OPERATIONAL ANALYSIS OF UNSIGNALIZED INTERSECTION AND ROUNDABOUT UNDER MIXED TRAFFIC FLOW CONDITION OF SELECTED INTERSECTIONS IN ADAMA CITY

M.Sc. THESIS

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A THESIS SUBMITTED TO THE

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DECLARATION

I hereby certify that this research work titled "Operational Analysis of unsignalized intersection and roundabout under mixed traffic flow condition of Selected intersections in Adama city" is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where material has been used from other sources it has been properly acknowledged/ referred.

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This MSc thesis titled "Operational Analysis of unsignalized intersection and roundabout under mixed traffic flow condition of Selected intersections in Adama city" has been submitted for examination with my approval as thesis advisor.

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "Operational Analysis of unsignalized intersection and roundabout under mixed traffic flow condition of Selected intersections in Adama city" submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Road and Transport Engineering, the Graduate Program of the Department of Civil Engineering, and has been carried out by Darara Berhanu Kuru ID. No GPRoTrR/008/11, under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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We, the undersigned, members of the Board of Examiners of the final open defense by Darara Berhanu Kuru have read and evaluated his/her thesis entitled “Operational Analysis of unsignalized intersection and roundabout under mixed traffic flow condition of Selected intersections in Adama city”, and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of Masters of Science in Road and Transport Engineering.

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SGS Approval	Signature	Date

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LISTS OF ACRONYMS

HCM	*Highway Capacity Manual*
TRB	*Transportation Research Board*
LOS	*Level of service*
Pcph	*Passenger car per hour*
PHF	*Peak-hour factor*
Veh	*Vehicle*
FHWA	*Federal Highway Agency*
AACRA	*Addis Ababa City Roads Authority*
CBD	*Central Business District*
SIDRA	*Signalized (un-Signalized) Intersection Design and Research Aid*
TDM	*Travel Demand Management*
PCU	*Passenger Car Unit*
TRRL	*Transport Research Record Laboratory*
UI	*Unsignalized Intersections.*

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ABSTRACT

Urban traffic congestion is currently severe in most cities in the world due to economic and social development. Prevalently, Ethiopia is one of the countries that are in rapid economic development. This influences the travel pattern of the community from their origin to any destination. Adama is one of busy transportation center in Ethiopia. The city is situated along the road that connects Addis Ababa with Dire Dawa. A large number of trucks use this same route to travel to and from the seaports of Djibouti. A little attention has been paid to junctions and their capacities. There are a few studies conducted to assess the capacity of junctions in Adama. The main objective of this study was to evaluate operational performance on both unsignalized and roundabout junctions which are necessary for transport professional's and governors to put tangible information for future traffic growth of Adama city. This thesis addresses the most important element of operational performance of unsignalized intersections and roundabouts traffic intersections in Adama City on capacity analysis. The methodology employed for this study was the quantitative descriptive research design which used quantitative data and analyzed using SIDRA 8.1 intersection software model. The movements of the vehicles were observed at three roundabouts and seven unsignalized intersections for a duration of two hours. The necessary geometric data for the analysis (average entry width, circulatory road width, number of entry and circulatory lanes, and island diameter), traffic movement data with vehicle characteristics and pedestrian volume were collected. The capacity analysis was done based on the gap acceptance method that is adopted by SIDRA 8.1 software program. Based on analysis results, Derartu Tulu roundabouts and six of selected unsignalized intersections i.e Mebrathayil,Sartera,Wonji,Tikur Abay and Alem Hotel junctions are currently serving in poor condition of level of service of F by having v/c ratio >1. Genda Gara and Medanihalem roundabouts are serving at their best operating condition having level of service A.The rest two unsignalized intersection Geda and Diamond Cafe have level of service of D and C respectively. The overall level of service of the unsignalized intersections and roundabouts are serving at poor level of service. Thus, after carrying out additional detail investigation and taking into consideration future traffic growth appropriate improvement should be made by concerned bodies

Keywords: ***Round about, unsignalized intersections, capacity, Gap acceptance, LOS, Degree of Saturation***

1.INTRODUCTION

1.1 Background

Junctions are usually considered as the critical points within the urban road network and the evaluation of their performance provides valuable understanding and useful indication about the performance of the network. As many studies show, the failure of junction directly reduces the roadway corridor capacity and its improvement can be a very cost-effective means of increasing a corridor's traffic capacity (Rodgers's et.al ,2010).

Traffic conflicts between vehicular movements are created when two or more roads crossed each other. Such conflicts may cause delay and traffic congestion with the possibility of road accidents (HCM,2010). Traffic congestion is one of many serious global problems of both developed and developing countries. It always exerts a negative externality up on the society. It poses severe threat to economy as well as the environment (Najneen et.al, 2010).

Congestion become a common characteristic in urban road transportation system of the cities of developing countries which results in high operation cost, loss of time, high delay, high travel time and increase in fuel consumption (Haregewoin, 2010) Thus, each intersection requires traffic control. It is regulated with stop signs, traffic lights, and roundabout. Evaluation of junction capacity of unsignalized intersection roundabout is very important since it is directly related to delay, level of service, accident, operation cost, and capacities.

Adama city is one of the fastest growing cities in Ethiopia. The city is working to become the second most significant conference destination in Ethiopia after Addis Ababa which losses 5-8milion birr per intersection only for vehicle and fuel cost annually (Taddesse, 2011). Accordingly, traffic congestion is growing extremely in the city. This is due to the increase in traffic volume with high percentage of heavy vehicle together with high pedestrian volume, even though the city administration tries to mitigate the effect

Hence, studies of unsignalized and roundabout junctions are significant because such junctions often control the capacity of the city streets to accommodate traffic. The study is also important for urban planners, traffic engineers and other concerned bodies who aim to mitigate the problems.

1.2 Statement of the problem

Urban traffic congestion is currently severe in most cities in the world due to economic and social development. Prevalently, Ethiopia is one of the countries that are in rapid economic development. This influences the travel pattern of the community from their origin to any destination. Generally, transportation system is affected by economic development that results in increase of vehicles on roads.

Adama is a one of busy transportation center. The city is situated along the road that connects Addis Ababa with Dire Dawa. A large number of trucks use this same route to travel to and from the seaports of Djibouti. This and other phenomenon increase road congestion, especially during peak hours. Traffic congestion is also influenced by road network. In a road network, intersection is a major cause of bottlenecks thus contributing to congestion. However; the problem being recognized by all road users and transport professionals; little attention has been paid to junctions and their capacities.There are no any studies conducted to assess the capacity of unsignalized intersection and roundabout junctions in Adama city.

Hence, assessing capacity and level of services on both unsignalized and roundabout junctions are necessary for transport professional's and governors to put tangible information for future traffic growth of Adama city. Therefore, it is necessary to evaluate the operational analysis and capacity of roundabouts and unsignalized intersection for proper traffic operation and to answer at least the following questions.

1.3 Research Questionaries

- ✓ What is the existing level of service of junctions in Adama city?
- ✓ What parameters affect the capacities of intersections in Adama city?
- ✓ What improvement measures should be taken to mitigate the problems?

1.4 Significance of the study

Traffic congestion and long queues at intersections during peak hour are the major problems in the cities. Growing numbers of road users lead to increasing of traveling times. Evaluation of junction capacity is very important since it is directly related to delay, level of service, accident, operation cost, and environmental issues. For more than three decades modern roundabouts and unsignalized intersections have been used successfully throughout the world as a junction control device. In Adama city there are unsignalized intersections and roundabouts. Since little attention has been paid to the design and capacity evaluation of those junctions, nobody knows their capacities or level of services. Therefore, it is vital to evaluate the capacity of unsignalized intersections and roundabouts for proper traffic operation, which has significance of giving a clear picture for the planners and traffic engineers involved in highway junction design and traffic operation tasks in the city and government authorities can come up with solutions for the traffic congestions, traffic delays, and level of services, accidents and operating costs.

1.5 Objectives

1.5.1 General Objectives

The main objective of this study was to:

- To analyze operational performance of selected unsignalized intersections and roundabouts in Adama city.

1.5.2 Specific Objectives

The specific objectives include

- To evaluate present level of service of the intersections.
- Determination of capacity of selected junctions considering mixed traffic flow.
- To evaluate traffic performance of unsignalized intersections and roundabouts under mixed traffic conditions

1.6 Scope of the study

This research is generalized and focused on operational analysis of selected traffic junctions in Adama city. Therefore, this study focused on determining operational analysis criteria such as capacity, delay and queue to existing level of service selected junctions and recommend future improvement measures to mitigate the problems and also clear picture for the planners and traffic engineers involved in highway junction design.

1.7 Limitation of the study

The primary performance metric used in many countries for intersection operations is the average delay per vehicle, which is calculated as a function of capacity and (demand) volume-to-capacity ratio, with capacity being directly influenced by the gap acceptance and follow-up headway behaviors of drivers. These two parameters should be determined according to the real condition of each sites. However, in this study gap acceptance and follow-up headway and other calibration parameters SIDRA software are taken as default values of different countries.

2. LITERATURE REVIEW

2.1 General

Intersection is an area shared by two or more roads. This area is designated for the vehicles to turn 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 (FHWA,2000). A small error in judgment can cause severe accidents. Thus, each intersection requires traffic control. It is regulated with stop signs, traffic lights, and roundabout. Therefore, both from the accident perspective and the capacity perspective, the study of intersections is very important for the traffic engineers especially in the case of urban scenario (HCM,2010). The common type of at grade intersection is the unsignalized intersection and roundabout, which is used to regulate low volume of traffic flow between the major and minor streets. The two-way stop-controlled (TWSC) and all-way stop-controlled (AWSC) are among the types of operation for unsignalized intersection. Troutbeck and Brilon (1992) stated that unsignalized intersection operates without positive indication or control to the driver. It depends on the driver's decision to take the right opportunity to enter the major street. This behavior is defined as gap acceptance. In this case, the driver in the minor street will wait for an adequate gap before entering the major street. On the other hand, small gaps are typically rejected.

2.2 Unsignalized Intersection

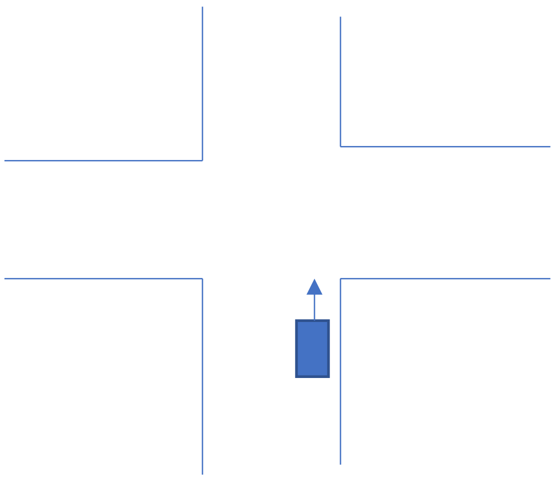
2.2.1 Two-Way Stop- Controlled (TWSC)

Two-way STOP-controlled (TWSC) intersections are common in the United States. One typical configuration is a four-leg intersection, where one street—the major street— is uncontrolled, while the other street—the minor street—is controlled by STOP signs. The other typical configuration is a three-leg intersection, where the single minor-street approach (i.e., the stem of the T configuration) is controlled by a STOP sign. Minor street approaches can be public streets or private driveways. (HCM 2010)

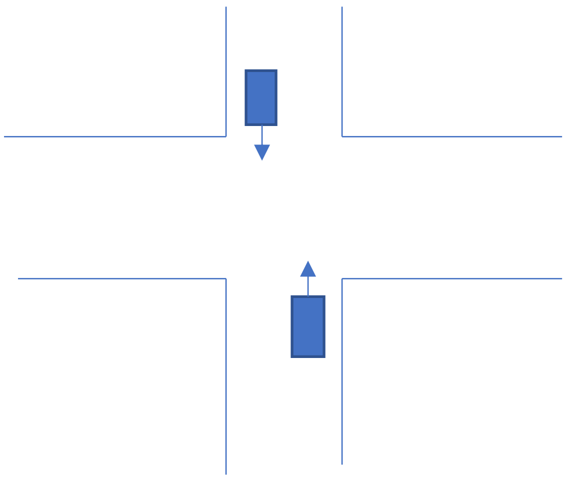
Capacity analysis of TWSC intersections requires a clear description and understanding of the interaction between travelers on the minor, or STOP controlled, approach with travelers on the major street. Both gap acceptance and empirical models have been developed to describe this interaction. (HCM 2010)

2.2.2 All-Way Stop-Controlled Intersections.

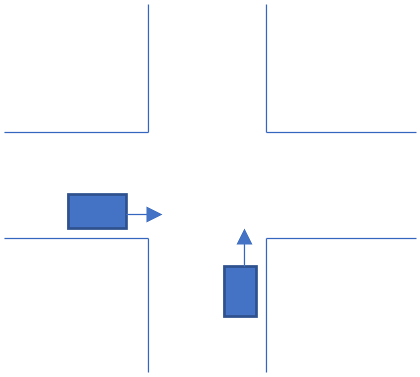
AWSC intersections require every vehicle to stop at the intersection before proceeding. Because each driver must stop, the decision to proceed into the intersection is a function of traffic conditions on the other approaches. If no traffic is present on the other approaches, a driver can proceed immediately after stopping. If there is traffic on one or more of the other approaches, a driver proceeds only after determining that no vehicles are currently in the intersection and that it is the driver's turn to proceed. Field observations indicate that standard four-leg AWSC intersections operate in either a two-phase or a four-phase pattern, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north-south and east-west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach intersection) as shown in Figure 2.1.(HCM,2010)



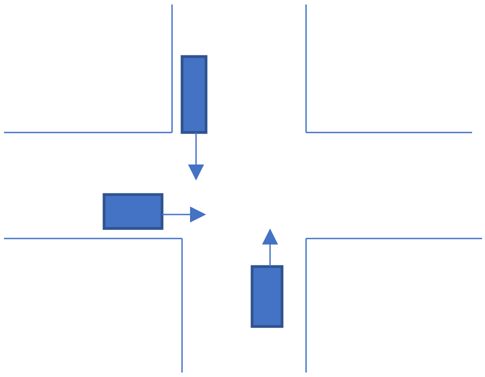
Case 1: Vehicles on subjected approach.



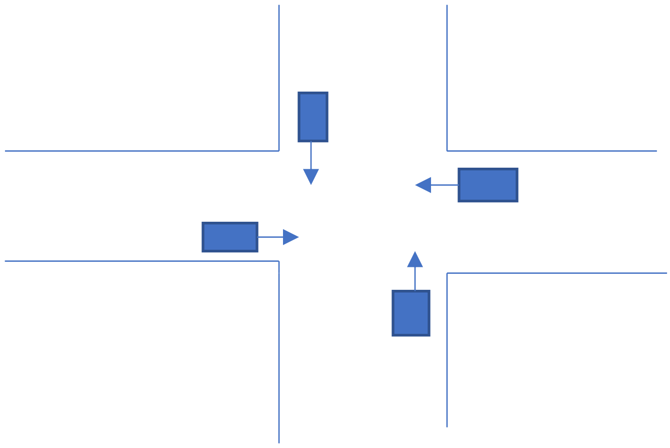
Case 2: Vehicles on subjected approach and Opposite



Case 3: Vehicles on subject and conflicting approaches



Case 4: Vehicles on subject and two other approaches



Case 5: Vehicles on all approaches

Figure 2. 1 Analysis case for AWSC intersection. (HCM,2010)

If traffic is present on the other approaches, as well as on the subject approach, the saturation headway (the time between subsequent vehicle departures) on the subject approach will increase somewhat, depending on the degree of conflict that results between the subject approach vehicles and the vehicles on the other approaches. In Case 2, some uncertainty is introduced with a vehicle on the opposing approach, and thus the saturation headway will be greater than for Case 1. In Case 3, vehicles on one of the conflicting approaches further restrict the departure rate of vehicles on the subject approach, and the saturation headway will be longer than for Case 1or Case 2. In Case 4, two vehicles are waiting on opposing or conflicting approaches, and saturation headways are even longer. When vehicles are present on all approaches, as in Case 5, saturation headways are the longest of any of the cases because the potential for conflict between vehicles is greatest. The increasing degree of potential conflict translates directly into longer driver decision times and longer saturation headways. Because no traffic signal controls the stream movement or allocates the right of-way to each conflicting traffic stream, the rate of departure is controlled by the interactions between the traffic streams. Therefore, the operation at an AWSC intersection can be described numerically by a few key time-based terms:

- The saturation headway, h_{sl}-, is the time between departures of successive vehicles on a given approach for a particular case (case i), as described above, assuming a continuous queue.
- The departure headway, hÿ, is the average time between departures of successive vehicles on a given approach accounting for the probability of each possible case.
- The service time, fs, is the average time spent by a vehicle in first position waiting to depart.

2.3 Typical Conflicts at Unsignalized Intersections

Conflicts at an intersection are different for different types of intersection. Consider a typical four-legged intersection as shown in figure 2.2. The number of conflicts for competing through movements are 4, while competing right turn and through movements are 8. The conflicts between right turn traffics are 4, and between left turn and merging traffic is 4. The conflicts created by pedestrians will be 8 taking into account all the four approaches. Diverging traffic also produces about 4 conflicts. Therefore, a typical four-legged intersection has about 32 different types of conflicts as shown in figure 2.2 below

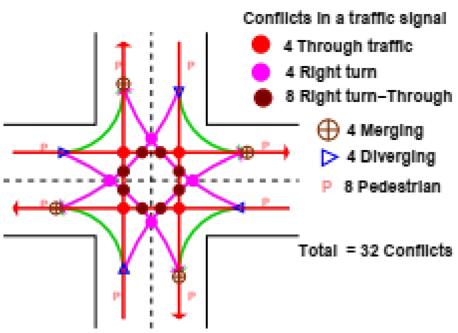


Figure 2. 2 conflicts at f 4-leg intersection (FHWA,2000)

2.4 Level of Service Criteria for TWSC and AWSC Intersections

Level of service (LOS) for a TWSC intersection is determined by the computed or measured control delay. For motor vehicles, LOS is determined for each minor-street movement (or shared movement) as well as major-street left turns by using criteria given figure Table 2.1 LOS is not defined for the intersection as a whole or for major-street approaches for three primary reasons: (a) major-street through vehicles are assumed to experience zero delay; (b) the disproportionate number of major-street through vehicles at a typical TWSC intersection skews the weighted average of all movements, resulting in a very low overall average delay for all vehicles; and (c) the resulting low delay can mask important LOS deficiencies for minor movements. As table 2.1 below notes, LOS F is assigned to the movement if the volume-to-capacity ratio for the movement exceeds 1.0, regardless of the control delay The LOS criteria for TWSC intersections are somewhat different from the criteria used in for signalized intersections, primarily because user perceptions differ among transportation facility types. The expectation is that a signalized intersection is designed to carry higher traffic volumes and will present greater delay than an unsignalized intersection. Unsignalized intersections are also associated with more uncertainty for users, as delays are less predictable than they are at signals, which can reduce users' delay tolerance. (HCM, 2010)

Table 2. 1 LOS Criteria at TWSC and AWSC intersection. (HCM,2010)

Control Delay (s/Vehicle)	**LOS by Volume-to-Capacity Ratio**	
	V/C≤1	**V/C >1**
0-10	A	F
>10-15	B	F
>15-25	C	F
>25-35	D	F
>35-50	E	F
>50	F	F

2.5 Capacity model for AWSC intersections

Throughout the world, all-way stop-controlled (AWSC) intersections are used to control intersecting traffic when the traffic flows are nearly balanced on all approaches, or as an interim transition from stop-control to signalization. AWSC intersections require each driver to stop, judge whether it is safe to enter, and finally, accomplish a turning maneuver. The capacity of all highway facilities depends on how they operate. For AWSC intersections, ‘‘yield to the right’’ is the nominal operating rule, but the actual decision making is more complex. More relevant are the probabilities that specific conflict situations will arise, and the fact that time s ‘‘lost’’ while these conflicts are resolved. In contrast, at signalized intersections, right-of-way is signal controlled, and queuing models can be employed. At two-way stop-controlled (TWSC) intersections, hierarchical relationships allow priority queuing relationships to determine capacity and delays. At roundabouts, a similar situation pertains in that vehicles on the entryways must wait for gaps in the circulating roadway flows before being serviced.

2.5.1 HCM capacity model

Delay

The delay experienced by a motorist is made up of a number of factors that relate to control, geometries, traffic, and incidents. Control delay is the difference between the travel time that is actually experienced and the reference travel time that would result during conditions inthe absence of traffic control or conflicting traffic. Equation 2.1can be used to compute control delay for each lane.

$d = t_s + 900T\left[(x-1)^2 + \sqrt{(x-1)^2} + {}^{hdx}/450T\right] + 5$……………………..…Equation 2.1

where

d= average control delay, sec/veh

x= $\frac{v}{3600}$ h_d= degree of saturation

t_s= service time(s)

hd= departure headway (s)

T= time period, T=1 for hour analysis, T=0.25 for 15 minutes analysis

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time with Equation 2.2

t_s= h_d- m…………………………………………………… Equation 2.2

where ts is the service time, is the departure headway, and m is the move-up time (2.0 s for Geometry Groups 1 through 4; 2.3 s for Geometry Groups 5 and 6)

Queue

According to Highway capacity manual 2010, queue length is computed as the product of the average delay per vehicle and the flow rate for the movement of interest. Equation 2.3 can be used to calculate the 95th percentile queue for each approach lane.

$$Q_{95} = 900T/h_d \left[(X-1) + \sqrt{(1-X)2 + \frac{h_d x}{150T}}\right] \text{.............................Equation 2.3}$$

Where

Q_{95}= Queue length, veh

x= volume to capacity ratio of the subject lane

h_d= departure headway (s)

T= time period, T=1 for hour analysis, T=0.25 for 15 minutes analysis

2.6 Roundabout

2.6.1 General

'Traffic circles have been part of the transportation system in the United States since 1905, when the Columbus Circle designed by William Phelps Eno opened in New York City' (FHWA, 2000). These traffic circles were unlike modern roundabouts today as they gave entering traffic the right of way, thus causing the circulating traffic to give way. This developed numerous problems with roundabouts which involved locking up of traffic around the central Island, aiding high speed entry and the merging and weaving of vehicles leading to severe crashes. After numerous traffic mishaps within these traffic circle intersections in the United States, the Americans decided to abandon the traffic circle designed intersections. However, the British decided to continue to develop and refine the design of these traffic circles and came up with the mandatory give way rule that allowed the development of modern roundabouts to continue to become safe and effective intersections. 'In 1966, the United Kingdom adopted mandatory "give-way" rule at circular intersections, which required entering traffic to give way, or yield, to circulating traffic' (FHWA, 2000). By adopting this rule, roundabouts became free flowing as it did not allow vehicles to enter the roundabout until there was a sufficient gap in the circulating traffic. The differences of modern roundabouts from the traditional traffic circles include (FHWA, 2000):

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

The United States of America (USA) finally adopted the design of the modern roundabout in 1990 in Summerlin, Las Vegas. Since then USA have adopted the modern roundabout and as of December 2009, the number of modern roundabouts in the USA was approximately 2,300(FHWA, 2000). In 1984, the French government adopted the mandatory give- way rule to circulating traffic and as of mid-1997 there are about 15,000 modern roundabouts in France (Jacquemart, 1998). In addition to their popularity in Great Britain and France, roundabouts are very common in Germany, Switzerland, Spain and Portugal. 'Outside of Europe the modern roundabout is a standard feature in Australia and it is becoming more common in New Zealand, South Africa and Israel' (Jacquemart, 1998). According to Federal Highway Administration 2000, roundabout.

