ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

Performance Evaluation of AALRT Level Crossings along North- South Line

Case study on Adey Ababa and Sebategna Level Crossings

A thesis submitted to the graduate school of Addis Ababa University in partial fulfillment of the requirements for the degree of Masters of Science in Civil Engineering (Road and Transport Stream)

BY

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Declaration

I, Lemmi Gurmessa, declare that "Performance Evaluation of AALRT Level Crossings along North- South Line" a case study on Adey Ababa and Sebategna Level Crossing has not been presented for award of any degree or diploma in this or other universities. All sources of materials used for this thesis work have been fully acknowledged.

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EXECUTIVE SUMMARY

Railway grade crossings are the intersection of two modes of transportation with very different physical and operational characteristics. There are many variables factors that influence a motorist’s ability to react appropriately at grade crossings. These factors include the information available as the driver approaches, the crossing and human factors such as competing decisions, distractions, and impaired driving.

This paper is intended to study the performance evaluation of AALRT Level crossings where road traffic (Road Vehicles), Pedestrians and LRT vehicles (Trains) are computing for crossing each other along the North –south line. This area is one of the important places that belongs to both the rail line and the road as well the bottle neck for the operation of trains. Performance evaluation using Average control delay and Level of service of level crossings were the main topic covered with in the paper.

Primary data (Traffic Volume data at crossings) have been collected using Video camera for Six hours at each level crossings. Based on the collected Traffic volume, the traffic distribution to each direction has been counted carefully to minimize the error including pedestrian’s volume.

Sidra Intersection 8 software along with Highway capacity manual 2010 (HCM 2010) have been used to evaluate the Performance of theses Level Crossings at 3, 6, 10,15- and 20-minutes’ headway of Addis Ababa Light Rail Transit (AALRT) operation.

Intersection models with different Conditions have been considered to see the effects of the presence of pedestrians, Light rail Vehicle (LRV) Frequencies and the type of Traffic Control (Signaling and Yield Sign control).

According to the results the presence of pedestrian is sounder in Yield control methods than the Signal control for the same Intersection geometry, LRV frequency and Traffic Flows.

Comparison of the Signaling and Yield Control results at the existing traffic condition and Level Crossings geometry conditions shows that Signaling control will perform better than the Yield control.

According the result of both from Signaling and Yield control models, both level crossings will not perform at LOS D (Target LOS) at the existing Geometric and Traffic Flow conditions.

Finally, the Level crossings were modelled with traffic movement and Geometric modifications under signaling control and the result shows that these level crossings will perform at least at LOS D if the recommended modifications have been set accordingly.

KEY Words: ***Level Crossing, Pedestrian Crossing, Addis Ababa Light Rail Transit, Intersection Performance, LOS.***

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Abbreviations

AALRT Addis Ababa Rail Transit

ABCL: automatic barrier crossing locally monitored

AHB: automatic half barrier crossing

AOCL: automatic open crossing locally monitored

CB-OD: controlled barrier crossing with obstacle detection

e.g. Example

E-W East west

FP (MSL): footpath crossing with miniature stop lights

GDP Gross Domestic Product

HCM Highway Capacity Manual

Ht or H Time head way

LC Level Crossings

LOS Level of Service

LRV Light Rail Vehicle

LT Left Turn

MCB: manually controlled barrier crossing

MCB (CCTV): manually controlled barrier crossing with closed circuit television

MUTCD:

MCG: manually controlled gated crossing

N-S North South

NCHRP National Cooperative Highway Research Program

OC: open crossing

ORR Office of Railway Regulation

RT Right Turn

TMO: train crew (or other peripatetic railway staff) operated crossing

UWC (MSL): user worked crossing with miniature stop lights

UWC (T): user worked crossing with telephone

WHO World Health Organization

CHAPTER 1 INTRODUCTION

1.1 Background

Addis Ababa, the Capital city of Ethiopia, the center of Oromia Region, the head quarts for different international organization and National Organization, the Center of Politics and Economy, faces transportation challenges in day-to-day activities. The government of Ethiopia has set different programs for the sustainable solution of transportation problem across the country. In the same way, Addis Ababa City administration has implemented the policy of public transport encourages by sharing in providing Anbessa Buses and Shager Express Buses into the transportation logistics. On the other hand, the Federal Government has constructed the Light rail transit line in Addis Ababa to solve the transportation problems and challenges.

Addis Ababa Light Rail Transit or AALRT is a light rail transportation system in Addis Ababa, Ethiopia. Ethiopia is the only country that has introduced light rail transit as one of transport mode from in eastern and sub-Saharan Africa (Clelie February 2018), (Railwaygazette 2015). AALRT is objected to increase the use of public transportation and to reduce the problem that were raised in the city even though the implementation is limited to only two Corridors, i.e., West-East and South – North.

The north- south line phase I project starts from Menelik II Square and ends at Kaliti. The total length of the line is 16.97km. There are 22 stations, among which 9 are elevated stations, 2 underground station and 11 ground stations (China Railway Group 2009). This North-South line has a total of 4 at level crossings in which one of them is at roundabout.

The east-west line phase I project starts from Hayat and ends at Torayiloch. The total length of the line is 17.4km. There are 22 stations, among which 5 are elevated stations, 1 underground station and 16 ground stations including 5 common stations at the common line. On this corridor there are a total of 5 at level crossings which includes two roundabouts (China Railway Group 2009).

1.2 Statement of the problem

AALRT is one of the latest transportation systems in Addis Ababa. It is design to serve the passenger with in head time of 6-15 minutes in one direction flow. However; the annual report of AALRT for fiscal year 2014 E.C (2021/2022 G.C) shows that by the year of 2014 E .C, the service head time was 10-15 minutes at peak hour and 15 minute at off peak hour for both the N-S line and E-W line which satisfy the operation plan for that year (Ethiopia Railway Corporation 2022).

Level crossings like any Junctions and Intersections, have been the potential place for the congestion of traffic. At level crossings all arrived traffics are willing to take place or cross each other based on their destination place. On the other hand, the Light Rail Vehicle (LRV) has a priority over the others vehicles which could lead the Level crossings more congested if the LRV has frequently interrupted the crossing traffic over the Level crossings which in turn lead to reduce the level of service of the adjacent road and the intersection (Level Crossings).

It is well Known that intersection and Level crossings are the potential place for conflict if not managed properly. At these level crossings there are different traffic conflicts between the road users and the rail.

- ✓ Road Vehicles with Rail Vehicles,
- ✓ Rail Vehicles with Pedestrians,
- ✓ Road Vehicles with Pedestrians; and
- ✓ Road Vehicles with other Road vehicles.

Due to these conflicts at level crossings, there are congestion on the road that cross the rail line and as well as on adjacent roads/parallel to rail line.

To make the intersection safe and sufficient during service, the intersection shall be evaluated weather it is facilitated with crossing safety tools including its geometry. In line with this, at some AALRT level crossings one can observe some negative impacts on the adjacent roads such as delays and Queuing at and near the peak hours.

Having in mind that the Railway infrastructure (Stations, Track Geometry, Man power, Rail vehicles, etc.) is enough to serve at 6minutes head time, meanwhile, we need to know the effect of this design head time at selected level crossings along North – South line with answering for the following research Questions.

- What would be the effect of this head time on the adjacent road around the level crossing?
- Which level crossing can sustain the 6 minutes AALRT operation with out or with small modification?
- What would be the optimum operation time so that the Level crossing operates at least at LOS D?
- What type of remedial measurement shall be taken to increase the performance of Level crossings at least to LOS D?

1.3 Hypothesis

Decreasing the time head way (Increasing the frequency of LRV) of AALRT operations will increases the negative impacts on the road section adjacent to the rail line and will lowers the performance of the level crossings. Additionally, there could be some level crossings that could never operate at 6 minutes (One direction) or 3 minutes (both direction) of AALRT operations without modifications while others can operate at least at LOS of D.

1.4 Objectives

1.4.1 Main Objective

- To evaluate the performance of AALRT level Crossings at 6 minutes' head time of AALRT operations.

1.4.2 Specific Objectives

- To determine the LOS of the adjacent roads at around the level crossings at 6 minutes' head time of AALRT Operation
- To identify the level crossings which can accommodate the traffic operation at LOS of at least D while Light Rail Vehicle operation is at 6 minutes.
- To know the effect of pedestrians and LRV head time on the level crossings
- To recommend the level crossings which need some modification to operate the light rail vehicle at 6 minutes' head way.

1.5 Scope of the Research

AALRT consists of nine (9) Level crossings, 4 on North – South line and 5 along East- West line and Pedestrian crossings at each Level Stations. Covering at all these points may needs more time as well as resources and out of this research scope. Hence, the number of crossings to be included in this research are limited to only two-Level crossings along the South – North Line, Adey Ababa and Sebategna Level Crossings. The research evaluates the performance of the selected level crossings based on two methods of intersection controls; Yield (Giveaway) and Signaling control system and two performance measures (Control delay and queue length).

1.6 Thesis Organization

The thesis has organized into five main Parts as summarized below.

The first part is the introductory section which consists of back ground, Statement of problem, Objective of the thesis and scope of the thesis.

The second section is given for literature reviews. In this section literatures related to level crossings, intersection performance evaluations, Performance measure elements and other subtopics intended as input for this research has been reviewed in detail.

The third part is the methodology and material of the research through which the thesis was done. Under this section the procedure for the completion of the thesis has been set. Traffic data collection, analysis and interpretation methods were selected. Additionally, the methods of Level crossing were selected for the analysis.

The fourth part is the section where the main objectives of the thesis has got answer. Under this section, the level crossings model analysis output has been interpreted and discussed in detail. The performance evaluation of Yield control and Signal control for the selected level crossings has been evaluated and compared for different light rail vehicle frequencies.

The last section, Section 5, summarizes the main findings of the research which contains conclusion and recommendation.

Lastly, References and appendices are incorporated with in the paper. Tables, Charts and figures were included in the report for more elaboration where required.

1.7 Significant of the thesis

This paper is significant for the policy maker and transport planner related to the Addis Ababa Light Rail Transit regarding the determination of operational head time. it is also significant for the Addis Ababa Transport Bureau for the decision in between the Yield control and Signal control of intersections and remedial measurement related to Level crossings' geometry change/Modification.

CHAPTER 2 LITERATURE REVIEW

2.1 Level Crossing

U.S. Department of transportation defines that "Highway-railroad grade crossings are intersections where a highway crosses a railroad at-grade. They are also called level crossings in other countries such as Canada, Australia, and the United Kingdom" (U.S. Department of Transportation 2020). Rail-highway grade crossings are potential points of conflict between roadway traffic and trains. The train has right of-way in these conflicts, and it is the driver's responsibility to yield to the train (NCHRP 2002).

Level crossings can be dangerous if there is poor sight distance to a signal display, or to approaching trains; traffic control is inadequate; vehicles queue across tracks due to congestion; there are a lack of pedestrian facilities; pavement is not maintained; signaling equipment is located too close to the road (Road Safety 2020).

To avoid collisions, warning/control devices are required at grade crossings just like intersecting roads need stop signs or traffic signals.

2.1.1 Types of Level Crossings

The classification of Level crossings depends based on the type of road to be crossed (according to Indian Classification) and based on the way of controlling Traffic at level crossings. According to Indian classification there are 5 types of level crossings. Namely: Special class, 'A' class, 'B' class, 'C' class and 'D' class (Ministry of Railways 2009) as described below.

Special class: These are the busiest level crossings in terms of road traffic. Most of the busy level crossings on the national highway are special class level crossings. Normally the gates are open to road traffic but whenever a train passes by, the gates are closed to road traffic. The gates of the level crossings are interlocked with signals. They are manned round the clock by three gatemen working 8-hour shifts.

'A' class: These level crossings are also busy in terms of road traffic. All level crossings on important roads are mostly 'A' class level crossings. In this case also, the gates are normally open to road traffic. All other provisions are the same as for special class level crossings except that these level crossings are provided with only two gatemen who work in 12-hour shifts, as these crossings are not as busy as special class level crossings.

'B' class: These level crossings are relatively less busy. Normal B class level crossings can be found on metaled roads. The gates are normally closed to road traffic, but can be kept open to road traffic provided that the gates are interlocked with signals. They are provided with two gatemen working 12-hour shifts.

'C' class: These level crossings are mostly provided on unmetalled roads. Some of these level crossings are unmanned because of low volume of road traffic.

'D' class: These level crossings are provided for cattle; they are normally used by cattle or pedestrians.

On the other hand, Railway level crossings can be classified as active and passive level crossings based on the road traffic control mechanism. Active level Crossings have active warning and control devices such as bells, flashing lights, and automatic gates in combined with static control such as traffic sign and Pavement Markings (NCHRP 2002) (U.S. Department of Transportation 2020) (Road Safety 2020).

At a passive rail-highway grade crossing, the traffic-control system does not inform roadway users of the approach or presence of trains, locomotives, or railroad cars (NCHRP 2002), (U.S. Department of Transportation 2020). Passive control systems provide warnings through signs and line markings (Road Safety 2020).

Figure 2-1: Level Crossing Control Type (Beanland, V., Lenné, M. G., Salmon, P. M., & Stanton, N. A. (in press) 2015)

According to office of Rail Regulation (ORR), the basic protection and warning arrangements for railway level crossing is well described in table 2-1 below. Accordingly, the railway level crossing can be protected or unprotected from train movements with different ways of controlling the road traffic from conflict.

As per (ORR 2011), Protection from train movements ensures that trains are not authorized to pass over the crossing until the crossing is closed and the crossing area has been checked to be clear. Unprotected crossings depend on a warning being given to crossing users of an approaching train so that they can be clear before the train arrives. It is unlikely that the train can be stopped if the crossing is not clear.

Telephones are fitted to several crossing types for a range of purposes. At a user worked crossing with telephone, UWC (T), the warning of an approaching train is achieved by contacting the signaler. For this to be effective the user must make the call and the signaler must be able to advise how close the nearest train is.

Table 2-1: Level crossing types – basic protection and warning arrangements (ORR 2011)

Protection from Train Movement	**Crossing Confirmed Clear**	**Warning arrangements**	**Full barriers/ gates**	**Half barriers**	**No barriers**	**Telephone "protection"**
Protected	By signaler or crossing keeper		MCG			
			MCB			
			MCB (CCTV)			
	By obstacle detector		CB-OD			
	By driver			ABCL		
					AOCL	
	By train crew/other		TMO			
Unprotected		Approaching train		AHB		
					UWC(MSL)	
					FP(MSL)	
		Telephone				UWC (T)
		Line of Sight			OC	
					UWC	
					FP/BW	

Where,

MCG: Manually Controlled Gated Crossing

MCB: Manually Controlled Barrier Crossing

MCB (CCTV): Manually Controlled Barrier crossing with closed circuit television

CB-OD: Controlled Barrier crossing with obstacle detection

ABCL: Automatic Barrier Crossing Locally monitored

AOCL: Automatic Open Crossing Locally monitored

TMO: Train crew (or other peripatetic railway staff) Operated Crossing

AHB: Automatic Half Barrier crossing

UWC (MSL): User Worked Crossing with Miniature Stop Lights

FP (MSL): Footpath Crossing with Miniature Stop Lights

UWC (T): User Worked Crossing with Telephone

OC: Open Crossing

UWC: User Worked Crossing

FP/BW: footpath or bridleway crossing

2.1.2 Level crossing capacity

Capacity is defined as the maximum hourly rate at which persons/vehicles can reasonable expected to traverse a point or Uniform section of a lane or road way during given period (Usually 15 minutes) (May 1990). Capacity would be the maximum volume that could be accommodated by the highway at the study location (Roses 2004).

The Capacity of railway level crossing consists of railway and road capacity part. Both transport modes have own measure units. But for mixed view, there is a lack of common measure system and universal units. Train operations are prioritized at railway crossing, theoretically until the entire capacity is used. Therefore, calculating only, the remaining road capacity can already be considered as sufficient (Martin Sojka 2016).

According to Martin Sojka (2016), Railway demands capacity only if train is coming and the necessary capacity is then reserved with priority (either supported by technique or at passive crossing through right of way). The road transport uses remaining capacity. In practice, road transport uses only one part of the remaining capacity and the rest stays as general reserve. Even if enough reserves exist in the system, the delay appears systematically, because the almost continuous flow of road traffic is regularly disturbed by trains. In these moments road traffic is stopped and delayed even if the level crossing capacity is widely not reached.

In the case of real lack on capacity, which means the requirements of both transport modes is not possible to satisfy over a longer period of time, the real impact affects road transport mode only. The railway system does not need any additional capacity, because, if necessary, the whole capacity will be allocated for the railway use only.

2.2 Pedestrian Crossings

Pedestrian crossings are the crossings that allowed only for the cross of pedestrians across the Railway line or /and Highway section.

According to HCM (2010), the methodology provides a variety of measures for evaluating facility performance in terms of its service to pedestrians. Each measure describes a different aspect of the pedestrian trip along the facility. One measure is the LOS score. This score is an indication of the typical pedestrian's perception of the overall facility travel experience. A second measure is the average speed of pedestrians traveling along the facility. A third measure is based on the concept of "circulation area." It represents the average amount of sidewalk area available to each pedestrian walking along the facility. A larger area is more desirable from the pedestrian perspective (HCM 2010).

2.3 Road Safety

Traffic safety is the major problem in a developing country. According to WHO (2020) report approximately 1.35 million people lost their lives due to traffic accident in the world. About 20-50 million people suffer non-fatal injuries, with many incurring a disability as a result of the injury. The road traffic crashes cost most countries 3% of their gross domestic product (GDP). WHO (2020) States that 93 % of the world’s fatalities on the road occur in the low-middle income countries, even though these countries have approximately 60% of the World’s Vehicles.

Based on the data traffic accident reports of 2016 by WHO, the rates of road traffic death are highest in Africa (26.6/100,000 people) and South East Asia (2.7/100,000 people) While Europe is the lowest fatal rates (9.3/100,000 people) and America is the next smallest fatal rates with 15.6/100,000 people (W. H. WHO 2018).

According to the WHO data Published in 2018 Road traffic accident deaths in Ethiopia reached 29,386 or 4.81 % of the total death with death rate of 36.78 per 100,000 people at world rank of 24 (WHO 2018).

Among the various types of road accident, accidents between trains and road vehicles are the strongest and most expensive accidents (Behzad Dezhkam & Seyed Mehrdad Eslami 2017). By its property Rail transportation is the Safest and Reliability mode of land transport with the lowest accident rate but high fatality when compared with road traffic. For the same traffic, the risk of fatality occurring is eight times greater in road than in rail transport, whereas the risk of injury is 200 times more likely to occur on Highway than rail transport. (V.A.Profillidis 1995)

According to U.S. Department of transportation, 94% of all rail-related fatalities and injuries occur at railroad crossings or due to trespassing. Sadly, almost all of these deaths and injuries are preventable (U.S. Department of Transportation 2020).

2.4 Traffic Characteristics Data

2.4.1 Flow Rate (Traffic Volume)

Traffic flow is of fundamental importance in developing and designing strategies for intersection control, rural highways, and freeway segments. The demand flow rate for an intersection traffic movement is defined as the count of vehicles arriving at the intersection during the analysis period divided by the analysis period duration. Volume is the number of vehicles passing a point during specified time period and rate of flow is the rate at which vehicles pass a point during the specified time (Roses 2004). Flow rate is expressed as an hourly flow rate but may represent an analysis period shorter than 1h.

There are four important parameters of Flow rate characteristics ((May 1990): Existing Traffic Demand, Service Volume, Capacity and Saturation Flow rate.

Demand is the number of vehicles (person) that desire to travel past a point during the specified period and frequently higher than the actual volumes where congestions exist ((Roses 2004). According Adolf D. May (1990) definition, Existing Traffic demand is the unique flow rate value that plays an important role when analyzing the oversaturated or Congested situation and represents the flow rate at which vehicles would like to be serviced. The existing traffic demand is numerically equal to the measured flow rate if there is no oversaturation upstream or congestion at the site in question.

According HCM 10, Demand flow rate represents the flow rate of vehicles arriving at the intersection. When measured in the field, this flow rate is based on a traffic count taken upstream of the queue associated with the subject intersection. This distinction is important for counts during congested periods because the count of vehicles departing from a congested approach will produce a demand flow rate that is lower than the true rate.

Service Volume is defined as the maximum hourly rate at which persons or vehicles can be reasonable be expected to transverse a point or short section of the lane or roadway during the study period (Usually 15 minutes) under prevailing road way, Traffic and control conditions while maintaining the designated level of service (May 1990).

Adolf D. May (1990) also defines the capacity flow rates as the maximum hourly rate at which persons or vehicles can reasonably be expected to transvers a point or short section of the lane or roadway during the study period (Usually 15 minutes) under prevailing road way, Traffic and control conditions whereas the saturation flow rate is defined as the equivalent maximum hourly rate at which vehicles can transverse a lane or intersection approach under prevailing traffic and roadway conditions assuming that the green signal was available at all times that no loss times are experienced.

It has been noted that volume, demand and capacity are three different, even though all are expressed in the same units and may relate to the same locations. In the figure 2-2 below, a bottle neck location on a freeway, for each approaching leg and for the downstream freeway section, the actual Volume(V), the demand (d) and the capacity (c) of the segment are given.

