.state.fl.us/TrafficOperations/Doc_Library/PDF/roundabout_ guide8_07 pdf.

"Highway capacity manual" (HCM) transportation research board 2000, executive committee.

"Highway capacity manual" (HCM), Transportation research board 2010, executive committee

Homburger, W.S., J. W. Hall, R. C. Loutzenheiser, and W. R. Reilly. 1996. Spot Speed Studies. In Fundamentals of Traffic Engineering. Berkeley: Institute of Transportation Studies, University of California, Berkeley,

Huddart, K.W. "Signalling of Hyde Park Corner. Elephant and Castle and other Roundabouts". In PTRC 11th Summer Annual Meeting. Proceedings of Seminars K (Traffic Operation and Management). 1983.

Insurance Institute for Highway Safety Highway Loss Data Institute. (2016) IIHSHLDI: Highway Safety Research and Communication: HSRC. [Online] available from: http://www.iihs.org/iihs/topics/t/roundabouts/qand/ (April, 2016),

Manual on Uniform Traffic Control Devices (MUTCD) Millennium Edition, Federal Highway Administration, U.S. Department of Transportation, Washington DC, 2000.

Marjan Mosslemi, (December 2008). "Using Metering Signals at roundabouts with Unbalanced Flows to Improve the Traffic Condition" (A Case Study of Kannik Area in Stavanger) Linköping University SE-601 74 Norrköping, Sweden.

Nicholas J. Garber and Lester A. Hoel (2009) "Traffic and Highway Engineering (4th edition)" University of Virginia ISBN-10: 0-495-08250-3

PTV VISSIM 8 user manual (2015) PTV AG, Karlsruhe, Germany info@vision.ptvgroup.com, www.ptvgroup.com and vision-traffic.ptvgroup.com

PTV VISTRO user manual (2016) PTV AG, Karlsruhe Germany info@vision.ptvgroup.com, www.ptvgroup.com and vision-traffic.ptvgroup.com

R. Mallikarjuna Roll No. 212CE3051 (2012-2014), "Operational Analysis of roundabouts under Mixed Traffic Flow Condition" National Institute Of Technology Rourkela-769008 department of civil engineering national institute of technology rourkela-769008

Robertson, H. D. 1994. Spot Speed Studies. In Manual of Transportation Engineering Studies, ed. H. D. Robertson, J. E. Hummer, D. C. Nelson. Englewood Cliffs, N.J.: Prentice Hall, Inc.

Roger P. roess, Elena S. Prassas and William R. McShane USA, "Traffic Engineering Book 3rd edition" 2004)

Sharma, S. C. 1994. Seasonal Traffic Counts for a Precise Estimation of AADT. ITE Journal,Vol. 64, No. 9

Tan J. "Comparison of Capacity & between Roundabout Design and Signalized Junction Design". In Proceedings of 1st Swiss Transportation Research Conference March 2001. Monte Verita/Ascona.

Wondwossen T. (September, 2011). "Assessing and Quantifying the Level of Traffic Congestions at Major Intersections in Addis Ababa" (A case for east west corridor), Addis Ababa university school of civil engineering.

Appendix A: Spot Speed

Table 10 (a) Spot Speed Study Analysis at Gerji Emperial Roundabout, Megenagna Approach

Class	Speed class km/hr	Class mid value, U_i (km/hr)	Class mid value, U_i (mil/hr)	Class frequency fi (number of observation)	fi*Ui	Percentage of observations in the class	Cumulative percentage of observations	$f*(u_i - \hat{U})^2$
1	24-25.99	25	17.0	2	50	2.0%	2%	398.749
2	26-27.99	27	18.4	2	54	2.0%	4%	293.789
3	28-29.99	29	19.7	3	87	3.0%	7%	307.243
4	30-31.99	31	21.1	5	155	5.0%	12%	329.672
5	32-33.99	33	22.4	6	198	6.0%	18%	224.726
6	34-35.99	35	23.8	10	350	10.0%	28%	169.744
7	36-37.99	37	25.2	17	629	17.0%	45%	76.405
8	38-39.99	39	26.5	18	702	18.0%	63%	0.259
9	40-41.99	41	27.9	8	328	8.0%	71%	28.275
10	42-43.99	43	29.3	7	301	7.0%	78%	105.381
11	44-45.99	45	30.6	5	225	5.0%	83%	172.872
12	46-47.99	47	32.0	9	423	9.0%	92%	558.850
13	48-49.99	49	33.3	2	98	2.0%	94%	195.229
14	50-51.99	51	34.7	3	153	3.0%	97%	423.403
15	52-53.99	53	36.1	3	159	3.0%	100%	577.963
	Total	585	398	100	3912	100.0%		3862.560

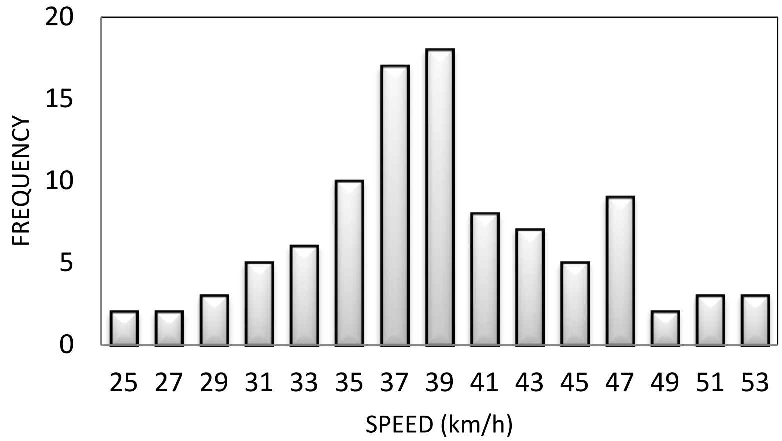
Table 10 (b) Spot Speed Study Analysis at Gerji Emperial Roundabout, Gerji Approach

Class	Speed class km/hr	Class mid value, U_i (km/hr)	Class mid value, U_i (mil/hr)	Class frequency fi (number of observation)	fi*Ui	Percentage of observations in the class	Cumulative percentage of observations	$f^*(u_i - \hat{U})^2$
1	13-13.99	13.5	9.2	2	27.0	2.0%	2%	107.165
2	14-14.99	14.5	9.9	4	58.0	4.0%	6%	159.770
3	15-15.99	15.5	10.5	7	108.5	7.0%	13%	198.117
4	16-16.99	16.5	11.2	4	66.0	4.0%	17%	74.650
5	17-17.99	17.5	11.9	6	105.0	6.0%	23%	66.134
6	18-18.99	18.5	12.6	9	166.5	9.0%	32%	48.442
7	19-19.99	19.5	13.3	11	214.5	11.0%	43%	19.166
8	20-20.99	20.5	13.9	11	225.5	11.0%	54%	1.126
9	21-21.99	21.5	14.6	7	150.5	7.0%	61%	3.237
10	22-22.99	22.5	15.3	12	270.0	12.0%	73%	33.869
11	23-23.99	23.5	16.0	9	211.5	9.0%	82%	64.642
12	24-24.99	24.5	16.7	5	122.5	5.0%	87%	67.712
13	25-25.99	25.5	17.3	4	102.0	4.0%	91%	87.610
14	26-26.99	26.5	18.0	2	53.0	2.0%	93%	64.525
15	27-27.99	27.5	18.7	2	55.0	2.0%	95%	89.245
16	28-28.99	28.5	19.4	2	57.0	2.0%	97%	117.965
17	29-29.99	29.5	20.1	2	59.0	2.0%	99%	150.685
18	30-30.99	30.5	20.7	1	30.5	1.0%	100%	93.702
	Total	353	240	100	2082.0	100%		1447.760

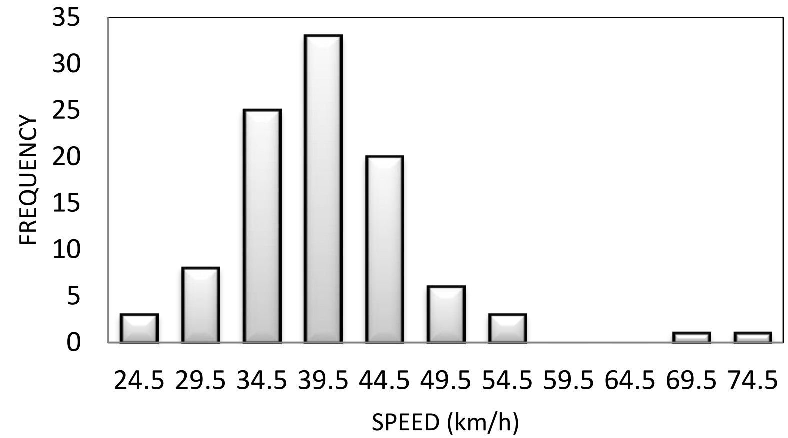
Table 10 (c) Spot Speed Study Analysis at Gerji Emperial Roundabout, Bole Approach

Class	Speed class km/hr	Class mid value, U_i (km/hr)	Class mid value, U_i (mil/hr)	Class frequency fi (number of observation)	fi*Ui	Percentage of observations in the class	Cumulative percentage of observations	$f*(u_i - \hat{U})^2$
1	22-26.99	24.5	16.7	3	73.5	3.0%	3%	693.120
2	27-31.99	29.5	20.1	8	236.0	8.0%	11%	832.320
3	32-36.99	34.5	23.5	25	862.5	25.0%	36%	676.000
4	37-41.99	39.5	26.9	33	1303.5	33.0%	69%	1.320
5	42-46.99	44.5	30.3	20	890.0	20.0%	89%	460.800
6	47-51.99	49.5	33.7	6	297.0	6.0%	95%	576.240
7	52-56.99	54.5	37.1	3	163.5	3.0%	98%	657.120
8	57-61.99	59.5	40.5	0	0.0	0.0%	98%	0.000
9	62-66.99	64.5	43.9	0	0.0	0.0%	98%	0.000
10	67-71.99	69.5	47.3	1	69.5	1.0%	99%	888.040
11	72-76.99	74.5	50.7	1	74.5	1.0%	100%	1211.040
	Total			100	3970	100%		5996.00