2.6.2 Basic Concepts of Roundabouts and definitions

A roundabout is a channelized intersection at which all traffic moves anticlockwise around a central traffic island. (AACRA Geometric Design Manual, 2003). This definition can be used for traffic circles also because it doesn't mention priority. Roundabouts are intersections of two or more roads that are made up of a one way-circulating roadway that has priority over approaching traffic. Yield signs control the approaching traffic and the driver can only make a right turn onto the circulating roadway. The only decision the entering motorist needs to make once they reach the yield line is whether or not a gap in the circulating traffic is large enough for them to enter. The vehicles then exit the circulating roadway by making a right turn toward their destination (FHWA-RD-00-067, 2000). Roundabouts are often confused with traffic circles or rotaries and it is important to be able to distinguish between them. According to FHWA-2000 information guide, roundabouts have five main characteristics that identify them when compared to traffic circles:

- Traffic control: Yield control is used on all entries at roundabouts. The circulatory roadway has no control.
- Priority to circulating vehicles: Circulating vehicles have the right of way in roundabouts. Some traffic circles require circulating traffic to yield to entering traffic.
- Pedestrian access: Pedestrian access is allowed only across the legs of the roundabout, behind the yield line. Some traffic circles allow pedestrian access to the central island.
- Parking: No parking is allowed within the circulatory roadway or at the entries. Some traffic circles allow parking within the circulating roadway.
- Direction of Circulation: All vehicles circulate counter clockwise and pass to the right of the central island of the roundabout. Some neighborhood traffic circles allow left-turning vehicles to pass to the left of the central island. A case in point can be in countries like In United Kingdom, Japan, India, Australia, New Zealand, South Africa, Kenya, Uganda, Tanzania, Zambia, Zimbabwe, and Malawi.

Besides to those five mentioned above, Thaweesak (1998) included additional features of roundabout, which distinguish them from other traffic circles. Approach Flare: Most roundabout approaches flare out at the entries and allow more vehicles to enter the circulating roadway at a more obtuse angle. This improves capacity, and allows entering vehicles to enter at similar speeds as the circulating vehicles unless a queue has developed at the entry. The size and angle of the flare is generally controlled by a raised traffic island that separates the entering and exiting traffic at an approach. This island also gives pedestrians a safe location to cross the approach in two stages. This is the old English principle and gives high capacity, but low safety due to high speed in some countries. Deflection: This characteristic is the geometry of the facility that requires vehicles to slow down as they maneuver through the roundabout. The size of the Center Island and angle of approach determine the deflection and potential speeds of entering and circulating vehicles. Generally, the effect of the roundabout is that traffic is required to slow down to negotiate the curve around the central island, but unlike full stop and signal-controlled intersections, vehicles entering a roundabout are not required to stop completely. This makes the facility more efficient under a wide range of traffic volumes, as motorists only need to find an acceptable gap for entrance.

2.6.3 Types of Roundabouts

Roundabouts are divided into three main types based generally on the size of the facility, as a function of how many vehicles it services, with the three types being mini roundabout, single lane roundabout, and multilane roundabout. (Rodegerdts et al,2010).

2.6.3.1 Mini Roundabout

According to (Robinson et, al,2000), to overcome the safety problems and excessive delays on minor approaches at physically constrained intersections, a mini roundabout can be used as an alternative design, as shown below in Figure 2-3. Mini roundabouts are appropriate in locations where the approaching roadways have an 85th percentile speed less than 30 mph (50km/h), and in some parts of the United States are beginning to be used extensively within subdivisions. Mini roundabouts are discouraged at locations with excessive U-turn traffic.

2.6.3.2 Single Lane Roundabout

At single lane roundabouts, the circulatory roadway should be designed with a minimum radius to accommodate the design vehicle, avoiding larger radii in order to maintain slow speeds in the facility. Moreover, it is common to provide a mountable curve on the interior of the roundabout with a drivable pad that allows heavy vehicles a straighter path through the facility while maintaining the requirement of passenger cars (and equivalent) to be diverted at slower speeds than they would be traveling straight through. Appropriate vehicle turning templates or a CAD-based computer program should be used to determine the swept path of the design vehicle through each of the turning movements. Usually the left turn movement is the critical path for determining circulatory roadway width. In accordance with AASHTO policy, a minimum clearance of 2 ft (0.6 m) should be provided between the outside edge of the vehicle’s tire track and the curb line. AASHTO Table III-19 (1994 edition), shown in Table 2-2, provides derived widths required for various radii for each standard design vehicle. (Robinson et, al,2000)

Table 2. 2 Recommended inscribed circle diameter ranges. (Robinson et, al,2000)

Site Category	Typical Design Vehicle	Inscribed Circle Diameter Range*
Mini-Roundabout	Single-Unit Truck	13–25m (45–80 ft)
Urban Compact	Single-Unit Truck/Bus	25–30m (80–100 ft)
Urban Single Lane	WB-15 (WB-50)	30–40m (100–130 ft)
Urban Double Lane	WB-15 (WB-50)	45–55m (150–180 ft)
Rural Single Lane	WB-20 (WB-67)	35–40m (115–130 ft)
Rural Double Lane	WB-20 (WB-67)	55–60m (180–200 ft)

* Assumes 90-degree angles between entries and no more than four legs.

2.6.3.3 Multiple Lane Roundabout

At multi-lane roundabouts, the width of lanes within the circulatory roadway is usually governed by light trucks and not the design vehicle's overhang while turning, as the design vehicle is assumed to utilize the mountable curb and adjacent area to perform their maneuver. The width required for the necessary number of adjacent lanes to travel simultaneously through the roundabout should be used to establish the circulatory roadway width. The combination of vehicle types to be accommodated side by side is dependent upon the specific traffic conditions at each site. If the entering traffic is predominantly passenger cars and single unit trucks (AASHTO P and SU vehicles), where semi-trailer traffic is infrequent, it may be appropriate to design the width for two passenger vehicles or a passenger car and a single unit truck side-by-side. If semi-trailer traffic is relatively frequent, such as being greater than 10 percent, it may be necessary to provide sufficient width for the simultaneous passage of a semitrailer in combination with a passenger vehicle or single unit truck. (Rodegerdts et al,2010).

2.6.4 Geometric Design of Roundabouts

The first-come-first-serve nature of yield control for vehicles entering the roundabout resolves the question of conflict resolution in the absence of signalization. The primary goal of geometric design at these facilities is then to aid (or force) drivers to slow their speeds down while navigating the facility, with slow speeds being a key component of the safety benefits experienced at these facilities. Other considerations that impact the design of a roundabout are the site constraints, speed of roadways approaching, requirement to pass heavy vehicles, and various geometric attributes unique to each individual site (Rodegerdts et.al ,2010).Three fundamental elements must be determined in the preliminary design stage of a roundabout before defining the details of the geometry: the optimal roundabout size, position, and alignment and arrangement of approach legs [Alphand, and Noelle(1991)]. Some of the basic geometric design elements for a roundabout are shown in Figure 2.3

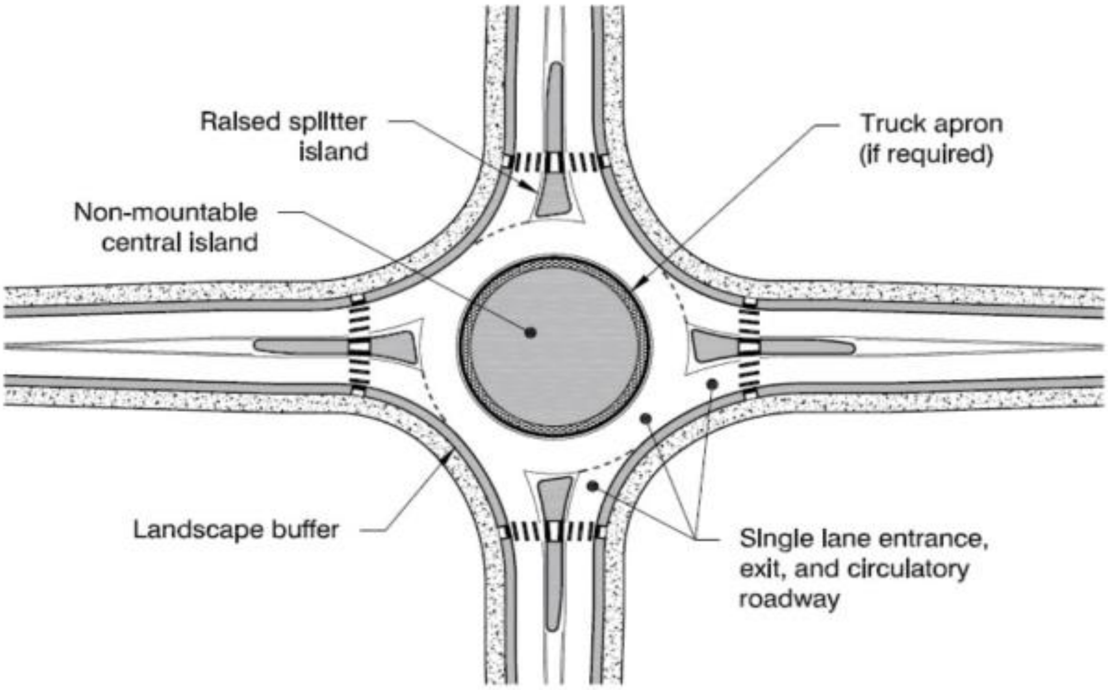


Figure 2. 3 Basic geometric elements of a roundabout. (Alphand and Noelle (1991).

2.6.5 Basic Elements of Roundabout

Good geometric design will improve not only capacity but also safety, which is a major concern for road design. Basic elements for design consideration of roundabouts are shown in table 2.3:

Table 2. 3 Basic elements of roundabouts (HCM,2010).

Approach Curve:	Approach Curve is used to slow down the operating speed of vehicles coming from a high-speed environment.
Entry Curve:	Entry Curve is used to deflect and slow entering vehicles to an appropriate speed to safely circulate the roundabout

Entry Width:	Entry Width is the width of the entry where it meets the circulating Carriageway
Holding Line:	Holding Line is pavement marking that defines where the vehicles have to give-way to the circulating traffic. It is generally marked along the inscribed circle
Circulating Carriageway:	Circulating Carriageway is a curved path used by vehicles to travel around the central Island. This is defined by line marking
Circulating Carriageway Width	Circulating Carriageway Width defines the roadway width for vehicle circulation around central island. Circulating Carriageway Width has to be wide enough to accommodate the largest design vehicles turning path.
Exit Width**:**	Exit Width is the width of the exit where it meets the circulating carriageway.
Exit Curve:	Exit Curve is generally bigger/flatter than the entry curve to allow and flow**.** vehicles to exit at faster speed to improve traffic capacity

2.6.6 Evaluation methods of Roundabout Capacity Evaluation

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

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

- ✓ The empirical method, and
- ✓ The analytical or gap acceptance-based method.

2.6.6.1 Empirical (statistical) model

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

2.6.6.2 Analytical (Gap-acceptance) model

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

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

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

- ➢ The estimation of the critical gap is not easy;
- ➢ The geometric factors are not directly taken into account;
- ➢ The inconsistent gaps are not accounted for in theory (forced right of way when

traffic is congested, circulating drivers give up right of way, different gap accepted by different vehicles, the rejection of large gap before accepting a smaller one, etc.)

2.6.6.2.1 Gap Acceptance Capacity Models.

Roundabout capacity has been estimated using various capacity models developed based on the gap acceptance theory. The gap acceptance method estimates the capacity based on the distribution of headways within the circulating flow, the critical-headway and the follow-up time.

Headway Distribution

The gap acceptance models assume that the headways (i.e. the time between consecutive vehicles passing the conflict point) of the circulating flow follows a certain distribution. The distribution follows an M1 (negative exponential), M2 (shifted negative exponential), or M3 (bunched exponential) (Cowan, 1997). The distributions are expressed as shown in equation 2.4,2.5 and 2.6:

$F(t) = 1 - e^{\lambda t}$ for $t \geq 0$............................Equation 2.4

$F(t) = 1 - e^{-(\lambda t - \Delta)}$ for $t \geq 0$............................Equation 2.5

$F(t) = 1 - \alpha e^{-(\lambda t - \Delta)}$ for $t \geq 0$............................Equation 2.6

Where

F(t) = the cumulative probability that the headway is less than or equal to t

Δ = the minimum headway between the circulating vehicles (sec)

λ = the delay constant (sec)

α =the proportion of free vehicles

The delay constant λ is calculated using the equation no. 2.7 (Cowan, 1997)

$\lambda = q_c\alpha / 1 - q_c\Delta$...Equation 2.7

Where

qc =The circulating flow (pcu/h)

All distributions were developed based on the assumption that the arrival of vehicles follows a Poisson distribution. The M1 distribution is the simplest form but does not headway. The M2 distribution is the M1 distribution with headways shifted by minimum non-zero headway. The M3 distribution has additional assumption of bunching‖ of vehicles within the circulating flow in congested conditions. Trout beck (1994) suggested the proportion of free vehicles at a roundabout is dependent on the circulating flow as given in equation no. 2.8

$$\alpha = 0.75(1 - qc) \text{ .... Equation 2.8}$$

Alternatively, Akçelik (2003) suggested that α can be estimated using equation 2.9 below

$$\alpha = (\frac{\max((1-qc)}{1-(1-kd)\Delta qc} ,0.001) \text{ ....Equation 2.9}$$

Where k_{d} is a constant (2.2 for roundabouts).

The equation no. 2.4,2.5 and 2.6 assume that the proportion of free vehicles decreases as the circulating flow increases due to shorter headways

Critical Headway

Critical headways are estimated using the distributions of gap acceptance and rejection data. Three methods are commonly used for estimating the critical headway:

- ✓ The graphical method
- ✓ The maximum likelihood method
- ✓ The probability equilibrium method

The graphical method determines the critical headway by using cumulative distributions of individual entry vehicles accepted and rejected gaps. A gap is considered accepted if the driver of the entering vehicle perceives that the gap is sufficiently long enough for them to enter the roundabout. Otherwise the gap is rejected. The critical headway is then determined at the point of intersection between the two cumulative distributions curves of the accepted gaps and rejected gaps plotted on the same graph. The critical headway is negatively correlated with higher circulating flow and higher speed of the circulating flow (Xu and Tian, 2008). Also, the critical headway is affected by the waiting time of entrance vehicles (Polus et al., 2003). As waiting time increases, drivers will become more aggressive and will accept shorter gaps. Consequently, this will reduce the critical headway. This could lead to forced entry maneuver, also known as gap forcing. When vehicles accept gaps, which are shorter than the gap required to enter, the speed of the circulating flow will decrease.

2.6.6.2.2 Tanner Capacity Model

The headway distribution functions can be used in conjunction with gap-acceptance parameters to derive the capacity estimation models. These models are macroscopic analytical models which express the capacity in an exponential function of the circulating flow. The exponential function is reasonable because the rate of reduction in capacity generally decreases as the circulating flow increases and capacity never reaches zero. For example, the capacity model adapted in the Highway Capacity Manual (HCM) 2000 (TRB, 2000) assumes that headways follow an M1 distribution and is described as given in equation 2.10.

$$ce = \frac{3600.qc.e^{-qctc}}{1-e^{-qctf}}$$ ……………………………………..Equation 2.10

Where c_e= the entry capacity (pcu/h)

t_c= the critical headway (sec)

t_f= follow-up time (sec)

This capacity model was revised in the HCM 2010 (TRB, 2010) as shown in equation no. 2.11.

$$c_e = \frac{3600}{tf} . e^{\frac{-(tc-0.5tc)qc}{3600}}$$ …………………………………Equation 2.11

The above capacity model is an exponential regression model developed based on a gap acceptance theory (Akçelik, 2011). the HCM 2000 the critical headways were assumed to be different for different roundabout geometry. Geometry is classified in terms of the numbers of circulating lanes and entry lanes. In this model, shorter critical headways were used for a multi-lane roundabout than a one-lane roundabout.

Delay

The below equation 2.12 provides a delay estimation model to be used in determining delay for each approach or critical lane. This model is based on the HCM 2010.This delay model and is consistent with recommendations from NCHRP Report 572. The delay estimates resulting from this model should be used to determine LOS

$$d = \frac{3600}{c} + 900T\left[(x-1)^2 + \sqrt{(x-1)^2} + {}^{3600}{}_{c}x/450T\right] + 5$$ ……………..Equation 2.12

Where

d= average control delay, sec/veh

x= volume to capacity ratio of the subject lane

c= capacity of the subject lane, veh/h

T= time period, T=1 for hour analysis, T=0.25 for 15 minutes analysis

Queuing

Queue lengths should be estimated using below equation for each single-lane approach and for the critical lane on each multilane approach. As shown the below equation will result in the 95th-percentile queue to occur during the peak period. (HCM,2010)

$$Q_{95} = 900T\left[(X-1)+\sqrt{(1-X)2+\frac{\frac{3600x}{c}}{150T}}\right]\left(\frac{C}{3600}\right) \text{ ....Equation 2.13}$$

Where

Q_{95}= Queue length, veh

x= volume to capacity ratio of the subject lane

c= capacity of the subject lane, veh/h

T= time period, T=1 for hour analysis, T=0.25 for 15 minutes analysis

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

Kimber in his initial laboratory report (1980) states that the dependence of entry capacity on circulating flow depends on the roundabout geometry. Kimber defines five geometric parameters which have an effect on the capacity. These are entry width and flare, the inscribed circle diameter (a line that bisects the center island and the circulating lane twice), and the angle and radius of the entry. Similarly, Kimber in his paper (1989) states that gap acceptance is not a good estimator of capacity in the United Kingdom. He further states that, single-lane entries are the basis for the simplest case for gap acceptance models; while, empirical models apply also to multilane entries. Kimber reasons that gap acceptance models do not increase capacity correctly when additional entry lanes are added. Perhaps Kimber's reasoning in this publication was due to its creation date. Many new ideas have been put forth on how additional lanes affect capacity in a gap acceptance model.Kimber makes two interesting comments in his paper (1989), the first being that many circumstances exist where driver response to yield signs conforms to gap acceptance assumptions. However, he questions whether or not gap acceptance is a sufficient description of this interaction. The main flaw of the gap acceptance theory is that it poorly evaluates capacity for at-capacity roundabouts. Flannery et al (1998) comment that congested roundabouts are very scarce in the United States. Therefore, the empirical regression model might be difficult to use since it requires a saturated facility to be calibrated. The second comment by Kimber is that because of driver behavior and geometric variation, it is not safe to transfer theories from one country to another. Fisk, in a 1991 article, agreed that regression models should not be transferred from region to region, or between roundabouts of different geometrical configurations. Fisk writes that because a regression model requires a great deal of data for calibration, it may work well at a specific facility, but cannot be universal. Further, Fisk feels that gap acceptance models demonstrated reliable predictions for both capacity and delay of New Zealand roundabouts. Fisk believed that by changing vehicle class parameters or providing a range of critical gap values, gap acceptance modeling could be used in other locations. Akcelik (ARR 321, 1998) contends that while Kimber objects to the "simple gap acceptance method", the model presented for use in the SIDRA software package goes beyond the simple approach. One main addition to Akcelik's gap acceptance approach is the modeling of the roundabout based on approach lane use. Furthermore, Akcelik writes that the method presented in his report improves capacity prediction during heavy flow conditions and especially for multilane roundabouts with uneven approach demands. Many of the additional elements used in SIDRA are parameters used to enhance its basic gap acceptance theory. The parameters that deal with the entering traffic stream include the in-scribed diameter, average entry lane width, the number of circulating and entry lanes, the entry capacity (based on the circulating flow rate), and the ratio of the entry flow to the circulating flow. These additional model elements demonstrate the detailed nature of the SIDRA model (AACRA also recommend SIDRA for capacity evaluation). Another important component of Akcelik's formulation is the identification of the dominant and sub-dominant entry lanes based on their flows. The dominant lane has the highest flow rate, and all others are subdominants. The purpose of this component is that dominant and sub-dominant entry lanes can have different critical gap and follow up times. The distinction between dominant and sub-dominant lanes appears to be quite important because vehicles using the leftmost entry lane must find a gap in both circulating lanes, as opposed to the right entry lane, which must only deal with traffic in the outer most circulating lane.

SIDRA also includes a passenger car equivalent (pce) factor for heavy vehicles. In this regard, Akcelik (1997) recommended that pce per hour be used in place of vehicles per hour when the proportion of heavy vehicles surpassed 5percent. Many other authors concurred Akcelik's recommendation. In their Roundabout Design Workshop (2001), Ray and Rodegertds explain that heavy vehicles primarily affect roundabout capacity due to their size, not because of their slower acceleration and speed. The U.S. DOT's Roundabout Guide (2000) suggests typical PCE conversion factors for adjusting entering and circulating volumes. These include a 1.5 factor for recreational vehicles and buses, and a 2.0 factor for tractor-trailers. For the purpose of this thesis, the gap acceptance theory appears to be the most appropriate basis for the capacity evaluation. Because the empirical formulation has some drawbacks, for example, data has to be collected at over-saturated flow (or at capacity) level. It is a painstaking task to collect enough data to ensure reliability of results, and this method is sometimes inflexible under unfamiliar circumstances, for example, when the value is far out of the range of regressed data. Consequently, researchers looked for other reliable methods of determining roundabout capacity. Many researchers agree that a gap acceptance theory (Analytical Method) is a more appropriate tool. An advantage of this method is that the gap acceptance technique offers a logical basis for the evaluation of capacity. Secondly, it is easy to appreciate the meaning of the parameters used and to make adjustments for unusual conditions. Moreover, gap acceptance conceptually relates traffic interactions at roundabouts with the availability of gap in the traffic streams (Thaweesak, 1998). Further, investigation into which theory is more appropriate shows that the gap acceptance model is felt to be more transferable from country to country and location to location than is the empirical regression model. List et al (1994) investigated multilane roundabouts in New York State using gap acceptance-based models. They commented that it is possible to transfer capacity equations from overseas.

