Performance Evaluation of selected intersection in Jimma city

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Performance analysis of selected intersection in Jimma city

A Project submitted to the department of Civil Engineering of Jimma University in Partial fulfillment of the requirements for the BSc Degree in Civil Engineering

A Project submitted to Civil Engineering department

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Jimma, Ethiopia

DECLARATION

We declare that this document is the independent outcome of our group and all sources of materials used in this document have been duly acknowledged. We again honestly declare that this Research is not submitted to any other institution anywhere for the awards of any practical work.

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Abstract

Intersection plays an important role in the road network, where traffic flowing in different directions cross each other. Because of disturbance of pedestrians, mixed traffic and lost time, capacity of the intersection is much lower than their approach links and it is significant point of conflicts within a roadway system. Traffic signals, roundabouts, stop and yield controls are commonly used in several at grade junction in urban areas to maximize traffic efficiency and safety by separating conflicting traffic movements in time. The performance of intersections are evaluated in terms of capacity, degree of saturation, queue length, delay and average speed. And estimation of these performance measures to evaluate the intersection is important for design, operations and planning purposes in

traffic management as well as in improving the performance of intersections in urban areas.

In Jimma city most of the intersections are closely spaced with high pedestrian and other motorized vehicular traffics. Because of this their performance is highly affected and are prone to vehicle to vehicle and vehicle to pedestrian accidents.

The objective of this study is to evaluate the performance of roadway intersection in Jimma city which is located around merkato and to study the special effect of pedestrians, and other motorized vehicles in the performance of the given intersection.

Traffic and geometric data for the selected intersection have been collected. And the result of this research shows high magnitude of degree of saturation, delay and queue length and low magnitude of capacity for the selected intersection in the city. Volume of pedestrians and Bajaj traffics are the main causes for this.

In Jimma as well as in other big cities of Ethiopia's regional states, number of Tricycles should be controlled and the number other public transports like buses and minibuses should be increased to improve the performance of major intersections in these cities. Based on the results of the analysis and the forecasted traffic volume, we have proposed a roundabout as a solution for the existing intersection. In addition to this, parking areas for vehicles and a separate way for pedestrians to cross the road at the intersection areas should be prepared to increase the performance of the intersection.

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Acronyms

AACRA-Addis Ababa City Road Authority

AASHTO-American associate state Highway and Transportation Officials

ERA-Ethiopian Road Authority

HCM-Highway Capacity Manual

IRC-Indian Road Congress

MUTCO-Manual on Uniform Traffic Control Device

PCU-Passenger Car Units

PHF-Peak Hour Factor

TRL-Transportation Road Research Lab

TWLTL-Two way Left turn lane

TWSC-Two Way Stop Control

CHAPTER ONE

INTRODUCTION

1.1 Background

Traffic congestion on urban roads results in tremendous economic loss, additional delay and user cost. Intersections play an important role in the road network, where traffic flows in different directions converge. Because of disturbance of pedestrians, mixed traffic and lost time for beginning and clearance etc., capacity or the maximum rate of flow at which persons or vehicles can reasonably expected to traverse an intersection is much lower than their approach links. Thus, intersections are usually the bottleneck of the network and are the greatest and immediate source of traffic accidents. Hence, level of service at intersection significantly affects the overall level of service of the road. The critical aspect of increasing capacity of any road lies in increasing capacity of the intersection. Traffic signals, roundabouts, stop and yield controls are commonly used in several at grade junction in urban areas to maximize traffic efficiency and safety by separating conflicting traffic movements in time [2]. And Intersection performance measures (capacity, degree of saturation, delay and queue length etc.) used to evaluate the intersection performance [1] appropriately at these days based on different manuals.

Capacity which is the maximum sustainable flow rate that can be achieved during a specified time period under a given (prevailing) road, traffic and control conditions and degree of Saturation or the ratio of arrival (demand) flow rate to capacity during a given flow period are relevant measure of performance of intersections. Delay which is additional travel time experienced by a vehicle or pedestrian with reference to a base travel time is also one of the measure performances of intersections.

Among others queue length is a more useful performance measure of intersections because it is relevant to the design of appropriate queuing space, e.g. for short lane design to avoid queue spillback into adjacent lanes, for phasing design to avoid blockage of upstream signals in paired intersection situations, and for signal coordination offset design to prevent interruption of platoons by downstream queue.

Jimma is the largest city in South – Western Ethiopia with traffic composition of pedestrian, bicycle, and motorized vehicles such as Bajaj, taxi, bus, truck etc. The city has several straight roadways with a number of different intersection controls such as roundabouts. Like all others the performance of intersections in the city would be evaluated in terms of capacity, degree of saturation, average queue length, maximum queue length and delay.

1.2 Statement of the problem

Most of the researches related to performance evaluation of intersection in Ethiopia are focused only on highly congested junction areas in Addis Ababa. And there is no sufficient study on the performance of intersections in other cities of the country to handle the problem of congestion, delay and accident in the intersection areas before they become congested like in Addis Ababa. And studies on the performance of intersection for Jimma city will be important to maximize traffic efficiency and safety in the intersection for future use.

Most of the intersections in the major road ways of Jimma city are closely spaced with high traffic of pedestrians and motorized vehicle such as Tricycle, taxi, bus, truck etc. in the peak hours so that their performance is highly affected by it. In addition to this, growing vehicular and pedestrian traffic in the city may also reduce performance of the intersections more than ever in the near future.

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Figure 1.1 Intersection under congestion

1.3 Objective

1.3.1 General objective

The general objective of this study is to evaluate the performance (capacity, degree of saturation, delay, queue length, and level of service) of roadway intersection in Jimma city.

1.3.2 Specific objective

Specific objectives of this study are:

- To evaluate the performance of the selected intersection that is located around Merkato in Jimma city.
- To study the special effect of pedestrians and Vehicles in the performance of intersections. Depending on the result of analysis to give recommendation in designing as well as improving the performance of the intersection for the future in the city.

1.4 Significance of the Study

The performance of intersections in Jimma city is not studied until now and the result of this research will be important for designing and improving the performance of intersections in the future for the city. And also it will be important for design, operations and planning purposes in traffic management in the city as well as for other cities which have similar traffic and topographic conditions in Ethiopia in general. It also can serve as a teaching material for future students who are interested in the design and analysis of intersections. The significance of the study can be stated as follows:

- The study is carried on a particular area, it could be helpful to have a deeper knowledge about the traffic analysis in general and intersection in particular.
- The findings obtained from the study would be helpful to gain information and knowledge about the analysis and design of intersections.
- Finally, it also helps to carry out further research to refine the conceptual and methodology of the present study.

1.5 Scope of the Study

The scope of this study is mainly concerned with evaluation of performance of the traffic intersection in Jimma town around Merkato in front of Geda supermarket.

1.6. Limitation

One of the main objectives of this thesis is to assess and evaluate the performance of the traffic intersection within Merkato area. To do such kind of research, one will obviously need tremendous amount of traffic and geometrical data. However, due to lack of previous traffic data, we were forced to limit our research based on recently collected traffic data.

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CHAPTER TWO

LITERATURE REVIEW

2.1 Intersection traffic control

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/her route, intersection geometry, speed and direction of other vehicles etc. A small error in judgment can cause severe accidents. It also causes delay and it depends on type, geometry, and type of control. Overall traffic flow depends on the performance of the intersection. It also affects the capacity of the road. Therefore, both from the accident perspective and the capacity perspective, the study of intersections are very important for traffic engineers especially in the case of urban scenario [3].

2.1.1 Conflicts and levels of control at an intersection

Conflicts at an intersection are different for different types of intersection. Consider a typical four-legged intersection as shown in figure 2.1. The numbers 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 [4]. This is shown in figure 2.1

The essence of the intersection control is to resolve these conflicts at the intersection for the safe and efficient movement of both vehicular traffic and pedestrians.

Two methods of intersection controls are there: time sharing and space sharing. The type of intersection control that has to be adopted depends on the traffic volume, road geometry, cost involved, importance of the road etc.

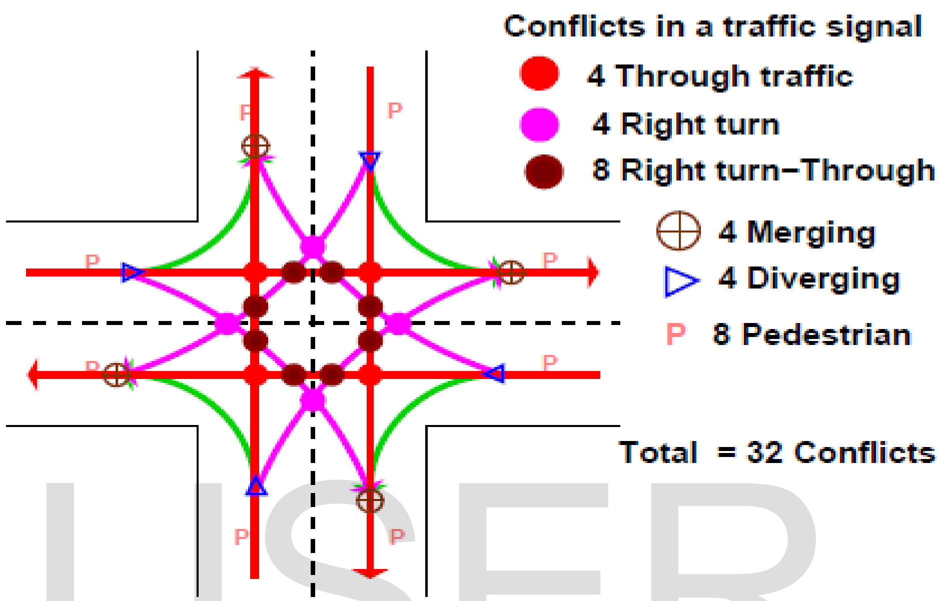

Figure 2.1: Conflicts at an intersection

The control of intersection can be exercised at different levels. They can be either passive control, semi control, or active control. In passive control there is no explicit control on the driver. In semi control, some amount of control on the driver is there from the traffic agency. Active control means the movement of the traffic is fully controlled by the traffic agency and the drivers cannot simply cross the intersection according to his choice.

2.1.2 Passive Control

When the volume of traffic is less, no explicit control is required. Here the road users are required to obey the basic rules of the road. Passive control like traffic signs, road markings etc. are used to complement the intersection control [4]. Some of the intersection control that are classified under passive control are as follows:

2.1.2.1 No control

If the traffic coming to an intersection is low, then by applying the basic rules of the road like driver on the left side of the road must yield and that through movements will have priority than turning movements. The driver is expected to obey these basic rules of the road.

2.1.2.2 Traffic signs

With the help of warning signs, guide signs etc. it is able to provide some level of control at an intersection. Give way control, two-way stop control, and all-way stop control are some examples. The GIVE WAY control requires the driver in the minor road to slow down to a minimum speed and allow the vehicle on the major road to proceed. Two way stop control requires the vehicle drivers on the minor streets should see that the conflicts are avoided. Finally an all-way stop control is usually used when it is difficult to differentiate between the major and minor roads in an intersection. In such a case, STOP sign is placed on all the approaches to the intersection and the driver on all the approaches are required to stop the vehicle. The vehicle at the right side will get priority over the left approach [3]. The traffic control at 'at-grade' intersection may be uncontrolled in cases of low traffic. Here the road users are required to obey the basic rules of the road. Passive control like traffic signs, road markings etc. are used to complement the intersection control.

2.1.2.3 Traffic signs plus marking

In addition to the traffic signs, road markings also complement the traffic control at intersections. Some of the examples include stop line marking, yield lines, arrow marking etc.

2.1.3 Semi control

In semi control or partial control, the drivers are gently guided to avoid conflicts. Channelization and traffic rotaries are examples of such type of controlling mechanisms

2.1.3.1 Channelization

The traffic is separated to flow through definite paths by raising a portion of the road in the middle usually called as islands distinguished by road markings. The conflicts in traffic movements are reduced to a great extent in such a case.

In channelized intersections, as the name suggests, the traffic is directed to flow through different channels and this physical separation is made possible with the help of some barriers in the road like traffic islands, road markings etc.

2.1.3.2 Traffic rotaries

Overview

Rotary intersections or round about are special form of at-grade intersections laid out for the movement of traffic in one direction around a central traffic island. Essentially all the major conflicts at an intersection namely the [4] collision between through and right-turn movements are converted into milder conflicts namely merging and diverging. The vehicles entering the rotary are gently forced to move in a clockwise direction in orderly fashion. They then weave out of the rotary to the desired direction [5].

Advantages and disadvantages of rotary

The key advantages of a rotary intersection are listed below:

1. Traffic flow is regulated to only one direction of movement, thus eliminating severe conflicts between crossing movements.
2. All the vehicles entering the rotary are gently forced to reduce the speed and continue to move at slower speed. Thus, none of the vehicles need to be stopped, unlike in a signalized intersection.

3. Because of lower speed of negotiation and elimination of severe conflicts, accidents and their severity are much less in rotaries.
4. Rotaries are self-governing and do not need practically any control by police or traffic signals.
5. They are ideally suited for moderate traffic, especially with irregular geometry, or intersections with more than three or four approaches.

Although rotaries offer some distinct advantages, there are few specific limitations for rotaries which are listed below.

1. All the vehicles are forced to slow down and negotiate the intersection. Therefore, the cumulative delay will be much higher than channelized intersection.
2. Even low traffic, the vehicles are forced to reduce their speed.
3. Rotaries require large area of relatively flat land making them costly at urban areas.
4. The vehicles do not usually stop at a rotary. They accelerate and exit the rotary at relatively high speed. Therefore, they are not suitable when there is high pedestrian movements.

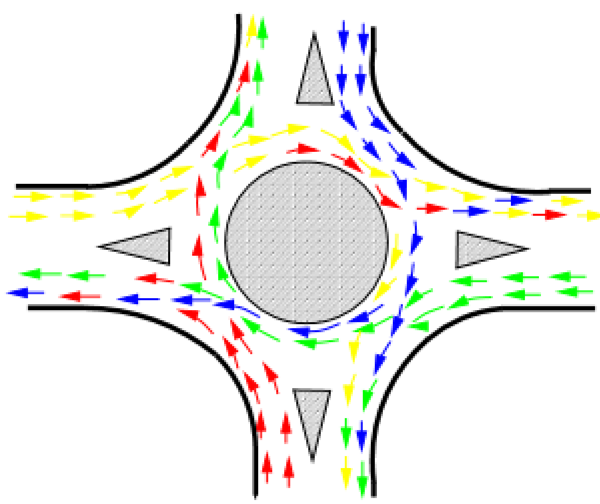

Figure 2.2 Traffic operations in a rotary

Guidelines for the selection of rotaries

Because of the above limitations, rotaries are not suitable for every location. There are few guidelines that help in deciding the suitable of a rotary. They are listed below.

1. Rotaries are suitable when the traffic entering from all the four approaches are relatively equal.
2. A total volume of about 4000 vehicles per hour can be considered as the upper limiting case and a volume of 500 vehicles per hour is the lower limit.
3. A rotary is very beneficial when the proportion of the right-turn traffic is very high; typically if it is more than 30 percent.
4. Rotaries are suitable when there are more than four approaches or if there is no separate lanes available for right-turn traffic. Rotaries are ideally suited if the intersection geometry is complex [3].

Traffic operations in a rotary

As noted earlier, the traffic operations at a rotary are three; diverging, merging and weaving. All the other conflicts are converted into these three less severe conflicts.

1. Diverging: It is a traffic operation when the vehicles moving in one direction is separated into different streams according to their destinations.
2. Merging: Merging is the opposite of diverging. Merging is referred to as the process of joining the traffic coming from different approaches and going to a common destination into a single stream.
3. Weaving: Weaving is the combined movement of both merging and diverging movements in the same direction.

These movements are shown in figure below. It can be observed that movements from each direction split into three; left, straight, and right turn.

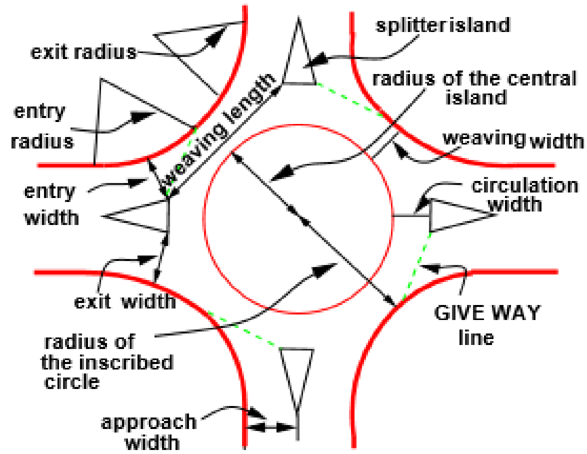

Figure 2.3 Design of a rotary

Design elements

The design elements include design speed, radius at entry, exit and the central island, weaving length and width, entry and exit widths. In addition the capacity of the rotary can also be determined by using some empirical formula.

Design speed

All the vehicles are required to reduce their speed at a rotary. Therefore, the design speed of a rotary will be much lower than the roads leading to it. Although it is possible to design roundabout without much speed reduction, the geometry may lead to very large size incurring huge cost of construction. The normal practice is to keep the design speed as 30 and 40 kmph for urban and rural areas respectively.

Entry, exit and island radius

The radius at the entry depends on various factors like design speed, super-elevation, and coefficient of friction. The entry to the rotary is not straight, but a small curvature is introduced. This will force the driver to reduce the speed. The entry radius of about 20 and 25 meters is ideal for an urban and rural design respectively. The exit radius should be higher than the entry radius and the radius of the [5] rotary island so that the vehicles will discharge from the rotary at a higher rate. A general practice is to keep the exit radius as 1.5 to 2 times the entry radius. However, if pedestrian movement is higher at the exit approach, then the exit radius could be set as same as that of the entry radius. The radius of the central island is governed by the design speed, and the radius of the entry curve. The radius of the central island, in practice, is given a slightly higher radius so that the movement of the traffic already in the rotary will have priority. The radius of the central island which is about 1.3 times that of the entry curve is adequate for all practical purposes.