As of the example illustrated in Roses (2004) in the following figure the capacity is 2000 Veh/hr/lane and the capacity of the two-lane approach legs will be 400veh/hr/ln whereas for the downstream with three legs will be 6000 veh/hr/ln. The queue at the approaching legs is observed due to the fact that the capacity of the downstream legs is less than the demand (3800+3600=7400) V/eh/hr whereas the demand flow at the approaching legs is less than the capacity of the respective approaching legs.

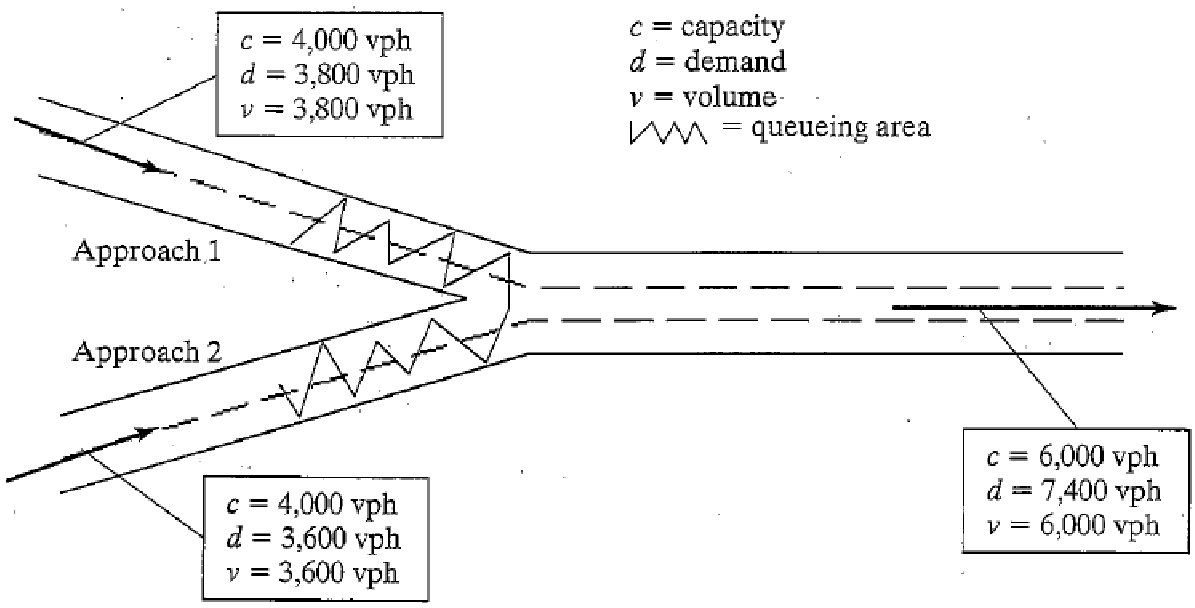

Figure 2-2: Volume, Demand and Capacity at a bottle neck location (Roses 2004).

2.4.2 Peak Hour Factor and Intersection Peak Hour Volume

Peak hour factor is a measure of the variability of demand during the peak hour. It is the ratio of the volume during the peak hour to the maximum rate of flow during a given time period within the peak hour. For intersections, the time period used is 15 min, and the PHF is given as below (HCM 10).

$$PHF = \frac{\text{Volume During PeakVolume}}{4 * \text{volume during peak 15 min within peak hour}}$$

The use of a single peak hour factor for the entire intersection is intended to avoid the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-minanalysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-minperiod that are in apparent conflict with demand volumes from another 15-minperiod, whereas in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-mincount data are likely available, permitting the determination of actual 15-mindemand and avoiding the need to use a peak hour factor (HCM 10).

Peak Hour Volume (PHV) is the maximum number of vehicles that pass a point on a highway during a period of 60 consecutive minutes ((Nicholas J. Garber and Lester A. Hoel 2009).

According to Nicholas J. Graber (2009) PHVs are used for: Functional classification of highways; Design of the geometric characteristics of a highway, for example, number of; lanes, intersection signalization, or channelization; Capacity analysis; Development of programs related to traffic operations, for example, one-way; street systems or traffic routing and Development of parking regulations

2.4.3 Left turn and Right Turn Treatments

At an intersection a vehicle could go Through, Left Turn, U-Turn or Right Turn based on their destination wishes. The right turn movement has non considerable effect with the traffic flow except with the pedestrian movements and normally allowed during all signal phases. The through movements from all direction intersect (Cross) each other. Left turns from any approaches are Unique in that they interfere with the traffic from all directions. Thus, in order to facilitate a left turn, a traffic on some directions should yield or stop (Shin 1997).

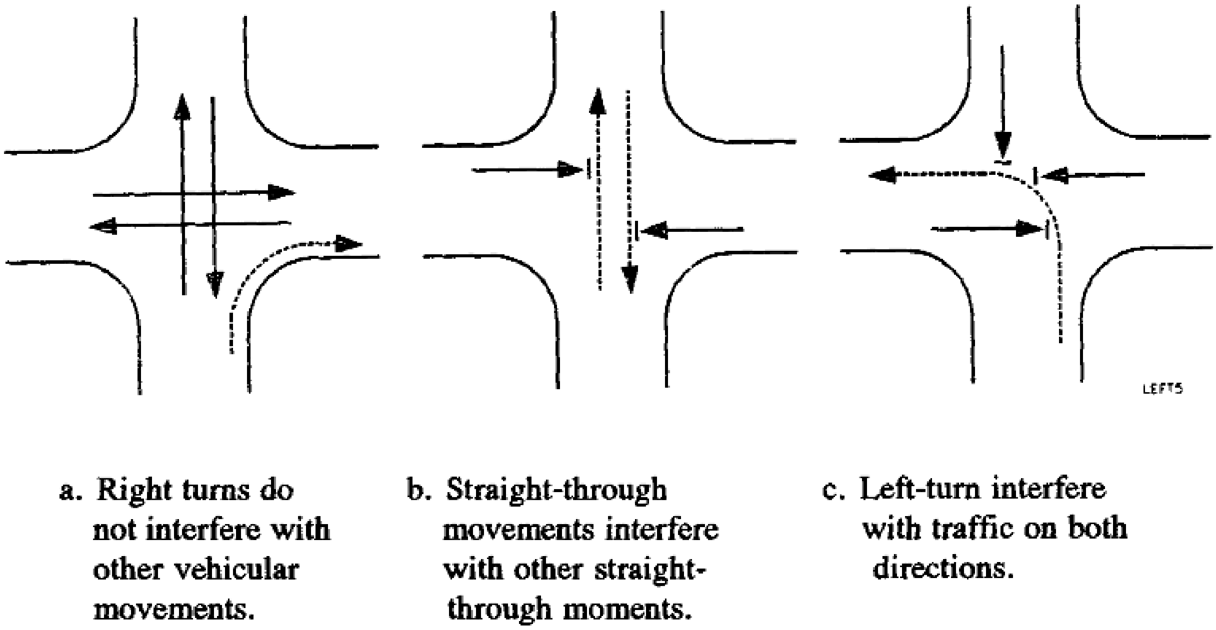

Figure 2-3: Interface among the Directional Movements at an intersection (Shin 1997)

2.4.3.1 Unprotected and Protected Left Turns

Protected/Permitted left-turn phasing is a left-turn movement of traffic at a signalized intersection having a separate left-turn phase in the signal cycle to provide a protected green arrow interval, as well as a non-protected circular green interval. Use of the protected/permitted left-turn phasing technique is based on the assumption that the need for a protected left-turn interval has been established. One of the basic precepts of the protected/ permitted left-turn phasing is that the protected arrow is displayed only when needed in a traffic demand condition. The protected/permitted left-turn phasing technique is an efficient concept opposed to a collision reduction concept; it will probably offer safer operation than permissive operation only (Spokane 2018).

Unprotected (permitted) left-turn phasing occurs when there is no exclusive phase provided for left-turn vehicles. Left-turns are permitted during a circular green when there are sufficient gaps in opposing traffic flow to allow a left-turning vehicle to safely make the turn. Exceptions would be at those locations where turns are restricted by signs or other devices. Separate left-turn lanes may or may not be provided (Spokane 2018). The permitted left turn movement is very complex. It involves the conflict between a left turn and an opposing through movement. The operation is affected by the left turn flow rate and the opposing flow rate, the number of opposing lanes, whether left turns flow from an exclusive left turn lane or from shared lane, and the details the signal timing (Roses 2004).

A protected left turn movement is made without an opposing vehicular flow; the signal plan protects left turning vehicles by stopping the opposing vehicle through movement. This requires that the left turns and the opposing through flow be accommodated in separate Signal phases and leads to multiphase (more than two) signalization (Roses 2004).

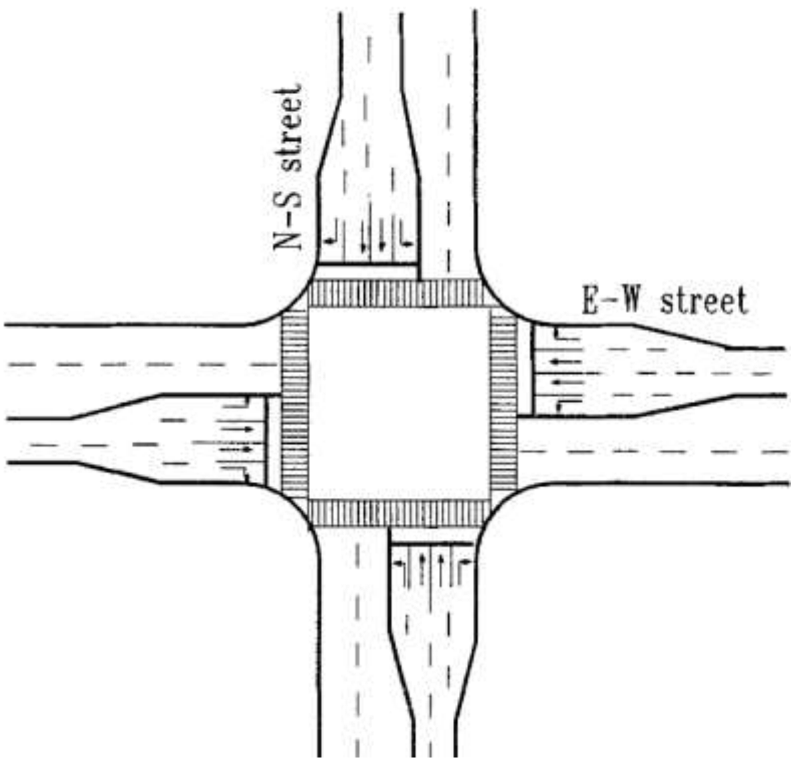

Figure 2-4: Protected Left Turn ((Shin 1997)

2.4.3.2 Left Turn Treatment

During the Signal timing and phase determination, decision on the left turn treatment is one of the things to be considered. There are some criteria to determine whether left turns are treated as permitted or protected. Among the criteria, a cross product method (Roses 2004) and a method of Provided by Traffic Engineering handbook (Pusey 2000) are discussed below.

A) Cross product

The cross-product Criteria and not absolute and they provide a starting point for considering whether or not left turn protection is needed for a particular left turn movement (Roses 2004). According to Cross Product Criteria, the left turn shall be protected if the following satisfied.

- $V_{lt} \geq 200 veh/hr$
- $V_{lt} * ({V_o}/{N_o}) \geq 50{,}000 veh/hr$

Were,

- V_{lt} = left-turn flow rate, veh/hr.
- V_O = opposing through movement, veh/hr.
- N_o = number of lanes for opposing through

B) Traffic Engineering Handbook Method

Based on the research study done by Transportation Research Board (Asante 1993), Traffic Engineering handbook (Pusey 2000) provides criteria for left turn movement treatments (Roses 2004).

Accordingly, permitted left turn phasing should be provided when the following condition exist;

1. Left-turn demand flow within the peak hour, as plotted in figure 2-5 below,
2. The sight distance for left-turning vehicles is not restricted
3. Fewer than eight left-turn accidents have occurred within the last three years at any one approach with permitted-only phasing

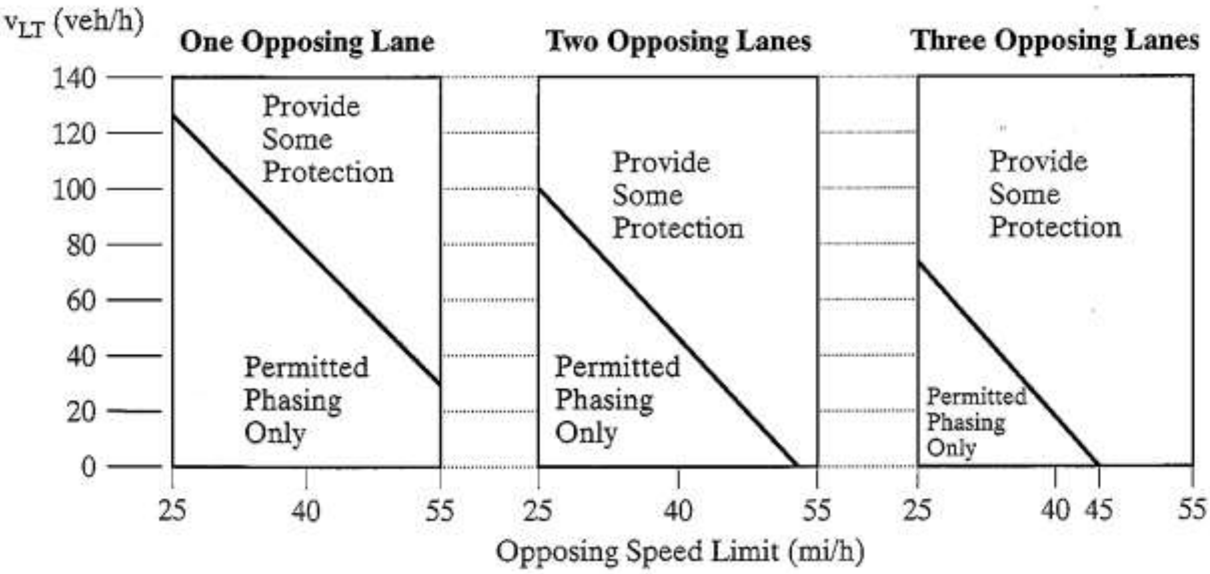

Figure 2-5:Reccommended Selection Criteria for left turn Protection (Roses 2004)

On the other hand, the handbook also states the case when fully left turn will be protected such that fully protected phasing is recommended when any two the following criteria are met (Roses 2004):

1. Left-turn flow rate is > 320 veh/hr.
2. Opposing flow rate is > 1100 veh/hr.
3. Opposing speed limit is ≥ 45 mi/hr.

Additionally, fully protected phasing is also recommended when any one of the following criteria are met (Pusey 2000):

1. There are three opposing traffic lanes, and the opposing speed is ≥ 45 mi/hr.
2. Left-turn flow rate is > 320 veh/hr., and heavy vehicles > 2.5%.
3. The opposing flow rate > 1100 veh/hr., and left turns > 2.5%.
4. Seven or more left turn accidents within three years under compound phasing
5. The average stopped delay to left-turn traffic is acceptable for fully protected phasing, and the engineer decides that additional left turn accidents would occur under the compound phasing option.

During traffic flow analysis at the intersection, left turn movements shall be converted to equivalent through movements using left turn adjustment (see table 2-2 below). For the intermediate opposing flow rate interpolation-based estimation is recommended for Left turn adjustment (E_{LT}) but values should be rounded to the nearest tenth

Table 2-2: Through Vehicle Equivalent for left turning Vehicle, E_{LT} (Roses 2004)

Opposing Flow, V_o (veh/hr.)	**Number of Opposing Lane**			**Left Turn Treatments**
	1	**2**	**3**	
0	1.1	1.1	1.2	Permitted Left Turns
200	2.5	2.0	1.8	
400	5.0	3.0	2.5	
600	10.0	5.0	4.0	
800	13.0	8.0	6.0	
1000	15.0	13.0	10.0	
≥1200	15.0	15.0	15.0	
E_{LT} for all protected left turns = 1.05				

Note: _ Opposing Volume, V_o, includes only the through volume on the opposing approach, in veh/hr.

Similarly, converting the right turn flow to equivalent through vehicles based on the pedestrian's volume in conflicting cross walks as given in table 2-3 below (Roses 2004).

Table 2-3: Through Vehicle equivalent for Right-Turning Vehicle (E_{RT}) (Roses 2004)

Pedestrians Volume in conflicting Crosswalks (Peds/hr.)	**Equivalent (E_{RT})**
None (0)	1.18
Low (50)	1.21
Moderate (200)	1.32
High (400)	1.52
Extreme (800)	2.14

Note:

- For right turns the "conflicting crosswalk" is the crosswalk through which right-turning vehicles must pass,
- Pedestrian volumes in the table 2-3 for ERT represent typical situations in moderate-sized communities.

- Interpolation for ERT is not recommended.

Once ELT and ERT are determined, all right- and left-turn volumes must be converted to units of "through-vehicle equivalents." Subsequently, the demand intensity per lane is found for each approach or lane group (Roses 2004).

$$V_{LTE} = V_{LT} * E_{LT} \text{ and } V_{RTE} = V_{RT} * E_{ET}$$

Where:

V_{LTE} = left-turn volume in through-vehicle equivalents, tvu/hr

V_{RTE} = Right-turn volume in through-vehicle equivalents, tvu/hr

The total equivalent volume and equivalent volume per lane in each approach or lane group is determined as (Roses 2004):

$$V_{EQ} = V_{LTE} + V_{TH} + V_{RTE} \quad \text{and} \quad V_{EQL} = {}^{V_{EQ}}/_{N}$$

Where:

V_{EQ} = total volume in a lane group or approach, tvu/hr.

V_{EQL} = total volume per lane in a lane group or approach, tvu/hr./ln

N = number of lanes

2.4.4 Traffic Congestion

Traffic congestion has become a major issue with the metropolis facing the most. Due to urbanization and the high use of roadway vehicles the problem of traffic congestion is increasing day by day. Some private are companies have an important role in being congestion in large cities because they provide easy services of two and four-wheeler vehicles (Manoj Kumar 2021).

The congestion results from the imbalance between the supply of and the demand for transportation facilities (Falcocchio,John C. and Herert S.Levinson 2015).

To tackle the problem of traffic congestion, various measures have been developed for the identification and quantification of traffic congestion by various researchers. These measures can be helpful for finding the degree of traffic congestion and the performance of the roadway which can be categorizing these measures into three parts: Travel time based, speed-based, and level of service based (Manoj Kumar 2021).

According to FHA Final report (FHWA 2022), there are seven sources of Congestion which can be categorized into three main categories as elaborated below.

2.4.4.1 Category 1 — Traffic-Influencing Events

A. **Traffic Incidents** – Are events that disrupt the normal flow of traffic, usually by physical impedance in the travel lanes. Events such as vehicular crashes, breakdowns, and debris in travel lanes are the most common form of incidents. In addition to blocking travel lanes physically, events that occur on the shoulder or roadside can also influence traffic flow by distracting drivers, leading to changes in driver behavior and ultimately degrading the quality of traffic flow. Even incidents off of the roadway (a fire in a building next to a highway) can be considered traffic incidents if they affect travel in the travel lanes.

B. **Work Zones** – Are construction activities on the roadway that result in physical changes to the highway environment. These changes may include a reduction in the number or width of travel lanes, lane "shifts," lane diversions, reduction, or elimination of shoulders, and even temporary roadway closures. Delays caused by work zones have been cited by travelers as one of the most frustrating conditions they encounter on trips.

C. **Weather** – Environmental conditions can lead to changes in driver behavior that affect traffic flow. Due to reduced visibility, drivers will usually lower their speeds and increase their headways when precipitation, bright sunlight on the horizon, fog, or smoke are present.

2.4.4.2 Category 2 — Traffic Demand

1. **Fluctuations in Normal Traffic** – Day-to-day variability in demand leads to some days with higher traffic volumes than others. Varying demand volumes superimposed on a system with fixed capacity also results in variable (i.e., unreliable) travel times, even without any Category 1 events occurring.
2. **Special Events** – Are a special case of demand fluctuations where traffic flow in the vicinity of the event will be radically different from "typical" patterns. Special events occasionally cause "surges" in traffic demand that overwhelm the system.

2.4.4.3 Category 3 — Physical Highway Features

Traffic Control Devices – Intermittent disruption of traffic flow by control devices such as railroad grade crossings and poorly timed signals also contribute to congestion and travel time variability.

1. **Physical Bottlenecks ("Capacity")** – Transportation engineers have long studied and addressed the physical ***capacity*** of roadways; the maximum amount of traffic capable of being handled by a given highway section.

2.5 Intersection Control

Intersection being the place where traffic shares the same space at different times needs efficient design and consideration for vehicles from different approaches. The only solution at the intersection is ordering the vehicles at different times to occupy the same space. If any vehicles occupy the same space at the same time, there will be a conflict which is not desirable.

Several methods of controlling conflicting streams of vehicles at intersections are in use. The choice of one of these methods depends on the type of intersection and the volume of traffic in each of the conflicting streams. Guidelines for determining whether a particular control type is suitable for a given intersection have been developed and should be at least meet the MUTCD requirement for applying any of the option of controlling methods (Roses 2004).

According to MUTCD, there are three basic levels of intersection control. The first is the application of basic Rules of the Road where right-of-way is not explicitly assigned through the use of traffic signals, STOP, or YIELD signs. In essence, the responsibility for avoiding a potential conflict is assigned to the vehicle on the left. It’s often specified that through vehicles have the right-of-way over turning vehicles at uncontrolled intersections (Roses 2004).