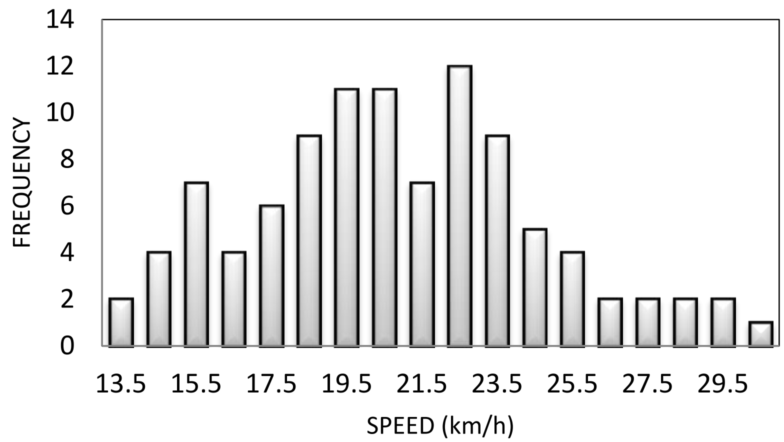
Appendix B: Spot Speed Study Frequency Distribution Histogram



(a)



(b)



(c)

Figure 10 Spot Speed Study Frequency Distributions at Gerji Emperial Roundabout, (a) Megenagna (b) Bole and (c) Gerji Approach

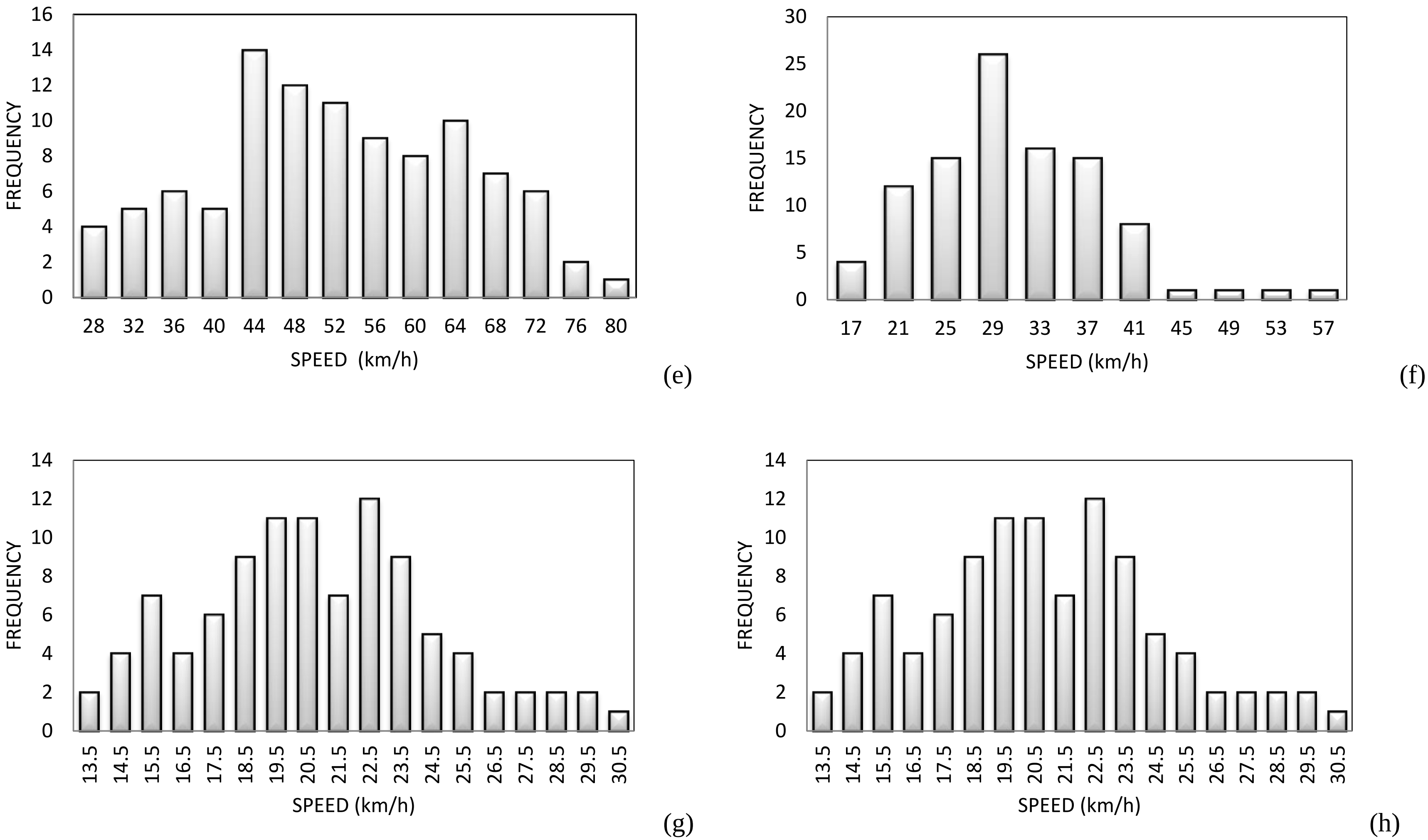
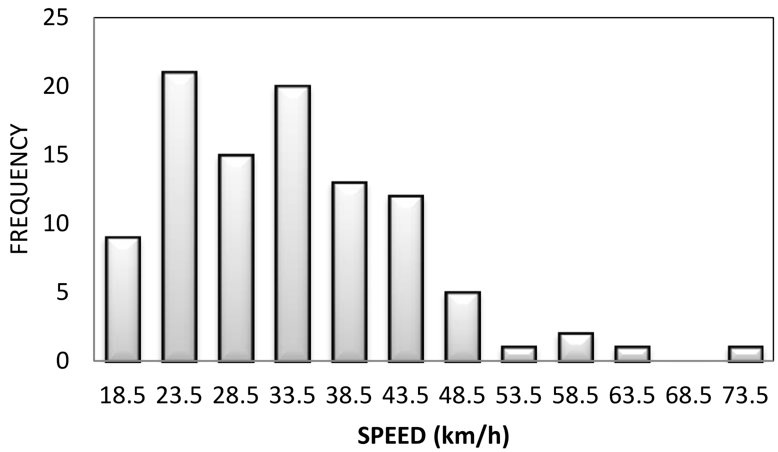
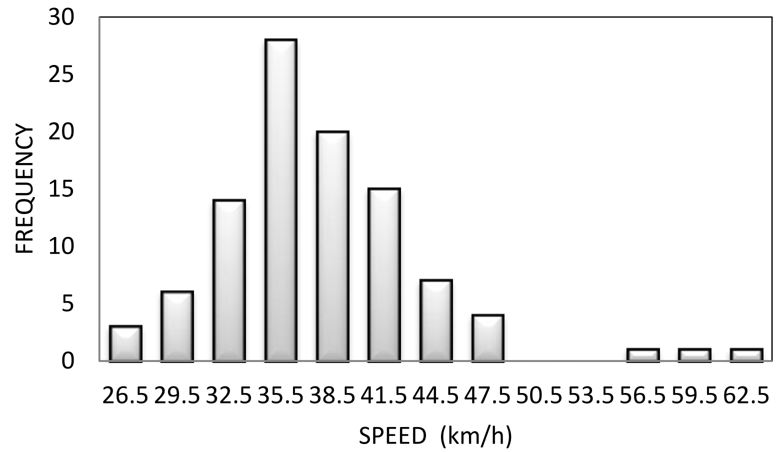


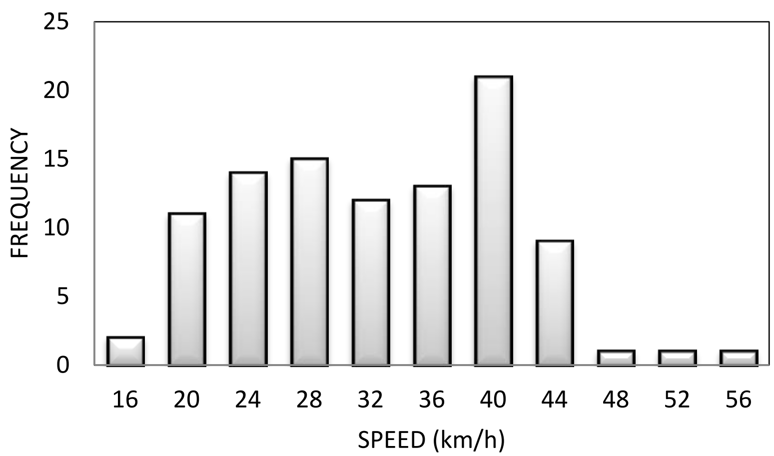
Figure 10 Spot Speed Study Frequency Distributions at Bole Mikael Roundabout, (e) Bole (f) Kadisco (g) Rwanda and (h) Bulbula Approach



(i)



(j)



(k)

Figure 10 Spot Speed Study Frequency Distributions at Saris Abo Roundabout, (i) Maseltegna (j) Kadisco and (k) Abo Church Approach

Appendix C: Traffic Volume

APPROACH	MOVEMENT (8AM – 9AM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	218	19	10	2	2	2	16	0	0	0	0	0	29	0	3	1	0	0	3	0	1	0	0	0
MEGENAGNA	286	18	6	2	1	1	41	2	0	0	0	0	26	2	1	0	0	0	4	2	2	0	0	0
GERJI	69	4	1	0	0	0	85	2	0	0	0	0	42	4	0	0	0	3	0	0	0	0	0	0
24 STREET	42	0	0	0	0	0	40	3	0	1	0	1	39	0	1	0	0	0	0	0	0	0	0	0

APPROACH	MOVEMENT (9AM – 10AM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	227	19	13	10	5	2	19	3	2	1	0	0	23	4	4	1	0	4	2	1	0	0	0	0
MEGENAGNA	234	22	19	10	5	2	18	2	4	1	0	0	29	3	4	0	2	0	5	2	1	0	0	0
GERJI	63	2	14	4	3	1	78	5	10	3	1	2	37	8	5	2	1	0	0	0	0	0	0	0
24 STREET	28	2	7	5	0	6	22	2	2	7	0	0	28	4	15	3	0	4	0	0	0	0	0	0

