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SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

THE EFFECTS OF TRAFFIC MIXES WITH 3-WHEELERS ON PERFORMANCE OF UNSIGNALIZED INTERSECTIONS IN HARAR CITY

By: BELAY SEYOUM GONFA

A Thesis Submitted to the School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science in Road and Transport Engineering

Advisor: BIKILA TEKLU (PhD)

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Addis Ababa, Ethiopia

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Approval by Board of Examiners

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Declaration

I, the undersigned, certify that this research work titled *"The Effects of Traffic Mixes with 3-Wheelers on Performance of Unsignalized Intersections in Harar City"* is my original work performed under the supervision of my research advisor Dr. Bikila Teklu and it has not been presented elsewhere for assessment and for a degree in any other university.

*Signature:*________________

Belay Seyoum Gonfa

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ABSTRACT

In heterogeneous traffic conditions, the performances of an intersection are described by vehicle and driver characteristics which are different from traffic conditions in homogeneous conditions. The absence of lane discipline results in vehicular movement that is influenced by the presence of vehicles in the front as well as on the sides. This led to a complex traffic behavior and it cannot be analyzed by using conventional microscopic and macroscopic traffic variables. In this study the relationship between different vehicle compositions and the performance measurement parameters for two roundabouts and one unsignalized T-intersection in Harar city are assessed by application of gap acceptance procedures.

All data which describes actual condition of the intersections were collected to calibrate the existing condition into gap acceptance procedures of highway capacity manual. Critical gaps follow up times and passenger car equivalents representing the existing traffic conditions of the study area are estimated. The analysis is made in 15minute interval over a range of 24 consecutive time series to indicate the trends of change of the performance of intersections in relation to the change of vehicle compositions and flow characteristics. Highway capacity manual 2000 was used for measurement of performances and multiple regressions was the analysis method applied to indicate the effects of vehicle classes in the study

The results are interpreted by five performance measurement parameters. They are approach capacity, degree of saturation, queue length, control delay and LOS. The model which indicates the relationship between these parameters and the vehicle compositions are formulated in the form of multiple linear regression equations with R^2 greater than 0.85 for most approaches of the selected intersections. Cross validation was made on the models to estimate how accurately a predictive model will perform in practice. Peak hour performances are compared with the presence of bajaj vehicles and with bajajs replaced by minibus taxis.

Finally, the result indicates that some of the approaches of the intersections selected are providing service closer or above the capacity during peak hour of traffic flows. Besides, the presences of Bajajs were significantly affecting the performances of the intersection. A replacement of bajaj vehicles by 12 seat minibus taxis indicates a very good improvement of performances of the intersections in Harar city. For Arategna roundabout the average delay will be reduced by 7 minutes per vehicle if bajajs are replaced by minibus taxis during peak hour. The queue lengths will be reduced by 85 vehicles and the capacity will be increased by 396veh/hr for peak periods. Sillassie roundabout will be improved by decrease of average delay by 95 sec/veh, decrease of queue length by 34 vehicles and increase of capacity by 531veh/hr during peak hour if bajajs are replaced by minibus taxis. Thus, the effect of traffic mix with three wheelers is very significant on performances of unsignalized intersections.

KEY WORDS: Unsignalized, Bajajs (3-wheelers), Roundabout, Capacity, Composition, Delay, Performance, Intersection, Traffic Flow

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List of Acronyms

AASHTO – American Association of State Highway and Transportation Officials

HCM – Highway Capacity Manual

FHWA – Federal Highway Administration

RT –Right Turn

LT – Left Turn

TT – Through Movement

UT – U-Turn

TT1 – Through Movement 1

TT2 – Through Movement 2

RT1- Right Turn 1

RT 2 – Right Turn 2

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

As the traffic in the existing road system in cities grows, congestion, delay, environmental pollution and high energy consumption become a serious problem. From the elements of existing road system, intersections are considered those locations with complex nature. Thus, a thorough understanding of them needs to be achieved in order to design them in the most effective manner. (Mathew et al, 2007)

Unsignalized intersections are the most commonly found intersection on rural roads as well as urban roads. They seem to be trade-off between safety and delay. The safe gap required to cross becomes an individual judgment and error or misjudgment leads to accident. Being over cautious to cross increases the stopped delay. Delay of traffic at the uncontrolled intersection affects the overall performance of the road network. (Rao, 2015)

Most of the capacity analysis procedures of unsignalized intersections are based on stochastic models; i.e., gap-acceptance theory. Capacity is a function of the availability of critical gaps. A critical gap is the minimum time interval between successive major-stream vehicles, in which a minor-street vehicle can make a maneuver. In addition to capacity and delay, estimation of level of service and queue length, estimation of operating cost, fuel consumption and pollutant emissions for evaluating intersection are also important for design, operations and planning purposes in traffic management as well as in improving the performance of intersections in urban areas (Prasetijo, 2007). The detail analysis of intersections can be also made by micro-simulation models that are developed recently. But many researchers believe that the use of these models has got some limitations in relation to actual conditions.

Several capacity analysis models exist and can be classified into two broad categories - theoretical and empirical. Highway Capacity Manual (HCM 2000 or 2010) capacity model is an analytical (exponential regression) model with clear basis in gap-acceptance theory.

In heterogeneous traffic conditions, intersection capacity is described by vehicle and driver characteristics which are different from traffic conditions in homogeneous conditions. The absence of lane discipline results in vehicular movement that is influenced by the presence of vehicles in the front as well as on the sides. This led to a complex traffic behavior and it cannot be analyzed by using conventional microscopic and macroscopic traffic variables. (Brilon, 2002)

Most of the intersections in Ethiopia are unsignalized. The vast majority of unsignalized intersections have yield signs on minor road approaches. Other unsignalized intersections are either stop-controlled or uncontrolled, observing the “right before left” principle. Owners and users of adjacent land often have a direct interest in intersection design, particularly where the intersection is surrounded by retail, commercial, historic or institutional land uses. Primary concerns include maintenance of vehicular access to private property, turn restrictions, consumption of private property for right-of-way, and provision of safe, convenient pedestrian access.

Harar city is one of the most sought after cities in the country with traffic composition of pedestrian and motorized vehicles such as bajaj, taxi, bus, truck etc. The city has several straight roadways with a number of different intersection controls such as roundabouts, traffic signals, stop and yield control intersections. From all intersections found in the city, only one of them is signalized while all the others are unsignalized intersections. The effects of different vehicle categories on the performance of unsignalized are very different in nature. Therefore time series analyses of the effects of different vehicle sizes on intersection performance are essential in the study of traffic flows. Specifically, the relationship between Bajaj composition and performance parameters are needed to be studied since Bajaj mode of transportation is becoming abundant public transportation mode in Harar city and other cities in Ethiopia.

1.2 Statement of the Problem

The investigation and performance evaluation for unsignalized intersection for different cities will help us to handle with the problem of congestion, delay, emission and low level of service. From all intersections found in Harar city, only one of them is signalized. Therefore unsignalized intersections are the most abundant intersections in that city.

Most of the intersections in Harar city are characterized by mixes of different sizes of vehicles starting from very small three wheelers to large truck & trailers. These will highly affect the traffic operation around the intersections. A study made by Rao (2015) in India indicated that different vehicle categories have different effects on the traffic flow around the intersection. Mallikarjuna (2014) also showed that heavy vehicles and light vehicles have different kind of effects on performance parameters of intersections.

In contrast to traffic flow condition in developed countries, by which many softwares developed, the traffic condition in Harar city is totally different. Apart from the different driver classes, vehicles with various performance and dimensional characteristics (especially traffic is predominantly occupied by small sized vehicles such as 3 wheelers (locally called Bajajs), non-lane discipline, and creeping behavior of drivers are characterized a totally complex traffic environment. It requires special attention in modeling traffic flow behavior. Traffic control devices such as pavement markings and traffic signs are not fully applied for the intersection approaches in Harar city. Hence, the use of softwares, which considers the best road conditions with all its control mechanisms, will not provide the best results for these kinds of unsignalized intersections. Thus, manual method of performance analysis is necessary to capture the conditions of the intersections. The study shall be performed by investigating the change of the performances from time to time so that the trends of the variation of performance parameters and vehicle compositions can be known under the prevailing conditions.

1.3 Research Questions

The basic research questions are as follow:

- What are the parameters to estimate the performance of unsignalized intersections under the given traffic and road conditions?
- How the performance measurement parameters are varying from time to time for a given intersections under heterogeneous traffic flows?
- What will be the performance improvement of the intersections if 3-wheelrs are replaced by minibus taxis as means of public transport in Harar city?

1.4 Objective of the Research

1.4.1 General Objective

This research aims to study the effect of compositions of different vehicle classes on performance measurement parameters of unsignalized intersections in Harar city.

1.4.2 Specific Objectives

The specific objectives addressed in this research are the followings:

- To estimate passenger car units (PCU), critical gaps and follow up times for the traffic movements at selected intersections.
- To analyze the intersection performance measurement parameters; ***capacity, degree of saturation, control delay, queue length*** *and* ***level of service*** for intersection approaches.
- To develop a model to analyze the effect of vehicle compositions with performance parameters.
- To compare the performances of unsignalized intersections with the presence of bajajs/ 3-wheelers and without the presence of bajajs, i.e. when bajaj vehicles are removed and replaced by minibus taxis.

1.5 Scope of the Research

This research is carried on two roundabouts and one unsignalized T-intersection which are located in Harar city. These intersections are considered as the major intersections in the city and located on the major road within the city. The findings of this research are specific to these selected intersections. The study analyzes the effect of vehicle composition on intersection performance parameters (capacity, degree of saturation, control delay, queue length and level of service). The study of the effect of pedestrians, geometry and any other variables other than vehicles compositions are out of scope of this research. In comparing the effect of bajajs and minibus taxis on unsignalized intersections, the comparison is from traffic performance point of view only. Detail cost benefit analysis of the comparison is not discussed in this research.

1.6 Limitations of the Research

The analysis methods used in this research are based on Highway Capacity Manual (HCM 2000) procedures. Even if this study tried to calibrate the data to the manual, the procedures may not fit the existing conditions of the intersections under this study. The manual do not have a complete procedure for multi lane roundabout analysis. In addition, the data collection period is limited to a single day for an intersection and only for six hours per day based on the recommendations from FDOT manuals. It may be difficult to capture the actual fluctuation of traffic flows for the intersections during this time periods.

1.7 Research Outline

This thesis consists of the five chapters:

- The first chapter of this thesis gives a general introduction of the overall thesis content and the general background of parameters involved in the analysis of roundabouts. The problem statements and objectives are discussed in this chapter
- The second chapter reviews the relevant literatures related capacity and other performance analysis of roundabouts and TWSC intersections.
- Chapter 3 discusses the study methodology carried out for this study. The relevant data collected for this research and the methodologies and equipments used for data collection are discussed.
- Chapter 4 discusses data analysis and results. The findings and theirs interpretation are discussed in this chapter.
- Chapter 5 is about conclusion of the findings and future scope of the studies based on the findings of this thesis.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction about Intersection

Intersection is an area shared by two or more roads. This area is designated for the vehicles to change their flow course to different directions to reach their desired destinations. Its main function is to guide vehicles to their respective directions. Traffic intersections are complex locations on any highway. This is because vehicles moving in different direction want to occupy same space at the same time. In addition, the pedestrians also seek same space for crossing. Drivers have to make split second decision at an intersection by considering his route, intersection geometry, speed and direction of other vehicles etc. A small error in judgment can cause severe accidents. It also causes delay and it depends on type, geometry, and type of control. (Roess, 2004)

Intersections are a key feature of street design in four respects: (Mathew et al, 2007)

i. Focus of activity - the land near intersections often contains a concentration of travel destinations.
ii. Conflicting movements - Pedestrian crossings and motor vehicle and bicycle turning and crossing movements are typically concentrated at intersections.
iii. Traffic control - at intersections, movement of users is assigned by traffic control devices such as yield signs, stop signs, and traffic signals. Traffic control often results in delay to users traveling along the intersecting roadways, but helps to organize traffic and decrease the potential for conflict.
iv. Capacity - in many cases, traffic control at intersections limits the capacity of the intersecting roadways, defined as the number of users that can be accommodated within a given time period.

The term intersection encompasses not only the area of pavement jointly used by the intersecting streets, but also those segments of the intersecting streets affected by the design. Thus, those segments of streets adjacent to the intersection for which the cross-section or grade has been modified from its typical design are considered part of the intersection. (Patel, 2015)

Intersections are generally classified into three general categories: At-grade intersections, Grade-separated without ramps, and Grade-separated with ramps (commonly known as interchanges). Most highways intersect at grade, and the intersection area should be designed to provide adequately for turning and crossing movements, with due consideration to sight distance, signs, and alignments. The basic types of at-grade intersections are **T**, **Y** or three-leg intersections, which consist of three approaches; four-leg or cross intersections, which consist of four approaches; multi leg intersections, which consist of five or more approaches; and roundabouts.

At grade intersections can be also classified as signalized and unsignalized intersections based on the traffic control mechanisms. Intersections at grade can be eliminated by the use of grade-separation structures that permit the cross flow of traffic at different levels without interruption. The advantage of such separation is the freedom from cross interference with resultant saving of time and increase in safety for traffic movements. (Kumar, 2014)

Roundabouts are a type of circular intersections in which traffic travels counterclockwise (in right-hand traffic countries) around a central island. Specific design and traffic control features define and distinguish roundabouts from traffic circles. These features include yield control of all entering traffic, channelized approaches that deflect traffic flow, and appropriate geometric curvature to ensure that travel speeds on the circulatory roadway within the specified limit. (FHWA, 2000)

Intersection design is a complex process where factors related to operational efficiency such as capacity, delay and emissions are an important consideration along with safety features and geometrical constraints. A poorly designed intersection may contribute to traffic congestion, increase in vehicular emissions and road accidents. The operational efficiency of intersections largely depends on the prevailing road, traffic and control conditions. In recent years, vehicular emissions have also been a major factor in intersection design. Environmentally-friendly alternatives are more important than ever before to minimize carbon footprints contributed by transport sector. (Borkloe et al, 2013)

2.2 Factors Affecting Traffic Movements at Intersection

The main factors which affect the traffic movements at intersections can be categorized as human factors, traffic factors and features of physical elements.

Human factors includes driving habits, ability to make decisions, driver expectancy, decision and reaction time, conformance to natural paths of movement, and pedestrian use and habits.

The following features of road intersections are also the major factors which affect intersection performances;

- Number of lanes
- Design speed
- Gradient
- Lane, shoulder and median width
- Traffic volume and composition of highway users, including trucks and transit vehicles
- Turning volumes

- Intersection sight distance
- proximity of adjacent intersections and
- Types of adjacent intersections

All roadway users are affected by intersection design as described below: (Nurhussien, 2015)

- **Pedestrians:** Key elements affecting intersection performance for pedestrians are:
  - Amount of right-of-way provided for the pedestrian including both sidewalk and crosswalk width, accuracy of slopes and cross slopes on curb cut ramps and walkways, audible and/or tactile cues for people with limited sight, and absence of obstacles in accessible path
  - Crossing distance and resulting duration of exposure to conflicts with motor vehicle and bicycle traffic
  - Volume of conflicting traffic, and
  - Speed and visibility of approaching traffic.
- **Bicyclists:** Key elements affecting intersection performance for bicycles are:
  - Degree to which pavement is shared or used exclusively by bicycles
  - Relationship between turning and through movements for motor vehicles and bicycles
  - Traffic control for bicycles
  - Differential in speed between motor vehicle and bicycle traffic and
  - Visibility of the bicyclist.
- **Motor vehicles:** Key elements affecting intersection performance for motor vehicles are:
  - Type of traffic control
  - Vehicular capacity of the intersection, determined primarily from the number of lanes and traffic control (although there are other factors)
  - Ability to make turning movements
  - Visibility of approaching and crossing pedestrians and bicycles, and
  - Speed and visibility of approaching and crossing motor vehicles.
- **Transit:** When transit operations involve buses, they share the same key characteristics as vehicles. In addition, transit operations may involve a transit stop at an intersection area, and

influence pedestrian, bicycle, and motor vehicle flow and safety. In some cases, the unique characteristics of light-rail transit must be taken into account.

2.3 Passenger Car Units

The traffic flow on any given section of road is composed of vehicles of different types, which have all different road–space requirements due to their respective size and performance characteristics. In order to allow this in highway capacity measurements, traffic volumes are expressed in passenger car units (PCUs) which represent the equivalent traffic impedance values of various types of vehicle as compared with a value of unity for the passenger car. (Khanorkar, 2014)

The PCU has been defined by the United Kingdom Transport and Road Research Laboratory as follows:

> *"On any particular section of road under particular traffic conditions, if the addition of one vehicle of a particular type per hour will reduce the average speed of the remaining vehicles by the same amount as the addition of, say x cars of average size per hour then one vehicle of this type is equivalent to x PCU."*

Many researchers developed different approaches of PCU estimation. The PCU value is affected by many factors like traffic volume, traffic composition, road geometry, speed and many others. The selection of the methods is thus based on the appropriateness of the method to the highways and intersections under study.

Chandra & Sikdar (1999) have developed a PCUs factor for a vehicle type based on dynamic and static vehicle performance and geometric variables. The procedures of analyzing the capacity calibrates for a specific set of ideal conditions, one of them is that the traffic stream contains only passenger cars. The adjustment factor for the presence of vehicles other than cars is based on PCUs. This adjustment factor correlates with the flow rates of passenger cars only and mixed traffic streams that are equivalent in terms of drivers' perception of the level of service LOS.

Muhammad Adnan (2014) studied on passenger car equivalent in heterogeneous traffic environment. Four different methods were used to estimate the PCU of vehicles. The study found that method that incorporate vehicles speed along with projected area of vehicles were provide appropriate estimate of PCE values.

Dhamaniya and Chandra (2014) worked on midblock capacity of urban arterial roads in India. They considered the speed and size of the vehicle as the prime variables for determination of PCU. The variation in PCU for different types of vehicles was established graphically. Rakha et al. (2007) estimated the truck equivalency factor for freeway sections at different grades. PCEs are developed for broader range of vehicle weight to power ratio in the INTEGRATION software using HCM procedure. The authors estimated PCE for truck at different LOS and 2 to 5 percent grades, when their proportion in the mix is more than 25 percent which was beyond the limit of HCM 2000. Rongviriyapanich and Suppattrakul (2005) estimated PCU values as the ratio between the headway of car following a car and subject vehicle following a same type of vehicle.

2.4 Capacity and Level of Service of Intersection

Highway Capacity Manual (HCM) 2000 defines capacity as;

> *"The capacity of a facility is the maximum hourly rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions."*

Highway capacity analysis serves three general purposes: (AASHTO, 2004)

i. Transportation planning studies: Highway capacity analysis is used in these studies to assess the adequacy or sufficiency of existing highway networks to service current traffic. In addition, it is used to estimate the time in the future when traffic growth may overtake the capacity of a highway or perhaps reach a level of congestion below capacity that is considered undesirable.
ii. Highway design: Knowledge of highway capacity is essential to properly fit a planned highway to traffic demands. Highway capacity analysis is used both to select the highway type and to determine dimensions such as the number of lanes and the minimum lengths of weaving sections.
iii. Traffic operational analysis: Highway capacity analysis is used in these analyses for many purposes, but especially for identifying bottleneck locations (either existing or potential). It is also used on preparing estimates of operational improvements that may be expected to result from prospective traffic control measures or from spot alterations in the highway geometry.

Capacity analysis, therefore, is a set of procedures for estimating the traffic-carrying ability of facilities over a range of defined operational conditions. It provides tools to assess facilities and to plan and design improved facilities.

Quality of service requires quantitative measures to characterize operational conditions within a traffic stream. **Level of service (LOS)** is a quality measure describing operational conditions within a traffic stream, generally in terms of such service measures as speed and travel time, freedom to maneuver, traffic interruptions, and comfort and convenience. (HCM 2000)

HCM 2000 provides six LOS defined for each type of facility that has analysis procedures available. Letters designate each level, from A to F, with LOS A representing the best operating conditions and LOS F the worst. Each level of service represents a range of operating conditions and the driver's perception of those conditions. Safety is not included in the measures that establish service levels.

2.4.1 Analysis of STOP Controlled Intersections

At two way stop controlled (TWSC) intersections, the stop-controlled approaches are referred to as the minor street approaches; they can be either public streets or private driveways. The intersection approaches that are not controlled by stop signs are referred to as the major street approaches.

Figure 2.1 illustrates the priority of movements at a typical four-leg and a typical T-intersection. In a four-leg intersection, the highest priority (Priority 1) movements include the through and right-turn movements on the major street, and the pedestrian movements crossing the minor street. These movements have the right of way over all minor-street movements at a STOP sign. Priority 2 movements include left turns from the major street, right turns from the Minor Street, and pedestrians crossing the major street. Left turns from the major street have first access to gaps in the opposing vehicular traffic stream; right turns from the minor street have first access to merge into gaps in the right-most approaching major-street lane. Pedestrians crossing the major street also have the right of way over vehicles seeking the same gaps. The through movements from the minor street are Priority 3, while left turns from the minor street are Priority-4.

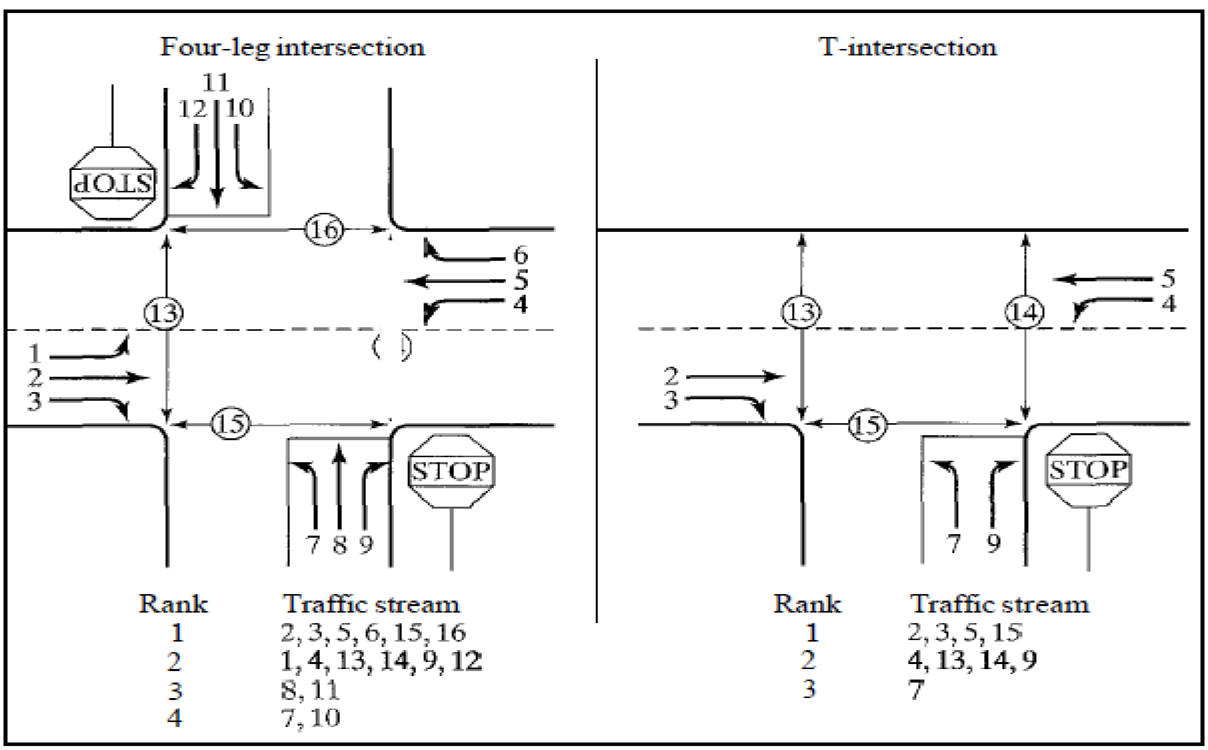

Figure 2.1: Priority of Movements at a TWSC Intersection (Source: HCM 2000)

Capacity analysis at TWSC intersections depends on a clear description and understanding of the interaction of drivers on the minor or stop-controlled approach with drivers on the major street. Both gap acceptance and empirical models have been developed to describe this interaction. Procedures described in HCM rely on a gap acceptance model developed and refined in Germany.

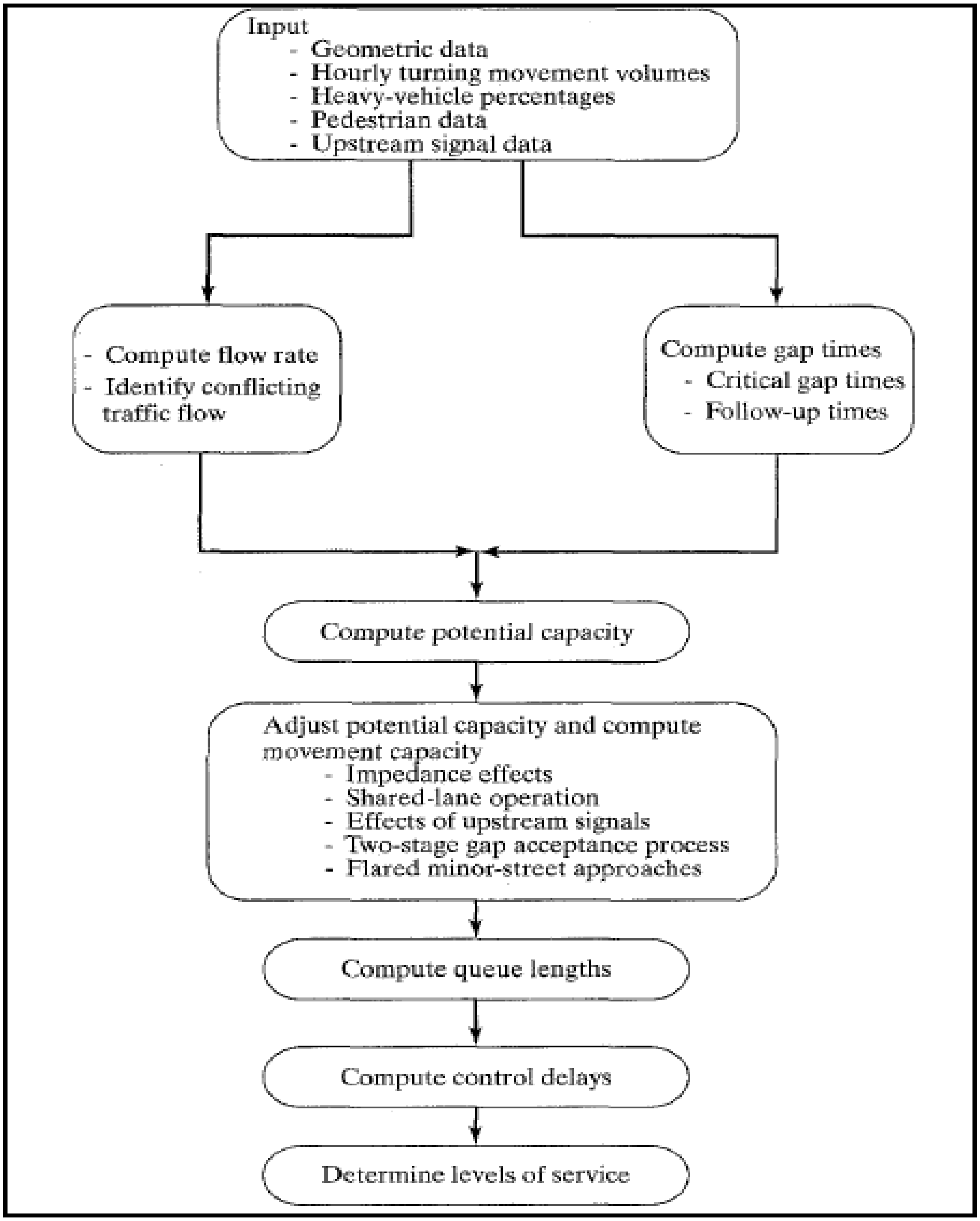

Figure 2.2: Flow chart for analysis of TWSC Intersections (Source: HCM 2000)

Level of service (LOS) for a TWSC intersection is determined by the computed or measured control delay and is defined for each minor movement. LOS is not defined for the intersection as a whole.

Table 2.1: Level of Service Criteria for TWSC intersections (Source HCM 2000)

Level of Service	Average Control Delay (s/veh)
A	0-10
B	>10-15
C	>15-25
D	>25-35
E	>35-50
F	>50

2.4.1.1 Determining Conflicting Volume

Each movement seeking gaps does so through a different set of conflicting traffic movements. Major-street left turns seeks gaps through the opposing through movement, the opposing right turn movement, and pedestrians crossing the far side of the minor street. Minor-street right turns seek to merge into the right-most lane of the Major Street, which contains through and right-turning vehicles. Each right turn from the minor street must also cross the two pedestrian paths shown. Through movements from the minor street must cross all major street vehicular and pedestrian flows. Minor street left turns must deal not only with all major-street traffic flows but with two pedestrian flows and the opposing minor-street through and right-turn movements. (HCM 2000)

The calculation of the "conflicting volume" (*vcx*) for movement "x" is presented in figure 2.3.

2.4.1.2 Critical Gaps and Follow-up Times

The "critical gap" (t_{cx}) for movement "x" is defined as the minimum average acceptable gap that allows intersection entry for one minor-street (or major-street left turn) vehicle. The term average acceptable means the average driver would accept (or choose to utilize) a gap of this size. The gap is measured as the clear time in the traffic stream(s) defined by all of the conflicting movements. Thus, the model assumes that all gaps shorter than *tcx* are rejected (or unused), while all gaps equal to or larger than *t,,* would be accepted (or used). (HCM 2000)

The "follow-up time" (t_f .) for movement *"x"* is the minimum average acceptable time for a second queued minor-street vehicle to use a gap large enough to admit two or more vehicles. Base critical gaps and follow-up times must be adjusted to account for a number of conditions, including heavy-vehicle presence, grade, and the existence of two stages gap acceptance. The critical gap is computed separately for each minor movement by Equations available on HCM 2000.