AASHTO suggests that, to ensure safe operation with no control, both drivers should be able to stop before reaching the collision point when they first see each other, otherwise higher level of intersection control, Yield, stop or Signals, should be used based on the (MUTCD 2003) recommendations.

2.5.1 Control by GIVE WAY and STOP Signs

GIVE WAY signs and STOP signs are used to control traffic at intersections other than those controlled by means of roundabouts or traffic signals, by allocating priority to traffic on one of the intersecting roads (MUTCD 2003). MUTCD recommends GIVE WAY and STOP signs, generally based on the inadequate of available sight distances of the approaching roads to the junction or intersection.

All drivers on approaches with yield signs are required to slow down and yield the right of way to all conflicting vehicles at the intersection. Stopping at yield signs is not mandatory, but drivers are required to stop when necessary to avoid interfering with a traffic stream that has the right of way. Yield signs are therefore usually placed on minor-road approaches, where it is necessary to yield the right of way to the major road traffic. The most significant factor in the warrant for yield signs is the approach speed on the minor road (Nicholas J. Garber and Lester A. Hoel 2009). It is often required that drivers on YIELD-controlled approaches slow to 8-10 mi/hr before entering the major street (Roses 2004).

A STOP sign shall be used when, for minor road traffic, the sight distance, Y, in either direction along the major or uncontrolled road as shown in Figure 2.6 is less than the distance given for the corresponding major road speed in table 2-4. The STOP sign shall be used in the same way and for the same purpose as specified for the GIVE WAY sign but with the additional requirement that a complete stop is necessary for safety before entering the intersection.

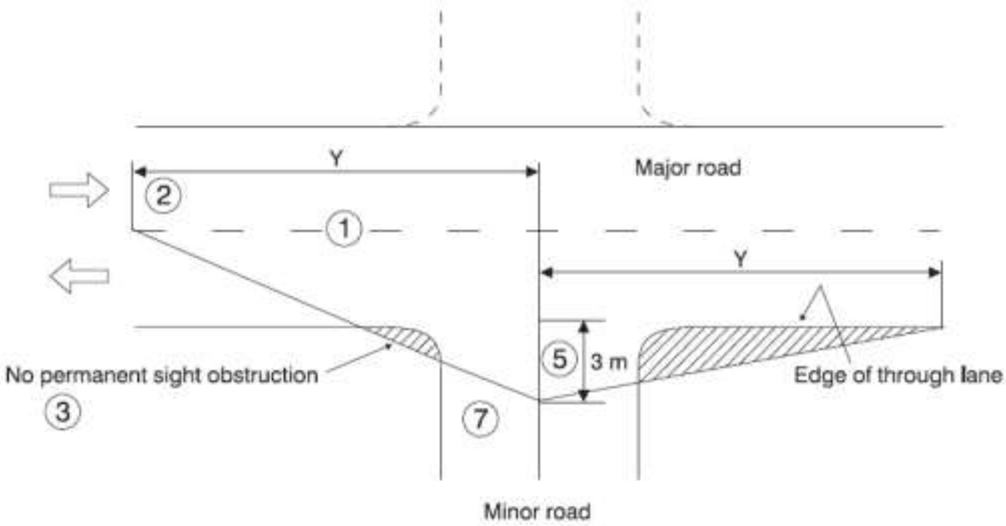

Figure 2-6: Sight distance restrictions requiring use of stop signs (MUTCD 2003)

Table 2-4: Sight distance restrictions requiring use of stop signs (MUTCD 2003)

Major Road Speed (Km/h), Note [1]	Distance along major road, y(m), Note [2]
40	20
50	30
60	40
70	55
80	65
90	80
100	95
110	115
120	140

As indicated in (MUTCD 2003), the intention of controlling intersection by GIVE WAY and Stop Signs more emphasized on the safety of the movement at the intersection by reducing the probability of conflicts that would happen due to scarcity of Sight distances.

2.5.2 Control by Traffic Signal Sign

One of the most effective ways of controlling traffic at an intersection is the use of traffic signals. Traffic signals can be used to eliminate many conflicts because different traffic streams can be assigned the use of the intersection at different times. Since this results in a delay to vehicles in all streams, it is important that traffic signals be used only when necessary (Nicholas J. Garber and Lester A. Hoel 2009).

2.5.2.1 Signal Warranty

MUTCD (2003) specifies eight different warrants that justify the installation of a traffic signal along with comprehensive engineering study to be conducted to determine whether or not installation of signal is justified. These warranties are:

A. Warrant 1, Eight-hour vehicular volume

B. Warrant 2, Four-hour vehicular volume

C. Warrant 3, Peak hour

D. Warrant 4, Pedestrian volume

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[1] The posted or general speed limit is used, unless the 85th percentile speed is significantly higher.

[2] When checking sight distance, the height of the observer’s eye is 1.1 m and the height of the object is 0.0 m.

E. Warrant 5, School crossing

F. Warrant 6, Coordinated signal system

G. Warrant 7, Crash experience

H. Warrant 8, Roadway network

Satisfaction of a traffic signal warrant or warrants shall not in itself require the installation of a traffic signal un less professional judgment based on experience and Traffic signal shall not be implemented if none of the warrants are met. At least one of the Warrants must be satisfied to implement the signals.

The reader can get the description and preconditions of these warrants from Traffic Engineering books ((Nicholas J. Garber and Lester A. Hoel 2009), (Roses 2004), MUTCD and others.

If traffic signal is not properly design, it will lead to extra delays, excessive disobedience, increased use of less adequate route as road users, Significant increases in the frequency of collisions. attempt to avoid traffic control signal (Roses 2004).

2.5.3 Signal Timing

Traffic Signal warrants will help traffic engineer only in deciding whether a traffic signal should be used at an intersection. The efficient operation of the signal also requires proper timing of the different color indications, which is obtained by implementing the necessary design.

The common terms to be define for the determination of signal timing are, Head ways, Saturation flows and Signal Cycle with its components as each of the terms discussed in next sub-sections.

2.5.4 Time Head Way and Discharge headways

I. Time Head way

Time headway (h or Ht) is the difference between the time the front of a vehicle arrives at a point on the highway and the time the front of the next vehicle arrives at that same point. (Nicholas J. Garber and Lester A. Hoel 2009). It is the time interval between the fronts (or rears) of two successive vehicles measured at a fixed point along the road (SC van As & HS Joubert 2002). Time headway is usually expressed in seconds (Nicholas J. Garber and Lester A. Hoel 2009).

The time head way between vehicles is an important flow characteristic that affect the safety, Level of Service, Driver Behavior and Capacity of Transportation System. A minimum head time headway must be present to provide safety in the event that the lead vehicles decelerate. The capacity of the system is governed primarily by the minimum time headway and the time headway distribution under capacity flow conditions (May 1990). According to Adolf D. May (1990) time headway and its distribution are equally important in other than highway traffic system such as Pedestrian, Rail, Water and Air transportation.

II. Discharge Headways

In a signalized intersection, When the light turns GREEN, there is a queue of vehicles waiting to be discharged, that were stopped during the preceding RED phase. As the queue moves, headway measurements are taken as follows (Roses 2004) :

- First headway is the time elapse between the start of GREEN signal and the time that the front wheel of first vehicle crossing the stop line,
- Second headway is the time laps between that the 1st and 2nd vehicles' front wheels crossing the stop line,
- Subsequent headways measured similarly,
- Only headways through the last vehicle in queue (at the start of GREEN light) are considered to be operating under saturated conditions.

Accordingly, if many queues of vehicles are observed at a given location and the average headway is plotted VS the queue position of the vehicle, a trend similar to that in figure 2-7 below.

As shown in figure 2-6 below, the first headway is relatively long, the second headway is shorter. Each successive headway is little bit smaller and finally the headway tends to level out (Roses 2004). This generally occurs when queued vehicles have fully accelerated by the time; they cross the stop line. At this point, a stable moving queue has been stablished. The constant headway achieved is referred to as Saturation headway, as it is the average headway that can be achieved by a saturated, Stable moving queue of vehicles passing through the signal. It is given the symbol ‘h’ in units of second/Vehicles.

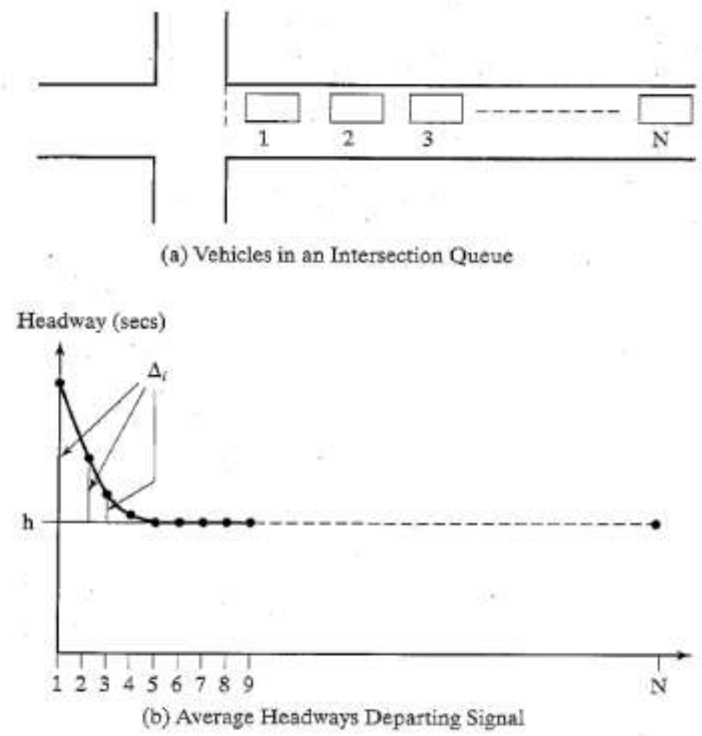


Figure 2-7: Flow From a queue at signalized Intersection (Roses 2004)

2.5.5 Saturation Flow Rate

According to (HCM 2010) definition, saturation flow rate represents the maximum rate of flow for a traffic lane, as measured at the stop line during the green indication. The base saturation flow rate represents the saturation flow rate for a traffic lane that is 12ft wide and has no heavy vehicles, a flat grade, no parking, no buses that stop at the intersection, even lane utilization, and no turning vehicles. Typically, one base rate is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. It has units of passenger cars per hour per lane (pc/h/ln). usually, the base saturation is taken to be 1900 passenger cars/h of green time per lane (Nicholas J. Garber and Lester A. Hoel 2009).

According to Roses (2004) the default value for saturation flow rate, s, is 1615 tvu/hg for typical conditions of lane width, heavy-vehicle presence, grades, parking, pedestrian volumes, local buses, area type, and lane utilization and can be range from 1500tvu/hg to 1700tvu/hg.

HCM 2010 Provides the way of adjusting for this ideal saturation flow for the prevailing conditions to obtain the saturation flow for the lane group being considered.

$$s = (s_o)(N)(f_w)(f_{HV})(f_g)(f_p)(f_a)(f_{bb})(f_{Lu})(f_{RT})(f_{LT})(f_{Lpb})(f_{Rpb})$$

Equation 2.1

Were,

- *s* _ saturation flow rate for the subject lane group, expressed as a total for all lanes in lane group under prevailing conditions (veh/h/g)
- s_o _ ideal saturation flow rate per lane, usually taken as 1900 (veh/h/ln)
- *N* _ number of lanes in lane group
- *fw* _ adjustment factor for lane width
- *fHV* _ adjustment factor for heavy vehicles in the traffic stream
- *fg* _ adjustment factor for approach grade
- *fp* _ adjustment factor for the existence of parking lane adjacent to the lane group and the parking activity on that lane
- *fa* _ adjustment factor for area type (for CBD, 0.90; for all other areas, 1.00)
- *fbb* _ adjustment factor for the blocking effect of local buses stopping within the intersection area
- *fLu* _ adjustment factor for lane utilization
- *fRT* _ adjustment factor for right turns in the lane group
- *fLT* _ adjustment factor for left turns in the lane group
- *fLpb* _ pedestrian adjustment factor for left-turn movements
- *fRpb* _ pedestrian adjustment factor for right-turn movements

(Nicholas J. Garber and Lester A. Hoel 2009) has summarized each of the above factors as presented in table 2-5 below.

Table 2-5: Adjustment Factors for Saturation Flow Rates (Nicholas J. Garber and Lester A. Hoel 2009)

Factor	Formula	Definition of Variables	Notes
Lane width	$f_w = 1 + \frac{(W - 12)}{30}$	W = lane width (ft)	$W \geq 8.0$ If $W > 16$, two-lane analysis may be considered
Heavy vehicles	$f_{HV} = \frac{100}{100 + \% HV(E_T - 1)}$	$\% HV$ = percent heavy vehicles for lane group volume	$E_T = 2.0$ pc/HV
Grade	$f_g = 1 - \frac{\% G}{200}$	$\% G$ = percent grade on a lane group approach	$-6 \leq \% G \leq +10$ Negative is downhill
Parking	$f_p = \frac{N - 0.1 - \frac{18N_m}{3600}}{N}$	N = number of lanes in lane group N_m = number of parking maneuvers/h CBD=Central Business District	$0 \leq N_m \leq 180$ $f_p = \geq 0.050$ $f_p = 1.000$ for no parking
Bus blockage	$f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N}$	N = number of lanes in lane group N_B = number of buses stopping/h	$0 \leq N_B \leq 250$ $f_{bb} = \geq 0.050$
Type of area	$f_a = 0.900$ in CBD $f_a = 1000$ in all other areas		
Lane utilization	$f_{LU} = v_g/(v_{g1}N)$	v_g = unadjusted demand flow rate for the lane group, veh/h v_{g1} = unadjusted demand flow rate on the single lane in the lane group with the highest volume N = number of lanes in the lane group	
Left turns	Protected phasing: Exclusive lane: $F_{LT} = 0.95$ Shared lane: $f_{LT} = \frac{1}{1.0 + 0.05P_{LT}}$	P_{LT} = proportion of LTs in lane group	See pages 474 through 483 for non-protected phasing alternatives
Right turns	Exclusive lane: $f_{RT} = 0.85$ Shared lane: $f_{RT} = 1.0 - (0.15)P_{RT}$ Single lane: $f_{RT} = 1.0 - (0.135)P_{RT}$	P_{RT} = proportion of RTs in lane group	
Pedestrian-bicycle blockage	LT adjustment: $f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$ RT adjustment: $f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$	P_{LT} = proportion of LTs in lane group A_{pbT} = permitted phase adjustment P_{LTA} = proportion of LT protected green over total RT green P_{RT} = proportion of RTs in lane group P_{RTA} = proportion of RT protected green over total RT green	See pages 485 to 490 for step-by-step procedure

2.5.6 Signal Cycle

The main objectives of signal timing at an intersection are to reduce the average delay of all vehicles and the probability of crashes. These objectives are achieved by minimizing the possible conflict points when assigning the right of way to different traffic streams at different times (Nicholas J. Garber and Lester A. Hoel 2009). The signal consists different time interval which consists of Green, Yellow and all red intervals which adding all together gives a full of one cycle consists of different phases.

2.5.6.1 Yellow Interval and Total Lost

A) Yellow Interval

The main purpose of the yellow indication after the green is to alert motorists to the fact that the green light is about to change to red and to allow vehicles already in the intersection to cross it. The required yellow interval is the time period that guarantees that an approaching vehicle can either stop safely or proceed through the intersection without speeding (Nicholas J. Garber and Lester A. Hoel 2009).

The Institute of Transportation Engineers, ITE, recommends the following for yellow interval as in equation 2.2 (Ewing 1999), (Roses 2004).

$$\boldsymbol{y = t + \frac{1.47S_{85}}{2(a+Gg)}}$$ **Equation 2.2**

Were,

- y = length of the yellow interval, s
- t = driver reaction time, s (commonly 1s)
- S_{85} = 85th percentile speed of approaching vehicles, or speed limit, as appropriate, mi/hr
- a = deceleration rate of vehicles, ft/s2 (commonly 10 ft/s2)
- G = grade of approach, %
- g = acceleration rate due to gravity (32.2 ft/s2)

Similarly, the ITE general formula have been modified for the minimum yellow interval time including the dilemma zone given in equation 2.3 below (Nicholas J. Garber and Lester A. Hoel 2009).

$$\boldsymbol{\tau_{min} = \delta + \frac{W+L}{u_o} + \frac{u_o}{2(a+Gg)}} \qquad \textbf{Equation 2.3}$$

Were,

- $\delta =$ perception-reaction time (sec)
- u_o = speed limit during the yellow interval (ft/sec)
- W = Width of intersection (ft)
- L = Length of vehicle (ft)
- *a*= constant rate of braking deceleration (usually 10 ft/sec2) and other are as defined previously

B) Clearance Lost Time and Start Up lost

i. Clearance (all red) intervals

For a vehicle that just entered the intersection legally on yellow, the all-red must provide sufficient time for the vehicle to cross the intersection and clear its back bumper past the far curb line (or x-walk) before conflicting vehicles are given the GREEN.

The ITE (Ewing 1999) recommends the following for all red intervals:

$$ar = \begin{cases} \frac{W+L}{1.47S_{15}} \text{\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_} (No\ pedestrian\ Traffic) \\ \frac{P+L}{1.47S_{15}} \text{\_\_\_\_\_\_\_\_\_\_\_\_} (Significant\ Pedestrain\ Traffic) \\ \max\left[\left(\frac{W+L}{1.47S_{15}}\right), \left(\frac{W+L}{1.47S_{15}}\right)\right] \text{\_} Some\ Pedestrian\ Traffic \end{cases} \qquad \text{Equation 2.4}$$

Where:

- *ar* = length of the all-red interval, s
- w = distance from the departure STOP line to the far side of the farthest conflicting traffic lane, ft
- P = distance from the departure STOP line to the far side of the farthest conflicting crosswalk, ft
- L = length of a standard vehicle, usually taken to be 18-20 ft
- S_{15} = 15^{th} percentile speed of approaching vehicles, or speed limit, as appropriate, mi/hr

ii. All RED intervals

The average head way described in section 2.5.4 above is actually greater than normal head way ‘h’. the first; several headways are larger than ‘h’ and those are at the first three or four headways are involved additional time (Δi) as drivers react to the GREEN signal and acceleration (Roses 2004). These additional times are added and referred to as the startup lost time (l_1). HCM (2010) recommends the value of Startup lost time 2 seconds per phase.

$$\boldsymbol{l_1 = \sum_i \Delta_i} \qquad \textbf{Equation 2.5}$$

Where;

- l_1 = start-up lost time, s/phase
- Δi = incremental headway (above "h" seconds) for vehicle i, s

The additional lost time associated with stopping of the queue at the end the GREEN signal is called Clearance lost (l_2) and is usually difficult to measure in field. In such a situation, it is defined as the time interval between the last vehicle's front wheels crossing the stop line, and the initiation of the GREEN for the next phase. The clearance lost time occurs each time a flow of vehicles is stopped (Roses 2004).

The clearance lost time values vary with the Yellow and all-red timings of the signal

$$\boldsymbol{l_2 - y + ar - e} \qquad \textbf{Equation 2.6}$$

Were,

- y=length of yellow interval, seconds,
- *ar* =length of all red intervals, Seconds
- e=encroachment of vehicles into yellow and all red, seconds, 2.0seconds

Total lost time is the summation of startup and clearance lost time of each phase.

$$\boldsymbol{L = l_1 + l_2} \qquad \textbf{Equation 2.7}$$

2.5.6.2 Cycle length and green time

Critical lane volume is one of the factors that govern the design of cycle length (Roses 2004). Hence, before determining the cycle length determining number of phases and the critical traffic volume in each predetermined phase shall be known. The cortical Volume is affected by the number of Left and Right turnings where Right turnings are particularly affected by the number of crossing pedestrians. Roses (2004) provides equivalent factors for Left turning and right turning traffics as discussed in section 2.4.3.

In addition to the critical lane group volume, Saturation Flows is very important parameters to determine of cycle length.

I general, the cycle length can be determined by the following equation (Roses 2004).

$$C_{des} = \frac{L}{1 - \left(\frac{V_c}{S * PHF * (v/c)}\right)}$$ **Equation 2.8**

Wh
ere,

- C_{des} = desirable cycle length, s
- s = saturation flow rate (tvu/hg)
- L = total lost time per cycle, s/cycle
- PHF = peak-hour factor
- v/c = target v/c ratio for critical movements in the intersection (Generally, 0.85 ‐ 0.95)

2.6 Performance Measures

An intersection's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road user group. HCM 10 recommends the Performance measures for Automobile (Vehicles) of the following parameters. Volume-to-capacity ratio, Average delay, queue storage ratio and Level of Service (LOS). It is useful for describing intersection performance to elected officials, policy makers, administrators, and the public. Speed, Travel Time and Delay are all related measures that are commonly used as indicator of performance for traffic facilities (Roses 2004).

Moreover, HCM 10 more emphasis LOS criteria for the performance measure of the intersection for Automobile modes. Accordingly, LOS can be characterized for the entire intersection, each intersection approach, and each lane group. Control delay alone is used to characterize LOS for the entire intersection or an approach. Control delay and volume-to-capacity ratio are used to characterize LOS for a lane group. Delay quantifies the increase in travel time due to traffic signal control. It is also a surrogate measure of driver discomfort and fuel consumption. The volume-to-capacity ratio quantifies the degree to which a phase's capacity is utilized by a lane group (HCM 10).