APPROACH	MOVEMENT (10AM – 11AM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	218	19	10	2	2	2	16	0	0	0	0	0	29	0	3	1	0	0	3	0	1	0	0	0
MEGENAGNA	286	18	6	2	1	1	41	2	0	0	0	0	26	2	1	0	0	0	4	2	2	0	0	0
GERJI	69	4	1	0	0	0	85	2	0	0	0	0	42	4	0	0	0	3	0	0	0	0	0	0
24 STREET	42	0	0	0	0	0	40	3	0	1	0	1	39	0	1	0	0	0	0	0	0	0	0	0

APPROACH	MOVEMENT (11AM – 12AM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	186	9	17	6	4	3	16	1	2	1	1	0	36	1	2	0	0	1	4	1	1	1	0	0
MEGENAGNA	205	14	11	9	4	0	43	1	1	1	0	0	43	0	1	0	0	0	11	1	1	0	0	0
GERJI	49	0	1	1	0	1	65	1	4	1	0	1	52	1	1	0	0	0	0	0	0	0	0	0
24 STREET	33	0	1	2	0	0	33	1	2	0	0	1	27	0	4	2	0	1	0	0	0	0	0	0

APPROACH	MOVEMENT (12AM – 1PM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	111	8	7	5	3	2	13	0	2	1	0	0	21	0	2	0	0	1	3	3	1	1	0	0
MEGENAGNA	102	12	9	8	3	0	34	1	1	0	0	0	26	0	0	0	0	0	9	1	1	0	0	0
GERJI	38	0	1	1	0	1	41	0	4	1	0	1	35	1	1	0	0	0	4	0	2	0	1	0
24 STREET	29	0	1	2	0	0	22	1	2	0	0	1	22	0	3	2	0	1	1	2	2	1	0	0

APPROACH	MOVEMENT (1PM – 2PM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	108	5	7	4	4	2	9	0	2	2	1	1	21	1	2	0	0	2	2	1	1	1	0	0
MEGENAGNA	123	5	7	5	5	2	18	2	1	0	0	1	18	0	1	0	0	0	4	1	1	0	0	0
GERJI	19	0	1	0	0	1	20	0	3	1	0	1	13	1	0	0	0	0	0	0	0	0	0	0
24 STREET	12	0	1	1	0	0	11	1	3	1	0	1	9	0	5	2	0	2	3	1	2	0	0	0

APPROACH	MOVEMENT (2 PM – 3 PM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	112	0	1	2	3	3	20	0	2	0	0	1	21	1	2	2	1	1	2	2	3	3	2	0
MEGENAGNA	114	10	9	2	2	2	32	0	1	0	0	0	11	0	2	1	0	0	13	3	2	0	1	0
GERJI	41	1	0	1	0	2	51	1	3	1	0	1	8	0	0	0	0	1	3	0	0	0	0	0
24 STREET	18	0	0	0	0	0	32	0	2	1	0	0	32	0	3	0	0	0	2	0	0	0	0	0

APPROACH	MOVEMENT (3 PM – 4 PM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	199	1	2	2	1	4	29	0	2	1	0	1	31	1	2	1	1	1	1	0	0	0	0	0
MEGENAGNA	267	14	13	4	0	2	45	1	1	0	0	0	29	0	2	2	0	0	17	1	2	0	0	0
GERJI	59	1	0	1	0	3	77	1	4	1	0	1	11	0	0	0	0	1	0	0	0	0	0	0
24 STREET	27	0	0	0	0	0	51	0	3	2	0	0	49	0	4	0	0	0	0	0	0	0	0	0

APPROACH	MOVEMENT (4 PM – 5 PM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	252	20	12	11	3	9	23	0	1	2	0	1	29	1	2	4	0	2	1	0	1	1	0	0
MEGENAGNA	255	13	8	11	2	4	54	4	1	0	0	0	28	1	0	1	0	1	3	0	0	0	0	0
GERJI	42	1	2	0	0	4	55	0	4	0	1	2	33	1	3	0	0	1	1	0	0	0	0	0
24 STREET	54	1	1	2	0	2	64	1	2	0	0	0	38	2	2	2	0	1	0	0	0	0	0	0

APPROACH	MOVEMENT (5 PM – 6 PM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	222	30	7	4	1	4	15	0	1	2	0	1	29	4	0	1	0	1	4	1	1	1	0	0
MEGENAGNA	231	22	12	2	1	3	46	3	2	0	0	0	29	0	0	1	0	1	1	0	0	0	0	0
GERJI	58	0	1	1	0	1	73	2	1	3	1	1	45	3	0	0	0	0	0	0	0	0	0	0
24 STREET	49	0	0	0	0	3	76	1	2	0	0	1	38	1	1	2	0	1	0	0	0	0	0	0

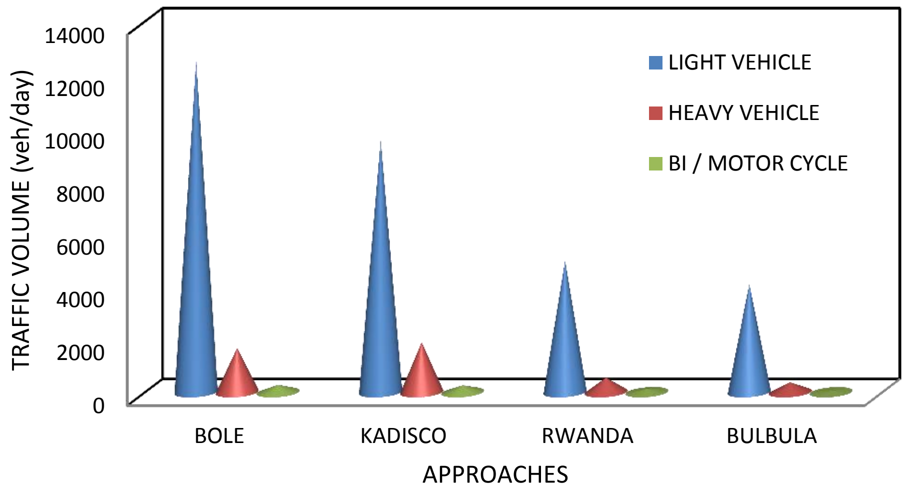
APPROACH	MOVEMENT (6 PM – 7 PM)																							
	THROUGH						LEFT TURN						RIGHT TURN						U-TURN					
	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC	PC	BUS	S&MT	LT	TT	BIC
BOLE	241	31	12	9	1	2	13	0	1	0	0	1	23	5	1	0	0	1	0	0	0	0	0	0
MEGENAGNA	238	15	6	8	3	2	59	5	0	1	0	0	21	1	0	0	0	1	0	0	0	0	0	0
GERJI	32	0	1	0	0	1	46	2	1	3	2	1	34	2	1	0	0	0	0	0	0	0	0	0
24 STREET	38	3	1	1	0	1	18	1	0	0	0	0	25	0	1	2	0	0	0	0	0	0	0	0

Appendix D: Pedestrian Volume

	Pedestrian volume				
	Approaches	8:00 AM – 9:00 AM		9:00 AM – 10:00 AM	
		15 minutes	Ped/h	15 minutes	Ped/h
Bole Mikael Roundabout	Bole	143	572	96	384
	Kadisco	201	804	217	868
	Rwanda	81	324	52	208
	Bulbula	136	544	104	416
	Total		2244		1876
	Approaches	4:00 PM – 5:00 PM		5:00 PM – 6:00 PM	
		15 minutes	Ped/h	15 minutes	Ped/h
	Bole	201	804	132	528
	Kadisco	113	452	238	952
	Rwanda	42	168	72	288
	Bulbula	109	436	183	732
	Total		1860		2500

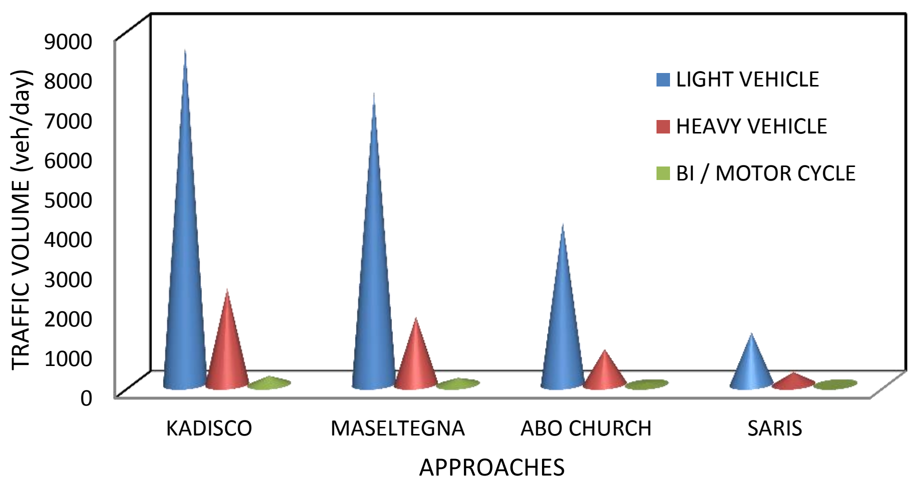
	Pedestrian volume				
	Approaches	8:00 AM – 9:00 AM		9:00 AM – 10:00 AM	
		15 minutes	Ped/h	15 minutes	Ped/h
Saris Abo Roundabout	Kadisco	831	3324	846	3384
	Maseltegna	743	2972	733	2932
	Abo Church	311	1244	265	1060
	Saris	106	424	116	464
	Total		7964		7840
	Approaches	4:00 PM – 5:00 PM		5:00 PM – 6:00 PM	
		15 minutes	Ped/h	15 minutes	Ped/h
	Kadisco	993	3972	1194	4776
	Maseltegna	901	3604	948	3792
	Abo Church	342	1368	456	1824
	Saris	286	1144	302	1208
	Total		10088		11600

Appendix E: Traffic Count Analysis



(a)