3. MATERIALS AND METHODS

3.1 Research Approach

The research approach in this thesis involves quantitative data and analysis using SIDRA Intersection software model to determine the level of service of intersections. To do this, primary data were collected directly through field surveys of the selected junctions and sub-sequent analysis of the data. This helps to generate inductive conclusion on the Level of Service (LOS) of considered junctions in Adama City. Since it is impossible to assess the Level of Service of all intersections in the City due to the limitation of budget and time, representative samples were taken at different locations of the City to drive a generalized conclusion. The LOS criterion was according to HCM-2010 and determined using the widely used Sidra Intersection software version 8.1. The flow chart in Figure 3.1 shows the overall activities carried out during the determination of the level of service of the junctions

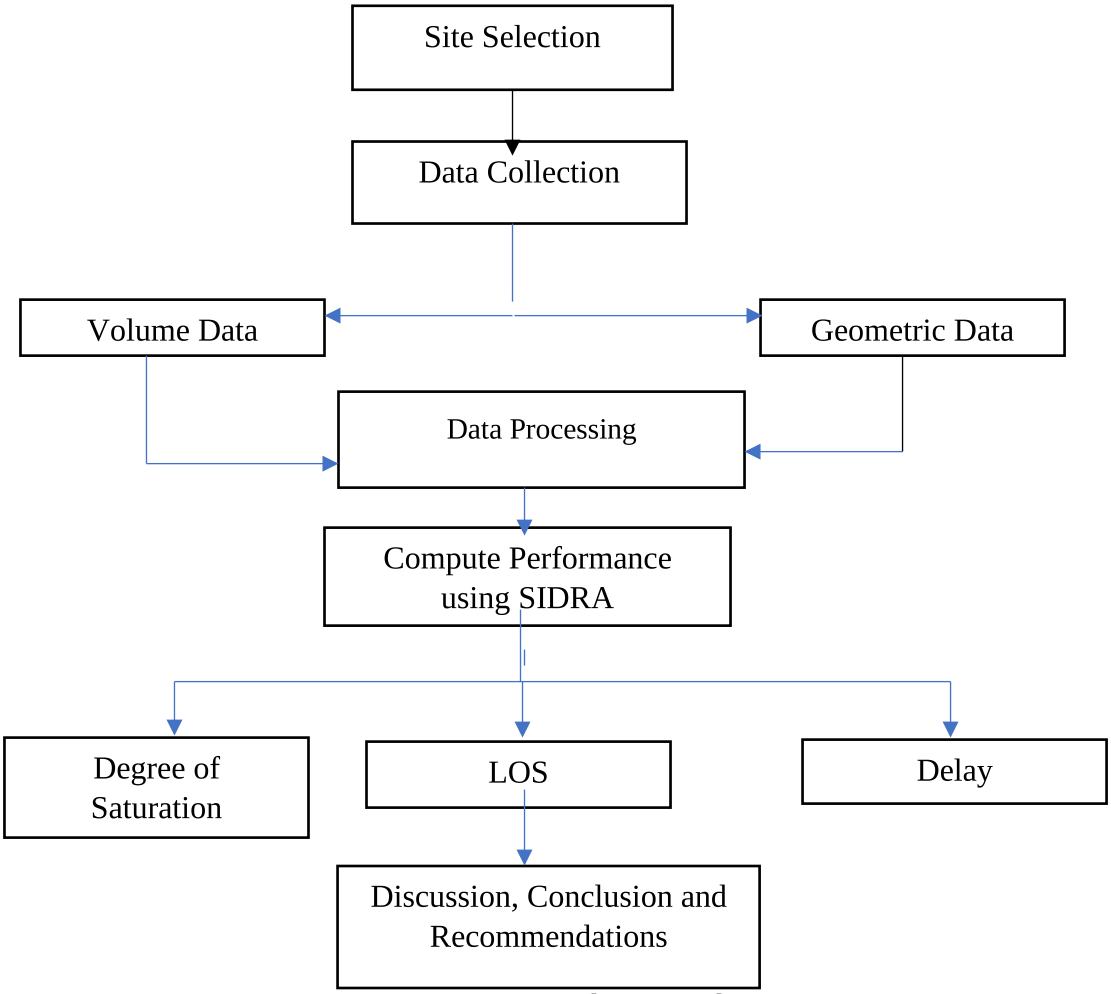


Figure 3. 1 Research approach.

3.2 Calibration of Sidra Intersection Software

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

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

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

Table 3. 1 Default Values used in this research

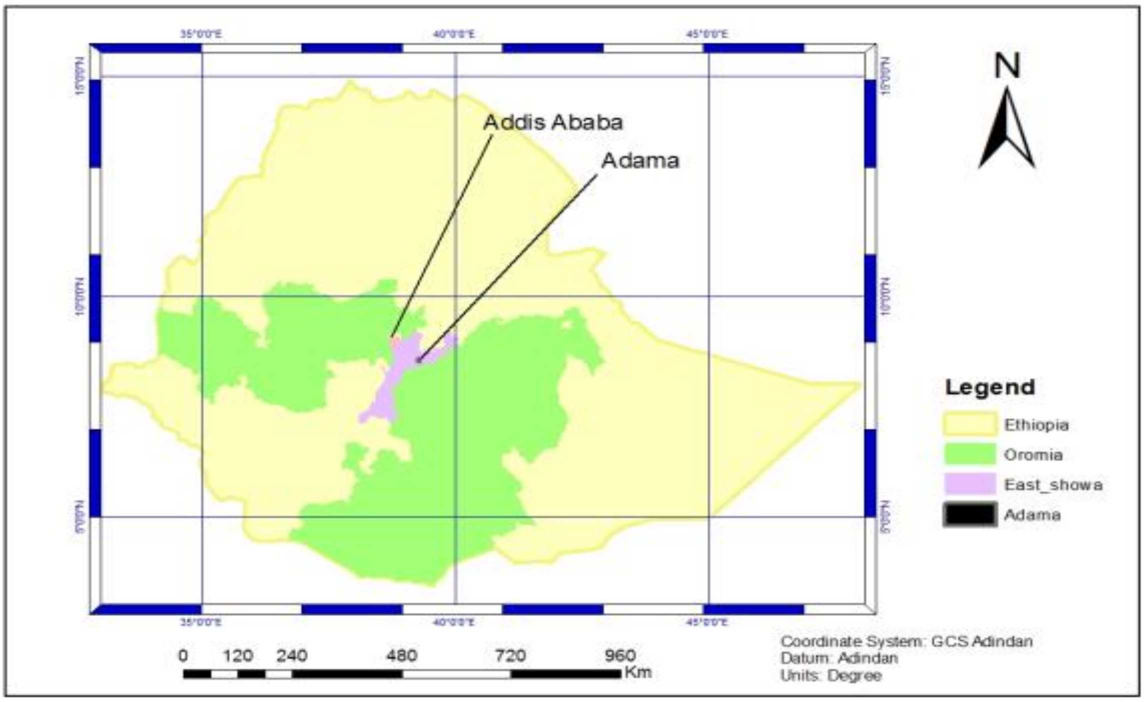
Parameters	Default Values
Critical Gap	4.1 sec
Follow up headway	2.6 sec
Basic Saturation Flow	1950 ve/hr
Lane utilization	100%

3.3 Description of Study Area

Adama is a city in central Oromia Region, Ethiopia. Adama forms a Special Zone of Oromia and is surrounded by East Shewa Zone. It is located at 8.54°N 39.27°E at an elevation of 1712 meters, 99 km southeast of Addis Ababa. The city sits between the base of an escarpment to the west, and the Great Rift Valley to the east. Adama is a busy transportation center. The city is situated along the road that connects Addis Ababa with Dire Dawa. A large number of trucks use this same route to travel to and from the seaports of Djibouti and Asseb (though the latter is not currently used by Ethiopia, following the Eritrean-Ethiopian War). Additionally, the new Addis Ababa-Djibouti Railway runs through Adama.

Adama city is working to become the second most significant conference destination in Ethiopia after Addis Ababa according to the Culture and Tourism Office. The town has several international standard hotels to provide adequate service for any type of conference. The hotels provide services such as swimming pools, internet access, parking lots and meeting halls that make them desirable to tourists and event organizers. If city wants to achieve this goal of being a conference destination focus must be given to assess the cause and evaluate the level of service of intersections and provide the proper mitigation measures.

In order to evaluate the capacity of roundabouts and unsignalized intersections in Adama, three roundabouts and seven unsignalized intersections are selected (as illustrated in Table 3.2). The intersections are selected from major road corridors which represent significant traffic activities in Adama. Thus, the selected intersections for this research connect major road corridors, the ring road and the Central Business District (CBD) of the city road network.



Source: Shape File CSA 2012

Figure 3. 2 Geographical location of Adama city. (Source: Shape file CSA)

3.3.1 Description of Selected Study Intersection

In the city there exist a number of intersections with different type. From these intersections 10 of them has been selected as major intersection on the basis of the importance they give. The intersections are listed in Table 3.2 below.

Table 3. 2 Selected roundabouts and unsignalized intersections

No	Intersection Name	Types of intersections
1	Derartu Tullu	4-Leged Roundabouts
2	Genda Gara	3-Leged Roundabout
3	Kera Condominium	4-Leged Roundabout
4	Mebrat Hayil	4-Leged Intersection
5	Alem Hotel	4-Leged Intersection
6	Tikur Abay	3-Leged Intersection
7	04 Kebele	4-Leged Intersection
8	Geda Resort	4-Leged Intersection
9	Wonji Junction	4-Leged Intersection
10	Sartera	4-Leged Intersection

The chosen intersection names were adopted from the locality or public ally declared names by the Government. The locations of the junctions contribute to high traffic volume and congestion due to the limited flow capacity. Hence, the intersections were selected as the subject of this research study. The Google Map Images of selected intersections are as shown on Appendix-A

3.4 Data Collection

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

3.4.1 Traffic Data

The traffic count surveys were carried out at all selected junctions for four hours; in the morning 7:00 am-9:00 am and in the afternoon 4:00 pm-6:00 pm at 15-minute intervals during working days by setting up video cameras. The peak hour was determined by finding the four consecutive 15-minute periods with the highest total volume. The highest traffic volume in each direction was recorded for use in the analysis of this research. Traffic volumes can vary greatly throughout the day, by day of week, by time of year, and even at

5-minute intervals during the peak hour. Traffic volumes can also experience additional fluctuations due to accidents, special incidents, or weather and will also change over time depending on the growth dynamism of the City. However; normally, traffic surveys are performed using the average weekday peak hour traffic counts. On normal commuter routes, there are morning and afternoon peak hours. High pedestrian volume also has a significant effect on capacity. Because of this numbers of pedestrian were recorded at peak hours along the direction of their movements. The number of counted vehicles and pedestrians are shown in Table 3.3. For detailed information on the vehicles and pedestrian counts, please see Appendix B. The surveys were carried out during the month of Decembere, 2020.

3.5 Video with manual transcription method

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

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

Therefore, video camera was arranged at convenient height where maximum view could be captured and visibility was maximized. Based on the recorded data in video camera, data were manually counted and tabulated as given in table from Table 3.2

Table 3. 3 Hourly Traffic Volumes (PCU) at the Junctions during the time period of survey

Time	Derartu Tulu	Genda Gara	Kera Condomi	Alem Hotel	Tikur Abay	04 kebele	Geda Resort	Wonji	Sartera	Mebrat Hayil
7:00-8:00 AM	3752	1009	987	1158	1478	756	509	1513	1096	1176
8:00-9:00 AM	4823	1145	1047	1063	1356	1234	567	1719	1103	1202
4:00-5:00 PM	3246	1098	1014	921	1524	705	671	1625	1100	1277
5:00-6:00 AM	4685	1118	846	1168	1375	1186	614	1523	989	1164

The traffic counts shown in Table 3.2 help to specify the peak hour period. The peak hour at all intersection was found to be between 5:00-6:00p.m and Figures 3.3 show the peak hour during the time period of survey at each intersection (for detail see appendix –B)

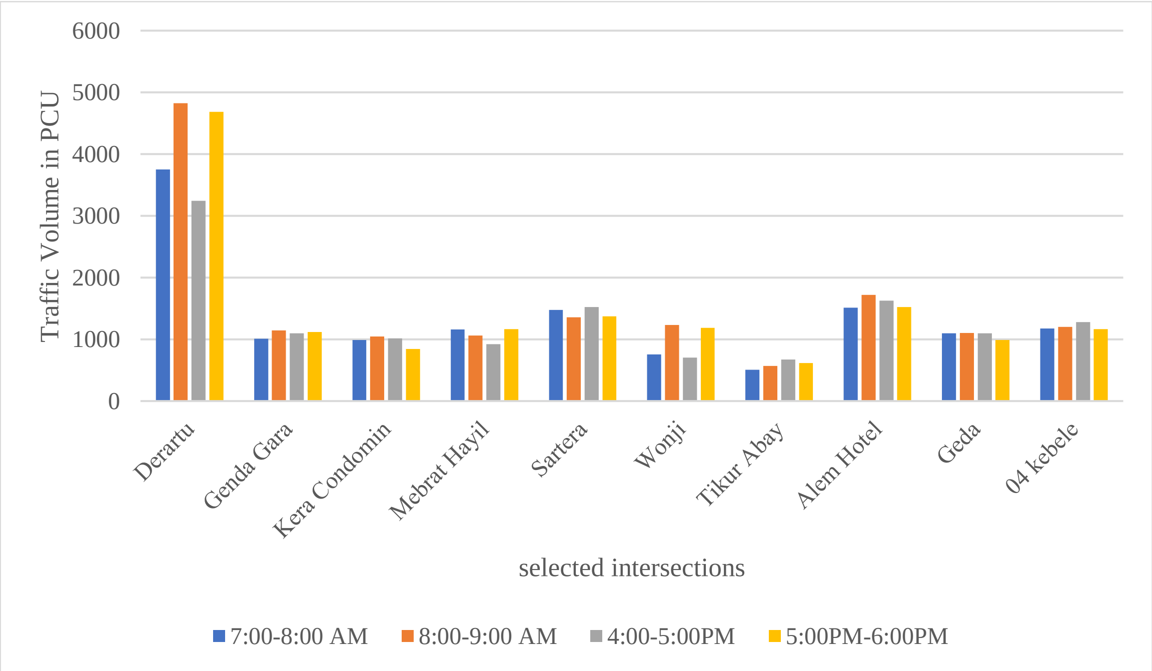
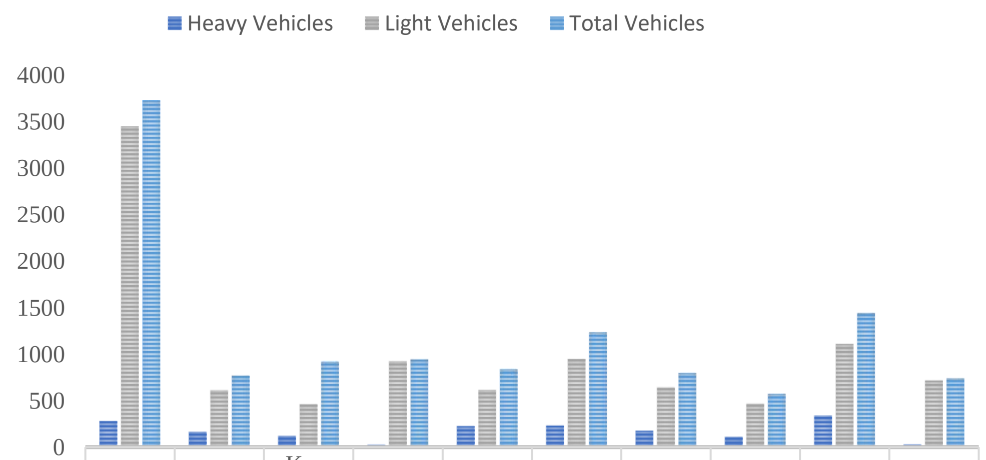


Figure 3. 3 Hourly traffic volumes (PCU) during the period of survey at each intersection

Table 3. 4- Vehicles volume of intersections at peak hour.

S. N	Junction Name	Heavy Vehicles			Light Vehicles	Total no. of vehicles	Total Traffic (PCU)	% of Heavy Vehicles
		Bus & Dump Truck	Truck & Trailer	Total				
1	Derartu Tullu	243	34	277	3452	3732	4823	4.7
2	Genda Gara	130	30	160	608	768	1118	12.5
3	Kera condom	81	37	118	460	917	1047	9
4	Mebrat Hayil	21	-	21	921	941	1277	1.5
5	Alem Hotel	164	58	222	612	834	1164	18
6	Tikur Abay	175	54	229	950	1234	1524	12
7	04 Kebele	145	30	175	639	796	1092	4.8
8	Geda Resort	85	23	108	463	571	671	11.4
9	Wonji	295	43	338	1107	1445	1709	20
10	Sartera	18	6	24	714	738	1103	1.5



	Derartu	Genda Gara	Kera Condo minium	Mebrat Hayil	Sartera	Wonji	Tikur abay	Alem Hotel	Geda	04Kebel e
Heavy Vehicles	277	160	118	21	222	229	175	108	338	24
Light Vehicles	3452	608	460	921	612	950	639	463	1107	714
Total Vehicles	3732	768	917	941	834	1234	796	571	1445	738

Figure 3. 4- Maximum Peak Hour Vehicle Volumes Distribution at Intersections

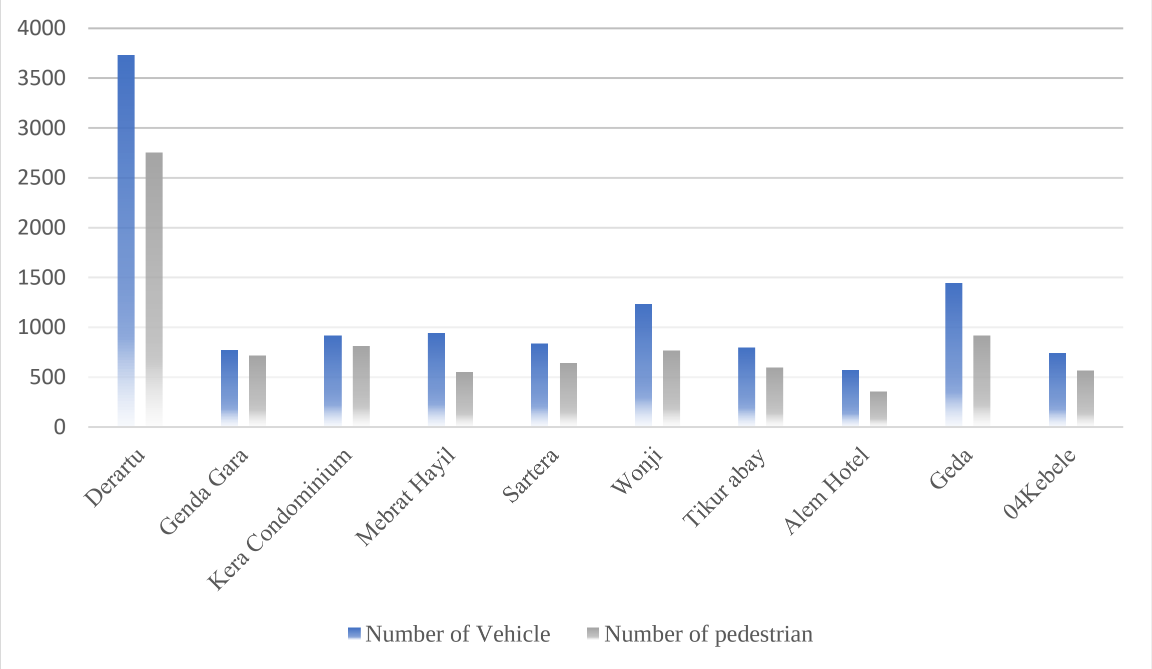


Figure 3. 5 Maximum peak hour vehicle volumes vs pedestrian at each junction

Figure 3.4 and Figure 3.5 clearly show the maximum and minimum numbers of vehicle and pedestrian traffic at surveyed junctions. For the most part when there is increased traffic volume, there are more pedestrians. The reason for this can mostly be attributed to land use. The maximum numbers of vehicles and pedestrians traffic exist at Derartu Tulu that are located at the major road corridor, central business district and residential part of Adama. The minimum numbers of vehicle and pedestrian traffic were counted at Alem Hotel junction where there are less residential areas and commercial activities.The maximum percentages of heavy vehicles is seen at wonji and Genda Gara of the intersections as shown in Table 3.3. This is because; heavy vehicles use these junctions to enter Adama and to exit to Addis Ababa. However; both Mebrat Hayil and and Sartera junction have 1.5% of heavy vehicles, out of the total number of vehicles counted at those junctions. This is because; heavy vehicles are not allowed to inter the city at the data collection time of the day.

Table 3. 5 Entry Peak Hour Traffic Flow & Percentage Share on Each Leg.

No	Junction Name	Approach Leg	Entry Traffic on Leg (PCU)	% of Traffic Share
1	Derartu Tullu	Mebrat Hayil	1141	23.7
		Aba Geda	1378	28.6
		Asella Road	1861	38.6
		Harar Road	443	9.2
2	Genda Gara	Road to Bole	574	51.3
		Ali Bira Road	544	48.7
3	Kera condominium	Sartera	321	30.7
		Medanihalem	214	20.4
		Wonji	236	22.5
		Mission Church	280	26.7
4	Mebrat Hayil	Sartera	335	26.2
		St Marry Church	265	20.8
		Post office	324	25.4
		Tele	353	27.6
5	Alem Hotel	Adama University	160	13.7
		Amede	454	39
		Post office	334	28.6
		Sike Road	252	21.6
6	Tikur Abay	Post office	573	37.5
		Micheal Church	505	33.1
		Kereyu street	448	29.3
7	04 Kebele	Diamond Café	157	22.1
		Bole road	546	23.4
		Hospital	246	20.3
		Sillase church	143	21
8	Geda Resort	Bole road	240	35.7
		Genda hara	113	16.8
		Genda Gara	154	22.9
		Goro School	166	24.7
9	Wonji	Aba geda road	590	34.3
		Franko	588	34.2
		Kera	102	5.9
		Adama steel	468	27.2
10	Sartera	Mebrat	204	18.5
		Old bus station	408	37
		Atena tera	237	21.5
		Kera	255	23

3.6 Geometric Data

As per the requirement of the SIDRA Intersection Version 8.1, different geometric data are required, such as, island diameter, circulatory width, number of circulatory lanes, entry lanes, entry lane number and average lane width at the entry of roundabout junction. These data were measured using tape meter and also by observing the roundabout existing configuration. The collected geometric data are summarized in Table 3.5 and Table 3.6

Table 3. 6 Geometric data of each roundabout.

Junction Name	Approach Leg	Entry Lane	Number of circulatory lanes	Island Diameter(m)	Average Lane width(m)	Circulatory Road width(m)
Derartu Tulu	Mebrat Hayil	2	2	20	3.3	3
	Aba Geda	2	2	20	3	3
	Asella Road	2	2	20	3.3	3
	Harar Road	2	2	20	3	3
Genda Gara	Ali Bira	2	2	30	3.6	3
	West Road	2	2	30	3.6	3
Medanihalem	Sartera	2	2	15	3	3
	Kera	2	2	15	3	3
	Wonji	2	2	15	3	3
	Mission	2	2	15	3	3

Table 3. 7 Geometric data of each Unsignalized intersection.