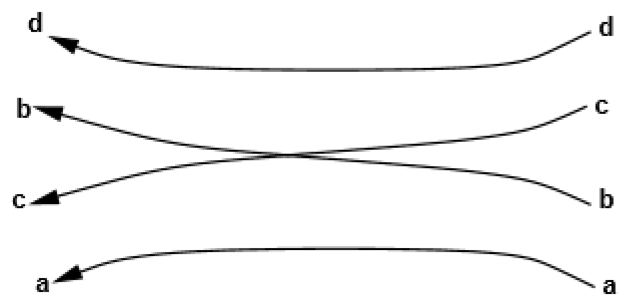

Figure 2.4 Weaving operations in a rotary

Width of the rotary

The entry width and exit width of the rotary is governed by the traffic entering and leaving the intersection and the width of the approaching road. The width of the carriageway at entry and exit will be lower than the width of the carriageway at the approaches to enable reduction of speed. Indian Road Congress (IRC) suggests that a two lane road of 7 m width should be kept as 7 m for urban roads and 6.5 m for rural roads. Further, a three lane road of 10.5 m is to be reduced to 7 m and 7.5 m respectively for urban and rural roads.

The width of the weaving section should be higher than the width at entry and exit. Normally this will be one lane more than the average entry and exit width. Thus weaving width is given as,

$$W_{weaving} = \left(\frac{e_1 + e_2}{2}\right) + 3.5\text{m} \qquad \textbf{Equation 2.1}$$

Where e_1 is the width of the carriageway at the entry and e_2 is the carriageway width at exit. Weaving length determines how smoothly the traffic can merge and diverge. It is decided based on many factors such as weaving width, proportion of weaving traffic to the non-weaving traffic etc. This can be best achieved by making the ratio of weaving length to the weaving width very high. A ratio of 4 is the minimum value suggested by IRC [3]. Very large weaving length is also dangerous, as it may encourage over-speeding.

Capacity

The capacity of rotary is determined by the capacity of each weaving section. Transportation road research lab (TRL) proposed the following empirical formula to find the capacity of the weaving section.

$$Q_w = \frac{280w\left[1+\frac{e}{w}\right]\left[1-\frac{P}{3}\right]}{1+\frac{w}{l}} \qquad \textbf{Equation 2.2}$$

Where e is the average entry and exit width, i.e. $\frac{e_1 + e_2}{2}$, w is the weaving width, l is the length of weaving, and p is the proportion of weaving traffic to the non-weaving traffic [4].

Therefore,

$$\mathrm{P} = \frac{b+c}{a+b+c+d} \qquad \textbf{Equation 2.3}$$

This capacity formula is valid only if the following conditions are satisfied.

1. Weaving width at the rotary is in between 6 and 18 meters.
2. The ratio of average width of the carriage way at entry and exit to the weaving width is in the range of 0.4 to 1.

3. The ratio of weaving width to weaving length of the roundabout is in between 0.12 and 0.4.

4. The proportion of weaving traffic to non-weaving traffic in the rotary is in the range of 0.4 and 1.

5. The weaving length available at the intersection is in between 18 and 90 m.

2.1.4 Active control

Active control implies that the road user will be forced to follow the path suggested by the traffic control agencies. He cannot maneuver according to his wish. Traffic signals and grade separated intersections come under this classification.

2.1.4.1 Traffic signals

Control using traffic signal is based on time sharing approach. At a given time, with the help of appropriate signals, certain traffic movements are restricted where as certain other movements are permitted to pass through the intersection. Two or more phases may be provided depending upon the traffic conditions of the intersection. When the vehicles traversing the intersection is very large, then the control is done with the help of signals. The phases provided for the signal may be two or more. If more than two phases are provided, then it is called multiphase signal [3].

The signals can operate in several modes. Most common are fixed time signals and vehicle actuated signals. In fixed time signals, the cycle time, phases and interval of each signal is fixed. Each cycle of the signal will be exactly like another. But they cannot cater to the needs of the fluctuating traffic. On the other hand, vehicle actuated signals can respond to dynamic traffic situations. Vehicle detectors will be placed on the streets approaching the intersection and the detector will sense the presence of the vehicle and pass the information to a controller. The controller then sets the cycle time and adjusts the phase lengths according to the prevailing traffic conditions [5].

2.1.4.2 Grade separated intersections

The intersections are of two types. They are at-grade intersections and grade-separated intersections. In at-grade intersections, all roadways join or cross at the same vertical level. Grade separated intersections allows the traffic to cross at different vertical levels. Sometimes the topography itself may be helpful in constructing such intersections. Otherwise, the initial construction cost required will be very high. Therefore, they are usually constructed on high speed facilities like expressways, freeways etc. These type of intersection increases the road capacity because vehicles can flow with high speed and accident potential is also reduced due to vertical separation of traffic.

As we discussed earlier, grade-separated intersections are provided to separate the traffic in the vertical grade. But the traffic need not be those pertaining to road only. When a railway line crosses a road, then also grade separators are used. Different types of grade-separators are flyovers and interchange. Flyovers itself are subdivided into overpass and underpass. When two roads cross at a point, if the road having major traffic is elevated to a higher grade for further movement of traffic, then such structures are called overpass. Otherwise, if the major road is depressed to a lower level to cross another by means of an under bridge or tunnel, it is called under-pass. Interchange is a system where traffic between two or more roadways flows at different levels in the grade separated junctions. Common types of interchange include trumpet interchange, diamond interchange, and cloverleaf interchange [3].

1. **Trumpet interchange**: Trumpet interchange is a popular form of three leg interchange. If one of the legs of the interchange meets a highway at some angle but does not cross it, then the interchange is called trumpet interchange. A typical layout of trumpet interchange [3] is shown in figure below.

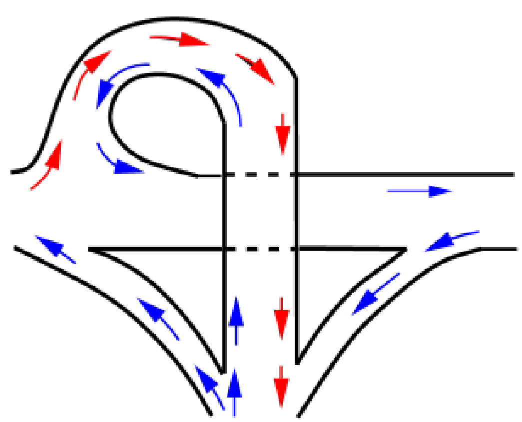

Figure 2.5 Trumpet Interchange

2. Diamond interchange: Diamond interchange is a popular form of four-leg interchange found in the urban locations where major and minor roads crosses. The important feature of this interchange is that it can be designed even if the major road is relatively narrow. A typical layout of diamond interchange [3] is shown in the figure below.

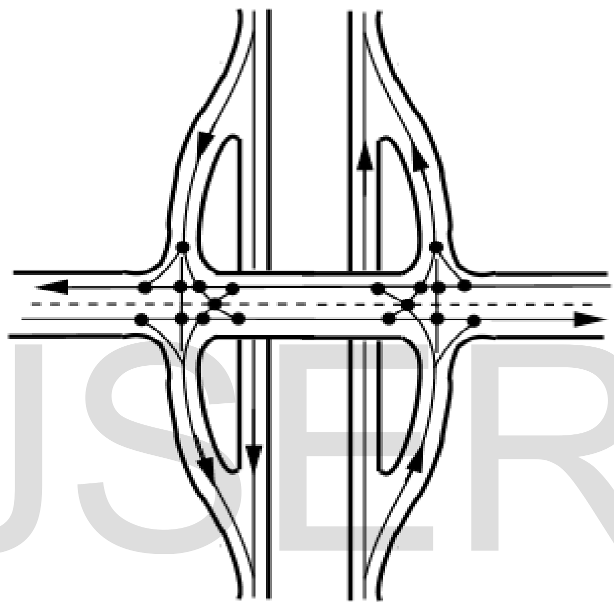

Figure 2.6: Diamond Interchange

3. Clover leaf interchange: It is also a four leg interchange and is used when two highways of high volume and speed intersect each other with considerable turning movements. The main advantage of cloverleaf intersection is that it provides complete separation of traffic. In addition, high speed at intersections can be achieved. However, the disadvantage is that large area of land is required. Therefore, cloverleaf interchanges are provided mainly in rural areas [3]. A typical layout of this type of interchange is shown in the figure below

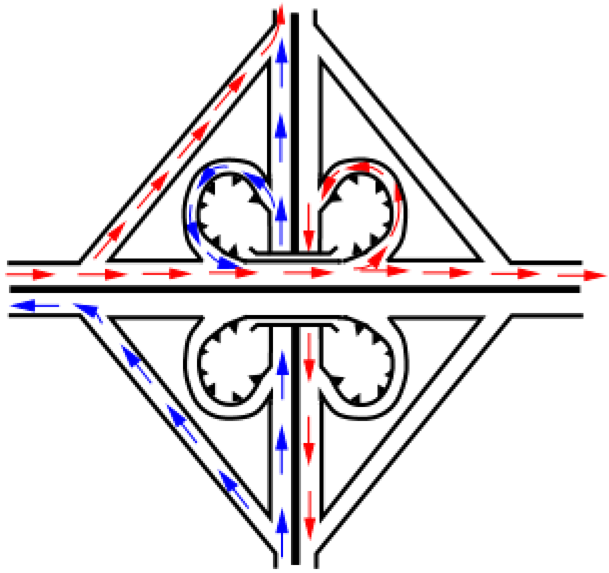

Figure 2.7 Clover leaf interchange

2.1.4.3 Channelized intersection

Vehicles approaching an intersection are directed to definite paths by islands, marking etc. and this method of control is called channelization. Channelized intersection provides more safety and efficiency. It reduces the number of possible conflicts by reducing the area of conflicts available in the carriageway If no channelizing is provided the driver will have less tendency to reduce the speed while entering the intersection from the carriageway. The presence of traffic islands, markings etc. forces the driver to reduce the speed and becomes more cautious while maneuvering the intersection. A channelizing island also serves as a refuge for pedestrians and makes pedestrian crossing safer. Channelization of traffic through a three-legged intersection and a four-leg intersection is shown in the figures below [4].

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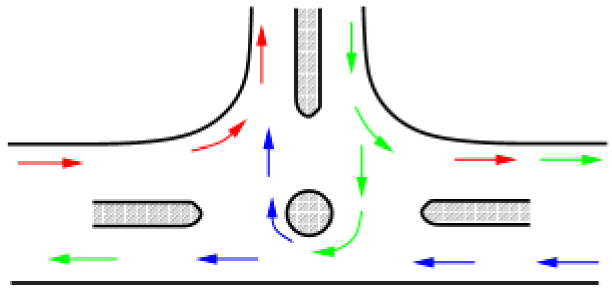

Figure 2.8 Channelization of traffic through a three-legged intersection

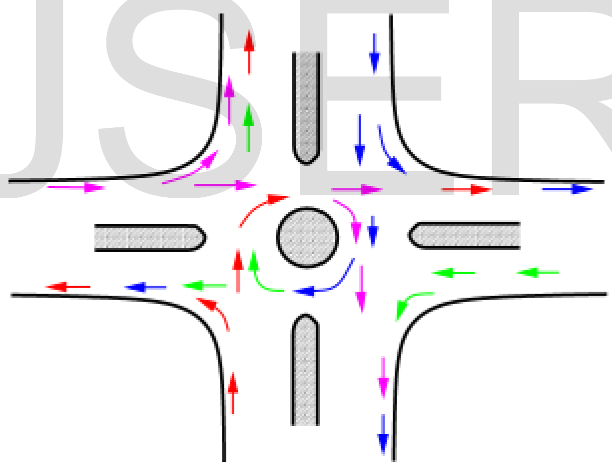

Figure 2.9 Channelization of traffic through a four-legged intersection

2.2 Intersection performance measures

It is said that "If you cannot tell how your system performed yesterday, you cannot expect to manage it today". Transportation system performance measures constitute an invaluable source of information for decisions related to infrastructure resource allocation, investment plan monitoring and project evaluation. The advent of Intelligent Transportation Systems has further increased the significance of obtaining timely and accurate transportation performance measures, which can be used for either optimizing traffic management strategies or informing travelers with respect to their optimal travel paths. The challenge for transportation system has been diverted from developing basic infrastructure to managing and operating the existing transportation resources and delivering better services to road travelers under various conditions. Performance measurement becomes the critical tool to meet such challenges. Intersection performance measures are used to determine or evaluate the service quality provided by the existing road facilities and the control plans at an intersection. Intersection measures usually can be calculated or estimated based on traffic data collected from the subject intersection and its surrounding intersections [6]. Capacity, degree of saturation, delay, queue length, and level of service are the main performance measures of intersections.

2.3 Capacity Analysis at Two Way Control Intersection

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

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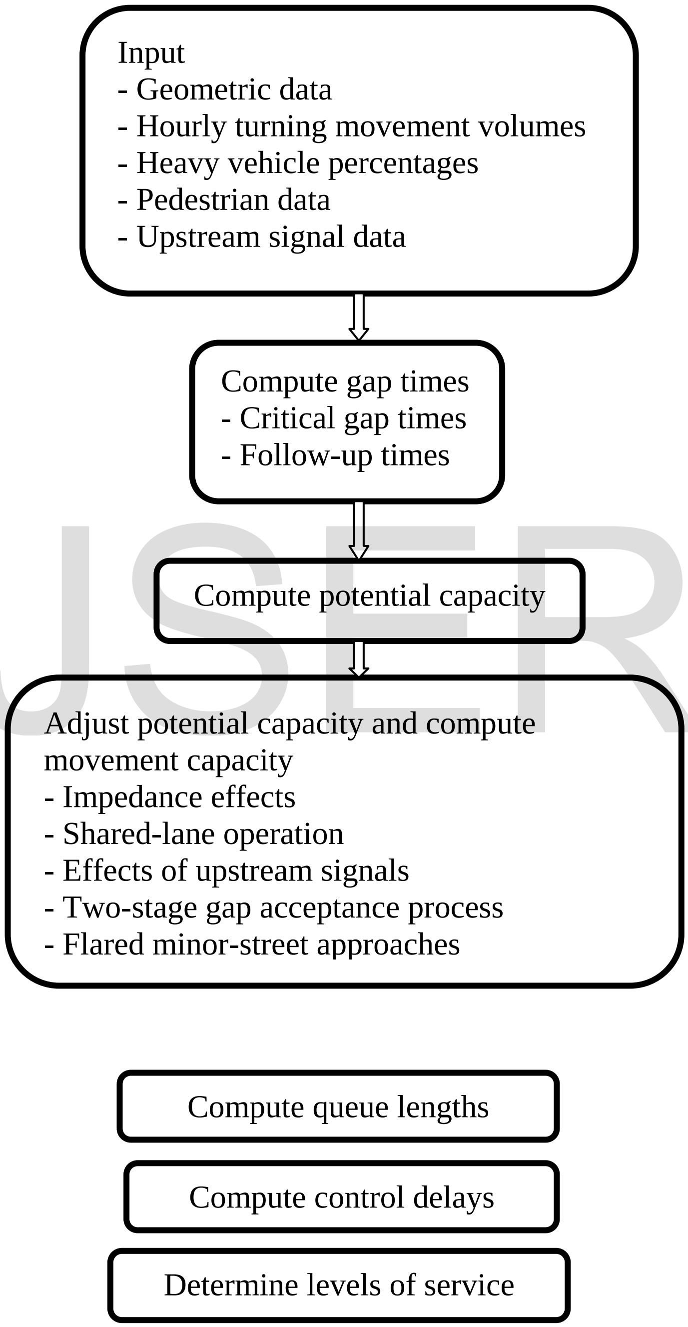
TWSC UN signalized intersection methodology



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2.3.1 Level of Service criteria

Level of service (LOS) for a TWSC intersection is determined by the computed or measured control delay and is defined for each minor movement. LOS is not defined for the intersection as a whole [1, 7]. LOS criteria are given in Figure 3.3.

Level of Service	Average Control Delay (s/veh) (d)		Degree of saturation (v/c ratio) (x)
	Intersection	roundabout	
A	0-10	d≤10	0<x≤0.85
B	>10-15	10<d≤20	0<x≤0.85
C	>15-25	20<d≤35	0<x≤0.85
D	>25-35	30<d≤50 0<d≤50	0<x≤0.85 0.85<x≤0.95
E	>35-50	50<d≤70 0<d≤70	0<x≤0.95 0.95<x≤1.00
F	>50	70<d	1.00<x

Table 2.1 Level of Service Criteria for TWSC Intersection and roundabout

The LOS criteria for TWSC intersections are somewhat different from the criteria used for signalized intersections primarily because different transportation facilities create different driver perceptions. The expectation is that a signalized intersection is designed to carry higher traffic volumes and experience greater delay than unsignalized intersection.

2.3.2 Input Data Requirements

Data requirements for the TWSC intersection require detailed descriptions of the geometrics, control, and volumes at the intersection.

Key geometric factors include number and use of lanes, channelization, two-way left-turn lane (TWLTL) or raised or striped median storage (or both), approach grade, and existence of flared approaches on the minor street.

The number and use of lanes are critical factors. Vehicles in adjacent lanes can use the same gap in the traffic stream simultaneously (unless impeded by a conflicting user of the gap). When movements share lanes, only one vehicle from those movements can use each gap. A TWLTL or a raised or striped median (or both) allows a minor-stream vehicle to cross one major traffic stream at a time. The grade of the approach has a direct and measurable effect on the capacity of each minor movement. Compared with a level approach, downgrades increase capacity and upgrades decrease capacity. A flared approach on the minor street increases the capacity by allowing more vehicles to be served simultaneously [1].