2.6.1 Traffic delay at Traffic Signal

Delay and Queening are one of the parameters for determining the Performance of intersection and operational quality of signalized intersections (Roses 2004).

Delay can be defined as amount of time consumed in traversing the intersection (difference between arrival time and departure time) whereas Queuing is the number of vehicles forced to queue behind the stop-line during a red signal phase (Roses 2004).

The results obtained from the volume adjustment, saturation flow rate, and capacity analysis modules are used to determine the average control time delay per vehicle in each lane group and hence the level of service for each approach and the intersection as a whole (Nicholas J. Garber and Lester A. Hoel 2009).

According to (Roses 2004) delay can be categorized into different parts as shown in figure below.

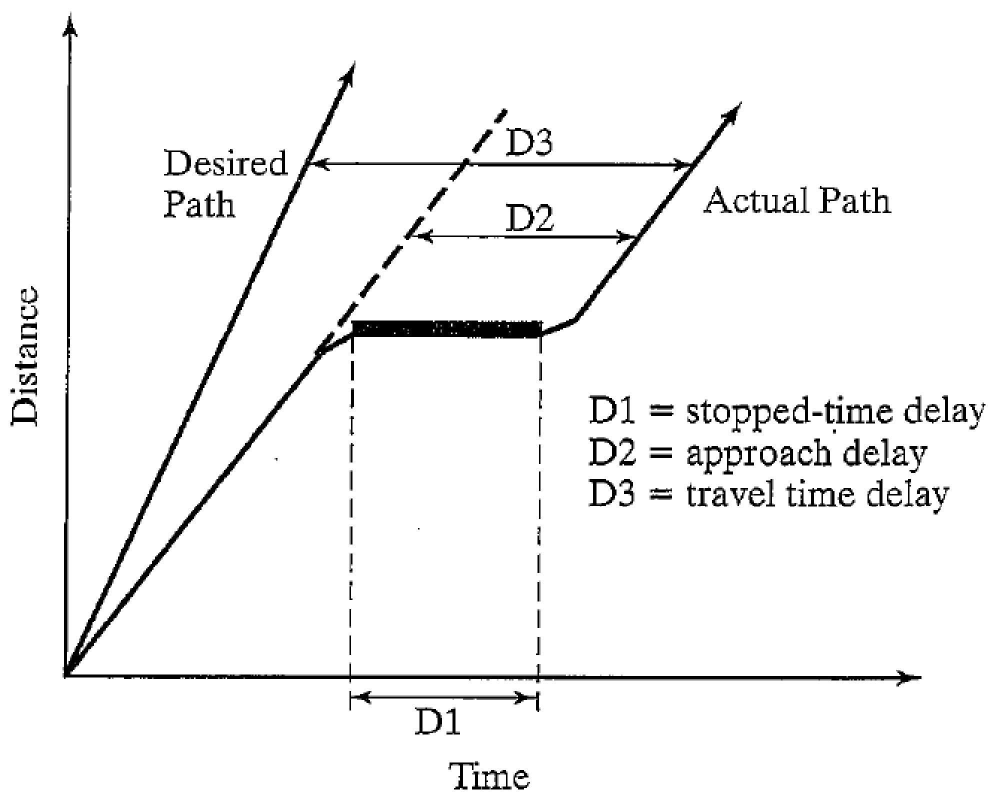


Figure 2-8: Illustration of delay measures (Roses 2004)

Stopped time delay includes only the time spent stopped at the signal. It begins when the vehicles are fully stopped and ends when the vehicles begin to accelerate. Approach delay includes additional delay loses due to deceleration and acceleration found by extending the velocity slope of the approaching vehicle if no signal existed. Travel time delay is the difference in time between a hypothetical desired velocity line and the actual vehicle path. Time in queue delay cannot be effectively shown using one vehicle, as it involves joining and departing a queue of several vehicles (Roses 2004).

2.6.1.1 Webster's Uniform Delay Model

Webster's Uniform delay Model was developed based on the assumption of stable and Simply uniform arrival function. Webster's model for uniform delay is the area of the triangle formed by the arrival and departure functions as illustrated in figure 2-9 below.

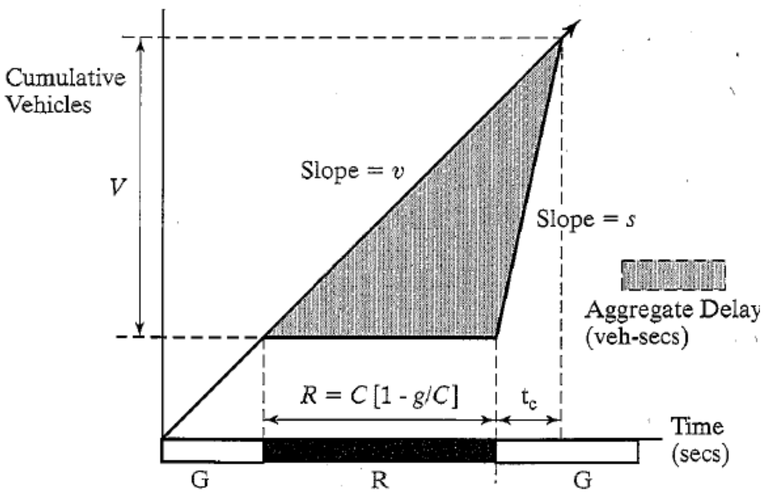


Figure 2-9: Webster's Uniform Delay Model illustration (Roses 2004)

According to Webster's model, aggregate delay is on-half of the base (Length of red phase) times the height (Total vehicles in queue).

$$UD_a = \frac{1}{2}RV \qquad \textbf{Equation 2.9}$$

Where,

- UDa=Aggerate Uniform delay (Vehicles-second)
- R=Length of the red phase (Seconds)
- V=total vehicles in queue (Vehicles)

Since the Original delay model concept was developed in terms of Red phase, it is rational to use

cycle length and green phases. Hence, webster substitutes Red with corresponding cycle and green phases factor as presented in equation 2 below.

$$R = C(1 - \frac{g}{c}) \qquad \textbf{Equation 2.10}$$

Where,

- C=Cycle Length, second

- g=effective green time

Webster also define the height of the triangle, V, as total vehicles in queue which includes vehicles arriving during RED, R, plus those that join the end of the queue while it is moving out of the intersection during time tc which leads to determine tc by setting number of vehicles arriving during the R + tc equal to the number of vehicles departing during the period t.

$$\boldsymbol{V}(\boldsymbol{R}+\boldsymbol{t_c}) = \boldsymbol{S}\boldsymbol{t}_c$$

$$\boldsymbol{R} + \boldsymbol{t}_c = \frac{\boldsymbol{S}}{\boldsymbol{v}}\boldsymbol{t}_c$$

$$R = \boldsymbol{t}_c\left[\frac{\boldsymbol{S}}{\boldsymbol{v}} - \boldsymbol{1}\right]$$

$$\boldsymbol{t}_c = \frac{R}{\left[\frac{S}{v} - \boldsymbol{1}\right]}$$

Substituting for t_c in V, and $V = R\frac{vS}{[s-v]}$ and $R = C(1-\frac{g}{c})$, then V= $\frac{vS}{[s-v]} * C(1-\frac{g}{c})$

$$\boldsymbol{UD_a} = \frac{1}{2}\boldsymbol{RV} = \frac{1}{2}\boldsymbol{C^2}\left(\boldsymbol{1}-\frac{\boldsymbol{g}}{\boldsymbol{c}}\right)^{\boldsymbol{2}}\frac{\boldsymbol{vS}}{[\boldsymbol{s}-\boldsymbol{v}]}$$ **Equation 2.11**

Average uniform delay per vehicle (dividing UDa by the number of vehicles arriving during the cycle, vC:

$$\boldsymbol{UD} = \frac{1}{2}\boldsymbol{RV} = \frac{1}{2}\boldsymbol{C}\frac{\left(\boldsymbol{1}-\frac{\boldsymbol{g}}{\boldsymbol{C}}\right)^{\boldsymbol{2}}}{\boldsymbol{1}-\frac{\boldsymbol{v}}{\boldsymbol{s}}}$$ **Equation 2.12**

Where;

UD = average uniform delay per vehicles, s/veh

C = cycle length, second

g = effective green time, second

v = arrival flow rate, veh/hr

s=saturation flow, Veh/hr

Alternatively, the capacity, c, can be used rather than the saturation flow rate, s, s = c/(g/C)

$$UD = \frac{1}{2}C\frac{\left[1-\frac{g}{C}\right]^2}{\left[1-\left(\frac{g}{C}\right)\left(\frac{v}{c}\right)\right]} = \frac{0.50C\left[1-\frac{g}{C}\right]^2}{1-\left(\frac{g}{C}\right)X}$$

Equation 2.13

Where X = v/c ratio, or degree of saturation and the maximum value of X is 1.0 as the uniform flow model assumes no overflow.

For the case of over flow, the value of X set to be equal with 1 (X=1) (Roses 2004). The total delay will be the sum of Overflow delay (OD) and Uniform delay (UD)which will be obtain by setting X=1.

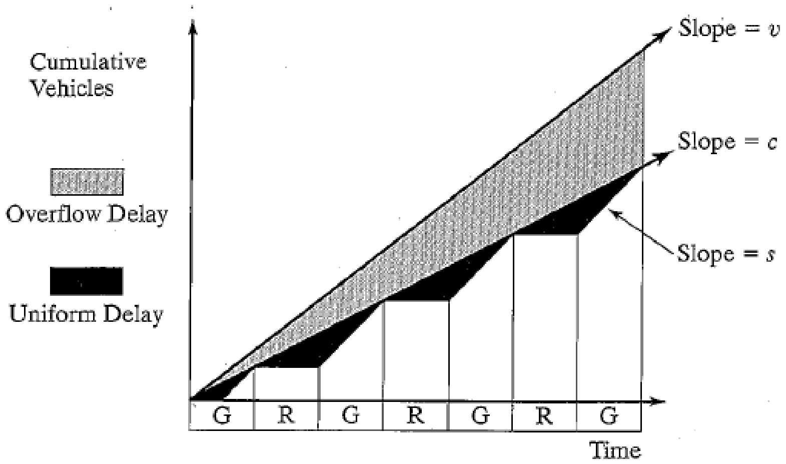


Figure 2-10: an oversaturated Period Illustrated (Roses 2004)

Figure 2-11 below illustrate how the overflow is estimated.

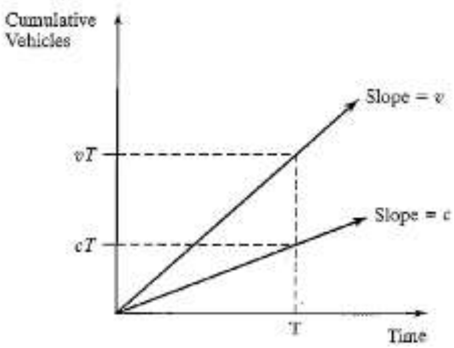


Figure 2-11: Derivation of the Overflow delay formula (Roses 2004)

$$\boldsymbol{OD_a = \frac{1}{2}T(vT - cT) = \frac{T^2}{2}(v - c) \quad and \quad OD = \frac{OD_a}{cT} = \frac{T}{2}\left(\frac{v}{c} - 1\right) = \frac{T}{2}(x - 1)} \quad \textbf{Equation 2.14}$$

Where,

- $v = \frac{approaching}{arrival}\ flow\ rate(\frac{V}{h})$
- $c = Capacity\ flow\ rates\ \left(\frac{V}{h}\right)$
- $x = \frac{v}{c}$ =volume to capacity ratio)
- $ODa = Aggregate\ overflow\ delay, veh - secs$
- OD =Average overflow delay per vehicle, s/veh
- T = the analysis period, hr

However, the overflow model, with overflow delay of zero seconds per vehicle at X = 1.0 is also unrealistic and some studies shows that Webster models is practically acceptable for some ranges of X (Roses 2004).

- Uniform delay model: v/c ≤ 0.85
- Simple overflow delay model: v/c ≥ 1.15

Accordingly, the webster delay model will be as summarized in equation **2.7 below.**

$$\boldsymbol{Delay}\left(\frac{s}{veh}\right) = \begin{cases} \boldsymbol{UD} = \frac{0.5C\left(1-{g}/{c}\right)^2}{1-\left({g}/{C}\right)X}, \boldsymbol{X \leq 0.85} \\ \boldsymbol{UD + OD = \frac{1}{2}C(1 - {g}/{C}) + \frac{T}{2}(x-1), X \geq 1.15} \end{cases}$$ **Equation 2.15**

2.6.1.2 Akcelik Delay Model

As Webster Delay model is practically acceptable for $X \leq .85$ and X ≥ 1.15, To address this inconsistency problem Akcelik proposed a delay model and is used in the Australian Road Research Board’s signalized intersection. In his delay model, overflow component is given by (Roses 2004),

$$\boldsymbol{\frac{cT}{4}\left[(1-X) + \sqrt{(1-X)^2 + \frac{12(X-X_O)}{cT}}\right]}$$ **Equation 2.16**

In which$X_o = 0.67 + \frac{sg}{600}$, $X \geq X_0$, and if $X \leq X_0$ then overflow delay is zero, and

where,

- T is the analysis period, h,
- X is the v/c ratio,
- c is the capacity, veh/hour,
- s is the saturation flow rate, veh/sg (vehicles per second of green) and
- g is the effective green time, sec

then, the total model will be the sum of Webster Uniform model and Over flow model of Akcelik for 0.85 < X < 1.15.

$$\boldsymbol{Delay}\left(\frac{s}{veh}\right) = \boldsymbol{\frac{1}{2}C(1 - {g}/{C}) + \frac{cT}{4}\left[(1-X) + \sqrt{(1-X)^2 + \frac{12(X-X_O)}{CT}}\right]}$$ **Equation 2.17**

2.6.1.3 HCM Delay Model

Highway Capacity Manual ((HCM 2010) has provided the equation for Delay models consisting of Uniform delay model (d_1), Incremental (over flow) delay model (d_2) and Initial delay model (d_3). An Initial Delay model represent a term covering delay from an existing or residual queue at the beginning of the analysis period.

$$\boldsymbol{d = d_1 PF + d_2 + d_3} \qquad \textbf{Equation 2.18}$$

$$PF = \left(\frac{(1-P)}{1-(g/_{C})}\right) X\, f_p$$

Where

- d = control delay, s/veh
- d1 = uniform delay component, s/veh
- PF = progression adjustment factor that accounts for the effect of platooned arrival patterns
- d2 = overflow delay component, s/veh
- d3 = delay due to pre-existing queue, s/veh
- $PF = \left(\frac{(1-P)}{1-(g/_{C})}\right) * \ f_p$
- P = proportion of vehicles arriving during the green interval,
- f_p= supplemental adjustment factor for platoon arriving during the green

A) **HCM Uniform Model (d1)**

In the Webster uniform delay model, it is assumed that the flow is stable and Simply uniform arrival function. The HCM delay methodology removes these assumptions to allow more accurate estimation of uniform delay through accounting for (1) traffic movements in platoons, (2) movements with multiple green periods, and (3) movements with multiple saturation-flow rates (e.g., protected–permitted left-turns), by implementing an incremental queue accumulation procedure (Anurag 2016).

$$d_1 = \left(\frac{C}{2}\right) * \left\{\frac{\left(1-g/_{C}\right)^2}{1-\left[\min(1,X)*g/_{C}\right]}\right\} \qquad \text{Equation 2.19}$$

Where,

- X = v/c ratio
- C = cycle length, s
- g = effective green time, seconds
- c = capacity, veh/h, $c = S * \frac{g}{C}$
- S=Saturation flow rate

B) HCM Incremental Delay model (d2)

According to the HCM, the incremental delay consists of two components which account for the following: (1) delay due to random cycle by cycle fluctuations in demand that occasionally exceed capacity, and (2) delay due to sustained oversaturation during the analysis period. Intersection Controlling System (Roses 2004).

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\left(\frac{8kIX}{cT}\right)}\right] \qquad \text{Equation 2.20}$$

Where;

- c = capacity, veh/h
- T = analysis period, h, 0.25h
- k = incremental delay factor for actuated controller settings; 0.50 for all pre-timed controllers
- I = upstream filtering/metering adjustment factor; 1.00 for all individual intersection analyses
- X = v/c ratio

C) Initial delay model (d3)

HCM provides a room for additional delay incurred due to an initial queue of unmet demand in the previous time period.

2.6.2 Level of Service (LOS)

Level-of-Service (LOS) of a traffic facility is a concept introduced to relate the quality of traffic service to a given flow rate. Level-of-Service is introduced by HCM (2010) to denote the level of quality one can derive from a local under different operation characteristics and traffic volume. HCM (2010) proposes LOS as a letter that designate a range of operating conditions on a particular type of facility. Six LOS letters are defined by HCM (2010), namely A, B, C, D, E, and F, where A denote the best quality of service and F denote the worst. These definitions are based on Measures of Effectiveness (MoE) of that facility. Typical measure of effectiveness include speed, travel-time, density, delay etc. for highway segment and Vehicle delay per hour for intersection (HCM 2010), (Nicholas J. Garber and Lester A. Hoel 2009). There will be an associated service volume for each of the LOS levels. A service volume or service flow rate is the maximum number of vehicles, passengers, or the like, which can be accommodated by a given facility or system under given conditions at a given LOS (HCM 2010).

LOS can be characterized for the entire intersection, each intersection approach, and each lane group. Control delay alone is used to characterize LOS for the entire intersection or an approach. Control delay and volume-to-capacity ratio are used to characterize LOS for a lane group. Delay quantifies the increase in travel time due to traffic signal control. It is also a surrogate measure of driver discomfort and fuel consumption. The volume-to-capacity ratio quantifies the degree to which a phase's capacity is utilized by a lane group. The following paragraphs describe each LOS (HCM 10).

A lane group can incur a delay less than 80 s/veh when the volume to capacity ratio exceeds 1.0. This condition typically occurs when the cycle length is short, the signal progression is favorable, or both. As a result, both the delay and volume-to-capacity ratio are considered when lane group LOS is established. According HCM 10, A ratio of 1.0 or more indicates that cycle capacity is fully utilized and represents failure from a capacity perspective (just as delay in excess of 80 s/veh represents failure from a delay perspective).

Table 2-6: Level of Service at Intersection Control (HCM 10)

Control Delay (S/Veh)		<10	10 – 20	20 - 35	35 – 55	55 – 80	>80
Volume-to-Capacity Ratio	≤1.0	A	B	C	D	E	F
	> 1.0	F	F	F	F	F	F

The level of service at any intersection on a highway has a significant effect on the overall operating performance of that highway. Thus, improvement of the level of service at each intersection usually results in an improvement of the overall operating performance of the highway (Nicholas J. Garber and Lester A. Hoel 2009).

2.7 Similar or related Studies and findings on AALRT Level crossings

A MSc thesis done by Habatmu Sied in 2015 *entitled "**Pedestrian Level Crossing Safety on the East West Line of Addis Ababa Light Rail Transit**"* tried to cover the safety performance of pedestrian's level crossings at or near by the level stations along the east- west line of AALRT. The researcher used the Train Pedestrian Values (TPV) which is the product of the maximum number of pedestrians and the number of trains passing over the crossing within a period of 15 minutes to classify the pedestrian's level crossing nearby or at Level Stations. Finally, based on TPV calculated for each level Crossings nearby or at Level Stations, the researcher recommended appropriate precaution for pedestrian's crossings. It was recommended that Road Markings, Audible warnings and Pedestrian Signals at all pedestrian's Level crossings as listed in table 2-5 below.

Table 2-7: An appropriate precaution for pedestrians in each level crossing (Stations) (Habtamu 2015)

No.	Stations	Pedestrians	Width of footway	Road marking	Audible warning	Pedestrians signals	Tactile thresh	Guard rails
1	Torhailoch	A	> = 3.6m	✓	✓	✓	* ✓	**X**
2	Coca cola	B	> = 3.6m	✓	✓	✓	**X**	**X**
3	Yodranos Hotel	A	> = 3.6m	✓	✓	✓	**X**	**X**
4	Chemical corporation	A	> = 4m	✓	✓	✓	✓	**X**
5	Mazoria (traffic head quarter)	A	> = 4m	✓	✓	✓	✓	**X**
6	Lem hotel	A	> = 4m	✓	✓	✓	✓	**X**
7	Megenagna square	A	> = 4m	✓	✓	✓	**X**	**X**
8	Gurd shola	A	> = 4m	✓	✓	✓	**X**	**X**
9	Salite mehret church	B	> = 4m	✓	✓	✓	✓	**X**
10	Civil service	B	> = 3.6m	✓	✓	✓	**X**	**X**
11	Saint michael church	A	> = 4m	✓	✓	✓	✓	**X**
12	CMC 1	B	> = 4m	✓	✓	✓	✓	**X**
13	Meri CMC 2	A	> = 4m	✓	✓	✓	**X**	**X**
14	Ayat 2	B	> = 3.6m	✓	✓	✓	✓	**X**

Moreover, the research was limited by some data since the research was done before the implementation of AALRT's operation by considering some parameters from design (e.g. Head way time =6 minute) and other converting from corresponding area (e.g. Pedestrians volume from the road near by the crossing or station).