Junction Name	Approach Name	Number of Entry lane	Number of exit lane	Median Width(m)
` Mebrat Hayil	Sartera	2	2	1.2
	St Mary Church	2	2	1.5
	Post office	2	2	1.2
	Tele	2	2	1.5
Sar Tera	Mebrat Hayil	2	2	1
	Old Bus Station	2	2	1.2
	Atena Tera	2	2	N/A

	Kera	2	2	1
Wonji Junction	Aba Geda	2	2	1.2
	Post office	2	2	1.2
	Kera	2	2	1
	Expressway	2	2	1.2
Tikur Abay Hotel	Post office	2	2	1.2
	Micheal Church	2	2	1.2
	New Bus station	2	2	1
Alem Hotel	ASTU	2	2	1.2
	Amede Market	2	2	1.2
	Post office	2	2	1.2
	Sike Road	2	2	1.2
Geda Resort	Sillase Churdc	2	2	1.2
	Geda Resort	2	2	1.2
	Industry Zone	2	2	1
	Genda Gara	2	2	1
Diamond Café	Alem Hotel	2	2	1.2
	ASTU	2	2	1.2
	O4	2	2	N/A
	Genda Gara	2	2	N/A

3.7 Method of Analysis.

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

3.7.1 Methods of Analysis for Roundabout Junction

As noted in the Highway Capacity Manual (Transportation Research Board, 2000), intersection analysis models can be classified into two types: empirical and analytical models. Empirical models use observations at many different intersections under all types of conditions to develop regression equations that match intersection characteristics with intersection capacity. Analytical models estimate capacity based on gap-acceptance relationships that do not require observations under congested conditions. Since the Empirical Method totally depends on the geometric elements of the roundabout, it is sometimes difficult to find the necessary geometric features (elements) on Adama roundabouts. In this regard, the Analytical Method is more realistic than Empirical Method since it considers the traffic environment. Therefore, the Analytic Method is utilized in this research. The basis of determining the capacity of a roundabout can be seen from the HCM (2010) shown in Figure 3.6

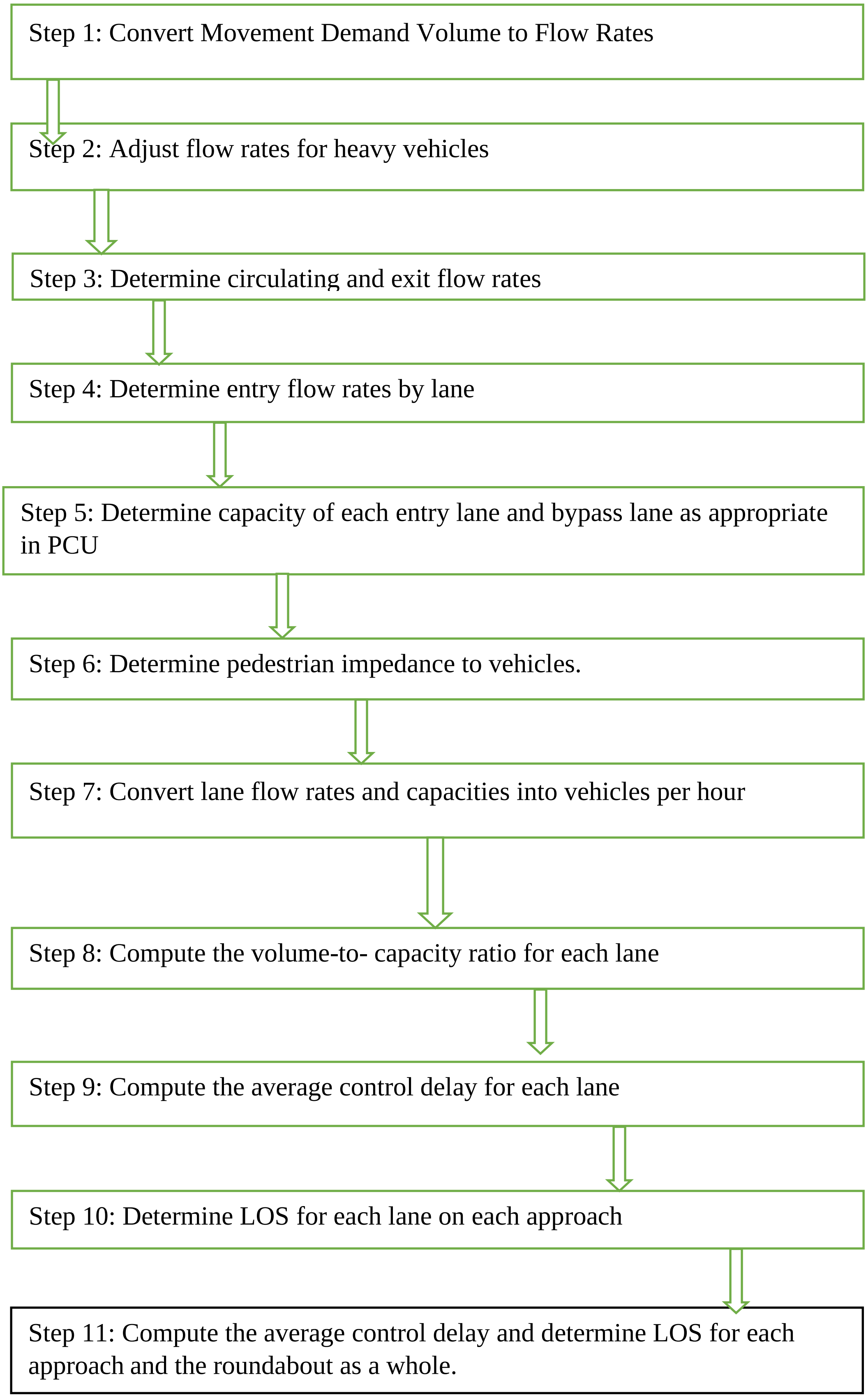


Figure 3. 6 Roundabout Analysis Methodology (HCM 2010)

3.7.2 Methods of Unsignalized intersection analyses.

TWSC intersections require only drivers on the minor-street approaches to stop before proceeding into the intersection. Left-turning drivers from the major street may have to yield to oncoming major-street through or right-turning traffic but are not required to stop in the absence of oncoming traffic. The methodologies presented rely on the required input data listed previously to compute the potential capacity of each minor movement, which is ultimately adjusted, if appropriate, to compute a movement capacity for each movement. The movement capacity can be used to estimate the control delay by movement, by approach, and for the intersection as a whole. Queue lengths can also be estimated once movement capacities are determined (HCM 2010) as shown in Figure 3.7.

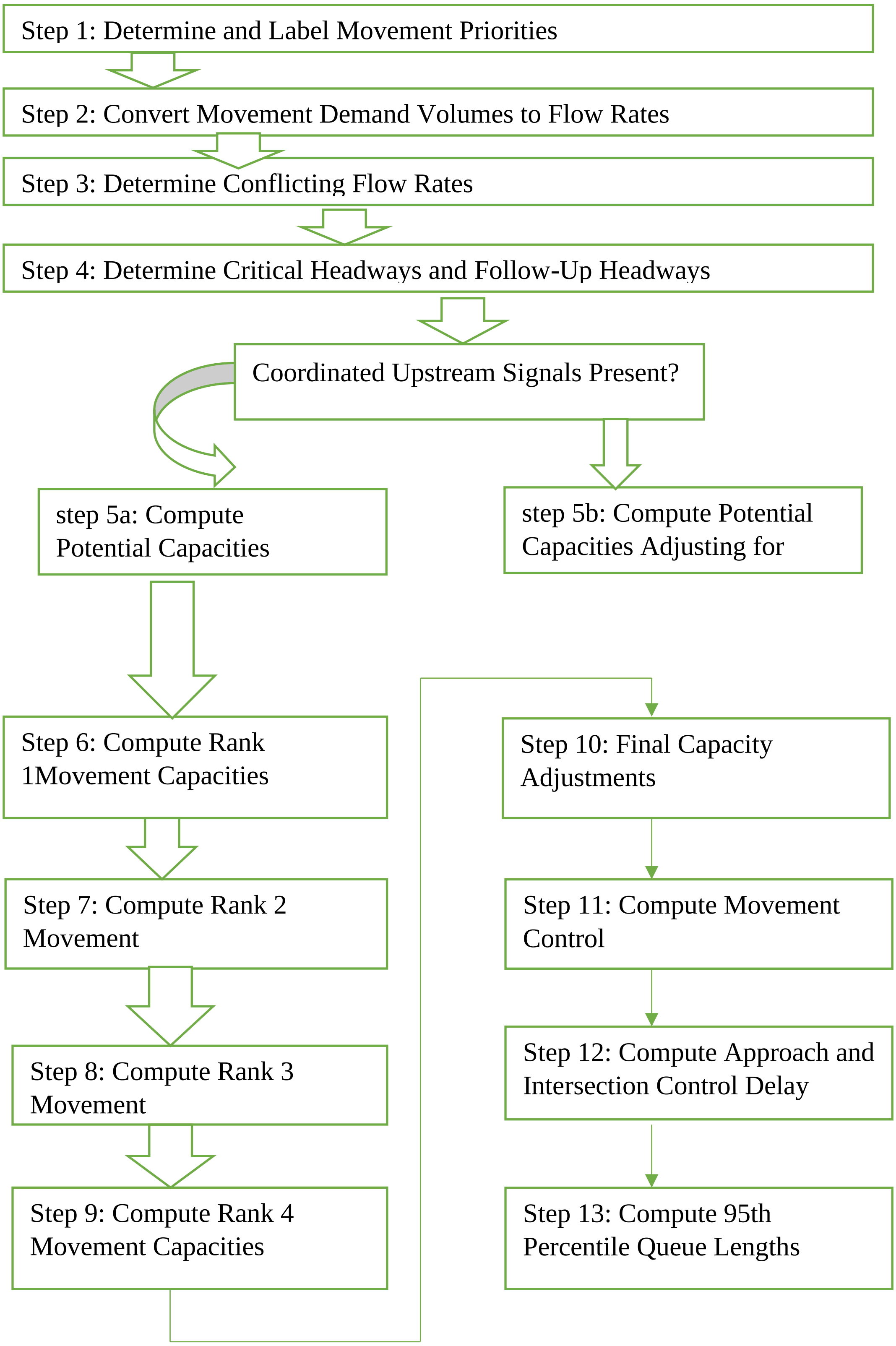


Figure 3. 7 Unsignalized intersection analysis methodology (HCM 2010)

3.8 Analysis using Software

There are different computer software's to analyze traffic operations at roundabouts and signalized intersections. The software can be of two types: macroscopic and microscopic models. The macroscopic models use traffic volume flows to model intersections as isolated locations. On the other hand, the microscopic models simulate the movement of individual vehicles, thereby allowing a network-wide analysis. For research, one of the macroscopic models (SIDRA) software program is used to analyze traffic operations at roundabout. In fact, AACRA also recommends SIDRA Intersection software for capacity analysis, which was developed by using analytic methods with some geometric elements. For this research, the Signalized & Un-signalized Intersection Design and Research Aid (SIDRA) software is preferred for capacity analysis for the following reasons:

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

4.RESULTS AND DISCUSSIONS.

During the analysis of the study at selected unsignalized intersections and round abouts using the collected data following procedures mentioned from the research methodology with the aid of SIDRA Intersection Version 8.1 Software package, the results are presented in the following sections with brief explanations.

4.1 Roundabout Junctions

4.1.1 Derartu Tulu Roundabout

Geometric Data

- ✓ Number of approaches or legs - 4
- ✓ Number of circulating lane-2
- ✓ Inscribed circle diameter - 30m

4) Central island diameter -20m

5) Truck apron width – 0

Table 4. 1-Geometric data for Derartu Tulu Roundabout

	Mebrat Hayil	Aba Geda	Asella Road	Harar road
Number of entry lane	2	2	2	2
Average Lane width (m)	3.3	3.3	3.3	3.3

Table 4. 2-Traffic volume data for Derartu Tulu Roundabout

Junction Name	**Approach Leg**	**Entry Traffic on leg (PCU)**	**% of traffic share**
	Mebrat Hayil	1141	23.7
	Aba Geda	1378	28.6
	Asella Road	1861	38.6
	Harar Road	443	9.2
Total		4823	

An excel program is used to analyze the traffic count shown in Table 3.2 to specify the peak hour. From site investigation and traffic count, the following were observed: a) The peak hour at Derartu Tulu roundabout is found to be between 8:00-9:00 a.m. The total traffic volume during this hour was 4823. Figures 4.1 shows the peak hour traffic during the time of survey

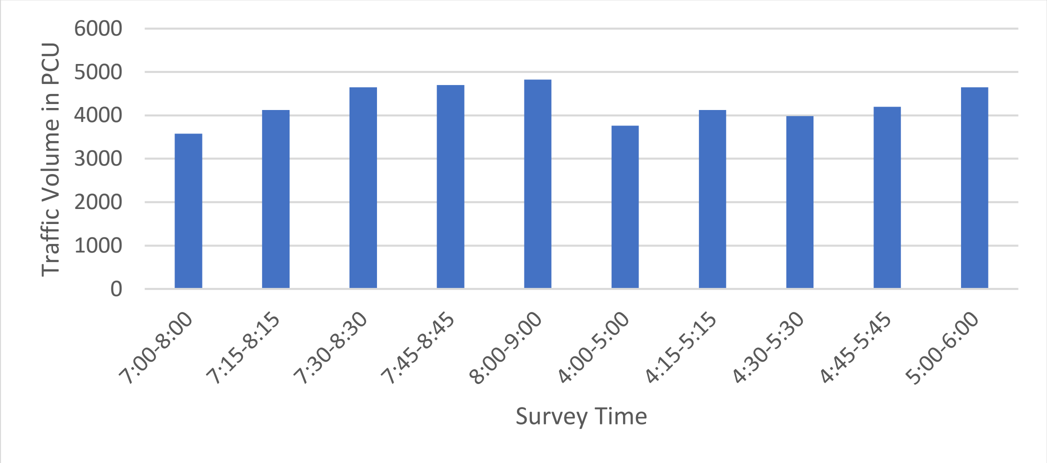


Figure 4. 1 Traffic Volume at Derartu Tulu Roundabout counted within 15 minutes Time Interval

b) The percentage of heavy vehicles at Derartu Tulu roundabout was concentrated on the approaches that come from Harar Road and Abagega, as shown in Table 4.3

Table 4. 3 Percentage of Heavy Vehicle for All Approaches at Derartu Tulu Roundabout

Approach	Percentage of Heavy Vehicle at Peak Hour
Mebrat Hayil	1.5
Aba Geda	3.5
Asella Road	1.7
Harar Road	4

Derartu Tulu Roundabout Analysis Results in SIDRA Intersection Output

To evaluate the existing LOS, the SIDRA Software Program was used and lane LOS values were based on degree of saturation per lane and Intersection & Approach LOS values were based on worst degree of saturation for any lane. Table 4.4 below shows the detail.

Table 4. 4-Output Summary of Derartu Tulu Roundabout

Mov ID	Turn	Demand Flows Total veh/h	Demand Flows HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
Movement Performance - Vehicles												
South: Asella Road												
1	L2	884	2.2	1.515	252.0	LOS F	201.3	1430.6	1.00	3.81	7.04	9.3
2	T1	484	1.0	1.515	252.0	LOS F	201.3	1430.6	1.00	3.81	7.04	9.3
3	R2	568	0.7	0.320	2.9	LOS A	1.5	10.3	0.20	0.13	0.20	53.5
Approach		1937	1.5	1.515	178.9	LOS F	201.3	1430.6	0.76	2.73	5.03	13.0
East: Harar Road												
4	L2	244	12.0	1.076	86.3	LOS F	29.2	215.4	1.00	2.63	6.52	24.1
5	T1	372	2.5	1.076	86.1	LOS F	29.2	215.4	1.00	2.63	6.52	20.3
6	R2	179	0.0	0.091	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	54.7
Approach		795	4.9	1.076	66.8	LOS E	29.2	215.4	0.77	2.04	5.05	24.9
North: Mebrat Hayil												
7	L2	148	0.5	0.941	55.8	LOS D	11.6	81.4	0.95	1.68	3.66	26.1
8	T1	316	0.3	0.941	55.8	LOS D	11.6	81.4	0.95	1.68	3.66	26.2
9	R2	737	0.7	0.356	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	52.3
Approach		1201	0.6	0.941	21.6	LOS C	11.6	81.4	0.37	0.65	1.41	35.0
West: Aba Geda Road												
10	L2	1	2.1	0.003	5.6	LOS A	0.0	0.1	0.41	0.29	0.41	43.8
11	T1	1	2.2	0.003	5.6	LOS A	0.0	0.1	0.41	0.29	0.41	48.3
12	R2	1	3.7	0.001	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	54.6
Approach		3	2.7	0.003	3.7	LOS A	0.0	0.1	0.27	0.20	0.27	49.1
All Vehicles		3936	1.9	1.515	108.1	LOS F	201.3	1430.6	0.64	1.95	3.93	17.5

	Approaches				Intersection
	South	East	North	West	
LOS	F	E	C	A	F

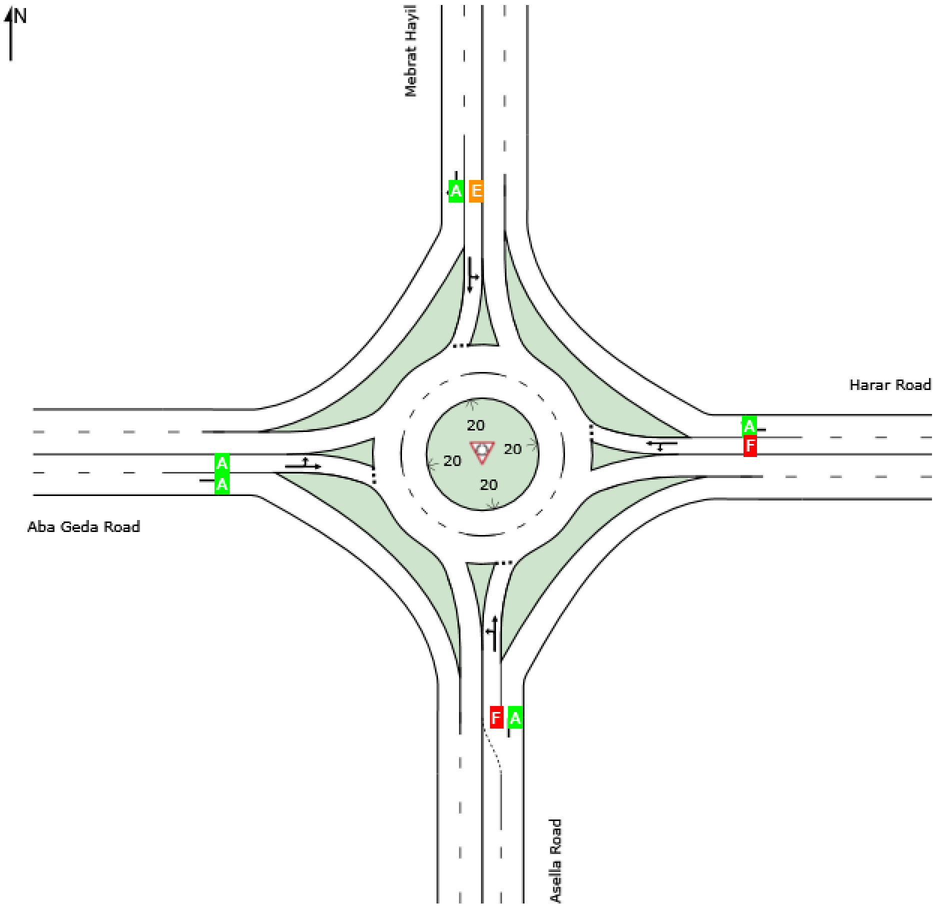


Figure 4. 2 Level of Service (LOS) at Derartu Tulu Roundabout

The results of capacity analysis show that the level of service (LOS) of the Derartu Tulu Roundabout is "F" and it is noticed that all approaches road to Asella approach have level of service (LOS) of "F"; but road to Harar approach has a level of service of "E".

The capacity of Derartu Tulu Roundabout is inadequate to control traffic flow at peak hour time due to the following problems observed;

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

This Roundabout has an inscribed circle diameter of 33m; however, according to FHWA, 2,000 double lane urban roundabout that has a typical inscribed circle diameter should be in the range of 45 to 55m.This shows that the inscribed circular diameter of Derartu Tulu Roundabout is below the standard and it is one of the problems that has lower the capacity. From Table 3.3 it is observed that there is unbalanced traffic flow at legs or approaches at this Roundabout. According FHWA it is not recommended to build roundabouts as a mechanism of traffic control devices when there is unbalanced traffic on the legs (FHWA-RD-00-067). Hence, this is also one of the problems that have great impact on the capacity and other factors that have impact on Level of Service at Derartu Tulu Roundabout since the junction connects the road from Addis Ababa to Dire Dawa trunk road. Furthermore, the percentage of heavy vehicle which is greater than 4.7% and the pedestrian number is high and separate pedestrian cross is not provided; as a result, the pedestrian conflict with vehicle at approaches reduce speed and increases delay at junction lowering the Level of Service of the roundabout. Therefore, currently at peak hour the Derartu Roundabout is not functioning well and so traffic police man regulates traffic flow.

4.1.2 Genda Gara Roundabout

Geometric Data

- ✓ Number of approaches or legs – 2
- ✓ Number of circulating lane-2
- ✓ Inscribed circle diameter - 30m

4) Central island diameter -20m
5) Truck apron width – 0

Table 4. 5-Geometric data for Genda Gara Roundabout

	Road to Bole	Ali Bira Road
Number of entry lane	2	2
Average Lane width (m)	3.3	3.3

Table 4. 6-Traffic volume data for Genda Gara Roundabout

Junction Name	**Approach Leg**	**Entry Traffic on leg (PCU)**	**% of traffic share**
	Road to Bole	574	51.3
	Ali Bira Road	544	48.7
Total		1118	

An excel program was used to analyze the traffic count shown in Table 3.2 to specify the peak hour. From site investigation and traffic count, the following were observed: a) The peak hour at Genda Gara was found to be between 5:00-6:00 p.m. The total traffic volume during this hour was 1118. Figures 4.3 show the peak hour traffic (pcu) during the survey time at Junction.

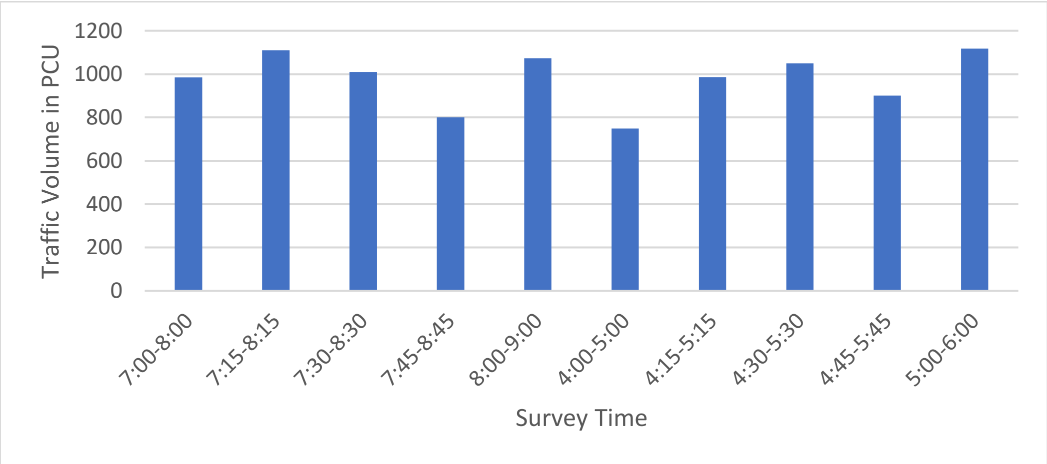


Figure 4. 3 Traffic Volume at Genda Gara Roundabout counted within 15 minutes Time Interval

b) The percentage of heavy vehicles at Genda Gara Roundabout was not significant at all approaches as shown in Table 4.7

Table 4. 7 -Percentage of Heavy Vehicle for All Approaches at Genda Gara Roundabout

Approach	Percentage of Heavy Vehicle at Peak Hour
Genda Hara	1.5
Ali Bira Road	2.7

Genda Gara Roundabout Analysis Results in SIDRA Intersection Output

To evaluate the existing LOS, the SIDRA Software Program was used and lane LOS values were based on degree of saturation per lane and Intersection & Approach LOS values were based on worst degree of saturation for any lane. Table 4.8 below shows the detail.