Volumes must be specified by movement. For the analysis to reflect conditions during the peak 15 min, the analyst must divide the full hour volumes by the peak-hour factor (PHF) before beginning computations. If the analyst has peak 15-min flow rates, they can be entered directly with the PHF set to 1.0. The adjusted flow rate for movement x is designated asV_x.

By convention, subscripts 1 to 6 define vehicle movements on the major street, and subscripts 7 to 12 define movements on the minor street. Pedestrian flows impede all minor-street movements. Pedestrian volumes must be specified by movement. Subscripts 13 to 16 define the pedestrian movements

The presence of traffic signals upstream from the intersection on the major street will produce nonrandom flows and affect the capacity of the minor-street approaches if the signal is within 0.25 mi of the intersection. The basic capacity model assumes that the headways on the major street are exponentially distributed. To assess the effect on capacity, a separate analysis is provided that requires the signalized intersection data (cycle length, green time), the saturation flow rate, and information on platooned flow.

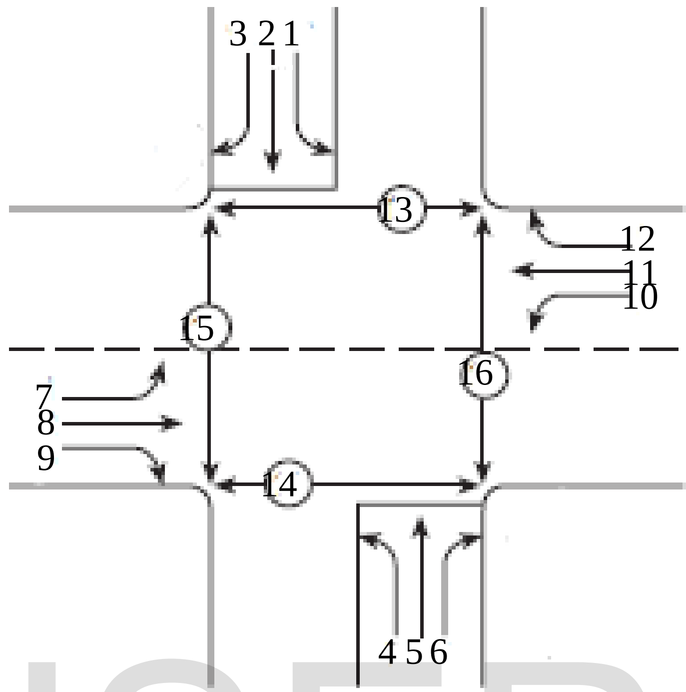
2.3.3 Priority of Streams

In using the methodology, the priority of right-of-way given to each traffic stream must be identified. Some streams have absolute priority, whereas others have to give way or yield to higher-order streams. Figure 3.4 shows the relative priority of streams at four-leg intersections.

Movements of Rank 1 (denoted by the subscript i) include through traffic on the major street and right-turning traffic from the major street. Movements of Rank 2 (subordinate to 1 and denoted by the subscript j) include left-turning traffic from the major street and right-turning traffic onto the major street.

Movements of Rank 3 (subordinate to 1 and 2 and denoted by the subscript k) include through traffic on the Minor Street (in the case of a four-leg intersection) and left turning traffic from the minor street (in the case of a T-intersection). Movements of Rank 4 (subordinate to all others and denoted by the subscript l) include left-turning traffic from the minor street. Rank 4 movements only occur at four-leg intersections [1].

Figure 2.10 Traffic streams at a TWSC intersection



Rank	Traffic Stream
1	2, 3,5,6,15,16
2	1, 4, 13, 14,9,12
3	8, 11
4	7, 10

For example, if a left-turning vehicle on the major street and a through vehicle from the minor street are waiting to cross the major traffic stream, the first available gap of acceptable size would be taken by the left-turning vehicle. The minor-street through vehicle must wait for the second available gap. In aggregate terms, a large number of such left-turning vehicles could use up so many of the available gaps that minor-street through vehicles would be severely impeded or unable to make safe crossing movements. Because right-turning vehicles from the minor street merely merge into gaps in the right-hand lane of the stream into which they turn, they require only a gap in that lane, not in the entire major-street traffic flow (this may not be true for some trucks and vans with long wheelbases that encroach on more than one lane in making their turn).

Furthermore, a gap in the overall major-street traffic could be used simultaneously by another vehicle. For this reason, the method assumes that right turns from the minor street do not impede any of the other flows using major-street gaps.

Pedestrian movements also have priorities with respect to vehicular movements. While this may be a policy issue varying by jurisdiction, both the American Association of State Highway and Transportation Officials (AASHTO) and the *Manual on Uniform Traffic Control Devices* (MUTCD) infer that pedestrians must use acceptable gaps in major-street (Rank 1) traffic streams and that pedestrians have priority over all minor-street traffic at a TWSC intersection [1]. Specific rankings are shown in Figure 2.10.

2.3.4 Conflicting Traffic

Each movement at a TWSC intersection faces a different set of conflicts that are directly related to the nature of the movement. These conflicts are shown in Figure 3.4, which illustrates the computation of the parameter$V_{c,x}$, the conflicting flow rate for movement x, that is, the total flow rate that conflicts with movement x (veh/h).

The right-turn movement from the minor street, for example, is in conflict with only the major-street through movement in the right-hand lane into which right-turners will merge. Figure 2.10 includes one-half of the right-turn movement from the major street, because only some of these turns tend to inhibit the subject movement.

Left turns from the major street are in conflict with the total opposing through and right-turn flows, because they must cross the through flow and merge with the right-turn flow. The method does not differentiate between crossing and merging conflicts. Left turns from the major street and the opposing right turns from the major street are considered to merge, regardless of the number of lanes provided in the exit roadway.

Minor-street through movements have a direct crossing or merging conflict with all movements on the major street, as indicated in Figure 2.10, except the right turn into the subject approach. Only one-half of this movement is included in the computation, for the reasons discussed above. In addition, field research has shown that the effect of left-turn vehicles is twice their actual number. This effect is reflected in Figure 2.10.

The left turn from the minor street is the most difficult maneuver to execute at a TWSC intersection, and it faces the most complex set of conflicting flows, which include all major-street flows, in addition to the opposing right-turn and through movements on the minor street. Only one-half of the opposing right-turn and through movement flow rate is included as conflicting flow rate because both movements are stop-controlled and their effect on left turns is diminished. The additional capacity impedance effects of the opposing right-turn and through movement flow rates are taken into account elsewhere in the procedure.

Pedestrians may also conflict with vehicular traffic streams. Pedestrian flow rates, also defined as V_x, with x noting the leg of the intersection being crossed, should be included as part of the conflicting flow rates, since they, like vehicular flows, define the beginning or ending of a gap that may be used by a minor-stream vehicle. Although it recognizes some peculiarities associated with pedestrian flows, this method takes a uniform approach to vehicular and pedestrian movements [1].

While regulations or practices may vary between jurisdictions, this methodology assumes that pedestrians crossing the subject or opposing approaches have Rank 1 status and that pedestrians crossing the two conflicting approaches to the left or right of the subject minor-street approach have Rank 2 status. The conflicting pedestrian flow rates are identified in Figure 2.10.

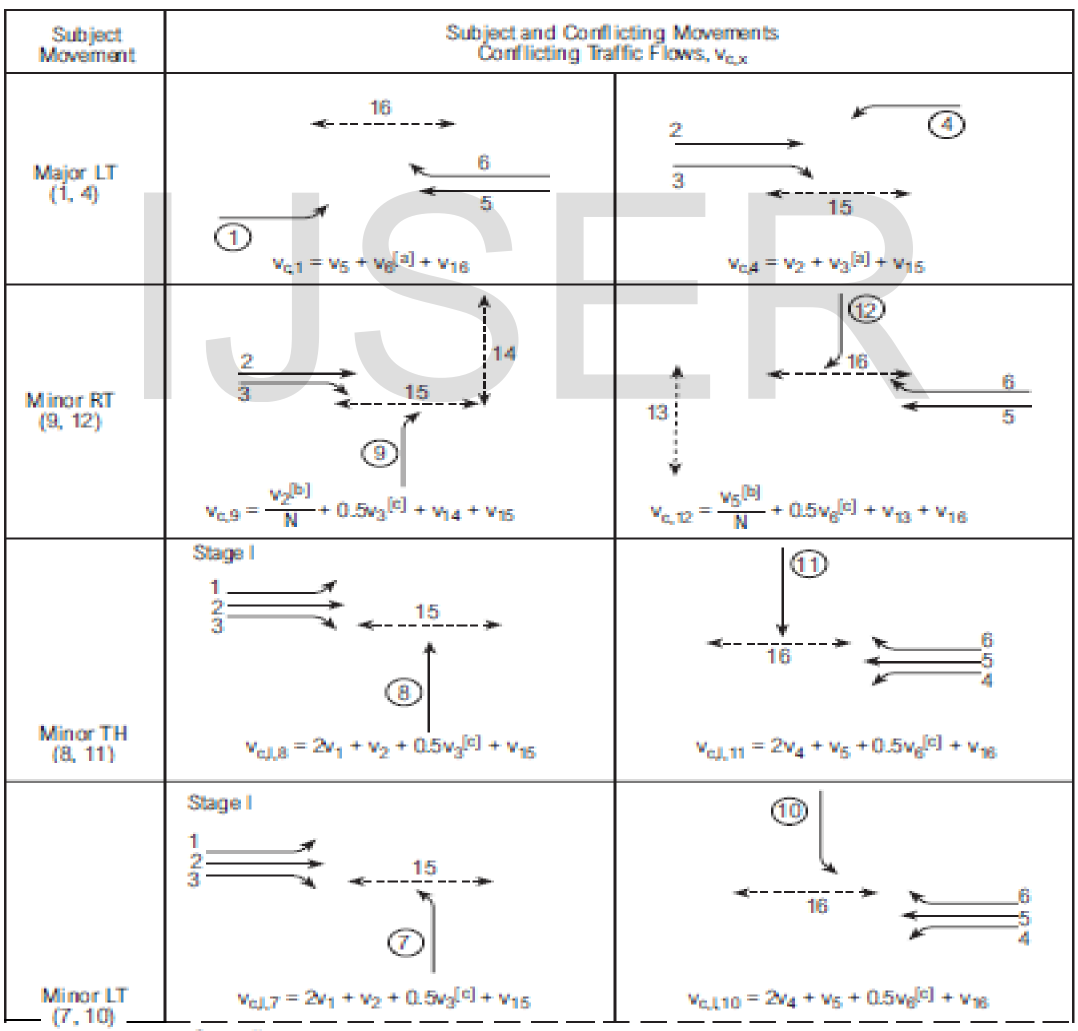
Figure 2.10 also identifies the conflicting flow rates for each stage of a two-stage gap acceptance process that takes place at some intersections where vehicles store in the median area. If a two-stage gap acceptance process is not present, the conflicting flow rates shown in the rows labeled Stage I and Stage II should be added together and considered as one conflicting flow rate for the movement in question.

2.3.5 Critical Gap and Follow-up time

The critical gap, t_c is defined as the minimum time interval in the major-street traffic stream that allows intersection entry for one minor-street vehicle. Thus, the driver's critical gap is the minimum gap that would be acceptable.

A particular driver would reject any gaps less than the critical gap and would accept gaps greater than or equal to the critical gap. Estimates of critical gap can be made on the basis of observations of the largest rejected and smallest accepted gap for a given intersection. The time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street gap, under a condition of continuous queuing on the minor street, is called the follow-up time, t_f. Thus, t_f is the headway that defines the saturation flow rate for the approach if there were no conflicting vehicles on movements of higher rank [1].

Table 2.2 Definition and computation of conflicting flows



Subject Movement	Subject and Conflicting Movements Conflicting Traffic Flows, $v_{c,x}$	
Major LT (1, 4)	$v_{c,1} = v_5 + v_6^{[a]} + v_{16}$	$v_{c,4} = v_2 + v_3^{[a]} + v_{15}$
Minor RT (9, 12)	$v_{c,9} = \frac{v_2^{[b]}}{N} + 0.5v_3^{[c]} + v_{14} + v_{15}$	$v_{c,12} = \frac{v_5^{[b]}}{N} + 0.5v_6^{[c]} + v_{13} + v_{16}$
Minor TH (8, 11)	Stage I $v_{c,I,8} = 2v_1 + v_2 + 0.5v_3^{[c]} + v_{15}$	$v_{c,I,11} = 2v_4 + v_5 + 0.5v_6^{[c]} + v_{16}$
Minor LT (7, 10)	Stage I $v_{c,I,7} = 2v_1 + v_2 + 0.5v_3^{[c]} + v_{15}$	$v_{c,I,10} = 2v_4 + v_5 + 0.5v_6^{[c]} + v_{16}$

Base values of t_c and t_f for passenger cars are given in Table 3.6. The values are based on studies throughout the United States and are representative of a broad range of conditions. Adjustments are made to account for the presence of heavy vehicles, approach grade and two-stage gap acceptance. The critical gap is computed separately for each minor movement by Equation 2.4.

$$t_{c,x} = t_{c,base} + t_{c,HV} P_{HV} + t_{c,G} G - t_{c,T} - t_{3,LT}$$ **Equation 2.4**

Where

$t_{c,x}$ = critical gap for movement x (s),

$t_{c,base}$ = base critical gap from Table 2.3 (s),

$t_{c,HV}$ = adjustment factor for heavy vehicles (1.0 for two-lane major streets and 2.0 for four-lane major streets) (s),

P_{HV} = proportion of heavy vehicles for minor movement,

$t_{c,G}$ = adjustment factor for grade (0.1 for Movements 9 and 12 and 0.2 for Movements 7, 8, 10, and 11) (s),

G = percent grade divided by 100,

$t_{c,T}$ = adjustment factor for each part of a two-stage gap acceptance process (1.0 for first or second stage; 0.0 if only one stage) (s), and

$t_{3,LT}$ = adjustment factor for intersection geometry (0.7 for minor-street left-turn movement at three-leg intersection; 0.0 otherwise) (s) [1].

Table 2.3 Base critical gaps and follow-up times for TWSC intersection

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

The follow-up time is computed for each minor movement using Equation 2.5. Adjustments are made for the presence of heavy vehicles.

$$t_{f,x} = t_{f,base} + t_{f,Hv}\, P_{HV} \qquad \textbf{Equation 2.5}$$

Where

$t_{f,x}$ = follow-up time for minor movement x (s),

$t_{f,base}$ = base follow-up time from Table 2.3 (s),

$t_{f,Hv}$= adjustment factor for heavy vehicles (0.9 for two-lane major streets and 1.0 for four-lane major streets), and

P_{HV} = proportion of heavy vehicles for minor movement.

Values from Table 2.3 are considered typical. If smaller values for t_c and t_f are observed, capacity will be increased. If larger values for t_c and t_f are used, capacity will be decreased [1]. More accurate capacity estimates will be produced if field measurements of the critical gap and follow-up time can be made.

It should be noted that the critical gap data for multilane sites account for the actual lane distribution of traffic flows measured at each site. This accounts for the higher value of critical gap for the minor-street right turn (6.9 s) compared with the value for the minor through movement (6.5 s).

2.3.6 Potential Capacity

The gap acceptance model used in this method computes the potential capacity of each minor traffic stream in accordance with Equation 2.6.

$$C_{p,x} = V_{c,x} \frac{e^{\frac{-v_{c,x} t_{c,x}}{3600}}}{1-e^{\frac{-v_{c,x} t_{f,x}}{3600}}} \qquad \textbf{Equation 2.6}$$

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Where

$c_{p,x}$ = potential capacity of minor movement x (veh/h),

$v_{c,x}$ = conflicting flow rate for movement x (veh/h),

$t_{c,x}$ = critical gap (i.e., the minimum time that allows intersection entry for one minor-stream vehicle) for minor movement x (s), and

$t_{f,x}$= follow-up time (i.e., the time between the departure of one vehicle from the minor street and the departure of the next under a continuous queue condition) for minor movement x (s) [1].

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

• Traffic from nearby intersections does not back up into the subject intersection.

• A separate lane is provided for the exclusive use of each minor-street movement.

• An upstream signal does not affect the arrival pattern of the major-street traffic.

• No other movements of Rank 2, 3, or 4 imped the subject movement.

2.3.7 Impedance Effects

Vehicle Impedance

Vehicles use gaps at a TWSC intersection in a prioritized manner. When traffic becomes congested in a high-priority movement, it can impede lower-priority movements (i.e., streams of Ranks 3 and 4) from using gaps in the traffic stream, reducing the potential capacity of these movements.

Major traffic streams of Rank 1 are assumed to be unimpeded by any of the minor traffic stream movements. This rank also implies that major traffic streams are not expected to incur delay or slowing as they travel through the TWSC intersection. Empirical observations have shown that such delays do occasionally occur, and they are accounted for by using adjustments provided in the procedures.

Minor traffic streams of Rank 2 (including left turns from the major street and right turns from the minor street) must yield only to the major-street through and right-turning traffic streams of Rank 1. There are no additional impedances from other minor traffic streams, and so the movement capacity of each Rank 2 traffic stream is equal to its potential capacity as indicated by Equation 2.7.

$$c_{m,j} = c_{p,j}$$ **Equation 2.7**

Where

j denotes movements of Rank 2 priority.