Christine Nyakona (2019) in the MSc thesis titled "***Harmonizing Pedestrians Mobility at Level Crossing (the case of Sebategna level crossing in Addis Ababa)***" Showed that improving and adjusting the infrastructure geometry (Shared level Crossing, Lane Configuration, Conflict Areas) in place can improve pedestrian safety and mobility. The outer used three scenarios to harmonize the pedestrian's mobility using Vissim Software.

- Existing Condition/Situation
- scenario one: providing separate pedestrian crosswalk away but close to the existing shared level crossing and adjusting the vehicular lanes
  - Head way time = 15 Minute
- Scenarios Two: Scenario one plus Coordinating Signal Control

The results of the findings were summarized in table and provided as table 2-6 below.

Table 2-8: Simulation results for existing, test scenario 1 and test scenario 2 (Nyakona 2019)

	Parameters	**Existing Situation**	**scenario one**	**Scenario Two**
Parameters used for modelling	Approaching lane Width (m)	3.5	3.5	3.5
	Right Turn Lane width (m)	3 lanes (8m)	3.0	3.0
	Pedestrian length and width respectively (m)	2.4,10	2.4, 10	2.4,10
	Time headway of Train (Minute)	15	15	15
	Coordinated Signal Control	**X**	**X**	✓
Performance Measured after the analysis	Queue length (m)	64.66	25.63	90.46
	Maximum Queue Length (m)	227.4	194.04	317.85
	Vehicles delay, all (s)	50.52	14.47	170.13
	Pedestrian Average stop, all (s)	0.01	0.03	0.04
	Conflicts number	120	110	51

According to Christine Nyakona (2019) implementing scenario 1 is cheaper while scenario 2 is the safest for pedestrian crossing.

Three Scholars, Demelash Berhane Mengesha, Murad Mohammed and Alemayehu Gebissa (2019) studied the performance of AALRT level crossing at two selected intersections with the title of "***Assessment of the Influence of at Grade Road-Rail Crossing on Traffic Performance in Addis Ababa, Case of Sebategna and Adey Ababa Road-Rail at Grade Intersection***". The outers used the actual traffic Volume counted manually with the support of video camera at peak hour and the actual level crossing dimension on the site and analyzed in MS Excel (Performance) and Vissim Software for LOS and Vehicles delay (Demelash Berhane Mengesha et al.: 2019). The paper had considered three scenarios at both selected level crossings as listed below.

i. The actual case (All users were considered at level crossings);
ii. Without Light Rail Vehicle (LRV);
iii. Without pedestrians.

The result of the analysis in Vissim software is summarized in table 2-8 below.

Table 2-9: Result summary of Level crossings for Adey Ababa intersection and Sebategna (Demelash Berhane Mengesha et al.: 2019)

<table>
<tr><th rowspan="2">Road-Rail Crossing name</th><th rowspan="2">Scenario No</th><th rowspan="2">Average Delay, sec./veh</th><th colspan="3">Additional Delay due to:</th><th rowspan="2">Peak hour volume conflict with LRV</th><th rowspan="2">Loss description</th></tr>
<tr><th>Pedestrian</th><th>LRV</th><th>Other</th></tr>
<tr><td rowspan="3">Adey Ababa</td><td>1</td><td>21.95</td><td rowspan="3">24 % or
5.37 Sec/veh</td><td rowspan="3">7 % or
1.51 sec/veh</td><td rowspan="3">69 % or
15.017 sec/veh</td><td rowspan="3">35%</td><td rowspan="3">LOS_E and LOS_F</td></tr>
<tr><td>2</td><td>20.48</td></tr>
<tr><td>3</td><td>16.5</td></tr>
<tr><td rowspan="3">Sebategna</td><td>1</td><td>29.69</td><td rowspan="3">44 % or
13.22 Sec/veh</td><td rowspan="3">5% or
1.46 sec/veh</td><td rowspan="3">51% or
15.01 sec/veh</td><td rowspan="3">48%</td><td rowspan="3">Between LOS_D and LOS_F</td></tr>
<tr><td>2</td><td>28.25</td></tr>
<tr><td>3</td><td>16.5</td></tr>
</table>

In their discussion of the result, the outers argued that the existence of pedestrian adds greater severity on vehicular delay than that of LRV. According to the outers, additional delay due to LRV Crossing have a considerably less effect on the existing traffic performance problem. Additionally, the outers recommended three significant measures that assumed to increase the performance of these level crossings; Alternative route for vehicle destination of Merkato Separate & clearly lane for pedestrian and vehicles at both Junction, and Pedestrian Over pass were the recommended measurements to increase the performance at the said level crossings.

In spite of the conclusion “Additional delay due to LRV Crossing have a considerably less effect on the existing traffic performance problem” the research should be extended at different head time including the design head time (6 seconds) which would be implemented by AALRT to support as the outers did not show the LRV head time at which these results obtained. Moreover, it is possible to guess they used the actual LRV head time which was about 15 minutes (one direction) during the research time, 2019.

Another three researchers, Robel Desta, Daric Tesfaye and Janos Toth (may 2021) conducted research on a titled **“Microscopic Traffic Characterization of Light Rail Transit Systems at Level Crossings**” a case study at ALRT Level crossings, particularly at Sebategna and CMC level Crossings. The study used to describe five different cases using VISSIM Simulation of traffic data collected at the study points.

1) **Without LRT:** the actual geometric and traffic characteristics are considered without the LRV and the LRT line.
2) **Actual LRV arrival:** the actual geometric and traffic characteristics are considered along with the LRV and the LRT line, combining the observed arrival frequency
3) **Twice actual LRV arrival:** the actual geometric and traffic characteristics are considered along with the LRV and the LRT line, incorporating a hypothetical twice arrival frequency
4) **Signalized actual LRV arrival:** the actual geometric and traffic characteristics are considered along with the LRV and the LRT line, combining the observed arrival frequency. Signal program is incorporated for facilitating the movement of the general traffic and LRV.
5) **Signalized twice actual LRV arrival:** the actual geometric and traffic characteristics are considered along with the LRV and the LRT line, incorporating a hypothetical twice arrival frequency. Signal program is incorporated for facilitating the movement of the general traffic and LRV.

The study was used the LOS and Delay to characterize the level crossings traffic as summarized in table 2-8 below for Sebategna Level crossing.

According to Robel et.al (2021) the delays for without LRT and Actual with LRV is the same, while increasing the frequency of LRV to twice of the actual will increases the delay of the traffic. Moreover, Introducing the signal will lead to more delay as per the result of the paper.

Table 2-10: Sebategna level crossing simulation results (Robel Desta, Daric Tesfaye and Ja´nos To´th 2021)

Controlling Methods	Cases	LOS	Delay (all), Seconds
All way Stop Control (AWSC)	Without LRT	C	23.5
	Actual LRV arrival	C	23.5
	Twice actual LRV arrival	D	33.1
Signalizing (C=80 seconds)	Signalized actual LRV arrival	E	67.3
	Signalized twice actual LRV arrival	F	98.5

However, the outers did not consider the effect of pedestrians and did not show clearly the actual head way of LRV during the study period.

Hence, the research shall be extended incorporating the effects of pedestrians with some modification of intersection geometry as well as some traffic movement management modifications, such as restricting left and /or U-turns.

2.8 Sidra Intersection Software

Sidra Intersection is a software package used for intersection (junction) and network capacity, level of service and performance analysis by traffic design, operations and planning professionals. (SIDRA 2023). This software features allow the users to use their own input values for different futures. Among these, Movement Classes, Origin -Destination (OD) Movements, Lane Based model, Flow Proportion and Lane Blockage, intersection Geometry, different Intersection models (Round about, Two-way Sign control, Signal Control), Demand and Sensitive analysis and graphically supported outs puts' (Results') summary (Amrozi 2023), (Akcelik 2022).

Sidra is preferable for Network construction, Phase settings, output speed and Convenience operation for vehicle average delay (CHEN Tianzia, JIN Shaochenb, YANG Hongxuc 2013). The software is flexible to use different Manuals such as HCM, common manual for Transportation Modelling operational analysis, which is already mounted with in the software as optional use with others. Sidra Intersection is a good predictor for Control delay at signal intersection and it is better than Highway Capacity Software (HCS 2000) (Bashar H. Al-Omari and Madhar M. Ta'amneh 2007).

There are some research papers that were already done and published on international journals which indicate that Sidra Intersection can be a tool for modeling and analyzing traffic characteristics. To list some, Kumala Terakegn with other two authors had published in which SIDRA Intersection tool had been used in the analysis (Tarekegn Kumala, Prof. Emer T. Quezon, Bogale Shiferaw 2016); Mulualem Birmeta in His MSc Thesis ((Mulualem 2020); Habatmu Mebratu with other Authors ((Habtamu Mebratu Yimer, Sharadchand B. Dugad, Prashant Kumar Gangwar and Abebe Halefom4 2021); Tsedey Ababa on her Msc thesis (Tsedey 2020); Sentayehu Leleisa with other Authors (Sentayehu Leleisa, Emer Tucay Quezon, Teyba Wedajo 2018) have used Sidra Intersection tools in their traffic modeling analysis.

CHAPTER 3 MATERIALS AND METHODS/ METHODOLOGY

3.1 Study Area

The city of Addis Ababa is the capital and largest city in the country of Ethiopia. Located in the center of Ethiopia, Addis Ababa (also known as Finfinne) currently has a population of **4.8** million people in the urban area and **2.7** million people in its city area (Population Stat May,2020). With the projection of Addis Ababa Population World Population Review estimated about 5.2million are living in 2022 and forecasted to be about 7.4 million by 2030. (World population Rview 2022)

Addis Ababa is a chartered city and also serves as the capital of the Oromia Region, the homeland of the Oromo people. The city is a few miles west of the East African Rift, which splits the country in two. Considered the political capital of Africa, Addis Ababa is where the African Union is headquartered and where the Organization of African Unity was once based. It is also the location for the United Nations Economic Commission of Africa. This has given the city political and diplomatic significance in the continent (Population Stat May,2020).

Addis Ababa Light Rail Transit open for operation February 1, 2015 ((Railwaygazette 2015) with two lines, North-South and East- west have a total of 9 level crossings.

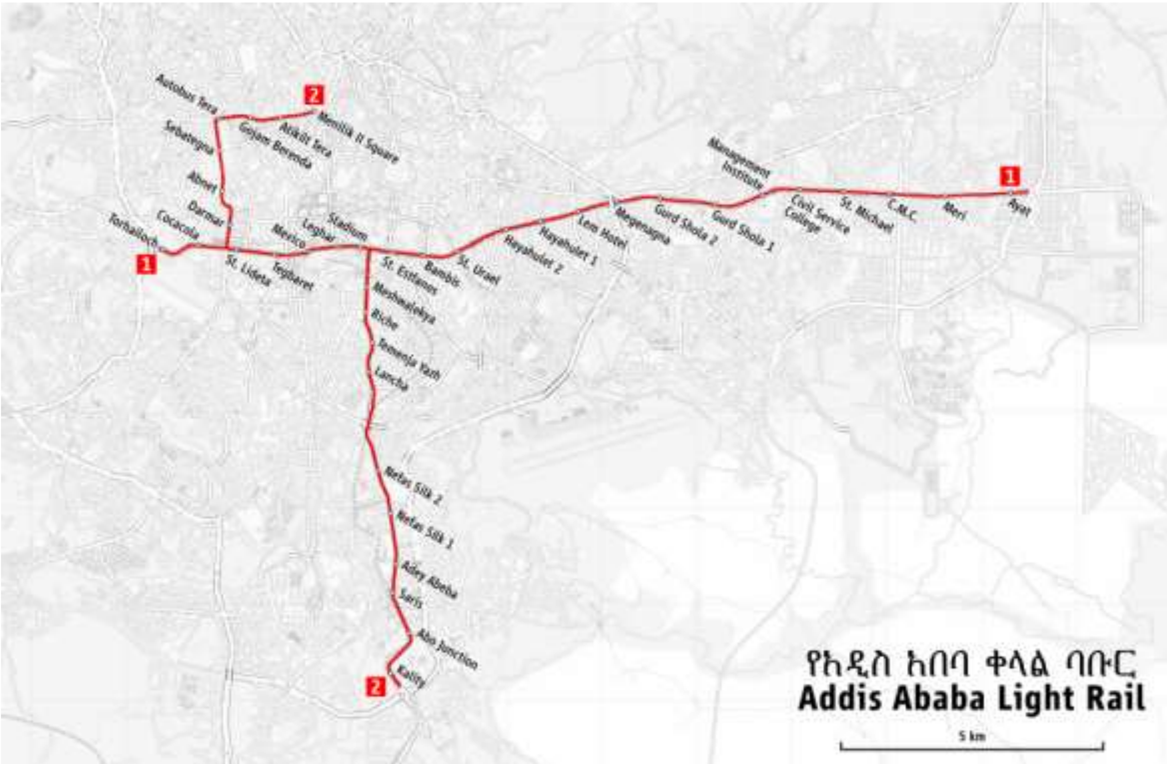


Figure 3-1: AALRT Map (AALRT 2019)

Among these crossings, 2 (Two) level crossings, **Adey Ababa and Sebategna**, along the North-South line are selected specific study area. These Level crossings are selected due to the fact that the traffic congestion observed in day-to-day activities at these level Crossings impacts the mobility of venders and other road users.

Sebategna Level Crossing (Google Image,2022)

b) Adey Ababa Level Crossing (Google Image. 2022)

Figure 3-2: Level Crossings along N-S line: a) Sebategna; b) Adey Ababa

3.2 Method

In order to achieve to objective of the research, the following procedure has set in the flow chart.

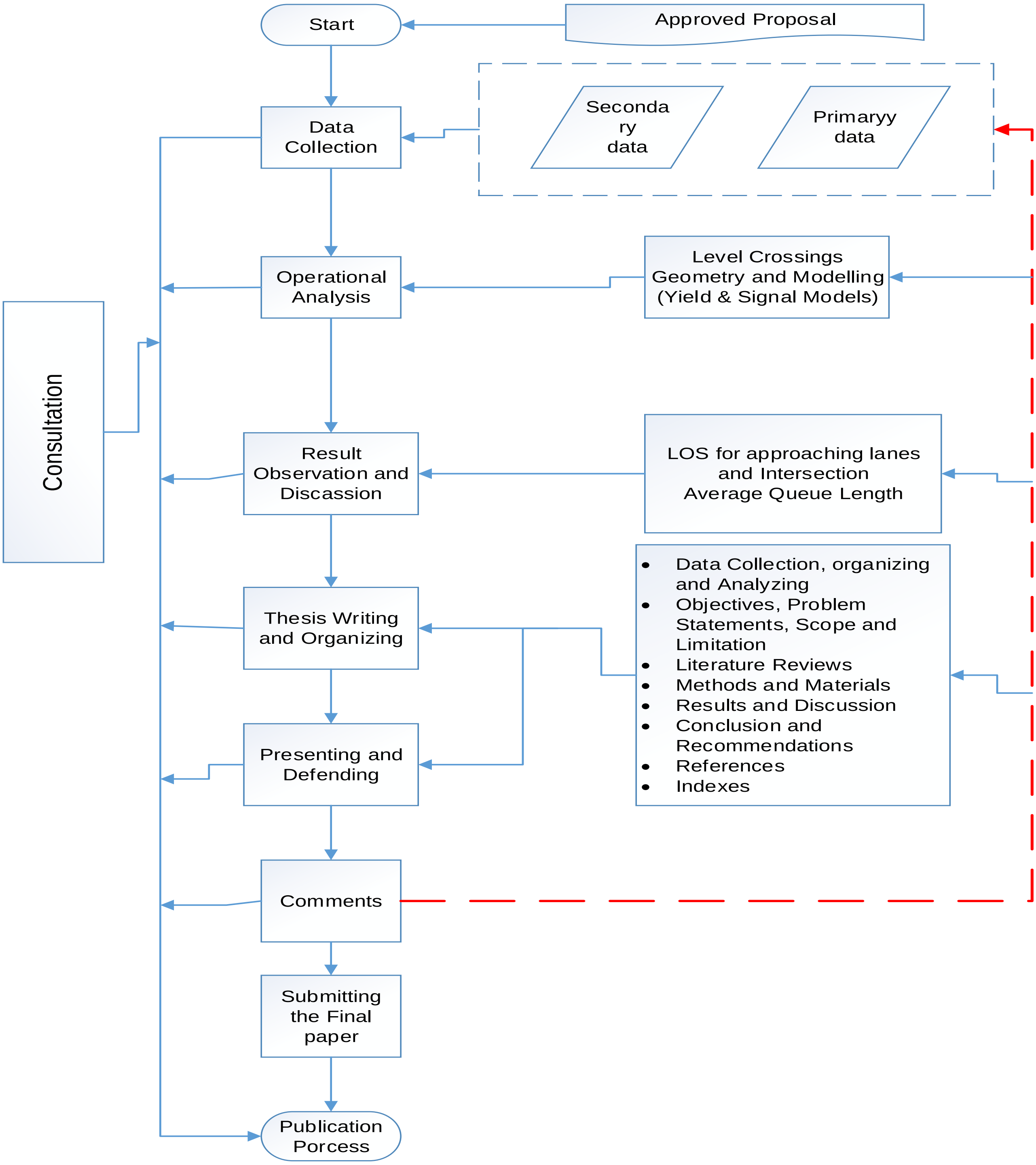


Figure 3-3: Flow Chart

3.3 Materials

- **Video Camera**: To collect Traffic and Pedestrian volume at the interested crossings
- **Pen and Note Book**: To note (Jot down) any observation during the data collection
- **Sidra Software**: To evaluate the performance of the interested crossings
- **Traffic and Transport Standards or Manuals:** To set the Bench marks for decision and discussion the results

3.4 Data Collection

3.4.1 Geometric Data

Geometric data of the intersection was collected from the site and from Addis Ababa Transport Bureau for the actual site condition.

Table 3-1: Geometric Data of Sebategna and Adey Ababa Level Crossing

<table>
<tr><th>Level Cross Name</th><th colspan="4">Sebategna Level Crossing</th></tr>
<tr><td>Approaches</td><td>Merkato</td><td>Abinat</td><td>Amanuel</td><td>Autobus Tara</td></tr>
<tr><td>Number of Lane/directions</td><td>3</td><td>3</td><td>1</td><td>3</td></tr>
<tr><td>Lane width</td><td>3.2*3=9.6m</td><td>3.2*3=9.6m</td><td>3.5m</td><td>3.2*3=9.6m</td></tr>
<tr><td>Left Turn (LT) Protection</td><td>Unprotected</td><td>Unprotected</td><td rowspan="2">LT +TH+RT</td><td>Unprotected</td></tr>
<tr><td>Right Turn (RT) Protection</td><td>RT with TH</td><td>RT with TH</td><td>Exclusive RT Lane</td></tr>
<tr><td>Level Cross Name</td><td colspan="4">Adey Ababa Level Crossing</td></tr>
<tr><td>Approaches</td><td>Saris</td><td>Addis Sefer</td><td>Gotera</td><td>Beretsige</td></tr>
<tr><td>Number of Lane/directions</td><td>3</td><td>2</td><td>3</td><td>2</td></tr>
<tr><td>Lane width</td><td>3.2*3=9.6m</td><td>3.2*3=9.6m</td><td>3.5m</td><td>3.2*3=9.6m</td></tr>
<tr><td>Left Turn (LT) Protection</td><td>Unprotected</td><td>Unprotected</td><td>Unprotected</td><td>Unprotected</td></tr>
<tr><td>Right Turn (RT) Protection</td><td>Shared with through</td><td>Shared with through</td><td>Shared with through</td><td></td></tr>
</table>

3.4.2 Traffic Volume data

3.4.2.1 Video Recording Method

To capture the traffic condition at the study time video camera was used. Video camera has its own advantage in collecting Traffic volume in which it will allow to record all events and incidents at the time of recording which is difficult to collect manually at field (KHALIL 2013). It is practically acceptable to use video camera for traffic volume recording and there are some of the scholars who were used the same methods of recording on their studies in Addis Ababa City Traffic study.

The Video Camera was mounted on buildings (See table 3.2 and figure 3.4) located nearby the level crossings and was focused to cover each approach of the level crossing legs.

Table 3-2: Video Camera Position Characteristics

Level crossing Name	Distance from intersection (m)	Boulding Location (X, Y)
Sebategna LC	110m south west	X=470646, Y= 997961
Adey Ababa	27 m North East	X=474072, Y=990185

A-Sebategna LC

B- Adey Ababa LC

Figure 3-4: Camera Position for Traffic data collection

The date and time for the Collection of traffic volume data has been selected based on the traffic flow experience in the city. In order to get the representative traffic data, consultation with traffic engineering experts from Addis Ababa Transport bureau was done and they shared their best experience on traffic volume collection. According these staffs, they had been collecting the traffic data at different intersection for the implementation of "Safe intersection" projects at all days of the weekdays. It was confirmed that the traffic data collected from Tuesday, Wednesday and Thursday were almost the same while those collected on Monday and Friday were different. For the implementation of the' Safe Intersection' projects they were collecting the traffic data one day from the Normal days (Tuesday –Thursday) and one day from the abnormal days, usually Friday, and finally the average of the data collected used for design and implementation.

Similarly, for this paper works, the traffic data was collected on Thursday and Friday for Sebategna LC and Tuesday and Wednesday for Adey Ababa LC.

Traffic Volume data were collected using video Camera for the interval of one hour with three times a day at the anticipated peak hours, Morning, mid-day and evening. Thus, the traffic volume was collected at each level crossing for total of six hours as described in table 3-3 below.