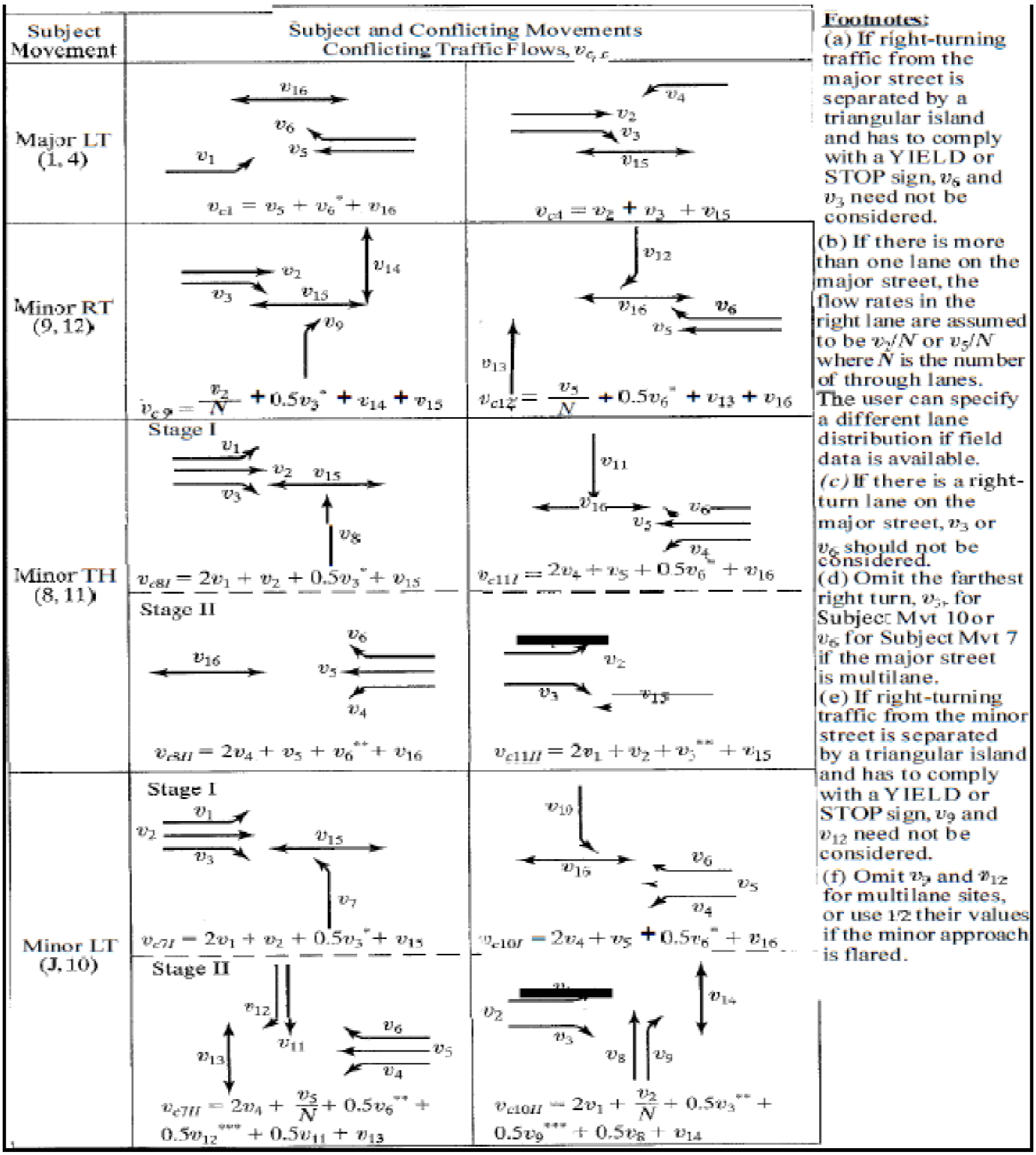

Figure 2.3: Definition and Computation of Conflicting Flows (Source: HCM 2000)

The following equation is recommended by HCM for determination of critical gaps.

$$\boxed{t_{c,x} = t_{c,base} + t_{c,HV} P_{HV} + t_{c,G}\, G - t_{c,T} - t_{3,LT}}$$

- $t_{c,x}$ = critical gap for movement x (s),
- $t_{c,base}$ = base critical gap from table 2 (s),
- $t_{c,HV}$ = adjustment factor for heavy vehicles (1.0 for two-lane major streets and 2.0 for four-lane major streets) (s),
- P_{HV} = proportion of heavy vehicles for minor movement,
- $t_{c,G}$ = adjustment factor for grade (0.1 for Movements 9 and 12 and 0.2 for Movements 7, 8, 10, and 11) (s),
- G = percent grade divided by 100
- $t_{c,T}$ = adjustment factor for each part of a two-stage gap acceptance process (1.0 for first or second stage; 0.0 if only one stage) (s), and
- $t_{3,LT}$ = adjustment factor for intersection geometry (0.7 for minor-street left-turn movement at three-leg intersection; 0.0 otherwise) (s).

Table 2.2: Base Critical Gaps and Follow-Up Times for TWSC Intersections (Source: HCM 2000)

Vehicle Movement	Base Critical Gap, $t_{c,base}$ (s)		Base Follow-up Time, $t_{f,base}$ (s)
	Two-Lane Major Street	Four-Lane Major Street	
Left turn from major	4.1	4.1	2.2
Right turn from minor	6.2	6.9	3.3
Through traffic on minor	6.5	6.5	4.0
Left turn from minor	7.1	7.5	3.5

The follow-up time is computed for each minor movement using the following equation. Adjustments are made for the presence of heavy vehicles. [HCM 2000]

$$t_{f,x} = t_{f,base} + t_{f,HV}\, P_{HV}$$

Where,

- $t_{f,x}$ = follow-up time for minor movement x (s),
- $t_{f,base}$ = base follow-up time from Exhibit 17-5 (s),
- $t_{f,HV}$ = adjustment factor for heavy vehicles (0.9 for two-lane major streets and 1.0 for four-lane major streets), and
- P_{HV} = proportion of heavy vehicles for minor movement

Many researchers recommend that the critical gap and follow-up times recommended by HCM should be adjusted to the actual site condition accordingly. It is preferable if the critical gap and follow-up data are collected for the intersections under study.

2.4.1.3 Potential Capacity

The potential capacity of a movement is denoted as $c_{p,x}$ (for movement x) and is defined as the capacity for a specific movement, assuming the following base conditions:

- Traffic from nearby intersections does not back up into the subject intersection.
- A separate lane is provided for the exclusive use of each minor-street movement.
- An upstream signal does not affect the arrival pattern of the major-street traffic.
- No other movements of Rank 2, 3, or 4 impede the subject movement

The gap acceptance model used in this method computes the potential capacity of each minor traffic stream in accordance with this formula; (HCM 2010)

$$\boxed{c_{p,x} = v_{c,x} \frac{e^{-v_{c,x} t_{c,x}/3600}}{1 - e^{-v_{c,x} t_{f,x}/3600}}}$$

Where,

- $c_{p,x}$ = potential capacity of minor movement x (veh/h),
- $v_{c,x}$ = conflicting flow rate for movement x (veh/h),
- $t_{c,x}$ = critical gap (i.e., the minimum time that allows intersection entry for one minor-stream vehicle) for minor movement x (s), and
- $t_{f,x}$ = follow-up time (i.e., the time between the departure of one vehicle from the minor street and the departure of the next under a continuous queue condition) for minor movement x (s).

2.4.1.4 Movement Capacity

Potential capacities must first be adjusted to reflect the impedance effects of higher-priority movements that may utilize some of the gaps sought by lower-priority movements. This impedance may come from both pedestrian and vehicular sources. The movement capacity is found by multiplying the potential capacity by an adjustment factor. The adjustment factor is derived as the product of the probability that each impeding movement will be blocking a subject vehicle. (HCM 2000)

Vehicles use gaps at a TWSC intersection in a prioritized manner. When traffic becomes congested in a high-priority movement, it can impede lower-priority movements (i.e., streams of Ranks 3 and 4) from using gaps in the traffic stream, reducing the potential capacity of these movements. Major traffic streams of Rank 1 are assumed to be unimpeded by any of the minor traffic stream movements. This rank also implies that major traffic streams are not expected to incur delay or slowing as they travel through the TWSC intersection. Empirical observations have shown that such delays do occasionally occur, and they are accounted for by using adjustments provided in the procedures. Minor traffic streams of Rank 2 (including left turns from the major street and right turns from the minor street) must yield only to the major-street through and right-turning traffic streams of Rank 1. There are no additional impedances from other minor traffic streams, and so the movement capacity of each Rank 2 traffic stream is equal to its potential capacity as indicated by equation below. [HCM 2000]

$$c_m,j = c_p,j \ ,$$

Where; c_m,j = movement capacity for movement j of rank 2 priority

c_p,j = potential capacity for movement j of rank 2 priority

Highway capacity manual 2000 provides the formulas for movement capacity for different rank movements. For further discussion HCM 2000 can be referred.

Minor-street vehicle streams must yield to pedestrian streams.

2.4.2 Analysis of Roundabouts

Many methods applicable to two-way stop-controlled and two-way yield controlled intersection capacity are used as the foundation for the evaluation of roundabout performances. Roundabout analysis models are generally divided into two categories: (HCM 2000)

i. statistical (empirical) models based on the regression of field data;
ii. Analytical (semi-probabilistic) models based instead on the gap-acceptance theory.

Empirical models correlate geometric features and performance measures, such as capacity, average delay and queue length, through the regression of field data. In this way they generate a relationship (generally linear or exponential) between the entering flow of an approach and the circulating flow in front of it. These models are better than analytical ones but require a great number of congested (oversaturated conditions) roundabouts for calibration and may have poor transferability to other countries. (Lenters, 2010)

Gap-acceptance models can be developed instead from uncongested sites: the driver on the approach (entering flow) needs to select an acceptable gap in the circulating stream, to carry out the entering maneuver. (Akcelik, 2009)

2.4.2.1 Entry and Circulating Flow Determination

Entry flow and circulating flow for each approach are the volumes of interest for roundabout capacity analysis, rather than turning movement volumes. (FHWA, 2000)

The relationship between the standard origin-to-destination turning movements at an intersection and the circulating and entry flows at a roundabout is important, yet is often complicated to compute, particularly if an intersection has more than four approaches. For conventional intersections, traffic flow data are accumulated by directional turning movement, such as for the northbound left turn. For roundabouts, however, the data of interest for each approach are the entry flow and the circulating flow. Entry flow is simply the sum of the through, left, and right turn movements on an approach. Circulating flow is the sum of the vehicles from different movements passing in front of the adjacent up stream splitter-island. At existing roundabouts, these flows can simply be measured in the field. Right turns are included in approach volumes and require capacity, but are not included in the circulating volumes downstream because they exit before the next entrance. (FHWA, 2000)

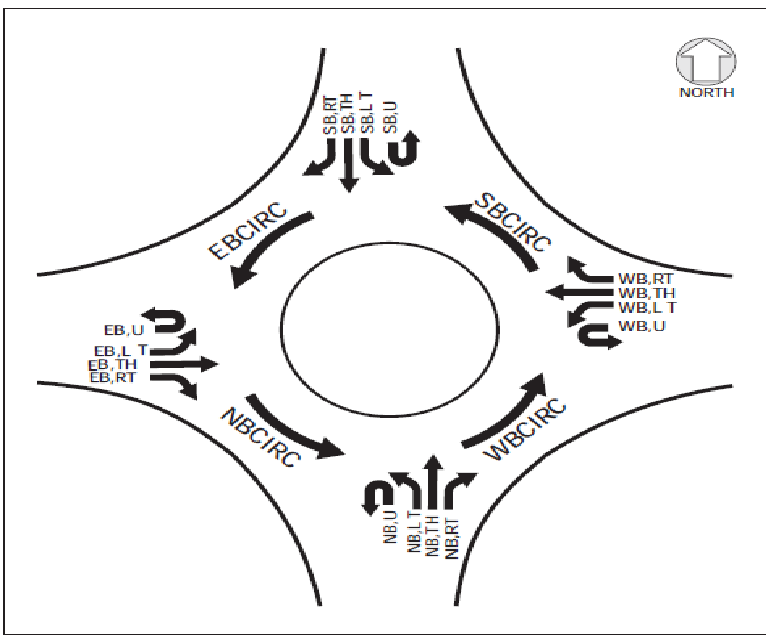

Figure 2.4: Traffic Flows for four-legged Roundabouts (Source: US Department of Transportation, FHWA)

For proposed or planned four-legged roundabouts, the following equation can be applied to determine conflicting (circulating) flow rates, as shown graphically in figure 2.4

$$V_{EB,circ} = VW_{B,LT} + V_{SB,LT} + V_{SB,TH} + V_{NB,U\text{-}turn} + V_{WB,U\text{-}turn} + V_{SB,U\text{-}turn} \quad (1)$$

$$V_{WB,circ} = V_{EB,LT} + V_{NB,LT} + V_{NB,TH} + V_{SB,U\text{-}turn} + V_{EB,U\text{-}turn} + V_{NB,U\text{-}turn} \quad (2)$$

$$V_{NB,circ} = V_{EB,LT} + V_{EB,TH} + V_{SB,LT} + V_{WB,U\text{-}turn} + V_{SB,U\text{-}turn} + V_{EB,U\text{-}turn} \quad (3)$$

$$V_{SB,circ} = V_{WB,LT} + V_{WB,TH} + V_{NB,LT} + V_{EB,U\text{-}turn} + V_{NB,U\text{-}turn} + V_{WB,U\text{-}turn} \quad (4)$$

2.4.2.2 Capacity of Roundabouts

The capacity of each entry to a roundabout is the maximum rate at which vehicles can reasonably be expected to enter the roundabout from an approach during a given time period under prevailing traffic and roadway (geometric) conditions. An operational analysis considers a precise set of geometric conditions and traffic flow rates defined for a 15-minute analysis period for each roundabout entry. While consideration of Average Annual Daily Traffic volumes (AADT) across all approaches is useful for planning purposes, analysis of this shorter time period is critical to assessing the level of performance of the roundabout and its individual components. (Lenters, 2010)

When the circulating flow is low, drivers at the entry are able to enter the roundabout without significant delay. The larger gaps in the circulating flow are more useful to the entering drivers and more than one vehicle may enter each gap. As the circulating flow increases, the size of the gaps in the circulating flow decrease, and the rate at which vehicles can enter also decreases. Note that when computing the capacity of a particular leg, the actual circulating flow to use may be less than demand flows, if the entry capacity of one leg contributing to the circulating flow is less than demand on that leg. (FHWA, 2000)

The geometric elements of the roundabout also affect the rate of entry flow. The most important geometric element is the width of the entry and circulatory roadways, or the number of lanes at the entry and on the roundabout. Two entry lanes permit nearly twice the rate of entry flow as of one lane. Wider circulatory roadways allow vehicles to travel alongside, or follow, each other in tighter bunches and so provide longer gaps between bunches of vehicles. The flare length also affects the capacity. The inscribed circle diameter and the entry angle have minor effects on capacity. (FHWA, 2000)

HCM 2010 describes an exponential model of capacity for single lane and two lane roundabouts. The manual states that the formula is a combination of simple lane base regression and gap acceptance models. In other words, the model can be viewed both as empirical (exponential regression) model and a gap- acceptance model.

$$c_{pce} = Ae^{(-Bv_c)} \quad \text{(pc/h)}$$

$$A = \frac{3,600}{t_f} \quad B = \frac{t_c - (t_f / 2)}{3,600}$$

Where;

- C_{pce} = Entry capacity (pc/h),
- v_c = circulating traffic flow (pc/h),
- t_c = critical gap (s), and
- t_f = follow-up time (s).

2.5 Performance Analysis of Intersections

Three performance measures are typically used to estimate the performance of a given roundabout design: degree of saturation, delay, and queue length. Each measure provides a unique perspective on the quality of service at which a roundabout will perform under a given set of traffic and geometric conditions. Whenever possible, the analyst should estimate as many of these parameters as possible to obtain the broadest possible evaluation of the performance of a given roundabout design. In all cases, a capacity estimate must be obtained for an entry to the roundabout before a specific performance measure can be computed. (FHWA, 2000)

A. Degree of Saturation

Degree of saturation is the ratio of the demand at the roundabout entry to the capacity of the entry. It provides a direct assessment of the sufficiency of a given design. While there are no absolute standards for degree of saturation, the Australian design procedure suggests that the degree of saturation for an entry lane should be less than 0.85 for satisfactory operation. When the degree of saturation exceeds this range, the operation of the roundabout will likely deteriorate rapidly, particularly over short periods of time. Queues may form and delay begins to increase exponentially.

B. Delay

Delay is a standard parameter used to measure the performance of an intersection. The Highway Capacity Manual identifies delay as the primary measure of effectiveness for both signalized and unsignalized intersections, with level of service determined from the delay estimate. Currently, however, the Highway Capacity Manual only includes control delay, the delay attributable to the control device. Control delay is the time that a driver spends queuing and then waiting for an acceptable gap in the circulating flow while at the front of the queue. The formula for computing this delay is given in equation below: Figure 2.5 shows how control delay at an entry varies with entry capacity and circulating flow. Each curve for control delay ends at a volume-to-capacity ratio of 1.0, with the curve projected beyond that point as a dashed line.

$$d = \frac{3600}{c_{m,x}} + 900T \times \left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}} \right]$$

Where; d = average control delay, sec/veh;

v_x = flow rate for movement x, veh/h;

c_{mx} = capacity of movement x, veh/h; and

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

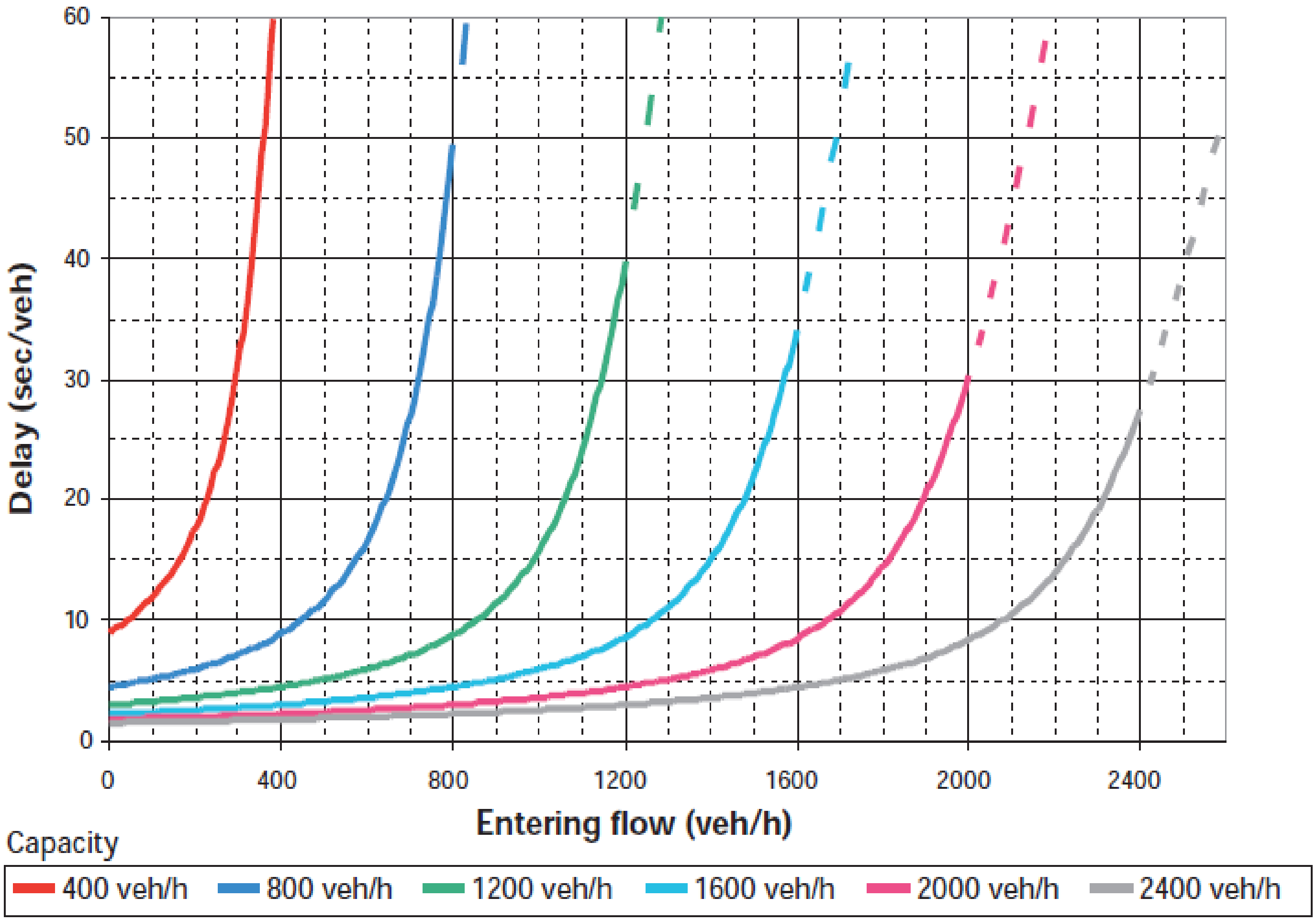

Figure 2.5: Control Delay as a function of Capacity and Entering Flow for ***Single Lane Roundabout*** *(Source: HCM 2000)*

C. Queue Length

Queue length is important when assessing the adequacy of the geometric design of the roundabout approaches. For design purposes, Highway capacity manual provides an equation and a graph for computation of queue lengths. Figure 2.6 shows how the 95th-percentile queue length varies with the degree of saturation of an approach. The x-axis of the graph is the degree of saturation, or the ratio of the entry flow to the entry capacity. Individual lines are shown for the product of T and entry capacity. To determine the 95th-percentile queue length during time T, enter the graph at the computed degree of saturation. Move vertically until the computed curve line is reached. Then move horizontally to the left to determine the 95th-percentile queue length. Alternatively, the equation below can be used to approximate the 95th-percentile queue. Note that the graph and equation are only valid where the volume-to-capacity ratio immediately before and immediately after the study period is no greater than 0.85 (in other words, the residual queues are negligible). [HCM 2000]

$$Q_{95} \approx 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(1-\frac{v_x}{c_{m,x}}\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\right]\left(\frac{c_{m,x}}{3600}\right)$$

Where: Q_{95}= 95th percentile queue, veh, v_x = flow rate for movement x, veh/h,

$c_{m,x}$= capacity of movement x, veh/h, and

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

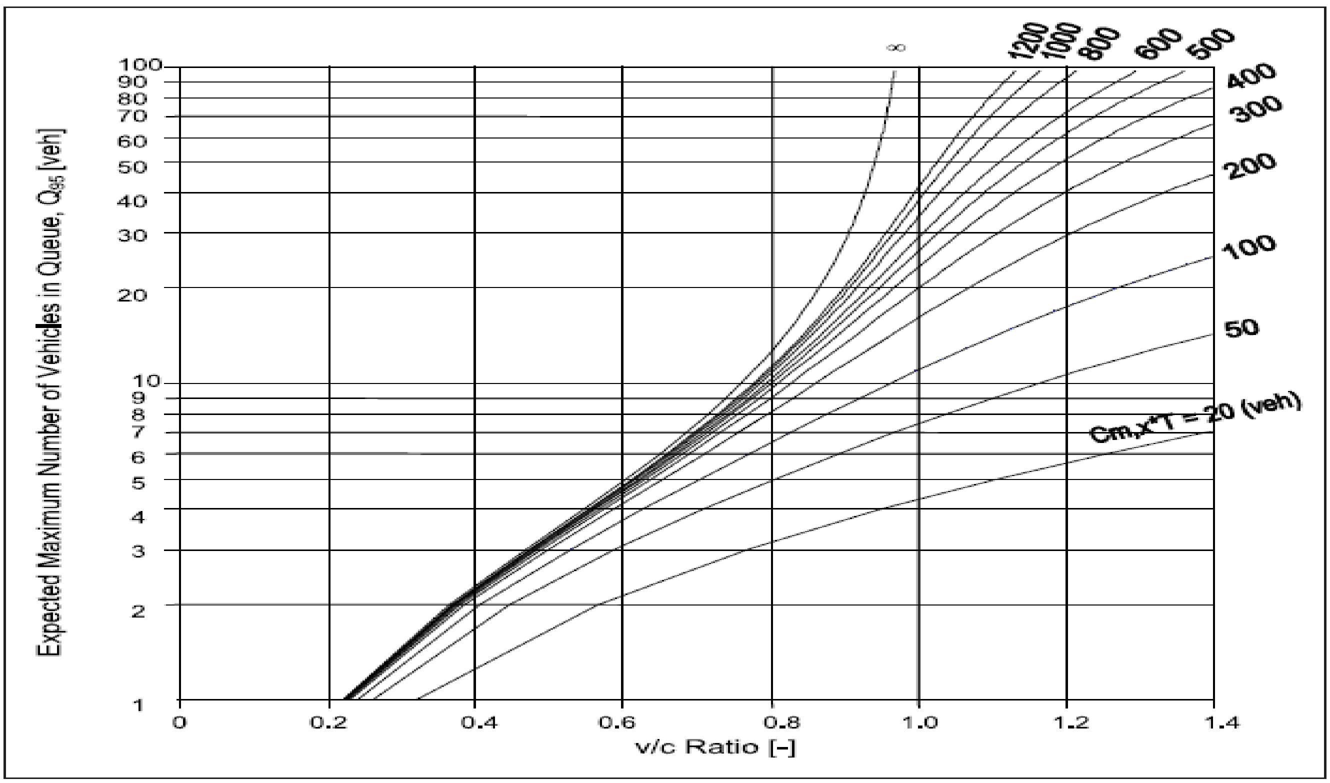

*Figure 2.6: 95th percentile Queue Length Estimation for **Single Lane Roundabout** (Source: HCM 2000)*

2.6 The Effect of Vehicle Types on Intersection Performance

Much research has been conducted to understand the effects of different vehicle types on the performance of intersections. For example, Webster and Cobbe (1966) estimated a passenger-car equivalency (PCE) value of 1.75 for heavy and medium goods vehicles at signalized intersection. Miller (1968) obtained a PCE estimate of 1.85 by measuring the additional time required by such trucks to cross an intersection.

Research by Kockelman (1998) indicates that length contributes negatively to highway flows. She analyzed different third-order-polynomial models of flow versus density interacted with other explanatory variables, and the elasticity of flow with respect to average vehicle length was estimated to be –17.4%. The average sales-weighted length of new vans and pickups is about 14% more than that of new passenger cars, while the average new sport-utility vehicle length is about equal to that of the average car (Kockelman, 1998).

In terms of vehicle performance, acceleration characteristics are likely to be highly correlated with horsepower-to-curb weight ratios. For new vehicle sales in 1997, these ratios for SUVs and pickups are about 10% less than that of cars, and almost 20% less in the case of vans. (Shabih et al, 2000)

One of the most important studies performed to determine the effect of heavy vehicles on unsignalized intersections in Turkey was carried out by Gedizlioğlu, 1979. In his study, Gedizlioğlu investigated the critical gap values that the heavy vehicles and passenger cars in the minor flow. Having aimed to examine the heavy vehicle effect on urban traffic, Gedizlioğlu chose three vehicle types for comparison, i.e. passenger car, minibus, and bus. The study showed that the critical gap value for minibuses was very close to, and even in some cases smaller than, that of passenger cars. Therefore, no PCE value was calculated for minibuses in the minor flow. When the ratio of the critical gap value for buses to the critical gap value for passenger cars *(Tbus/Tpassenger car)* was calculated, a difference was determined in the range “1.17~1.74 passenger car equivalence/vehicle”. Gedizlioğlu further emphasized that the values calculated for the ratio ($T_{bus}/T_{passenger\ car}$) were concentrated in the range “1.20~1.30”. (Gedizlioglu, 1979)

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Description of Study Area

Harar is a walled city in eastern Ethiopia. It was formerly the capital of Hararghe and now the capital of the modern Harari Region of Ethiopia. The city is located on a hilltop in the eastern extension of the Ethiopian Highlands, about five hundred kilometers from Addis Ababa at an elevation of 1,885 meters. Based on figures from the Central Statistical Agency in 2005, Harar had an estimated total population of 122,000.

The traffic movements in Harar is Characterized with a lot of small vehicles like Bajajs and old taxis, and pedestrians. According to the information from Harar city's transportation bureau, in 2016 there were about 300 three wheeler taxis (commonly known as bajajs), 65 old 4 seat taxis (locally called pejo taxis), 5 public buses, and 30 minibus taxis registered by the city authority to give service for about 150,000 population. These number of vehicles will fluctuate from time to time as number of vehicles in surrounding Oromia towns like Aweday, Haramaya, Babile and Fadis will also give service to the city's population. Not only within the city, there are also transportation between the city and the towns by all mentioned transport means including bajajs.

People use to move to the work early in the morning and back from work early in the afternoon in the city. Mid-day, 12:00pm to 2:00 pm, is the most unstable time of the traffic movement. Unsignalized intersections are the most abundant intersections in Harar. During the period of this research there was only one signalized intersections in the city.

Figure 3.1: Location of selected Intersections in Harar City (Source: Google Map)

3.1.1 Description of the Selected Intersections

In Harar city there are more than twenty unsignalized intersections. Roundabouts, T-intersections, four leg intersections, and signalized intersection are the most common one. Some of the well known intersection by their name are NOC intersection, Arategna intersection, Shash garage intersection, stadium intersection, Hajib intersection (signalized), Ras mekonnen roundabout, Sillasie intersection, Medhanialem intersection, Polis meda intersection, Shankor intersection, Dessie hotel roundabout, Shewa bar intersection, Mebrat hayil intersection, Yimaj intersection etc. From these intersections, the researcher of this study selected three well known intersections.

A. Arategna Intersection

Arategna intersection is a roundabout intersection located at an entrance to the centre of the city. The area is the well known commercial centre and characterized by a long queue length of vehicles during peak hour. It has four approaches known as NOC approach, Shash approach, Fadis approach and Warwari approach. NOC approach is a six lane (two directions) highway while both Fadis approach and Shash approach are a four lane two way highways. Warwari approach is a two lane (one lane in one direction) approach.

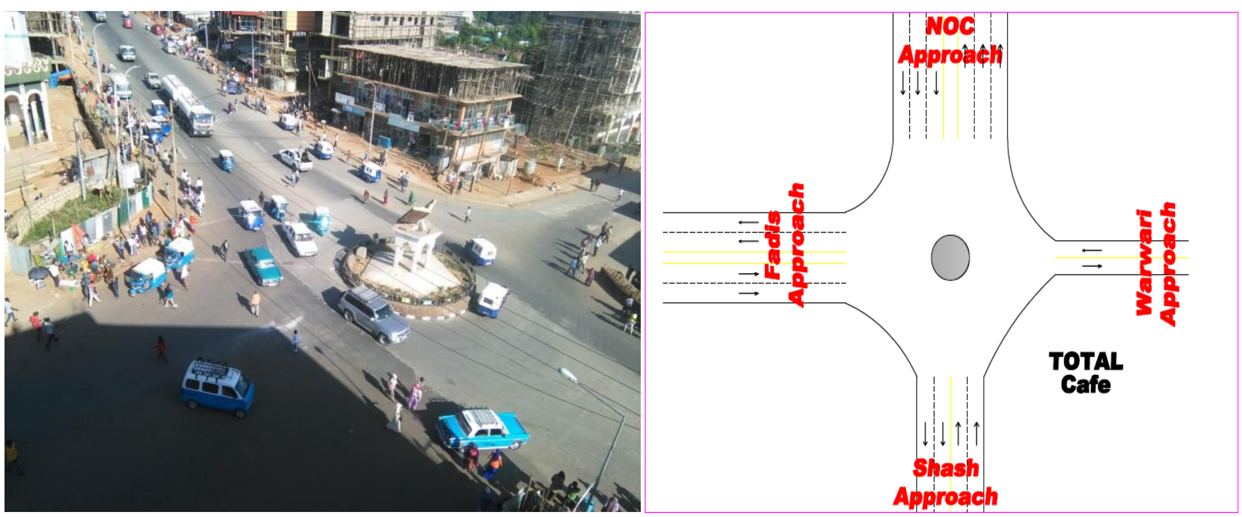

Figure 3.2: Arategna Roundabout (photo and plan view)

B. Shash Garage Intersection

The intersection which is located around Shash garage on the way from Arategna to Hajib is an unsignalized T – Intersection. It has three approaches known as Bira approach, Hajib approach and Shash approach. Both Shash and Hajib approach is a major highway with four lanes in both directions while Bira approach is a minor road with two lanes in both directions. The STOP sign is placed at the end of Bira approach. The area is characterized by high volume of Bajaj vehicles.