Minor traffic streams of Rank 3 must yield not only to the major traffic streams, but also to the conflicting major-street left-turn movement, which is of Rank 2. Thus, not all gaps of acceptable length that pass through the intersection will normally be available for use by Rank 3 traffic streams, because some of these gaps are likely to be used by the major-street left-turning traffic. The magnitude of this impedance depends on the probability that major-street left-turning vehicles will be waiting for an acceptable gap at the same time as vehicles of Rank 3. A higher probability that this situation will occur means greater capacity-reducing effects of the major-street left-turning traffic on all Rank 3 movements [1].

What is of interest to the analyst, therefore, is the probability that the major-street left-turning traffic will operate in a queue-free state. This probability is expressed by Equation 2.8: If major-street through and left-turn movements are shared, use Equation 2.19. Also use Equation 2.8 to compute the probability of queue-free state for Rank 3 movements.

$$P_{o,j} = 1 - \frac{v_j}{c_{m,j}}$$ **Equation 2.8**

Where j = 1, 4 (major-street left-turn movements of Rank 2).

The movement capacity $c_{m,k}$, for all Rank 3 movements is found by calculating a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. The capacity adjustment factor is denoted by f_k for all movements k and for all Rank 3 movements and is given by Equation 2.9.

$$f_k = \prod_j p_{o,j}$$ **Equation 2.9**

Where

$p_{o,j}$= probability that conflicting Rank 2 movement j will operate in a queue-free state, and k = Rank 3 movements. The movement capacity for the Rank 3 movements is computed using Equation 2.10.

$$c_{m,k} = (c_{p,k})f_k$$ **Equation 2.10**

Also account for pedestrian impedance, if significant Rank 4 movements (i.e., only the minor-street left turns at a four-leg intersection) can be impeded by the queues of three higher-ranked traffic streams:

- Major-street left-turning movements (Rank 2),
- Minor-street crossing movements (Rank 3), and
- Minor-street right-turning movements (Rank 2).

If the intersection has three legs, then the minor-street left turn is a Rank 3 movement and should be evaluated using Equations 2.8 through 2.10.

The probability that each of these higher-ranked traffic streams will operate in a queue-free state is central to determining their overall impeding effects on the minor-street left-turn movement. At the same time, it must be recognized that not all of these probabilities are independent of each other. Specifically, queuing in the major-street left-turning movement affects the probability of a queue-free state in the minor-street crossing movement. Applying the simple product of these two probabilities will likely overestimate the impeding effects on the minor-street left-turning traffic.

Figure 2.11 can be used to adjust for the overestimate caused by the statistical dependence between queues in streams of Ranks 2 and 3. The mathematical representation of this curve is given by Equation 2.11.

$$P' = 0.65P'' - \frac{P''}{P''+3} + 0.6\sqrt{P''}$$ **Equation 2.11**

Where

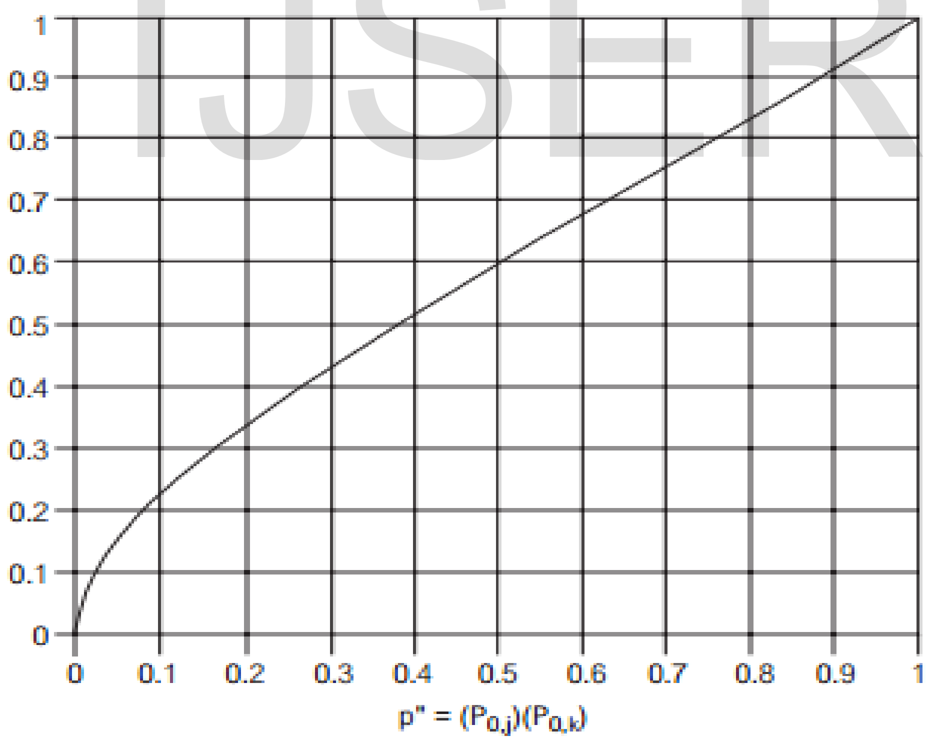
p'= adjustment to the major-street left, minor-street through impedance factor;

p''= $(p_{o,j})$ $(p_{o,k})$;

$p_{o,j}$ = probability of a queue-free state for the conflicting major-street left-turning traffic; and

$p_{o,k}$= probability of a queue-free state for the conflicting minor-street crossing traffic

Figure 2.11 Adjustment to impedance factors for major left turn, minor through



The capacity adjustment factor for the Rank 4 minor-street left-turn movements can be computed by Equation 2.12 [1].

f_l=$(p')(p_{o,j})$ **Equation 2.12**

Where

l = minor-street left-turn movement of Rank 4 (Movements 7 and 10 in Figure 2.10), and

j = conflicting Rank 2 minor-street right-turn movement (Movements 9 and 12 in Figure 2.10).

The variable $p_{o,j}$ should be included in Equation 2.12 only if movement j is identified as a conflicting movement. Refer to Table 3.5 and the associated notes.

Finally, the movement capacity for the minor-street left-turn movements of Rank 4 can be determined from Equation 2.13

$c_{m,j}$= (f_l) $(c_{p,j})$ Equation 2.13

Where

l indicates movements of Rank 4 priority.

Rank 4 movements occur only at four-leg intersections. Equations 2.11 to 2.13 are only required when evaluating four-leg intersections.

Pedestrian Impedance

Minor-street vehicle streams must yield to pedestrian streams. Table 2.4 shows the relative hierarchy between pedestrian and vehicular streams used in this methodology. A factor accounting for pedestrian blockage is computed by Equation 2.14 on the basis of pedestrian volume, the pedestrian walking speed, and the lane width.

$$f_{bp} = \frac{V_x(\frac{W}{S_p})}{3600}$$ Equation 2.14

Where

f_{bp}= pedestrian blockage factor, or the proportion of time that one lane on an approach is blocked during 1 h;

v_x= number of groups of pedestrians, where x is Movement 13, 14, 15, or 16, as described in w = lane width (m); and

s_p = pedestrian walking speed, assumed to be 1.22 m/s [1].

Table 2.4 Relative pedestrian/vehicle hierarchy

Vehicle Stream	Must Yield to Pedestrian Stream		Impedance Factor for Pedestrians, $p_{p,x}$
V_1	V_{16}		$P_{P,16}$
V_4	V_{15}		$P_{P,15}$
V_7	V_{15},V_{13}		$(P_{P,15})(P_{P,13})$
V_8	V_{15},V_{16}		$(P_{P,15})$ $(P_{P,16})$
V_9	V_{15},V_{14}		$(P_{P,15})$ $(P_{P,14})$
V_{10}	V_{16},V_{14}		$(P_{P,16})$ $(P_{P,14})$
V_{11}	V_{15},V_{16}		$(P_{P,15})$ $(P_{P,16})$
V_{12}	V_{16},V_{13}		$(P_{P,16})$ $(P_{P,13})$

The pedestrian impedance factor for pedestrian movement x, $p_{p,x}$ is computed by Equation 2.15

$$p_{p,x} = 1 - f_{bp}$$ **Equation 2.15**

If pedestrians are present to a significant degree, $p_{p,x}$is included as a factor in Equations 2.9 and 2.12. Equation 2.9 becomes

$$f_k=\prod_j (p_{o,j})p_{p,x}$$ **Equation 2.16**

Where $p_{p,x}$ takes on the values shown in Table 2.4. Equation 2.12 becomes

$$f_l = p' p_{o,j}\, p_{p,x}$$ **Equation 2.17**

where $p_{p,x}$ takes on the value $p_{p,13}\, p_{p,15}$ for Stream 7 and $p_{p,14}\, p_{p,16}$ for Stream 10.

2.3.8 Shared-Lane Capacity

Minor-Street Approaches Where several movements share the same lane and cannot stop side-by-side at the stop line, Equation 2.18 is used to compute shared-lane capacity.

$$C_{SH} = \frac{\sum_y V_y}{\sum_y \left(\frac{V_y}{C_{m,y}}\right)}$$ **Equation 2.18**

Where

C_{SH} = capacity of the shared lane (veh/h),

v_x = flow rate of the y movement in the subject shared lane (veh/h), and

$c_{m,y}$ = movement capacity of the y movement in the subject shared lane (veh/h) [1].

2.3.9 Major-Street Approaches

The methodology implicitly assumes that an exclusive lane is provided to all left turning traffic from the major street. In situations where a left-turn lane is not provided, major-street through (and possibly right-turning) traffic could be delayed by left-turning vehicles waiting for an acceptable gap. To account for this possibility, the factors $p*_{0,1}$ and $p*_{0,4}$ may be computed as an indication of the probability that there will be no queue in the respective major-street shared lanes [1].

$$P_{o,j} = 1 - \frac{1 - P_{o,j}}{1 - \left(\frac{V_{i1}}{S_{i1}} + \frac{V_{i2}}{S_{i2}}\right)}$$ **Equation 2.19**

Where

$P_{o,j}$= probability of queue-free state for movement j assuming an exclusive left-turn lane on the major street,

j = 1, 4 (major-street left-turning traffic streams),

i_1 = 2, 5 (major-street through traffic streams),

i_2 = 3, 6 (major-street right-turning traffic streams),

s_{i1} = saturation flow rate for the major-street through traffic streams (veh/h) (this parameter can be measured in the field),

s_{i2} = saturation flow rate for the major-street right-turning traffic (veh/h) (this parameter can be measured in the field),

v_{i1} = major-street through flow rate (veh/h), and

v_{i2} = major-street right-turning flow rate (or 0 if an exclusive right-turn lane is provided) (veh/h).

By using $P^*_{0,1}$ and $P^*_{0,4}$ in lieu of $p_{0,1}$ and $p_{0,4}$ (as computed by Equation 2.8), the potential for queues on a major street with shared left-turn lanes may be taken into account [1].

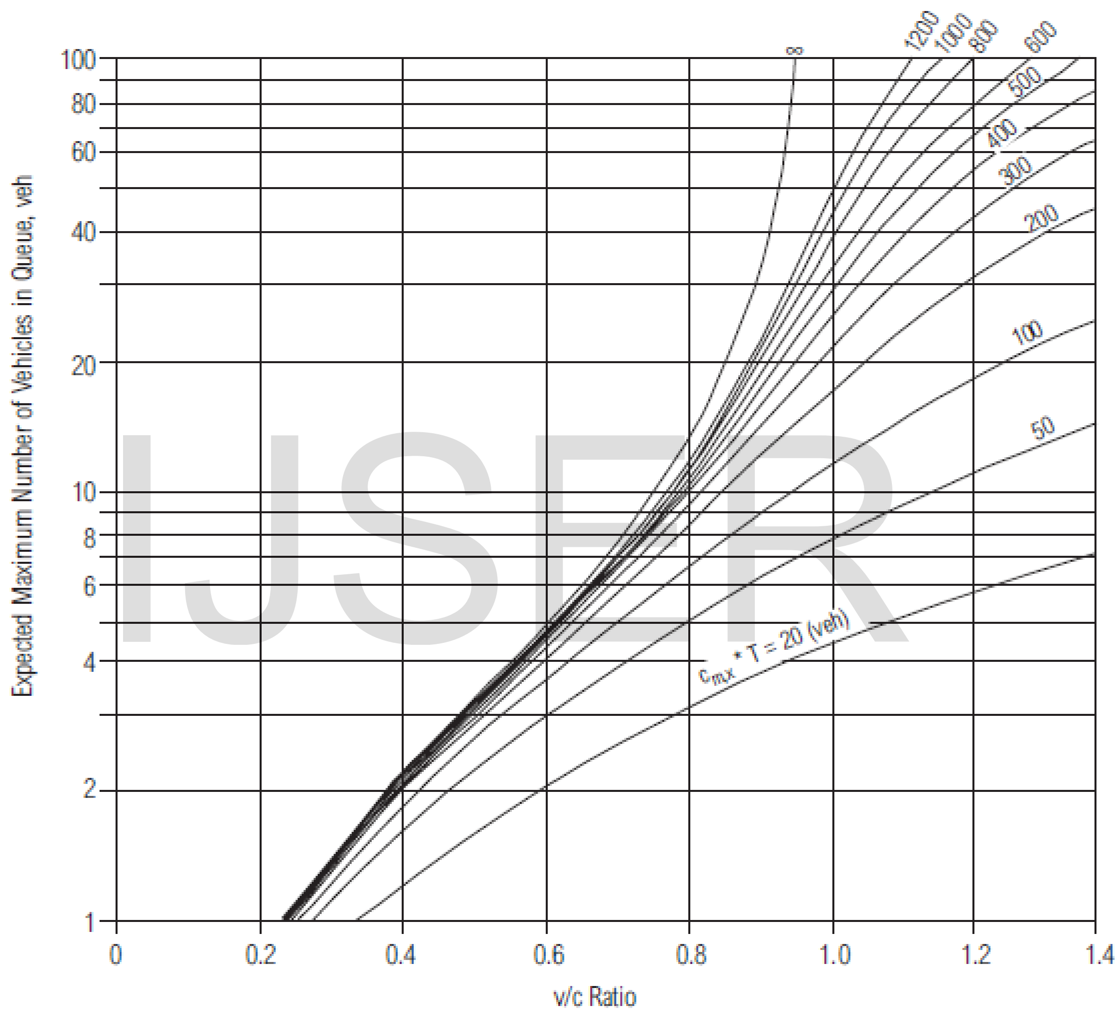
2.3.10 Estimating queue lengths

Estimation of queue length is an important consideration at un-signalized intersections. Theoretical studies and empirical observations have demonstrated that the probability distribution of queue lengths for any minor movement at an un-signalized intersection is a function of the capacity of the movement and the volume of traffic being served during the analysis period. Figure 2.13 can be used to estimate the 95th percentile queue length for any minor movement at an un-signalized intersection during the peak 15-min period on the basis of these two parameters.

The mean queue length is computed as the product of the average delay per vehicle and the flow rate for the movement of interest.

The expected total delay (vehicle-hours per hour) equals the expected number of vehicles in the average queue; that is, the total hourly delay and the average queue are numerically identical. For example, 4 vehicle-hours/hour of delay can be used interchangeably with an average queue length of four (vehicles) during the hour [1].

Figure 2.12 95th-percentile queue length



Equation 2.20 is used to calculate the 95th-percentile queue.

$$Q_{95} = 900T \left[\frac{V_x}{C_{m,x}} - 1 + \sqrt{\left(\frac{V_x}{C_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{C_{m,x}}\right)\left(\frac{V_x}{C_{m,x}}\right)}{150T}}\right] \left(\frac{C_{m,x}}{3600}\right) \qquad \text{Equation 2.2}$$

Where

Q_{95} = 95th-percentile queue (veh),

v_x = flow rate for movement x (veh/h),

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

T = analysis time period (h)

(T = 0.25 for a 15-min period) [1].

2.3.11 Control delay

The delay experienced by a motorist is made up of a number of factors that relate to control, geometrics, traffic, and incidents. Total delay is the difference between the travel time actually experienced and the reference travel time that would result during base conditions, in the absence of incident, control, traffic, or geometric delay. The portion of total delay attributed to control measures, either traffic signals or stop signs, is quantified and this delay is called control delay. Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. With respect to field measurements, control delay is defined as the total elapsed time from the time a vehicle stops at the end of the queue to the time the vehicle departs from the stop line [1]. This total elapsed time includes the time required for the vehicle to travel from the last in-queue position to the first-in-queue position, including deceleration of vehicles from free-flow speed to the speed of vehicles in queue.