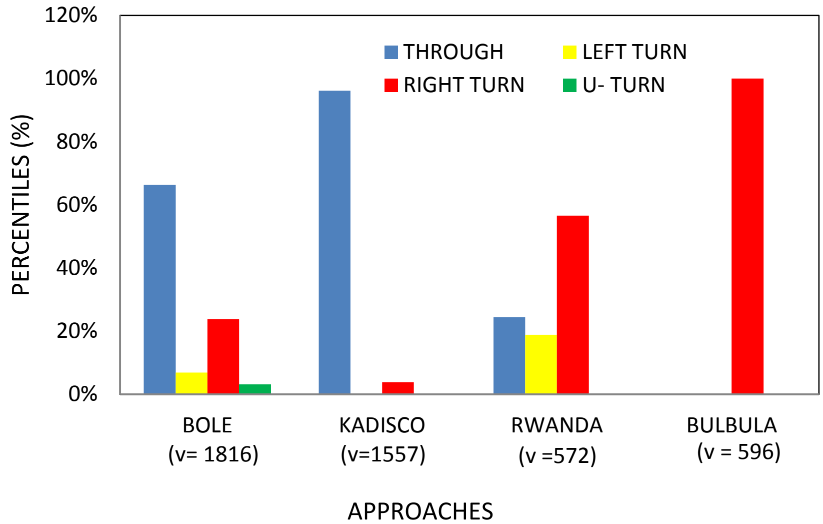
Figure 15 (a) Traffic Count Summary at Bole Mikael Roundabout, Wednesday



(b)

Figure 15 (b) Traffic Count Summary at Saris Abo Roundabout, Wednesday

Appendix F: Through Vehicle Dominancy Analysis



(a)

Figure 16 (a) Percentile Proportioning of Each Turning Movement per Hour per Approach at Bole Mikael Roundabout

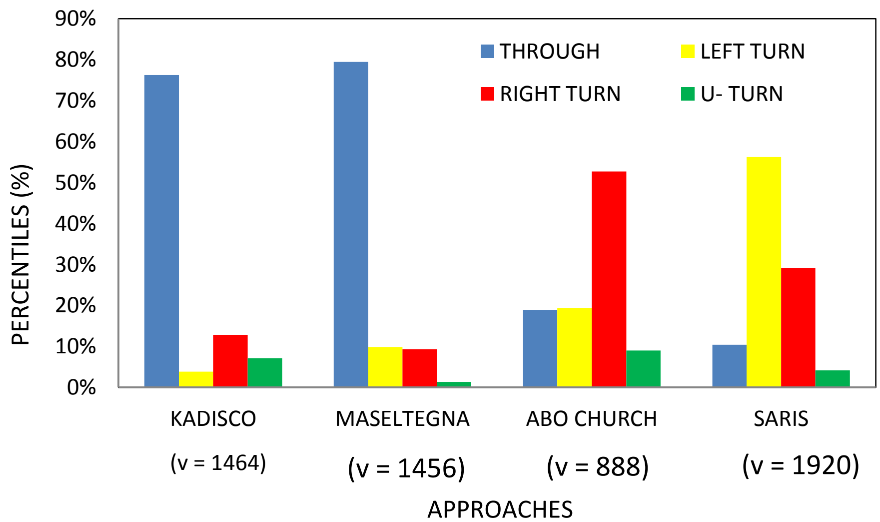
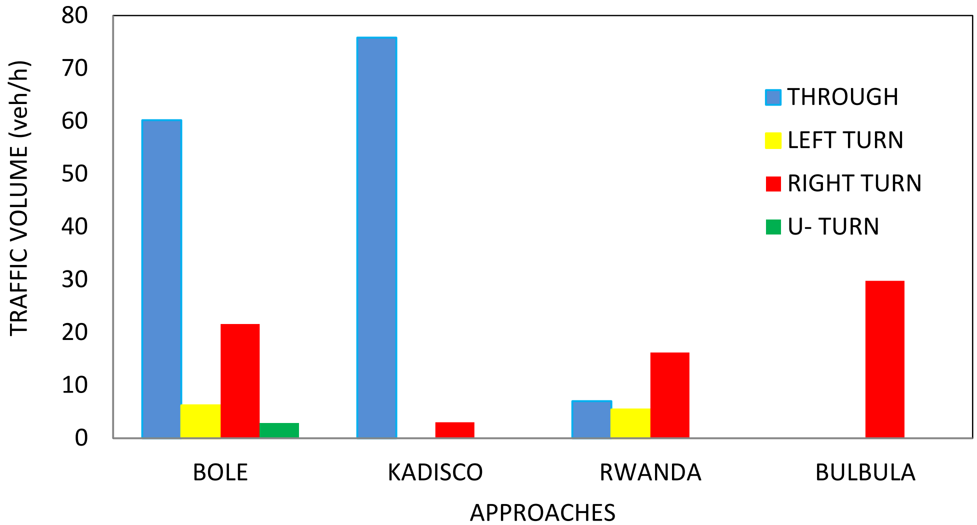


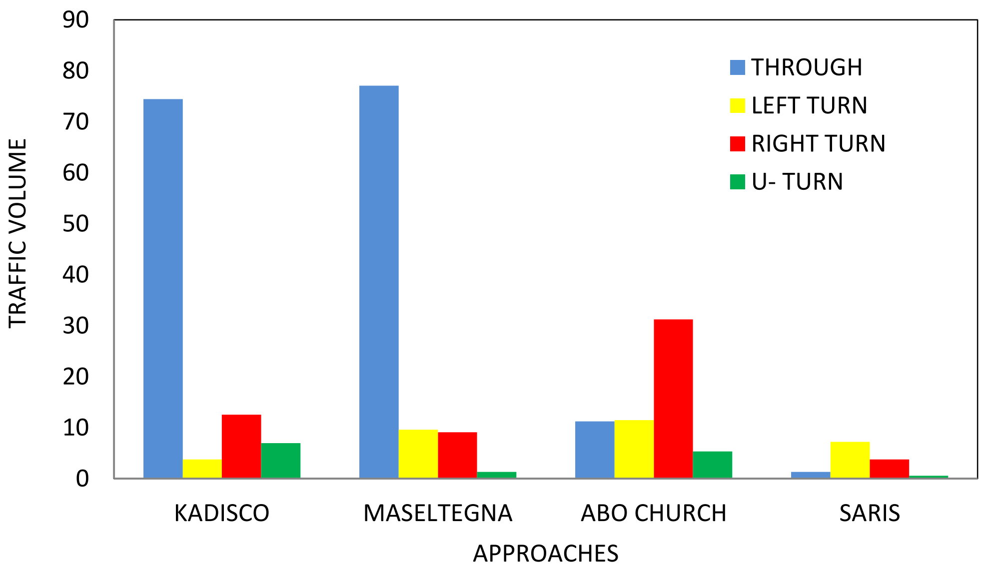
Figure 16 (b) Percentile Proportioning of Each Turning Movement per Hour per Approach at Saris Abo Roundabout

Appendix G: Turning Movement Analysis Per-Phase



(a)

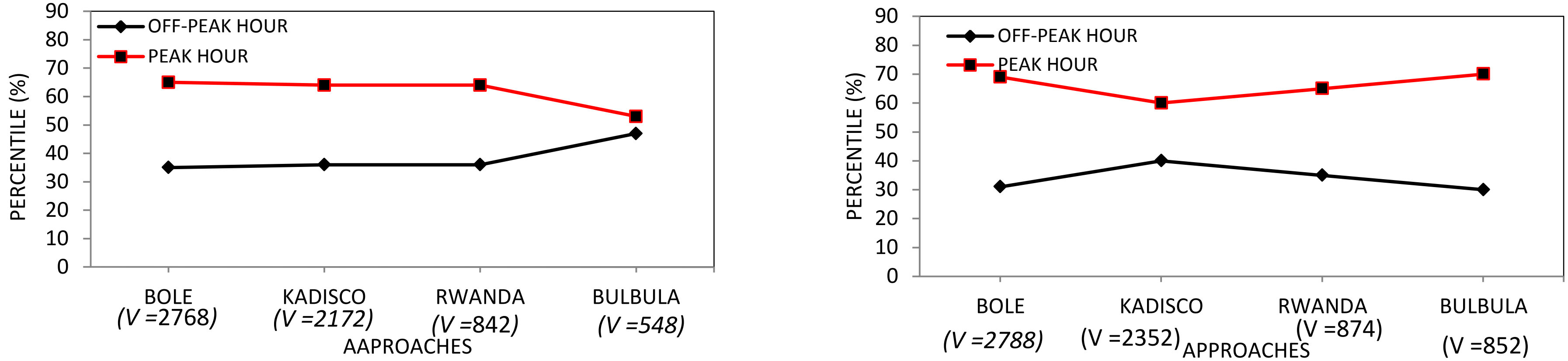
Figure 17 (a) Peak Hour Turning Movement Study per Phase per Approaching at Bole Mikael Roundabout



(b)

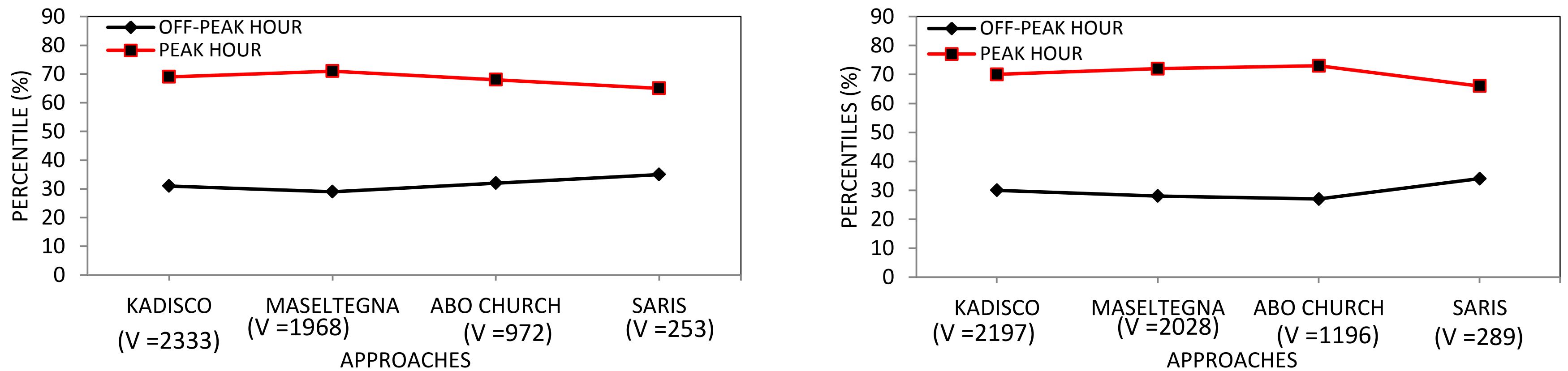
Figure 17 (b) Peak Hour Turning Movement Study per Phase per Approaching at Saris Abo Roundabout