Table 3-3: Time of Traffic Volume Collection

Level Crossing Name	**Traffic Volume collected Time**		
	Morning	**Mid-day**	**Evening**
Sebategna LC	8:00 – 9:00am	12:00pm –7:00pm	5:00pm – 6:00pm
Adey Ababa LC	8:00 – 9:00am	12:00pm –7:00pm	5:00pm – 6:00pm

The Collected Traffic Volume at this intersection was extracted from the Recorded video into traffic flows to different directions for the analysis.

3.4.2.2 Approaching traffic Volume

Once the traffic and pedestrian volume extracted from Camera Video at the morning, Mid-Day and evening, the maximum from the average of the 2 days has been used for the perform evaluation of the Level crossings.

According to the collection of traffic flow, the traffic flow at Sebategna Level crossing are almost the same throughout the day with the highest traffic flow at the morning and that of Adey Ababa almost the same at Mid-day and Evening but little higher at the morning. For the design purpose the morning session (with higher Traffic Volume) for both level crossings are taken. The two-day average traffic volumes are shown in table 3-4 and 3-5 for both level crossings.

Table 3-4: Average Traffic Flows at Sebategna Level Crossings

Approach	Morning (8:00am-9:00am)				
	Right Turn	Through	Left Turn	U- Turn	Total
Abinat	373	479	116	130	1098
Merkato	144	105	287	24	558
Amanuel	86	148	118	4	355
Autobus Tera	67	339	174	83	662
Total from all Approach entering to the intersection					**2,673**
Mid-Day (12:00pm-1:00pm)					
Abinat	246	421	58	90	815
Merkato	121	69	240	24	454
Amanuel	74	77	56	9	215
Autobus Tera	77	618	205	74	974
Total from all Approach entering to the intersection					**2,457**
Afternoon (11:00pm -12:00pm)					
Abinat	252	360	44	78	734
Merkato	79	68	210	35	391
Amanuel	80	63	55	7	205
Autobus Tera	78	633	122	62	894
Total from all Approach entering to the intersection					**2,223**

Table 3-5: Average Traffic Flows at Adey Ababa Level Crossings

Approach	Morning (8:00am-9:00am)				
	Right Turn	Through	Left Turn	U- Turn	Total
Saris	49	428	70	49	701
Addis Sefer	94	135	89	0	317
Gotera	121	621	186	251	1179
Beretsige	28	118	359	0	504
Total from all Approach entering to the intersection					**2701**
Mid-Day (12:00pm-1:00pm)					
Saris	15	383	46	159	602
Addis Sefer	97	153	114	0	363
Gotera	103	603	119	203	1027
Beretsige	18	85	44	0	547
Total from all Approach entering to the intersection					**2539**
Afternoon (11:00pm -12:00pm)					
Saris	45	395	75	179	693
Addis Sefer	102	110	10	0	312
Gotera	96	656	139	232	1122
Beretsige	49	117	282	0	447
Total from all Approach entering to the intersection					**2574**

3.4.2.3 Traffic Composition

The traffic data was collected by differentiating the vehicles into main Categories: Light Vehicles (Including Car, Taxi, Small and Medium Bus, Two Wheel Vehicles, Three Vehicles and Cyclists), Heavy Vehicles (Consists of Trucks and Large Buses) and Light Rail Vehicle (LRV). The composition of heavy vehicles was figured out from the collected data as below in tables.

Table 3-6: Heavy Vehicle Composition (in Number) at Sebategna Level Crossing, at the Morning

Approach	Movement Direction											
	Morning				Mid-Day				Evening			
	Right Turn	Through	Left Turn	U-Turn	Right Turn	Through	Left Turn	U-Turn	Right Turn	Through	Left Turn	U-Turn
Abinat	62	32	12	8	46	34	10	14	39	37	8	9
Merkato	28	15	53	3	21	9	41	2	11	3	30	2
Amanuel	6	8	7	-	5	11	6	1	4	6	6	-
Autobus Tera	6	22	10	5	6	15	7	4	8	29	11	5

Table 3-7: Heavy Vehicle Composition (in Number) at Adey Ababa Level Crossing

Approach	Movement Direction											
	Morning				Mid-Day				Evening			
	Right Turn	Through	Left Turn	U-Turn	Right Turn	Through	Left Turn	U-Turn	Right Turn	Through	Left Turn	U-Turn
Saris	2	22	2	9	-	26	1	3	1	36	3	6
Addis Sefer	1	2	1	-	1	2	1	-	-	2	-	-
Gotera	1	37	1	-	2	56	2	4	-	34	2	1
Beretsige	1	1	3	-	2	-	2	-	-	2	3	-

3.4.3 Pedestrian Volume

The pedestrians at the level crossing were collected together with the traffic flows using Video recorder camera and then the pedestrian's number for each leg are counted from the recorded video.

Table 3-8: Pedestrian Volume at Sebategna and Adey Ababa Level Crossings

Level crossing	**Approaches**	**Direction**	**Pedestrian Volume (each direction)**	**Total Ped. Volume at each leg**
	Abinat Approach	Amanuel to Merkato	2289	4585
		Merkato to Amanuel	2296	

Sebategna Level Crossings	Merkato Approach	Abinat to Autobus Tera	952	2380
		Autobus Tera to Abinat	1428	
	Autobus Tera approach	Amanuel to Merkato	1651	2304
		Merkato to Amanuel	653	
	Amanuel Approach	Abinat to Autobus Tera	479	798
		Autobus Tera to Abinat	319	
Total Pedestrian Volumes at Sebategna LC				**10,067**
Adey Ababa Level crossing	Saris Approach	Beretsige to Addis Sefer	721	1361
		Addis Sefer to Beretsige	640	
	Addis Sefer Approach	Saris to Gotera	455	892
		Gotera to Saris	437	
	Gotera Approach	Beretsige to Addis Sefer	445	794
		Addis Sefer to Beretsige	349	
	Beretsige Approach	Saris to Gotera	722	
		Gotera to Saris	568	1290
Total Pedestrian Volumes at Adey Ababa LC				4337

3.4.4 Level Crossings Control Data

AALRT level crossings are the complex intersection where different users are in compete for the crossings. The current existing/Condition of controlling the level crossings is summarized in table 3-9 below in detail.

Table 3-9: Level Crossing protection and warning arrangements at Sebategna and Adey Ababa LC

Level Crossing Name	Protection form Train Movement	Warning Arrangements	Level crossing Type
Sebategna LC	**Un protected (No barrier).** -Traffic Police control the entry of Vehicles to the LC. The pedestrian sometimes interrupts the flow of traffic for crossing.	**Train approach:** Bells and Flashing lights that warns the Road Vehicles when the train approaches to the LC. There is Railway passenger station from one side nearby these crossings.	Active (Open Crossing)
Adey Ababa LC			

3.5 Modelling

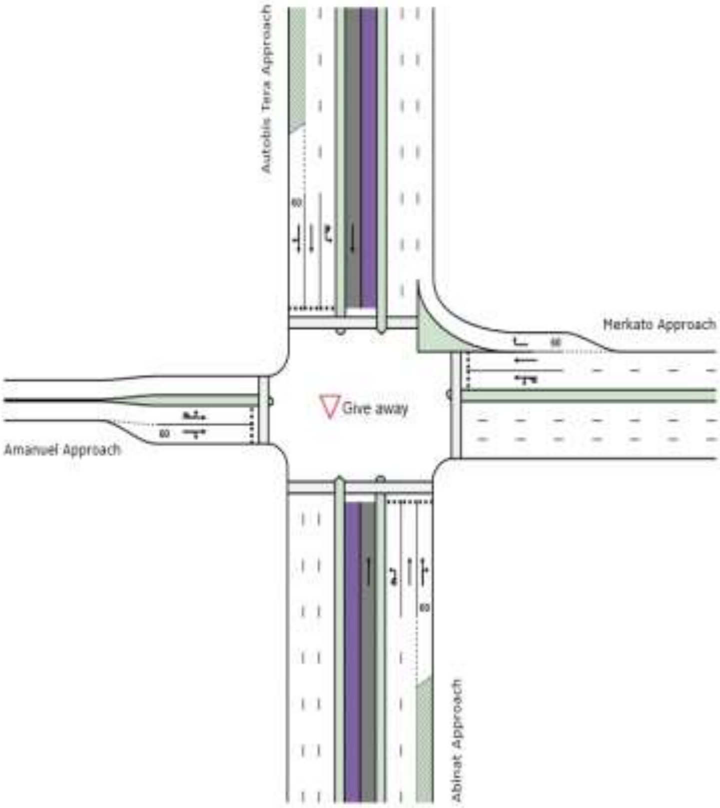
3.5.1 Give Way Model

The Road Vehicles from all approaches is controlled by the traffic Police which can be modelled as yield sign Control. in this model, the North – South movements are considered as Major Movements (has priority) whereas the approaching from east and west are considered as Minor roads. A YIELD sign control is applied for analysis of the Intersection.

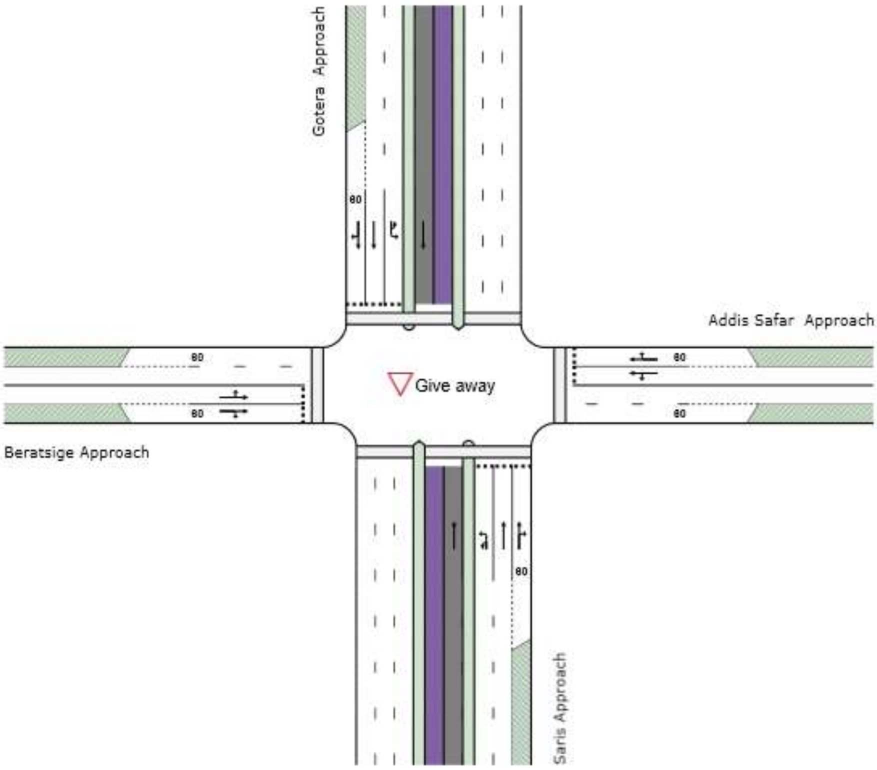
Accordingly,

- Priority for LRT vehicles at all time,
- All Left Turn gives priority for all through traffic Movements and left turn (Merging traffic) from Right hand movements.
- All through movements from all direction follows the right hand, Rule;
- Right Turn from all approaches gives Priority for crossing pedestrians and all merging vehicles and U-Turn Movements;
- As there are some vehicles such as taxi and buses use the external lane of Major Roads for loading and unloading, these lanes are modelled with allowing parking's;
- Pedestrian movements treated to be served when the traffic from one direction gives priority and no needs to treat separately for the case of give way model.
- Pedestrian effects are considered by using Right turn and Left turn adjustment according to HCM10.

In consideration of the above assumptions the Level crossings are modelled in Sidra Intersection 8.0 Plus for the analysis. The hourly Volume has been used whereas the Heavy vehicles and Left turns adjustments are set in the software's so that Sidra Intersection 8.0 Plus software will consider according to the set adjustment factor during analysis. As it was observed during the data collection, the left turn is not protected (Uncontrolled type of) left turns are also modelled in the approaching lanes. The actual Geometric size of each approaching lanes are used for the modelling. The following figures shows give way model for both Level crossings.



a) Sebategna LC



a) Adey Ababa LC

Figure 3-5: Give Away Model for Sebategna Level Crossing, Sidra Intersection-8

3.5.2 Signaling Model

Give a way Intersection control is mainly used when minor roads intersect with major roads and there will be a gap for a vehicle from the minor roads to cross the intersection otherwise the vehicles from the minor roads will be subjected to excessive delay and queening.

In the case of Sebategna and Adey Ababa AALRT level crossings, the intersection is controlled by Traffic Police/Traffic Control Person which allows the vehicles to enter into the intersection from different directions based on the available queening in random cases. Such type of control is subjective and sometimes difficult to model in Software's such as Sidra Intersection-8 using give away or Stop intersection control.

On the other hand, the Traffic and Pedestrian Volume at these Intersections are high and could warrant for the use of traffic signal control for better efficient performance of these intersection. Below the details of traffic signal for both intersections are discussed.

3.5.2.1 Check the Signal Warranty

According to MUTCD, peak hour warrant is used to justify the installation of traffic signals at intersections where traffic conditions during one hour of the day or longer result in undue delay to traffic on the minor street.

MUTCD sets eight criteria's (Warrants) in addition to engineering decision to justify the installation of Signals at the intersection. At least one of them must satisfy to be signalizing.

Table 3-10: Warranties for Signal justification

S.No	Warrant No		Section in MUTCD
1	Warrant 1	Eight-Hour Vehicular Volume	Section 4C.02 of MUTCD 2009
2	Warrant 2	Four-Hour Vehicular Volume	Section 4C.03 of MUTCD 2009
3	Warrant 3	Peak Hour	Section 4C.04 of MUTCD 2009
4	Warrant 4	Pedestrian Volume	Section 4C.05 of MUTCD 2009
5	Warrant 5	School Crossing	Section 4C.06 of MUTCD 2009
6	Warrant 6	Coordinated Signal System	Section 4C.07 of MUTCD 2009
7	Warrant 7	Crash Experience	Section 4C.08 of MUTCD 2009
8	Warrant 8	Roadway Network	Section 4C.09 of MUTCD 2009

As additional, Traffic Engineering hand book (Anurag 2016) adds additional warranty for at grade (level) crossings.

According to Anurag P. (2016), The Intersection Near a Grade Crossing signal warrant is intended for use at a location where none of the conditions described in the other eight traffic signal warrants are met, but the proximity to the intersection of a railroad grade crossing on an intersection approach controlled by a STOP or YIELD sign is the principal reason to consider installing a traffic control signal. The reason is due to concern that vehicular traffic from the STOP- or YIELD sign-controlled approach may queue onto or beyond the grade crossing.

It is also important to note that satisfying or even exceeding the MUTCD signal warrant criteria does not require the installation of a traffic signal. These warrants are intended to be minimum conditions to consider the need for traffic signal control. (Anurag 2016), (Nicholas J. Garber and Lester A. Hoel 2009).

From these warrantees Sebategna and Adey Ababa level crossings were checked against for warrantee three (Peak Hour criteria) as a base in addition to the 9th criteria set by Anurag P. (2016).

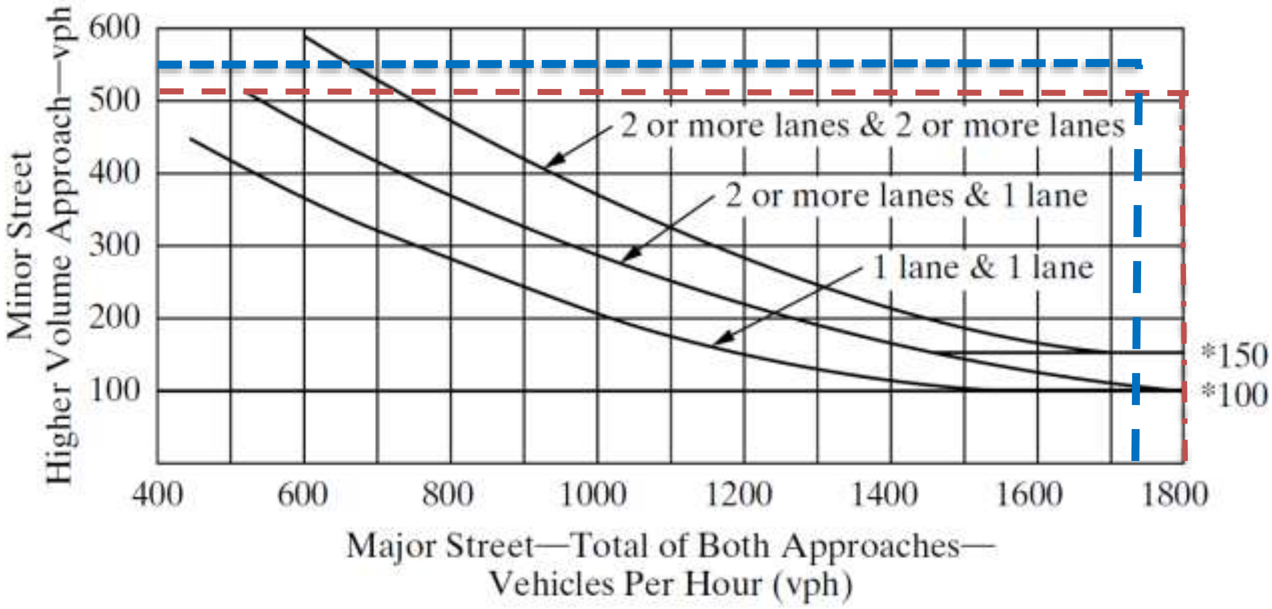
This warrant is used to justify the installation of traffic signals at intersections where traffic conditions during one hour of the day or longer result in undue delay to traffic on the minor street. One of two conditions (A and B) should be satisfied for the warrant to be satisfied.

The justification for traffic signals at Sebategna and Adey Ababa Level crossings have been checked from the Collected traffic volume data within the peak hour. Accordingly, the collected data show that both Level crossings warrantee the traffic signaling as shown in table 3-11 below under MUCTD Condition A.

Table 3-11: warranty condition for Traffic Signal

Level Crossing	**Total Approach from Major Roads (North +South Approaches)**	**Minor Road Approach**		
		East Approach	**West Approach**	**Maximum**
Sebategna	1763	560	356	560
Adey Ababa	1880	317	504	504

The result is checked against the graph (Condition A of Warrantee 3) as shown in figure 3-6 below.



Legends: Adey Ababa Level Crossing

Sebategna Level Crossing

Figure 3-6: Warranty for Traffic Signal at Intersection, Condition A (MUTCD)

As shown in figure 3-6 above, both level crossings Satisfy minimum criteria for signalization under warrantee 3 (Peak hour) condition A. Thus, both level crossings can be controlled using signal controlling systems which will be discussed in sections 4.2 and 4.3.

CHAPTER 4 ANALYSIS AND RESULTS

4.1 Introduction

In this section the results of models with different intersection controlling system are discussed. Two types of models are used for analysis. These are:

1. Yield Sign control: to analysis the performance of the existing condition
2. Signal Control:
    a. At the existing traffic and intersection geometry condition; and
    b. with some traffic movement management and Geometric modification.

in the next section, 4.2, 4.3 and 4.4 the results of these models are discussed in detail including methods of modelling.

4.2 Yield Sign Control Result

At the existing condition, Adey Ababa and Sebategna Level crossings are performing through Yield sign control in such a way that rail has priority over all modes (Vehicle, Pedestrian and other non-motor Vehicles) at the intersection.

Accordingly, Priority has been given to the LRV in Sidra Intersection 8 software. In order to see the effects of LRV on LOS of the level crossings and Queue length, the Frequencies' of LRV were set to be 20min, 15min, 10min, 6min and 3min. In the priority rule, the direction that has no priority will wait until gets gap to proceed. The analysis was done considering the pedestrian's volume and at the Right turn the pedestrians were set to have priority over the turning Traffic. The results of the Analysis are shown in table 4-1 below for both level crossings.

4.2.1 Effects of Pedestrian on Yield Control

At the intersection pedestrian has priority over the right turn vehicles such that Right tuning traffic needs pedestrian's gap to proceed otherwise remain stopping until the acceptable gap is get. In the yield controls the only chance/probability for vehicles to cross is based the availability of acceptable gap and its priority over the other direction. For direction with probability of acceptable gaps are low, the delay along these approaches will be high which in turn increases the dalliance of the intersection.

Study on the Yield control model at Sebategna and Addis Ababa Level crossings shows the same. Setting priority for pedestrian over the right turning, Priority for LRV and Right hand rule in Sidra Intersection Software model, the result shows that pedestrians have severe effects on the intersection performance. At 6 minutes of LRV frequency; the result shows that the average of delay at Adey Ababa LC with pedestrian is about 26 minutes and without pedestrian is about 6.7 minutes for the same models. The same is true for Sebategna LC with 20.7 minutes with pedestrians and 3.5 minutes/vehicles without pedestrians for the same models.

4.2.2 Effects of LRV frequency on Yield Control

As it is seen in the above, the pedestrian effects are severing on the performance of intersection controlled by yield Sign. To study the LRV frequency effects on the Intersection using the same models but only varying the LRV volume both with and without pedestrians have been checked. The frequency of the LRV were set to be 20minutes, 15 minutes, 10 minutes,6 minutes and 3 minutes from which LRV Volume is computed. According to the result, LRV frequency has almost the same effects on both cases with near to zero seconds (about 10 sec/veh) of additional to the traffic delay as shown in table 4-1 below.