Average control delay for any particular minor movement is a function of the capacity of the approach and the degree of saturation. The analytical model used to estimate control delay (Equation 2.21) assumes that the demand is less than capacity for the period of analysis. If the degree of saturation is greater than about 0.9, average control delay is significantly affected by the length of the analysis period. In most cases, the recommended analysis period is 15 min. If demand exceeds capacity during a 15-min period, the delay results calculated by the procedure may not be accurate. In this case, the period of analysis should be lengthened to include the period of oversaturation.

$$d = \frac{3600}{C_{m,x}} + 900T \left[\frac{V_x}{C_{m,x}} - 1 + \sqrt{\left(\frac{V_x}{C_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{C_{m,x}}\right)\left(\frac{V_x}{C_{m,x}}\right)}{450T}} \right] + 5 \quad \textbf{Equation 2.21}$$

Where

d = control delay (s/veh),

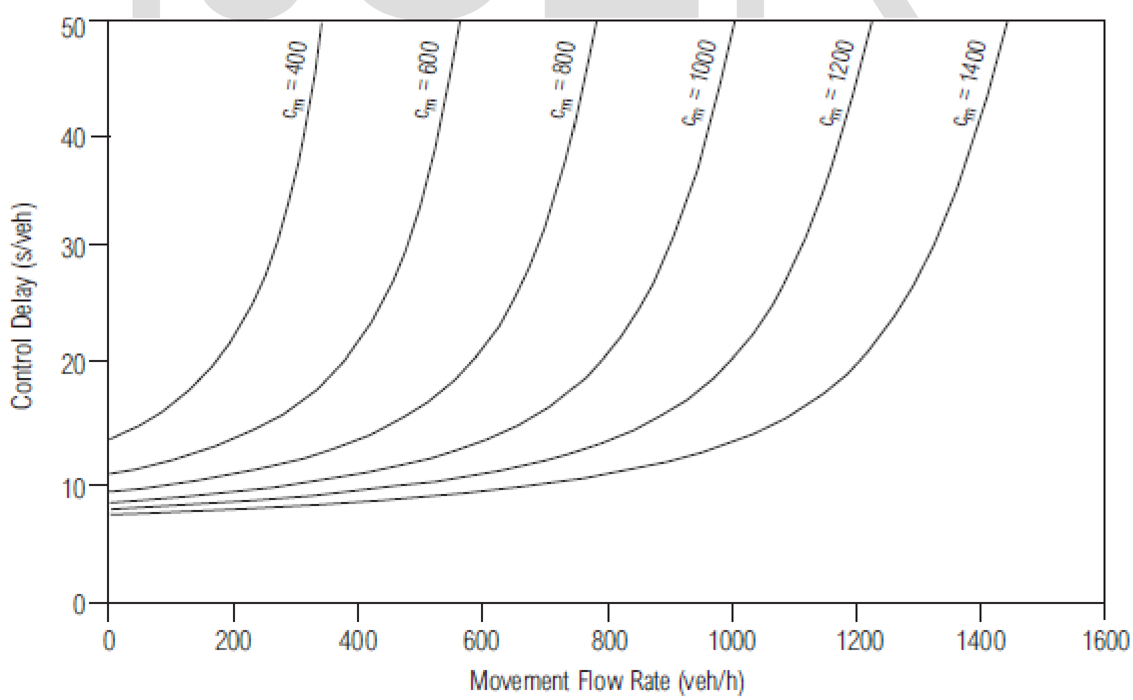
v_x = flow rate for movement x (veh/h),

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

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

The constant value of 5 s/veh is included in Equation 2.21 to account for the deceleration of vehicles from free-flow speed to the speed of vehicles in queue and the acceleration of vehicles from the stop line to free-flow speed [1]. This equation is depicted graphically in Figure 2.14 for a discrete range of capacities and a 15-min analysis period

Figure 2.13 Control delay and flow rate



CHAPTER THREE

RESEARCH METHODOLOGY

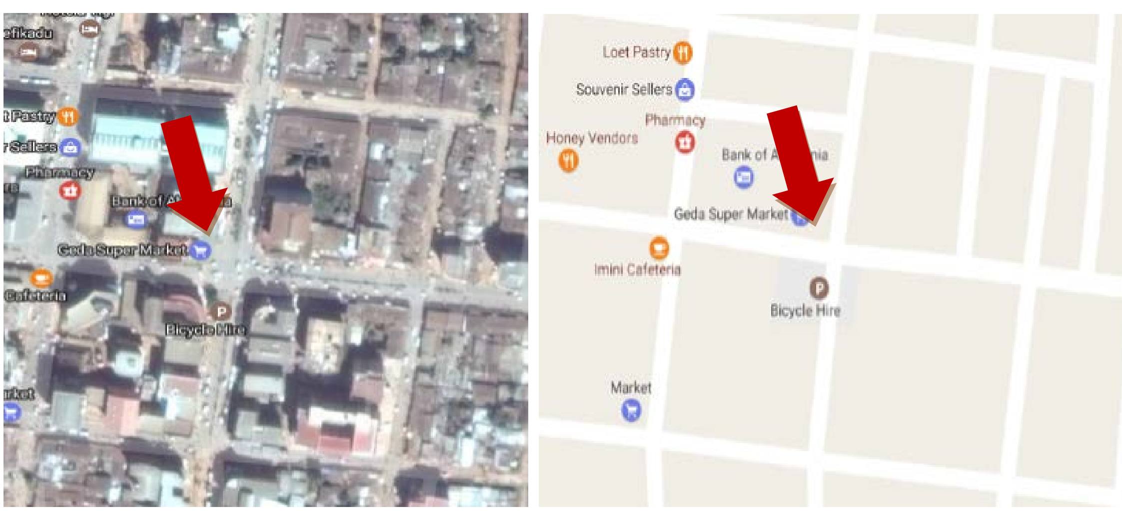
3.1General

This thesis has been compiled by intensive review of literatures related with the title of the research and by associating it with the real ground conditions. After thorough study of literatures on the topic, field observation have been made on the current intersection conditions of Jimma city to select intersection for the study. Geometric data, site plan and other necessary information about the intersection have been collected from concerned bodies in the city. By direct counting of pedestrians and motorized vehicles and measurement of geometric elements of the selected intersections in the city have been conducted. An interview to the traffic polices, road users and to different organization have done on the issues concerning about the performance of intersection in the city. Historical traffic or traffic growth rate data of the city had been collected to forecast the traffic for prediction of intersection performance.

Study Area

This study is conducted on an intersection located around Merkato near Geda Supermarket. And the overall study period is 2 Months.

Figure 3.1 Location of the study area [11]



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3.2 Data collection

To select intersection for the study and to decide on the peak hour, field observation have been made on the current intersections of Jimma city. Since traffic and geometric data's are too important to achieve the objective of this thesis, it was found necessary to collect these data.

As a result, after consulting with our advisor, we collected general information on appropriate forms for vehicle traffic volume and pedestrian counts and how to measure geometric elements of the intersections.

3.2.1 Traffic data collection

In order to encompass a good image about the site and the flow condition, repeated visual visits and information from traffic polices are taken. From these repeated visual visits a decision on the peak flow period time is arrived. And the traffic volume is clearly higher in the day time than at the morning times. And from among the day times the time from 6:30-6:45 is peak flow period and to determine the peak hour exactly the data is collected at 15 minute intervals. Since both vehicle and pedestrian traffics have a great influence on the performance of intersections, traffic count is made for both vehicle and pedestrian traffics. The traffic count was made on Monday, Friday and Saturday. From these days, Saturday is the peak flow day.

In Jimma city there is high volume of Tricycles and pedestrians compared with other vehicular traffics. And hence Tricycles, pedestrians, light vehicles (cars, taxi, minibus, pickups) and heavy vehicles (bus, trucks, truck-trailers) are counted separately as well and this are shown in Appendix A. For analysis and design of intersections such a mixed traffic should be converted into passenger cars. To change all vehicle into a passenger cars (light vehicles in this case) passenger car unit (pcu) for each vehicles is needed. Actually pcu value is depends on the characteristics of vehicle, traffic stream, roadway, environment and control and climatic conditions. But there is scarce research that specifies pcu values of Jimma as well as for Ethiopia as a whole. And therefore, we adopted Passenger car unit values recommended by the Indian Road Congress (IRC)

(Table 3.1) which have mixed traffic and high proportion of motorcycles and tricycles in their traffic composition like in Jimma.

Table 3.1 PCU values recommended by the Indian Road Congress (IRC)

Vehicle Type	Passenger Car Units
Motorcycles	**0.5**
Tricycles	**0.7**
Buses	**2.0**

From Table 3.1 pcu value of 0.5 for motorcycles is recommended but because in the intersection under study the number of motorcycles is very small and thus is neglected. Here tricycles include motorized and non-motorized tricycles that used for goods transport. But non-motorized tricycles are not common in Jimma. Therefore by considering recommended pcu value on IRC for this paper pcu value of 0.7 is taken for Tricycles.

Proportion of heavy vehicles relative to other vehicular traffic in Jimma city is very small and Bus and medium Trucks are the major compositions of heavy vehicles in the city.

There are many literatures that recommend pcu value for heavy vehicles. And most of these literatures recommend pcu value for bus and medium trucks in the vicinity of 2. And in this paper pcu value of 2 is taken for heavy vehicles. The traffic data is counted on Monday, Friday and Saturday at 2-3,6-7 and 9-10 hours by dividing each hour within 15 minutes divisions. These days are selected based on site visits and the information gathered from traffic polices of all the seven days and the maximum congested days are selected for vehicle counting. The detail 15 minute traffic data is show on **appendix A.**

Table 3.2 Average number of vehicles (in pcu) and pedestrians for each 15 minute Sintervals of peak flow period in intersections

Time	Number of vehicles	Number of pedestrians
6:00-6:15	439	1345
6:15-6:30	485	1371
6:15-6:30	531	1547
6:45-7:00	429	1408

Figure 3.1 Average number of vehicles (in pcu) and pedestrians for each 15 minute interval in intersections

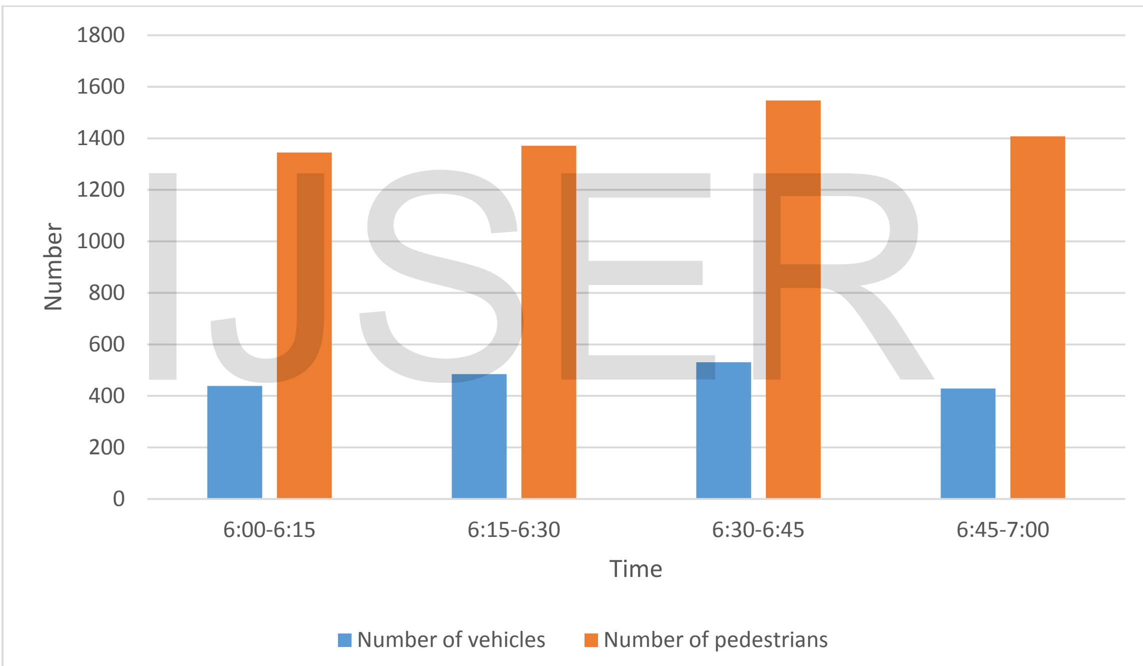

Figure 3.1 show the time from 6:30-6:45 is the 15 minute interval which has higher traffic volume than other 15 minute intervals of peak flow period (6:00-7:00) and both vehicles and pedestrians traffic decreases in both directions from this time. The time from 6:00-7:00 pm is taken as peak hour for the area and used in this paper. And in this peak hour number of vehicles and pedestrians for each intersection from their approach legs are shown in the table below.

Table 3.3 Number of vehicles and pedestrians during the peak hour

Time (PM)	Tricycles			Light vehicles			Heavy vehicles			Total Vehicles (pcu)			Percentage of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City View approach														
6:00-7:00	36	336	220	108	96	88	4	16	8	143	364	258	3.7	1671
Dashen Bank approach														
6:00-7:00	88	500	12	12	80	8	8	24	0	90	478	17	5.5	1323
City Center approach														
6:00-7:00	152	48	44	128	48	60	12	8	8	259	98	107	6.03	1536
Buna Bank Approach														
6:00-7:00	24	116	68	16	16	120	0	0	0	33	91	168	0	1148

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3.3 Geometric data collection

For performance evaluation of intersections in addition to traffic data's geometric data are also necessary and these data's are collected and are showed as the figure below.

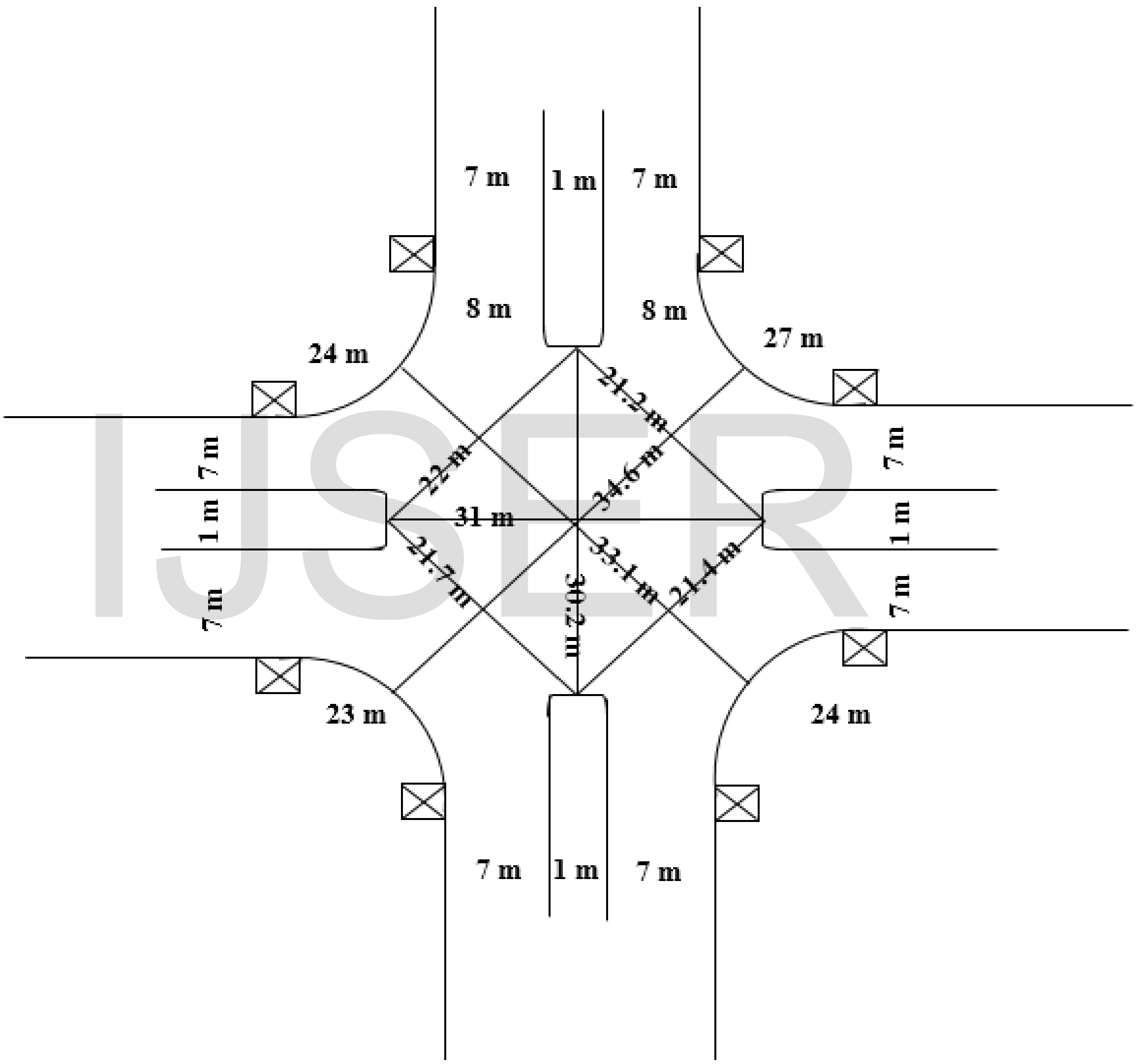


Figure 3.2 Geometrical data for the intersection analyzed

CHAPTER FOUR

RESULT AND DISCUSSION

4.1 Introduction

Existing intersection analysis models fall generally into two categories which are Empirical models and Analytical models. Empirical models rely on field data to develop relationship between geometric design features and performance measures such as capacity and delay. Instead Analytical models are based on the concept of gap acceptance theory. The choice of an analysis approach depends on the calibration data available. For this specific research we used Empirical model of intersection analysis. This is done through analysis procedures on HCM.

4.2 Models, criteria or values used for analysis

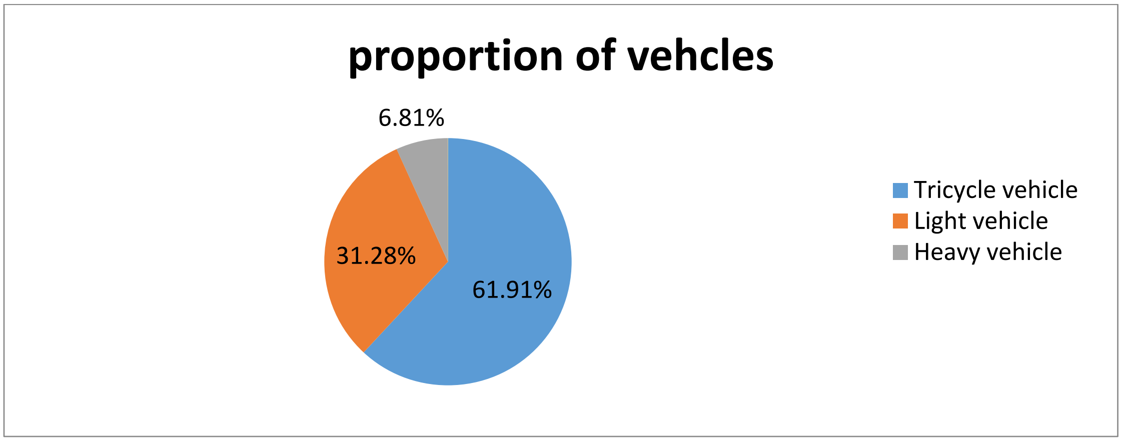
In addition to traffic and geometric data, criteria or value used for analysis of each performance measures (saturation flow, capacity, delay, queue length, and level of ervice) should be specified.