Appendix H: AM & PM Peak Hour Comparison with off-Peak Hour



(a) (b)

Figure 18 (a) AM Peak and Off Peak, (b) PM Peak and Off Peak Hour Traffic Count Comparison at Bole Mikael Roundabout



(c) (d)

Figure 19 (c) AM Peak and Off Peak, (d) PM Peak and Off Peak Hour Traffic Count Comparison at Saris Abo Roundabout

Appendix I: VISTRO Output

Gerji Emperial Roundabout: Base Condition Alternative 1

Control Type	Roundabout											
Analysis Method	HCM											
Name	BOLE			MEGENAGNA			GERJI			24 STREET		
Show Name	☑			☑			☑			☑		
Approach	*Northeastbound*			*Southwestbound*			*Northwestbound*			*Southeastbound*		
Lane Configuration	┐║├			┐║├			┤║├			┤║├		
Turning Movement	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*
Base Volume Input [veh/h]	112	1104	144	132	1168	152	396	348	212	132	192	216
Total Analysis Volume [veh/h]	*127*	*1255*	*164*	*155*	*1374*	*179*	*445*	*391*	*238*	*157*	*229*	*257*
Intersection Settings												
Analyze Intersection?	☑											
Analysis Period	15 minutes											
Population < 10000 (Signal Warrants)	☑											
Number of Conflicting Circulating Lanes	3			3			3			3		
Circulating Flow Rate [veh/h]	*718*			*1192*			*1859*			*2401*		
Exiting Flow Rate [veh/h]	*502*			*638*			*1708*			*2214*		
Demand Flow Rate [veh/h]	*112*	*1104*	*144*	*132*	*1168*	*152*	*396*	*348*	*212*	*132*	*192*	*216*
Adjusted Demand Flow Rate [veh/h]	*127*	*1255*	*164*	*155*	*1374*	*179*	*445*	*391*	*238*	*157*	*229*	*257*

Lanes	BOLE			MEGENAGNA			GERJI		24 STREET	
Overwrite Calculated Critical Headway	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
User-Defined Critical Headway [s]	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
Overwrite Calculated Follow-Up Time	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
User-Defined Follow-Up Time [s]	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
A (intercept)	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*
B (coefficient)	*0.00075*	*0.00075*	*0.00070*	*0.00075*	*0.00075*	*0.00070*	*0.00075*	*0.00070*	*0.00075*	*0.00070*
HV Adjustment Factor	*0.84*	*0.84*	*0.84*	*0.83*	*0.83*	*0.83*	*0.80*	*0.80*	*0.73*	*0.73*
Entry Flow Rate [veh/h]	*151*	*844*	*844*	*188*	*938*	*938*	*629*	*710*	*416*	*469*
Capacity of Entry and Bypass Lanes [ve	*660*	*660*	*684*	*463*	*463*	*491*	*281*	*308*	*187*	*211*
Pedestrian Impedance	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
Capacity per Entry Lane [veh/h]	*555*	*555*	*575*	*383*	*383*	*407*	*226*	*247*	*136*	*153*
X, volume / capacity	*0.23*	*1.28*	*1.23*	*0.41*	*2.03*	*1.91*	*2.24*	*2.31*	*2.23*	*2.23*

Movement, Approach, & Intersection Results	BOLE			MEGENAGNA			GERJI		24 STREET	
Average Lane Delay [s/veh]	*9.55*	*161.90*	*142.93*	*17.70*	*495.70*	*442.23*	*607.98*	*631.61*	*628.76*	*621.12*
Lane LOS	*A*	*F*	*F*	*C*	*F*	*F*	*F*	*F*	*F*	*F*
95th-Percentile Queue Length [veh]	*0.88*	*28.64*	*26.76*	*1.91*	*54.58*	*51.92*	*39.73*	*45.03*	*25.30*	*28.04*
95th-Percentile Queue Length [m]	*6.68*	*218.23*	*203.89*	*14.59*	*415.90*	*395.60*	*302.77*	*343.10*	*192.79*	*213.66*
Approach Delay [s/veh]	*139.67*			*425.21*			*620.50*		*624.71*	
Approach LOS	*F*			*F*			*F*		*F*	
Intersection Delay [s/veh]	*404.41*									
Intersection LOS	*F*									

Gerji Emperial Roundabout: Base Condition Alternative 2

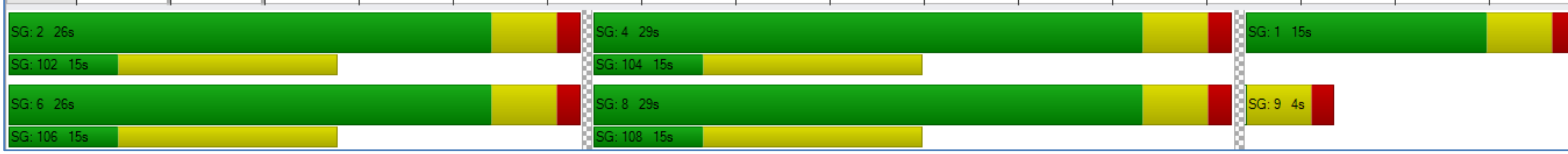
g / C, Green / Cycle	0.36	0.36	0.36	0.36	0.36	0.36	0.47	0.31	0.31	0.31
(v / s)_i Volume / Saturation Flow Rate	0.50	0.51	0.51	0.54	0.55	0.56	1.09	0.49	8591246.02	0.48
so, Base Saturation Flow per Lane [veh/	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Arrival type		3			3		3		3	
s, saturation flow rate [veh/h]	252	1420	1363	289	1430	1374	449	1196	0	1017
c, Capacity [veh/h]	103	507	487	103	511	491	310	376	103	320
X, volume / capacity	1.23	1.42	1.44	1.51	1.53	1.57	1.58	1.55	1.53	1.52
d, Delay for Lane Group [s/veh]	199.67	222.01	231.04	306.79	272.42	288.32	285.98	286.10	314.83	273.61
Lane Group LOS	F	F	F	F	F	F	F	F	F	F
Critical Lane Group	☐	☐	☐	☐	☐	☑	☑	☐	☑	☐
50th-Percentile Queue Length [veh]	6.43	35.87	35.67	9.48	43.21	43.67	26.58	33.53	9.74	27.21
50th-Percentile Queue Length [m]	49.02	273.33	271.78	72.26	329.24	332.77	202.55	255.48	74.19	207.30
95th-Percentile Queue Length [veh]	11.58	55.47	55.47	17.07	67.62	68.78	44.65	53.38	17.53	43.87
95th-Percentile Queue Length [m]	88.24	422.67	422.69	130.08	515.29	524.13	340.25	406.76	133.54	334.32

Movement, Approach, & Intersection Results

d_M, Delay for Movement [s/veh]	199.67	225.86	231.04	306.79	279.25	288.32	285.98	286.09	286.10	314.83	273.61	273.61
Movement LOS	F	F	F	F	F	F	F	F	F	F	F	F
Critical Movement	☐	☐	☐	☐	☐	☐	☐	☐	☐	☑	☐	☐
d_A, Approach Delay [s/veh]		224.26			282.70			286.05			283.67	
Approach LOS		F			F			F			F	
d_I, Intersection Delay [s/veh]						265.37						
Intersection LOS						F						

Sequence

Ring 1	2	4	1	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 2	6	8	9	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-



Control Type	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Protected	Permissive	Permissive	Protected	Permissive	Permissive
Signal group	0	8	0	0	4	0	1	2	0	9	6	0

Gerji Emperial Roundabout: Future Condition Alternative 1

Control Type	Roundabout											
Analysis Method	HCM											
Name	BOLE			MEGENAGNA			GERJI			24 STREET		
Show Name	☑			☑			☑			☑		
Approach	*Northeastbound*			*Southwestbound*			*Northwestbound*			*Southeastbound*		
Lane Configuration	┓|┣			┓|┣			┫┣			┫┣		
Turning Movement	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*
Base Volume Input [veh/h]	112	1104	144	132	1168	152	396	348	212	132	192	216
Total Analysis Volume [veh/h]	*159*	*1568*	*205*	*194*	*1718*	*224*	*556*	*489*	*298*	*196*	*286*	*321*
Intersection Settings												
Analyze Intersection?	☑											
Analysis Period	15 minutes											
Population < 10000 (Signal Warrants)	☑											
Number of Conflicting Circulating Lanes	3			3			3			3		
Circulating Flow Rate [veh/h]	*898*			*1491*			*2322*			*3002*		
Exiting Flow Rate [veh/h]	*628*			*798*			*2133*			*2768*		
Demand Flow Rate [veh/h]	*140*	*1380*	*180*	*165*	*1460*	*190*	*495*	*435*	*265*	*165*	*240*	*270*
Adjusted Demand Flow Rate [veh/h]	*159*	*1568*	*205*	*194*	*1718*	*224*	*556*	*489*	*298*	*196*	*286*	*321*

Lanes	BOLE			MEGENAGNA			GERJI		24 STREET	
Overwrite Calculated Critical Headway	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
User-Defined Critical Headway [s]	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
Overwrite Calculated Follow-Up Time	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
User-Defined Follow-Up Time [s]	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
A (intercept)	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*
B (coefficient)	*0.00075*	*0.00075*	*0.00070*	*0.00075*	*0.00075*	*0.00070*	*0.00075*	*0.00070*	*0.00075*	*0.00070*
HV Adjustment Factor	*0.84*	*0.84*	*0.84*	*0.83*	*0.83*	*0.83*	*0.80*	*0.80*	*0.73*	*0.73*
Entry Flow Rate [veh/h]	*189*	*1054*	*1054*	*235*	*1173*	*1173*	*787*	*887*	*520*	*586*
Capacity of Entry and Bypass Lanes [ve	*577*	*577*	*603*	*370*	*370*	*398*	*198*	*223*	*119*	*139*
Pedestrian Impedance	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
Capacity per Entry Lane [veh/h]	*485*	*485*	*508*	*306*	*306*	*330*	*159*	*179*	*87*	*101*
X, volume / capacity	*0.33*	*1.83*	*1.75*	*0.63*	*3.17*	*2.95*	*3.97*	*3.99*	*4.37*	*4.24*