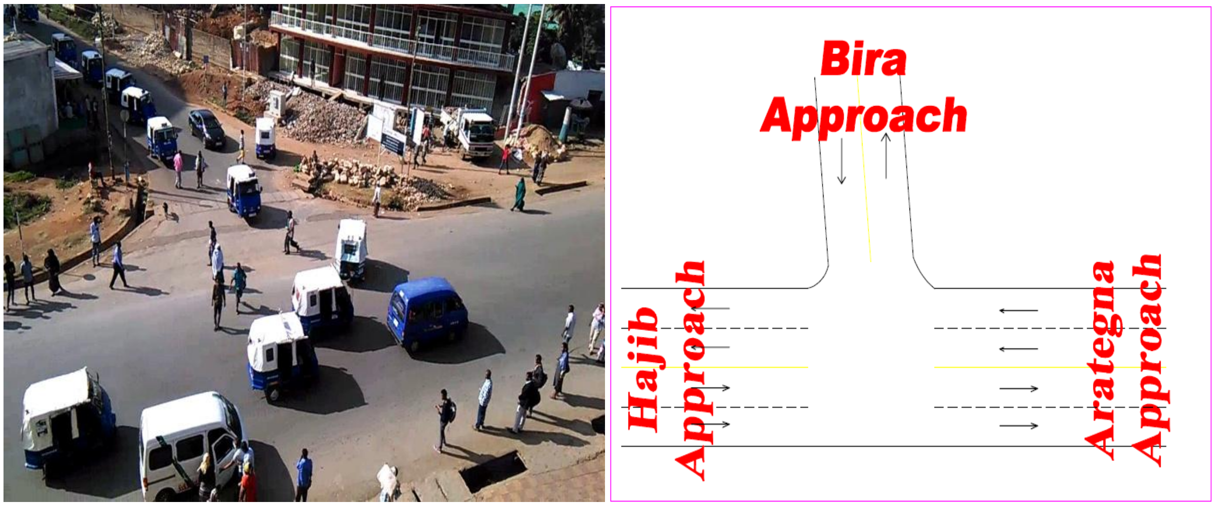

Figure 3.3: Shash Garage Intersection (Photo and plan view)

C. Sillassie Intersection

Sillassie intersection is a special kind of intersection where there are two islands side by side; a roundabout and a triangular yellow colored island to channelize the movement of vehicles. It is located at the heart of central business district (CBD) in Harar city. It has four approaches known as Rasmekonnen approach, Bote approach, Shewabar approach and Andegna menged approach. Both Rasmekonnen approach and Andegna menged approach are the part of the major highway in the city which has a six lane in both directions. They have a wide divided median which serves as a green area too. The other two approaches (Bote and Shewabar) are a two way two lane roads characterized by congested pedestrians because of the roadside marketing practices.

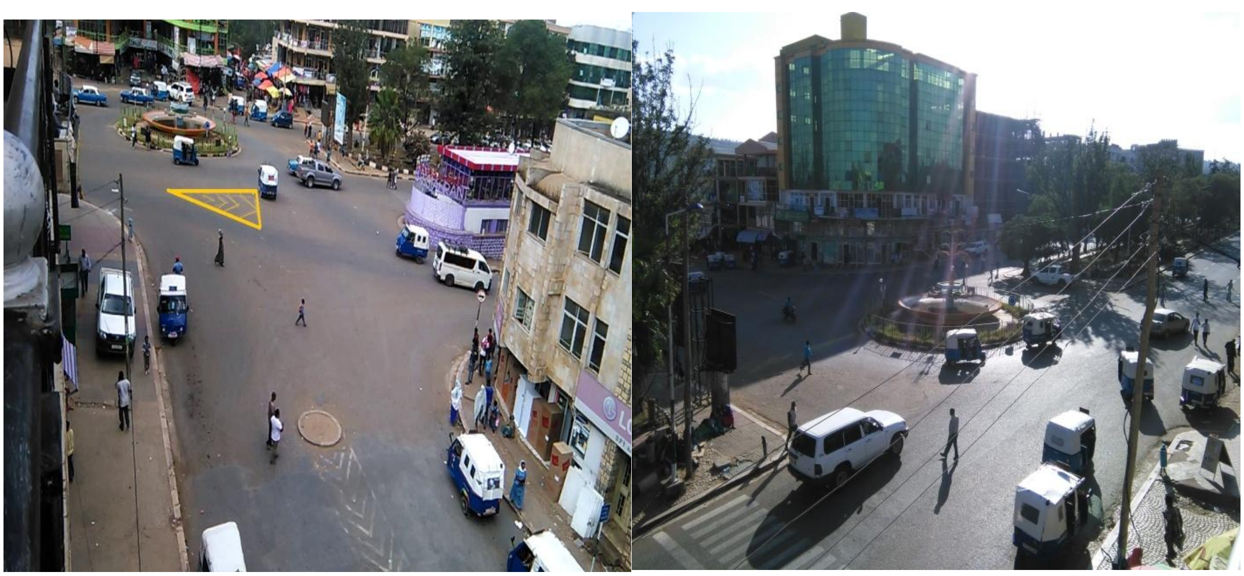

Figure 3.4: Sillassie Intersection (Views from different perspective)

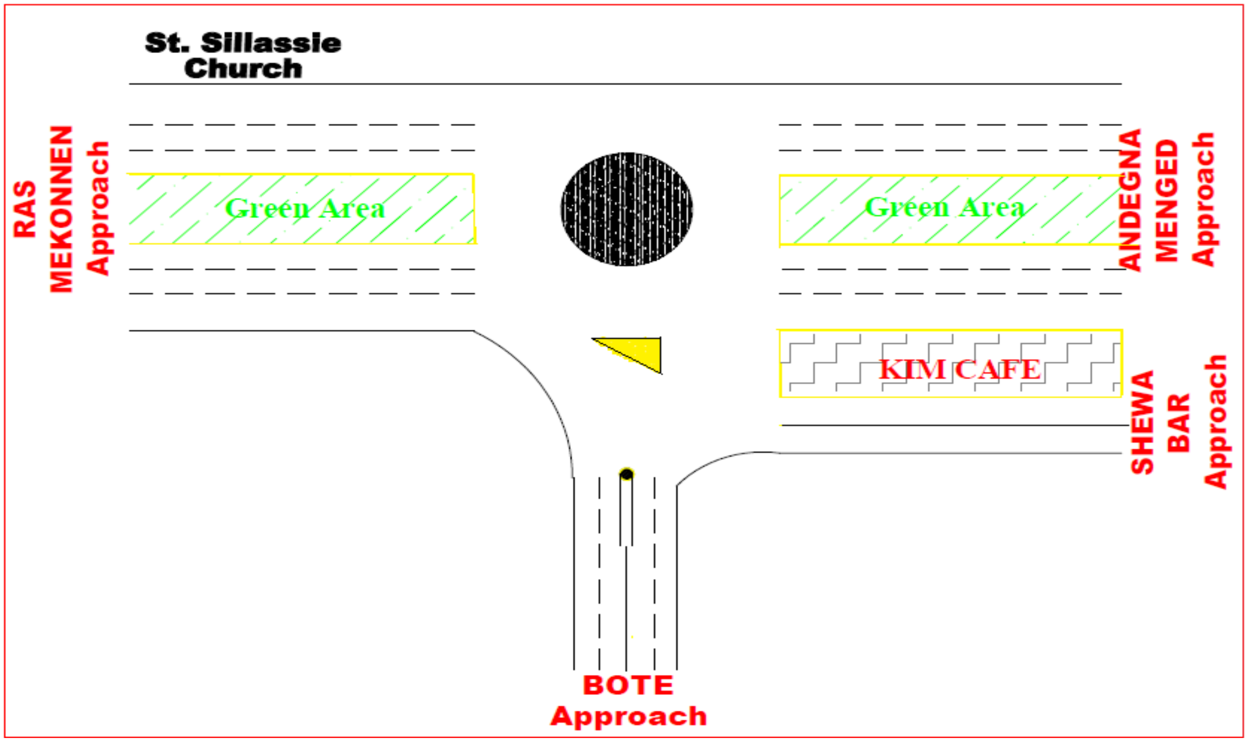

Figure 3.5: Plan view of Sillassie intersection

3.2 Study Design

The research designs that are used by the researcher are quantitative and descriptive type of research. Quantitative data were used to determine all parameters which are related to performance characteristics of unsignalized intersections like capacity, level of service, delay, queue length and degree of saturation. Descriptive type of research is chosen by the researcher to assess the performance evaluation of unsignalized intersections. After the performances of the intersections were analyzed in a consecutive time series, the change of the parameters over the time period and the variables which has a significant effect on the change was analyzed.

3.3 Methods of Data Collection

The major primary data of this research were collected using video cameras in accordance with the clearly defined points and recommendations by traffic data collection handbooks.

3.3.1 Traffic Volume Data

Traffic counts are the most basic of traffic studies and are the primary measure of demand. These data can help identify critical flow time periods, determine the influence of large vehicles or pedestrians on vehicular traffic flow or document traffic volume trends. The length of the sampling period depends on the type of count being taken and the intended use of the data recorded. For example, an intersection count may be conducted during the peak flow period. If so, manual count with 15-minute intervals could be used to obtain the traffic volume data.

According to traffic analysis handbook prepared by American Federal Department of Transportation (FDOT), the volume count for intersection shall consider the following:

- The count should be for less than 12 months.
- The count should be on typical day (Tuesday, Wednesday or Thursday)
- During counting weather and incident must not affect flow

Handbook of simplified practices for traffic studies prepared by Iowa department of transportation and Iowa state university suggested that 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, and afternoon peaks, and 12 hours for daytime periods (Iowa DOT, 2002). This research aims at studying the performances of the intersections at morning and afternoon peak periods and thus a 4 hour traffic count is suggested by Iowa department of transportation. But the researcher of this study took a 6 hour volume count to properly analyze a change of performances from time to time. This is because the analysis is recommendable to be done during morning and afternoon peak periods. Travel time data collection hand book suggests that the volume count for morning peak hour should encompasses all congestions for a minimum of 3 hours. The same is true for afternoon peak hours. Thus, a 6 hour volume count can be used for analysis.

Accordingly, a video recording was done in the morning session from 8:00 AM to 11:00 AM and in the afternoon session from 2:00PM to 5:00PM. The time period is selected to capture the traffic movements during stable traffic flow and also to consider the work or school trips around 8:00AM and 5:00PM. The days of the week selected for data collection are based on FDOT recommendation and they are Tuesday, Wednesday and Thursday for the three intersections considered in this research. Monday and Friday are typically excluded from data collection because a small number of weekdays are sampled and these days' high variation from conditions during the middle of the week would necessitate a much larger sample of weekdays. Monday and Friday are also the start and end of working days in the week and the traffic nature are different from the others.

The vehicles were counted in 5min interval separately for left turn (LT), right turn (RT), through movement (TT), and U- turns (UT) for each approach of the intersections and the three consecutive 5min count will be a 15 minute volumes which are used for the analysis. The vehicle classification used in counting are based on Ethiopian Road Authority pavement design manual classification. It includes: Bajaj, car and Taxi, 4WD, mini bus, medium bus, large bus, medium truck, large truck, and truck and trailers.

Volume count format for three intersections selected in this research is indicated in table 3.1 below. Each count is done separately on different days; Tuesday for Shash garage intersection, Wednesday for Arategna roundabout, and Thursday for Sillassie roundabout. The six hour count can be separated into 24 time series traffic streams in 15 minute interval, which were analyzed separately, to investigate the trends of change of intersection performance parameters. The analysis for peak hour determination is discussed in section 4.1 of this report.

Table 3.1 Sample for traffic volume count format

Before Noon (8:00AM-11:00AM)											
5min Traffic count			Vehicle Categories								
	Approach	Direction of Movement	Bajajs	Cars &Taxis	Mini bus	M/ Bus	L/ Bus	M/ Trucks	L/ Trucks	T/T	Total
1	EB	Left Turn	////								
		Through turn		///							
		Right Turn			/////						
		U-Turn				///					
	WB	LT					////	`			
		TT									
		RT									
		UT									
	NB	LT									
		TT									
		RT									
		UT									
	SB	LT									
		TT									
		RT					////				
		UT				//					
	EB	LT			/////						
		TT		/////							

Before Noon (8:00AM-11:00AM)											
5min Traffic count			Vehicle Categories								
	Approach	Direction of Movement	Bajajs	Cars &Taxis	Minibus	M/ Bus	L/ Bus	M/ Trucks	L/ Trucks	T/T	Total
2		RT	///								
		UT									
	WB	LT									
		TT									
		RT									

3.3.2 Travel Time and Speed Data

The speed of approaching vehicles collected in this research is a space mean speed. The **space-mean speed** is the average speed of vehicles traveling a given segment of roadway during a specified period of time and is calculated using the average travel time and length for the roadway segment. Using the video recorders for the movement of vehicles over a predetermined length of approach roads, the travel time can be known by taking start and end time of the travelling vehicle. The recorder starts the first stopwatch as the driver passes the first checkpoint, recording the elapsed time as the driver passes the final check point. Then dividing the segment length by the time taken to the driver to finish the segment, the speed of that particular vehicle can be found. This procedure is followed for different vehicle categories depending on the sample size and the average speed of the vehicles can be taken as the representative speed for a given approach and a given vehicle group.

3.3.2.1 Sample Size

Sample size requirements for the test vehicle technique dictate the number of "runs" that must be performed for a given roadway during the time period(s) of interest. The use of minimum samples sizes ensures that the average travel time and average speed obtained from the test vehicle is within a specified error range of the true average travel time for the entire vehicle population. The standard equation for sample size is as follow:

Sample size, $n = (\frac{t\ x\ s}{\varepsilon})^2$ where;

- t = t-statistic from Student's t distribution for specified confidence level;
- s = standard deviation of travel time; and
- ε = maximum specified allowable error

According to travel time data collection hand book prepared by American Federal Highway Administration (FHWA), by using the above formula, a minimum number of test vehicle to be selected for 30 min time period for freeways and urban arterial streets are suggested. For this research, the table for urban arterial streets is selected.

Table3.2: Sample size on arterial streets (Source: FHWA, travel time collection hand book)

Traffic Signal Density (signals per mile)	Average Coefficient of Variation, (%) Table 3-2	Sample Sizes (iterative calculations using Equation 3-4)		
		90% Confidence, ± 10 % Error	95% Confidence, ± 10 % Error	95% Confidence, ± 5 % Error
Less than 3	9	5	6	15
3 to 6	12	6	8	25
Greater than 6	15	9	12	37

According to table 3.2, if traffic signal density is less than three per mile, which is the case for this research, for 95% confidence interval and 5% error, the minimum number of vehicles is 15 in 30 minute time period. Since the data in this research are arranged in 15 minute interval, the researcher took travel time of 8 vehicles in 15 minute time period and the average will be taken for a given vehicle class during that time.

Table 3.3: Average speed and travel time at Arategna roundabout

	Fadis Approach		NOC Approach		Warwari Approach		Shash Approach	
	Length=	60M	Length=	150M	Length =	60M	Length =	100M
Vehicle category	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)
Bajaj	3.24	18.5	8.24	18.2	3.85	15.6	5.68	17.6
Cars and Taxi	3.02	19.87	6.00	25	2.45	24.5	4.26	23.5
4WD	2.25	26.7	5.56	27	2.25	26.7	4.98	20.1
Mini Bus	2.40	25	4.29	35	2.40	25	3.13	32
Medium Bus	3.26	18.4	7.77	19.3	-	-	5.62	17.8
Large Bus	-	-	6.25	24	3.00	20	6.41	15.6
M/Truck	3.33	18	6.52	23	3.16	19	4.35	23
L/Truck	3.33	18	6.52	23	2.40	25	4.55	22
T/T	3.75	16	8.33	18	3.53	17	6.67	15

Table 3.4: Average speed and travel time at Shash garage intersection

	Bira Approach		Arategna Approach		Hajib Approach	
	Length =	80M	Length =	120M	Length =	150M
Vehicle category	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)
Bajaj	5.03	15.91	4.29	28	7.35	20.4
Cars and Taxi	3.20	25	3.43	35	5.00	30
4WD	2.50	32	3.29	36.5	4.49	33.4
Mini Bus	2.50	32	3.00	40	5.00	30
Medium Bus	-	-	4.14	29	6.52	23
Large Bus	-	-	4.44	27	7.50	20
M/Truck	5.52	14.5	4.21	28.5	6.82	22
L/Truck	-	-	4.62	26	8.11	18.5

Table 3.5: Average speed and travel time at Sillassie intersection

	Ras Mekonnen Approach		Bote Approach		Shewabar Approach		AndegnaMenged Approach	
	Length=	100M	Length=	50M	Length =	60M	Length =	110M
Vehicle category	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)	Travel time (sec)	Average speed (km/hr)
Bajaj	5.00	20	3.13	16	4.00	15	4.40	25
Cars and Taxi	3.97	25.2	3.85	13	3.00	20	3.61	30.5
4WD	4.17	24	2.00	25	2.86	21	3.61	30.5
Mini Bus	3.33	30	1.75	28.6	2.36	25.4	3.19	34.5
Medium Bus	4.35	23	2.67	18.7	3.00	20	3.81	28.9
Large Bus	-	-	3.13	16	-	-	4.26	25.8
M/Truck	3.33	30	1.79	28	3.75	16	3.67	30
L/Truck	4.00	25	2.50	20	3.16	19	3.67	30

3.3.3 Gap Acceptance Data

Gap acceptance studies are used to determine the critical gap. Critical gaps are then used to evaluate level of service and capacity at priority intersections. This study measures gaps in the major stream and driver behavior on the minor road. Different types of drivers will accept different gaps, more aggressive drivers will accept smaller gaps and conservative drivers accept larger gaps.

To determine gap acceptance times at an intersection, a field study must be performed. In this thesis, a video camera was used to record maneuvers in the intersection, while recording, an observer had to indicate the arrival time and departure time of a major stream vehicle to the intersection reference point. Also it is important to indicate in the video if a vehicle stopped in the minor stream of the intersection has accepted or rejected a gap. If a vehicle is not present, the gap is classified as untested. By using a stopwatch, the times between the consecutive major stream moving vehicles are recorded from the video.

For roundabouts, the gap acceptance is between the approach flow and the circulating flow. The time gap between consecutive circulating flows was taken and the behavior of the driver on the approach flow is examined. When the approach drivers accept the gap, the gap is recorded as "accepted gap" and when the approach drivers rejected the gap, the gap is recorded as a "rejected gap."

The methods used to determine the critical gap and follow-up times from the collected data are discussed in chapter 4 of this report. Table 3.6 below provides the way the gap was identified as accepted gaps and rejected gaps. The sample size for the gaps to be tested was determined using the following equation:

$$n = \left(\frac{Z\alpha/2 * \sigma}{E}\right)^2$$

Where E is the margin of error equal to 0.5, this was used as a starting point to determine a sample size. Sigma (σ) is the theoretical standard deviation with a value of 2.0 and a critical value $z_{\alpha/2}$ for a 95% confidence level of 1.96.

$$n = \left(\frac{1.96 * 2}{0.5}\right)^2 = 61 \text{ samples per approach}$$

Accordingly, the minimum number gaps to be tested are ***61 gaps.*** Actually in this study the numbers of gaps tested are above the minimum mentioned here. Even if there is a variation of gap acceptance for different vehicle categories, in this research it is assumed that the vehicles are tested at random. But the selection is distributed based on the percentage compositions of the vehicles; i.e. for high percentage vehicles, more numbers of vehicles are tested and vice versa.

The average of gap acceptance for different vehicle categories is used in the analysis of this study.

The sample data for data of gap acceptance for Arategna intersection is presented in table 3.8. Similar table was produced for other intersections.

Table 3.6: Gap acceptance data collection for Arategna intersection

	Fadis Approach			**NOC Approach**			**Warwari Approach**			**Shash Approach**		
Vehicle	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection
1	6	*		4.3		*	2.6		*	7.4	*	
2	1.8		*	7.6	*		7	*		2.3		*
3	3.3		*	10.5	*		9.2		*	4.6	*	
4	2		*	18.8	*		6		*	3.7		*
5	2.4		*	3.1		*	6.4	*		8.1	*	
6	3.1		*	5.7	*		2.6	*		19.7	*	
7	4.2	*		6.6	*		2.7		*	7	*	
8	3.2		*	3.8	*		6.5	*		5.2	*	
9	5.7	*		19.4	*		3.8	*		3.7		*
10	3		*	13.7	*		1.9		*	6.3	*	
11	3.9		*	5.7		*	1.9		*	11.3	*	
12	8	*		19.5	*		4.1		*	11.8	*	
13	6.7		*	5		*	2.2		*	1.9		*
14	2.3		*	5.4	*		1.7		*	3.2		*
15	7.1	*		1.6		*	1.9		*	3.6		*
16	2.5		*	16.1	*		3.5		*	2.3		*
17	2		*	15.5	*		2.2		*	4.8		*
18	3.3		*	2.2		*	3.4		*	11.9	*	
19	2.2		*	4.9	*		1.8		*	2.2		*
20	2.1		*	4		*	2.1		*	3.7		*
21	2.7		*	2.5		*	3.1	*		2.5		*
22	1.8		*	3.3	*		2.8		*	6.5	*	
23	2.1	*		26.6	*		3.8	*		11.8	*	
24	2.1		*	4.8	*		4		*	4.8	*	

	Fadis Approach			**NOC Approach**			**Warwari Approach**			**Shash Approach**		
Vehicle	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection
25	5.7		*	6.1	*		3.1		*	5.7		*
26	2.8		*	5.7	*		2.9		*	1.3		*
27	2.3		*	3		*	2.8		*	2.4		*
28	3.8		*	13.6	*		1.5		*	13.7	*	
29	2		*	2.5		*	2.6		*	4.2	*	
30	2.7		*	5.5		*	3.5		*	8.7	*	
31	3.6	*		4.6		*	6.6	*		5.3		*
32	2.3		*	2.6		*	5.1	*		6.1	*	
33	2.4	*		5.9	*		4.9	*		3.6		*
34	2.1		*	31.1	*		4.6	*		12.4	*	
35	1		*	5.7		*	1.7		*	2.3		*
36	3.4		*	30.2	*		2.2		*	4.9	*	
37	1.5	*		7.9	*		3.3		*	6.8	*	
38	3		*	5.4		*	5.8	*		3.5		*
39	2.8		*	9.7	*		5.7		*	13.4	*	
40	2.8		*	3.7		*	5.5		*	3.6	*	
41	2.3		*	17.9	*		7.3	*		4.7		*
42	2.5	*		4.8		*	7.2		*	7.6	*	
43	3.5		*	4.1	*		2.1		*	7	*	
44	5.3	*		8.4	*		5.2	*		5.2	*	
45	5.4	*		7.2	*		4.4		*	3.7		*
46	6	*		13.9	*		3.2		*	6.3	*	
47	2.6		*	17.5	*		1.4		*	11.3	*	
48	4.9	*		3.1	*		5.6	*		11.8	*	
49	4.4	*		4.2		*	9.4	*		1.9		*
50	4.3		*	2.7		*	5.8		*	3.2		*
51	6.2	*		3.5	*		2.9	*		3.6		*
52	2.2		*	6	*		3.3		*	7	*	
53	3.3	*		6.8	*		6.1		*	5.2	*	
54	4.3	*		6.9	*		2.3		*	3.7		*

	Fadis Approach			**NOC Approach**			**Warwari Approach**			**Shash Approach**		
Vehicle	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection	Gap (sec)	Acceptance	Rejection
55	3.1		*	6	*		5.2		*	6.3	*	
56	1.7		*	4.1		*	2.3		*	11.3	*	
57	2.6		*	12.8	*		1.7		*	11.8	*	
58	5.5	*		12.7	*		2.1		*	1.9		*
59	4.2		*	4.4		*	4.8	*		3.2		*
61	3.2		*	35.3	*		5.7	*		3.6		*
62	3.5		*	11.5	*		2.9		*	4.7	*	

3.3.4 Intersection Geometry Data

For performance evaluation of intersections, in addition to traffic data, geometric data are also necessary and these data are collected. During field measurement, alignment of the legs, width of traffic lanes, crosswalks, number of lanes, median and the method of treating and channelization of turning movements has been measured on each approach for the selected intersection. The summary of the geometric data collected for the three intersections are described in the following tables and figures.

Table 3.7: Geometric data for Arategna intersection

Arategna Intersection							
Type:-	**Roundabout**						
Approach	No. of Lane *(one way)*	Lane Width	Median Type	Median Width	Deflection Angle	Cross Walk	RT Channelization
Fadis	2	3.5m	Divided	2m	90°	No	No
NOC	3	3.5m	Divided	3m	90°	Yes	No
Warwari	1	4m	Undivided	0	90°	No	No
Shash	2	3.2m	Undivided	0	90°	No	No
# of Circulating Lane			3				
Circulating Width			13m				
Island Diameter			8m				

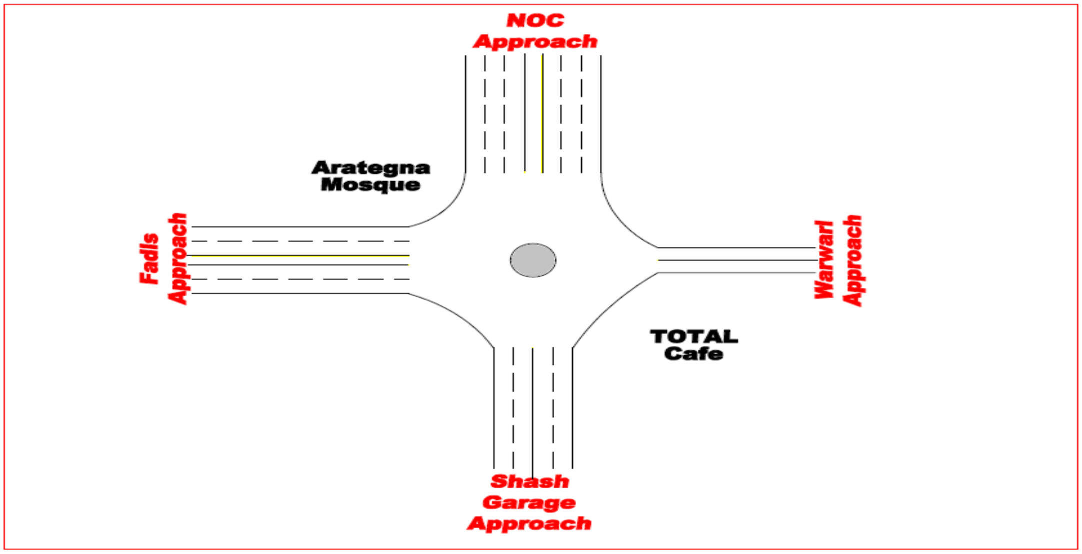

Figure 3.6: Layout of Arategna intersection

Table 3.8: Geometric data for Shash garage intersection

Shash Garage Intersection							
Type:-	**Unsignalized T- Intersection**						
Approach	No. of Lane *(one way)*	Lane Width	Median Type	Median Width	Deflection Angle	Cross Walk	RT Channeliz ation
Bira	1	4m	Undivided	0	85°	No	No
Arategna	2	3.5m	Undivided	0	90°	No	No
Hajib	2	3.5m	Undivided	0	90°	No	No

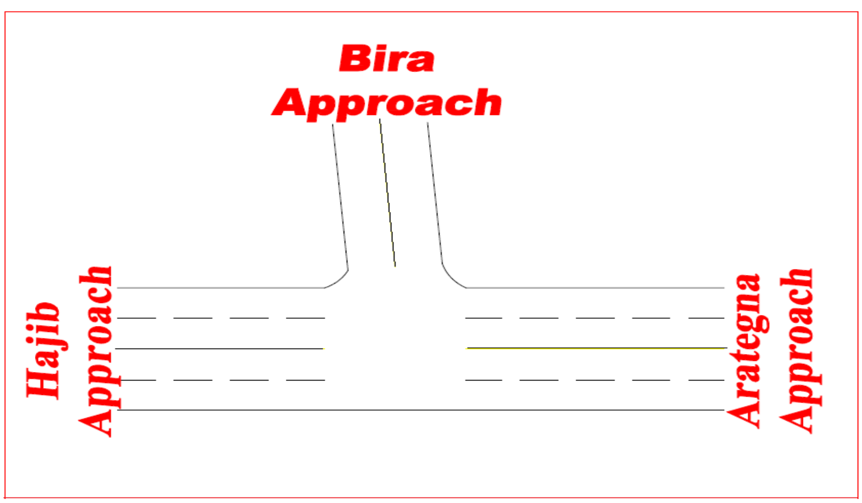

Figure 3.7: Layout of Shash garage intersection

Table 3.9: Geometric data for Sillassie intersection

Sillassie Intersection							
Type:-	**Roundabout (With double Island/channelization at Bote approach)**						
Approach	No. of Lane (one way)	Lane Width	Median Type	Median Width	Deflection Angle	Cross Walk	RT Channelization
Ras Mekonnen	3	3.5m	Divided	11m	90°	Yes	No
Bote	2	3m	Undivided	0	90°	Yes	Yes
Shewa Bar	1	4.5m	Undivided	0	90°	No	No
Andegna Menged	3	3.5m	Divided	11m	90°	Yes	No
# of Circulating Lane			2				
Circulating Width			10m				
Island diameter			18m				

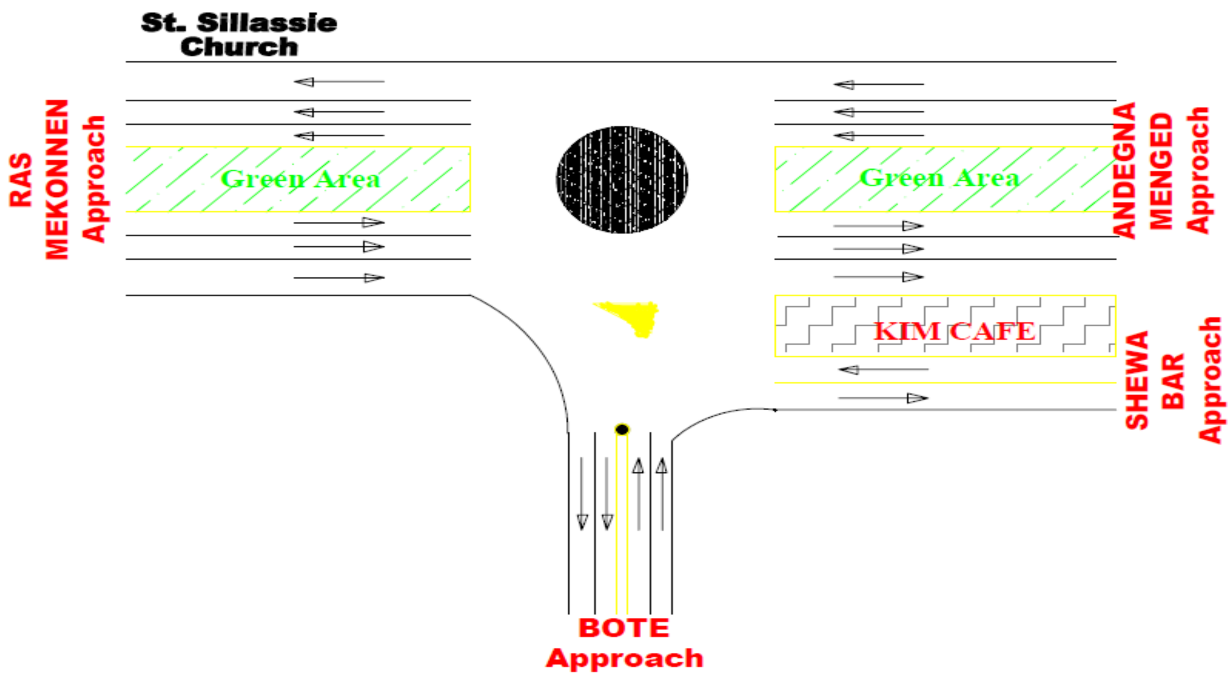

Figure 3.8: Layout of Sillassie intersection

3.4 Research Materials

All data collected for this research are by the help of different equipments for different data types. The analyses are made by use of different software. The frequently used equipments and software are discussed as below.

3.4.1 Equipments

The main equipment used in collecting necessary data for this research is videotaping recorder available on smart phones like Samsung and Huawei. Traffic volumes can be counted by viewing videotapes recorded with a camera at a collection site. A digital clock in the video image can prove useful in noting time intervals. Then volumes and travel time over a fixed distance can be extracted on the notepad on a required time interval.

Other equipment which is frequently used is meter tapes for measuring geometric characteristics of intersections.

3.4.2 Software

In this study the use of Microsoft Excel is applied greatly for analysis purpose. All the data were feed on excel and the analysis tables and graphs were the output of excel too. Other Microsoft office applications were used over the entire period of this research for writing purpose (e.g. Word), and drawing purposes (e.g. paint). AutoCAD was also used to draw the geometry of the intersections. Stata 11 software is used to analyze the statistical relationship between variables.

3.5 Analysis Method

The performances of the selected intersections in this study are analyzed by using gap acceptance procedures presented in highway capacity manuals (HCM). The steps used by HCM are thoroughly discussed in chapter two of this report. The gap acceptance models are calibrated for the intersections in this study by using the actual data on the site. Passenger car equivalents, critical gaps and follow up times representing the prevailing conditions of the intersections are estimated before analyzing the performances of the intersections.