From the analysis of the intersections almost all the intersections have poor performance and to know the root causes of this among other thing is distinguishing proportion of each vehicle types and proportion of total vehicles to pedestrian traffics at the intersection areas is the one. And to do this total number of each vehicle type and pedestrians for each intersection in the peak hour is determined and shown in the Table 4.2.

Table 4-1 Total number of each vehicle type and pedestrians traffic for each intersection in an hour

Intersection	Tricycles	light vehicles	heavy vehicles	Total vehicles	Pedestrians
	1544	780	170	2494	5678

Figure 4.8 Proportion of vehicle type at the intersections in Jimma city



4.3 Analysis of the existing intersection

Fig 4.9 Geometrics and movements

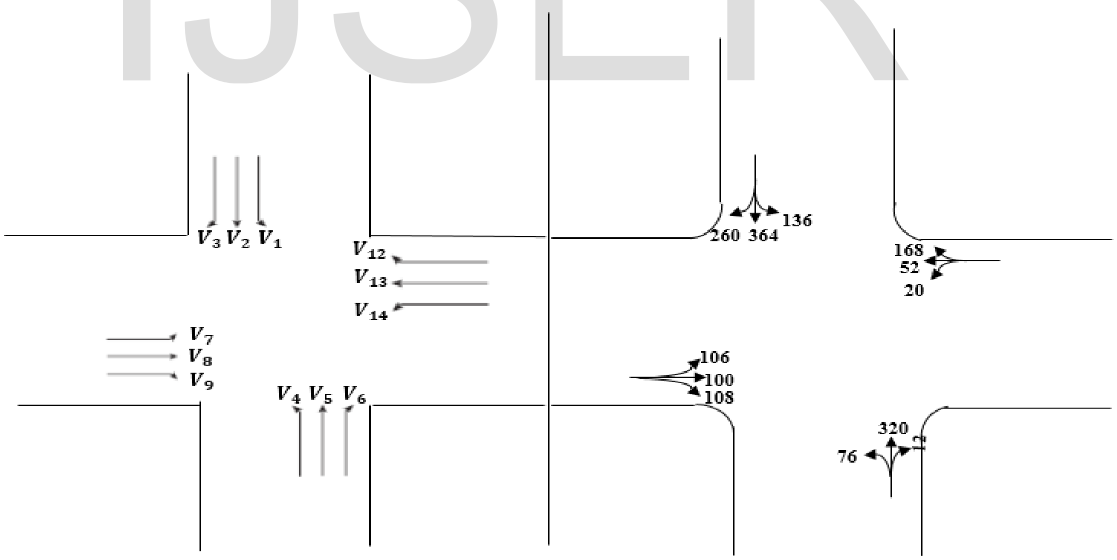


Table 4.2 Calculation of circulating flow

<table>
<tr>
<td>Movement capacity $c_{m,x}$accounting for impedance</td>
<td>Major L_T(1,4)

$V_{C,1}$=V_5+V_6+V_{16}

$V_{C,1}$=320+12+0

$V_{C,1}$=132veh/hr

$$C_{p,x} = V_{c,x}\frac{e^{\frac{-v_{c,x}t_{c,x}}{3600}}}{1-e^{\frac{-v_{c,x}t_{f,x}}{3600}}}$$
$$C_{p,1} = 332\frac{e^{\frac{-332*4.21}{3600}}}{1-e^{\frac{-332*2.25}{3600}}}$$
$$C_{p,1} = 1202\ veh/hr$$

$C_{m,x}$=$C_{p,x*}p_{p,x}$

$C_{m,1}$=$C_{p,1}$ *$p_{p,1}$

$p_{p,1}$=1
$C_{m,1}$=1202
$P_{o,x}$ = 1-$\frac{V_x}{C_{m,x}}$
$P_{o,1}$=1-$\frac{V_1}{C_{m,1}}$=1-$\frac{136}{1202}$=0.89</td>
<td>$V_{C,4}$=V_2+V_3+V_{15}

$V_{C,4}$=364+260+0

$V_{C,4}$=624 veh/hr

$$C_{p,x} = V_{c,x}\frac{e^{\frac{-v_{c,x}t_{c,x}}{3600}}}{1-e^{\frac{-v_{c,x}t_{f,x}}{3600}}}$$
$$C_{p,4} = 624\frac{e^{\frac{-624*4.21}{3600}}}{1-e^{\frac{-624*2.25}{3600}}}$$

$C_{p,4}$=931 veh/hr

$C_{m,x}$=$C_{p,x*}p_{p,x}$

$C_{m,4}$=$C_{p,4*}p_{p,4}$,$p_{p,4}$=1

$P_{o,4}$ = 1-$\frac{V_4}{C_{m,4}}$=1-$\frac{76}{931}$=0.92</td>
</tr>
</table>

Movement capacity $c_{m,x}$accounting for impedance	Major TH(8,11) $v_{c,8}$=$2V_1$+V_2+$0.5V_3$+V_{15} $v_{c,8}$=2*136+364+0.5*260=766 $C_{p,x} = V_{c,x}\frac{e^{\frac{-v_{c,x}t_{c,x}}{3600}}}{1-e^{\frac{-v_{c,x}t_{f,x}}{3600}}}$ $C_{p,8} = 766\frac{e^{\frac{-766*6.56}{3600}}}{1-e^{\frac{-766*4.05}{3600}}}$=330 veh/hr $p_{o,x}$=1-$\frac{v_x}{c_{m,x}}$ $p_{o,8}$=1-$\frac{v_8}{c_{m,8}}$=1-$\frac{100}{330}$=0.60 f_8 =$p_{o,4}$*$p_{o,1}$*$p_{p,8}$ f_8=0.71 $C_{m,8}$=$C_{p,8}$*f_8 $C_{m,8}$=330*.71=235	$V_{c,11}$=$2V_4$+V_5+$0.5V_6$+V_{16} $V_{c,11}$=2*76+320+0.5*12+0=478 $C_{p,x} = V_{c,x}\frac{e^{\frac{-v_{c,x}t_{c,x}}{3600}}}{1-e^{\frac{-v_{c,x}t_{f,x}}{3600}}}$ $C_{p,11} = 478\frac{e^{\frac{-478*6.56}{3600}}}{1-e^{\frac{-478*4.05}{3600}}}$=481 veh/hr $p_{o,x}$=1-$\frac{v_x}{c_{m,x}}$ $p_{o,11}$=1-$\frac{v_{11}}{c_{m,11}}$=0.85 f_{11} =$p_{o,4}$*$p_{o,1}$*$p_{p,11}$ f_{11}=0.92*0.89*0.88=0.73 $C_{m,11}$=$C_{p,11}$*f_{11} $C_{m,11}$=481*0.73=352

Movement capacity $c_{m,x}$ accounting for impedance	Major $_{LT,}$(7,10) $v_{c,7}$=2V_1+V_2+0.5V_3+V_{15} $v_{c,7}$=2*136*364+0.5*260=766 $$C_{p,x} = V_{c,x}\frac{e^{\frac{-v_{c,x}t_{c,x}}{3600}}}{1-e^{\frac{-v_{c,x}t_{f,x}}{3600}}}$$ $C_{p,7} = 766\frac{e^{\frac{-766*7.16}{3600}}}{1-e^{\frac{-766*3.55}{3600}}}$=315 veh/hr p_7''=$p_{o,11}$*f_{11}=0.73*0.85=0.63 p_7'=0.65p_7''*$\frac{p''_7}{p_7''+3}+0.6\sqrt{p''_7}$ p_7'=0.65*o.63-$\frac{o.63}{0.63+3}$+0.6$\sqrt{0.63}$=0.72 $p_{p,7}$=1-$\frac{v_7}{c_{m,7}}$1-$\frac{276}{136}$=-ve(take $p_{p,7}$=1) f_7=p_7'*$p_{o,12}$*$p_{p,7}$=0.72*0.68*0.86=0.43 $c_{m,7}$=f_7*$C_{p,7}$=0.43*315=36	$V_{c,10}$=2V_4+V_5+0.5V_6+V_{16} $V_{c,10}$=2*76+320+0.5*12=478 $$C_{p,x} = V_{c,x}\frac{e^{\frac{-v_{c,x}t_{c,x}}{3600}}}{1-e^{\frac{-v_{c,x}t_{f,x}}{3600}}}$$ $C_{p,10} = 478\frac{e^{\frac{-478*7.16}{3600}}}{1-e^{\frac{-478*3.55}{3600}}}$=492 veh/hr f_{10}=p_{10}'*$p_{o,9}$*$p_{p,10}$ $p_{o,10}$=1-$\frac{v_{10}}{c_{m,10}}$=1-$\frac{20}{492}$=0.88 f_{10}=0.48*0.75*0.88=0.32 p_{10}'=0.65p_{10}''*$\frac{p''_{10}}{p_{10}''+3}$ + 0.6$\sqrt{p''_{10}}$=0.48 $p_{p,10}$=0.88 p_{10}''=$p_{o,8}$*f_8=0.65*0.71=0.47 p_9'=0.75 $C_{m,10}$=$C_{p,10}$*f_{10}=492*0.32=158

Table 4.3 Vehicle volumes and adjustments

vehicle volumes and adjustments												
	Vehicle volume and adjustments											
Movements	1	2	3	4	5	6	7	8	9	10	11	12
Volume(veh/hr)	168	408	300	76	448	12	388	108	80	20	116	288
PHF	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
HFR(veh/hr)	168	408	300	76	448	12	388	108	80	20	116	288
P_{HV}	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06

Pedestrian volumes and adjustments				
Movement	13	14	15	16
Flow, (ped/hr)	1671	1323	1536	1148
Lane width(m)	3.5	3.5	3.5	3.5
Walking speed(m/s)	1.22	1.22	1.22	1.22
Percentage blockage f_b	0.130	0.106	0.123	0.092
Default walking speed= 4ft/sec (1.22m/sec) Average number of pedestrians crossing a road in group (N)=7				

Table 4.4 critical gap and follow-up time

Critical Gap and Follow-Up Time

$$t_c = t_{c,base} + T_{c,HV}\, P_{HV} + t_{c,G} G - t_{t,T} - t_{3,LT}$$

	Major LT		Minor RT		Minor TH		Minor LT	
Movement	1	4	9	12	8	11	7	10
$t_{c,base}$	4.1	4.1	6.2	6.2	6.5	6.5	7.1	7.1
$T_{c,HV}$	1	1	-	-	-	-	-	-
P_{HV}	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
$t_{c,G}$	1	1	0.1	0.1	0.2	0.2	0.2	0.2
G	0.05	0.05	0.005	0.005	0.005	0.005	0.005	0.005
$t_{3,LT}$	0	0	0	0	0	0	0	0
$t_{c,T}$	0	0	0	0	0	0	0	0
t_c	4.21	4.21	6.26	6.26	6.56	6.56	7.16	7.16

$$t_f = t_{f,base} + t_{f,HV} P_{HV}$$

	Major LT		Minor RT		Minor TH		Minor LT	
Movement	1	4	9	12	8	11	7	10
$t_{f,base}$	2.2	2.2	3.3	3.3	4	4	3.5	3.5
$t_{f,HV}$	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
P_{HV}	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
t_f	2.25	2.25	3.35	3.35	4.05	4.05	3.55	3.55

Table 4.5 Impedance and capacity calculation

Impedance and Capacity Calculation		
Step:1 RT from Minor street	V_9	V_{12}
Conflicting flows	$V_c,9= 655$	$V_c,12= 498$
Potential movement	$C_p,9= 460$	$C_p,12= 565$
Pedestrian impedance capacity	$P_p,9= 0.86$	$P_p,12= 0.88$
Movement capacity	$C_m,9=C_p,9* p_p,9=460$	$C_m,12=C_p,12 *P_p,12=499$
Probability of queue free state	$P_0,9=0.75$	$P_o,12= 0.68$
Step:2 LT from Major street	V_4	V_1
Conflicting flow	$V_c,4=624$	$V_c,1=332$
Potential movement	$C_p,4=931$	$C_p,1=1202$
Pedestrian impedance capacity	$P_p,4= 0.87$	$P_p,1=0.89$
Movement capacity	$C_m,4=C_p,4*P_p,4=850$	$C_m,1= C_p,1*Pp,1=1069$
Probability of queue free state	$P_o,4=0.92$	$P_o,1=0.89$
Step:3 TH Minor street	V_8	V_{11}
Conflicting flows	$V_c,8=766$	$V_c,11=478$
Potential movement	$C_p,8=330$	$C_p,11=481$
Pedestrian impedance capacity	$P_p,8=0.86$	$P_p,11=0.88$
Capacity adjustment factor due to impeding movement	$f_8=P_o,4*P_o,1*P_p,8=0.71$	$f11=P_o,4*P_o,1*P_o,11=0.73$
Movement capacity	$C_m,8=C_p,8*f8=235$	$C_m,11= C_p,11*f11=352$
Probability of queue free State	$P_o,8=0.6$	$P_o,11=0.83$
Step:4 LT from Minor street	V_7	V_{10}
Conflicting flows	$V_c,7=766$	$V_c,10=478$
Potential capacity	$C_p,7=315$	$C_p,10=492$
Pedestrian impedance factor	$P_p,7=0.86$	$P_p,10=0.88$
Major left, minor through impedance factor	P’’7=0.63	P”10=0.47
Major left, minor through adjusted impedance factor	P’7=0.72	P’10=0.75
Capacity adjustment factor due to impeding movements	$f7=P'7*P_o,12*P_p7=0.43$	f10= P’10*Po.9*Pp10=0.32
Movements capacity	$C_m,7=f7*C_p,7=136$	$C_m,10=f10*Cp,10=158$

4.3.5 Shared lane capacity

$$C_{SH} = \frac{\sum_y V_y}{\sum_y\left(\frac{V_y}{C_{m,y}}\right)}$$

Table 4.6 Shared lane capacity

	V(veh/hr)						C_{SH}
Lane	7	8	9	7	8	9	
	706	100	108	136	235	460	180
Lane	10	11	12	10	11	12	
	20	52	168	158	352	565	294

4.3.6 Control delay and LOS

$$d = \frac{3600}{C_{m,x}} + 900T\left[\frac{V_x}{C_{m,x}} - 1 + \sqrt{\left(\frac{V_x}{C_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{C_{m,x}}\right)\left(\frac{V_x}{C_{m,x}}\right)}{450T}}\right] + 5$$

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

Table 4.7 control delay and los

Lane	V	c_m	$\frac{v}{c}$	Queue length(Q_{95}) s/veh	Control delay(d)	LOS	Delay and LOS
7,8,9	108	180	0.6	3.29	50	E	50,E
10,11,12	168	294	0.5	2.95	44	E	44,E

Lane	V	c_m	$\frac{v}{c}$	Queue length(Q_{95}) s/veh	Control delay(d)	LOS	Delay and LOS
1	136	1202	0.12	0.38	8	A	8,A
4	76	931	0.09	0.27	6	A	6,A

Level of service for unsignalized intersections is based on the delay experienced by each movement within the intersection, rather than on the overall stopped delay per vehicle at the intersection. This difference from the method used for signalized intersections is necessary since the operating characteristics of uncontrolled intersections are substantially different. Driver expectations and perceptions are entirely different. For uncontrolled intersections, the through traffic on the major (uncontrolled) roadway experiences no delay at the intersection. Conversely, vehicles turning left from the minor roadway experience more delay than other movements and at times can experience substantial delay. Vehicles on the minor roadway, which are turning right or going across the major roadway, experience less delay than those turning left from the same approach. Due to this situation, the LOS assigned to uncontrolled intersection is based on average delay for vehicles on the minor roadway approach. In this regard the governing LOS of existing intersection is found to be LOS E [10].