Table 4. 8-Output Summary of Genda Gara Roundabout

Movement Performance - Vehicles												
Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Ali Bira Road												
1	L2	714	9.8	0.386	8.6	LOS A	0.0	0.0	0.00	0.66	0.00	52.7
Approach		714	9.8	0.386	8.6	LOS A	0.0	0.0	0.00	0.66	0.00	52.7
West: Road to West												
12	R2	631	2.9	0.283	4.1	LOS A	0.0	0.0	0.00	0.52	0.00	55.0
Approach		631	2.9	0.283	4.1	LOS A	0.0	0.0	0.00	0.52	0.00	55.0
All Vehicles		1344	6.6	0.386	6.5	LOS A	0.0	0.0	0.00	0.60	0.00	53.7

	Approaches		Intersection
	South	West	
LOS	A	A	A

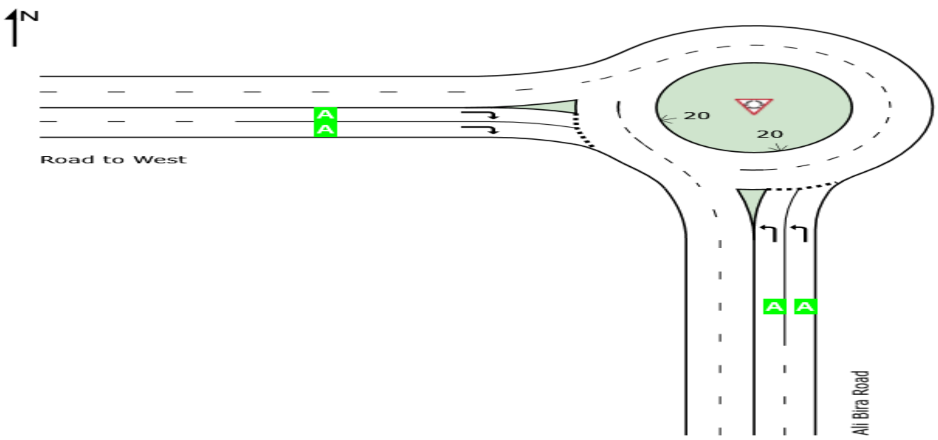


Figure 4. 4 Level of Service (LOS) at Genda Gara Roundabout

As the results of capacity analysis shows, the level of service (LOS) of the Genda Hara approach is “A”, Ali Bira road is “A”.therefore, the overall level of service of Genda Gara Roundabout is “A”.

4.2 Unsignalized Junction

4.2.1 Post-Office Signalize Junction

Table 4. 9-Geometric data for Mebrat Hayil unsiignalized Junction

Junction Name	Approach Name	No.of Entry Lane	No. of Exit Lane	Lane Width (m)	Median Width (m)
Mebrat Hayil	Sartera	2	2	3.3	1.2
	St Marry Church	2	2	3.3	1.2
	Post office	2	2	3.3	1.5
	Tele	2	2	3.3	1.2

Table 4. 10-Traffic Volume Data for Mebrat Hayil unsignalized Junction

Junction Name	**Approach Leg**	**Entry Traffic on Leg (PCU)**	**% of Traffic Share**
Mebrat Hayil	Sartera	335	26.2
	St Marry Church	265	20.8
	Post office	324	25.4
	Tele	353	27.6

An excel program was used to analyze the traffic volume count shown in Table 4.2 to specify the peak hour. From site investigation and traffic account, the following were observed: a) The peak hour at Post office signalized intersection is found to be between 5:00-6:00 p.m. The total traffic volumes during this hour at intersection was 4028 pc/h. Figure 4.5 show the peak hour during the time period of survey at Junction.

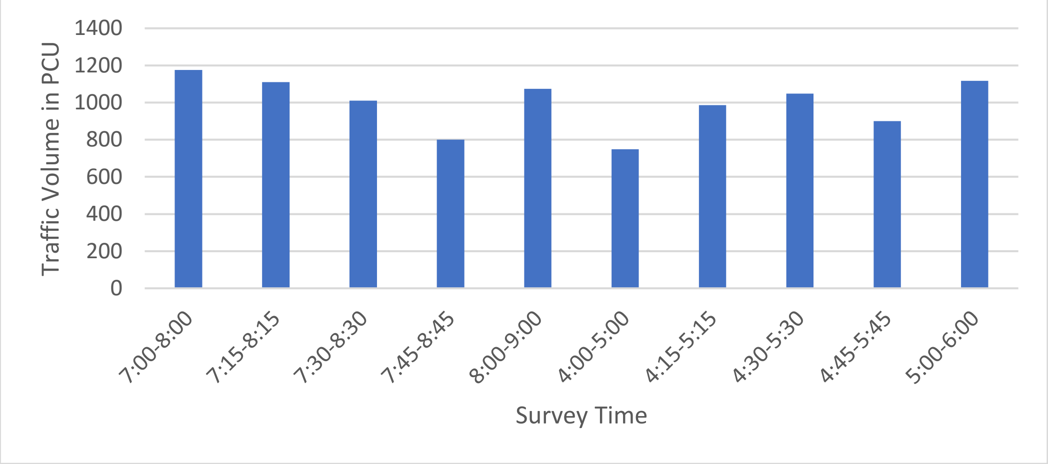


Figure 4. 5 Traffic Volume at Mebrat Hayil intersection counted within 15 minutes Time Interval

c) The percentages of heavy vehicles at Mebrat Hayil unsignalized Junction were concentrated on sartera and post office approaches as shown in

Table 4. 11.-Percentages of Heavy Vehicles for all approach at the Mebrat Hayil Junction

Approach	Percentage of Heavy Vehicle
Sartera	1.1
St Marry Church	3.2
Post office	3.6
Tele	3.7

Mebrat Hayil unsignalized Intersection Analysis Results in SIDRA Intersection Output

To evaluate the existing LOS SIDRA software program was used and lane LOS values are based on average delay per lane and Intersection and Approach LOS values are based on average delay for all lanes and Table 4.12 below shows the detail.

Table 4. 12-Output Summary of Mebrat Hayil unsignalized Intersection

Movement Performance - Vehicles												
Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Post office												
1	L2	172	0.1	0.766	35.5	LOS A	5.8	40.5	1.00	1.79	4.47	28.1
2	T1	126	0.8	0.766	35.7	LOS A	5.8	40.5	1.00	1.79	4.47	28.0
3	R2	43	0.2	0.126	12.5	LOS A	0.4	3.0	0.91	1.26	2.05	43.0
Approach		341	0.4	0.766	32.6	LOS D	5.8	40.5	0.99	1.72	4.16	29.3
East: St Mary Church												
4	L2	53	0.1	0.688	33.1	LOS A	4.4	30.8	1.00	1.63	3.78	29.1
5	T1	171	0.8	0.688	33.3	LOS A	4.4	30.8	1.00	1.63	3.78	29.1
6	R2	53	0.2	0.185	14.8	LOS A	0.7	4.7	0.95	1.28	2.17	40.8
Approach		276	0.6	0.688	29.7	LOS D	4.4	30.8	0.99	1.57	3.47	30.8
North: Bole Road												
7	L2	48	0.0	1.468	251.3	LOS F	40.6	284.3	1.00	4.51	16.45	6.6
8	T1	452	0.0	1.468	251.4	LOS F	40.6	284.3	1.00	4.51	16.45	6.6
9	R2	78	0.0	0.254	15.5	LOS A	1.0	6.8	0.95	1.30	2.28	40.2
Approach		578	0.0	1.468	219.6	LOS F	40.6	284.3	0.99	4.08	14.54	7.4
West: Sartera												
10	L2	53	1.0	0.816	53.9	LOS A	6.4	45.0	1.00	1.82	4.63	21.9
11	T1	158	0.2	0.816	54.0	LOS A	6.4	45.0	1.00	1.82	4.63	22.0
12	R2	263	1.0	0.911	66.8	LOS D	8.9	62.9	1.00	2.09	5.77	19.1
Approach		474	0.7	0.911	61.1	LOS F	8.9	62.9	1.00	1.97	5.26	20.2
All Vehicles		1668	0.4	1.468	105.0	LOS F	40.6	284.3	0.99	2.58	7.96	13.7

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

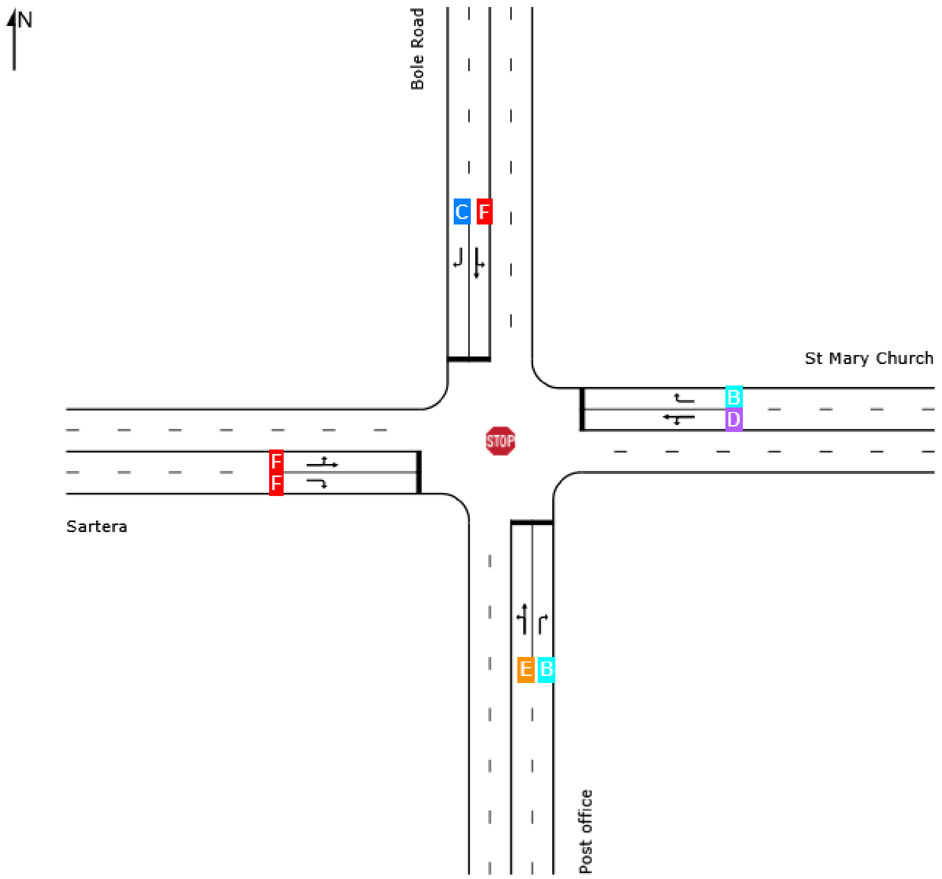


Figure 4. 6 Level of Service (LOS) at Mebrat Hayil unsignalized intersection

4.2.2 Wonji Junction

Table 4. 13 Geometric data for Wonji unsiignalized Junction

Junction Name	Approach Name	No.of Entry Lane	No. of Exit Lane	Lane Width (m)	Median Width (m)
Wonji	Aba geda road	2	2	3.3	1.2
	Franko	2	2	3.3	1.2
	Kera	2	2	3.3	1.5
	Adama steel	2	2	3.3	1.2

Table 4. 14-Traffic Volume Data for Wonji unsignalized Junction

Junction Name	**Approach Leg**	**Entry Traffic on Leg (PCU)**	**% of Traffic Share**
Wonji	Aba geda road	590	34.3
	Franko	588	34.2
	Kera	102	5.9
	Adama steel	468	27.2

An excel program was used to analyze the traffic volume count shown in Table 4.2 to specify the peak hour. From site investigation and traffic account, the following were observed: a) The peak hour at wonji unsignalized intersection is found to be between 8:00-9:00 a.m. The total traffic volumes during this hour at intersection was 1719 pc/h. Figure 4.7 show the peak hour during the time period of survey at Junction

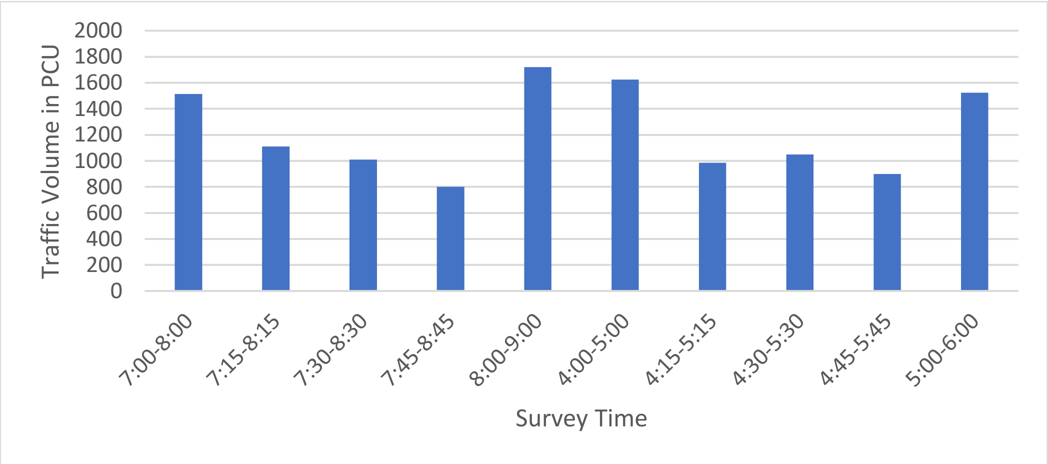


Figure 4. 7 Traffic Volume at Wonji intersection counted within 15 minutes Time Interval

c) The percentages of heavy vehicles at wonji unsignalized intersection were concentrated on Franko and Abageda approaches as shown in

Table 4. 15.-Percentages of Heavy Vehicles for all approach at the Mebrat Hayil Junction

Approach	Percentage of Heavy Vehicle
Aba geda road	2.3
Franko	3.2
Kera	1.2
Adama steel	1.3

Wonji Unsignalized Intersection Analysis Results in SIDRA Intersection Output To evaluate the existing LOS SIDRA software program was used and lane LOS values are based on average delay per lane and Intersection and Approach LOS values are based on average delay for all lanes and Table 4.16 below shows the detail.

Table 4. 16-Output Summary of Wonji unsignalized Intersection

Movement Performance - Vehicles												
Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Exit to Expressway												
1	L2	118	3.5	0.489	26.7	LOS A	2.3	16.9	1.00	1.43	2.85	41.9
2	T1	149	2.8	1.367	218.2	LOS F	28.2	201.5	1.00	3.58	12.26	13.1
3	R2	220	1.9	1.367	218.0	LOS F	28.2	201.5	1.00	3.58	12.26	13.1
Approach		487	2.6	1.367	171.8	LOS F	28.2	201.5	1.00	3.06	9.98	15.7
East: Post Office												
4	L2	199	11.0	0.583	25.2	LOS A	3.2	24.1	1.00	1.53	3.23	42.6
5	T1	317	14.2	0.935	62.0	LOS D	10.5	82.2	1.00	2.35	6.66	29.8
6	R2	38	5.0	0.935	61.5	LOS D	10.5	82.2	1.00	2.35	6.66	29.9
Approach		554	12.4	0.935	48.7	LOS E	10.5	82.2	1.00	2.05	5.43	33.4
North: Kera Condominium												
7	L2	31	2.0	0.157	18.1	LOS A	0.6	4.1	1.00	1.27	2.17	46.5
8	T1	32	6.0	0.491	29.4	LOS A	2.4	17.1	1.00	1.43	2.85	40.6
9	R2	79	4.0	0.491	29.2	LOS A	2.4	17.1	1.00	1.43	2.85	40.7
Approach		141	4.0	0.491	26.9	LOS D	2.4	17.1	1.00	1.39	2.70	41.8
West: Aba Geda												
10	L2	117	4.0	0.359	17.4	LOS A	1.5	10.8	0.96	1.35	2.49	46.8
11	T1	360	9.0	1.094	105.8	LOS F	17.9	134.4	1.00	3.03	9.69	22.0
12	R2	38	3.0	1.094	105.5	LOS F	17.9	134.4	1.00	3.03	9.69	22.0
Approach		515	7.4	1.094	85.7	LOS F	17.9	134.4	0.99	2.65	8.06	25.0
All Vehicles		1697	7.4	1.367	93.5	LOS F	28.2	201.5	1.00	2.47	7.31	23.7

4.3 Summary of Results

The results of analysis for the four intersections under this study are summarized in Table 4.17,4.18 and Table 4.19. The performances were measured in terms of v/c ratio or degree of saturation, average control delay per vehicle and level of service was applied using the

HCM 2010 Manual. Detailed analysis results of SIDRA Intersection Output are available in Appendix-B.

4.3.1 Volume to capacity ratio

Volume to capacity ratio or degree of saturation provides a direct assessment of the sufficiency of a given junction. There are no absolute standards for the 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 junctions will likely deteriorate rapidly. As the output of the analysis in table 4.1 shows that the degree of saturation for each seven unsignalized intersections and three roundabouts. Detailed analysis using SIDRA software are available in Appendix A-C. From Table 4.17, it is seen that one roundabout and six unsignalized intersections have very low effective capacity compared to their entry flow. They are with the F level of service. Actually, the intersection performance or capacity depends on the approaches or legs performance and always their v/c ratio is taken from the maximum v/c ratio of the legs.

Table 4. 17 Volume to capacity ratio of each intersections

Junction Name	Type of Junction	Number of Legs	Degree of Saturation (V/C)
Derartu Tullu	Roundabout	4	2.37
Genda Gara	Roundabout	2	0.25
Medihanialem	Roundabout	4	0.36
Mebrat Hayil	UI	4	1.47
Sartera	UI	4	1.29
Wonji Junction	UI	4	1.37
Tikur Abay	UI	3	1.16
Alem Hotel	UI	4	1.07
Geda Resort	UI	4	0.95
Dimond Café	UI	4	0.63

4.3.2Average delay per vehicle

Delay is a standard parameter used to determine the performance of an intersection. The Highway capacity manual identifies delay as the primary measure of effectiveness for both signalized and un-signalized intersection, with the level of service determined from the delay estimate. The output of the sidra analysis in Table 4.18 shows that the average delay per vehicle of roundabouts and unsignalized intersections.

Table 4. 18 Average delay (control) of each intersection

Junction Name	Type of Junction	Number of Legs	Control Delay (sec.)
Derartu Tullu	Roundabout	4	147
Genda Gara	Roundabout	2	6.5
Medihanialem	Roundabout	4	8.1
Mebrat Hayil	UI	4	105
Sartera	UI	4	89.3
Wonji Junction	UI	4	93.5
Tikur Abay	UI	3	73.8
Alem Hotel	UI	4	68.2
Geda Resort	UI	4	34.2
Dimond Café	UI	4	22.7

4.3.3 Level of service (LOS)

Quality of service requires quantitative measures to characterize operational conditions. 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. Six levels of service (LOS) are 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. Accordingly, the output of the analysis using SIDRA intersection version 8.1 Software indicates that the level of service of one roundabout and five unsignalized intersections are at the least level of service i.e. LOS F,two roundabouts are serving below their capacity i.e. LOS A and the two unsignalized intersections are satisfactory having v/c less than 1 as shown in table 4.3 ,this indicates that the quality of service is best.

Table 4. 19 Level of Service (LOS)

Junction Name	Type of Junction	(V/C) Ratio	Control Delay (sec.)	LOS
Derartu Tullu	Roundabout	2.37	147	F
Genda Gara	Roundabout	0.25	6.5	A
Medihanialem	Roundabout	0.36	8.1	A
Mebrat Hayil	UI	1.47	105	F
Sartera	UI	1.29	89.3	F
Wonji Junction	UI	1.36	93.5	F
Tikur Abay	UI	1.16	73.8	F
Alem Hotel	UI	1.07	68.2	F
Geda Resort	UI	0.95	34.2	D
Dimond Café	UI	0.63	22.7	C

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

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

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

5.2 Recommendations

To further enhance the results of this study the following recommendations are formulated:

- ✓ Further research should be conducted to extend all aspects of this research, such as by collecting more data in order to improve results. From this data traffic volume is the one and it is recommended to count at least for seven days. Similarly, during collecting travel speed and travel time automatic data collection method are better to minimize error.
- ✓ It is better to separate the pedestrians from vehicular traffic at the intersections where high pedestrian flows were observed since they affect normal traffic flows.
- ✓ From the analysis result, it shows the intersections are serving above their capacity. Therefore, the city administration should consider this issue and formulate capacity improvement methods

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APPENDIX-A (SUMMARIZED TRAFFIC DATA)

Table 1 Vehicle and pedestrian flow data of Derartu Tulu Roundabout

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Derartu Tulu	Mebrat Hayil	Bajaj	978	175
		Light Vehicles	625	
		Bus and DT	18	
		Trailer	-	
	AbaGeda Road		250	265
		Light Vehicles	1124	
		Bus and DT	114	
		Trailer	6	
	Asella Road	Bajaj	1268	114
		Light Vehicles	1098	
		Bus and DT	86	
		Trailer	11	
	Harar Road	Bajaj	523	105
		Light Vehicles	421	
		Bus and DT	25	
		Trailer	17	

Table 2 Vehicle and pedestrian flow data of Genda Gara Roundabout

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Genda Gara	Road to west	Bajaj	152	56
		Light Vehicles	354	
		Bus and DT	84	
		Trailer	9	
	Ali Bira Road	Bajaj	357	85
		Light Vehicles	254	
		Bus and DT	46	
		Trailer	21	

Table 3 Vehicle and pedestrian flow data of Medanihalem Roundabout.

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Medanihalem	Sartera	Bajaj	253	114
		Light Vehicles	117	
		Bus and DT	41	
		Trailer	3	
	Kera Condominium	Bajaj	178	95
		Light Vehicles	84	
		Bus and DT	19	
		Trailer	6	
	Wonji Junction	Bajaj	143	145
		Light Vehicles	109	
		Bus and DT	21	
		Trailer	15	
	Mission church	Bajaj	201	52
		Light Vehicles	150	
		Bus and DT	12	
		Trailer	6	

Table 4 Vehicle and pedestrian flow data of Mebrat Hayil Unsignalized intersection.

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
` Mebrat Hayil	Sartera	Bajaj	246	156
		Light Vehicles	194	
		Bus and DT	10	
		Trailer	-	
	St Mary Church	Bajaj	-	142
		Light Vehicles	259	
		Bus and DT	3	
		Trailer	-	

	Post office	Bajaj	-	93
		Light Vehicles	315	
		Bus and DT	9	
		Trailer	-	
	Tele	Bajaj	394	134
		Light Vehicles	153	
		Bus and DT	2	
		Trailer	-	

Table 5 Vehicle and pedestrian flow data of Sartera Unsignalized intersection.