Movement, Approach, & Intersection Results	BOLE			MEGENAGNA			GERJI		24 STREET	
Average Lane Delay [s/veh]	*12.64*	*400.77*	*364.46*	*33.31*	*1012.39*	*908.26*	*1394.64*	*1396.14*	*1613.93*	*1543.40*
Lane LOS	*B*	*F*	*F*	*D*	*F*	*F*	*F*	*F*	*F*	*F*
95th-Percentile Queue Length [veh]	*1.41*	*56.12*	*53.61*	*4.04*	*87.32*	*84.50*	*62.81*	*70.45*	*39.92*	*44.25*
95th-Percentile Queue Length [m]	*10.78*	*427.62*	*408.53*	*30.77*	*665.35*	*643.87*	*478.58*	*536.84*	*304.18*	*337.20*
Approach Delay [s/veh]	*350.24*			*870.67*			*1395.43*		*1576.55*	
Approach LOS	*F*			*F*			*F*		*F*	
Intersection Delay [s/veh]	*913.49*									
Intersection LOS	*F*									

Gerji Emperial Roundabout: Future Condition Alternative 2

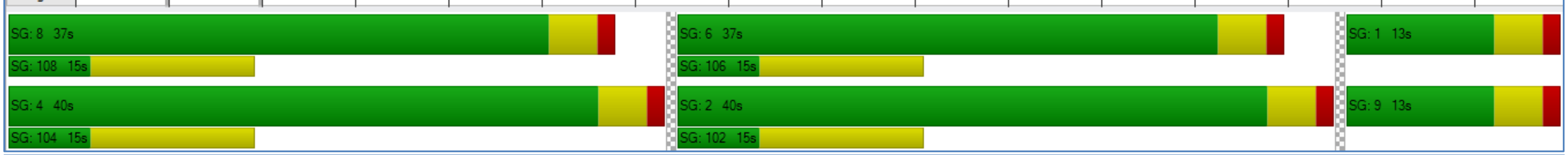
g / C, Green / Cycle	0.35	0.35	0.35	0.39	0.39	0.39	0.48	0.39	0.45	0.35
(v / s)_i Volume / Saturation Flow Rate	0.92	0.62	0.65	1.03	0.68	0.71	4.07	0.61	32353935.85	0.60
so, Base Saturation Flow per Lane [veh/	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Arrival type		3			3		3		3	
s, saturation flow rate [veh/h]	173	1420	1364	189	1430	1374	150	1197	0	1017
c, Capacity [veh/h]	77	504	484	121	554	532	191	463	155	361
X, volume / capacity	2.05	1.76	1.83	1.60	1.75	1.83	3.19	1.58	1.27	1.68
d, Delay for Lane Group [s/veh]	562.34	380.52	411.86	346.52	375.29	407.36	1030.82	300.80	199.30	348.70
Lane Group LOS	F	F	F	F	F	F	F	F	F	F
Critical Lane Group	☑	☐	☐	☐	☐	☐	☐	☐	☑	☐
50th-Percentile Queue Length [veh]	12.89	59.99	61.85	12.01	65.15	67.34	55.29	45.40	8.23	39.86
50th-Percentile Queue Length [m]	98.21	457.14	471.32	91.49	496.42	513.11	421.30	345.97	62.70	303.77
95th-Percentile Queue Length [veh]	23.20	95.21	98.80	21.61	103.44	107.68	96.51	71.89	14.70	64.46
95th-Percentile Queue Length [m]	176.78	725.46	752.88	164.69	788.24	820.49	735.40	547.77	111.98	491.19

Movement, Approach, & Intersection Results

d_M, Delay for Movement [s/veh]	562.34	394.13	411.86	346.52	389.23	407.36	1030.82	381.39	300.80	199.30	348.70	348.70
Movement LOS	F	F	F	F	F	F	F	F	F	F	F	F
Critical Movement	☐	☐	☐	☐	☐	☐	☑	☐	☐	☐	☐	☐
d_A, Approach Delay [s/veh]		409.86			387.26			632.37			312.24	
Approach LOS		F			F			F			F	
d_I, Intersection Delay [s/veh]						437.56						
Intersection LOS						F						

Sequence

Ring 1	8	6	1	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 2	4	2	9	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-



Control Type	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Protected	Permissive	Permissive	Protected	Permissive	Permissive
Signal group	0	8	0	0	4	0	1	2	0	9	6	0

Bole Mikael Roundabout: Base Condition Alternative 1

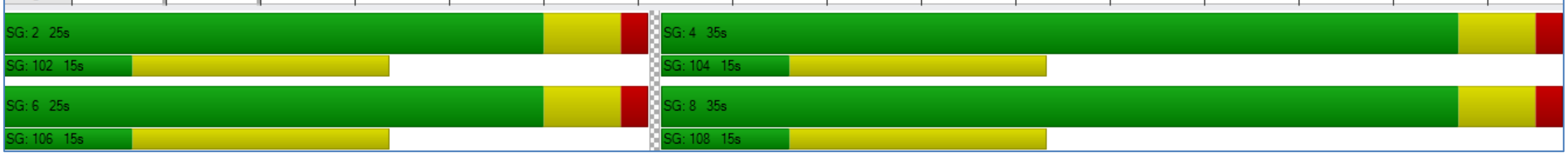
Control Type	Roundabout											
Analysis Method	HCM											
Name	KADISCO			BOLE			BULBULA			RWANDA		
Show Name	☑			☑			☑			☑		
Approach	*Northeastbound*			*Southwestbound*			*Northwestbound*			*Southeastbound*		
Lane Configuration												
Turning Movement	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*
Base Volume Input [veh/h]	150	1365	61	100	1204	432	105	64	426	108	140	324
Total Analysis Volume [veh/h]	*169*	*1534*	*69*	*109*	*1309*	*470*	*111*	*67*	*448*	*120*	*156*	*360*
Intersection Settings												
Analyze Intersection?	☑											
Analysis Period	15 minutes											
Population < 10000 (Signal Warrants)	☐											
Number of Conflicting Circulating Lanes		3			3			3			3	
Circulating Flow Rate [veh/h]		*405*			*386*			*2092*			*1696*	
Exiting Flow Rate [veh/h]		*282*			*267*			*1896*			*1575*	
Demand Flow Rate [veh/h]	*150*	*1365*	*61*	*100*	*1204*	*432*	*105*	*64*	*426*	*108*	*140*	*324*
Adjusted Demand Flow Rate [veh/h]	*169*	*1534*	*69*	*109*	*1309*	*470*	*111*	*67*	*448*	*120*	*156*	*360*
Lanes												
Overwrite Calculated Critical Headway	☐	☐	☐	☐	☐	☐		☐			☐	
User-Defined Critical Headway [s]	4.00	4.00	4.00	4.00	4.00	4.00		4.00			4.00	
Overwrite Calculated Follow-Up Time	☐	☐	☐	☐	☐	☐		☐			☐	
User-Defined Follow-Up Time [s]	3.00	3.00	3.00	3.00	3.00	3.00		3.00			3.00	
A (intercept)	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*		*1130.00*			*1130.00*	
B (coefficient)	*0.00075*	*0.00075*	*0.00070*	*0.00075*	*0.00075*	*0.00070*		*0.00070*			*0.00070*	
HV Adjustment Factor	*0.87*	*0.87*	*0.87*	*0.90*	*0.90*	*0.90*		*0.93*			*0.97*	
Entry Flow Rate [veh/h]	*196*	*927*	*927*	*122*	*990*	*990*		*672*			*655*	
Capacity of Entry and Bypass Lanes [ve	*834*	*834*	*851*	*846*	*846*	*863*		*262*			*345*	
Pedestrian Impedance	*0.65*	*0.65*	*0.65*	*0.79*	*0.79*	*0.79*		*1.00*			*1.00*	
Capacity per Entry Lane [veh/h]	*469*	*469*	*479*	*598*	*598*	*610*		*244*			*336*	
X, volume / capacity	*0.36*	*1.71*	*1.68*	*0.18*	*1.49*	*1.46*		*2.57*			*1.90*	
Movement, Approach, & Intersection Results												
Average Lane Delay [s/veh]	*13.76*	*350.03*	*334.55*	*8.28*	*247.96*	*235.30*		*750.13*			*441.29*	
Lane LOS	*B*	*F*	*F*	*A*	*F*	*F*		*F*			*F*	
95th-Percentile Queue Length [veh]	*1.62*	*47.90*	*46.84*	*0.66*	*44.07*	*42.83*		*52.30*			*43.14*	
95th-Percentile Queue Length [m]	*12.36*	*364.98*	*356.93*	*5.05*	*335.80*	*326.36*		*398.54*			*328.71*	
Approach Delay [s/veh]		*310.66*			*226.58*			*750.13*			*441.29*	
Approach LOS		*F*			*F*			*F*			*F*	
Intersection Delay [s/veh]	*351.18*											
Intersection LOS	*F*											