4.4 Average Annual daily traffic Determination

4.4.1 Conversion of Day Time Traffic to Average Daily Traffic

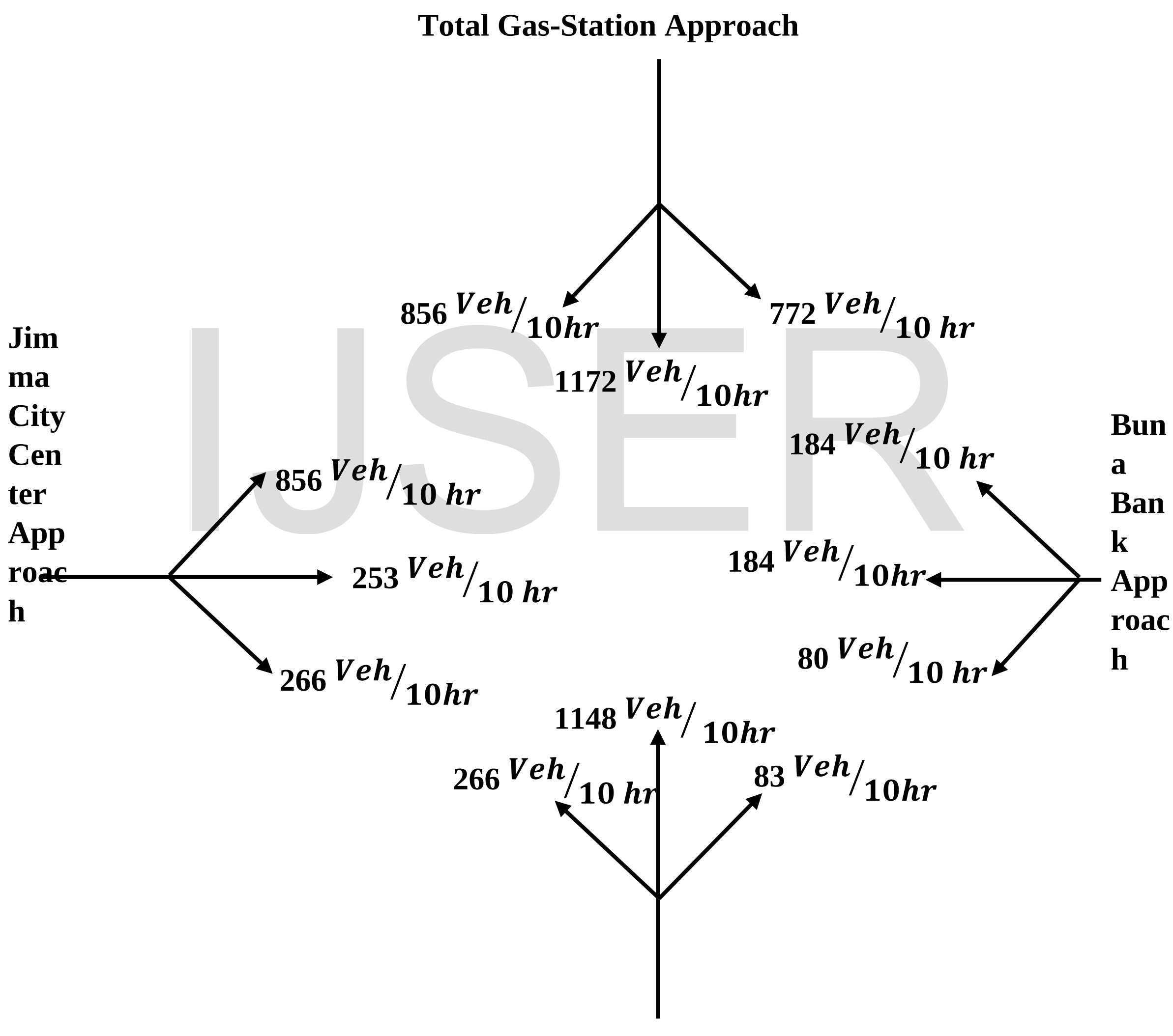
In order to convert Day Time Traffic to Average Daily Traffic and substantial to Annual Average Daily Traffic, derived factors based on the duration of counts shall be used. For the purpose of illustration the following has been assumed:

Monday and Saturday are selected as a representative days for weekdays and weekend respectively. The count was made for 10 hours per day has been obtained as follows.

We have used a 96% confidence limit for the 24 hour traffic flow considering 4% tolerance [10].

Monday

Figure 4.10 AADT determination



The above traffic volume is representative of Monday's 10 hour traffic data and same procedure is followed for Saturday's traffic data and it's shown on **Appendix B**.

Table 4.8 Average daily traffic Determination

<table>
<tr><td colspan="2">Monday</td></tr>
<tr><td>
1. 10 hours traffic count for total gas station approach.

Right turn=v_R=856 veh/10hr,

For straight movement v_s=1172 veh/10hr

For left side movement v_l=772 veh/10 hr is counted.

A) Right turn direction

10hr=856 veh

16 hr=x

$x_{24hrADT_R}=\frac{16hr*856veh}{10hr}$*1.04=1424 veh/Day

B) Straight direction

$x_{24hrADT_s}=\frac{16hr*1172veh}{10hr}$*1.04=1950 veh/Day

C) Left turn direction

$x_{24hrADT_R}=\frac{16hr*772veh}{10hr}$*1.04=1284 veh/Day
</td><td>
2. Jimma city center approach 10 hours traffic count v_R=266, v_s=253, and v_l=939

A) ADT_l=1562 veh/Day

B) ADT_S=420 veh/Day

C) ADT_R=443 veh/Day

3. Dashen bank approach v_R=83, v_s=1148, v_l=266

A) ADT_l=442 veh/Day

B) ADT_S=1910 veh/Day

C) ADT_R=138 veh/Day

4. Buna International bank approach v_R=552, , v_s=184, v_l=80

A) ADT_l=183 veh/Day

B) ADT_S=306 veh/Day

C) ADT_R=849 veh/Day
</td></tr>
</table>

Table 4.9 total average annual daily traffic

Total AADT	
1. Jimma city center AADT $AADT_L$=$\frac{5*1562+1650*2}{7}$=1587 $AADT_R$=$\frac{5*443+648*2}{7}$=502 $AADT_R$=$\frac{5*420+15502*2}{7}$=443 TOTAL AADT=2532 2. Dashen bank approach AADT $AADTl_L$=$\frac{5*442+708*2}{7}$=518 $AADT_R$=$\frac{5*138+172*2}{7}$=148 $AADT_s$=$\frac{5*1910+2077*2}{7}$=1958 TOTAL AADT=2624	3. Total gas station approach AADT $AADT_R$=$\frac{5*1424+1640*2}{7}$=1486 $AADT_S$=$\frac{5*1950+2218*2}{7}$=2027 $AADT_L$=$\frac{5*1284+1311*2}{7}$=1292 TOTAL AADT =4805 4. Buna International Bank approach $AADT_R$=$\frac{5*849+1212*2}{7}$=953 $AADT_s$=$\frac{5*306+274*2}{7}$=297 $AADT_l$=$\frac{5*183+186*2}{7}$=184 TOTAL AADT =1434

From the forecasted traffic data the functional classification of road is trunk road [9].

4.5 Traffic forecasting

The current traffic volume will increase tremendously year to year as the economic capacity of the community increase and it should be forecasted to sustain the coming traffic volume after years and here in our study we forecasted the existing traffic volume for 20 years.

$PHT_{O,1}$=136 $PHT_{O,3}$= 260 $PHT_{O,5}$=320 $PHT_{O,7}$=106 $PHT_{O,9}$=108 $PHT_{O,11}$=52

$PHT_{O,2}$=364 $PHT_{O,4}$=76 $PHT_{O,6}$=12 $PHT_{O,8}$=100 $PHT_{O,10}$=20 $PHT_{O,12}$=168

Annual growth rate r=5%

Estimated service year of interchange=20 years

$PDT_{15,01}$=$PHT_{O1}(1+r)^{x}$

$PDT_{15,01}$=136$(1+0.05)^{20}$

$PDT_{15,01}$=361

$PDT_{15,02}$ = $PHT_{O,2}(1+r)^{x}$

$PDT_{15,02}$= 364 $(1+0.05)^{20}$

$PDT_{15,02}$=967

$PDT_{15,03}$=$PHT_{O,3}(1+r)^{x}$

$PDT_{15,03}$=260$(1+0.05)^{20}$

$PDT_{15,03}$=690

$PDT_{15,04}$=$PHT_{O,4}(1+r)^{x}$

$PDT_{15,04}$=76$(1+0.05)^{20}$

$PDT_{15,04}$=202

$PDT_{15,05}$=$PHT_{O,5}(1+r)^{x}$

$PDT_{15,05}$=320$(1+0.05)^{20}$

$PDT_{15,05}$=849

$PDT_{15,06}$=$PHT_{O,6}(1+r)^{x}$

$PDT_{15,06}$=12$(1+0.05)^{20}$

$PDT_{15,06}$=32

$PDT_{15,07}$=106$(1+0.05)^{20}$ = 282

$PDT_{15,08}$=100$(1+0.05)^{20}$=266

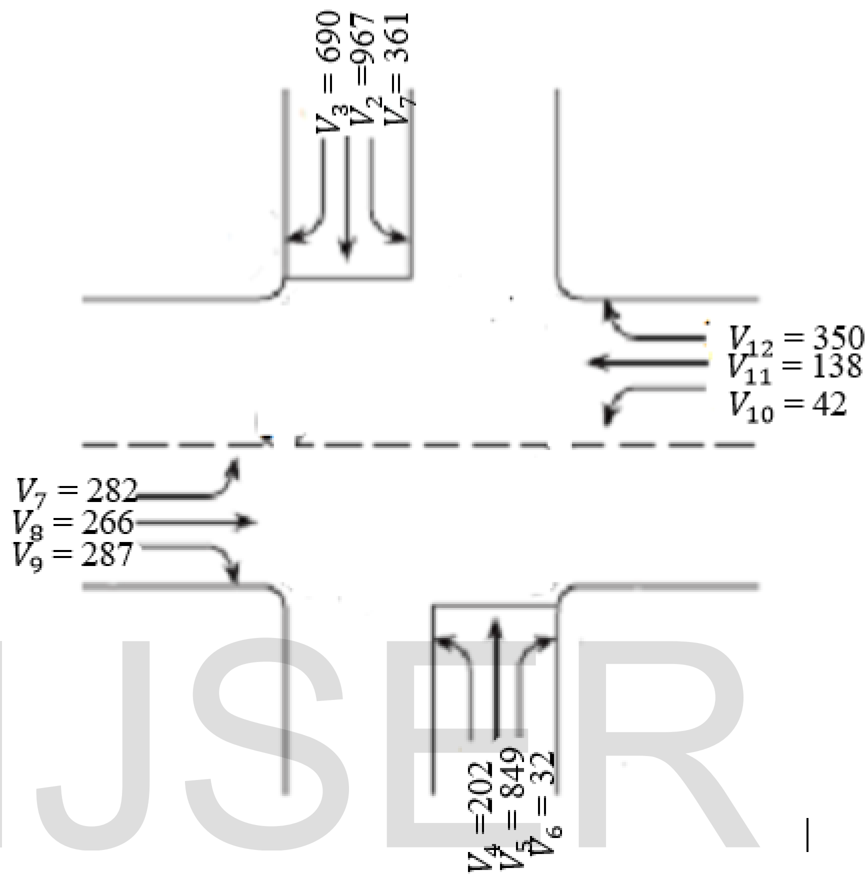
$PDT_{15,09}$=108$(1+0.05)^{20}$ = 287

$PDT_{15,010}$=20$(1+0.05)^{20}$=53

$PDT_{15,11}$=52$(1+0.05)^{20}$ = 138

$PDT_{15,12}$=168$(1+0.05)^{20}$=446

Figure 4.11 forecasted traffic movements in each approaches



4.6 Proportion of weaving traffic to non-weaving traffic Determination

$$p_{TB}=\frac{V_{2+}V_3+V_6+V_8}{V_2+V_3+V_6+V_8+V_1+V_9}=\frac{967+690+32+226}{967+690+32+226+361+287}=0.747$$

$$p_{TC}=\frac{V_{5+}V_{12}+V_8+V_9}{V_5+V_{12}+V_8+V_9+V_7+V_6}=\frac{446+849+266+287}{446+849+266+287+32+282}=0.621$$

$$p_{CD}=\frac{V_{3+}V_{11}+V_5+V_6}{V_3+V_{11}+V_5+V_6+V_{12}+V_4}=\frac{690+138+849+32}{690+138+849+32+202+466}=0.725$$

$$p_{BO}=\frac{V_2+V_3+V_6+V_8}{V_2+V_3+V_6+V_8+V_1+V_9}=\frac{967+287+138+446}{967+287+138+446+690+53}=0.712$$

Thus the proportion of weaving traffic to non-weaving traffic is highest in the direction of total gas station to Buna International Bank approach.

4.7 Determining parameters of rotary

4.7.1 Entry, Exit and island radius

$R_{exit}>R_{entry}$

$R_{exit}=1.5R_{entry}$ up to $2R_{entry}$

$R_{central\ island}=1.3*R_{entry}$ where w_{exit} =8.2m and 8.8m

w_{entry} =8.8m and 8m

$w_{circulating}$=1 to 1.2 max entry width

Weaving length (L) =4W

Weaving length=$\frac{l_1+l_2}{2}$+3.5

$w_{circulating}=1.1*w_{entry}$ =1.1*8.8=9.68

$R_{central\ island}$=x to $2w_{circulating}$

$R_{central\ island}$=34.6m-2(9.68) =15.24m

$R_{central\ island\ calculated}$=15.24 > R_{min}=15m for 35km/hr [8].

4.7.2 Weaving width from city center to total

$W_{weaving,CT}=\frac{e_1+e_2}{2}+3.5=\frac{8.2+8}{2}+3.5=11.6$m

$w_{weaving,TB}=\frac{e_1+e_2}{2}+3.5=\frac{8.8+8.8}{2}+3.5=12.3$

$w_{weaving,BD}=\frac{e_1+e_2}{2}+3.5=\frac{8.8+8}{2}+3.5=11.9$

$w_{weaving,CD}=\frac{e_1+e_2}{2}+3.5=\frac{8.8+8.8}{2}+3.5=12.3$

Weaving length is calculated as

$=W_{weaving,CT}+w_{weaving,TB}+w_{weaving,BD}+w_{weaving,CD}$

L = 11.6+12.3+11.9+12.3=48.1m

Therefore, the capacity of the rotary will be capacity of their weaving section.

$$Q_w = \frac{280w(1+\frac{l}{w})(1-\frac{p}{3})}{1+\frac{w}{l}} = \frac{280*11.6(1+\frac{8.1}{11.6})(1-\frac{0.854}{3})}{1+\frac{11.6}{48.1}} = 3180 \text{ veh/hr}$$

V_T=1722 veh/hr (current)

$\boldsymbol{Q_w > V_T}$ **Safe!**

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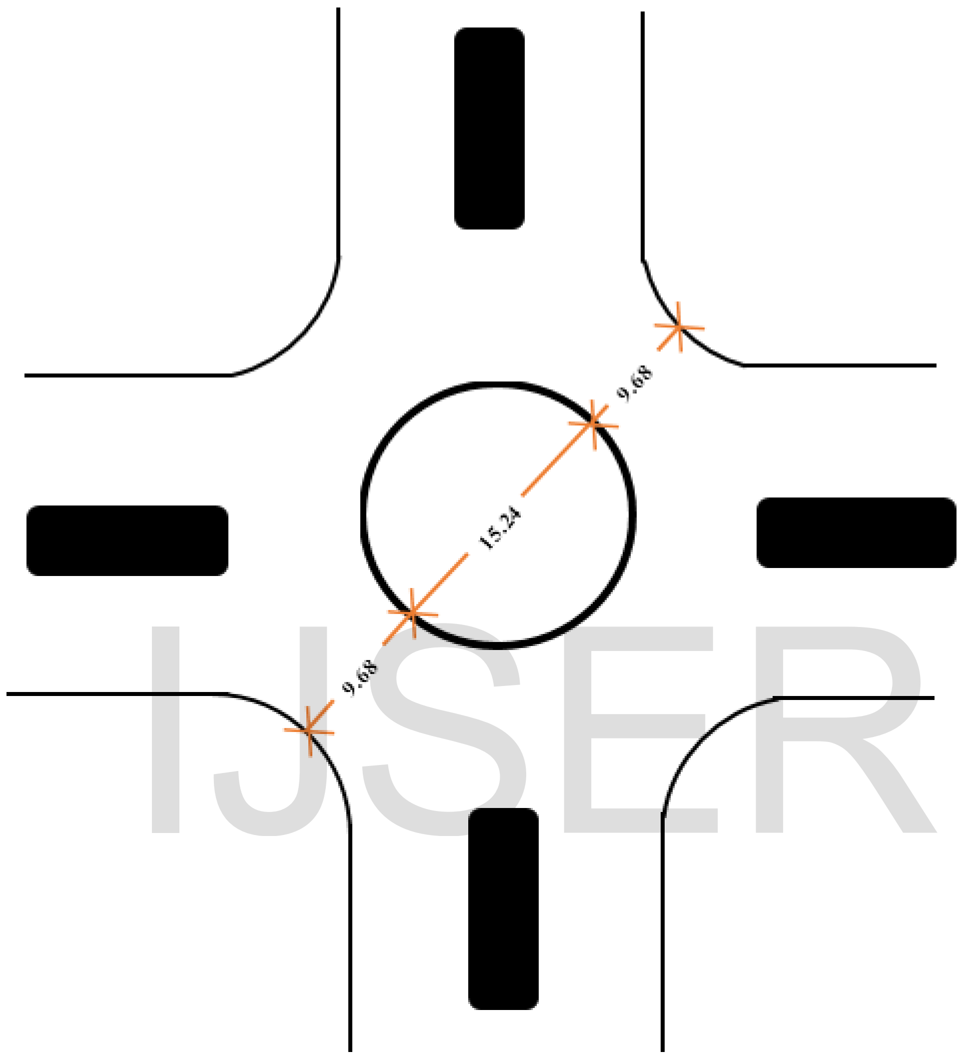
Figure 4.10 geometry of proposed roundabout



CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusions

The analysis of the selected intersection in Jimma city shows high values of degree of saturation, delay and queue length and low values of capacity in general and bad level of service. As a result of the above characteristics, congestion, tremendous economic loss, additional delay and road user cost rises from day to day. Proportion of pedestrian's traffic at the Geda intersection in Jimma city is 61% of the entire traffic and it is the root cause for the poor performance of the intersection. And among the total vehicular traffic Tricycles cover 61.91% and thus they have a great influence on the performance of intersection. Furthermore the performance of the intersection is highly affected by parking lanes at the intersections approaches. In addition to this, intersections are closely spaced in the city and it has also its own effect on the performance of the intersections.

5.2 Recommendation

The results of our analysis revealed that Geda intersection have inadequate capacity. And because of this it needs further design and improvement. For the analyzed intersection we recommend that number of tricycles should be controlled and the number of other public transports like buses and minibuses should be increased to improve the performance. Though we recommended a roundabout for the coming 20 years, the intersection should be arranged in the form of two way control till the implementation of the roundabout. Besides, as much as possible parking areas for vehicles and separate way for pedestrians should be prepared. And for the future intersections should be designed carefully and properly in Jimma Town.