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Sar Tera	Mebrat Hayil	Bajaj	176	81
		Light Vehicles	89	
		Bus and DT	14	
		Trailer	3	
	Old Bus station	Bajaj	316	63
		Light Vehicles	245	
		Bus and DT	2	
		Trailer	1	
	Atena tera	Bajaj	274	73
		Light Vehicles	96	
		Bus and DT	-	
		Trailer	2	
	Kera	Bajaj	221	85
		Light Vehicles	141	
		Bus and DT	2	
		Trailer	-	

Table 6 Vehicle and pedestrian flow data of Wonji Junction

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Wonji Junction	Aba Geda	Bajaj	-	43
		Light Vehicles	359	
		Bus and DT	114	
		Trailer	16	
	Post office	Bajaj	-	53
		Light Vehicles	411	
		Bus and DT	106	
		Trailer	9	
	Kera condominium	Bajaj	86	64
		Light Vehicles	32	
		Bus and DT	10	
		Trailer	6	
	Expressway	Bajaj	81	74
		Light Vehicles	305	
		Bus and DT	65	
		Trailer	12	

Table 7 Vehicle and pedestrian flow data of Tikur Abay UI

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Tikur Abay Hotel	Post office	Bajaj	269	89
		Light Vehicles	312	
		Bus and DT	65	
		Trailer	14	
	Micheal Church	Bajaj	273	51
		Light Vehicles	253	
		Bus and DT	54	

		Trailer	17	
	Kereyu Street	Bajaj	321	32
		Light Vehicles	157	
		Bus and DT	56	
		Trailer	23	

Table 8 Vehicle and pedestrian flow data of Alem Hotel UI

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Alem Hotel	ASTU	Bajaj	125	34
		Light Vehicles	86	
		Bus and DT	6	
		Trailer	1	
	Amede Market	Bajaj	108	14
		Light Vehicles	286	
		Bus and DT	56	
		Trailer	15	
	Post office	Bajaj	-	65
		Light Vehicles	147	
		Bus and DT	87	
		Trailer	13	
	Sike Road	Bajaj	156	45
		Light Vehicles	93	
		Bus and DT	15	
		Trailer	29	

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Table 9 Vehicle and pedestrian flow data of Geda Resort

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
Geda Resort	Sillase Church	Bajaj	78	23
		Light Vehicles	54	
		Bus and DT	27	
		Trailer	16	
	Bole	Bajaj	45	51
		Light Vehicles	152	
		Bus and DT	22	
		Trailer	16	
	Industry Zone	Bajaj	63	16
		Light Vehicles	57	
		Bus and DT	12	
		Trailer	3	
	Genda Gara	Bajaj	174	28
		Light Vehicles	53	
		Bus and DT	8	
		Trailer	1	

Table 10 Vehicle and pedestrian flow data of Diamond Café UI (5:00PM-6:00PM)

Junction Name	Approach Leg	Vehicle type	Traffic Volume	Pedestrian Volume
	Alem Hotel	Bajaj	92	11
		Light Vehicles	25	
		Bus and DT	6	

		Trailer	10	
Diamond Café	ASTU	Bajaj	52	23
		Light Vehicles	45	
		Bus and DT	6	
		Trailer	1	
	O4	Bajaj	152	29
		Light Vehicles	85	
		Bus and DT	5	
		Trailer	-	
	Genda Gara	Bajaj	129	34
		Light Vehicles	65	
		Bus and DT	5	
		Trailer	7	

APPENDIX- B (GOOGLE IMAGES SELECTED INTERSECTIONS)

Figure 1 Derartu Tullu Roundabout. (Source: Google Map,2020)

Figure 2 Genda Gara Roundabout. (Source: Google Map,2020)

Figure 3 Alem Hotel Unsignalized intersections. (Source: Google Map,2020)

Figure 4 Sartera unsignalized intersections. (Source: Google Map,2020)

Figure 5 Medanihamlem roundabout. (Source: Google Map,2020)

Figure 6 Diamond cafe unsignalized intersection. (Source: Google Map,2020)

Figure 7 Mebrat Hayil Unsignalized intersection. (Source: Google Map,2020)

Figure 8 Tikur Abay Unsignalized intersection (Source: Google Map,2020)

Figure 9 Wonji junction ((Source: Google Map,2020)

Figure 10 04 kebele intersection ((Source: Google Map,2020)

APPENDIX-C

(Movement Summary of Each Intersections)

MOVEMENT SUMMARY

Site: 01 [Derartu Tulu]

New Site

Site Category: Existing

Roundabout

Movement Performance - Vehicles												
Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Asella Road												
1	L2	884	2.2	1.184	97.0	LOS F	104.6	743.3	1.00	1.67	3.06	19.8
2	T1	484	1.0	1.184	92.0	LOS F	104.6	743.3	1.00	1.67	3.06	19.8
3	R2	568	0.7	0.297	3.5	LOS A	0.0	0.0	0.00	0.48	0.00	54.9
Approach		1937	1.5	1.184	68.3	LOS E	104.6	743.3	0.71	1.32	2.16	24.5
East: Harar Road												
4	L2	244	12.0	2.366	644.8	LOS F	132.4	976.4	1.00	3.53	7.96	4.2
5	T1	372	2.5	2.366	639.5	LOS F	132.4	976.4	1.00	3.53	7.96	4.0
6	R2	179	0.0	0.091	3.4	LOS A	0.0	0.0	0.00	0.48	0.00	54.7
Approach		795	4.9	2.366	497.9	LOS F	132.4	976.4	0.77	2.84	6.17	4.5
North: Mebrat Hayil												
7	L2	148	0.5	1.124	106.6	LOS F	33.5	234.2	1.00	2.16	4.16	18.6
8	T1	316	0.3	1.124	101.7	LOS F	33.5	234.2	1.00	2.16	4.16	18.6
9	R2	737	0.7	0.356	3.5	LOS A	0.0	0.0	0.00	0.48	0.00	52.3
Approach		1201	0.6	1.124	42.0	LOS D	33.5	234.2	0.39	1.13	1.61	27.7
West: Aba Geda Road												
10	L2	1	2.1	0.003	10.8	LOS A	0.0	0.1	0.60	0.56	0.60	46.9
11	T1	1	2.2	0.003	4.9	LOS A	0.0	0.1	0.60	0.56	0.60	50.9
12	R2	1	3.7	0.001	3.5	LOS A	0.0	0.0	0.00	0.48	0.00	54.6
Approach		3	2.7	0.003	6.7	LOS A	0.0	0.1	0.40	0.53	0.40	51.0
All Vehicles		3936	1.9	2.366	147.0	LOS F	132.4	976.4	0.62	1.57	2.80	14.3

MOVEMENT SUMMARY Site: 02 [Genda Gara]

Site Category: Existing

Roundabout

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Ali Bira Road												
1	L2	714	9.8	0.252	8.6	LOS A	0.0	0.0	0.00	0.66	0.00	52.7
Approach		714	9.8	0.252	8.6	LOS A	0.0	0.0	0.00	0.66	0.00	52.7
West: Road to West												
12	R2	631	2.9	0.196	4.1	LOS A	0.0	0.0	0.00	0.52	0.00	54.0
Approach		631	2.9	0.196	4.1	LOS A	0.0	0.0	0.00	0.52	0.00	54.0
All Vehicles		1344	6.6	0.252	6.5	LOS A	0.0	0.0	0.00	0.60	0.00	53.7

MOVEMENT SUMMARY Site: 03 [Medanihalem Church]

New Site

Site Category: Existing

Roundabout

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Wonji Junction												
1	L2	114	4.0	0.239	10.0	LOS A	1.2	8.4	0.51	0.69	0.51	52.2
2	T1	105	4.0	0.239	4.9	LOS A	1.2	8.4	0.51	0.69	0.51	50.1
3	R2	84	4.3	0.127	7.2	LOS A	0.5	3.8	0.51	0.68	0.51	50.4
Approach		303	4.1	0.239	7.8	LOS A	1.2	8.4	0.51	0.69	0.51	51.1
East: Sartera												
4	L2	149	3.4	0.358	10.2	LOS A	1.8	13.1	0.54	0.71	0.54	50.7
5	T1	181	3.3	0.358	6.1	LOS A	1.8	13.1	0.54	0.71	0.54	51.0
6	R2	105	3.3	0.172	7.6	LOS A	0.7	4.0	0.52	0.71	0.52	46.3

Approach		436	3.3	0.358	7.9	LOS A	1.8	13.1	0.54	0.71	0.54	50.0
North: Mission Church												
7	L2	118	1.7	0.355	10.4	LOS A	1.8	12.7	0.58	0.73	0.58	47.3
8	T1	185	2.2	0.355	6.3	LOS A	1.8	12.7	0.58	0.73	0.58	50.6
9	R2	85	0.9	0.161	8.1	LOS A	0.6	4.5	0.56	0.74	0.56	49.2
Approach		388	1.8	0.355	8.0	LOS A	1.8	12.7	0.58	0.73	0.58	49.4
West: kera condominium												
10	L2	172	2.2	0.312	10.4	LOS A	1.6	11.5	0.58	0.74	0.58	49.3
11	T1	105	2.8	0.312	6.3	LOS A	1.6	11.5	0.58	0.74	0.58	50.4
12	R2	25	2.2	0.050	8.2	LOS A	0.2	1.3	0.53	0.69	0.53	51.4
Approach		302	2.4	0.312	8.8	LOS A	1.6	11.5	0.57	0.74	0.57	49.9
All Vehicles		1429	2.9	0.358	8.1	LOS A	1.8	13.1	0.55	0.72	0.55	50.1

MOVEMENT SUMMARY Site: 04 [Mebrat Hayil]

New Site

Site Category: Existing

Stop (All-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Post office												
1	L2	172	0.1	0.766	34.5	LOS A	4.8	40.5	1.00	1.79	4.47	28.1
2	T1	126	0.8	0.766	34.7	LOS A	4.8	40.5	1.00	1.79	4.47	28.0
3	R2	43	0.2	0.126	12.5	LOS A	0.4	3.0	0.91	1.26	2.05	43.0
Approach		341	0.4	0.766	32.6	LOS D	4.8	40.5	0.99	1.72	4.16	29.3
East: St Mary Church												
4	L2	53	0.1	0.688	33.1	LOS A	4.4	30.8	1.00	1.63	3.78	29.1
5	T1	171	0.8	0.688	33.3	LOS A	4.4	30.8	1.00	1.63	3.78	29.1
6	R2	53	0.2	0.185	14.8	LOS A	0.7	4.7	0.95	1.28	2.17	40.8
Approach		276	0.6	0.688	29.7	LOS D	4.4	30.8	0.99	1.57	3.47	30.8
North: Bole Road												
7	L2	48	0.0	1.468	251.3	LOS F	40.6	284.3	1.00	4.51	16.45	6.6
8	T1	452	0.0	1.468	251.4	LOS F	40.6	284.3	1.00	4.51	16.45	6.6
9	R2	78	0.0	0.254	14.5	LOS A	1.0	6.8	0.95	1.30	2.28	40.2
Approach		578	0.0	1.468	219.6	LOS F	40.6	284.3	0.99	4.08	14.54	7.4

West: Sartera												
10	L2	53	1.0	0.816	53.9	LOS A	6.4	44.0	1.00	1.82	4.63	21.9
11	T1	158	0.2	0.816	54.0	LOS A	6.4	44.0	1.00	1.82	4.63	22.0
12	R2	263	1.0	0.911	66.8	LOS D	8.9	62.9	1.00	2.09	4.77	19.1
Approach		474	0.7	0.911	61.1	LOS F	8.9	62.9	1.00	1.97	4.26	20.2
All Vehicles		1668	0.4	1.468	104.0	LOS F	40.6	284.3	0.99	2.58	7.96	13.7

MOVEMENT SUMMARY

Site: 05 [Sartera]

New Site

Site Category: Existing

Stop (All-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Old Bus Station												
1	L2	105	0.3	1.161	126.5	LOS F	22.4	157.3	1.00	3.37	11.45	16.0
2	T1	334	0.2	1.161	126.6	LOS F	22.4	157.3	1.00	3.37	11.45	16.0
3	R2	155	0.0	0.453	19.5	LOS A	2.1	14.5	0.98	1.40	2.74	42.4
Approach		594	0.2	1.161	98.7	LOS F	22.4	157.3	0.99	2.86	9.18	19.1
East: Mebrat Hayil												
4	L2	183	2.0	0.694	31.8	LOS A	4.5	31.9	1.00	1.65	3.85	34.6
5	T1	60	1.0	0.694	31.9	LOS A	4.5	31.9	1.00	1.65	3.85	29.7
6	R2	57	1.9	0.185	14.2	LOS A	0.7	4.8	0.94	1.28	2.16	41.2
Approach		300	1.8	0.694	28.5	LOS D	4.5	31.9	0.99	1.58	3.53	34.2
North: Atenatera												
7	L2	84	0.0	0.943	61.9	LOS D	11.0	76.8	1.00	2.33	6.87	20.1
8	T1	284	0.0	0.943	62.0	LOS D	11.0	76.8	1.00	2.33	6.87	24.6
9	R2	21	0.0	0.061	11.6	LOS A	0.2	1.4	0.90	1.24	1.96	43.8
Approach		389	0.0	0.943	59.3	LOS F	11.0	76.8	0.99	2.27	6.61	24.9
West: Kera												
10	L2	24	0.6	0.551	34.9	LOS A	2.8	19.8	1.00	1.46	3.02	27.9
11	T1	81	0.0	0.551	36.0	LOS A	2.8	19.8	1.00	1.46	3.02	27.9
12	R2	278	0.0	1.288	197.3	LOS F	20.7	144.6	1.00	2.93	9.49	11.4
Approach		383	0.0	1.288	153.0	LOS F	20.7	144.6	1.00	2.52	7.71	12.9

All Vehicles		1666	0.4	1.288	89.3	LOS F	22.4	157.3	0.99	2.42	7.22	19.6

MOVEMENT SUMMARY

Site: 06 [Wonji Junction]

New Site

Site Category: Existing

Stop (All-Way)

Movement Performance - Vehicles												
Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Exit to Expressway												
1	L2	118	3.5	0.489	26.7	LOS A	2.3	16.9	1.00	1.43	2.85	41.9
2	T1	149	2.8	1.367	218.2	LOS F	28.2	201.5	1.00	3.58	12.26	13.1
3	R2	220	1.9	1.367	218.0	LOS F	28.2	201.5	1.00	3.58	12.26	13.1
Approach		487	2.6	1.367	171.8	LOS F	28.2	201.5	1.00	3.06	9.98	14.7
East: Post Office												
4	L2	199	11.0	0.583	24.2	LOS A	3.2	24.1	1.00	1.53	3.23	42.6
5	T1	317	14.2	0.935	62.0	LOS D	10.5	82.2	1.00	2.35	6.66	29.8
6	R2	38	4.0	0.935	61.5	LOS D	10.5	82.2	1.00	2.35	6.66	29.9
Approach		554	12.4	0.935	48.7	LOS E	10.5	82.2	1.00	2.05	4.43	33.4
North: Kera Condominium												
7	L2	31	2.0	0.157	18.1	LOS A	0.6	4.1	1.00	1.27	2.17	46.5
8	T1	32	6.0	0.491	29.4	LOS A	2.4	17.1	1.00	1.43	2.85	40.6
9	R2	79	4.0	0.491	29.2	LOS A	2.4	17.1	1.00	1.43	2.85	40.7
Approach		141	4.0	0.491	26.9	LOS D	2.4	17.1	1.00	1.39	2.70	41.8
West: Aba Geda												
10	L2	117	4.0	0.359	17.4	LOS A	1.5	10.8	0.96	1.35	2.49	46.8
11	T1	360	9.0	1.094	104.8	LOS F	17.9	134.4	1.00	3.03	9.69	22.0
12	R2	38	3.0	1.094	104.5	LOS F	17.9	134.4	1.00	3.03	9.69	22.0
Approach		515	7.4	1.094	84.7	LOS F	17.9	134.4	0.99	2.65	8.06	24.0
All Vehicles		1697	7.4	1.367	93.5	LOS F	28.2	201.5	1.00	2.47	7.31	23.7

MOVEMENT SUMMARY

Site: 07 [Tikur Abay Hotel]

New Site

Site Category: Existing

Stop (All-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Mamo Mazemir Street												
2	T1	380	4.7	1.162	132.6	LOS F	20.4	150.1	1.00	3.19	10.43	18.9
3	R2	313	4.5	1.059	102.7	LOS F	14.4	104.4	1.00	2.63	8.02	22.4
Approach		693	4.6	1.162	119.1	LOS F	20.4	150.1	1.00	2.93	9.34	20.3
East: Kereyu Street												
4	L2	320	10.5	0.892	54.5	LOS D	8.9	67.8	1.00	2.15	4.88	31.5
6	R2	266	3.5	0.830	48.4	LOS A	7.0	50.3	1.00	1.91	4.97	33.5
Approach		586	7.3	0.892	52.3	LOS F	8.9	67.8	1.00	2.04	4.47	32.4
North: Post Office												
7	L2	285	6.6	0.725	31.2	LOS A	4.0	37.0	1.00	1.72	4.11	39.8
8	T1	409	4.3	0.939	57.5	LOS D	11.2	81.9	1.00	2.40	7.05	31.0
Approach		695	4.8	0.939	46.7	LOS E	11.2	81.9	1.00	2.12	4.84	34.1
All Vehicles		1974	6.2	1.162	73.8	LOS F	20.4	150.1	1.00	2.38	6.96	27.2

MOVEMENT SUMMARY

Site: 08 [Alem Hotel]

New Site

Site Category: Existing

Stop (All-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Sike street												
1	L2	120	6.0	0.535	30.8	LOS A	2.7	19.9	1.00	1.46	2.99	40.0
2	T1	164	3.0	0.743	46.0	LOS A	4.1	36.7	1.00	1.70	4.05	30.0
3	R2	24	4.9	0.743	46.0	LOS A	4.1	36.7	1.00	1.70	4.05	34.3

Approach		308	4.4	0.743	40.1	LOS E	4.1	36.7	1.00	1.61	3.64	34.2
East: Amede Market Center												
4	L2	153	7.0	0.527	24.2	LOS A	2.6	19.6	1.00	1.46	2.99	38.9
5	T1	309	7.0	1.054	98.7	LOS F	14.8	109.1	1.00	2.69	8.27	19.1
6	R2	27	2.0	1.054	98.4	LOS F	14.8	109.1	1.00	2.69	8.27	14.4
Approach		489	6.7	1.054	74.8	LOS F	14.8	109.1	1.00	2.31	6.62	22.4
North: ASTU												
7	L2	20	4.0	0.116	18.7	LOS A	0.4	3.0	1.00	1.26	2.12	42.9
8	T1	185	2.5	1.065	124.5	LOS F	12.0	84.4	1.00	2.29	6.62	16.1
9	R2	24	0.5	1.065	124.4	LOS F	12.0	84.4	1.00	2.29	6.62	16.1
Approach		229	2.4	1.065	116.2	LOS F	12.0	84.4	1.00	2.20	6.22	17.0
West: Post Office												
10	L2	49	11.0	0.212	17.8	LOS A	0.8	6.1	0.98	1.30	2.24	42.7
11	T1	197	19.0	0.794	51.4	LOS A	6.0	48.7	1.00	1.84	4.46	32.6
12	R2	14	13.0	0.794	51.1	LOS A	6.0	48.7	1.00	1.84	4.46	32.7
Approach		260	17.2	0.794	44.0	LOS E	6.0	48.7	1.00	1.74	4.04	33.8
All Vehicles		1287	7.5	1.065	68.2	LOS F	14.8	109.1	1.00	2.00	4.31	24.4

MOVEMENT SUMMARY

Site: 09 [Geda Resort]

New Site

Site Category: (None)

Stop (All-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Sillase Church												
1	L2	47	4.0	0.185	12.2	LOS A	0.7	4.9	0.97	1.28	2.19	8.6
2	T1	122	4.0	0.470	19.7	LOS A	2.2	16.2	1.00	1.42	2.81	6.4
3	R2	15	14.0	0.470	19.8	LOS A	2.2	16.2	1.00	1.42	2.81	6.4
Approach		184	4.8	0.470	17.8	LOS C	2.2	16.2	0.99	1.38	2.65	6.7
East: Gendagara												
4	L2	38	3.0	0.192	14.3	LOS A	0.7	4.1	1.00	1.28	2.22	7.9
5	T1	158	6.0	0.948	83.4	LOS D	9.2	67.1	1.00	2.09	4.70	1.3
6	R2	53	3.0	0.948	83.2	LOS D	9.2	67.1	1.00	2.09	4.70	1.6
Approach		248	4.9	0.948	73.0	LOS F	9.2	67.1	1.00	1.96	4.17	1.6

North: Bole												
7	L2	142	2.0	0.471	18.3	LOS A	2.2	14.7	0.99	1.41	2.81	6.7
8	T1	158	4.0	0.633	24.1	LOS A	3.7	26.8	1.00	1.57	3.47	4.0
9	R2	53	7.0	0.633	24.1	LOS A	3.7	26.8	1.00	1.57	3.47	4.3
Approach		353	3.6	0.633	22.4	LOS C	3.7	26.8	1.00	1.51	3.20	4.5
West: Industry zone												
10	L2	47	3.0	0.219	14.5	LOS A	0.8	4.9	0.99	1.29	2.26	7.1
11	T1	79	2.0	0.382	18.7	LOS A	1.6	11.7	1.00	1.36	2.57	4.7
12	R2	16	1.5	0.382	18.6	LOS A	1.6	11.7	1.00	1.36	2.57	4.7
Approach		142	2.3	0.382	17.3	LOS C	1.6	11.7	1.00	1.34	2.46	6.1
All Vehicles		927	4.2	0.948	34.2	LOS D	9.2	67.1	1.00	1.58	3.51	3.5

MOVEMENT SUMMARY

Site: 10 [Diamond Cafe]

New Site

Site Category: Existing

Stop (All-Way)

Movement Performance - Vehicles

Mov ID	Turn	Demand Flows Total veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Prop. Queued	Effective Stop Rate	Aver. No. Cycles	Average Speed km/h
South: Alem Hotel												
1	L2	78	0.0	0.254	16.2	LOS A	1.0	6.8	0.95	1.30	2.28	26.0
2	T1	38	0.5	0.219	16.1	LOS A	0.8	4.8	0.97	1.29	2.23	26.2
3	R2	21	1.0	0.219	16.0	LOS A	0.8	4.8	0.97	1.29	2.23	26.2
Approach		137	0.3	0.254	16.1	LOS C	1.0	6.8	0.96	1.30	2.26	26.1
East: Gendagara												
4	L2	31	2.0	0.101	13.0	LOS A	0.3	2.4	0.93	1.25	2.04	29.1
5	T1	153	0.5	0.560	24.5	LOS A	2.9	20.6	1.00	1.49	3.13	20.5
6	R2	37	1.0	0.560	24.4	LOS A	2.9	20.6	1.00	1.49	3.13	20.5
Approach		220	0.8	0.560	22.9	LOS C	2.9	20.6	0.99	1.45	2.98	21.4
North: Astu												
7	L2	13	0.0	0.090	20.5	LOS A	0.3	2.2	1.00	1.25	2.09	22.9
8	T1	11	0.0	0.629	49.9	LOS A	3.5	24.5	1.00	1.51	3.26	12.3
9	R2	91	0.0	0.629	49.8	LOS A	3.5	24.5	1.00	1.51	3.26	12.3

Approach		114	0.0	0.629	46.6	LOS E	3.5	24.5	1.00	1.48	3.13	12.9
West: 04												
10	L2	112	0.0	0.307	14.0	LOS A	1.2	8.5	0.94	1.32	2.36	27.1
11	T1	102	0.0	0.302	14.6	LOS A	1.2	8.2	0.92	1.32	2.33	27.5
12	R2	20	0.0	0.302	14.5	LOS A	1.2	8.2	0.92	1.32	2.33	27.5
Approach		234	0.0	0.307	14.8	LOS B	1.2	8.5	0.93	1.32	2.34	27.3
All Vehicles		704	0.3	0.629	22.7	LOS C	3.5	24.5	0.96	1.38	2.65	21.4

APPENDIX-D

(Delay (control) of Each Intersections)

DELAY (CONTROL) **Site: 01 [Derartu Tulu]**

Average control delay per vehicle, or average pedestrian delay (seconds)