Bole Mikael Roundabout: Base Condition Alternative 2

g / C, Green / Cycle	0.52	0.52	0.52	0.52	0.52	0.52	0.35			0.35		
(v / s)_i Volume / Saturation Flow Rate	0.81	0.55	0.61	0.41	0.58	0.76	0.62			0.63		
so, Base Saturation Flow per Lane [veh/	1900	1900	1900	1900	1900	1900	1900			1900		
Arrival type		3			3		3			3		
s, saturation flow rate [veh/h]	209	1465	1316	263	1553	1163	1011			1006		
c, Capacity [veh/h]	120	757	680	120	803	601	424			423		
X, volume / capacity	1.41	1.06	1.18	0.91	1.11	1.47	1.47			1.50		
d, Delay for Lane Group [s/veh]	255.84	64.85	108.46	90.35	82.49	235.85	247.19			259.23		
Lane Group LOS	F	F	F	F	F	F	F			F		
Critical Lane Group	☑	☐	☐	☐	☐	☐	☐			☑		
50th-Percentile Queue Length [veh]	9.24	19.46	25.72	3.50	23.56	43.25	32.83			34.12		
50th-Percentile Queue Length [m]	70.42	148.25	195.99	26.69	179.55	329.57	250.20			260.02		
95th-Percentile Queue Length [veh]	16.63	27.92	38.18	6.30	34.14	68.39	51.85			54.04		
95th-Percentile Queue Length [m]	126.75	212.73	290.92	48.03	260.15	521.15	395.10			411.77		
Movement, Approach, & Intersection Results												
d_M, Delay for Movement [s/veh]	255.84	85.62	108.46	90.35	131.08	235.85	247.19	247.19	247.19	259.23	259.23	259.23
Movement LOS	F	F	F	F	F	F	F	F	F	F	F	F
Critical Movement	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☑
d_A, Approach Delay [s/veh]		102.74			154.81			247.19			259.23	
Approach LOS		F			F			F			F	
d_I, Intersection Delay [s/veh]							161.31					
Intersection LOS							F					
Control Type	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive
Signal group	0	8	0	0	4	0	0	2	0	0	6	0

Sequence

Ring 1	2	4	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 2	6	8	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-



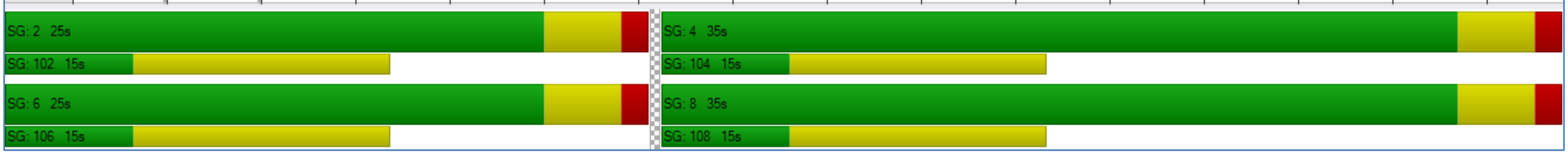
Bole Mikael Roundabout: Future Condition Alternative 1

Control Type	Roundabout											
Analysis Method	HCM											
Name	KADISCO			BOLE			BULBULA			RWANDA		
Show Name	☑			☑			☑			☑		
Approach	*Northeastbound*			*Southwestbound*			*Northwestbound*			*Southeastbound*		
Lane Configuration												
Turning Movement	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*
Base Volume Input [veh/h]	150	1365	61	100	1204	432	105	64	426	108	140	324
Total Analysis Volume [veh/h]	*211*	*1917*	*85*	*136*	*1636*	*587*	*138*	*84*	*561*	*150*	*194*	*450*
Intersection Settings												
Analyze Intersection?	☑											
Analysis Period	15 minutes											
Population < 10000 (Signal Warrants)	☐											
Number of Conflicting Circulating Lanes	3			3			3			3		
Circulating Flow Rate [veh/h]	*505*			*482*			*2614*			*2118*		
Exiting Flow Rate [veh/h]	*351*			*334*			*2370*			*1967*		
Demand Flow Rate [veh/h]	*188*	*1706*	*76*	*125*	*1505*	*540*	*131*	*80*	*533*	*135*	*175*	*405*
Adjusted Demand Flow Rate [veh/h]	*211*	*1917*	*85*	*136*	*1636*	*587*	*138*	*84*	*561*	*150*	*194*	*450*
Lanes												
Overwrite Calculated Critical Headway	☐	☐	☐	☐	☐	☐	☐			☐		
User-Defined Critical Headway [s]	4.00	4.00	4.00	4.00	4.00	4.00	4.00			4.00		
Overwrite Calculated Follow-Up Time	☐	☐	☐	☐	☐	☐	☐			☐		
User-Defined Follow-Up Time [s]	3.00	3.00	3.00	3.00	3.00	3.00	3.00			3.00		
A (intercept)	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*			*1130.00*		
B (coefficient)	*0.00075*	*0.00075*	*0.00070*	*0.00075*	*0.00075*	*0.00070*	*0.00070*			*0.00070*		
HV Adjustment Factor	*0.87*	*0.87*	*0.87*	*0.90*	*0.90*	*0.90*	*0.93*			*0.97*		
Entry Flow Rate [veh/h]	*244*	*1157*	*1157*	*152*	*1236*	*1236*	*841*			*817*		
Capacity of Entry and Bypass Lanes [ve	*774*	*774*	*794*	*788*	*788*	*807*	*182*			*257*		
Pedestrian Impedance	*0.65*	*0.65*	*0.65*	*0.79*	*0.79*	*0.79*	*1.00*			*1.00*		
Capacity per Entry Lane [veh/h]	*435*	*435*	*446*	*560*	*560*	*574*	*169*			*250*		
X, volume / capacity	*0.49*	*2.30*	*2.25*	*0.24*	*1.99*	*1.94*	*4.63*			*3.18*		
Movement, Approach, & Intersection Results												
Average Lane Delay [s/veh]	*18.29*	*614.49*	*588.33*	*9.69*	*467.36*	*445.92*	*1688.52*			*1023.20*		
Lane LOS	*C*	*F*	*F*	*A*	*F*	*F*	*F*			*F*		
95th-Percentile Queue Length [veh]	*2.59*	*75.78*	*74.47*	*0.95*	*74.54*	*72.96*	*80.41*			*72.21*		
95th-Percentile Queue Length [m]	*19.76*	*577.42*	*567.49*	*7.21*	*568.01*	*555.92*	*612.71*			*550.24*		
Approach Delay [s/veh]	*545.31*			*428.21*			*1688.52*			*1023.20*		
Approach LOS	*F*			*F*			*F*			*F*		
Intersection Delay [s/veh]	*707.67*											
Intersection LOS	*F*											

Bole Mikael Roundabout: Future Condition Alternative 2

g / C, Green / Cycle	0.52	0.52	0.52	0.52	0.52	0.52		0.35			0.35	
(v / s)_i Volume / Saturation Flow Rate	1.56	0.68	0.76	0.76	0.72	0.95		0.78			0.80	
so, Base Saturation Flow per Lane [veh/	1900	1900	1900	1900	1900	1900		1900			1900	
Arrival type		3			3			3			3	
s, saturation flow rate [veh/h]	135	1465	1318	178	1553	1165		1000			988	
c, Capacity [veh/h]	120	757	681	120	803	602		421			417	
X, volume / capacity	1.76	1.32	1.47	1.13	1.38	1.85		1.86			1.90	
d, Delay for Lane Group [s/veh]	403.06	168.90	233.77	152.47	195.47	401.87		417.77			436.87	
Lane Group LOS	F	F	F	F	F	F		F			F	
Critical Lane Group	☑	☐	☐	☐	☐	☐		☐			☑	
50th-Percentile Queue Length [veh]	14.15	41.33	49.47	5.72	48.75	71.07		52.47			54.18	
50th-Percentile Queue Length [m]	107.81	314.92	376.99	43.61	371.46	541.58		399.81			412.89	
95th-Percentile Queue Length [veh]	25.47	62.55	77.38	10.30	74.42	116.41		84.94			87.90	
95th-Percentile Queue Length [m]	194.06	476.64	589.63	78.50	567.09	887.07		647.25			669.81	
Movement, Approach, & Intersection Results												
d_M, Delay for Movement [s/veh]	403.06	199.90	233.77	152.47	261.64	401.87	417.77	417.77	417.77	436.87	436.87	436.87
Movement LOS	F	F	F	F	F	F	F	F	F	F	F	F
Critical Movement	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☑
d_A, Approach Delay [s/veh]		220.57			250.24			417.77			436.87	
Approach LOS		F			F			F			F	
d_I, Intersection Delay [s/veh]						300.34						
Intersection LOS						F						

Sequence																
Ring 1	2	4	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 2	6	8	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-



Saris Abo Roundabout: Base Condition Alternative 1

	ABO CHURCH			SARIS			MASELTEGNA			KADISCO		
Control Type	Roundabout											
Analysis Method	HCM											
Name	ABO CHURCH			SARIS			MASELTEGNA			KADISCO		
Show Name	☑			☑			☑			☑		
Approach	*Westbound*			*Northeastbound*			*Northwestbound*			*Southeastbound*		
Lane Configuration												
Turning Movement	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*
Base Volume Input [veh/h]	252	168	468	124	20	56	164	1156	136	160	1116	188
Total Analysis Volume [veh/h]	*293*	*195*	*544*	*141*	*23*	*64*	*171*	*1204*	*142*	*170*	*1187*	*200*
Intersection Settings												
Analyze Intersection?	☑											
Analysis Period	15 minutes											
Population < 10000 (Signal Warrants)	☐											
Number of Conflicting Circulating Lanes	1			1			1			1		
Circulating Flow Rate [veh/h]	*1730*			*1639*			*366*			*750*		
Exiting Flow Rate [veh/h]	*1534*			*1650*			*214*			*418*		
Demand Flow Rate [veh/h]	*252*	*168*	*468*	*124*	*20*	*56*	*164*	*1156*	*136*	*160*	*1116*	*188*
Adjusted Demand Flow Rate [veh/h]	*293*	*195*	*544*	*141*	*23*	*64*	*171*	*1204*	*142*	*170*	*1187*	*200*

Lanes	ABO CHURCH		SARIS		MASELTEGNA			KADISCO		
Overwrite Calculated Critical Headway	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
User-Defined Critical Headway [s]	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
Overwrite Calculated Follow-Up Time	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
User-Defined Follow-Up Time [s]	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
A (intercept)	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*
B (coefficient)	*0.00100*	*0.00100*	*0.00100*	*0.00100*	*0.00100*	*0.00100*	*0.00100*	*0.00100*	*0.00100*	*0.00100*
HV Adjustment Factor	*0.88*	*0.88*	*0.93*	*0.93*	*0.87*	*0.87*	*0.87*	*0.90*	*0.90*	*0.90*
Entry Flow Rate [veh/h]	*554*	*618*	*153*	*94*	*197*	*773*	*773*	*189*	*770*	*770*
Capacity of Entry and Bypass Lanes [ve	*201*	*201*	*180*	*180*	*784*	*784*	*784*	*534*	*534*	*534*
Pedestrian Impedance	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
Capacity per Entry Lane [veh/h]	*177*	*177*	*167*	*167*	*684*	*684*	*684*	*481*	*481*	*481*
X, volume / capacity	*2.76*	*3.08*	*0.85*	*0.52*	*0.25*	*0.99*	*0.99*	*0.35*	*1.44*	*1.44*