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APPENDIXES

Appendix A-Traffic data

Table A-1 Geda intersection vehicles and pedestrian traffics from each approach leg on Monday

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
2:00-2:15	7	112	1	8	0	18	0	1	2	13	101	3	2.56	163
2:15-2:30	3	38	0	1	2	8	0	1	2	4	39	4	6.38	159
2:30-2:45	5	51	0	6	1	13	0	1	8	10	65	3	12.82	189
2:45-3:00	7	70	1	0	1	12	0	1	13	5	87	4	14.58	207

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
6:00-6:15	5	1	68	3	0	8	1	1	13	9	82	3	15.96	315
6:15-6:30	13	98	2	1	8	0	2	7	1	15	91	4	9.09	309
6:30-6:45	5	70	1	6	13	1	1	8	1	12	78	4	10.64	321
6:45-7:00	3	51	3	1	18	2	0	6	1	4	66	7	9.09	351

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
9:00-9:15	8	51	2	0	18	1	0	3	1	6	60	5	5.63	331
9:15-9:30	3	80	3	12	20	2	0	5	2	15	86	9	6.36	325
9:30-9:45	4	38	2	1	15	1	3	4	0	10	46	5	11.47	312
9:45-10:00	9	50	0	0	5	1	0	2	0	7	44	1	3.84	341

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Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
2:00-2:15	1	3	3	2	15	1	1	0	2	5	18	8	9.68	151
2:15-2:30	1	5	4	0	8	0	1	1	0	3	14	3	10.00	159
2:30-2:45	1	12	8	0	12	2	0	1	0	1	23	8	3.12	184
2:45-3:00	2	9	3	1	8	2	0	0	1	3	15	7	4.00	214

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
6:00-6:15	2	5	10	1	2	28	1	1	0	5	8	35	4.17	265
6:15-6:30	0	5	10	0	4	13	0	0	1	0	8	22	3.33	245
6:30-6:45	1	7	8	0	0	20	0	1	0	1	7	28	2.78	281
6:45-7:00	2	8	7	1	4	10	1	0	0	5	10	15	3.33	267

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
9:00-9:15	0	8	10	2	1	18	1	0	0	4	7	25	2.78	197
9:15-9:30	0	5	8	0	2	12	1	0	0	2	6	18	3.84	201
9:30-9:45	1	2	4	0	6	12	0	1	0	1	10	15	3.70	244
9:45-10:00	1	2	3	0	4	14	0	0	1	1	6	19	1.12	233

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Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
2:00-2:15	8	72	9	24	8	9	1	0	0	32	15	60	0.93	124
2:15-2:30	2	51	18	18	12	16	1	1	1	22	27	54	2.91	147
2:30-2:45	0	54	22	15	17	16	0	1	1	15	35	56	1.88	130
2:45-3:00	2	79	20	25	11	18	1	0	1	29	25	76	1.54	151

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
6:00-6:15	14	72	20	18	12	21	0	2	1	28	67	37	2.27	418
6:15-6:30	7	70	31	21	20	12	1	1	0	28	71	34	1.50	402
6:30-6:45	5	80	32	0	21	8	1	2	1	6	81	33	2.50	437
6:45-7:00	1	70	25	12	18	17	1	3	1	15	73	37	4.00	388

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
9:00-9:15	4	50	30	8	20	10	0	0	1	11	55	33	1.01	275
9:15-9:30	3	80	25	11	10	8	1	0	1	16	66	28	1.82	247
9:30-9:45	12	39	20	18	27	10	1	1	0	69	57	24	1.33	288
9:45-10:00	5	53	25	10	12	1	1	2	1	16	54	21	4.39	293

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Approach														
2:00-2:15	20	3	5	9	2	4	0	0	0	23	5	8	0	157
2:15-2:30	17	4	4	6	4	0	0	2	0	18	11	3	6.25	164
2:30-2:45	28	3	7	18	2	6	5	0	0	47	5	11	7.93	135
2:45-3:00	45	18	20	20	10	8	5	2	1	62	27	24	7.08	153

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Approach														
6:00-6:15	30	2	5	25	2	0	2	0	1	50	4	6	5.00	361
6:15-6:30	40	4	10	21	8	3	0	0	2	49	11	14	2.70	352
6:30-6:45	38	10	14	25	1	1	0	1	1	52	10	13	2.67	374
6:45-7:00	34	8	7	21	4	1	1	1	0	47	12	6	3.07	345

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Approach														
9:00-9:15	25	2	5	10	2	1	2	0	1	32	4	7	6.97	188
9:15-9:30	35	1	7	21	1	1	0	0	1	46	2	8	1.76	194
9:30-9:45	34	1	4	18	1	2	0	0	1	42	2	7	1.96	207
9:45-10:00	35	6	7	24	5	2	0	1	1	49	12	9	2.86	202

Table A-2 Geda intersection vehicles and pedestrian traffics from each approach leg on Friday.

Time	Tricycle			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Approach														
2:00-2:15	25	4	3	10	0	2	0	0	0	28	3	5	0	122
2:15-2:30	16	5	6	4	3	0	0	1	0	16	9	5	3.33	157
2:30-2:45	26	5	10	17	5	4	6	0	0	48	9	11	8.82	143
2:45-3:00	42	16	18	26	11	7	7	1	0	70	25	20	6.96	137

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Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Approach														
6:00-6:15	32	1	7	27	3	1	1	0	0	52	4	6	1.61	378
6:15-6:30	45	5	11	22	9	5	1	0	0	56	13	13	1.22	386
6:30-6:45	46	11	15	26	1	1	1	1	0	60	11	12	2.41	397
6:45-7:00	37	9	8	23	3	0	2	0	0	53	10	6	2.89	295

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Approach														
9:00-9:15	25	3	6	13	1	1	3	0	0	37	4	6	6.38	201
9:15-9:30	38	2	8	27	2	1	0	1	1	54	6	9	2.89	212
9:30-9:45	35	0	5	21	2	2	1	1	0	48	4	6	3.45	194
9:45-10:00	39	7	8	26	6	3	1	0	0	56	11	9	1.32	222

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
2:00-2:15	5	125	2	9	2	30	2	0	3	17	4	124	3.45	142
2:15-2:30	3	46	3	0	1	9	1	1	1	5	6	44	5.45	156
2:30-2:45	6	62	0	5	2	15	1	0	7	12	2	73	9.19	163
2:45-3:00	8	76	0	4	2	22	1	0	15	12	4	106	13.11	132

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
6:00-6:15	12	102	3	4	27	1	1	9	0	15	117	4	7.35	339
6:15-6:30	8	78	2	8	16	1	0	11	0	14	93	3	10.00	346
6:30-6:45	9	80	2	4	18	1	1	8	1	13	90	5	9.26	368
6:45-7:00	5	61	6	2	25	0	0	5	0	6	78	5	5.62	337

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Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
9:00-9:15	10	60	3	2	29	1	1	4	0	11	79	4	5.32	101
9:15-9:30	3	88	4	11	28	3	1	6	0	16	102	6	5.64	135
9:30-9:45	5	42	3	2	19	2	4	7	0	14	63	5	12.19	144
9:45-10:00	9	54	0	5	5	0	0	3	0	12	49	0	4.92	165

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
2:00-2:15	0	4	5	3	2	17	0	0	0	3	21	5	0	135
2:15-2:30	1	5	7	0	0	13	0	0	0	1	18	4	0	121
2:30-2:45	2	7	17	2	3	16	0	0	0	4	28	8	0	132
2:45-3:00	1	6	10	1	3	16	0	0	0	2	23	8	0	145

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
6:00-6:15	3	7	14	0	5	33	0	0	0	3	10	43	0	384
6:15-6:30	2	7	17	2	5	20	0	0	0	4	10	32	0	391
6:30-6:45	2	6	6	0	4	23	0	0	0	2	9	28	0	402
6:45-7:00	3	9	6	0	6	14	0	0	0	3	13	19	0	374

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
9:00-9:15	2	9	15	3	3	20	1	0	0	6	10	31	2.13	294
9:15-9:30	3	7	10	0	4	19	0	0	1	3	9	28	2.50	221
9:30-9:45	2	3	8	1	7	18	0	0	0	3	10	24	0	235
9:45-10:00	1	5	4	0	5	20	0	1	0	1	11	23	2.86	227

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
2:00-2:15	9	81	9	13	27	14	0	1	1	34	22	73	1.55	157
2:15-2:30	3	63	23	18	23	17	0	1	0	26	37	62	0.79	178
2:30-2:45	2	63	28	20	18	20	0	0	2	20	40	69	1.55	156
2:45-3:00	4	88	27	28	27	12	0	0	1	31	31	91	0.65	152

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
6:00-6:15	12	88	29	26	18	22	0	3	2	35	86	47	2.98	426
6:15-6:30	8	75	39	25	23	22	0	2	0	31	80	50	1.24	413
6:30-6:45	10	91	35	15	24	10	0	3	2	22	94	39	3.22	446
6:45-7:00	8	75	26	15	28	21	0	4	1	21	89	42	3.29	402

	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
9:00-9:15	5	60	35	10	26	16	1	1	0	16	70	41	1.57	411
9:15-9:30	5	84	30	18	12	12	0	0	0	22	71	33	0	387
9:30-9:45	13	45	27	20	30	17	0	1	1	30	64	36	1.54	391
9:45-10:00	7	65	30	20	17	4	0	0	0	25	63	25	0	408

Table A-3 Geda intersection of vehicles and pedestrian traffics from each approach leg on Saturday

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
2:00-2:15	7	48	22	27	21	21	2	2	2	36	59	41	4.41	221
2:15-2:30	4	51	26	24	18	6	0	2	1	31	58	27	2.67	255
2:30-2:45	10	64	39	25	19	6	1	1	0	34	24	34	1.49	247
2:45-3:00	7	57	41	20	15	17	0	2	0	25	59	46	1.54	261

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
2:00-2:15	1	58	2	2	17	3	0	3	0	3	64	5	4.17	194
2:15-2:30	6	70	3	3	25	1	1	9	0	10	92	4	9.43	222
2:30-2:45	5	48	1	4	12	1	0	3	0	8	52	2	4.84	236
2:45-3:00	9	81	4	4	12	3	1	4	0	13	77	6	5.21	248

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Jimma Approach														
2:00-2:15	22	6	6	8	0	1	0	0	1	24	5	8	2.70	212
2:15-2:30	23	6	8	13	1	1	2	0	0	34	6	7	4.25	231
2:30-2:45	40	5	5	16	8	2	0	0	0	44	12	8	0	241
2:45-3:00	36	3	4	13	0	3	0	0	1	39	3	8	2.00	228

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
2:00-2:15	2	2	14	3	0	37	0	0	1	5	2	49	1.78	121
2:15-2:30	2	2	13	1	0	37	0	0	0	3	2	47	0	132
2:30-2:45	2	3	12	1	2	34	0	0	0	3	5	43	0	154
2:45-3:00	6	3	10	3	3	38	0	0	1	8	6	47	1.64	142

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
6:00-6:15	23	118	55	23	17	28	1	1	4	42	102	75	2.74	281
6:15-6:30	13	93	60	20	30	12	0	1	2	30	98	58	1.61	301
6:30-6:45	9	84	55	27	24	22	0	4	2	34	91	65	3.16	318
6:45-7:00	15	60	32	23	22	18	0	0	1	34	64	43	0.71	306

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Dashen Bank Approach														
6:00-6:15	12	85	0	4	28	0	0	3	0	13	63	0	0	287
6:15-6:30	12	81	5	2	19	1	0	5	0	11	86	5	3.79	271
6:30-6:45	22	79	3	3	20	0	0	2	0	19	80	3	1.49	291
6:45-7:00	15	73	0	5	20	2	2	6	0	20	84	2	6.30	281

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Jimma Approach														
6:00-6:15	48	9	17	21	2	5	2	0	0	59	9	17	2.35	319
6:15-6:30	53	6	16	23	5	3	2	0	1	79	10	17	2.83	311
6:30-6:45	43	12	11	32	12	15	3	2	2	69	25	27	5.18	280
6:45-7:00	53	16	7	29	8	7	1	1	1	69	20	21	2.73	308

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
6:00-6:15	3	10	24	4	1	27	0	0	0	7	8	44	0	251
6:15-6:30	4	17	23	0	2	27	0	0	0	3	14	44	0	245
6:30-66:45	6	12	17	0	4	30	0	0	0	5	13	42	0	263
6:45-7:00	2	12	13	0	3	27	0	0	0	2	12	37	0	236

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Total Gas Station Approach														
9:00-9:15	5	74	45	27	16	20	0	1	0	31	70	52	0.65	287
9:15-9:30	6	56	25	19	19	10	0	2	2	24	63	32	3.36	346
9:30-9:45	10	70	30	22	19	10	1	0	0	31	68	31	0.77	256
9:45-10:00	19	65	33	22	30	18	0	0	0	36	76	42	0	203

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Geda Super Market Approach														
9:00-9:15	13	86	1	1	15	2	1	2	0	13	80	3	3.13	324
9:15-9:30	7	56	6	3	13	1	0	5	0	8	63	6	6.49	316
9:30-9:45	4	60	2	3	6	1	1	5	0	8	58	3	8.69	315
9:45-10:00	9	63	2	1	9	0	0	3	0	8	60	2	4.29	297

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
City Center Jimma Approach														
9:00-9:15	41	3	10	16	2	3	2	1	1	49	7	12	5.88	271
9:15-9:30	41	5	7	21	5	1	0	0	0	50	9	6	0	226
9:30-9:45	31	6	6	18	2	3	1	0	0	42	7	8	1.75	272
9:45-10:00	37	7	12	21	1	2	1	0	0	49	6	11	1.52	293

Time	Motor Bicycles and Bajaj			Light Vehicles			Heavy Vehicles			Total Vehicles (pcu)			Percentage Of Heavy Vehicles	Pedestrians
To	L	S	R	L	S	R	L	S	R	L	S	R		
Buna Int. Bank Approach														
9:00-9:15	4	5	18	0	3	26	0	0	0	3	7	39	0	245
9:15-9:30	4	5	10	0	4	23	0	0	0	3	8	30	0	214
9:30-9:45	3	7	11	0	2	24	1	0	1	5	7	34	4.35	269
9:45-10:00	3	7	13	0	3	25	1	0	0	5	8	35	2.08	271

Table A-4 Geda intersection of vehicles and pedestrian traffics from each approach leg on Monday counted for 10 successive hours

Time	TOTAL GAS STATION APPROACH			CITY CENTER JIMMA APPROACH			DASHEN BANK APPROACH			BUNA INT. BANK APPROACH		
To	L	S	R	L	S	R	L	S	R	L	S	R
1:00-2:00	76	112	83	108	25	32	21	132	8	7	9	39
2:00-3:00	98	155	102	132	48	46	32	165	9	12	26	70
3:00-4:00	85	99	78	107	31	29	19	154	7	5	13	56
4:00-5:00	81	111	79	95	26	27	21	143	9	4	15	46
6:00-7:00	77	175	141	121	37	39	40	138	15	11	30	100
9:00-10:00	135	157	106	109	20	31	38	126	11	8	29	77
10:00-11:0	97	141	79	99	21	27	42	112	10	9	23	61
12:00-1:00	71	104	88	68	25	16	24	81	8	14	19	45
1:00-2:00	34	75	59	63	15	10	18	54	4	7	11	37
2:00-3:00	18	43	41	37	5	9	11	43	2	3	9	21
∑	772	1172	856	939	253	263	266	1148	83	80	184	552

Table A-5 Geda intersection of vehicles and pedestrian traffics from each approach leg on Saturday counted for 10 successive hours

Time	TOTAL GAS STATION APPROACH			CITY CENTER JIMMA APPROACH			GEDA SUPER MARKET APPROACH			BUNA INT. BANK APPROACH		
To	L	S	R	L	S	R	L	S	R	L	S	R
Morning Time												
1:00-2:00	81	123	101	97	14	19	59	131	9	11	7	79
2:00-3:00	126	200	148	141	26	31	92	285	17	19	15	89
3:00-4:00	78	89	78	83	19	21	63	113	14	20	17	80
4:00-5:00	87	89	88	97	87	13	42	98	16	11	12	95
Afternoon Time												
6:00-7:00	140	355	241	276	64	82	63	313	10	17	47	167
9:00-10:00	98	213	111	54	41	37	36	139	11	2	21	67
10:00-11:00	78	131	101	190	29	90	37	98	14	16	30	93
Night Time												
12:00-1:00	59	78	61	27	11	59	21	41	5	9	11	39
1:00-2:00	27	38	41	18	7	24	9	19	4	4	3	12
2:00-3:00	13	16	15	8	3	13	3	11	3	2	1	7
∑	787	1332	985	991	301	389	425	1248	103	111	164	728

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Appendix B- Average daily traffic data calculation

Saturday	
1. 16 hour traffic count for total gas station approach v_R =1576,v_S=2132,v_l=1260 A) ADT_R=1640 veh/Day B) ADT_S=2218 veh/Day C) ADT_l=1311 veh/Day 2. Dashen bank approach traffic count for 16 hours v_R=680 , v_S=2077, v_l=172 A) ADT_R=708 veh/Day B) ADT_S=2161veh/Day C) ADT_l=179 veh/Day	3. JIMMA city center approach traffic count for 16 hours v_R=623, v_S=482, v_l=1586 A) ADT_R=648 veh/Day B) ADT_S=502 veh/Day C) ADT_l=1650 veh/Day 4. Buna int'l bank approach traffic count for 16 hours v_R=1165, v_S=263, v_l=178 A) ADT_R=1212 veh/Day B) ADT_S=274 veh/Day C) ADT_l=186 veh/Day

Appendix C - Geda intersection of vehicle traffics from each approach leg on Saturday counted for 10 successive hours

Saturday