Table 4-1: Yield Control model results (without Pedestrian)

LRV frequencies (Minutes)	**Sebategna Level Crossing**		**Adey Ababa Level Crossing**	
	Average Delay (Sec/Veh)	Additional Delay due to LRV (sec/veh)	Average Delay (Sec/Veh)	Additional Delay due to LRV (sec/veh)
Without LRV	400.2	Reference	203.8	Reference
20	401.5	1.3	204.9	1.1
15	402.1	1.9	205.4	1.6
10	403.4	3..2	206.5	2.7
6	406.1	5.9	208.6	4.8
3	413.0	12.8	214.2	10.4

As it can be seen that the control in Yield sign will lead to excessive delay to the traffic irrespective of the Light Rail vehicle operations at both level crossings. Hence, these Level crossings shall be checked for the other intersection controlling mechanisms as discussed in the next sections (4.3 and 4.4) which would decrease the excessive delays.

The effect of pedestrian's and Light Rail Vehicle Frequency has been model with linear regression for both Level crossings. The equation is modelled by Varying the pedestrians' Volume and Light rail Vehicles Head time.

$$Delay\ (Minutes) = \alpha * Pedestrians'Volume\ (No) + \beta * LRV_{Head}Time\ (Minutes) + C$$

Equation 4.1

Where, each Coefficients are presented in table 4-2 below.

Table 4-2: Delay Model for Yield Traffic Control at Level Crossings

$\boldsymbol{Delay\ (Minutes) = \alpha * Pedestrians'Volume\ (No) + \beta * LRV_{Head}Time\ (Minutes) + C}$		
Coefficient	**Adey Ababa LC**	**Sebategna LC**
A	0.02	0.013
B	-0.004	-0.006
C	0.696	5.384
R	0.922	0.853
Remark	Pedestrians' Volume shall be only from one approach.	

According to the model coefficients, the pedestrian's volume has more effect the LRV head time. Increasing head time will decrease the delay, which mean that if the LRV frequency is so far, there will be more time that can be used by the road vehicles which in turn decreases the Delay at the Level crossings.

4.3 Signaling Model and Results

For intersection to be signalized it should satisfy at least one of the warranties for signalizing set by TUCD. Accordingly, as it is discussed in section 3.5.2, both level crossings satisfied one of the warranties; Peak hour warrant; for signalizing.

At the existing condition of traffic flows and Intersections geometry condition, the level crossings were modelled and set for the analysis in Sidra Intersection 8 software.

Sidra intersection 8 has a default as well as user defined values which uses to analyses based on the pre developed methods accessible for selection. In all cases the HCM 10 methods with Delay control as performance measures have been selected from the software privileges.

For the design of signals, the design parameters have been set based on the recommended values by the HCM 10 and some are computed from the available data using HCM Recommended methods. Some of the recommended parameter's values are presented in table 4-3 below in detail.

Table 4-3: Parameter selection for Signaling Models

S. No.	Parameter	Unit	Value	Remark/References
1	Saturation flow rate	Veh. /hr/ln	**1615**	Practical Default value for saturation flow rate (Roses 2004)
			1900	Base saturation flow rate per lane (HCM 10)
			1950	Default base saturation flow rate per lane (Sidra Intersection 8 Software)
2	Startup Lost, L1	Sec/Phase	2	Recommended default value (HCM 10)
3	Clearance Lost, L2	Sec/Phase	1 – 2	HCM 10
4	Yellow interval	Sec	3 – 6	HCM 10
5	Cycle length	Sec	<=120	
6	V/C ratio	-	0.85 – 0.95	
7	Approaching Speed	Kph	20 – 50	Addis Ababa City Administration's legal Speed Limit
8	Minimum Green Time	Sec	4 -15	HCM 10
9	Minimum Green Time	Sec	15-30	For Left turn phase (HCM 10)
			20 - 40	through phases serving the minor street approach (HCM 10)
			30 – 60	through phases serving the major street approach (HCM 10)
10	Left turn adjustment	Ratio	1.05	Protected left turning (HCM 10)
11	Right turn adjustment	Ratio	1.18	Unprotected left turning (HCM 10)

One of the major tasks in Signaling design is the determination of Cycle Length, Green and lost time required along with Signal phases. Different methods are available for these determination ((Roses 2004).

From the collected traffic data, at the first, weather left turn protection is required or not have been checked using Cross product Criteria ((Roses 2004) as presented in table 4-4 below.

Table 4-4: Left turn protection check

Approach	Movement	Volume (Veh/hr)	No. of Lanes	Sebategna Level Crossings		
				$V_{lt} \geq 200veh/hr$	$V_{lt} * (V_o/N_o) \geq 50,000veh/hr$	Remark (will be protected if both cases satisfied)
SB	RT	373	3	Vlt>200	27,798	Not satisfied
	TH	479		Required	No need	
	LT+UT	246				
EB	144	105	3	Vlt<200	4,391	Not satisfied
	TH	135		No need	No need	
	LT+UT	89				
NB	RT	67	3	Vlt>200	41,034	Not satisfied
	TH	339		Required	No need	
	LT+UT	257				
WB	RT	86	1	Vlt<200	16,470.0	Not satisfied
	TH	148		No need	No need	
	LT+UT	122				
Adey Ababa Level Crossing						
SB	RT	49	3	Vlt<200	24,633	Not satisfied
	TH	428		No need	No need	
	LT+UT	119				
EB	RT	94	2	Vlt<200	5,251	Not satisfied
	TH	135		No need	No need	
	LT+UT	89				
NB	RT	121	3	Vlt>200	62,345	**Satisfied**
	TH	621		Required	Required	
	LT+UT	437				
WB	RT	28	2	Vlt>200	24,232.5	Not satisfied
	TH	118		Required	No need	
	LT+UT	359				

Only North approaches of Adey Ababa meets the Cross-product requirement for the left turn lane protection. Accordingly, the outer lane of this approaching reserved to be used by Left and U_ turns.

Once the design cycle and no of phases are determined the next job is to determine the green time to each approaching lane according to their traffic volume. Sidra Intersection has on built system that can distribute effectively the green time to the approaching lanes with respective to the traffic volume. Hence, Sidra Intersection needs the design cycle as input to process the analysis. The cycle lane shall be designed either as of the methods previously discussed which would bring the lowest delay and lowest queue length at the intersection. On the other hand, the cycle time can be obtained by try and error considering the minimum control average delay and average minimum queue length(m).

The models, were first simulated with all parametres kept constant except the Cycle length. Then, the delay was recorded as a function cycle length which helps to select the optimum cycle length with lowest recorded delay.

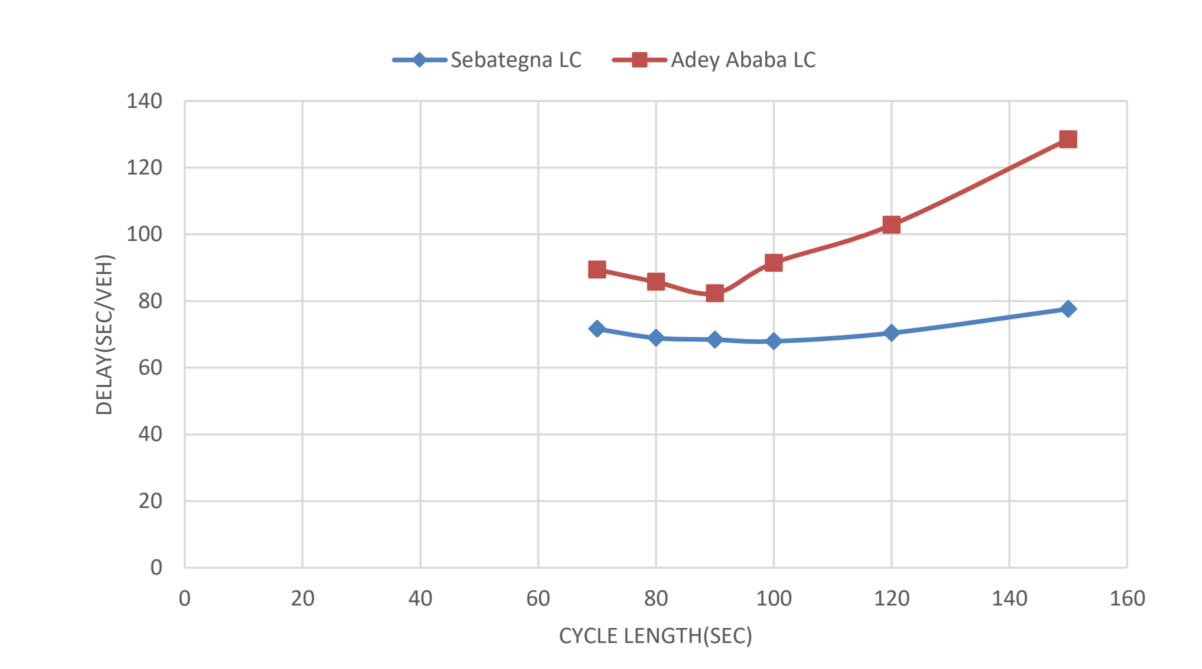


Figure 4-1: Determination of Cycle length

From figure 4-1 it can be observed that the performance measures (Delay in sec/veh) are achieved at the optimum cycle length of 90 seconds respectively for both Level Crossings. Accordingly, the signaling has been designed for each level with prioritization of LRV crossing as presented here below in tables and figures.

Table 4-5: Phase Summary for both LC

Phase	Adey Ababa Level Crossing				Adey Ababa Level Crossing			
	A	B	C	D	A	B	C	D
Phase Change Time (sec)	0	23	39	60	0	20	45	75
Green Time (sec)	19	12	17	10	16	21	26	11
Phase Time (sec)	23	16	21	14	20	25	30	15
Phase Split	31%	22%	28%	19%	22%	28%	33%	17%

The result of the model for this signal have been presented in the subsequent sections with detail discussion of the results.

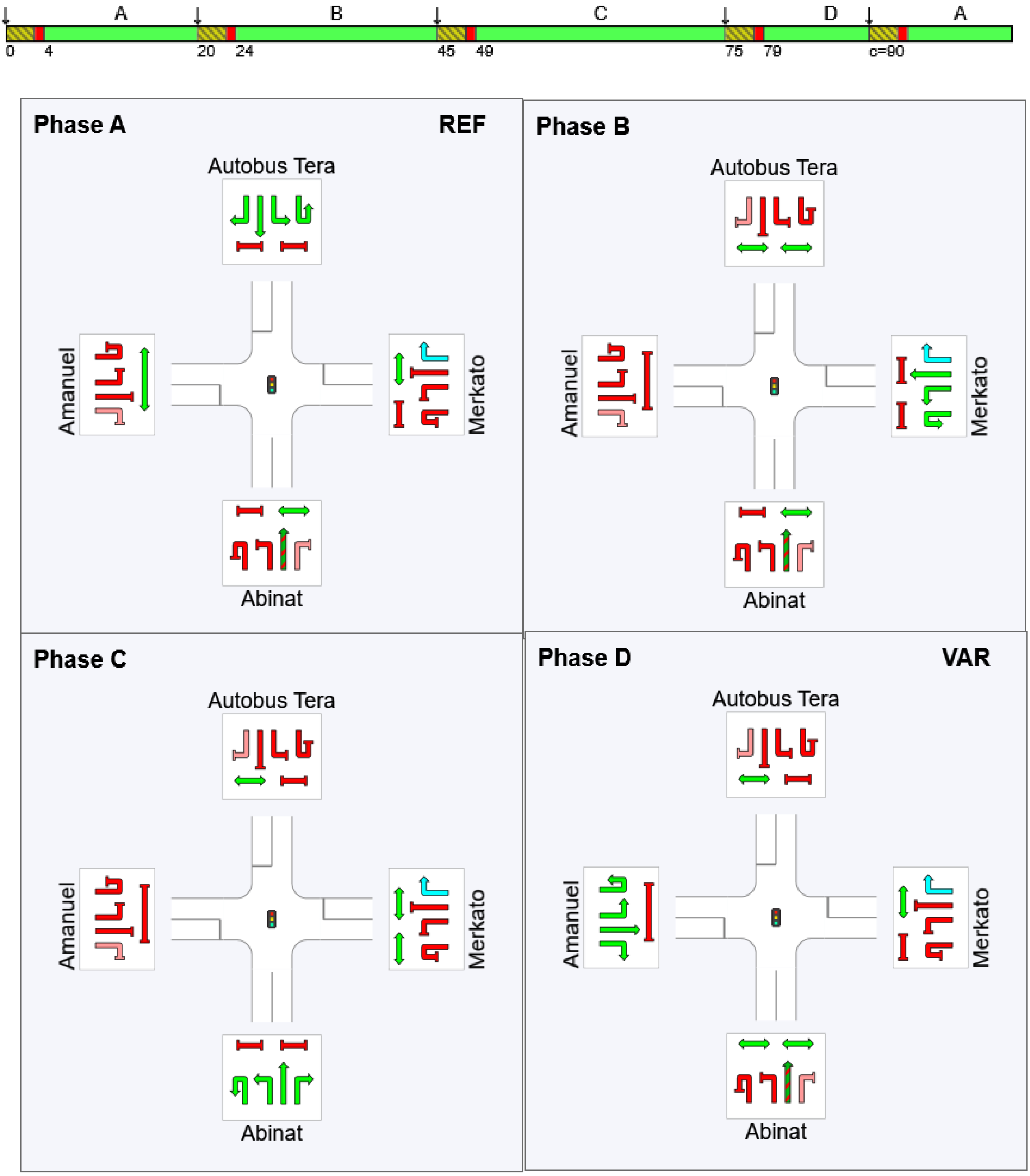


Figure 4-2: output phase sequence for Sebategna Level Crossing

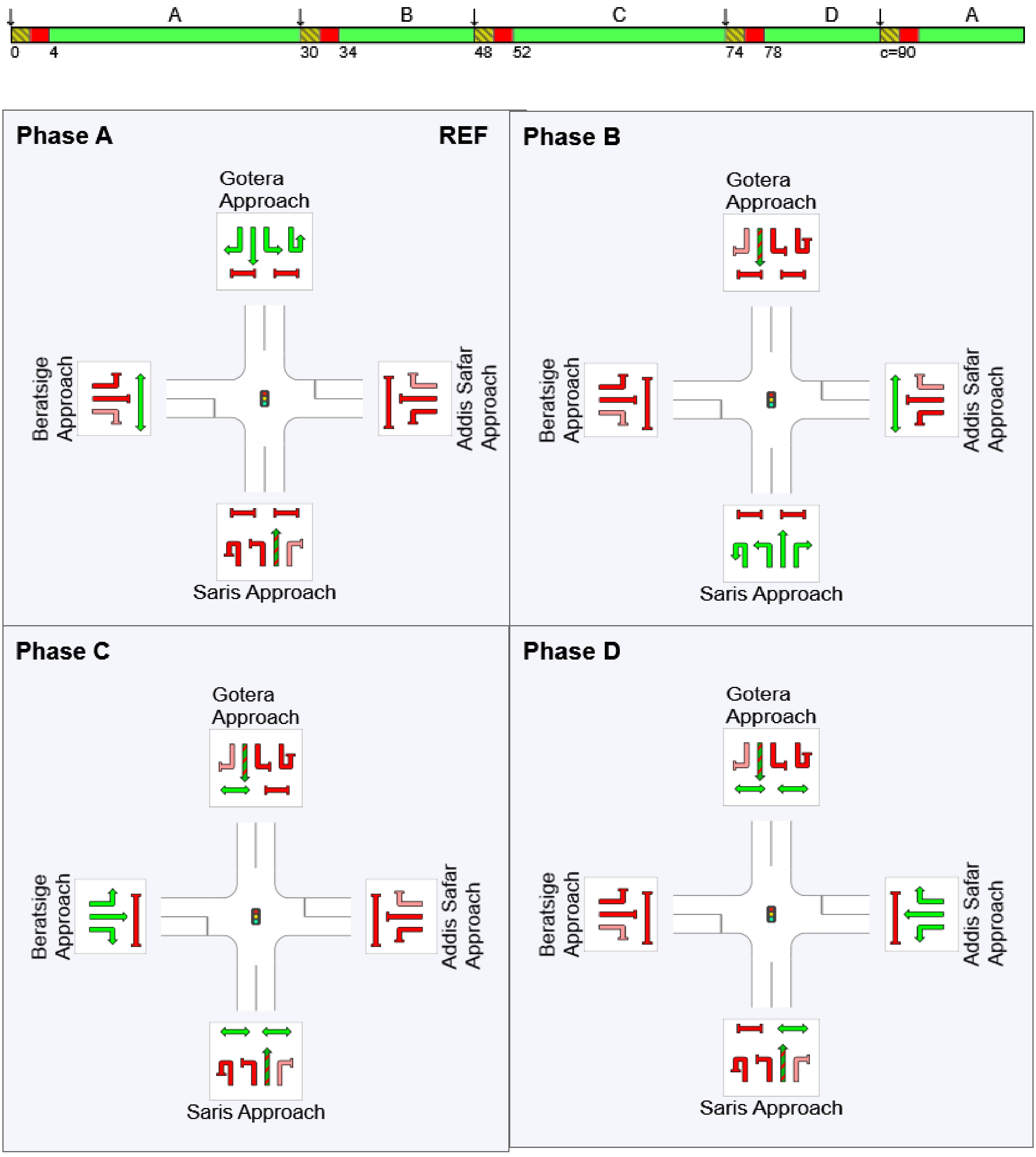


Figure 4-3: output phase sequence for Adey Ababa Level Crossing

Outputs of the analysis are summarized in the following tables for different frequencies of the Light Rail vehicle which has priority over the other traffic movements.

Table 4-6: Signal Control model results (Delay and LOS Criteria)

LRV Frequency		Sebategna LC					
		Approaches				Intersection	
head Time(minutes)	Volume	South	East	North	West	Delay (S/veh)	LOS
		45%	13%	27%	15%		
Without Pedestrian							
30	3	48.2	84.8	77.1	90.7	68.7	E
15	5	48.1	84.8	77	90.7	68.6	E
6	11	47.8	84.8	76.6	90.7	68.4	E
3	21	47.4	84.8	76	90.7	68	E
1	61	45.7	84.8	74	90.7	66.6	E
With Pedestrian							
30	3	109.8	98.5	73.5	59.5	91.7	F
15	5	109.6	98.5	73.3	59.5	91.6	F
6	11	109	98.5	73	59.5	91.3	F
3	21	109	98.5	73	59.5	91.3	F
1	61	104.3	98.5	70.4	59.5	88.7	F

LRV frequency		Adey Ababa LC					
		Approaches				Intersection	
		South	East	North	West	Delay S/Veh)	LOS
Head time(minutes)	Volume	26%	12%	44%	19%		
Without Pedestrian							
30	3	103.9	76.3	66.6	92.3	81.4	F
15	5	103.5	76.3	66.5	92.3	81.2	F
6	11	102.5	76.3	66.2	92.3	80.9	F
3	21	100.8	76.3	65.6	92.3	80.2	F
1	61	94.7	76.3	63.5	92.3	77.9	F
With Pedestrian							
30	3	72.2	69.2	118.4	109.3	100	F
15	5	72	69.2	118.2	109.3	99.8	F
6	11	71.3	69.2	117.6	109.3	99.4	F
3	21	70.1	69.2	116.6	109.3	98.6	F
1	61	65.8	69.2	112.8	109.3	95.7	F

From table 4-6, the result reshows that for both with and without Pedestrian, increasing LRV frequency will decrease the average delay of Vehicles at LC. This is due to the fact that, when LRV frequency increases, it will provide additional movement time for the major roads with larger traffic approaches (North and south approaches in both Level crossings) which leads to the decrease of delay for these major roads approaching.

It is observed that the presence of pedestrian at level crossings will result in additional delay to the roads' vehicles. According to the result, there are additional delays 18.4 secs/vehicles at Adey Ababa LC and 23 sec/Vehicle at Sebategna LC.

The effects of pedestrians on the intersection performance are sounder for intersection with higher performance than those of performing at lower level of service. For Intersection performing at LOS F, there is enough gap for the pedestrians to cross the intersection and their effect is invisible (unforeseen). This would be remaining true because at the first without pedestrian, the intersection is already performing at LOS F, which means that the vehicles are almost stopping and if there are any pedestrians, they can cross the intersection without creating any additional delay on the traffic but the performance of pedestrians will be changed based on the number of Pedestrian volume and available gap for crossings.

According to the result, the low performance of the level crossing is not due to the presence Light rail vehicle but due to the large number of the road traffic at the intersections. If mitigation solution for road traffic has been taken, especially for those are left turns, the Level crossings can operate even up to 3minutes of head way of the light rail vehicles.

The sensitivity analysis of the models has been analyzed by varying the pedestrians Volume, Light Rail Vehicle head time and for any increment of total approaching road vehicles up to 50% increments. Regression models have been developed from out puts of Sidra Software simulations.

$$Delay\ (Seconds) = \alpha * Pedestrians'Volume\ (No) + \beta * LRV_{Head}Time\ (Minutes) + \gamma(1 + \mathrm{P}) + C$$

Equation 4.2

Where, each Coefficients are presented in table 4-7.