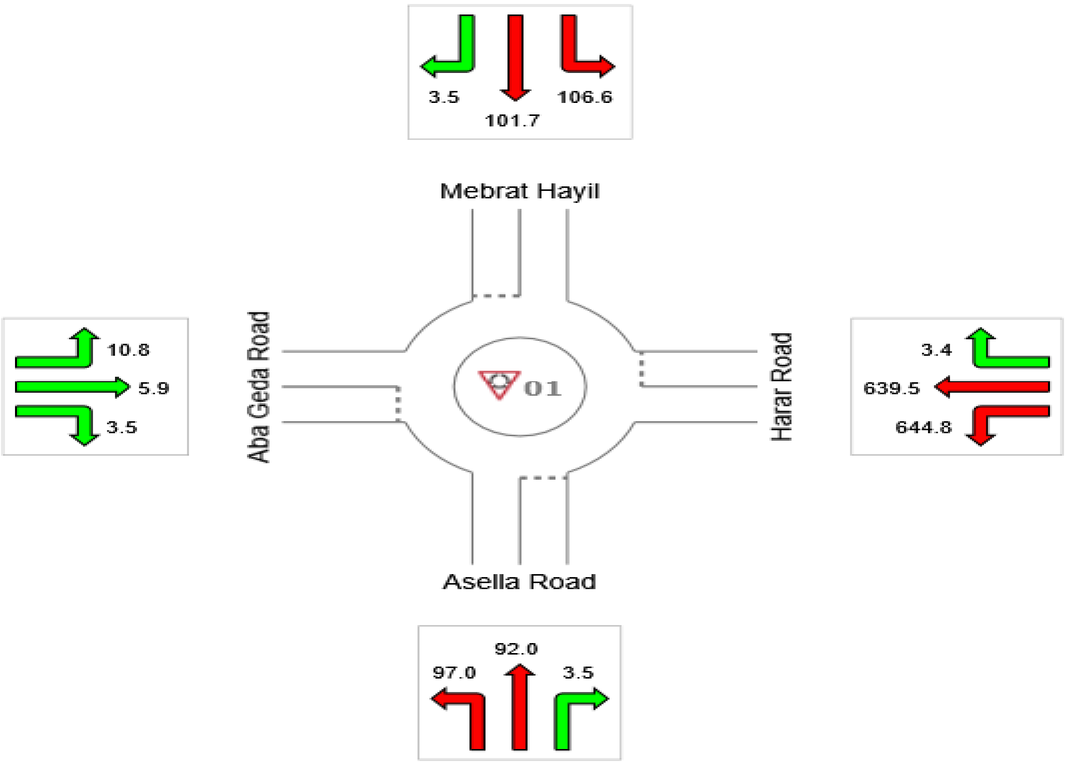
New Site

Site Category: Existing

Roundabout

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	68.3	497.9	42.0	6.7	147.0
LOS	E	F	D	A	F



Colour code based on Level of Service

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

DELAY (CONTROL)

Site: 02 [Genda Gara]

Average control delay per vehicle, or average pedestrian delay (seconds)

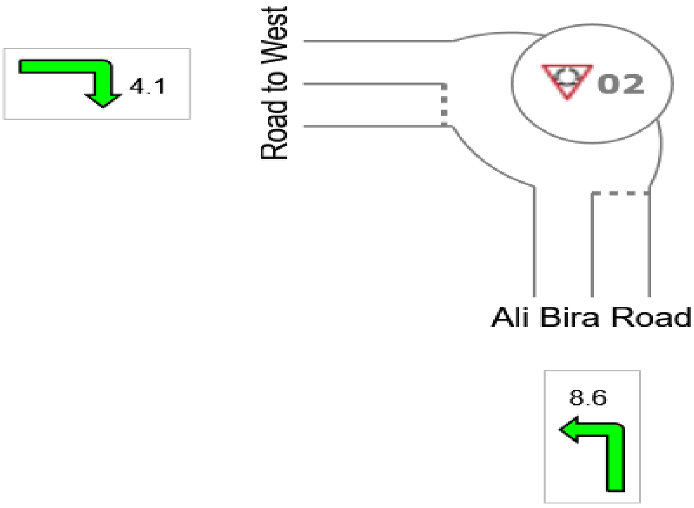
New Site

Site Category: Existing

Roundabout

All Movement Classes

	Approaches		Intersection
	South	West	
Delay (Control)	8.6	4.1	6.5
LOS	A	A	A



Colour code based on Level of Service

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

DELAY (CONTROL)

Site: 03 [Medanihalem Church]

Average control delay per vehicle, or average pedestrian delay (seconds)

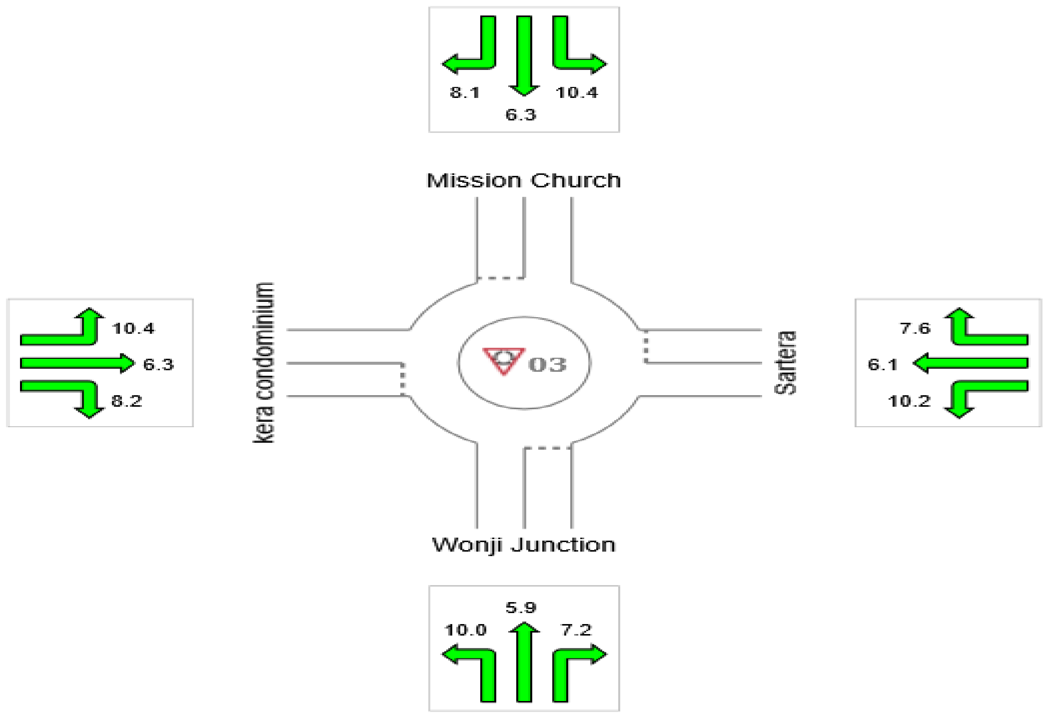
New Site

Site Category: Existing

Roundabout

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	7.8	7.9	8.0	8.8	8.1
LOS	A	A	A	A	A



Colour code based on Level of Service

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

DELAY (CONTROL)

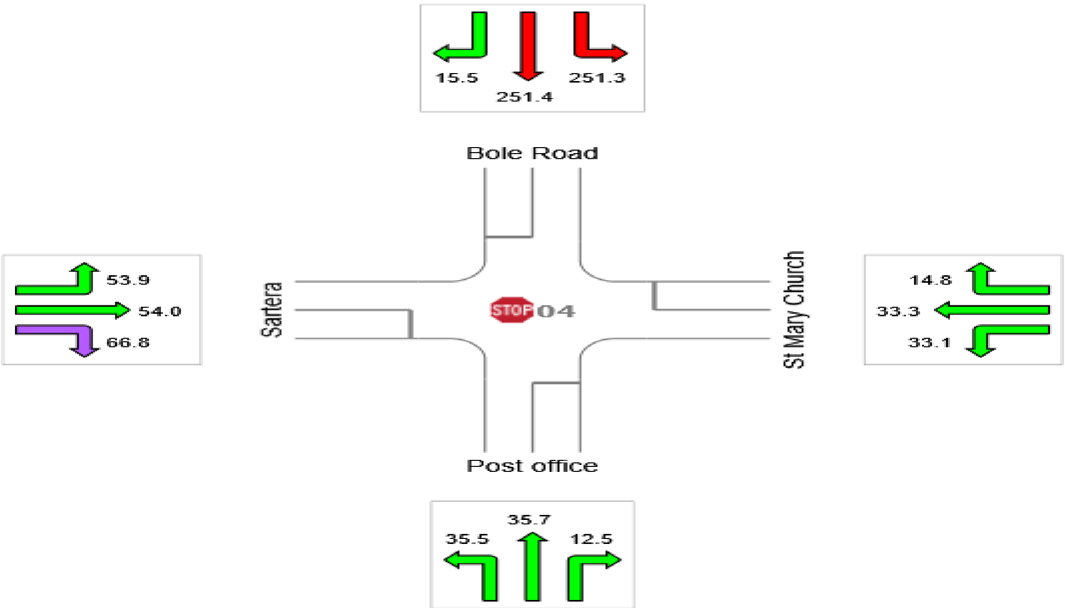
Site: 04 [Mebrat Hayil]

Average control delay per vehicle, or average pedestrian delay (seconds)

New Site
Site Category: Existing
Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	32.6	29.7	219.6	61.1	104.0
LOS	D	D	F	F	F



Colour code based on Level of Service

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

DELAY (CONTROL)

Site: 05 [Sartera]

Average control delay per vehicle, or average pedestrian delay (seconds)

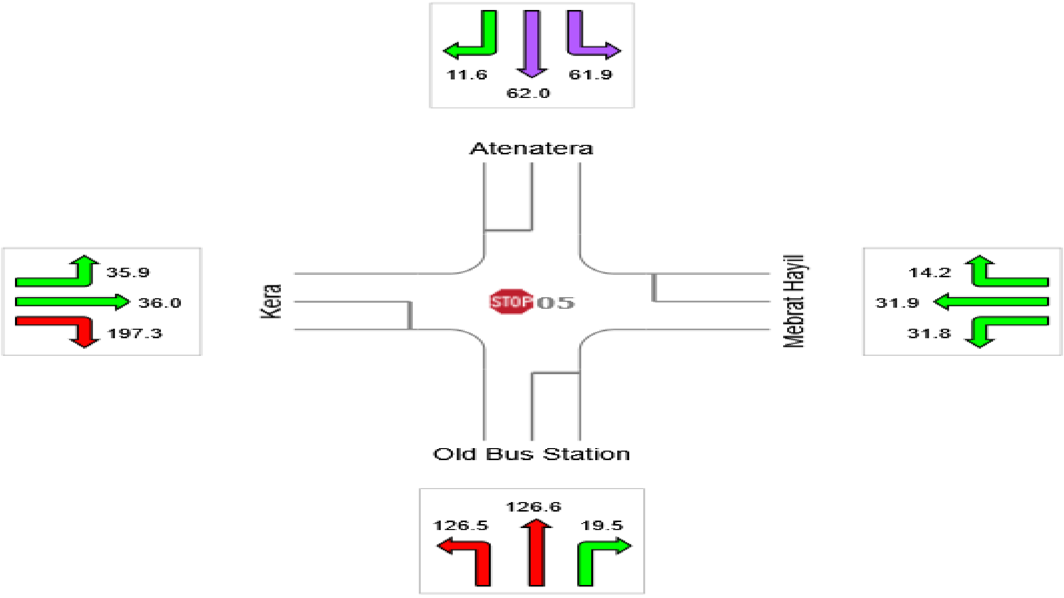
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	98.7	28.5	59.3	153.0	89.3
LOS	F	D	F	F	F



Colour code based on Level of Service

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

DELAY (CONTROL)

Site: 06 [Wonji Junction]

Average control delay per vehicle, or average pedestrian delay (seconds)

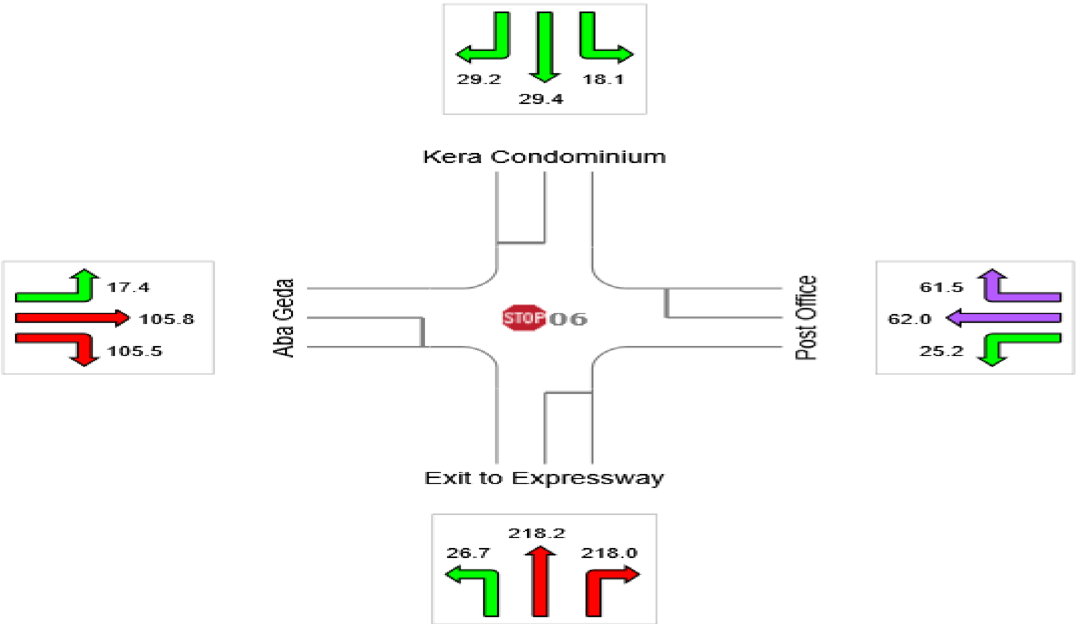
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	171.8	48.7	26.9	84.7	93.5
LOS	F	E	D	F	F



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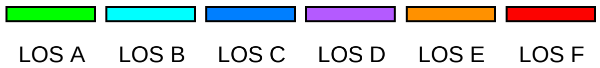


DELAY (CONTROL)

Site: 07 [Tikur Abay Hotel]

Average control delay per vehicle, or average pedestrian delay (seconds)

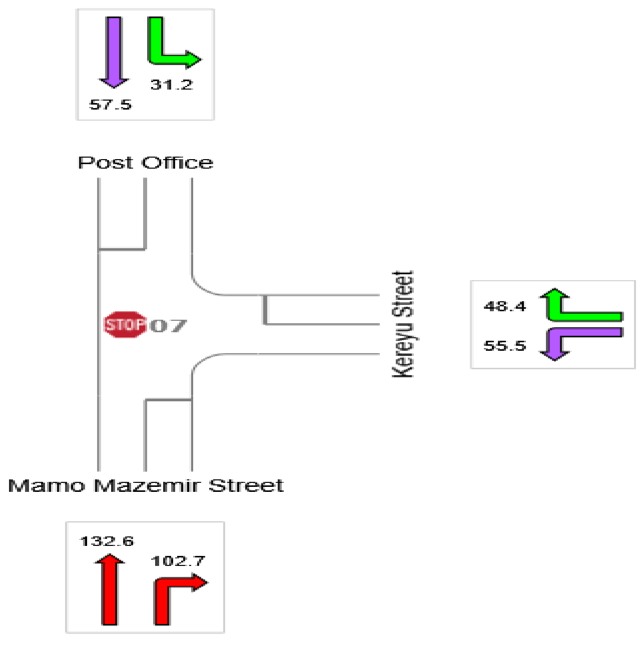
New Site

Site Category: Existing

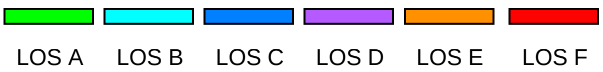
Stop (All-Way)

All Movement Classes

	Approaches			Intersection
	South	East	North	
Delay (Control)	119.1	52.3	46.7	73.8
LOS	F	F	E	F



Colour code based on Level of Service



DELAY (CONTROL)

Site: 08 [Alem Hotel]

Average control delay per vehicle, or average pedestrian delay (seconds)

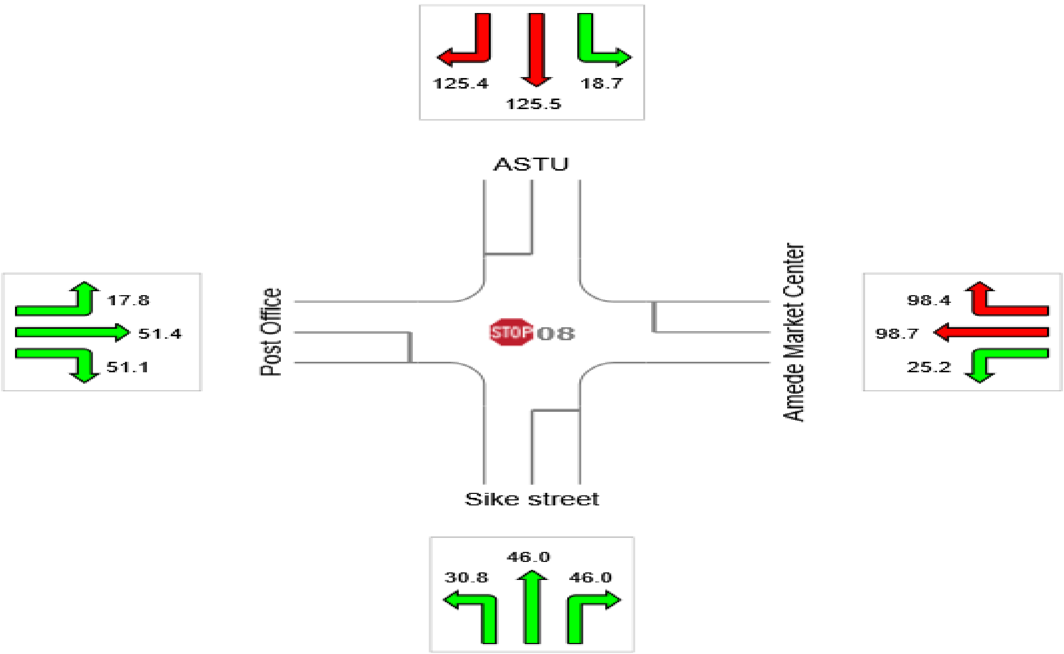
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	40.1	74.8	116.2	44.0	68.2
LOS	E	F	F	E	F



Colour code based on Level of Service

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DELAY (CONTROL)

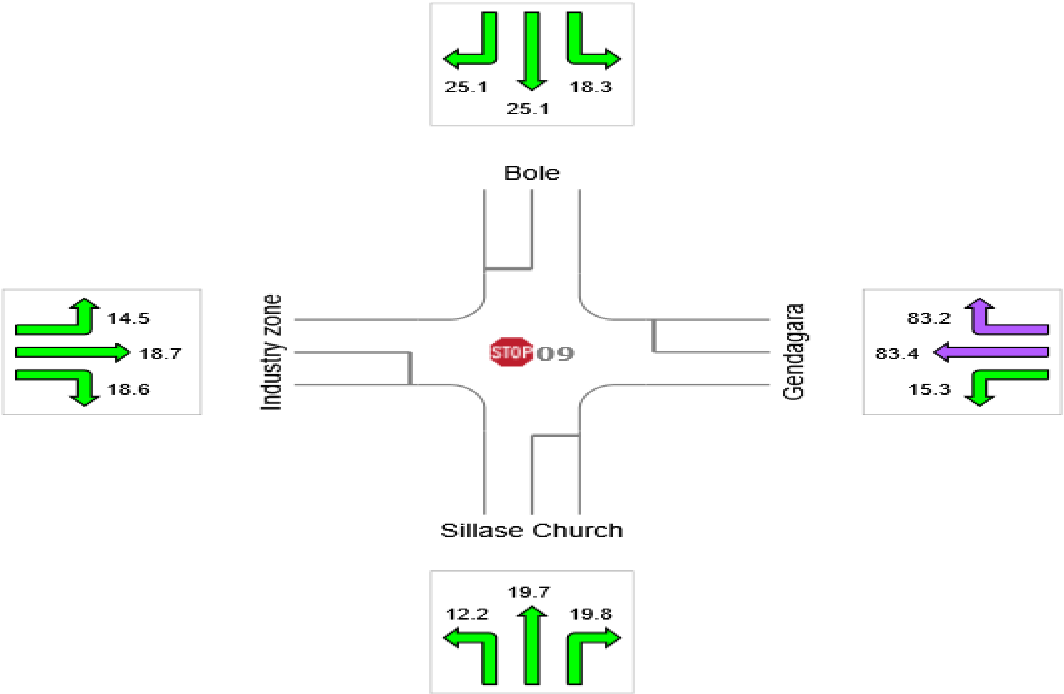
STOP Site: 09 [Geda Resort]

Average control delay per vehicle, or average pedestrian delay (seconds)

New Site
Site Category: (None)
Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	17.8	73.0	22.4	17.3	34.2
LOS	C	F	C	C	D



Colour code based on Level of Service

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

DELAY (CONTROL)

Site: 10 [Diamond Cafe]

Average control delay per vehicle, or average pedestrian delay (seconds)

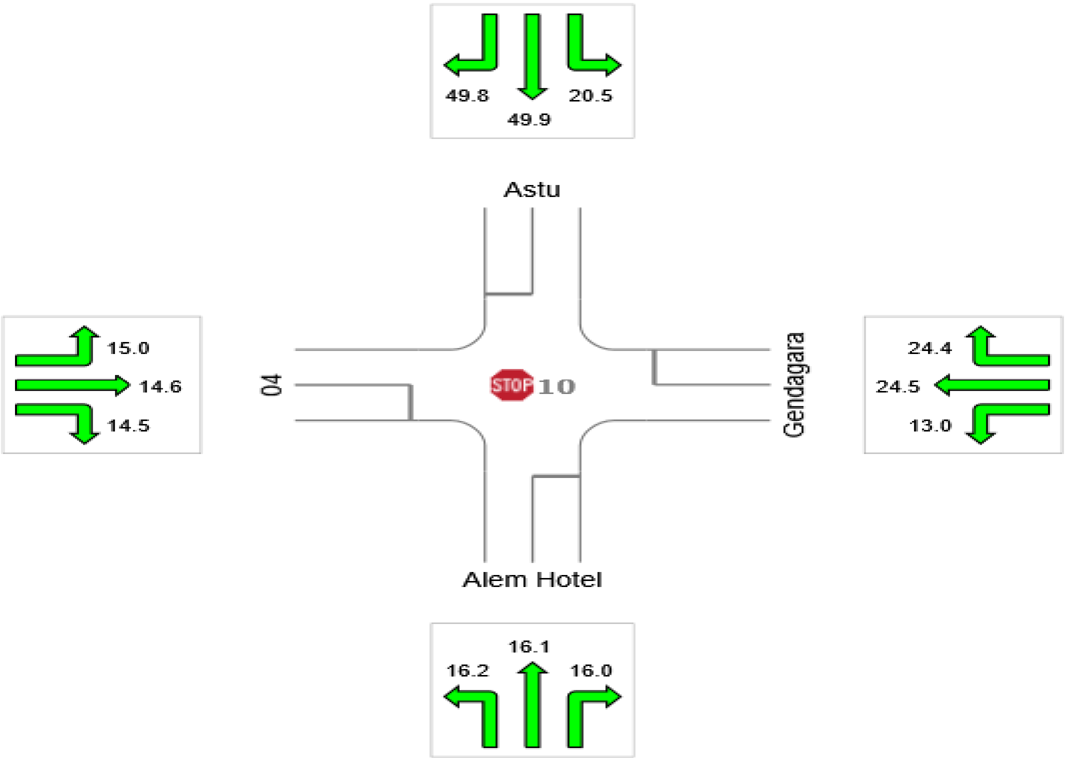
New Site

Site Category: Existing

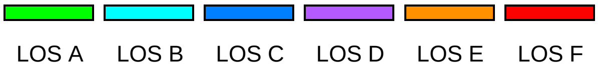
Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	16.1	22.9	46.6	14.8	22.7
LOS	C	C	E	B	C