The statistical software known as "stata" is used to develop a relationship between compositions of vehicle categories and performance measurement parameters over the selected time period. Based on the models developed, the effects of vehicle categories on the performance of the unsignalized intersections are analyzed.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Peak Hour Volumes

To locate the peak hour, we add four 15 minute interval volumes consecutively. From the cumulative, the one with the highest volume become the peak hour and the peak hour volume is just the sum of th volumes of the four 15 minute intervals within the peak hour.

According to table 4.1 below, the peak hour for Arategna, Shash garage, and Sillassie intersections become 8:00am to 9:00am, 2:00pm to 3:00pm, and 9:30am to 10:30am respectively. The consecutive tables, table 4.2 to 4.4, provides the peak hour volumes for different vehicle groups.

Table 4.1: Traffic Volume in 15min interval and peak hour determination

		Arategna Intersection		***Shash Garage Intersection***		***Sillassie Intersection***	
	Time	Volume	1hr Cum.*	Volume	1hr Cum.*	Volume	1hr Cum*.
AM	8:00 - 8:15	**719**	**2306**	531	1775	833	3245
	8:15 - 8:30	**624**	2104	482	1670	801	3158
	8:30 - 8:45	**512**	2003	389	1622	810	3166
	8:45 - 9:00	**451**	1944	373	1664	801	3185
	9:00 - 9:15	517	1913	426	1520	746	3242
	9:15 - 9:30	523	1911	434	1542	809	3345
	9:30- 9:45	453	1859	431	1512	**829**	**3347**
	9:45 - 10:00	420	1908	229	1536	**858**	3264
	10:00 - 10:15	515	2115	448	1752	**849**	3249
	10:15 - 10:30	471	2036	404	1785	**811**	3019
	10:30 - 10:45	502	1999	455	1831	746	2790
	10:45 - 11:00	627	1925	445	1850	843	2708
PM	2:00 - 2:15	436	1732	**481**	**1896**	619	2619
	2:15 - 2:30	434	1827	**450**	1883	582	2656
	2:30 - 2:45	428	1895	**474**	1841	664	2768
	2:45 - 3:00	434	1956	**491**	1791	754	2810
	3:00 - 3:15	531	2022	468	1769	656	2850

		Arategna Intersection		*Shash Garage Intersection*		*Sillassie Intersection*	
	Time	Volume	1hr Cum.*	Volume	1hr Cum.*	Volume	1hr Cum*.
	3:15 - 3:30	502	1973	408	1691	694	3052
	3:30- 3:45	489	1980	424	1760	706	3069
	3:45 - 4:00	500	2046	469	1856	794	2999
	4:00 - 4:15	482	2304	390	1880	858	2925
	4:15 - 4:30	509		477		711	
	4:30 - 4:45	555		520		636	
	4:45 - 5:00	758		493		720	

- **Note:** * The "***1hr cum***." Column in table 4.1 is the summation of the traffic volumes of four consecutive 15min volume.

Table 4.2: Peak Hour traffic volume for Arategna roundabout approaches

			Vehicle category									
Peak Hour	**Approach**	**Direction of Movement**	**Bajajs**	**Cars & Taxi**	**4WD**	**Mini Bus**	**Medium Bus**	**Large Bus**	**M/Truck**	**L/Truck**	**T/T**	**Total (Veh/hr)**
8:00 AM To 9:00 AM	Fadis	RT	46	32	13	23	9	0	0	0	0	123
		TT	19	7	11	5	8	0	6	5	3	64
		LT	35	8	5	11	11	0	8	7	0	85
		UT	0	0	0	0	0	0	0	0	0	0
			100	47	29	39	28	0	14	12	3	**272**
	NOC	RT	49	9	0	11	3	0	5	6	0	83
		TT	184	65	136	184	7	5	3	3	0	587
		LT	35	7	22	6	0	7	12	24	7	120
		UT	192	0	0	3	0	0	0	0	0	195
			460	81	158	204	10	12	20	33	7	**985**
	Warwari	RT	65	14	40	14	0	1	5	8	3	150
		TT	17	3	0	0	0	0	0	0	0	20
		LT	21	3	5	5	0	3	4	0	0	41
		UT	3	0	0	0	0	0	0	0	0	3
			106	20	45	19	0	4	9	8	3	**214**

			Vehicle category									
Peak Hour	**Approach**	**Direction of Movement**	**Bajajs**	**Cars & Taxi**	**4WD**	**Mini Bus**	**Medium Bus**	**Large Bus**	**M/Truck**	**L/Truck**	**T/T**	**Total (Veh/hr)**
	Shash	RT	16	8	7	0	6	0	1	0	0	38
		TT	177	40	99	138	10	5	6	17	2	494
		LT	35	35	17	26	9	1	0	3	0	126
		UT	49	81	10	36	0	0	1	0	0	177
			277	164	133	200	25	6	8	20	2	**835**

Table 4.3: Peak Hour traffic volume for Shash Garage Intersection approaches

Peak Hour	**Approach**	**Direction of Movement**	**Bajajs**	**Cars & Taxi**	**4WD**	**Mini Bus**	**Medium Bus**	**Large Bus**	**M/Truck**	**L/Truck**	**T/T**	**Total**
2:00 PM To 3:00 PM	Bira	LT	84	19	14	10	0	0	5	0	0	132
		RT	101	6	17	6	0	0	0	0	0	130
		UT	44	0	0	0	0	0	0	0	0	44
			229	**25**	**31**	**16**	**0**	**0**	**5**	**0**	**0**	**306**
	Arategna	RT	72	13	21	11	2	0	5	1	0	125
		TT	322	138	85	133	1	1	7	0	0	687
			394	**151**	**106**	**144**	**3**	**1**	**12**	**1**	**0**	**812**
	Hajib	LT	69	9	15	4	2	1	2	0	0	102
		TT	276	129	72	155	8	1	27	8	0	676
			345	**138**	**87**	**159**	**10**	**2**	**29**	**8**	**0**	**778**

Table 4.4: Peak Hour traffic volume for Sillassie Intersection approaches

Peak Hour	**Approach**	**Direction of Movement**	**Bajajs**	**Cars & Taxi**	**4WD**	**Mini Bus**	**Medium Bus**	**Large Bus**	**M/Truck**	**L/Truck**	**T/T**	**Total (veh/hr)**
	Ras Mekonnen	RT	69	9	13	6	0	0	3	0	0	100
		TT1	288	72	17	147	2	0	2	0	0	528
		TT2	295	99	81	49	6	0	10	2	0	542

Peak Hour	Approach	Direction of Movement	Bajajs	Cars & Taxi	4WD	Mini Bus	Medium Bus	Large Bus	M/Truck	L/Truck	T/T	Total (veh/hr)
9:30 AM To 10:30 AM		UT	67	5	6	6	0	0	0	0	0	84
			719	**185**	**117**	**208**	**8**	**0**	**15**	**2**	**0**	**1254**
	Bote	RT1	205	7	13	8	0	1	3	1	0	238
		RT2	85	4	23	3	1	0	4	2	0	122
		LT	89	10	20	9	1	0	1	1	0	131
		UT	61	1	1	0	0	0	0	0	0	63
			440	**22**	**57**	**20**	**2**	**1**	**8**	**4**	**0**	**554**
	Shewa Bar	TT	314	61	24	130	4	0	4	2	0	539
		LT	117	4	8	6	1	0	0	1	0	137
		UT	17	2	1	2	0	0	0	0	0	22
			448	**67**	**33**	**138**	**5**	**0**	**4**	**3**	**0**	**698**
	Andegna Menged	TT	396	99	93	38	8	2	10	2	0	648
		LT	70	8	18	9	0	0	7	1	0	113
		UT1	36	4	7	2	0	0	0	0	0	49
		UT2	17	4	6	4	0	0	0	0	0	31
			519	**115**	**124**	**53**	**8**	**2**	**17**	**3**	**0**	**841**

4.2 Passenger Car Units

In capacity analysis different types of vehicles offer different degree of interference to other traffic and it is necessary to bring all types to a common unit called as passenger Car Unit (PCU). The currently adopted PCU values in many researches do not represent the actual situation in the field and, hence, ways of estimating the value need to be developed for actual conditions.

Muhammad (2014) studied on passenger car equivalent in heterogeneous traffic environment. Four different methods were used to estimate the PCU of vehicles. The study found that the values obtained from speed method for large dimension vehicles are higher compared to those obtained from the headway methods. This is attributed to the dominancy of projected area parameter as speed difference between reference vehicles and larger dimension vehicles is not such significant. Further, the study found that method that incorporate vehicles speed along with projected area of vehicles were provide appropriate estimate of PCE values for poor lane disciplined flows.

Thus this study has adopted Chandra's method to estimate the PCU values of vehicles. *In a mixed traffic situation many categories of vehicles share the same roadway space. Vehicles do not move in lanes due to the poor lane discipline of many road users.* Chandra recommends that the conditions where the vehicles do not follow lane strictly is better reflected by the speed of the vehicle and the occupancy of the vehicle (Length and width of a vehicle).

$$\text{PCU}_\text{i} = \frac{\text{V}c/\text{V}i}{\text{A}c/\text{A}i} \qquad \text{Where;}$$

- V_C = Mean speeds for cars (c) in the traffic stream [km/h]
- V_i = Mean speeds for vehicles type i in the traffic stream [km/h]
- A_C = respective projected rectangular areas of cars [m2] on the road
- A_i = Respective projected rectangular area of vehicle type i [m2]

The numerator in the above equation is the function of volume of traffic stream as the speed of any vehicle type depends upon its category, own volume and volume of other vehicles. Therefore, speed of any vehicle type is true representation of overall interaction of a vehicle type due to presence of other vehicle of its own category and of other types. The denominator represents the carriageway occupancy with respect to standard car. In this study the average speed of the vehicles on the approach are considered. For a single intersection, since the performance of the intersection is the cumulative of the performance of the approaches, the PCU values of the approaches are considered similar. Table 4.5 indicates the calculated PCU for the selected intersections based on the Chandra's method. Many researches indicate that there is a change of PCU value over a time period due to change of vehicle volumes and compositions. But in this study PCU values are assumed to be constant over the change of time since the change of vehicle speeds is not very much significant for the intersections under study.

Table 4.5: PCU values for the selected intersections

				Arategna Intersection		**Sash Garage Intersection**		**Sillassie Intersection**	
Vehicle Type	Length	Width	Area	Average Speed	**PCU**	Averag e Speed	**PCU**	Average Speed	**PCU**
Bajaj	2.6	1.2	3.12	17.48	0.76	21.44	0.81	21.40	0.67
Cars and Taxi	3.72	1.45	5.39	23.00	1.00	30.00	1.00	24.80	1.00
4WD	4.58	1.77	8.11	26.30	1.31	35.00	1.29	24.80	1.50
Mini Bus	5.8	1.9	11.02	30.00	1.57	38.00	1.61	32.70	1.55
Medium Bus	7.1	2.1	14.91	21.50	2.96	28.20	2.90	25.00	2.70

				Arategna Intersection		**Sash Garage Intersection**		**Sillassie Intersection**	
Vehicle Type	Length	Width	Area	Average Speed	**PCU**	Averag e Speed	**PCU**	Average Speed	**PCU**
Large Bus	12.2	2.43	29.65	20.20	6.26	25.00	6.60	23.40	5.82
M/Truck	7.5	2.35	17.63	24.30	3.09	31.00	3.16	30.00	2.70
L/Truck	9.15	2.5	22.88	24.00	4.06	26.80	4.75	28.00	3.76
T/T	17.5	2.5	43.75	17.80	_	_	_	_	_

4.3 Flow Rate

The flow rate required for analysis of roundabouts and T- intersection are very different in characteristics.

For roundabouts the data of interest for each approach are the **entry flow** and the **circulating flow**. Entry flow is simply the sum of the through, left, right and U turn movements on an approach. Circulating flow is the sum of the vehicles from different movements passing in front of the adjacent upstream splitter island.

In chapter two of this research, figure 2.1 indicates the rank of the movements at T- intersections. The priority of right-of-way given to each traffic stream must be identified for each approach. Some streams have absolute priority, whereas others have to give way or yield to higher-order streams. The basic movements that should be analyzed by performance parameters are right and left turn movements from minor approach and left turn movements from major approach. These conflicts are shown in Exhibit 17-4 of HCM 2000, which illustrates the computation of the parameter $v_{c,x}$, the conflicting flow rate for movement x, that is, the total flow rate that conflicts with movement x (veh/h).

4.3.1 Entry and Circulating Flow for Arategna Roundabouts

For Arategna roundabout the entry flow and circulating flow for each approach are calculated as follows:

- Entry Flow,

$$V_E = V_{LT} + V_{RT} + V_{TT} + V_{UT}$$

- Circulating Flows;
  - Fadis approach:

$$V_{fad,circ} = V_{noc,TT} + V_{noc,LT} + V_{war,LT} + V_{noc,UT} + V_{war,UT} + V_{sha,UT}$$

  - NOC approach:

$$V_{noc,circ} = V_{war,LT} + V_{war,TT} + V_{sha,LT} + V_{fad,UT} + V_{sha,UT} + V_{war,UT}$$

- Warwari approach:

$$V_{war,circ} = V_{sha,TT} + V_{sha,LT} + V_{fad,\,LT} + V_{fad,UT} + V_{noc,UT} + V_{sha,U}$$

- Shash garage approach:

$$V_{sha,circ} = V_{fad,TT} + V_{fad,LT} + V_{noc,\,LT} + V_{war,UT} + V_{noc,UT} + V_{fad,UT}$$

In the tables below the designation for entry flow and circulating flow are Q_a and Q_c respectively. Table 4.6 provides a series of the entry and circulating flows in 15 minute time interval series for Arategna intersection approaches.

Table 4.6: Time series of entry and circulating flows for Arategna intersection approaches

Approach	**Fadis**		**NOC**		**Warwari**		**Shash Garage**	
15 min. Time series	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)
1	543	1952	1809	440	468	1649	1282	1076
2	499	1561	1318	668	329	1587	1364	764
3	340	1313	1204	320	291	1099	871	649
4	332	1083	996	344	136	1049	887	543
5	381	1130	1121	300	136	1216	1009	535
6	369	1442	1317	419	288	1188	1022	766
7	312	1124	1035	350	280	1005	809	528
8	287	1039	925	277	250	942	751	514
9	412	1261	1175	403	286	1214	996	669
10	367	1231	1115	348	255	1070	871	675
11	398	1274	1168	343	253	1135	916	678
12	482	1651	1490	557	400	1408	1197	870
13	379	820	826	362	364	1013	807	517
14	337	1039	925	277	250	959	751	547
15	276	806	710	365	215	937	835	374
16	320	747	782	321	280	913	776	445
17	390	782	753	394	291	1256	1047	439
18	280	827	859	393	314	1043	850	404
19	387	870	810	386	349	1096	905	513
20	359	831	845	486	356	1169	978	485
21	363	805	848	499	461	1014	893	452

Approach	**Fadis**		**NOC**		**Warwari**		**Shash Garage**	
15 min. Time series	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)
22	405	896	902	454	446	1134	923	529
23	506	1047	1046	608	538	1291	1086	638
24	902	1300	1308	645	555	1879	1425	979

4.3.2 Entry and Circulating Flow for Sillassie Roundabouts

For Sillassie roundabout the entry flow and circulating flow for each approach are calculated as follows:

- Entry Flow,

$$V_E = V_{LT} + V_{RT} + V_{TT} + V_{UT}$$

- Circulating Flow,
  1. Ras Mekonnen approach:

$$V_{ras,circ} = V_{and,LT} + V_{and,UT1} + V_{and,UT2}$$

  2. Bote approach:

$$V_{bot,circ} = V_{ras,TT1} + V_{ras,TT2} + V_{and,UT1} + V_{and,UT2} + V_{ras,UT}$$

  3. Shewa barr approach :

$$V_{shwb,circ} = V_{bot,LT} + V_{bot,RT2} + V_{ras,UT} + V_{and,UT2} + V_{ras,TT2}$$

  4. Andegna menged approach:

$$V_{and,circ} = V_{shwb,TT} + V_{bot,LT} + V_{ras,UT}$$

Table 4.7 provides a series of the entry and circulating flows in 15 minute time interval series for Sillassie intersection approaches.

Table 4.7: Time series of entry and circulating flows for Sillassie intersection approaches

Approach	**Ras Mekonnen**		**Bote**		**Shewa Barr**		**Andegna Menged**	
15 min Time series	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)
1	1223	206	486	1205	629	878	836	681
2	1193	192	484	1164	628	880	805	698
3	1167	132	485	1104	595	821	972	633
4	1204	180	523	1178	607	851	912	669

Approach	**Ras Mekonnen**		**Bote**		**Shewa Barr**		**Andegna Menged**	
15 min Time series	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)	Qa (vph)	Qc (vph)
5	1094	116	452	1034	511	771	893	562
6	1262	216	491	1249	676	874	729	794
7	1209	189	452	1182	673	875	790	757
8	1242	204	516	1254	655	889	779	736
9	1300	186	434	1274	696	926	846	729
10	1186	184	523	1148	641	860	870	695
11	1132	179	484	1121	563	806	845	597
12	1261	189	508	1224	665	973	831	764
13	894	114	321	887	425	572	642	462
14	804	113	364	796	467	569	587	545
15	928	145	358	931	473	618	690	566
16	1059	198	431	1077	641	758	777	690
17	978	83	321	938	411	620	660	492
18	897	225	334	955	583	622	750	678
19	979	111	332	949	586	668	728	694
20	970	64	375	913	847	644	620	759
21	1304	217	496	1287	667	913	810	723
22	982	177	391	997	594	697	769	698
23	911	170	349	912	522	613	684	608
24	991	145	429	988	589	732	736	683

4.3.3 Conflicting Flow for Shash garage T- intersection

For T – intersection, the data of interest are the **conflicting flows** for:

- Bira approach left turn (movement 7),
- Bira approach right turn (movement 9) and
- Hajib approach left turn (movement 4).

The **conflicting flows, V_c** for the above listed movements are calculated as follows:

- Hajib LT movement: $V_{c,4} = V_{arat,TT} + V_{arat,RT}$
- Bira RT movement: $V_{c,9} = V_{arat,TT}/2 + 0.5*V_{arat,RT}$
- Bira LT movement: $V_{c,7} = V_{arat,TT} + 0.5*V_{arat,RT} + 2*V_{haj,LT} + V_{haj,TT}/2$

Where, "*arat*", and "*haj*" stands for Arategna and Hajib approach respectively.

Table 4.8 provides a 15 minute interval time series of directional movements and conflicting movements for Shash garage intersection.

Table 4.8: approach flow and conflicting flow for Shash garage intersection

	Hajib (LT)		**Bira (LT)**		**Bira (RT)**	
15 min. Time series	V4, (vph)	Vc,4 (vph)	V7 (vph)	Vc,7 (vph)	V9, (vph)	Vc,9 (vph)
1	41	1132	62	566	151	1678
2	104	1062	177	531	221	1548
3	41	851	87	426	68	1271
4	86	791	86	396	81	1243
5	44	929	90	465	72	1391
6	22	933	47	466	110	1363
7	33	850	99	425	87	1348
8	35	481	51	241	49	768
9	35	916	65	458	58	1441
10	35	979	71	489	39	1370
11	49	929	67	465	96	1488
12	47	946	90	473	82	1465
13	138	843	103	422	129	1548
14	123	855	120	427	102	1437
15	112	848	115	424	124	1436
16	65	1040	139	520	193	1452
17	115	794	82	397	118	1498
18	44	826	53	413	109	1241
19	31	740	79	370	103	1188
20	138	823	110	411	122	1511
21	31	675	73	338	99	1096
22	129	953	110	477	146	1592
23	43	903	101	452	144	1457
24	146	887	179	443	201	1513

4.4 Critical Gap and Follow up Times

The method used to determine critical gap is similar to the one developed by Raff. To develop a gap acceptance graph, all accepted gaps chosen are sorted in ascending order and grouped in different classes. The number of observations in each group is counted and written down on a table as number of observed acceptances. Finally all data is accumulated to create the graph for total accepted gaps. The same process is done for rejected gaps, but the number of rejected observations is not accumulated; they are subtracted from the total number of observations until zero (0) is reached. When accumulated gaps are plotted, the critical gap is the point where both graphs intersect each other.

In this research, the critical gap is considered to be distributed normally over the whole period of data analysis. Since bajaj vehicles are trying to use a possible available gap without any lane disciplines, at a roundabout approaches, the gap acceptance for right turn, through turn, left turn and U turn are considered to be the same by the researcher.

Follow-up time is measured only under continuous queue conditions. Most intersection approaches in this research were not under continuous queue conditions. Measurements conducted in Germany (Gattis and Low, 1998) to estimate follow up time assumed that there is a fixed dependency of t_f and t_g and developed an equation as $\mathbf{t_f = 0.6 * t_c}$. The application of this equation is preferred in this research for follow up times.

Table 4.9 indicates gap acceptance calculation procedures for Fadis approach of Arategna intersection. On figure 4.1 the location of critical gap is shown. Gap acceptance tables and graphs of other approaches are attached in the appendix 1 at the end of this report. The summary of critical gaps and follow up times for all approaches are shown in table 4.10.

Table 4.9: Gap Acceptance/Rejection pattern for Fadis approach

Gap range (sec)	Median	#of observation	#of acceptance	#of rejection	Cumulative Acceptance	Cumulative Rejection
0-0.9	0.5	0	0	0	0	50
1-1.9	1.5	4	1	3	1	47
2-2.9	2.5	29	3	26	4	21
3-3.9	3.5	19	5	14	9	7
4-4.9	4.5	10	5	5	14	2
5-5.9	5.5	6	5	1	19	1
6-6.9	6.5	5	4	1	23	0
7-7.9	7.5	2	2	0	25	0
8-8.9	8.5	1	1	0	26	0

Gap range (sec)	Median	#of observation	#of acceptance	#of rejection	Cumulative Acceptance	Cumulative Rejection
9-9.9	9.5	0	0	0	26	0
10-10.9	10.5	0	0	0	26	0
11-11.9	11.5	0	0	0	26	0
>12	12	0	0	0	26	0

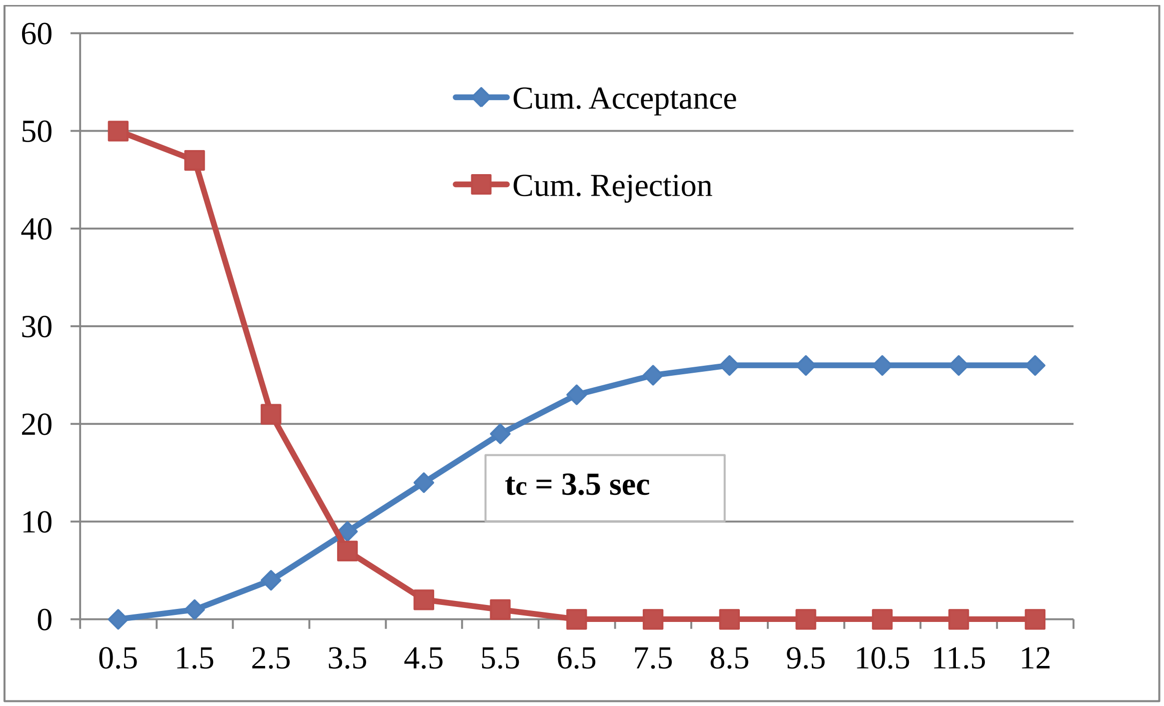

Figure 4.1: Critical gap for Fadis approach

Table 4.10: Critical gaps and follow up times for all intersection approaches

	Arategna Roundabout				**Sillassie Roundabout**				**Shash Garage Intersection**		
Approach	Fadis	NOC	warwari	Shash garage	Ras Mekonnen	Bote	Shewa Barr	Andegna Menged	Bira LT	Bira RT	Hajib LT
t_c (sec)	3.5	4.5	4	4.2	4.1	4	4.2	3.1	5.5	3.2	5.7
t_f (sec)	2.1	2.7	2.4	2.52	2.46	2.4	2.52	1.86	3.3	1.92	3.42

4.5 Performance Analysis

Five performance measures are typically used to estimate the performance of a given intersection design: entry *capacity, degree of saturation, delay, queue length, and level of service*. Each measure provides a unique perspective on the quality of service at which an intersection will perform under a given set of traffic and geometric conditions.

The analysis method used in this research was analytical (Gap acceptance) method. HCM 2000 and 2010 were used as guide to analyze the performance measurement parameters for every approach of the selected intersections. The measures were done over a series of 15minute interval consecutively and their change over a time series was analyzed in the form of empirical regression form, which will be described later in this report. A summary of formulas used in this analysis is shown in table 4.11 below. All the formulas are available on HCM 2000.

Table 4.11: Summary of performance measure evaluation formulas from HCM

	Roundabouts (Arategna & Sillassie intersection)	**Unsignalized T- intersection (Shash garage)**
Entry Capacity or (potential capacity for T-intersection	$q_{e,max} = A \cdot \exp(-B \cdot q_c)$ Where: $A = 3600 / t_f$ $B = (t_c - t_f / 2) / 3600$ t_c = critical headway (s) t_f = follow − up headway (s)	$c_{p,x} = v_{c,x} \dfrac{e^{-v_{c,x} t_{c,x} / 3600}}{1 - e^{-v_{c,x} t_{f,x} / 3600}}$
Movement Capacity	----------------------------	For rank 2- (Movement 4 and 9), $C_{m4} = C_{p,4}$ $C_{m9} = C_{p,9}$ For Rank 3 - (Movement 7), $C_{m,7} = (1 - (V_4/C_{m,4}))*C_{p,7}$
Degree of Saturation	V/C, or $x_i = \dfrac{v_i}{c_i}$	V/C, or $x_i = \dfrac{v_i}{c_i}$
Queue Length	$Q_{95} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \dfrac{\left(\frac{3,600}{c}\right)x}{150T}}\right]\left(\dfrac{c}{3,600}\right)$ T is **0.25**. or analysis period (15min)	$Q_{95} \approx 900T\left[\dfrac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\dfrac{v_x}{c_{m,x}} - 1\right)^2 + \dfrac{\left(\frac{3,600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\right]\left(\dfrac{c_{m,x}}{3,600}\right)$ (veh)

<table>
<tr><th></th><th colspan="3">Roundabouts (Arategna & Sillassie intersection)</th><th colspan="3">Unsignalized T- intersection (Shash garage)</th></tr>
<tr><td>Control Delay</td><td colspan="3">$d=\frac{3{,}600}{c}+900T\left[x-1+\sqrt{(x-1)^2+\frac{\left(\frac{3{,}600}{c}\right)x}{450T}}\right]+5\times\min[x,1]$ (s/veh)</td><td colspan="3">$d=\frac{3600}{c_{m,x}}+900T\left[\frac{v_x}{c_{m,x}}-1+\sqrt{\left(\frac{v_x}{c_{m,x}}-1\right)^2+\frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right]+5$</td></tr>
<tr><td rowspan="8">Level of Service (LOS)</td><td rowspan="2">Control Delay (s/veh)</td><td colspan="2">LOS by volume to capacity ratio</td><td rowspan="2">Control Delay (s/veh)</td><td colspan="2">LOS by volume to capacity ratio</td></tr>
<tr><td>v/c ≤ 1.0</td><td>v/c > 1.0</td><td>v/c ≤ 1.0</td><td>v/c > 1.0</td></tr>
<tr><td>0-10</td><td>A</td><td>F</td><td>0-10</td><td>A</td><td>F</td></tr>
<tr><td>>10-15</td><td>B</td><td>F</td><td>>10-15</td><td>B</td><td>F</td></tr>
<tr><td>>15-25</td><td>C</td><td>F</td><td>>15-25</td><td>C</td><td>F</td></tr>
<tr><td>>25-35</td><td>D</td><td>F</td><td>>25-35</td><td>D</td><td>F</td></tr>
<tr><td>>35-50</td><td>E</td><td>F</td><td>>35-50</td><td>E</td><td>F</td></tr>
<tr><td>>50</td><td>F</td><td>F</td><td>>50</td><td>F</td><td>F</td></tr>
</table>

Vehicles use gaps at a T- intersection in a prioritized manner. When traffic becomes congested in a high-priority movement, it can impede lower-priority movements from using gaps in the traffic stream, reducing the potential capacity of these movements. Therefore, movement capacity must be calculated taking account the effect of vehicle impedance and pedestrian impedance. In this research, the effect of pedestrian impedance was neglected.

While there are no absolute standards for degree of saturation, the Australian design procedure suggests that the degree of saturation for an entry lane should be less than 0.85 for satisfactory operation.

As volumes approach capacity, control delay increases exponentially, with small changes in volume having large effects on delay. The Highway Capacity Manual identifies delay as the primary measure of effectiveness for both signalized and unsignalized intersections, with level of service determined from the delay estimate.

Theoretical studies and empirical observations have demonstrated that the probability distribution of queue lengths for any minor movement at an unsignalized intersection is a function of the capacity of the movement and the volume of traffic being served during the analysis period (HCM 2000). The 95th percentile queue lengths for any minor movement at an unsignalized intersection during the peak 15-min period are calculated on the basis of these two parameters.

Level of service is determined based on the control delay value of the intersection approaches.

4.5.1 Performance Analysis of Arategna Intersection

The following tables, table 4.12 to table 4.15, provide the performance analysis parameters for each approach of the Arategna intersections. The analysis is made for consecutive 15 minute time interval and the change of the parameters over a time period is discussed later in this report.