Movement, Approach, & Intersection Results	ABO CHURCH		SARIS		MASELTEGNA			KADISCO		
Average Lane Delay [s/veh]	*850.03*	*991.29*	*88.69*	*45.81*	*8.27*	*55.27*	*55.27*	*13.29*	*233.51*	*233.51*
Lane LOS	*F*	*F*	*F*	*E*	*A*	*F*	*F*	*B*	*F*	*F*
95th-Percentile Queue Length [veh]	*43.17*	*50.01*	*5.86*	*2.60*	*0.99*	*15.26*	*15.26*	*1.58*	*34.19*	*34.19*
95th-Percentile Queue Length [m]	*328.96*	*381.08*	*44.63*	*19.85*	*7.52*	*116.28*	*116.28*	*12.02*	*260.53*	*260.53*
Approach Delay [s/veh]	*924.49*		*72.33*		*49.97*			*209.46*		
Approach LOS	*F*		*F*		*E*			*F*		
Intersection Delay [s/veh]	*316.68*									
Intersection LOS	*F*									

Saris Abo Roundabout: Base Condition Alternative 2

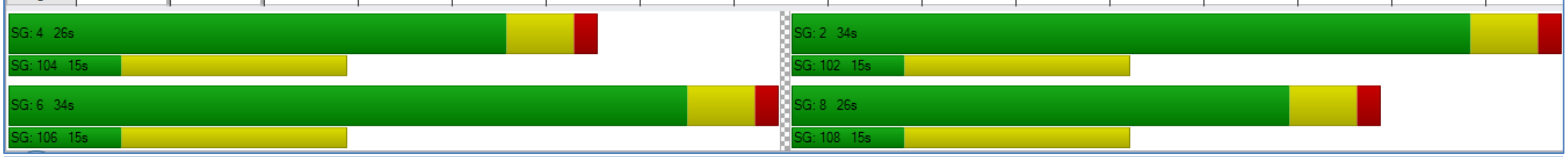
g / C, Green / Cycle	0.32	0.32	0.32	0.32	0.44	0.44	0.44	0.44	0.44	0.44
(v / s)_i Volume / Saturation Flow Rate	0.00	0.63	0.00	0.07	0.15	0.46	0.47	0.35	0.46	0.47
so, Base Saturation Flow per Lane [veh/	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Arrival type	3		3		3			3		
s, saturation flow rate [veh/h]	0	1173	0	1301	1162	1481	1427	486	1530	1455
c, Capacity [veh/h]	106	379	106	421	498	653	629	277	675	642
X, volume / capacity	2.77	1.95	1.33	0.21	0.34	1.04	1.06	0.61	1.04	1.06
d, Delay for Lane Group [s/veh]	855.03	458.82	234.15	17.79	18.40	66.05	70.38	34.71	65.63	72.37
Lane Group LOS	F	F	F	B	B	F	F	C	F	F
Critical Lane Group	☐	☑	☐	☐	☐	☐	☑	☐	☐	☐
50th-Percentile Queue Length [veh]	25.89	51.74	7.66	1.12	1.84	18.00	18.10	2.28	18.37	18.67
50th-Percentile Queue Length [m]	197.28	394.27	58.39	8.57	13.99	137.13	137.91	17.40	139.97	142.25
95th-Percentile Queue Length [veh]	45.94	84.00	13.79	2.02	3.30	25.72	26.06	4.11	26.19	26.89
95th-Percentile Queue Length [m]	350.03	640.11	105.11	15.42	25.18	196.02	198.54	31.31	199.60	204.89

Movement, Approach, & Intersection Results

d_M, Delay for Movement [s/veh]	855.03	458.82	458.82	234.15	17.79	17.79	18.40	67.93	70.38	34.71	68.37	72.37
Movement LOS	F	F	F	F	B	B	B	E	E	C	E	E
Critical Movement	☑	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
d_A, Approach Delay [s/veh]	571.31			151.59			62.57			65.21		
Approach LOS	F			F			E			E		
d_I, Intersection Delay [s/veh]	189.34											
Intersection LOS	F											

Sequence

Ring 1	4	2	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 2	6	8	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-



Control Type	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive	Permissive
Signal group	0	4	0	0	8	0	0	2	0	0	6	0

Saris Abo Roundabout: Future Condition Alternative 1

Control Type	Roundabout											
Analysis Method	HCM											
Name	ABO CHURCH			SARIS			MASELTEGAN			KADISCO		
Show Name	☑			☑			☑			☑		
Approach	*Westbound*			*Northeastbound*			*Northwestbound*			*Southeastbound*		
Lane Configuration												
Turning Movement	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*	*Left*	*Thru*	*Right*
Base Volume Input [veh/h]	252	168	468	124	20	56	164	1156	136	160	1116	188
Total Analysis Volume [veh/h]	*366*	*244*	*680*	*176*	*28*	*80*	*214*	*1505*	*177*	*213*	*1484*	*250*
Intersection Settings												
Analyze Intersection?	☐											
Analysis Period	15 minutes											
Population < 10000 (Signal Warrants)	☐											
Number of Conflicting Circulating Lanes	3			3			3			3		
Circulating Flow Rate [veh/h]	*2162*			*2299*			*457*			*938*		
Exiting Flow Rate [veh/h]	*1917*			*2063*			*267*			*522*		
Demand Flow Rate [veh/h]	*315*	*210*	*585*	*155*	*25*	*70*	*205*	*1445*	*170*	*200*	*1395*	*235*
Adjusted Demand Flow Rate [veh/h]	*366*	*244*	*680*	*176*	*28*	*80*	*214*	*1505*	*177*	*213*	*1484*	*250*
Lanes												
Overwrite Calculated Critical Headway	☐		☐	☐		☐	☐	☐	☐	☐	☐	☐
User-Defined Critical Headway [s]	4.00		4.00	4.00		4.00	4.00	4.00	4.00	4.00	4.00	4.00
Overwrite Calculated Follow-Up Time	☐		☐	☐		☐	☐	☐	☐	☐	☐	☐
User-Defined Follow-Up Time [s]	3.00		3.00	3.00		3.00	3.00	3.00	3.00	3.00	3.00	3.00
A (intercept)	*1130.00*		*1130.00*	*1130.00*		*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*	*1130.00*
B (coefficient)	*0.00075*		*0.00070*	*0.00075*		*0.00070*	*0.00075*	*0.00075*	*0.00070*	*0.00075*	*0.00075*	*0.00070*
HV Adjustment Factor	*0.88*		*0.88*	*0.93*		*0.93*	*0.87*	*0.87*	*0.87*	*0.90*	*0.90*	*0.90*
Entry Flow Rate [veh/h]	*693*		*772*	*191*		*117*	*987*	*987*	*204*	*942*	*942*	*278*
Capacity of Entry and Bypass Lanes [ve	*224*		*249*	*202*		*227*	*803*	*803*	*821*	*560*	*560*	*587*
Pedestrian Impedance	*1.00*		*1.00*	*1.00*		*1.00*	*0.63*	*0.63*	*0.63*	*0.63*	*0.63*	*0.63*
Capacity per Entry Lane [veh/h]	*197*		*220*	*187*		*210*	*443*	*443*	*454*	*316*	*316*	*331*
X, volume / capacity	*3.10*		*3.10*	*0.94*		*0.52*	*1.94*	*1.94*	*0.39*	*2.69*	*2.69*	*0.76*

Saris Abo Roundabout: Future Condition Alternative 2

g / C, Green / Cycle	0.54	0.50	0.36	0.36	0.54	0.50	0.50	0.36	0.36	0.36
(v / s)_i Volume / Saturation Flow Rate	0.00	1.10	0.00	0.12	39675835.35	1.12	0.20	0.77	0.69	0.28
so, Base Saturation Flow per Lane [veh/	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Arrival type	3		3		3			3		
s, saturation flow rate [veh/h]	0	837	0	877	0	1347	901	959	1393	909
c, Capacity [veh/h]	120	418	60	314	120	674	451	382	499	326
X, volume / capacity	3.05	2.21	2.93	0.34	1.78	2.23	0.39	1.94	1.91	0.77
d, Delay for Lane Group [s/veh]	999.80	581.55	973.36	31.15	439.36	590.06	21.23	479.72	456.83	49.95
Lane Group LOS	F	F	F	C	F	F	C	F	F	D
Critical Lane Group	☐	☐	☐	☐	☑	☐	☐	☑	☐	☐
50th-Percentile Queue Length [veh]	33.18	76.00	17.10	2.63	14.50	123.95	3.45	57.78	72.60	7.92
50th-Percentile Queue Length [m]	252.81	579.15	130.27	20.07	110.45	944.47	26.28	440.26	553.25	60.34
95th-Percentile Queue Length [veh]	59.72	127.51	30.77	4.74	26.09	203.18	6.21	93.84	115.91	12.53
95th-Percentile Queue Length [m]	455.05	971.63	234.49	36.13	198.82	1548.21	47.30	715.04	883.24	95.51

Movement, Approach, & Intersection Results

d_M, Delay for Movement [s/veh]	999.80	581.55	581.55	973.36	31.15	31.15	439.36	590.06	21.23	479.72	464.99	49.95
Movement LOS	F	F	F	F	C	C	F	F	C	F	F	D
Critical Movement	☑	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
d_A, Approach Delay [s/veh]	700.22			615.05			519.94			413.31		
Approach LOS	F			F			F			F		
d_I, Intersection Delay [s/veh]	529.53											
Intersection LOS	F											

Control Type	Protected	Permissive	Permissive	Permissive	Permissive	Permissive	Protected	Permissive	Permissive	Permissive	Permissive	Permissive
Signal group	1	4	0	0	8	0	9	2	0	0	6	0

Sequence

Ring 1	8	4	1	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 2	6	2	9	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ring 4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-