Colour code based on Level of Service



APPENDIX-E

(Degree of Saturation of Each Intersections)

DEGREE OF SATURATION

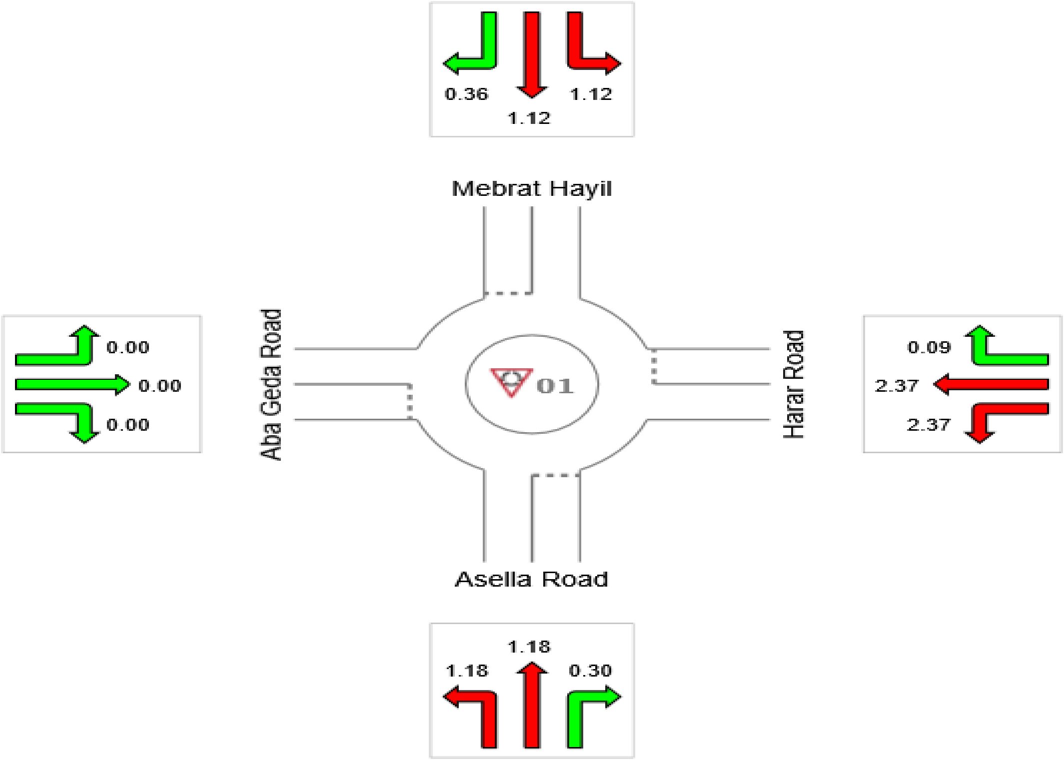
Site: 01 [Derartu Tulu]

Ratio of Demand Volume to Capacity (v/c ratio)

New Site
Site Category: Existing
Roundabout

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	1.18	2.37	1.12	0.00	2.37



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

Site: 03 [Medanihalem Church]

Ratio of Demand Volume to Capacity (v/c ratio)

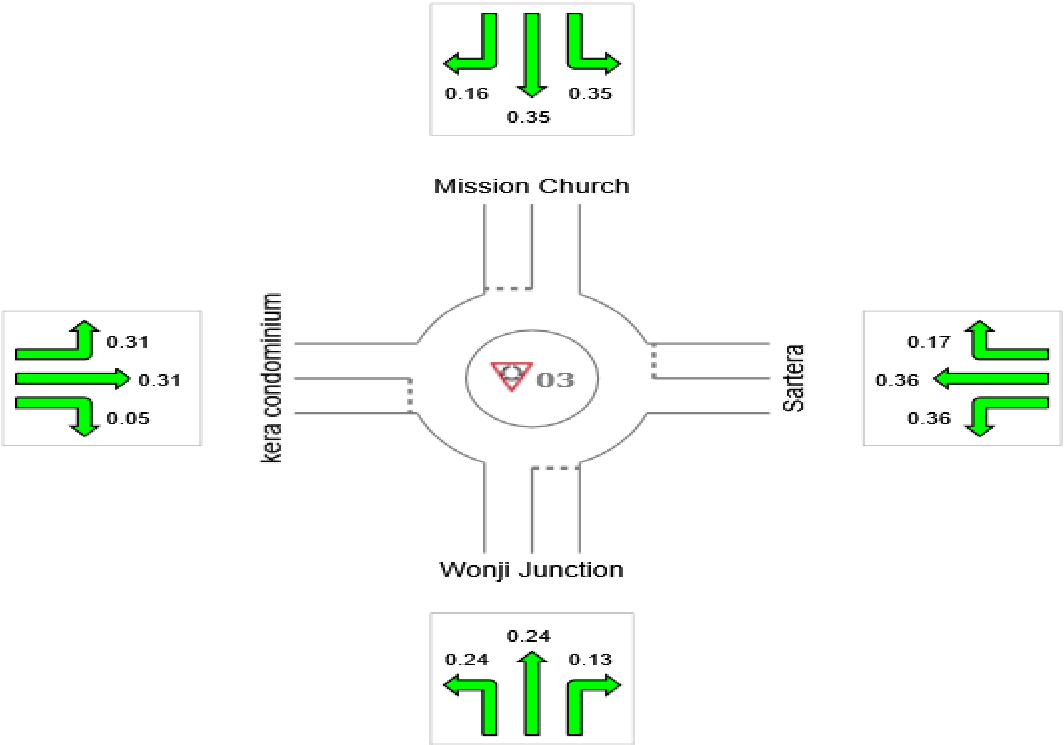
New Site

Site Category: Existing

Roundabout

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	0.24	0.36	0.35	0.31	0.36



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

Site: 04 [Mebrat Hayil]

Ratio of Demand Volume to Capacity (v/c ratio)

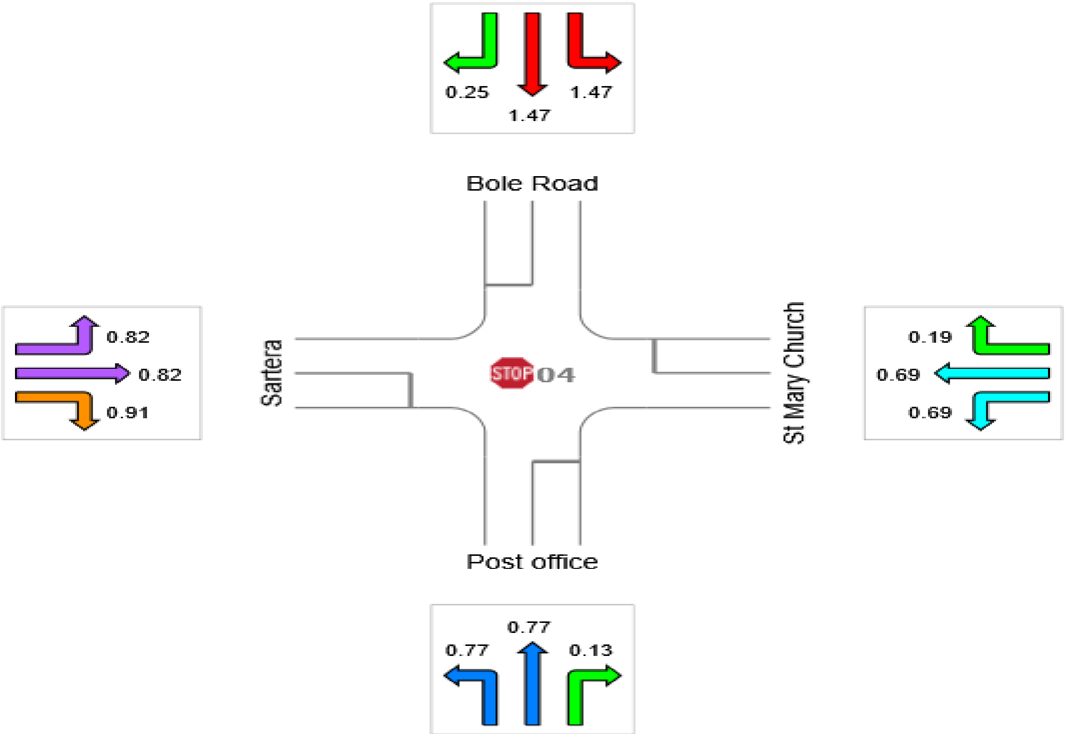
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	0.77	0.69	1.47	0.91	1.47



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

Site: 05 [Sartera]

Ratio of Demand Volume to Capacity (v/c ratio)

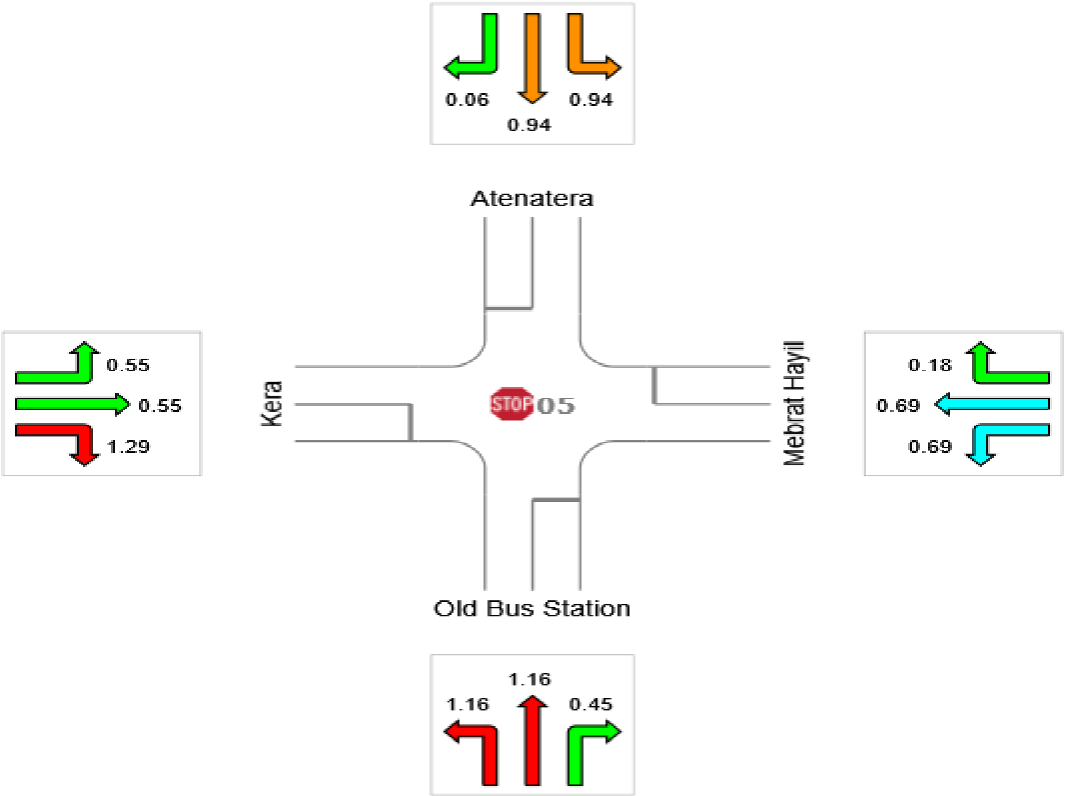
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	1.16	0.69	0.94	1.29	1.29



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

Site: 06 [Wonji Junction]

Ratio of Demand Volume to Capacity (v/c ratio)

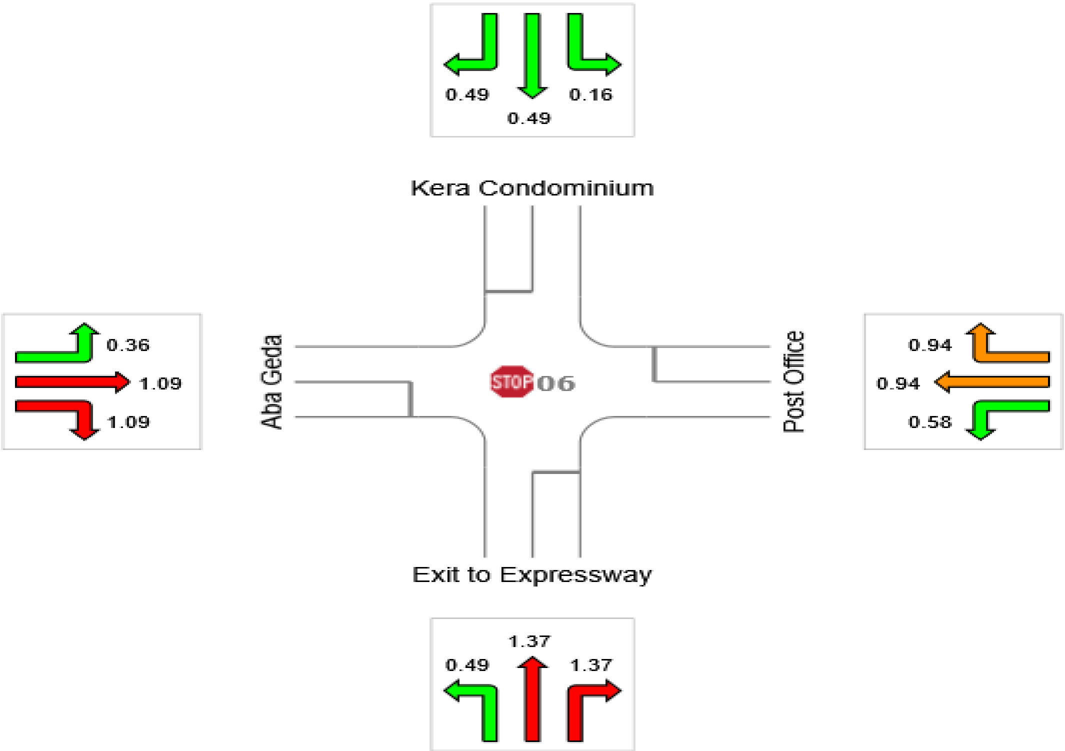
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	1.37	0.94	0.49	1.09	1.37



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

Site: 07 [Tikur Abay Hotel]

Ratio of Demand Volume to Capacity (v/c ratio)

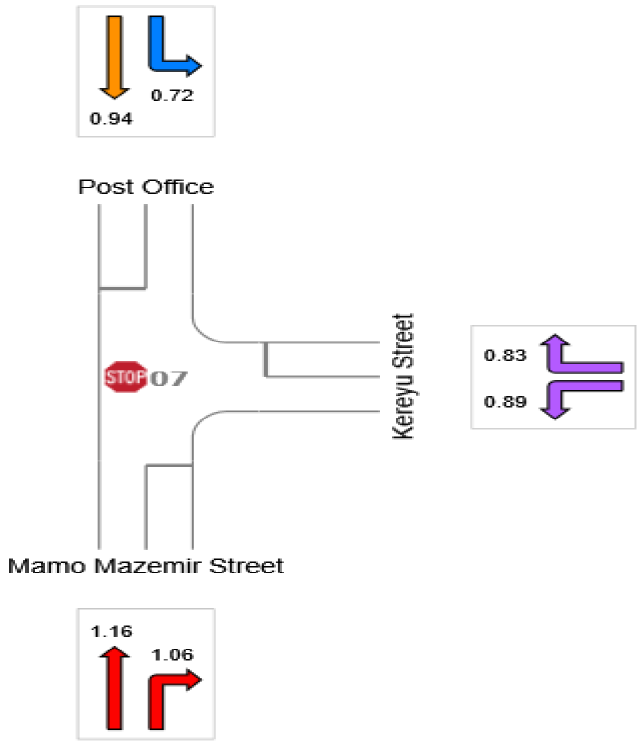
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches			Intersection
	South	East	North	
Degree of Saturation	1.16	0.89	0.94	1.16



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

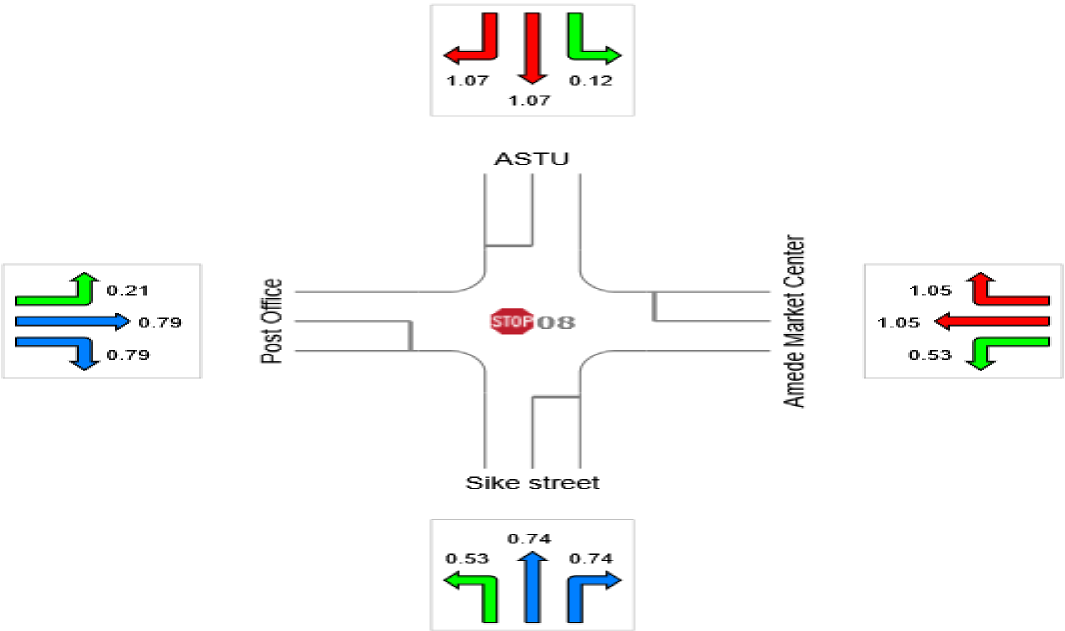
Site: 08 [Alem Hotel]

Ratio of Demand Volume to Capacity (v/c ratio)

New Site
Site Category: Existing
Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	0.74	1.05	1.07	0.79	1.07



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

Site: 09 [Geda Resort]

Ratio of Demand Volume to Capacity (v/c ratio)

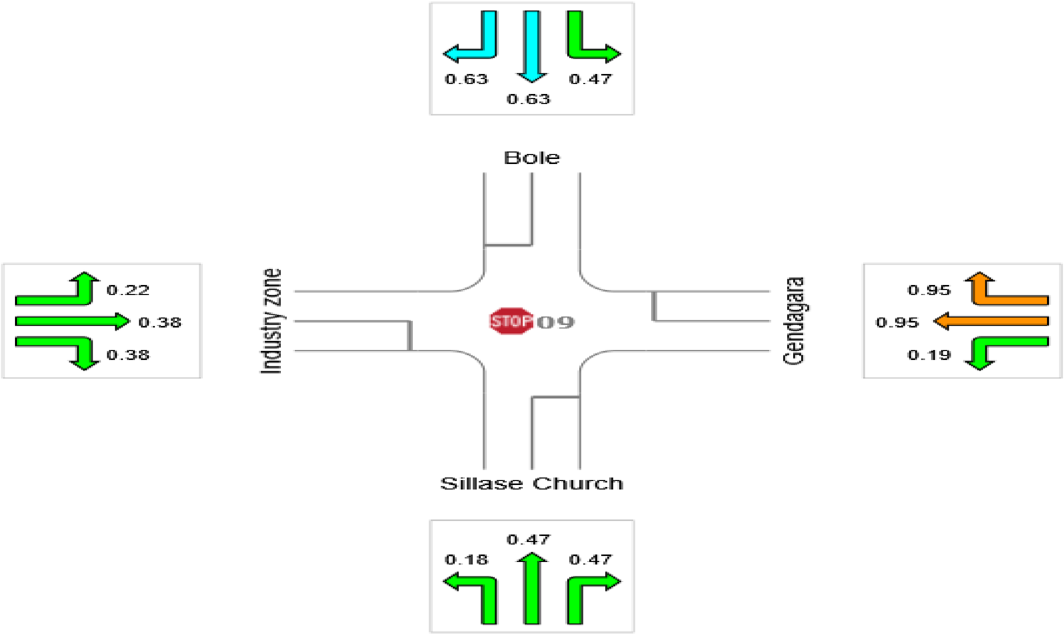
New Site

Site Category: (None)

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	0.47	0.95	0.63	0.38	0.95



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]

DEGREE OF SATURATION

Site: 10 [Diamond Cafe]

Ratio of Demand Volume to Capacity (v/c ratio)

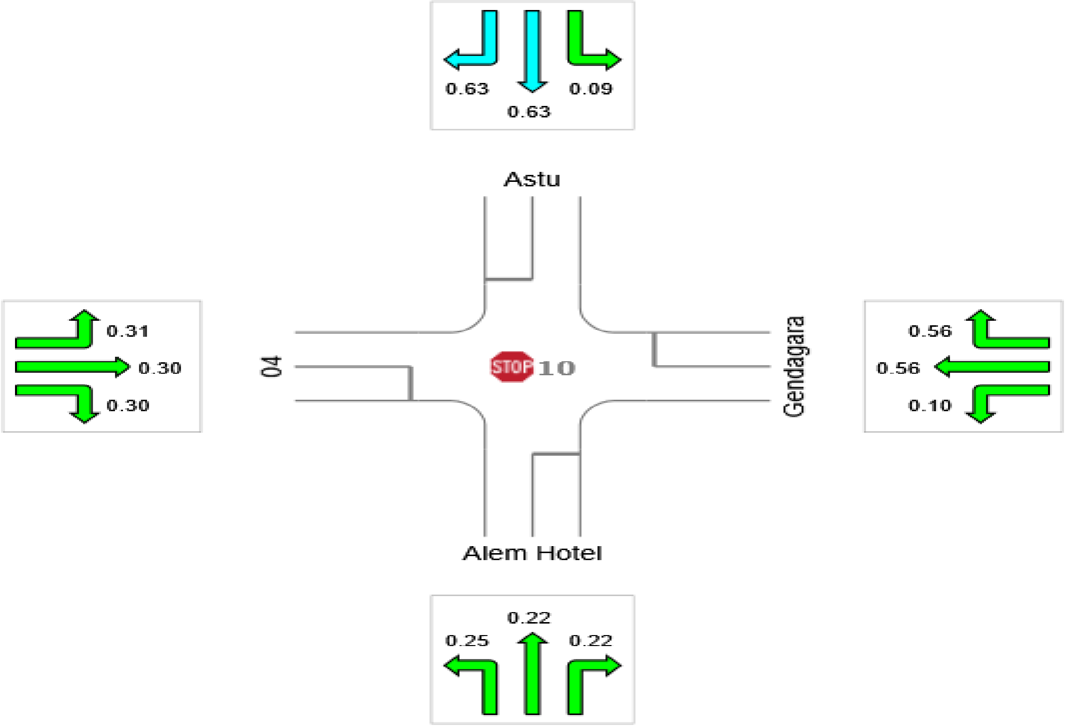
New Site

Site Category: Existing

Stop (All-Way)

All Movement Classes

	Approaches				Intersection
	South	East	North	West	
Degree of Saturation	0.25	0.56	0.63	0.31	0.63



Colour code based on Degree of Saturation

[< 0.6] [0.6 – 0.7] [0.7 – 0.8] [0.8 – 0.9] [0.9 – 1.0] [> 1.0]