*Table 4.12: Performance measures for **Fadis approach** over a given time series*

Time Series			Performance Measures				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	543	1952	454	1.20	21	136.0	F
2	499	1561	592	0.84	9	34.4	D
3	340	1313	701	0.48	3	12.3	B
4	332	1083	821	0.41	2	9.4	A
5	381	1130	794	0.48	3	11.0	B
6	369	1442	643	0.57	4	15.8	C
7	312	1124	798	0.39	2	9.3	A
8	287	1039	846	0.34	2	8.1	A
9	412	1261	727	0.57	4	14.1	B
10	367	1231	742	0.50	3	12.0	B
11	398	1274	720	0.55	3	13.7	B
12	482	1651	557	0.86	10	39.0	E
13	379	820	981	0.39	2	7.9	A
14	337	1039	846	0.40	2	9.0	A
15	276	806	990	0.28	1	6.4	A
16	320	747	1031	0.31	1	6.6	A
17	390	782	1007	0.39	2	7.8	A
18	280	827	976	0.29	1	6.6	A
19	387	870	948	0.41	2	8.4	A
20	359	831	974	0.37	2	7.7	A
21	363	805	991	0.37	2	7.6	A
22	405	896	932	0.43	2	9.0	A
23	506	1047	841	0.60	4	13.6	B
24	902	1300	707	1.27	34	153.8	F

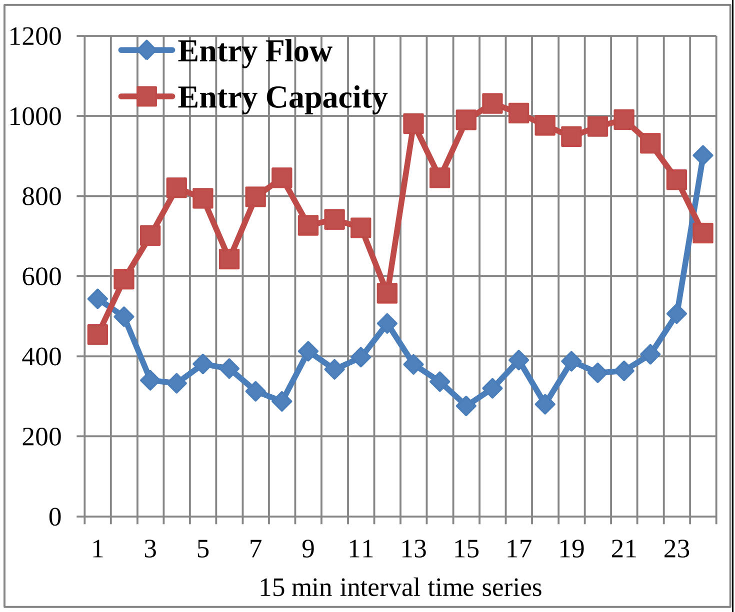

Figure 4.2: Entry flow versus entry capacity variations for Fadis approach

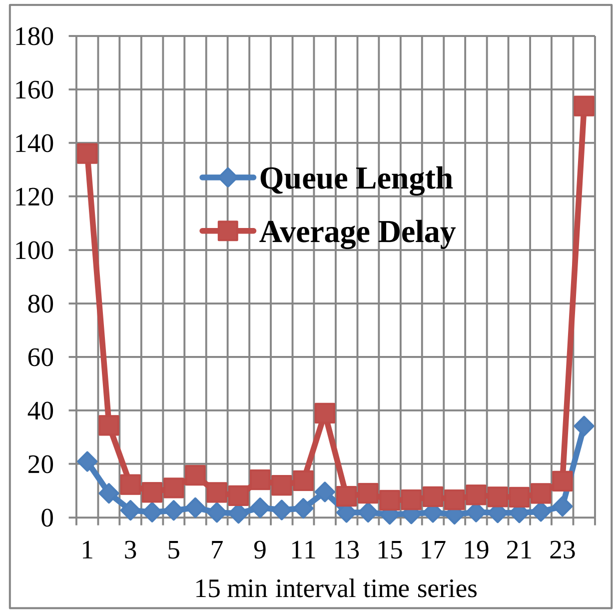

Figure 4.3: Queue length versus average delay variations for Fadis approach

According to table 4.12 and figure 4.2 and 4.3, Fadis approach has very good performances except for morning and afternoon peak periods. During these times it has a level of service F. The degree of saturation for most of time intervals is less than 0.85, while most of the time the approach is serving at a LOS A. About 85% of analysis time has a LOS A and LOS B. The performance of Fadis approach during a peak time is characterized by long delay and queue lengths. These peak times are morning session around 8:00AM and afternoon around 5:00PM. They are a time periods where there are school trips and work trips. The approach is serving at a LOS A in the afternoon from 2:00pm to 4:30 pm.

Table 4.13: Performance measures for ***NOC approach*** *over a given time series*

Time series	Entry Flow	Circulating Flow	**Performance Measures**				
			Entry Capacity	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	1809	440	907	1.99	119	464.5	F
2	1318	668	743	1.77	78	368.8	F
3	1204	320	1007	1.20	37	114.5	F
4	996	344	986	1.01	20	51.6	F
5	1121	300	1026	1.09	27	76.0	F
6	1317	419	924	1.43	58	212.5	F
7	1035	350	981	1.06	23	64.6	F

Time series	Entry Flow	Circulating Flow	Performance Measures				
			Entry Capacity	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
8	925	277	1047	0.88	13	27.0	D
9	1175	403	937	1.25	41	139.7	F
10	1115	348	984	1.13	30	91.3	F
11	1168	343	988	1.18	35	109.9	F
12	1490	557	819	1.82	90	387.7	F
13	826	362	971	0.85	11	24.8	C
14	925	277	1047	0.88	13	27.0	D
15	710	365	969	0.73	7	16.8	C
16	782	321	1007	0.78	8	18.7	C
17	753	394	944	0.80	9	20.9	C
18	859	393	946	0.91	13	32.2	D
19	810	386	951	0.85	11	25.3	D
20	845	486	871	0.97	16	45.1	E
21	848	499	862	0.98	17	48.6	E
22	902	454	896	1.01	19	53.2	F
23	1046	608	784	1.34	42	177.0	F
24	1308	645	758	1.73	75	347.0	F

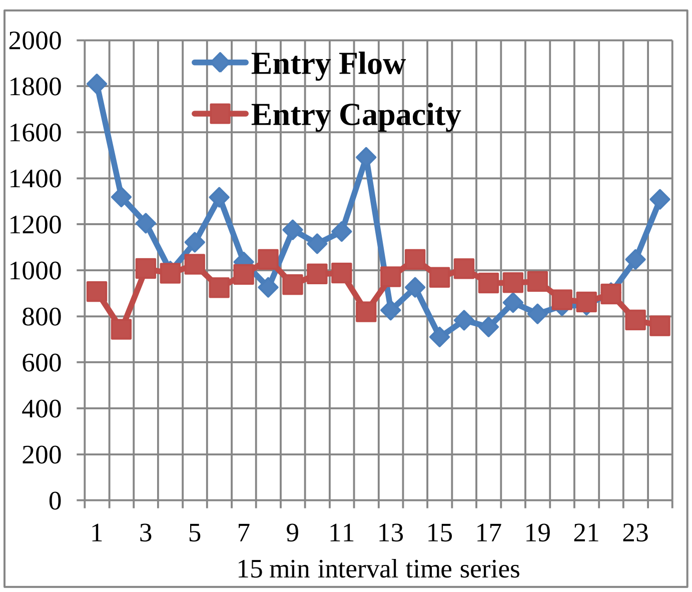

Figure 4.4: Entry flow versus entry capacity variations for NOC approach

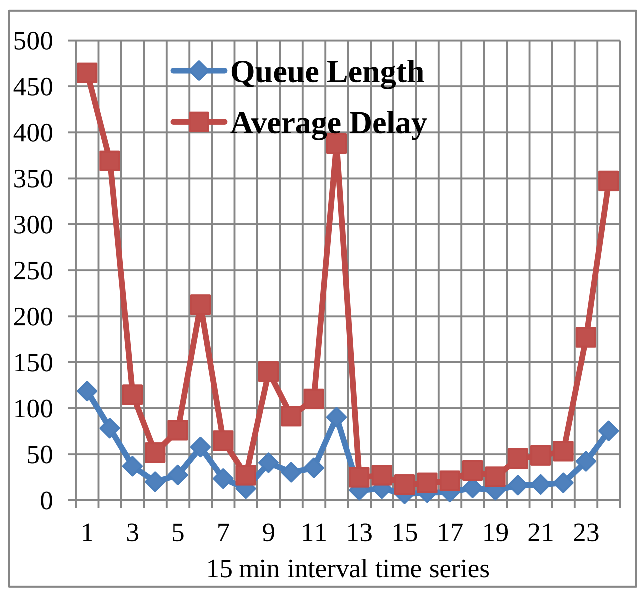

Figure 4.5: Queue length versus average delay variations for NOC approach

The performance of NOC approach for most of the time is closer or above the capacity. For most of the time session, especially in the morning period, the approach is servicing above the capacity with LOS F. At the morning and afternoon peak time, the average control delay reaches more than 7min/veh. There are queues on this approach at the peak hours. Even at the off-peak hours, NOC approach is not experiencing LOS A and B. In general NOC approach is the most congested approach despite the fact that it has three lanes on service. Even if there are many factors for the low performances, imbalanced traffic volumes on the approaches of the Arategna intersection is the basic factor for congestions. The geometry of the intersection has also its own contribution for low performances of the approach. The roundabouts has only 8M diameter central island which is not suitable for the heavy vehicles (like long buses and truck trailers) turning to the left or make U-turn. The vehicles which come from NOC approach (has 3 approaching lane of 3.5m wide) during peak time will face difficulty in moving through the roundabout if there is a presence of heavy vehicles. In general, imbalance of traffic flows among the approaches and poor geometry of the roundabout makes the Arategna intersection the most congested intersection in Harar city.

*Table 4.14: Performance measures for **Warwari approach** over a given time series*

Time series			**Performance Measures**				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	468	1649	416	1.12	17	113.5	F
2	329	1587	437	0.75	6	33.2	D
3	291	1099	638	0.46	2	12.6	B
4	136	1049	664	0.21	1	7.9	A
5	136	1216	582	0.23	1	9.2	A
6	288	1188	595	0.48	3	14.0	B
7	280	1005	687	0.41	2	10.8	B
8	250	942	721	0.35	2	9.4	A
9	286	1214	583	0.49	3	14.4	B
10	255	1070	653	0.39	2	11.0	B
11	253	1135	620	0.41	2	11.8	B
12	400	1408	502	0.80	7	33.9	D
13	364	1013	682	0.53	3	13.8	B
14	250	959	711	0.35	2	9.5	A
15	215	937	724	0.30	1	8.6	A
16	280	913	738	0.38	2	9.7	A
17	291	1256	565	0.51	3	15.5	C
18	314	1043	667	0.47	3	12.5	B
19	349	1096	639	0.55	3	14.9	B
20	356	1169	604	0.59	4	17.1	C

Time series			Performance Measures				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
21	461	1014	682	0.68	5	18.9	C
22	446	1134	621	0.72	6	22.8	C
23	538	1291	549	0.98	14	60.9	F
24	555	1879	348	1.59	32	308.4	F

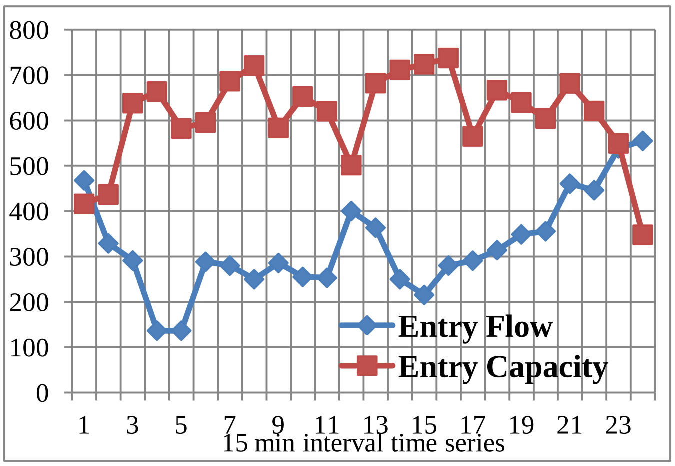

Figure 4.6: Entry flow versus entry capacity variations for Warwari approach

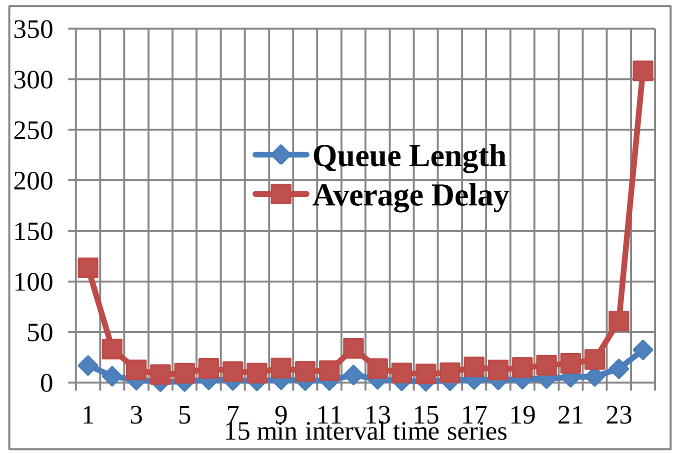

Figure 4.7: Queue length versus average delay variations for Warwari approach

The entry flow rate on Warwari approach is less compared to the other three approaches. But the circulating flow rate is high. This affects the performance of Warwari approach in some cases. But most of the times the performance of the Warwari approach are in a good condition. About 60% of the analysis time has a LOS A and LOS B. Warwari approach exceeds its capacity in the evening around 5:00pm and in the morning around 8:00AM. More than 85% of analysis time has a degree of saturation less than 0.85. The numbers of vehicles waiting in the queues are few except for morning and evening peak time i.e. 8:00-8:15AM and 4:30-5:00PM. At these times the queue length is from 14 vehicles up to 32 vehicles.

*Table 4.15: Performance measures for **Shash garage approach** over a given time series*

Time series			Performance Measures				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	1282	1076	593	2.16	91	544.7	F
2	1364	764	765	1.78	81	372.0	F
3	871	649	841	1.04	20	62.9	F

Time series			**Performance Measures**				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
4	887	543	917	0.97	16	43.6	E
5	1009	535	923	1.09	26	78.6	F
6	1022	766	764	1.34	41	178.3	F
7	809	528	928	0.87	11	27.9	D
8	751	514	939	0.80	9	21.2	C
9	996	669	827	1.20	33	121.8	F
10	871	675	823	1.06	21	69.9	F
11	916	678	821	1.12	25	89.2	F
12	1197	870	702	1.70	68	339.1	F
13	807	517	936	0.86	11	26.6	D
14	751	547	914	0.82	9	23.3	C
15	835	374	1053	0.79	9	19.0	C
16	776	445	993	0.78	8	19.1	C
17	1047	439	998	1.05	23	62.3	F
18	850	404	1027	0.83	10	21.9	C
19	905	513	939	0.96	16	41.9	E
20	978	485	962	1.02	20	54.1	F
21	893	452	988	0.90	13	30.7	D
22	923	529	928	0.99	18	49.4	E
23	1086	638	849	1.28	40	152.0	F
24	1425	979	642	2.22	103	568.7	F

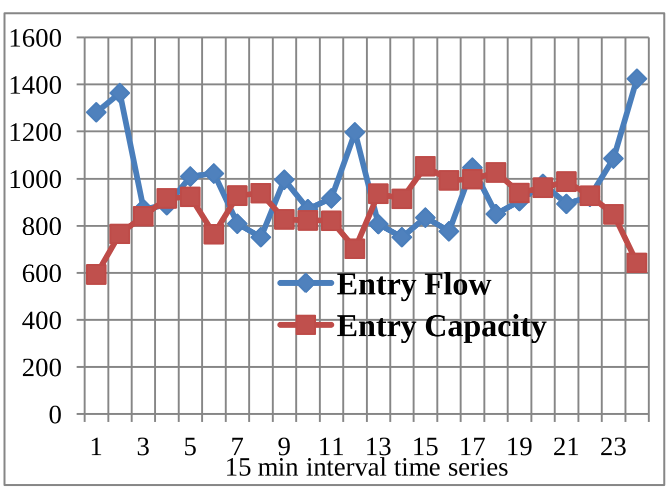

Figure 4.8: Entry flow versus entry capacity variations for Shash garage approach

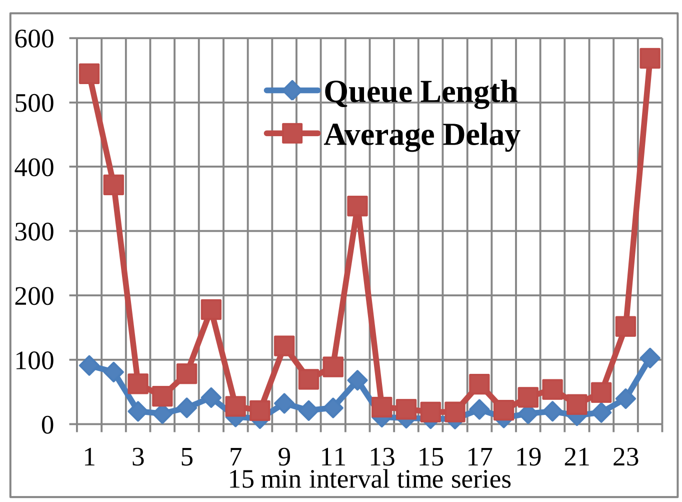

Figure 4.9: Queue length versus average delay variations for Shash garage approach

From six hour analysis time, Shash garage is performed at LOS F for 50% of analysis period. Especially in the morning and evening time, the approach was performing above the capacity. Shash garage has never performed at LOS A or B. Only 21% of the analysis time has a saturation flow rate of less than 0.85.

To summarize the performances of Arategna roundabout, the performance parameters at peak hour are extracted from tables 4.12 to table 4.15 and summarized in table 4.16 below. The peak hour for Arategna intersection is from 8:00AM to 9:00AM.

Table 4:16: Summary of performances of Arategna roundabout during peak hour

Time Range	Approach	Performance Measures				
		Entry Capacity	Degree of Saturation	Queue Length	Average Delay	LOS
		Qe (vph)	V/C	Q95 (veh)	D (s/veh)	
8:00AM To 8:15AM	Fadis	454	1.20	21	136.0	F
	NOC	907	1.99	119	464.5	F
	Warwari	416	1.12	17	113.5	F
	Shash G.	593	2.16	91	544.7	F
8:15AM To 8:30AM	Fadis	592	0.84	9	34.4	D
	NOC	743	1.77	78	368.8	F
	Warwari	437	0.75	6	33.2	D
	Shash G.	765	1.78	81	372.0	F
8:30AM To 8:45AM	Fadis	701	0.48	3	12.3	B
	NOC	1007	1.20	37	114.5	F
	Warwari	638	0.46	2	12.6	B
	Shash G.	841	1.04	20	62.9	F
8:45AM To 9:00AM	Fadis	821	0.41	2	9.4	A
	NOC	986	1.01	20	51.6	F
	Warwari	664	0.21	1	7.9	A
	Shash G.	917	0.97	16	43.6	E

During peak periods, Arategna approach is providing a service above its capacity. NOC and Shash garage approach are the two most congested approaches. Fadis and Warwari approaches showed better performances than the others two. This is because the approach flow of Fadis and Warwari approach is very small compared to the flow of NOC and Shash garage approaches.

4.5.2 Performance Analysis of Sillassie Intersection

The performance analysis in 15 minute interval for 24 consecutive time series for Sillassie roundabout approaches is presented in the following table, table 4.17 to table 4.20.

*Table 4.17: Performance measures for **Ras mekonnen approach** over a given time series*

Time series			Performance Measures				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	1223	206	1242	0.98	20	40.4	E
2	1193	192	1256	0.95	18	33.2	D
3	1167	132	1318	0.89	14	23.3	C
4	1204	180	1267	0.95	18	33.0	D
5	1094	116	1334	0.82	10	17.7	C
6	1262	216	1232	1.02	24	50.4	F
7	1209	189	1259	0.96	18	35.1	E
8	1242	204	1244	1.00	21	43.6	E
9	1300	186	1261	1.03	25	51.7	F
10	1186	184	1264	0.94	17	31.1	D
11	1132	179	1269	0.89	14	24.6	C
12	1261	189	1258	1.00	22	44.3	F
13	894	114	1336	0.67	6	11.3	B
14	804	113	1337	0.60	4	9.7	A
15	928	145	1304	0.71	6	12.8	B
16	1059	198	1249	0.85	11	20.6	C
17	978	83	1369	0.71	7	12.5	B
18	897	225	1223	0.73	7	14.2	B
19	979	111	1339	0.73	7	13.2	B
20	970	64	1391	0.70	6	11.8	B
21	1304	217	1231	1.06	27	60.9	F
22	982	177	1271	0.77	8	15.6	C
23	911	170	1278	0.71	7	13.0	B
24	991	145	1303	0.76	8	14.7	B

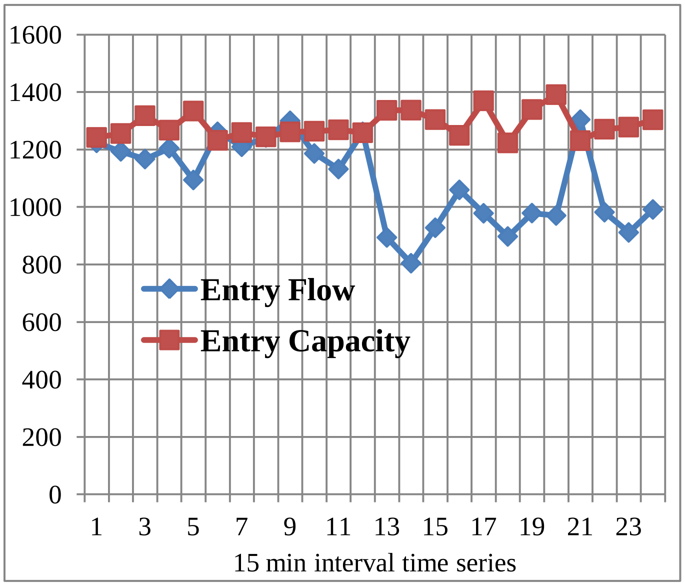

Figure 4.10: Entry flow versus entry capacity variations for Ras mekonnen approach

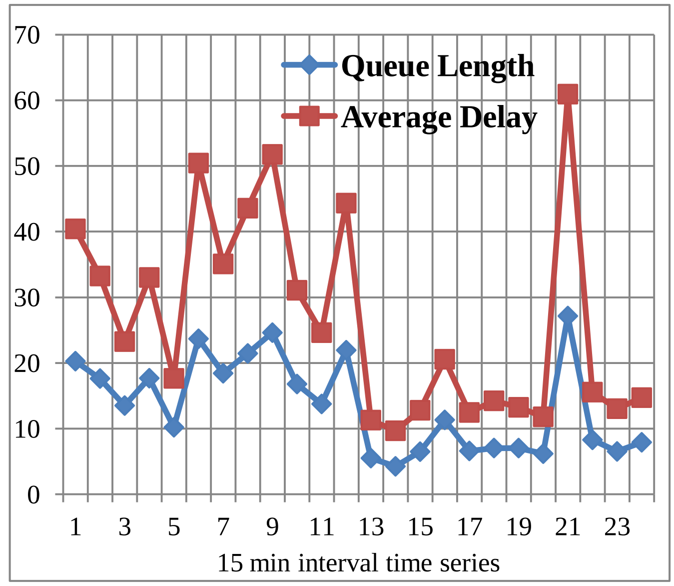

Figure 4.11: Queue length versus average delay variations for Ras mekonnen approach

As it is indicated in table 4.17, the entry flows of Ras mekonnen approach are greater than the circulating flows. For 15% of the analysis time Ras mekonnen approach is servicing above the capacity, i.e. LOS F. These are the times during peak hours (around 9:30AM to 10:30AM). During these times the average delay is closer to 50sec/min. The delay also changes repeatedly from time to time. In the morning time the performance of this approach is closer to capacity, while in the afternoon session the approach showed a good performance. The queue is decreased in the afternoon time.

Table 4.18: Performance measures for ***Bote approach*** *over a given time series*

Time series	Entry Flow	Circulating Flow	**Performance Measures**				
			Entry Capacity	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	486	1205	587	0.83	9	33.0	D
2	484	1164	607	0.80	8	29.3	D
3	485	1104	636	0.76	7	25.3	D
4	523	1178	600	0.87	10	37.9	E
5	452	1034	671	0.67	5	19.1	C
6	491	1249	568	0.86	10	38.5	E
7	452	1182	598	0.75	7	26.0	D
8	516	1254	566	0.91	11	45.8	E
9	434	1274	557	0.78	7	29.5	D

Time series			Performance Measures				
	Entry Flow	Circulating Flow	Entry Capacity	Degree of Saturation	Queue Length	Average Delay	LOS
	qa (vph)	qc (vph)	Qe (vph)	V/C	Q95 (veh)	D (s/veh)	
10	523	1148	614	0.85	9	34.8	D
11	484	1121	627	0.77	7	26.2	D
12	508	1224	579	0.88	10	39.6	E
13	321	887	752	0.43	2	10.4	B
14	364	796	807	0.45	2	10.3	B
15	358	931	727	0.49	3	12.1	B
16	431	1077	649	0.66	5	19.1	C
17	321	938	723	0.44	2	11.1	B
18	334	955	714	0.47	3	11.7	B
19	332	949	717	0.46	2	11.6	B
20	375	913	737	0.51	3	12.4	B
21	496	1287	551	0.90	11	44.5	E
22	391	997	691	0.57	4	14.6	B
23	349	912	738	0.47	3	11.5	B
24	429	988	696	0.62	4	16.2	C

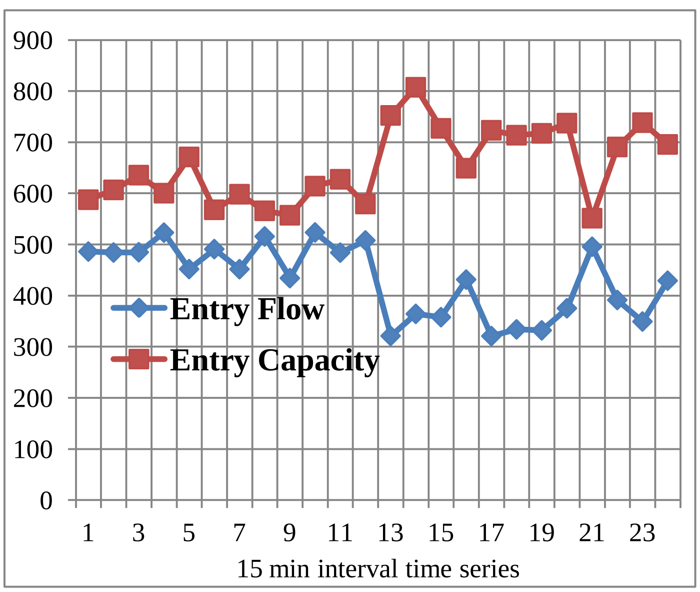

Figure 4.12: Entry flow versus entry capacity variations for Bote approach

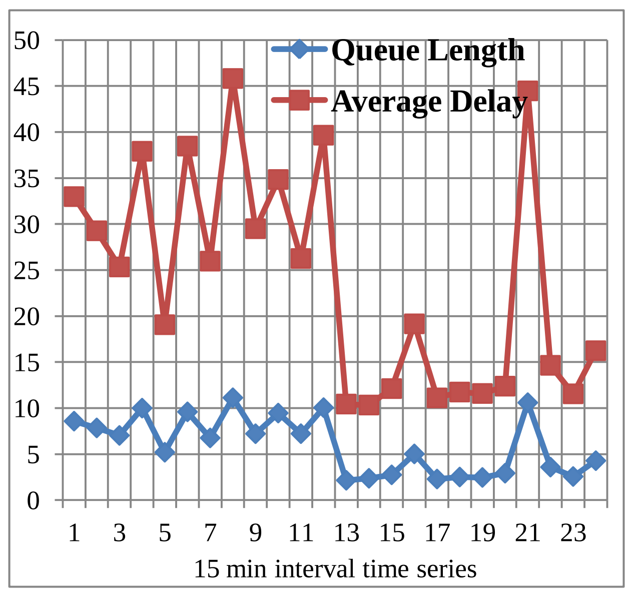

Figure 4.13: Queue length versus average delay variations for Bote approach

Bote approach is providing a service closer the capacity. There is a very high variation between approach flows and conflicting flows i.e. Conflicting flows are greater than approach flows. Even though the approach does not experience LOS F yet, in the morning time it is providing service very closer to its capacity. There is a fluctuation of delay from time to time abruptly. In the afternoon period this approach is working at a LOS B and C except from 4:00PM to 4:15PM where the approach is at LOS E. The maximum average delay for Bote approach is 46 sec/veh and it is experienced at 9:45AM to 10:00AM.

Table 4.19: Performance measures for ***Shewa barr approach*** *over a given time series*

Time series			Performance Measures				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	629	878	697	0.90	12	38.5	E
2	628	880	696	0.90	12	38.4	E
3	595	821	731	0.81	9	26.8	D
4	607	851	713	0.85	10	31.1	D
5	511	771	761	0.67	5	17.2	C
6	676	874	700	0.97	15	50.3	F
7	673	875	699	0.96	14	49.4	E
8	655	889	691	0.95	14	46.9	E
9	696	926	671	1.04	18	69.6	F
10	641	860	708	0.90	12	38.5	E
11	563	806	740	0.76	7	22.4	C
12	665	973	646	1.03	17	68.6	F
13	425	572	895	0.48	3	10.0	A
14	467	569	898	0.52	3	10.9	B
15	473	618	863	0.55	3	11.9	B
16	641	758	769	0.83	9	27.6	D
17	411	620	861	0.48	3	10.3	B
18	583	622	860	0.68	5	15.9	C
19	586	668	828	0.71	6	17.7	C
20	847	644	845	1.00	18	53.9	F
21	667	913	678	0.98	15	55.4	F
22	594	697	808	0.73	7	19.4	C
23	522	613	866	0.60	4	13.3	B
24	589	732	786	0.75	7	20.7	C

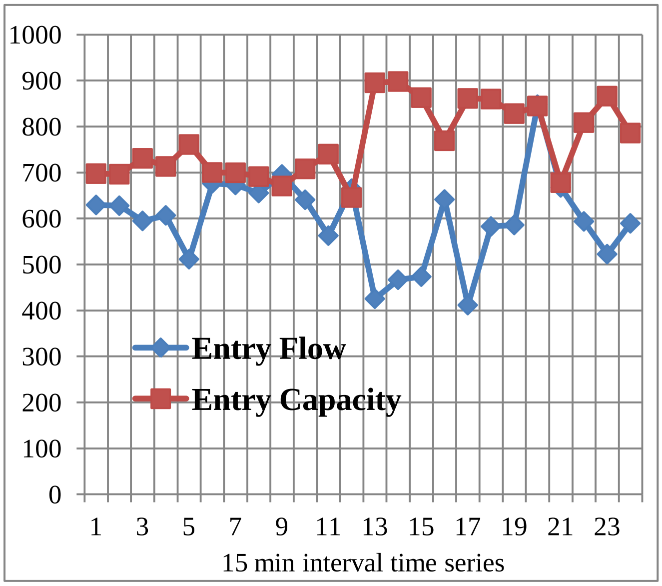

Figure 4.14: Entry flow versus entry capacity variations for Shewa barr approach

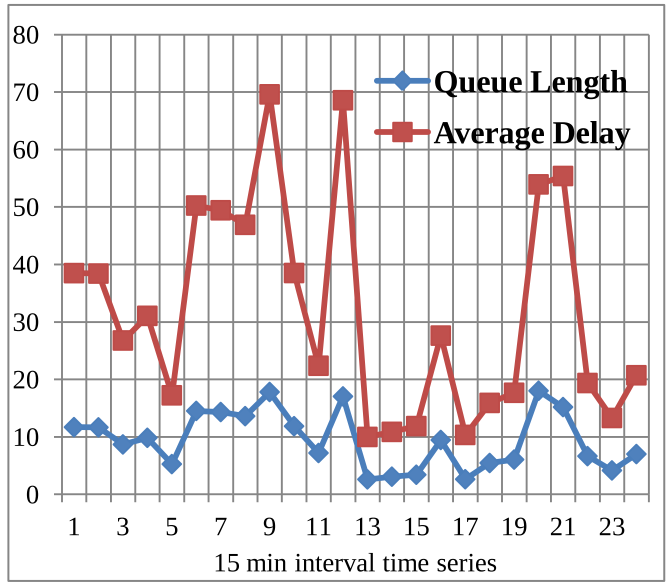

Figure 4.15: Queue length versus average delay variations for Shewa barr approach

Shewa barr approach is the approach which takes to/from the cities' bus station. It is characterized by loading and unloading of passengers, roads side businesses and a very complex traffic mixes. According to table 4.19, figure 4.14 and 4.15, the performance of Shewa barr approach is closer to the capacity. For about 20% of analysis time it is serving above the capacity (LOS F) while for another 20% it is very close to capacity (LOS E). During these times the average delay is about 1min/veh. Besides, average delay is changing abruptly in 15 minute intervals over the analysis period. A queue was observed for some period of the analysis times. The performances for morning and afternoon sessions do not indicate any significant differences. The volumes and traffic mixes are almost similar in complexity over the selected time periods and hence the performances.

Table 4.20: Performance measures for ***Andegna menged approach*** *over a given time series*

Time series			Performance Measures				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
1	836	681	1283	0.65	5	11.1	B
2	805	698	1271	0.63	5	10.8	B
3	972	633	1321	0.74	7	13.5	B
4	912	669	1293	0.71	6	12.7	B
5	893	562	1379	0.65	5	10.5	B
6	729	794	1200	0.61	4	10.6	B

Time series			Performance Measures				
	Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
7	790	757	1227	0.64	5	11.3	B
8	779	736	1242	0.63	5	10.8	B
9	846	729	1248	0.68	6	12.1	B
10	870	695	1273	0.68	6	12.1	B
11	845	597	1351	0.63	5	10.1	B
12	831	764	1221	0.68	6	12.4	B
13	642	462	1465	0.44	2	6.6	A
14	587	545	1394	0.42	2	6.6	A
15	690	566	1376	0.50	3	7.7	A
16	777	690	1277	0.61	4	10.1	B
17	660	492	1439	0.46	2	6.9	A
18	750	678	1286	0.58	4	9.6	A
19	728	694	1274	0.57	4	9.4	A
20	620	759	1225	0.51	3	8.4	A
21	810	723	1251	0.65	5	11.2	B
22	769	698	1271	0.60	4	10.1	B
23	684	608	1342	0.51	3	8.0	A
24	736	683	1282	0.57	4	9.4	A

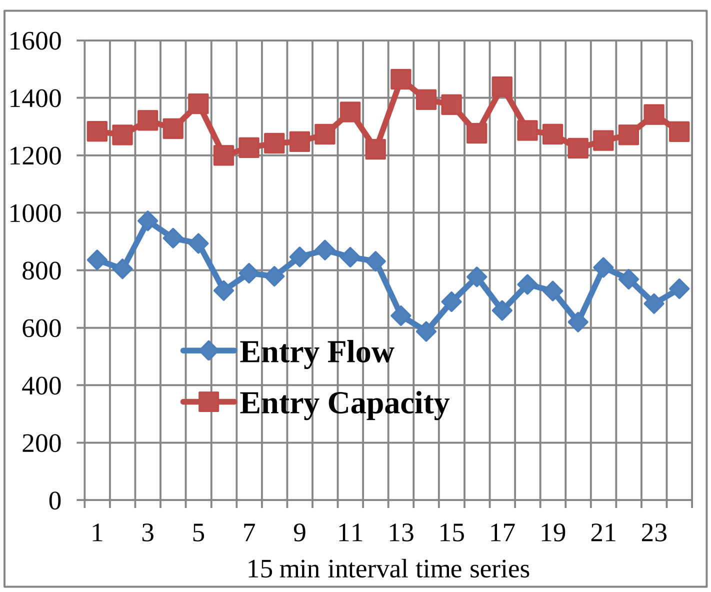

Figure 4.16: Entry flow versus entry capacity variations for Andegna menged approach

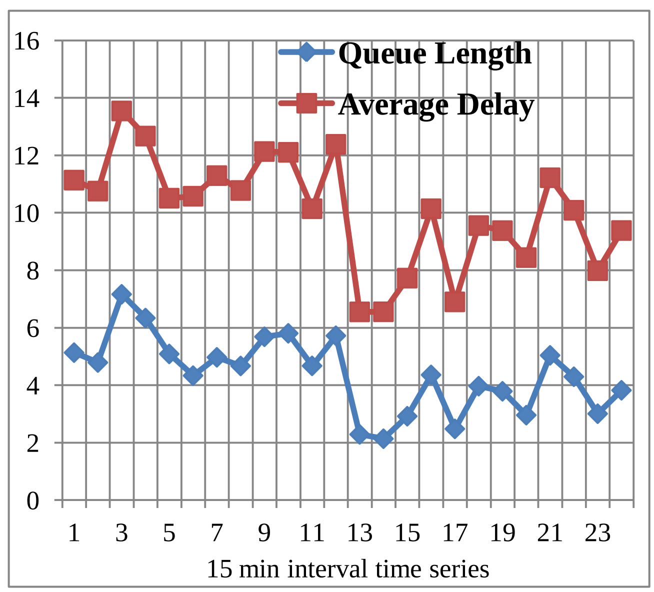

Figure 4.17: Queue length versus average delay variations for Andegna menged approach

Andegna menged is a three lane approach with minor pedestrian and roadside effects on the movements of the vehicles. It is the approach which performs at a blessing capacity. The flow is approximately in stable state. For all the times, the services are at LOS A and B. The maximum queue length observed during the analysis period is 7 veh and maximum average delay is 13 sec/veh, which are very small compared to others. The through movements of vehicles from andegna menged approach can cross the conflicting areas without any problems since the roundabout has a wider area for this purpose. This also helped the approach to operate at good conditions even during peak hour.

The summary for performances of Sillassie roundabout is indicated in table 4.21 below for peak hour. The peak hour for Sillassie intersection is from 9:30AM to 10:30AM.

Table 4:21: Summary of performances of Sillassie roundabout during peak hour

Time Range	Approach	Performance Measures				
		Entry Capacity	Degree of Saturation	Queue Length	Average Delay	LOS
		Qe (vph)	V/C	Q95 (veh)	D (s/veh)	
9:30AM To 9:45AM	Ras Mekonnen	1259	0.96	18	35.1	E
	Bote	598	0.75	7	26.0	D
	Shewa barr	699	0.96	14	49.4	E
	Andegna Menged	1227	0.64	5	11.3	B
9:45AM To 10:00AM	Ras Mekonnen	1244	1.00	21	43.6	E
	Bote	566	0.91	11	45.8	E
	Shewa barr	691	0.95	14	46.9	E
	Andegna Menged	1242	0.63	5	10.8	B
10:00AM To 10:15AM	Ras Mekonnen	1261	1.03	25	51.7	F
	Bote	557	0.78	7	29.5	D
	Shewa barr	671	1.04	18	69.6	F
	Andegna Menged	1248	0.68	6	12.1	B
10:15AM To 10:30AM	Ras Mekonnen	1264	0.94	17	31.1	D
	Bote	614	0.85	9	34.8	D
	Shewa barr	708	0.90	12	38.5	E
	Andegna Menged	1273	0.68	6	12.1	B

As discussed in previous tables, the performances of Sillassie intersection during peak hour are closer to the capacity except for Andegna menged approach. Ras mekonnen and Shewa barr approaches have performed above the capacity from 10:00AM to 10:15AM. Andegna menged is performing well with average delay of maximum 12 sec/veh during peak hour. In general the performance of Sillassie roundabout is better than Arategna roundabout during peak periods.

4.5.3 Performance Analysis of Shash Garage Intersection

The performance analysis in 15 minute interval for 24 consecutive time series for Shash garage approach movements are presented in the following table, table 4.22 to table 4.24.

*Table 4.22: Performance measures for **Hajib approach LT movements (movement 4)** over a given time series*

Time series			Performance Measures					
	Approach Flow	Conflicting Flow	**Potential Capacity**	**Movement Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	V_4 (vph)	**$V_{c,4}$ (vph)**	**$C_{p,4}$ (vph)**	**$C_{m,4}$ (vph)**	**V/C_m**	**Q_{95} (veh)**	**D (s/veh)**	
1	41	1132	286	286	0.14	0	15.4	C
2	104	1062	311	311	0.33	1	18.9	C
3	41	851	399	399	0.10	0	10.6	B
4	86	791	428	428	0.20	1	11.5	B
5	44	929	364	364	0.12	0	11.9	B
6	22	933	362	362	0.06	0	10.9	B
7	33	850	399	399	0.08	0	10.2	B
8	35	481	612	612	0.06	0	6.5	A
9	35	916	370	370	0.09	0	11.2	B
10	35	979	343	343	0.10	0	12.2	B
11	49	929	364	364	0.13	0	12.1	B
12	47	946	357	357	0.13	0	12.3	B
13	138	843	403	403	0.34	1	15.3	C
14	123	855	397	397	0.31	1	14.6	B
15	112	848	400	400	0.28	1	13.8	B
16	65	1040	319	319	0.20	1	15.1	C
17	115	794	426	426	0.27	1	12.9	B
18	44	826	411	411	0.11	0	10.4	B
19	31	740	454	454	0.07	0	8.8	A
20	138	823	412	412	0.34	1	14.8	B
21	31	675	489	489	0.06	0	8.2	A
22	129	953	354	354	0.37	2	17.7	C
23	43	903	375	375	0.11	0	11.4	B
24	146	887	383	383	0.38	2	17.0	C

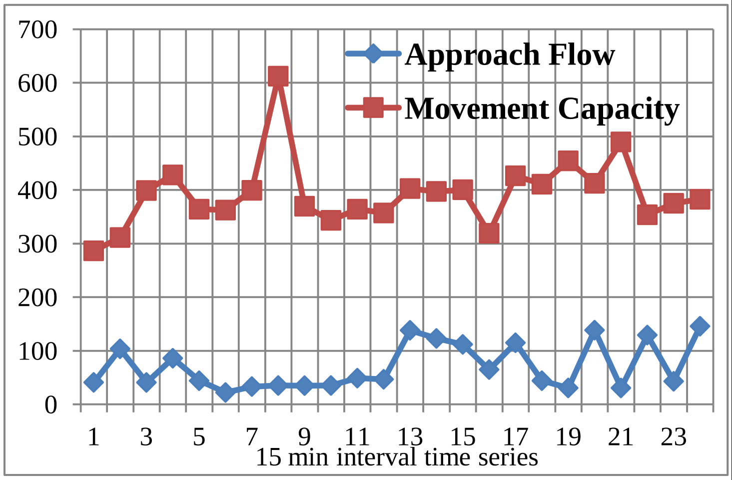

Figure 4.18: Entry flow versus entry capacity variations for Hajib approach LT movements

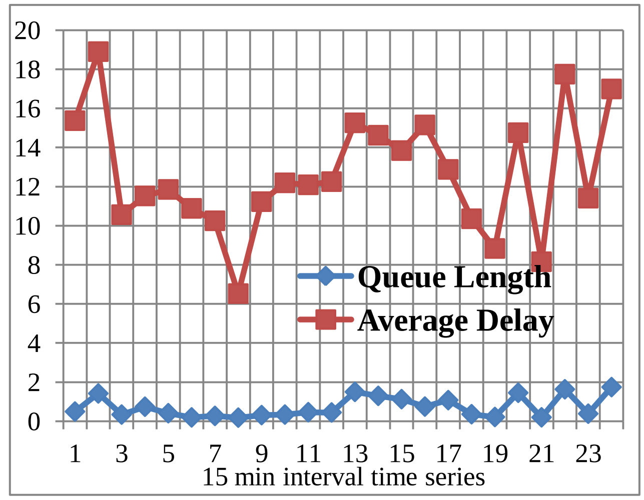

Figure 4.19: Queue length versus average delay variations for Hajib approach LT movements

Table 4.22 and figures 4.18 & 4.19 indicates that all the performance measurement parameters for Hajib LT movements are in a good condition. The queue length is almost zero. At all times the approach is providing a service at a rate of very smaller than the capacity. The minimum LOS is C at peak hour. The maximum degree of saturation experienced for these movements is 0.37 which indicates a good performance though.

Table 4.23: Performance measures for ***Bira approach LT movements (movement 7)*** *over a given time series*

Time series			Performance Measures					
	Approach Flow	Conflicting Flow	**Potential Capacity**	**Movement Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	V7 (vph)	**Vc,7 (vph)**	**Cp,7 (vph)**	**Cm,7 (vph)**	**V/Cm**	**Q95 (veh)**	**D (s/veh)**	
1	151	1678	165	141	1.07	8	159.3	F
2	221	1548	192	128	1.73	17	417.7	F
3	68	1271	265	238	0.29	1	22.6	C
4	81	1243	274	219	0.37	2	27.8	D
5	72	1391	231	203	0.35	2	28.9	D
6	110	1363	238	224	0.49	2	33.3	D
7	87	1348	242	222	0.39	2	28.3	D
8	49	768	470	443	0.11	0	9.7	A
9	58	1441	218	197	0.30	1	27.2	D
10	39	1370	236	212	0.19	1	21.7	C
11	96	1488	206	178	0.54	3	44.1	E
12	82	1465	212	184	0.45	2	36.8	E

Time series			Performance Measures					
	Approach Flow	Conflicting Flow	**Potential Capacity**	**Movement Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	V7 (vph)	**Vc,7 (vph)**	**Cp,7 (vph)**	**Cm,7 (vph)**	**V/Cm**	**Q95 (veh)**	**D (s/veh)**	
13	129	1548	192	126	1.02	7	152.8	F
14	102	1437	219	151	0.68	4	66.6	F
15	124	1436	219	158	0.79	5	80.7	F
16	193	1452	215	171	1.13	10	162.8	F
17	118	1498	203	149	0.79	5	85.5	F
18	109	1241	274	245	0.44	2	28.2	D
19	103	1188	292	272	0.38	2	22.9	C
20	122	1511	200	133	0.91	6	119.7	F
21	99	1096	324	304	0.33	1	19.1	C
22	146	1592	182	116	1.26	10	241.1	F
23	144	1457	213	189	0.76	5	66.1	F
24	201	1513	200	124	1.62	15	377.1	F

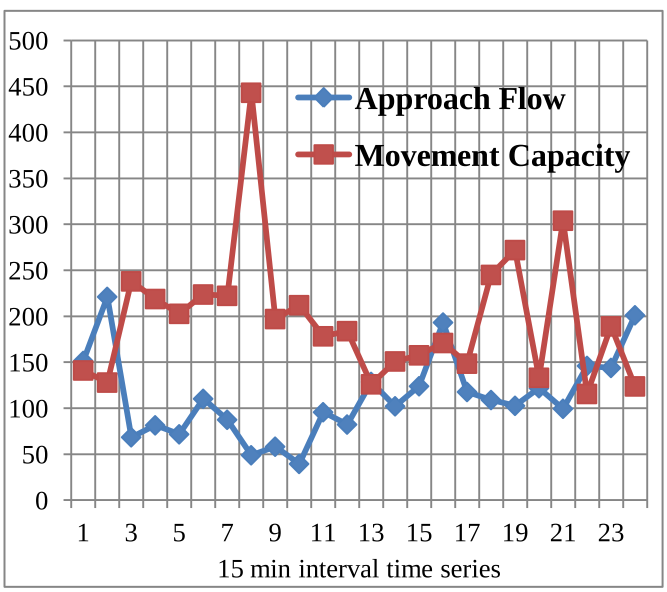

Figure 4.20: Entry flow versus entry capacity variations for Bira approach LT movements

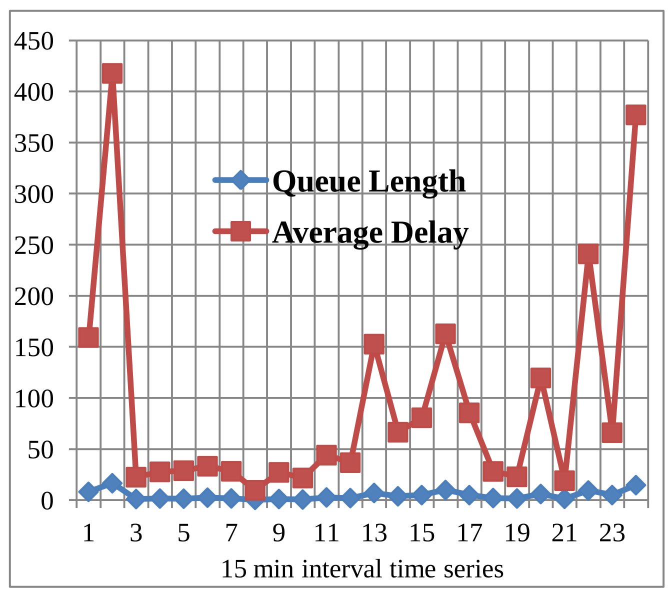

Figure 4.21: Queue length versus average delay variations for Bira approach LT movements

The left turn movements on Bira approach is performing at LOS F during peak hours and early morning and evening time. Even if the queue length for this approach is small, the average delay of the vehicles is large and the approach is working very close to the capacity. Sometimes when the flow rate is less than the capacity, the LOS can be F due to large average delay. Thus, the vehicles on Bira approach which tends to move left turn should wait for a long time gap to join the major street traffic stream.

*Table 4.24: Performance measures for **Bira approach RT movements (movement 9)** over a given time series*

Time series			Performance Measures					
	Approach Flow	Conflicting Flow	**Potential Capacity**	**Movement Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
	V9 (vph)	**Vc,9 (vph)**	**Cp,9 (vph)**	**Cm,9 (vph)**	**V/C_m**	**Q_{95} (veh)**	**D (s/veh)**	
1	62	566	1314	1314	0.05	0	3.1	A
2	177	531	1343	1343	0.13	0	3.7	A
3	87	426	1436	1436	0.06	0	3.0	A
4	86	396	1463	1463	0.06	0	2.9	A
5	90	465	1401	1401	0.06	0	3.1	A
6	47	466	1399	1399	0.03	0	2.8	A
7	99	425	1436	1436	0.07	0	3.0	A
8	51	241	1613	1613	0.03	0	2.5	A
9	65	458	1407	1407	0.05	0	2.9	A
10	71	489	1379	1379	0.05	0	3.0	A
11	67	465	1401	1401	0.05	0	2.9	A
12	90	473	1393	1393	0.06	0	3.1	A
13	103	422	1439	1439	0.07	0	3.1	A
14	120	427	1434	1434	0.08	0	3.2	A
15	115	424	1437	1437	0.08	0	3.1	A
16	139	520	1352	1352	0.10	0	3.5	A
17	82	397	1462	1462	0.06	0	2.9	A
18	53	413	1447	1447	0.04	0	2.8	A
19	79	370	1487	1487	0.05	0	2.8	A
20	110	411	1449	1449	0.08	0	3.1	A
21	73	338	1518	1518	0.05	0	2.7	A
22	110	477	1390	1390	0.08	0	3.2	A
23	101	452	1412	1412	0.07	0	3.1	A
24	179	443	1420	1420	0.13	0	3.5	A

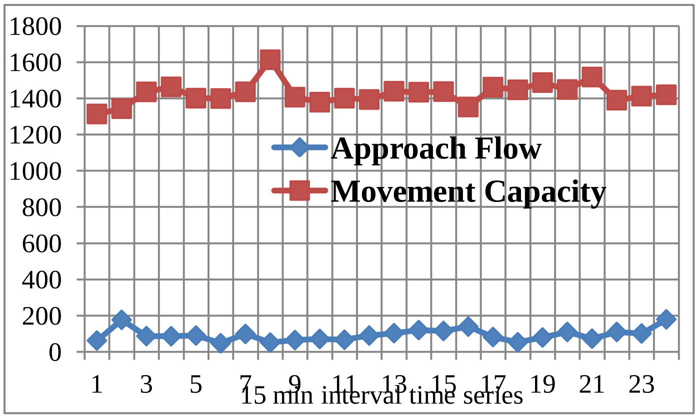

Figure 4.22: Entry flow versus entry capacity variations for Bira approach RT movements

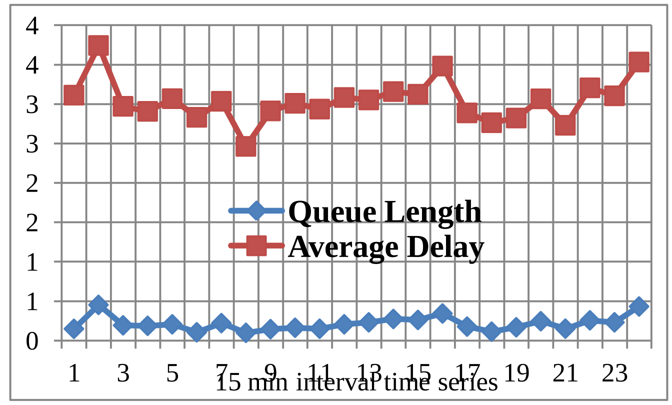

Figure 4.23: Queue length versus average delay variations for Bira approach RT movements

Right turn movement of Bira approach is under a very good condition during the analysis period. This means, the effect of major stream flow on right turn movement of Bira approach is very small. From table 4.24, the movements have a LOS A for the whole analysis period. There is no queue for these movements. The maximum time the vehicles wait to make right turn is four seconds. The maximum degree of saturation is 0.1 which indicates a maximum capacity for right turning movements compared to the existing flows.

The performances of Shash garage intersection movements during peak periods are summarized in table 4.25. The peak period for Bira intersection is from 2:00PM to 3:00PM.

Table 4.25: Summary of performances of Shash garage intersection during peak hour

Time Range	Movements	Performance Measures				
		Movement Capacity	Degree of Saturation	Queue Length	Average Delay	LOS
		Cm,4 (vph)	V/C	Q95 (veh)	D (s/veh)	
2:00PM To 2:15PM	Hajib LT	403	0.34	1	15.3	C
	Bira LT	126	1.02	7	152.8	F
	Bira RT	1439	0.07	0	3.1	A
2:15PM To 2:30PM	Hajib LT	397	0.31	1	14.6	B
	Bira LT	151	0.68	4	66.6	F
	Bira RT	1434	0.08	0	3.2	A
2:30PM To 2:45PM	Hajib LT	400	0.28	1	13.8	B
	Bira LT	158	0.79	5	80.7	F
	Bira RT	1437	0.08	0	3.1	A
2.45PM To 3:00PM	Hajib LT	319	0.20	1	15.1	C
	Bira LT	171	1.13	10	162.8	F
	Bira RT	1352	0.10	0	3.5	A

At Shash garage intersection Bira approach right turn movements are in a very good condition. There are no any queues for these movements. The Bira approach is flared at the end which helps a smooth right turning movements with minimum gap acceptance. But the left turn movements from Bira approach are the most congested traffic flows during peak hour. The average delays for these movements are 3 minutes per vehicle. The vehicles shall wait for the gaps of major stream traffic flows to make the left turn. Hajib left turn movements shall also wait for major stream available gaps to make the turn. The difference is that Bira LT movements give priority to Hajib LT movements too. This makes Bira LT movements the most congested flows at Shash garage intersection followed by Hajib LT movements and Bira RT movements.

Movements other than discussed above are priority movements. They are Hajib approach through movements, and Shash garage approach right turn and through movements. For T-intersections, it is not necessary to check for the performance of priority movements.

4.6 Statistical Analysis of the Effect of Vehicle Compositions on Performance Parameters of the Selected Intersections

One of the specific objectives of this research is to analyze the effects of the vehicle compositions on performance of the intersections over a selected time period. As it is indicated in the previous analysis (section 4.4), the volume of vehicles are changing from time to time differently for different vehicle classes. This change affects the performance parameters (i.e. degree of saturation, delay, queue length, and LOS) in a different manner. The composition of one type of vehicle may affect the performance the intersection more than the other type of vehicle. The main two statistical analyses performed in this study are:

- Multiple linear regression and
- Cross validation

a) Multiple Linear Regression Analysis

The analysis method used in this study to indicate the trend of vehicle composition changes and performance parameters is multiple regression analysis. Regression analysis generates an equation to describe the statistical relationship between a set of independent variables and dependent variable. The compositions of different vehicle classes are considered as independent variables while the performance measurement parameters are taken as dependent variables. Stata software is the potential program to analyze this kind of relationships and to investigate the significance of the variables on the outputs of the model.

The basic regression equation is;

$$\mathbf{Y} = \beta_0 + \beta_1\mathbf{X}_1 + \beta_2\mathbf{X}_2 + \varepsilon$$

Where;

- Y is dependent variable
- X are independent variables
- e is error term

In the above regression equation, β1 measures the effect of X1 on Y. Similarly β2 measures the effect of X2 on Y. The constant term (β0) measures the value of Y if both X1 and X2 are zero. The error term e includes other factors which effect Y other than X1 and X2.

`Stata` will generate a single piece of output for a multiple regression analysis. In the analysis, there are variables to check how well the model fit the data. These models are discussed as follows;

- **"R-squared":-** This row represents the R^2 value (also called the coefficient of determination), which is the proportion of variance in the dependent variable that can be explained by the independent variables. Technically, it is the proportion of variation accounted for by the regression model above and beyond the mean model.
- **Coefficients: -** Coefficients column labeled as **"Coef."** in stata indicate how much the dependent variable varies with an independent variable, when all other independent variables are held constant.

In this research there are 24 observations (i.e. 15 minute intervals of intersection performance analysis periods). The variables are as follows;

- **Independent Variables;**
  - X1 - Hourly flow rate of bajajs
  - X2 - Hourly flow rate of cars & taxis
  - X3 - Hourly flow rate of 4WDs
  - X4 - Hourly flow rate of mini buses
  - X5 - Hourly flow rate of medium buses
  - X6 - Hourly flow rate of large buses
  - X7 - Hourly flow rate of medium trucks
  - X8 - Hourly flow rate of large trucks and
  - X9 - Hourly flow rate of trucks and trailers
- **Dependent Variables;**
  - Y1 – Degree of saturation, v/c
  - Y2 – Queue length, Q_{95} (veh)
  - Y3 – Average delay, D (s/veh)

To evaluate whether multiple linear regression results may be translated into practice requires the development of methods that allow the cross-validation of the derived summary estimates in new independent settings.

b) Model Validation

Although methods have been developed to quantify and ascertain the effects of vehicle composition on performance parameters, more recently, particularly in the field of predictive modeling, the focus has been on developing statistical approaches that increase the validity of the results when applied in new populations. Cross-validation is a model validation technique for assessing how the results of a statistical analysis will generalize to an independent data set. It is mainly used in settings where the goal is prediction, and one wants to estimate how accurately a predictive model will perform in practice.

Cross-validation involves partitioning a sample of data into complementary subsets, performing the analysis on one subset (called the training set), and validating the analysis on the other subset (called the validation set or testing set). The data splitting ratio commonly used is partitioning the data set into two sets of 70% for training and 30% for test (Shao, 1997). Accordingly, from the 24 data sets in this study, training and test data are selected at random and theirs root mean square error is compared.

In a good model, Root Mean Square Error (RMSE) should be close for both testing data and training data. The RMSE is the square root of the variance of the residuals.

$$RMSE = \sqrt{\frac{1}{n}\sum_{1}^{n} e^2}$$

It indicates the absolute fit of the model to the data–how close the observed data points are to the model's predicted values. If the RMSE for the testing data is higher than the training data, there is a high chance that the model overfit. RMSE values can be extracted from stata output.

4.6.1 Statistical Analysis for Arategna Intersection

The statistical analysis by `stata` software for four approaches of Arategna intersection is discussed below in table 4.28. For example, the relationship between the vehicle compositions and performance measurement parameters for Fadis approach traffic movements can be described by the consecutive equations described in table 4.27. Table 4.26 indicates the stata output for the analysis between vehicle compositions and degree of saturation for the selected day.

Table 4.26 Descriptive statistics between degree of saturation and vehicle compositions for Fadis approach

Source	SS	df	MS
Model	1.4754467	7	.2107781
Residual	.110736676	16	.006921042
Total	1.58618338	23	.068964495

Number of obs	= 24
F(7, 16)	= 30.45
Prob > F	= 0.0000
R-squared	= 0.9302
Adj R-squared	= 0.8996
Root MSE	= .08319

vc	Coef.	Std. Err.	t	P>|t|	[95% Conf.	Interval]
bajajs	.0018549	.0005846	3.17	0.006	.0006156	.0030942
carstaxi	.0008132	.0019014	0.43	0.675	-.0032176	.004844
wd	.0022472	.0057552	0.39	0.701	-.0099534	.0144477
minibus	.0026786	.0023401	1.14	0.269	-.0022822	.0076394
mediumbus	.0160837	.0029398	5.47	0.000	.0098516	.0223158
mediumtruck	.0095835	.0040914	2.34	0.032	.0009102	.0182568
tt	.0261252	.0090833	2.88	0.011	.0068695	.0453809
_cons	-.3781934	.1053797	-3.59	0.002	-.6015885	-.1547984

From table 4.23 above, the relationship between degree of saturation and vehicle classes is described as below with R^2 of 0.9302.

$$Y1 = -0.3782 + 0.0019X1 + 0.001X2 + 0.0022X3 + 0.0028X4 + 0.0161X5 + 0.0096X7 + 0.0261X9$$

There are a direct relationships between the independent and dependent variables. An increase of one unit in any vehicle composition will increase the degree of saturation and in turn decreases the performances of the intersections even though their effect is different from vehicles to vehicles. From the relations developed above, Bajaj vehicles have more effect than cars & taxis on the degree of saturation. This means one increase of bajaj vehicles on Fadis approach will increase the degree of saturation by 0.002 while cars & taxis increase the v/c value by 0.001.

Table 4.27: Summary of statistical relationship of vehicle compositions and performance parameters for Arategna intersection approaches

Approaches	***Dependent Variables***	***Equation***	***Description***
Fadis	V/C	$Y1 = -0.3782 + 0.0019X1 + 0.001X2 + 0.0022X3 + 0.0028X4 + 0.0161X5 + 0.0096X7 + 0.0261X9$	▪ R^2 = 0.9302 ▪ RMSE (Training) = 0.098 ▪ RMSE (Test) = 0.00

Approaches	***Dependent Variables***	***Equation***	***Description***
Fadis	Q_{95} (veh)	$Y2 = -23.098 + 0.072X1 + 0.11X2 + 0.137X3 + 0.04X4 + 0.299X5 + 0.033X6 + 0.988X7 + 0.014X8 + 0.782X9$	▪ R^2 = 0.9173 ▪ RMSE (Training) = 56.369 ▪ RMSE (Test) = 6.638
	D (s/veh)	$Y3 = -108.4 + 0.3485X1 + 0.496X2 + 0.4846X4 + 1.565X5 + 0.4675X7 + 4.8534X9$	▪ R^2 = 0.8466 ▪ RMSE (Training) = 11.043 ▪ RMSE (Test) = 8.976
NOC	V/C	$Y1 = -0.422 + 0.001X1 + 0.003X2 + 0.003X3 + 0.002X4 + 0.003X5 + 0.004X6 + 0.011X7 + 0.003X8 + 0.007X9$	▪ R^2 = 0.9654 ▪ RMSE (Training) = 0.0652 ▪ RMSE (Test) =0.00
	Q_{95} (veh)	$Y2 = -89.04 + 0.0585X1 + 0.236X2 + 0.2048X3 + 0.2129X4 + 0.2952X5 + 0.8939X6 + 0.866X7$	▪ R^2 = 0.9698 ▪ RMSE (Training) = 3.746 ▪ RMSE (Test) =2.50
	D (s/veh)	$Y3 = -452.9 + 0.3448X1 + 1.106X2 + 0.963X3 + 0.892X4 + 0.328X5 + 3.196X6 + 4.308X7$	▪ R^2 = 0.9397 ▪ RMSE (Training) =28.41 ▪ RMSE (Test) =0.00
Warwari	V/C	$Y1 = -0.48 + 0.0012X2 + 0.013X3 + 0.0096X4 + 0.02X7 + 0.03X9$	▪ R^2 = 0.8868 ▪ RMSE (Training) = 0.128 ▪ RMSE (Test) =0.051
	Q_{95} (veh)	$Y2 = -16.02 + 0.1084X2 + 0.273X3 + 0.132X4 + 0.349X7 + 0.459X9$	▪ R^2 = 0.6908 ▪ RMSE (Training) = 4.65 ▪ RMSE (Test) = 3.17
	D (s/veh)	$Y3 = -142.77 + 1.416X2 + 2.242X3 + 1.161X4 + 2.343X7 + 1.54X9$	▪ R^2 = 0.5103 ▪ RMSE (Training) = 28.57 ▪ RMSE (Test) =13.46
		Note: The traffic volume for Warwari approach is very small and the occurrence of some of the vehicles is very rare. Thus the models are for the vehicles in appearance	

Approaches	*Dependent Variables*	*Equation*	*Description*
Shash garage	V/C	$Y1 = -1.14 + 0.0001X1 + 0.009X2 + 0.0025X4 + 0.004X5 + 0.006X6 + 0.007X7 + 0.019X8 + 0.0027X9$	▪ R^2 = 0.9684 ▪ RMSE (Training) = 0.107 ▪ RMSE (Test) = 0.0127
	Q_{95} (veh)	$Y2 = -123.96 + 0.660X2 + 0.130X4 + 0.507X5 + 0.167X6 + 0.585X7 + 1.179X8 + 0.578X9$	▪ R^2 = 0.9742 ▪ RMSE (Training) = 6.05 ▪ RMSE (Test) =3.343
	D (s/veh)	$Y3 = -801.76 + 0.017X1 + \mathbf{4.648X2} + 0.456X4 + 2.429X5 + 2.569X7 + 6.235X8$	▪ R^2 = 0.9453 ▪ RMSE (Training) = 54.716 ▪ RMSE (Test) = 5.65
		*Note: **Cars & taxi (x2)** is highly affecting delay more than bigger vehicles for Shash garage approach. This is because there is loading and unloading of passengers at the end of the approach which affect the vehicles following from behind.*	

From table 4.27, the RMSE for test and training variables showed a large difference for delay model of NOC, Warwari and shash garage approaches. Other approaches have approximately closer RMSE values for test and training variables which indicates the validity of the regression in practice. Equations in the above table for Arategna roundabout can be summarized by table 4.28 below.

Table 4.28: Summary of the effect of different vehicle classes on performance of Arategna intersection

Approach	Parameters	Coefficients									R^2
		Bajaj	Cars &Taxi	4WD	Mini bus	Medium bus	Large bus	Medium truck	Large truck	T/T	
Fadis	V/C	0.002	0.001	0.002	0.003	0.016	---	0.010	----	0.026	0.93
	Q_{95}	0.072	0.11	0.137	0.04	0.299	0.033	0.988	0.014	0.782	0.92
	D	0.349	0.496	---	0.485	1.565	---	0.468	----	4.853	0.85
NOC	V/C	0.001	0.003	0.003	0.002	0.003	0.004	0.011	0.003	0.007	0.97
	Q_{95}	0.059	0.236	0.205	0.213	0.295	0.894	0.866	---	----	0.97
	D	0.345	1.106	0.963	0.892	0.328	3.196	4.308	---	----	0.94
	V/C	---	0.001	0.013	0.01	---	---	0.02	---	0.03	0.89

Approach	Parameters	Coefficients									R^2
		Bajaj	Cars &Taxi	4WD	Mini bus	Medium bus	Large bus	Medium truck	Large truck	T/T	
Warwari	Q_{95}	---	0.108	0.273	0.132	---	---	0.349	---	0.459	0.69
	D	---	1.416	2.243	1.161	---	---	2.343	---	1.54	0.51
Shash G.	V/C	0.001	0.009	---	0.003	0.004	0.006	0.007	0.019	0.003	0.97
	Q_{95}	---	0.66	---	0.13	0.507	0.167	0.585	1.179	0.578	0.97
	D	0.017	4.648	---	0.456	2.429	---	2.569	6.235	---	0.95

From the approaches of Arategna intersection, NOC and Shash garage are more congested than the others. The effects of volume change of vehicles affect these two approaches more. But the composition of the vehicles which affect an approach will not affect the other approach by the same degree. For example, single unit change of bajaj vehicle affects the delay at Sash G. approach by 0.017 seconds while it affects the NOC approach by 0.345 seconds. But minibus vehicles affect Shash G. approach by 0.456 seconds and NOC approach by 0.892 seconds. Cars & taxis affect NOC and Shash G, approach than they affect Fadis and Warwari approach. For other vehicle classes the results are different as indicated in table 4.28.

4.6.2 Statistical Analysis for Sillassie Intersection

The statistical analysis by `stata` software for four approaches of Sillassie intersection is discussed below in table 4.30. Table 4.29 indicates the stata output for the analysis between vehicle compositions and V/C ratio for the selected day of ras mekonnen approach. Similar tables can be developed for other approaches and performance parameters.

Table 4.29: *Descriptive statistics between V/C ratio and vehicle compositions for Ras Mekonnen approach*

```
      Source |       SS       df       MS              Number of obs =      24
-------------+------------------------------           F(  7,    16) =  217.44
       Model |  .426578299     7  .060939757           Prob > F      =  0.0000
    Residual |  .004484154    16   .00028026           R-squared     =  0.9896
-------------+------------------------------           Adj R-squared =  0.9850
       Total |  .431062452    23  .018741846           Root MSE      =  .01674

------------------------------------------------------------------------------
          vc |      Coef.   Std. Err.      t    P>|t|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
      bajajs |    .000626   .0000731     8.56   0.000      .000471     .000781
          wd |   .0020638   .0002708     7.62   0.000     .0014896    .0026379
     minibus |   .0017807   .0001823     9.77   0.000     .0013942    .0021672
   mediumbus |   .0037923     .00127     2.99   0.009     .0011001    .0064845
    largebus |   .0087147   .0030706     2.84   0.012     .0022053     .015224
 mediumtruck |    .003632   .0008995     4.04   0.001     .0017252    .0055387
  largetruck |    .004446   .0015799     2.81   0.012     .0010967    .0077953
          tt |  (omitted)
       _cons |  -.1780052   .0471212    -3.78   0.002    -.2778977   -.0781126
```

Table 4.30: Summary of statistical relationship of vehicle compositions and performance parameters for Sillassie intersection approaches

Approaches	***Dependent Variables***	***Equation***	***Statistical description***
Ras mekonnen	V/C	$Y1 = -0.178 + 0.0006X1 + 0.0021X3 + 0.0018X4 + 0.0038X5 + 0.0087X6 + 0.0036X7 + 0.0044X8$	▪ R^2 = 0.9896 ▪ RMSE (Training) = 0.019 ▪ RMSE (Test) = 0.0035
	Q_{95} (veh)	$Y2 = -37.5 + 0.027X1 + 0.146X3 + 0.087X4 + 0.182X5 + 0.643X6 + 0.108X7 + 0.125X8$	▪ R^2 = 0.9674 ▪ RMSE (Training) = 1.547 ▪ RMSE (Test) = 0.112
	D (s/veh)	$Y3 = -75.6 + 0.051X1 + 0.338X3 + 0.181X4 + 0.189X5 + 0.971X6 + 0.109X7 + 0.264X8$	▪ R^2 = 0.9196 ▪ RMSE (Training) = 4.81 ▪ RMSE (Test) = 3.794
	V/C	$Y1 = -0.419 + 0.002X1 + 0.002X2 + 0.003X3 + 0.002X4 + 0.003X5 + 0.011X6 + 0.006X7 + 0.005X8$	▪ R^2 = 0.9196 ▪ RMSE (Training) = 0.028 ▪ RMSE (Test) = 0.00

Approaches	*Dependent Variables*	*Equation*	*Statistical description*
Bote	Q_{95} (veh)	$Y2 = -15.95 + 0.047X1 + 0.049X2 + 0.037X3 + 0.002X4 + 0.042X5 + 0.202X6 + 0.082X8$	▪ $R^2 = 0.9557$ ▪ RMSE (Training) = 0.953 ▪ RMSE (Test) = 0.00
	D (s/veh)	$Y3 = -54.27 + 0.166X1 + 0.103X2 + 0.212X3 + 0.088X5 + 0.718X6 + 0.074X8$	▪ $R^2 = 0.9354$ ▪ RMSE (Training) = 4.20 ▪ RMSE (Test) = 0.00
Shewa barr	V/C	$Y1 = -0.154 + 0.001X1 + 0.0003X2 + 0.0095X3 + 0.0008X4 + 0.0046X5 + 0.0065X6 + 0.011X8$	▪ $R^2 = 0.9429$ ▪ RMSE (Training) = 0.042 ▪ RMSE (Test) = 0.016
	Q_{95} (veh)	$Y2 = -16.16 + 0.045X1 + 0.0034X2 + 0.221X3 + 0.006X4 + 0.312X6 + 0.381X8$	▪ $R^2 = 0.9334$ ▪ RMSE (Training) = 0.953 ▪ RMSE (Test) = 1.696
	D (s/veh)	$Y3 = -56.3 + 0.149X1 + 0.096X2 + 0.773X3 + 0.590X6 + 1.367X8$	▪ $R^2 = 0.8363$ ▪ RMSE (Training) = 4.094 ▪ RMSE (Test) = 9.12
Andegna menged	V/C	$Y1 = -0.055 + 0.0005X1 + 0.0022X2 + 0.0004X3 + 0.0011X4 + 0.0025X5 + 0.003X6 + 0.0007X7 + 0.007X8$	▪ $R^2 = 0.9538$ ▪ RMSE (Training) = 0.022 ▪ RMSE (Test) = 0.00
	Q_{95} (veh)	$Y2 = -6.23 + 0.01X1 + 0.033X2 + 0.007X3 + 0.01X4 + 0.064X5 + 0.113X6 + 0.008X7 + 0.06X8 + 0.0072X9$	▪ $R^2 = 0.94$ ▪ RMSE (Training) = 0.439 ▪ RMSE (Test) = 0.00
	D (s/veh)	$Y3 = -5.0 + 0.015X1 + 0.049X2 + 0.006X3 + 0.017X4 + 0.058X5 + 0.063X6 + 0.007X7 + 0.208X8$	▪ $R^2 = 0.910$ ▪ RMSE (Training) = 0.747 ▪ RMSE (Test) = 0.320

Table 4.30 indicates the models can be taken as valid in practices for all approaches except for delay model of shewa barr approach based on their RMSE values.

Table 4.31: Summary of the effect of different vehicle classes on performance of Sillassie intersection

Approach	Parameters	Coefficients									R^2
		Bajaj	Cars &Taxi	4WD	Mini bus	Medium bus	Large bus	Medium truck	Large truck	T/T	
Ras Mekonnen	V/C	0.001	---	0.002	0.002	0.004	0.009	0.004	0.004	---	0.99
	Q_{95}	0.027	---	0.146	0.087	0.182	0.643	0.108	0.125	---	0.97
	D	0.051	---	0.338	0.181	0.189	0.971	0.109	0.264	---	0.92
Bote	V/C	0.002	0.002	0.003	0.002	0.003	0.011	0.006	0.005	---	0.92
	Q_{95}	0.047	0.049	0.037	0.002	0.042	0.202	---	0.082	---	0.96
	D	0.166	0.103	0.212	---	0.088	0.718	---	0.072	---	0.94
Shewa barr	V/C	0.001	0.001	0.01	0.001	0.005	0.007	---	0.011	---	0.94
	Q_{95}	0.045	0.003	0.221	0.006	---	0.312	---	0.381	---	0.93
	D	0.149	0.096	0.773	---	---	0.59	---	1.367	---	0.84
Andegna menged	V/C	0.001	0.002	0.001	0.001	0.003	0.003	0.001	0.007	---	0.95
	Q_{95}	0.01	0.033	0.007	0.01	0.064	0.113	0.008	0.06	---	0.94
	D	0.015	0.049	0.006	0.017	0.058	0.063	0.007	0.208	---	0.91

Truck and trailers are not available at Sillassie intersection. They are crossing the intersections at early morning or late evening. Therefore they are not considered in modeling. Vehicles which do not have continuous flows are also difficult to be described by the models. The effects of different vehicle classes on the performances are different from approach to approach as indicated in table 4.31.

4.6.3 Statistical Analysis for Shash Garage Intersection

The statistical analysis by `stata` software for three in need movements of Shash garage intersection is discussed below in table 4.33.

Table 4.32: Relationship between V/C ratio and vehicle compositions for Hajib LT movements

```
      Source |       SS       df       MS              Number of obs =      24
-------------+------------------------------           F(  7,    16) =  124.97
       Model |  .286411451     7  .040915922           Prob > F      =  0.0000
    Residual |   .00523856    16   .00032741           R-squared     =  0.9820
-------------+------------------------------           Adj R-squared =  0.9742
       Total |  .291650011    23  .012680435           Root MSE      =  .01809

------------------------------------------------------------------------------
          vc |      Coef.   Std. Err.      t    P>|t|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
      bajajs |   .0018551   .0002739     6.77   0.000      .0012744    .0024358
    carstaxi |   .0031549   .0008363     3.77   0.002      .0013819    .0049278
          wd |   .0029214   .0008288     3.52   0.003      .0011644    .0046783
     minibus |   .0045194   .0014248     3.17   0.006       .001499    .0075398
   mediumbus |    .016011   .0023455     6.83   0.000      .0110387    .0209832
    largebus |   .0069672   .0046886     1.49   0.157     -.0029722    .0169065
 mediumtruck |   .0105907   .0038333     2.76   0.014      .0024646    .0187168
  largetruck |  (omitted)
          tt |  (omitted)
       _cons |   .0046381   .0093945     0.49   0.628     -.0152774    .0245535
```

Table 4.33: Summary of statistical relationship of vehicle compositions and performance parameters for Shash garage intersection movements

Movement	***Dependent Variables***	***Equation***	***Statistical description***
Hajib LT, V4	V/C	$Y1 = 0.00464 + 0.00185X1 + 0.0032X2 + 0.0029X3 + 0.0045X4 + 0.016X5 + 0.007X6 + 0.0106X7$	▪ $R^2 = 0.9820$ ▪ RMSE (Training) = 0.018 ▪ RMSE (Test) = 0.00
	Q_{95} (veh)	$Y2 = -0.448 + 0.0107X1 + 0.0193X2 + 0.017X3 + 0.0079X4 + 0.854X5 + 0.088X6 + 0.0165X7$	▪ $R^2 = 0.8451$ ▪ RMSE (Training) = 0.283 ▪ RMSE (Test) = 0.00
	D (s/veh)	$Y3 = 9.41 + 0.0173X1 + 0.097X2 + 0.029X3 + 0.148X4 + 0.791X5 + 0.445X6 + 0.426X7$	▪ $R^2 = 0.7206$ ▪ RMSE (Training) = 1.68 ▪ RMSE (Test) = 0.00
Bira LT, V7	V/C	$Y1 = -0.304 + 0.0072X1 + 0.016X2 + 0.0172X3 + 0.0052X4 + 0.0358X5 + 0.083X6 + 0.011X7$	▪ $R^2 = 0.9335$ ▪ RMSE (Training) = 0.103 ▪ RMSE (Test) = 0.0158
	Q_{95} (veh)	$Y2 = -4.237 + 0.051X1 + 0.138X2 + 0.163X3 + 0.12X4 + 0.342X5 + 1.029X6 + 0.148X7$	▪ $R^2 = 0.9195$ ▪ RMSE (Training) = 1.30 ▪ RMSE (Test) = 0.243

Movement	***Dependent Variables***	***Equation***	***Statistical description***
	D (s/veh)	$Y3 = -77.528 + 0.513X1 + 3.657X2 + 3.988X3 + 3.693X4 + 13X5 + \mathbf{22.58X6} + 2.215X7$	▪ $R^2 = 0.8789$ ▪ RMSE (Training) = 27.53 RMSE (Test) = 24.38
		▪ **Note:** For minor approach left turn movements, the presence of heavy vehicle will affect the performance drastically. This is indicated in delay model for large buses.	
Bira RT, V9	V/C	$Y1 = -0.0001 + 0.0006(X1 + X2 + X3) + 0.0016X4 + 0.0072X5 + 0.001X7 + 0.003X8$	▪ $R^2 = 0.99$ ▪ RMSE (Training) = 0.004 ▪ RMSE (Test) = 0.0015
	Q_{95} (veh)	no queue exit	no queue exit
	D (s/veh)	$Y3 = -2.45 + 0.0054X1 + 0.0097X3 + 0.0116X4 + 0.0786X5 + 0.0011X7$	▪ $R^2 = 0.8744$ ▪ RMSE (Training) = 0.109 ▪ RMSE (Test) = 0.098

Table 4.33 above also indicates the validity of the models in practices since the RMSE values are closer to each other.

Table 4.34: Summary of the effect of different vehicle classes on performance of Shash garage intersection

Approach	Parameters	Coefficients									R^2
		Bajaj	Cars &Taxi	4WD	Mini bus	Medium bus	Large bus	Medium truck	Large truck	T/T	
Hajib LT	V/C	0.002	0.003	0.003	0.005	0.016	0.007	0.011	---	---	0.98
	Q_{95}	0.011	0.019	0.017	0.008	0.854	0.088	0.017	---	---	0.85
	D	0.017	0.097	0.029	0.148	0.791	0.445	0.426	---	---	0.72
Bira LT	V/C	0.007	0.016	0.017	0.005	0.036	0.083	0.011	---	---	0.94
	Q_{95}	0.051	0.138	0.163	0.120	0.342	1.029	0.148	---	---	0.92
	D	0.513	3.657	3.988	3.693	13	22.58	2.215	---	---	0.88
Bira RT	V/C	0.001	0.001	0.001	0.002	0.007	0.001	0.003	---	---	0.99
	Q_{95}	---	---	---	---	---	---	---	---	---	---
	D	0.005	---	0.010	0.012	0.079	---	0.001	---	---	0.87

Generally as discussed in the sections above, the effects of vehicle composition on performance measurements for different vehicle categories are different in nature and amount. The coefficients of the variables (vehicle types) in the equation formed indicate the how strongly the vehicle composition and performance measurement parameters are inter-related. The effects of some kind of vehicle are significant for some approaches while it is not significant for the other.

4.7 Comparison of the Performances of Intersections with the Presence of Bajajs and without Bajajs (Bajajs Replaced by Minibus Taxis)

The most common public transport modes used in Harar cities are bajajs, taxis and buses. Most of the bajaj vehicles have a carrying capacity of 3 passengers. Taxis can be in the form of passenger cars (with 4 seats) or in the form of minibuses (12 seats). Twelve seat taxis are the most widely public transport mode used in Addis Ababa in combination with public buses. These taxis are also available in other cities of the country including Harar city. But Bajaj taxis are more common outside Addis Ababa city.

From passenger carrying capacity point of view, one 12 seat minibus taxi can accommodate a passengers accommodated by four bajajs. This will definitely improve the performance of the traffic streams in many aspects. In this section the researcher tried to indicate the performance improvement that will be gained by **replacing** the bajaj vehicles with **12 seat taxis** from the traffic engineering point of view. Cost benefit analysis between the two transport modes is out of the scope of this study.

The comparison is made for the peak hour traffic volumes of intersections under the following assumptions;

- All bajaj vehicles will be replaced by minibus taxis at the ratio of 1:4.
- When bajajs are removed, speed of the traffic streams and lane utilization of vehicles will be improved. These will improve the passenger car equivalents of the vehicles. In the previous analysis the presence of bajaj vehicles and their non lane based discipline characteristics make the speed of the vehicles very slow and PCU very large. But without the presence of bajajs the researcher assumes a 15% improvement of PCU values of each vehicle classes. This assumption is based on the speed observation from the collected traffic flow data when there are a small percentage of bajaj vehicles on the road. An improvement in speed of all vehicle classes is observed which improves the performance of the traffic streams as a whole.
- Bajaj vehicles need a small gap to join the traffic stream. This made the average critical gaps of the intersection approach smaller, as we have seen the previous discussions. But when bajaj is not presented, the gap acceptance of the traffic flows will be increased as larger vehicles requires greater gaps. Thus, without the presence of bajaj, it is assumed that average critical gap of the approaches will be increased by ***one second*** as a whole. This assumption is also tested by observation of the gap acceptance of vehicles when the bajaj volumes are small from the recorded video data.

- The analysis is made with assumptions that the geometric conditions, the pedestrian characteristics and the roadway conditions will not be changed due to the replacement of bajajs with minibus taxis.

The performance measurement parameters with the presence of bajajs are discussed in the previous sections of this report. The analysis when bajajs replaced by minibus taxis is made by adding numbers of minibus taxis that accommodate the passengers previously using bajajs to the existing minibus taxis and making the bajaj volume zero. The passenger car units and gap acceptance units were adjusted according to the above assumptions. The process of determination of entry capacity, average control delay and queue length is the same as that of the analysis with the presence of bajajs discussed in the previous sections above. Tables 4.35 to 4.37 below indicate the performance of the intersections at peak hour when the bajajs are replaced by minibus taxis. Table 4.35 indicates the average values at peak hour for the performance variables of each approach with and without the presence of bajajs. The difference (improvement) is also indicated in the table. From the peak hour comparison we can conclude that there will be a good performance improvement for the whole period of the day in replacing bajajs with minibus taxis.

Table 4.35: Peak hour performance of Arategna intersection when bajajs are replaced by minibus taxis

Approach	Time series	**8:00am - 9:00am**		**Performance Measures**				
		Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
		qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q_{95} (veh)**	**D (s/veh)**	
Fadis	1	387	1410	501	0.77	7	31.5	D
	2	384	1130	623	0.62	4	17.7	C
	3	246	955	714	0.35	2	9.4	A
	4	253	780	818	0.31	1	7.9	A
NOC	1	1282	348	983	1.30	48	159.9	F
	2	931	526	842	1.11	25	85.3	F
	3	855	257	1065	0.80	9	19.5	C
	4	702	273	1050	0.67	5	13.4	B
Warwari	1	323	1197	591	0.55	3	15.9	C
	2	243	1160	609	0.40	2	11.8	B
	3	209	793	810	0.26	1	7.3	A
	4	94	772	823	0.11	0	5.5	A

Approach	Time series	8:00am - 9:00am		Performance Measures				
		Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
		qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
Shash G.	1	967	726	760	1.27	36	151.3	F
	2	1023	531	895	1.14	29	96.6	F
	3	656	445	962	0.68	6	14.8	B
	4	671	370	1024	0.65	5	13.2	B

Table 4.36: Peak hour performance of Sillassie intersection when bajajs are replaced by minibus taxis

Approach	Time series	9:30am - 10:30am		Performance Measures				
		Entry Flow	Circulating Flow	**Entry Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
		qa (vph)	**qc (vph)**	**Qe (vph)**	**V/C**	**Q95 (veh)**	**D (s/veh)**	
Ras Mekonnen	1	881	133	1316	0.67	6	11.4	B
	2	909	149	1300	0.70	6	12.4	B
	3	960	140	1309	0.73	7	13.6	B
	4	872	136	1313	0.66	5	11.3	B
Bote	1	303	860	768	0.39	2	9.7	A
	2	342	923	732	0.47	3	11.5	B
	3	300	946	718	0.42	2	10.6	B
	4	364	847	776	0.47	3	11.0	B
Shewa barr	1	489	626	856	0.57	4	12.5	B
	2	453	652	839	0.54	3	11.9	B
	3	496	692	812	0.61	4	14.2	B
	4	462	630	854	0.54	3	11.8	B
Andegna Menged	1	571	541	1396	0.41	2	6.4	A
	2	567	517	1417	0.40	2	6.2	A
	3	619	532	1404	0.44	2	6.8	A
	4	638	501	1431	0.45	2	6.8	A

Table 4.37: Peak hour performance of Shash garage intersection when bajajs are replaced by minibus taxis

Movements	Time series	2:00pm – 3:00pm		Performance Measures					
		Approach Flow	Conflicting Flow	**Potential Capacity**	**Movement Capacity**	**Degree of Saturation**	**Queue Length**	**Average Delay**	**LOS**
		V (vph)	**Vc (vph)**	**Cp (vph)**	**Cm (vph)**	**V/Cm**	**Q95 (veh)**	**D (s/veh)**	
Hajib LT	1	122	745	452	452	0.27	1	12.2	B
	2	106	753	447	447	0.24	1	11.7	B
	3	89	753	447	447	0.20	1	11.0	B
	4	52	929	364	364	0.14	0	12.2	B
Bira LT	1	93	1384	184	134	0.69	4	75.3	F
	2	82	1278	211	160	0.51	3	46.6	E
	3	104	1262	215	172	0.61	3	51.7	F
	4	167	1290	207	178	0.94	7	104.5	F
Bira RT	1	71	372	1196	1196	0.06	0	3.5	A
	2	97	377	1193	1193	0.08	0	3.7	A
	3	89	376	1193	1193	0.07	0	3.6	A
	4	110	464	1116	1116	0.10	0	4.1	A

Table 4.38: Peak hour comparison of performance parameters with and without the presence of bajajs

		With Bajajs				**Without Bajajs**				**Change (Improvement)**			
Intersection	Approach	Qe (vph)	V/C	Q95 (veh)	D (s/veh)	Qe (vph)	V/C	Q95 (veh)	D (s/veh)	Qe (vph) **(+)**	V/C **(-)**	Q95 (veh) **(-)**	D (s/veh) **(-)**
Arategna	Fadis	642	0.73	9	48.0	664	0.51	3	16.6	**22**	**0.22**	**5**	**31.4**
	NOC	911	1.49	63	249.9	985	0.97	22	69.5	**74**	**0.52**	**42**	**180.4**
	Warwari	539	0.63	7	41.8	708	0.33	2	10.1	**169**	**0.31**	**5**	**31.7**
	Shash G.	779	1.49	52	255.8	910	0.94	19	69.0	**131**	**0.55**	**33**	**186.8**
Sillassie	Ras M.	1257	0.98	20	40.4	1310	0.69	6	12.2	**53**	**0.29**	**14**	**28.2**
	Bote	584	0.82	9	34.0	749	0.44	2	10.7	**165**	**0.39**	**6**	**23.3**
	Shewa B.	692	0.96	14	51.1	840	0.57	4	12.6	**148**	**0.40**	**11**	**38.5**
	Andegna	1247	0.66	5	11.6	1412	0.42	2	6.5	**165**	**0.23**	**3**	**5.0**

		With Bajajs				Without Bajajs				Change (Improvement)			
Intersection	Approach	Qe (vph)	V/C	Q95 (veh)	D (s/veh)	Qe (vph)	V/C	Q95 (veh)	D (s/veh)	Qe (vph) (+)	V/C (-)	Q95 (veh) (-)	D (s/veh) (-)
Shash G.	Hajib LT	380	0.28	1	14.7	428	0.21	1	11.8	**48**	**0.07**	**0**	**2.9**
	Bira LT	151	0.90	7	115.7	161	0.69	4	69.5	**10**	**0.22**	**2**	**46.2**
	Bira RT	1416	0.08	0	3.2	1175	0.08	0	3.7	**-241**	**0.01**	**0**	**-0.5**

Table 4.38 above indicates that when numbers of bajajs are replaced by minibus taxis there is an improvement in the performance of the intersections. For example, for NOC approach of Arategna intersection, the average control delay at peak period with presence of bajajs is 250 sec/veh (4.2 min/veh). But when bajajs are replaced by minibus taxis the average control delay will become 70 sec/veh (1.2min/veh) which shows the decrease of delay by 180 sec or 3 min. The positive for Qe in table 4.32 indicates the increase in capacity and the negative sign for v/c, Q95, and D indicated the decrease of degree of saturation, queue length and average delays respectively. Only bira RT movements of shash garage intersection show the decrease in performance. Right turning is very easy for small vehicles without disturbing the major stream movements and the result for bira RT proves this scenario. Figures 4.24 to 4.27 show the difference in performance parameters under two conditions for the approaches of the intersections under study during peak periods.

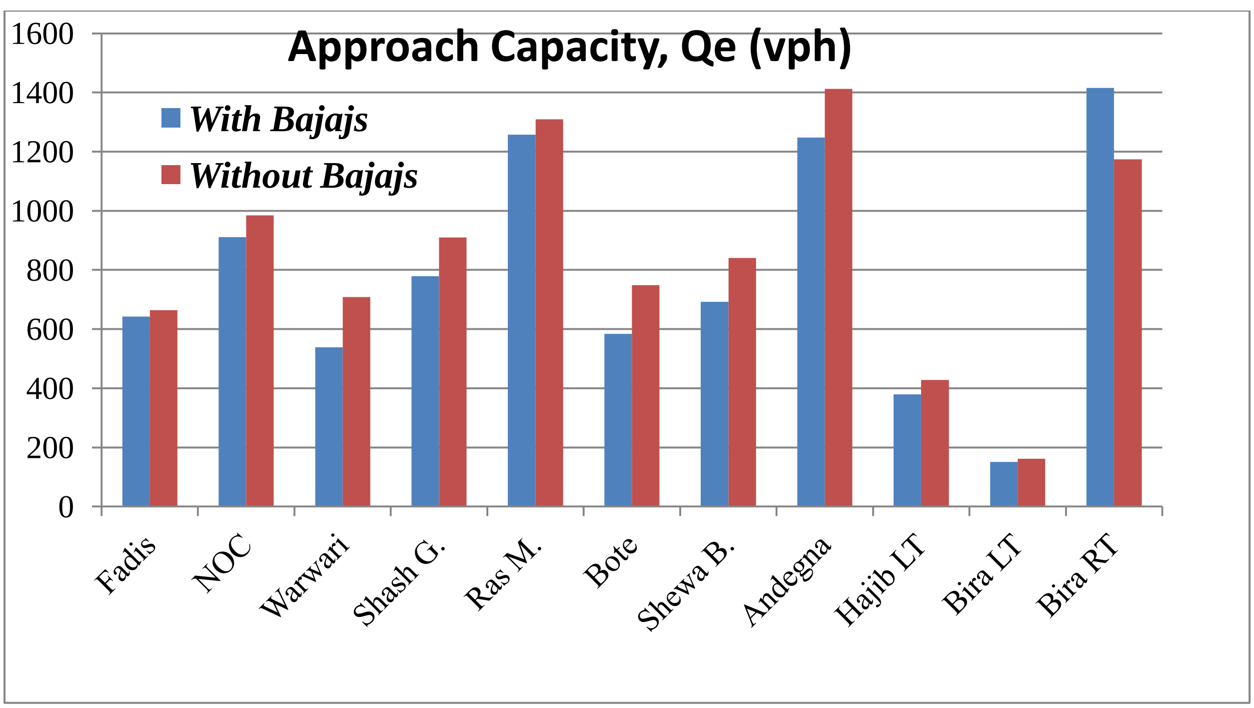

Figure 4.24: Approach capacity with and without Bajajs at peak hour

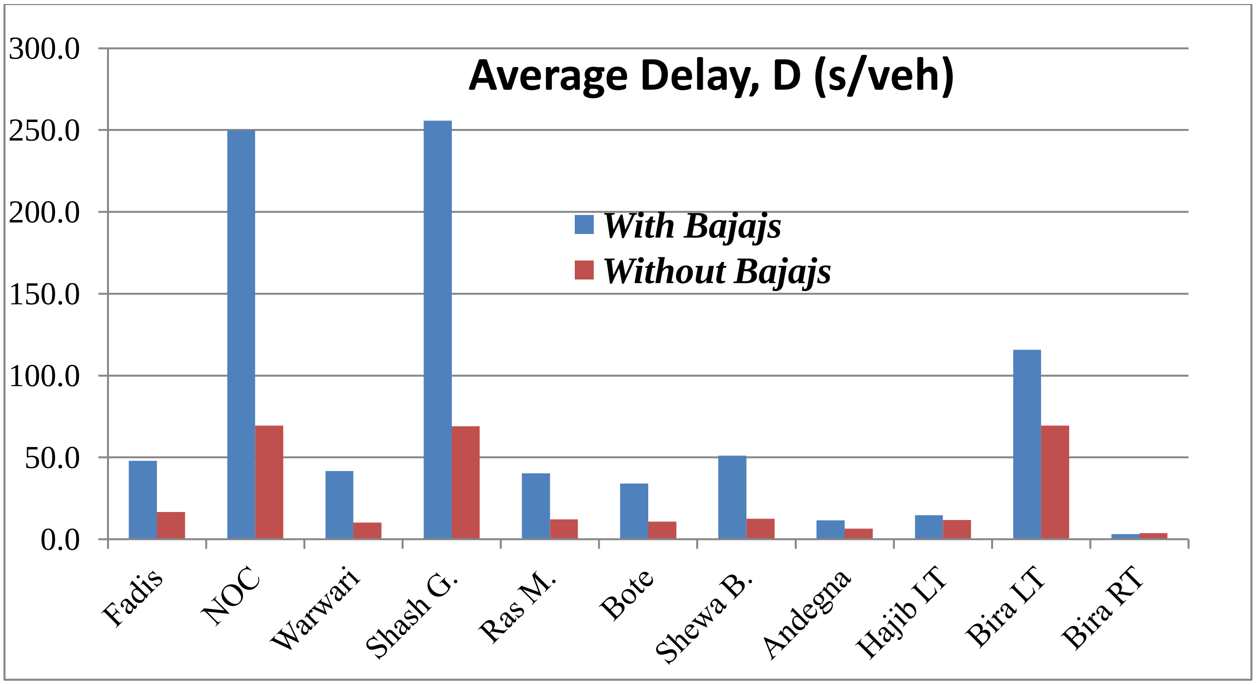

Figure 4.25: Average Delay with and without Bajajs at peak hour

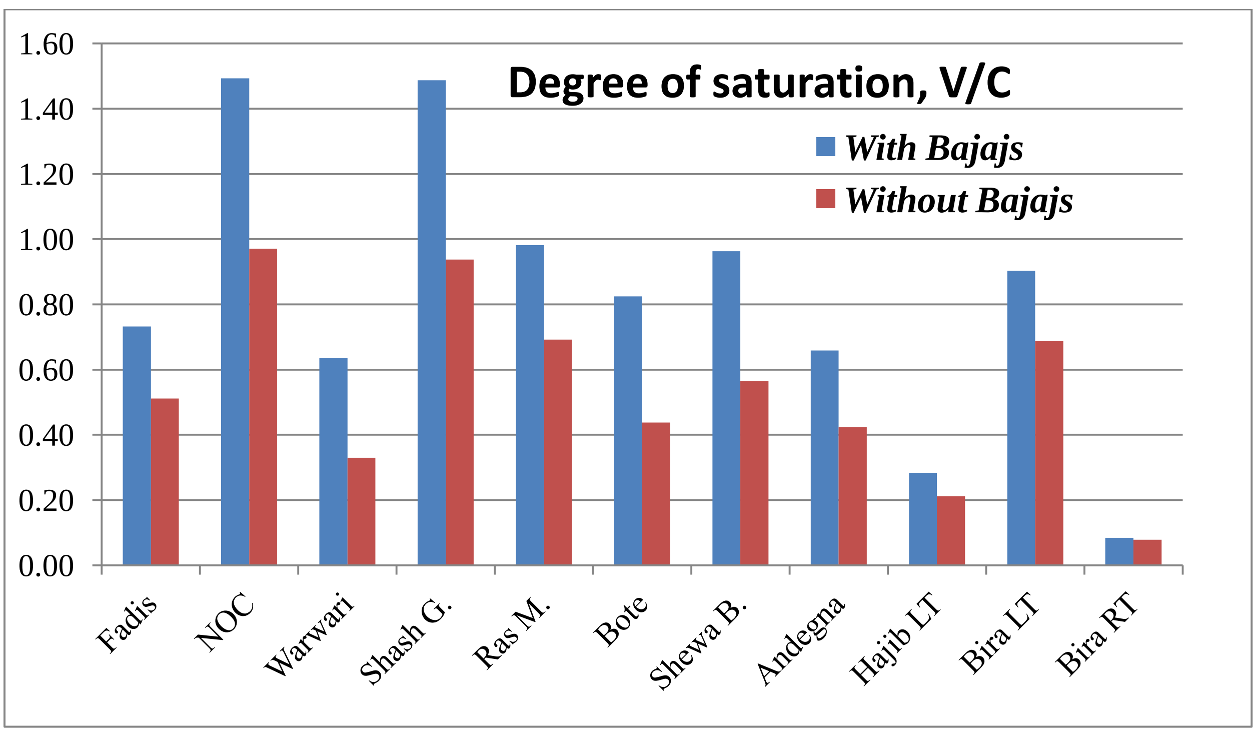

Figure 4.26: Degree of Saturation with and without Bajajs at peak hour

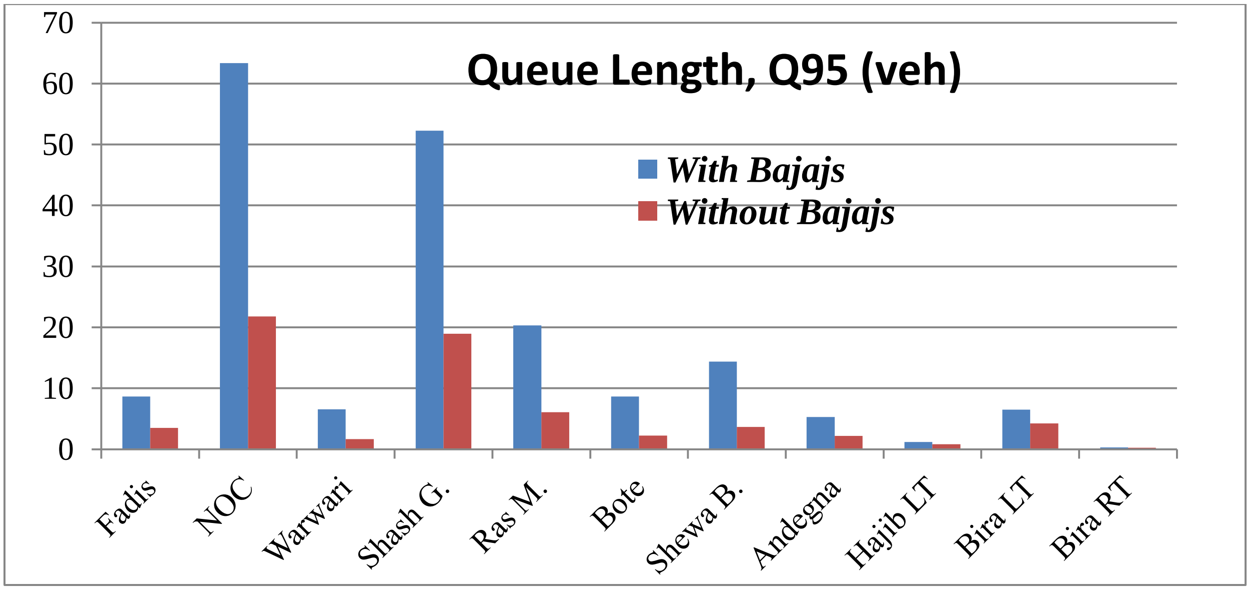

Figure 4.27: Queue Length with and without Bajajs at peak hour

For Arategna roundabout the average delay will be reduced by 7 minutes per vehicle if bajajs are replaced by minibus taxis during peak hour. The queue lengths will be reduced by 85 vehicles and the capacity will be increased by 396veh/hr for peak periods. But replacing bajajs by minibus taxi will not be a sufficient solution to improve the performance of Arategna intersections. After bajajs are removed the approaches of Arategna intersection are still providing service above their capacity. This can be from geometry of the roundabout or other factors affecting the performances of the intersection.

Sillassie roundabout will be improved by decrease of average delay by 95 sec/veh, decrease of queue length by 34 vehicles and increase of capacity by 531veh/hr during peak hour if bajajs are replaced by minibus taxis. There are times when LOS is improved from F to B by this replacement. The movements of Shash garage intersections also indicated a performance improvement by the replacement of bajajs by minibus taxis. But the improvement is very significant for Arategna and Sillassie intersections than for Shash garage intersection.

4.7.1 Reverse Effect of Replacing Bajajs by Minibus Taxis

As it is seen in the previous discussions, from traffic performance point of view, replacing three wheeler taxis by minibus taxis shows a significant improvement of congestion especially around intersections. But, three wheeler taxis offer a number of advantages over other modes. The reverse action related with replacing bajajs by minibuses will be losing the advantages that the smaller cars own.

Bajajs are comparatively cheap and can be easily afforded by a common man. Passengers will have to pay a less transport charge. It is cost effective not only for the consumers but also for the owners. Thus replacing existing bajaj vehicles with minibus taxis will have an effect on economy. An ordinary person will not afford minibus taxis and the passengers are also required to pay somehow greater money than bajaj's. Families that depend on incomes from bajajs may be in a problem when the uses of bajajs are limited due to the above reasons.

Bajajs has a lot greater turning radius than that of other transport modes which makes it more convenient to use in congested areas or busy roads. In remote areas where the roads are narrower or in busy cities where there is considerable traffic jams, these three wheelers are more preferable to use than larger vehicles to some extent. But during peak hours their presence can create a longer queue as they fill any small spaces they got on the road.

On the other hand, if three-wheelers are restricted or taken off the road, the passenger demand that they once catered for is likely to be satisfied in two other ways: by an increasing number of four-wheeled taxis, and by increasing purchases of private vehicles (either cars, motorbikes or motor cycles). Looking first at four-wheeled taxis, these have much higher air pollutant and GHG emissions than do three-wheelers (depending on the size of the four wheeler, as four stroke engines are more efficient than two strokes but only if in a similar sized vehicle), and they take up more area and therefore contribute more to congestion if their number is increased. Thus the replacement has a reverse effect on the traffic performance as numbers of bigger vehicles increased from time to time.

An added benefit of three-wheeler taxis is that they are especially suited to playing a feeder role for the city transportation. They provide service house to house which the other taxis will not provide. Thus, the replacement of bajajs by minibus taxis will have a reverse effect on passengers from feeder roads within the city.

Though the bajajs offer some advantages, the analysis around intersections presented that there will a huge improvement on performances of intersections by limiting the bajaj services and replacing by minibus taxis. But the researcher of this research suggests a more thorough study considering the advantages and disadvantages from both sides of transport modes. This work can be a base for the incoming researches that wanted to consider the cost benefit analysis as this study describes the traffic performance improvements in replacing bajajs by minibus taxis. The scope of this study is limited by investigating the bajaj effect from traffic performance point of view only.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The mix of traffic with different types of vehicle categories has made the analysis of the performance of the intersections more complex. They have different influences based on their size and volume. The presence of bajaj vehicles is decreasing the gap acceptance of the approach since Bajajs are accepting smaller gaps than the gaps accepted by other type of vehicles. They have also smaller passenger car equivalents than other vehicles. The passenger car equivalents are also different from intersection to intersection.

During peak periods, the performances of the intersections selected in this study are very close to the capacity for all approaches. For many approaches the delay is maximum and there are long queues during peak periods and the poor performance will extend for some times beyond peak hour. For Arategna roundabouts, NOC and Shash garage approach have poor performances for a long period of time. This is due to imbalance of traffic flows between these approaches and the other minor approaches. Besides, the geometry of the intersection is not sufficient enough to serve a big flow coming from these two approaches. From Sillassie intersection, Shewa barr and Ras mekonnen approaches showed a LOS F for some period of time intervals. But in general, Sillassie intersection has a better performance than Arategna intersection. Shash garage unsignalized T-intersection has a large delay for minor approach left turn movement. But minor approach right turn movement has an absolutely a best performance for the whole period of time.

The relationship between vehicle types and performance measures indicate that different vehicle type have different effect on degree of saturation, delay and queue length of the intersections. Their effect is dependent on their size and volumes or percentages of composition. If a small vehicle has an effect equal or closer to that of larger vehicles numerically, the effect of that small vehicle is very large when we consider their passenger car equivalents. The models developed for each approaches can significantly show the relationship between the vehicle compositions and the performance measurements.

By replacing Bajaj vehicles by minibus taxis as a means of public transport in Harar city, a performance improvement will be gained under the prevailing conditions. The improvement is significant for Arategna and Sillassie roundabouts. Shash garage intersection has also an improvement by removal of bajaj vehicles from the traffic stream. During peak period, by replacing of bajajs by minibus taxis, Arategna roundabout will have a decrease of average delay by 7min/veh, decrease of queue length by 85 vehicles and increase of capacity by 396 veh/hr. Sillassie roundabout will have a decrease of 95 sec/veh by average delay, decrease of queue length by 34 vehicles and an increase of capacity by 531 veh/hr during peak hour.

5.2 Recommendations

The data collected for the analysis of this study was from three intersections and for single day for each intersection i.e. Tuesday (Oct 31, 2017), Wednesday (Nov 01, 2017), and Thursday (Nov 02, 2017) for Arategna, Shash garage and Sillassie intersections respectively. To accurately estimate the relationship a more number of unsignalized intersections can be considered and the data also can be collected for longer time periods including weekends and weekdays. On the other hand, the model only takes into consideration the vehicle composition as explanatory variable for performance measurement, whereas the driver behavior characteristics, pedestrian characteristics, and geometry of the intersection can also decide the performance of a given intersection. Thus a revised model could be developed using gap acceptance concept along with the geometric elements to develop a robust model for Harar scenario and other cities of the country. If the effects of geometry and pedestrian are included in the analysis of all intersections, the accuracy of the models can be improved and can be representative for the city.

The study was conducted only for the unsignalized intersections, thus a study could be done for the performance evaluation for the signalized intersections for various compositions of vehicle groups. The effect of mix of bajaj with other vehicles in allocating signal timing for signalized intersection can be studied further. The model developed evaluated the performance of roundabout for individual approach only. Thus, another area of study could be to develop a model to compute total capacity of roundabouts based on origin destination survey. The assumptions made in replacing bajaj vehicles by minibus taxis shall be studied thoroughly and the cost benefit analysis shall be done in replacing the two vehicle classes.

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Appendices

Appendix 1: Gap Acceptance Data and Critical Gaps for Approaches

Table 1: Cumulative gap acceptance and rejection for Fadis approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	0	0	0	0	50
1-1.9	1.5	4	1	3	1	47
2-2.9	2.5	29	3	26	4	21
3-3.9	3.5	19	5	14	9	7
4-4.9	4.5	10	5	5	14	2
5-5.9	5.5	6	5	1	19	1
6-6.9	6.5	5	4	1	23	0
7-7.9	7.5	2	2	0	25	0
8-8.9	8.5	1	1	0	26	0
9-9.9	9.5	0	0	0	26	0
10-11	10.5	0	0	0	26	0
11-12	11.5	0	0	0	26	0
>12	12	0	0	0	26	0

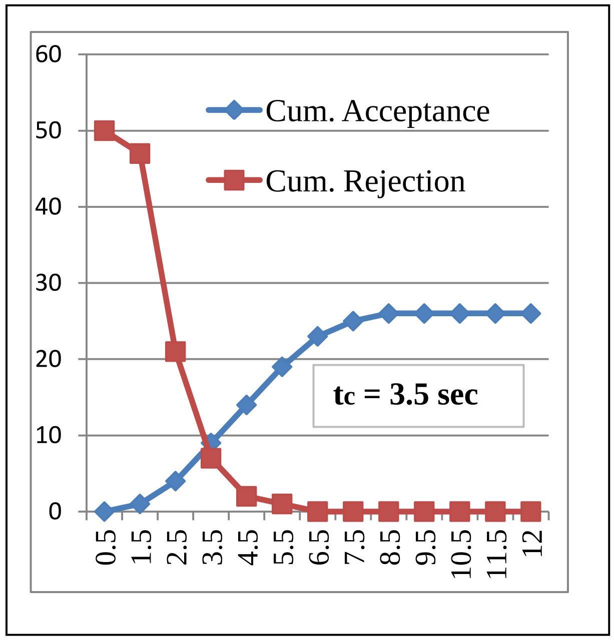

Table 2: Cumulative gap acceptance and rejection for NOC approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	0	0	0	0	28
1-1.9	1.5	1	0	1	0	27
2-2.9	2.5	6	0	6	0	21
3-3.9	3.5	9	5	4	5	17
4-4.9	4.5	12	4	8	9	9
5-5.9	5.5	10	4	6	13	3
6-6.9	6.5	8	8	0	21	3
7-7.9	7.5	3	3	0	24	3
8-8.9	8.5	4	1	3	25	0
9-9.9	9.5	1	1	0	26	0
10-10.9	10.5	1	1	0	27	0
11-11.9	11.5	1	1	0	28	0
>12	12	21	21	0	49	0

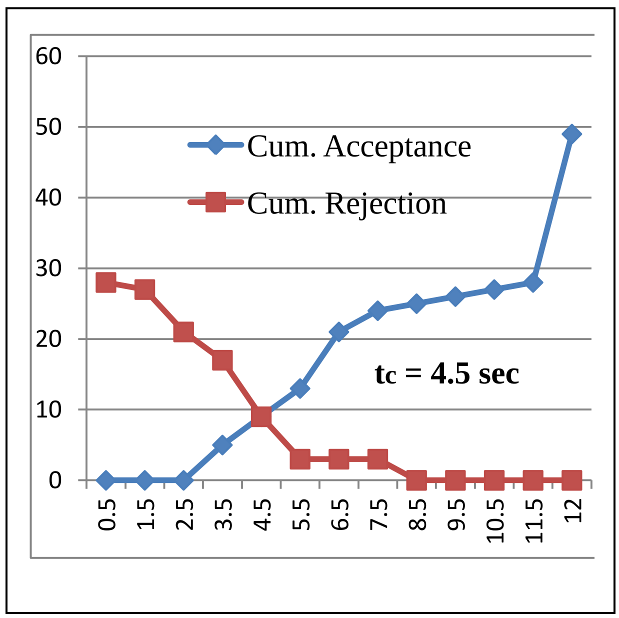

Table 3: Cumulative gap acceptance and rejection for Warwari approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	0	0	0	0	51
1-1.9	1.5	11	0	11	0	40
2-2.9	2.5	22	3	19	3	21
3-3.9	3.5	13	4	9	7	12
4-4.9	4.5	9	5	4	12	8
5-5.9	5.5	11	7	4	19	4
6-6.9	6.5	6	4	2	23	2
7-7.9	7.5	3	2	1	25	1
8-8.9	8.5	0	0	0	25	1
9-9.9	9.5	2	1	1	26	0
10-10.9	10.5	0	0	0	26	0
11-11.9	11.5	0	0	0	26	0
>12	12	0	0	0	26	0

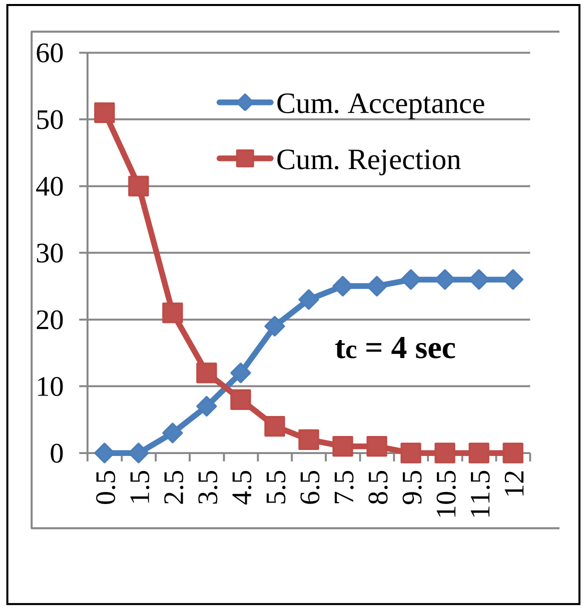

Table 4: Cumulative gap acceptance and rejection for Shash garage approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	0	0	0	0	34
1-1.9	1.5	5	0	5	0	29
2-2.9	2.5	6	0	6	0	23
3-3.9	3.5	18	3	15	3	8
4-4.9	4.5	8	6	2	9	6
5-5.9	5.5	6	3	3	12	3
6-6.9	6.5	8	7	1	19	2
7-7.9	7.5	8	7	1	26	1
8-8.9	8.5	2	2	0	28	1
9-9.9	9.5	0	0	0	28	1
10-10.9	10.5	0	0	0	28	1
11-11.9	11.5	11	10	1	38	0
>12	12	4	4	0	42	0

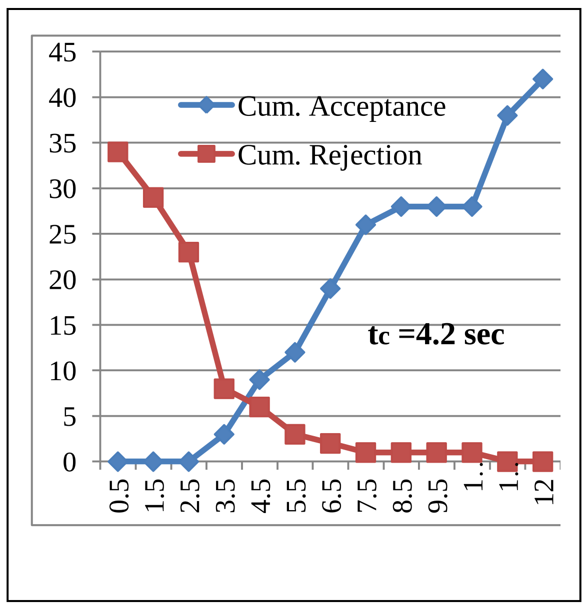

Table 5: Cumulative gap acceptance and rejection for Ras mekonnen approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	0	0	0	0	17
1-1.9	1.5	0	0	0	0	17
2-2.9	2.5	6	0	6	0	11
3-3.9	3.5	7	4	3	4	8
4-4.9	4.5	7	3	4	7	4
5-5.9	5.5	7	5	2	12	2
6-6.9	6.5	7	5	2	17	0
7-7.9	7.5	1	1	0	18	0
8-8.9	8.5	1	1	0	19	0
9-9.9	9.5	1	1	0	20	0
10-10.9	10.5	3	3	0	23	0
11-11.9	11.5	3	3	0	26	0
>12	12	27	27	0	53	0

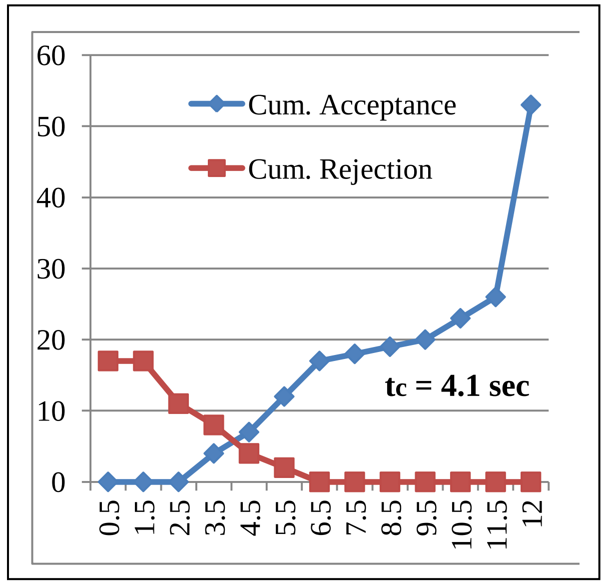

Table 6: Cumulative gap acceptance and rejection for Bote approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	1	0	1	0	28
1-1.9	1.5	6	0	6	0	22
2-2.9	2.5	9	0	9	0	13
3-3.9	3.5	8	4	4	4	9
4-4.9	4.5	11	7	4	11	5
5-5.9	5.5	7	4	3	15	2
6-6.9	6.5	10	8	2	23	0
7-7.9	7.5	3	3	0	26	0
8-8.9	8.5	1	1	0	27	0
9-9.9	9.5	3	3	0	30	0
10-10.9	10.5	2	2	0	32	0
11-11.9	11.5	0	0	0	32	0
>12	12	9	9	0	41	0

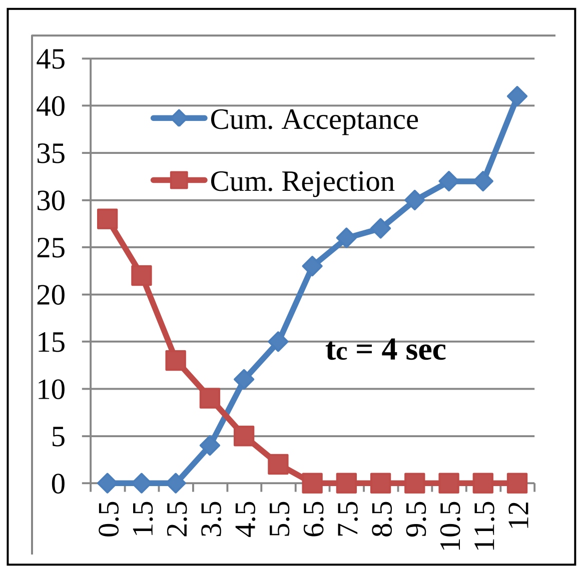

Table 7: Cumulative gap acceptance and rejection for Shewa barr approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	0	0	0	0	36
1-1.9	1.5	4	0	4	0	32
2-2.9	2.5	9	0	9	0	23
3-3.9	3.5	12	3	9	3	14
4-4.9	4.5	16	6	10	9	4
5-5.9	5.5	11	8	3	17	1
6-6.9	6.5	3	3	0	20	1
7-7.9	7.5	2	2	0	22	1
8-8.9	8.5	1	1	0	23	1
9-9.9	9.5	4	3	1	26	0
10-10.9	10.5	2	2	0	28	0
11-11.9	11.5	2	2	0	30	0
>12	12	4	4	0	34	0

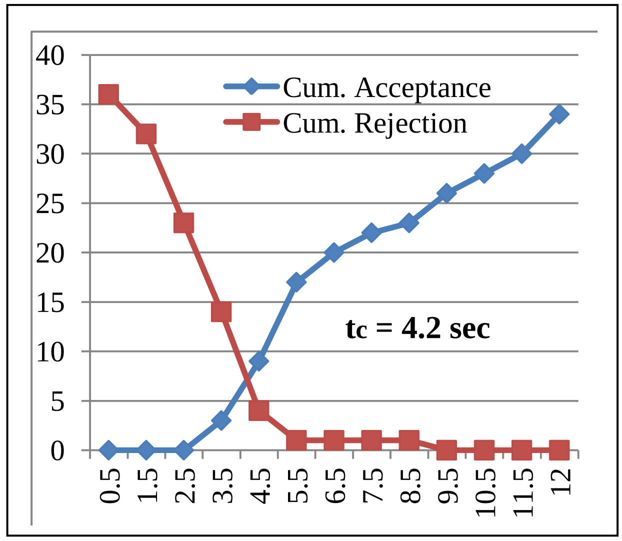

Table 8: Cumulative gap acceptance and rejection for Andegna approach

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	0	0	0	0	26
1-1.9	1.5	2	0	2	0	24
2-2.9	2.5	13	3	10	3	14
3-3.9	3.5	19	9	10	12	4
4-4.9	4.5	8	7	1	19	3
5-5.9	5.5	13	10	3	29	0
6-6.9	6.5	2	2	0	31	0
7-7.9	7.5	3	3	0	34	0
8-8.9	8.5	3	3	0	37	0
9-9.9	9.5	1	1	0	38	0
10-10.9	10.5	2	2	0	40	0
11-11.9	11.5	1	1	0	41	0
>12	12	3	3	0	44	0

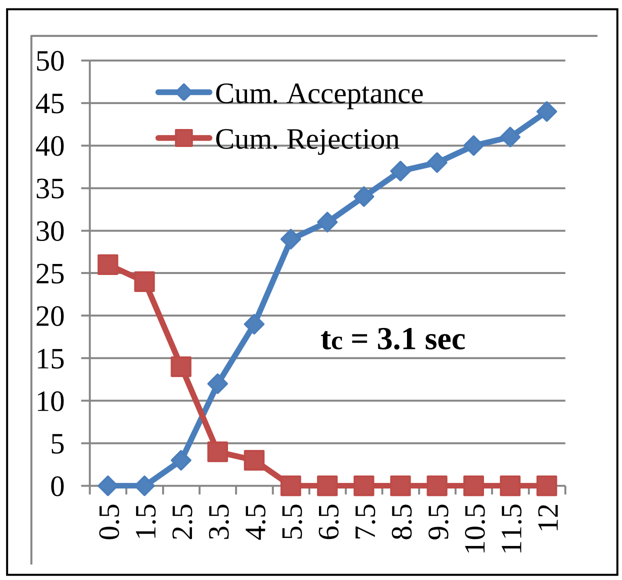

Table 9: Cumulative gap acceptance and rejection for Hajib left turn movements

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	1	0	1	0	43
1-1.9	1.5	8	0	8	0	35
2-2.9	2.5	16	0	16	0	19
3-3.9	3.5	10	0	10	0	9
4-4.9	4.5	3	1	2	1	7
5-5.9	5.5	5	2	3	3	4
6-6.9	6.5	9	5	4	8	0
7-7.9	7.5	4	4	0	12	0
8-8.9	8.5	1	1	0	13	0
9-9.9	9.5	1	1	0	14	0
10-10.9	10.5	1	1	0	15	0
11-11.9	11.5	0	0	0	15	0
>12	12	1	1	0	16	0

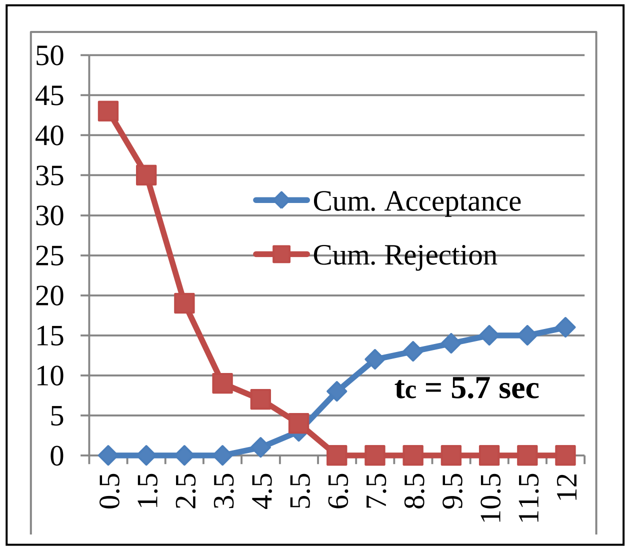

Table 10: Cumulative gap acceptance and rejection for Bira left turn movements

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	1	0	1	0	46
1-1.9	1.5	7	0	7	0	39
2-2.9	2.5	17	0	17	0	22
3-3.9	3.5	7	1	6	1	16
4-4.9	4.5	8	2	6	3	10
5-5.9	5.5	7	3	4	6	6
6-6.9	6.5	4	1	3	7	3
7-7.9	7.5	3	1	2	8	1
8-8.9	8.5	2	1	1	9	0
9-9.9	9.5	1	1	0	10	0
10-10.9	10.5	2	2	0	12	0
11-11.9	11.5	0	0	0	12	0
>12	12	1	1	0	13	0

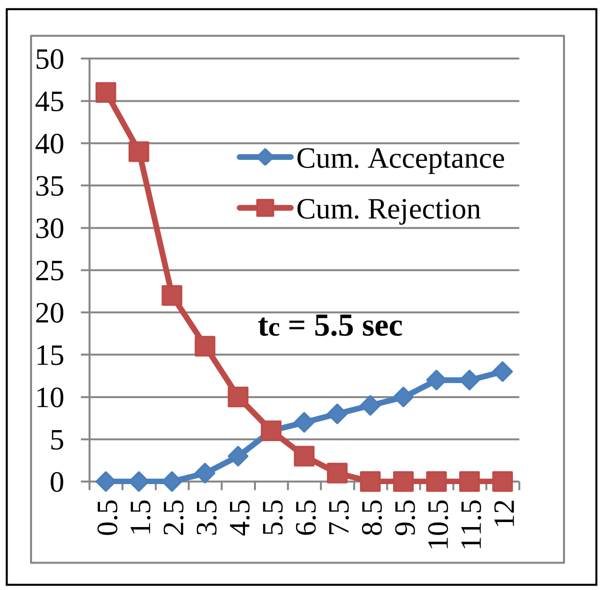

Table 11: Cumulative gap acceptance and rejection for Bira right turn movements

Gap range	Median	#of observation	#of acceptance	#of rejection	Cum. Acceptance	Cum. Rejection
0-0.9	0.5	1	0	1	0	32
1-1.9	1.5	10	0	10	0	22
2-2.9	2.5	14	2	12	2	10
3-3.9	3.5	11	5	6	7	4
4-4.9	4.5	7	5	2	12	2
5-5.9	5.5	6	4	2	16	0
6-6.9	6.5	3	3	0	19	0
7-7.9	7.5	4	4	0	23	0
8-8.9	8.5	3	3	0	26	0
9-9.9	9.5	1	1	0	27	0
10-10.9	10.5	0	0	0	27	0
11-11.9	11.5	0	0	0	27	0
>12	12	0	0	0	27	0