Table 4-7: Delay Model for Yield Traffic Control at Level Crossings

$Delay\ (Seconds) = \alpha * Pedestrians'\ Volume\ (No) + \beta * LRV\_Head\ Time\ (Minutes) + \gamma(1 + \frac{P(\%)}{100}) + C$		
Coefficient	**Adey Ababa LC**	**Sebategna LC**
α	0.132	0.096
β	0.629	0.095
γ	629.62	294.179
c	-616.232	-230.737
R^2	0.976	0.992

Remark	• Pedestrians' Volume shall be only from one approach and assumed to be uniform from all approaches. • P=Increments (%) of approaching Volume of traffic to the LC and, 0 ≤ P ≤ 50%
Restriction	If the Pedestrian Volume from each direction exceeds 250, 250 should be used

During the simulation, if pedestrians' volume from each approach is above 250, the total delay on road vehicles remains the same with that of pedestrians' volume of 250. This is due to the fact that for Signalized intersection, any additional pedestrians' volume mora than the capacity of the green time for pedestrians will result additional delay in pedestrians' rather than on road vehicles.

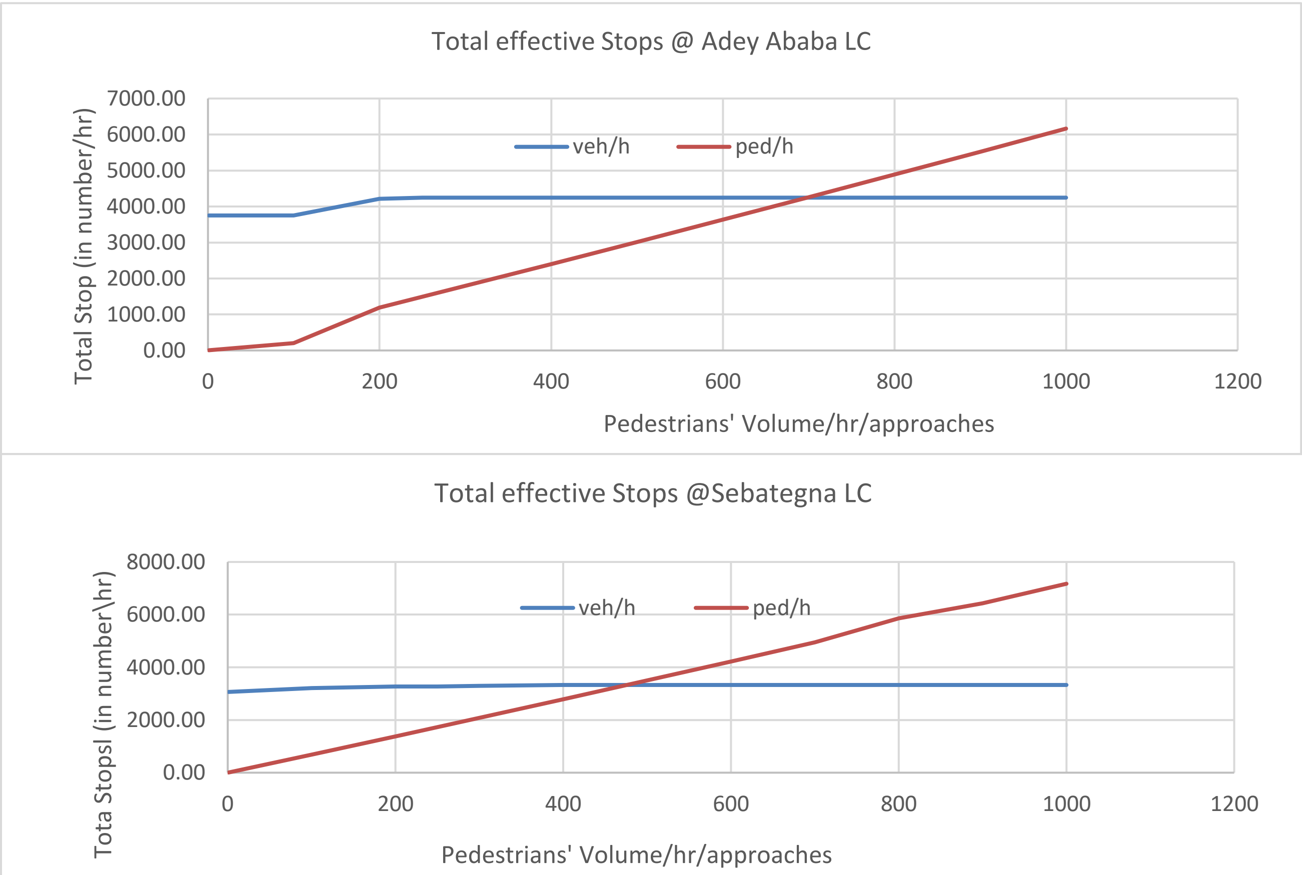


Figure 4-4: Effects of Pedestrians' Volume

4.4 Comparison of the Two Models

In the above sections (4.1, 4.2 and 4.3), the Level crossings were modelled for signaling and Yield Control of traffic mechanism. In order to check the better performance of the controlling system, comparison of the performance measures is very important. As a general, the signaling model results shows a better performance of the level crossing at both intersections whereas the burden of the Yield control on minor roads are distributed to all approaching lanes.

Table 4-8: Comparison of Performance Measures at 6 minutes of LRV frequency (Without Pedestrian scenario)

Adey Ababa LC					
S. No.	**Performance Measures**	**Unit**	**Yield sign**	**Signal**	**Better Performance**
1	Travel Speed (Average)	km/h	11.5	15.5	Signaling
2	Travel Time (Total)	veh-h/h	253.5	180.4	Signaling
3	Effective Intersection Capacity	veh/h	1650	2542	Signaling
4	Control Delay (Total)	veh-h/h	133.61	67.9	Signaling
5	Control Delay (Average)	Sec	169	80.7	Signaling
6	Control Delay (Worst Lane)	Sec	710.6	138.9	Signaling
7	Intersection Level of Service	LOS	F	F	The same
8	90% Back of Queue - Vehicles (Worst Lane)	Veh	90.9	30.7	Signaling
9	90% Back of Queue - Distance (Worst Lane)	M	639.1	215.0	Signaling
Sebategna LC					
1	Travel Speed (Average)	km/h	12.5	24.7	Signaling
2	Travel Time (Total)	veh-h/h	276	121.5	Signaling
3	Effective Intersection Capacity	veh/h	1896	2518	Signaling
4	Control Delay (Total)	veh-h/h	143.78	55.8	Signaling
5	Control Delay (Average)	sec	153.1	68.4	Signaling
6	Control Delay (Worst Lane)	sec	774.9	133.3	Signaling
7	Intersection Level of Service (LOS)	LOS	F	E	Signaling
8	90% Back of Queue - Vehicles (Worst Lane)	veh	78.8	28.4	Signaling
9	90% Back of Queue - Distance (Worst Lane)	M	589.7	248.4	Signaling

As shown in table 4-8 above, at both level crossing the level of service under the analyzed models are performing at LOS E & F even though the average of Control delay in signaling is much less than (about 20%) that of yield control system. For all selected performance measures, signaling shows superior performance for Vehicles whereas Yield control is in favor of pedestrians.

4.5 Comparison of the results against the Target LOS

As stated in the section 1.4, the LOS is targeted to be LOS D whereas the performance of the intersections is LOS F or E for both level crossings under the analyzed models which is different from the target. Hence, other models including modifying the intersections geometry shall be included and analyzed if they could perform at least at Level of Service D as presented in the next 4.7 section.

4.6 Other Optional Models

The models in section 4 shows that for both signaling and Yield control at the existing traffic and geometry conditions, the level crossings are performing at LOS F or E. on the other hand, one of the objectives of this paper is to set some modifications either on traffic movement or geometry modification so that both level crossings would perform at least at Level service of D. Since the Performance of these level crossings at the existing conditions do not answer the objective of the paper, other models are recommended and analyzed as presented in next sections.

Accordingly, the other optional models that could increase the performance of the intersections are:

1. Restricting Left turn and U-turn with/out intersection Geometry change;
2. Roundabout with Rail line passing through the Roundabout with traffic control of either Yield control or Signaling control;
3. Providing Pedestrians bridge; and
4. Grade Separation.

Having these possible options, the analysis should start with the 1st one and if it fails the second options will be proceeding until the last options (grade separation). For the selected level crossings, the 1st option will come up with the targeted performance as discussed in next sections.

4.6.1 Intersection Modification for Sebategna LC

4.6.1.1 Traffic Movement Management for Sebategna LC

This Level crossing is located in one of the busiest places in Addis Ababa. Modification of the geometry of the intersection would bring more congested and Conflict of interests with the business nearby the intersection. However, restricting and diverting some traffic movements through different approaches is selected as better proposal.

Accordingly, if the following traffic movement management is taken as measurement,

- U-Turn vehicles from all approaches are restricted, and Left Turns from North (Autobus Tera) approaches are restricted
- The Traffic movements from restricted direction will be distributed to different approaches with their best experiences. For example, Left and U turns from Autobus Tera would join one of the following:

C) Amanuel approach using Internal Collector roads and Left turn or Through from Amanuel,

D) Direct enter to Merkato through another route (Autobus Tera – Paisa Route),

E) Right turn from Autobus Tera and Join the Amanuel Approach from which Left turn and Through movements are possible

Figure 4-5 of the following shows the available optional routes for restricted traffic movements form North Approach (Autobus Tera) and left and U-turns from the same approach,

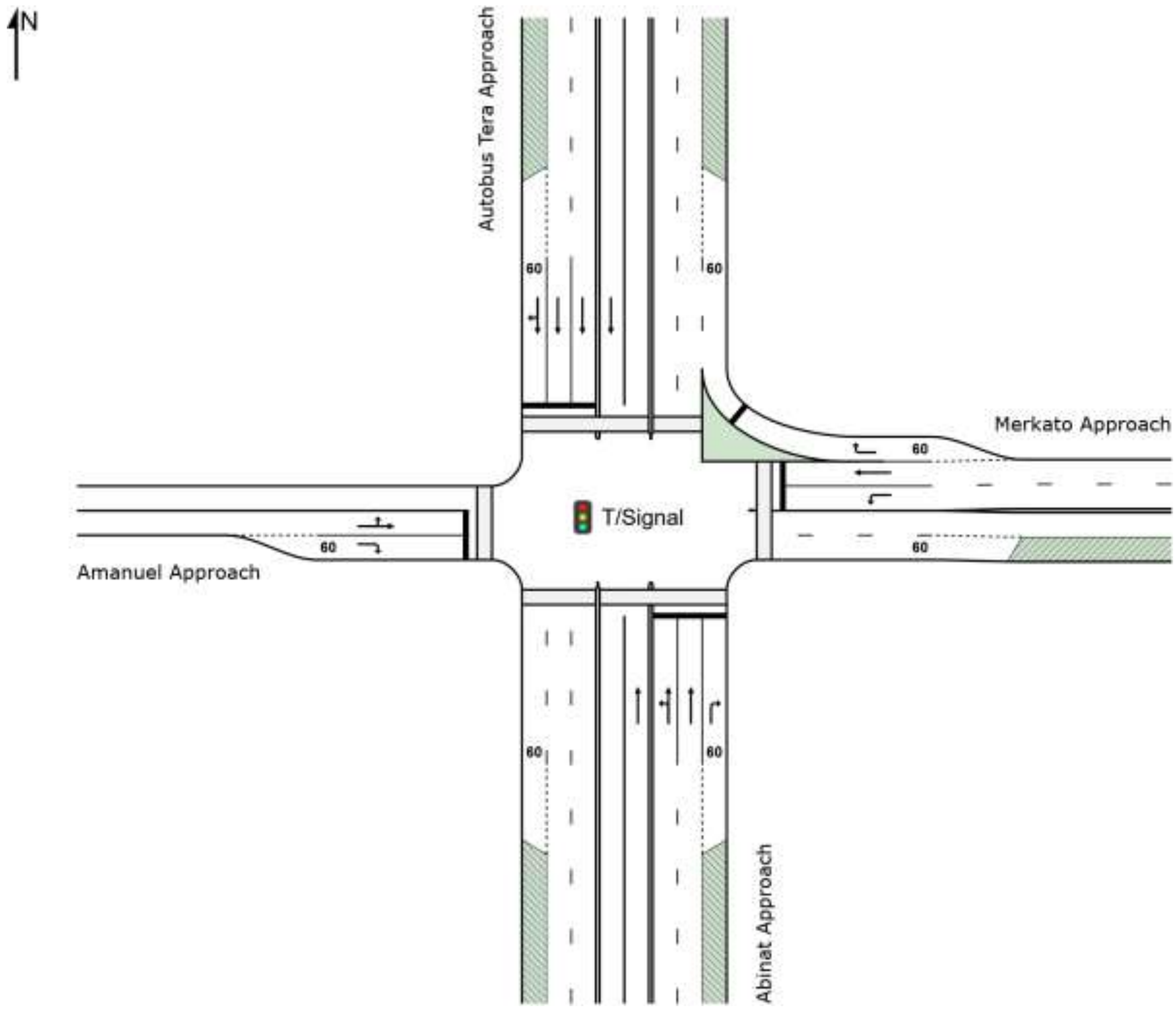


Figure 4-5: proposed Traffic Movement Management at Sebategna LC

Figure 4-6: Possible Rout options for Left and U-Turns from Autobus Tera Approaches

Similarly, the U-turns form the south (Abinat) Approaches could be distributed in such a way that fist Right Turn and join the Merkato approach from which Through and Left turn is possible.

Figure 4-7: Possible Route option for U-Turn Restricted from Abinat Approaches.

Unlike North south approaches, the East –West approaches are opened and possible to facilitate U-turn before the traffic inter to the intersection.

Figure 4-8: Proposed U-Turns from West- East Approaches

Incorporating these Traffic movement managements, the Level Crossing is modelled in Sidra Intersection 8 using the signal control for each movement. The pedestrians from each approach are also included for the analyses. Four phases have been used with Cycle length of 90 Seconds which has the highest performance as shown from Figure 4-9 below.

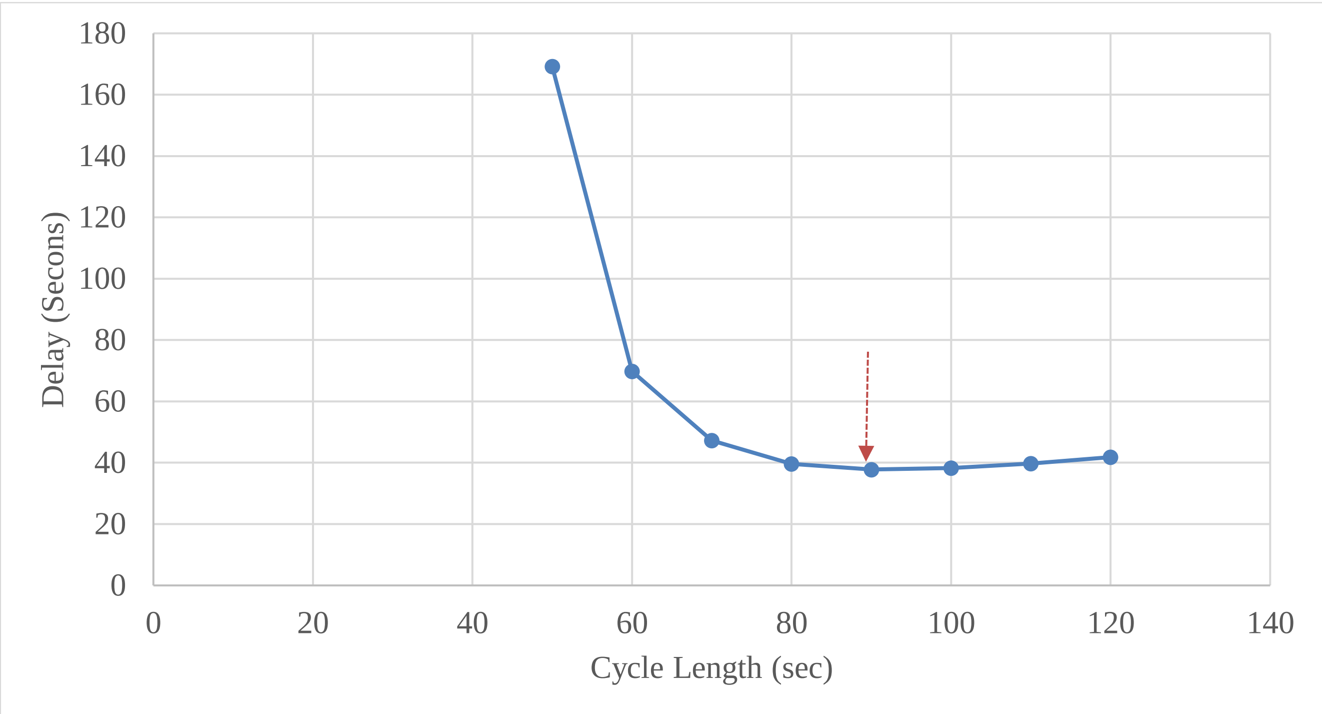


Figure 4-9: Cycle length Vs Delay at Sebategna LC (Modification case)

Timing in each phase has been made on the Distribution of traffic on each approaching lanes and movements group as shown in figure 4-10 below.

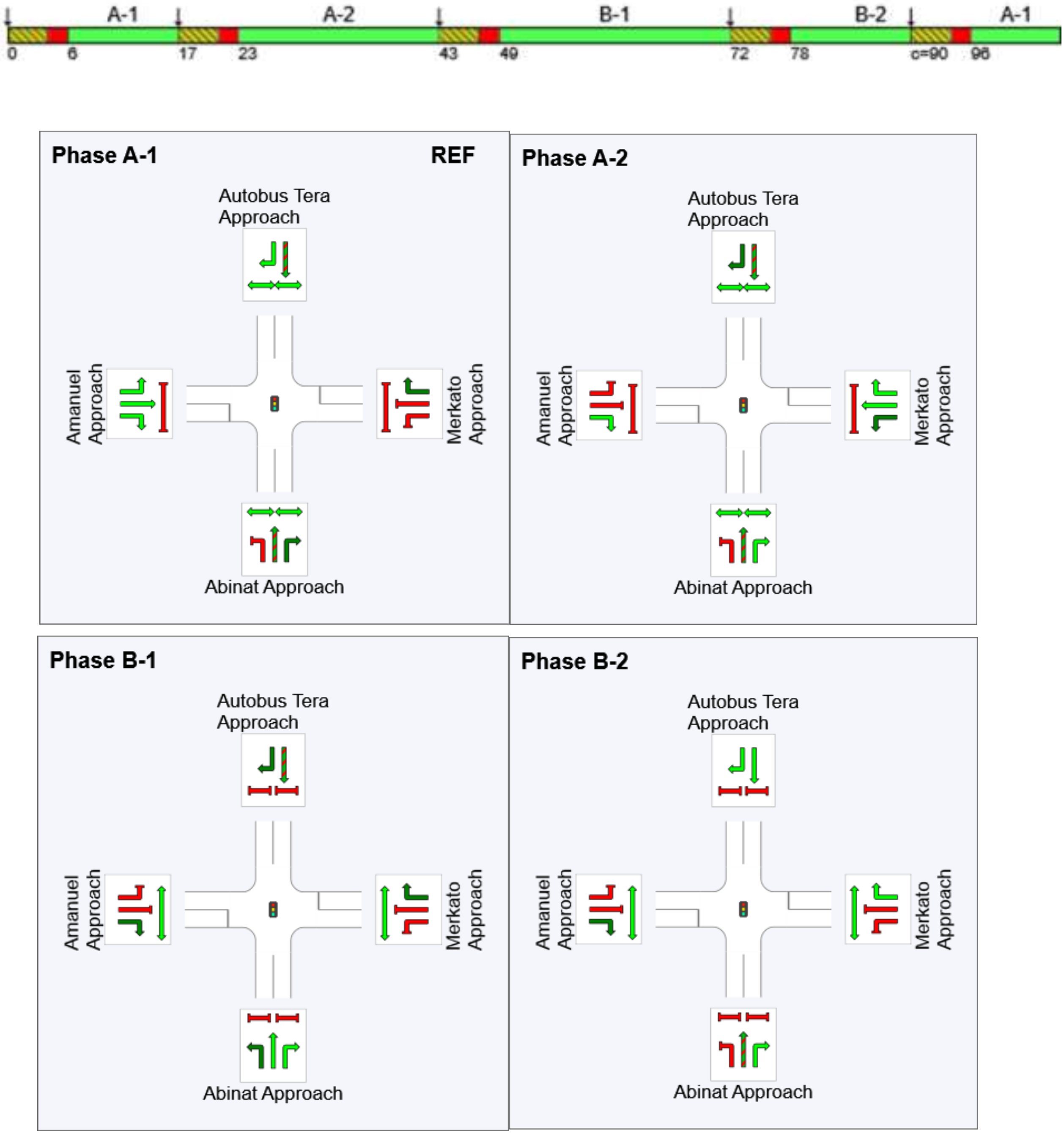


Figure 4-10: Phase sequence for Sebategna LC (Modification case)

Performance Evaluation for Sebategna LC after the above proposed traffic movement managements have been made shows that the Intersection will perform at Level of Service of D (37.8 seconds of Delay) for Vehicle and LOS E (43 seconds of Delay) for Pedestrians. However, the most congested Lane will be the West approaches with Left turn and Through lane with 84 seconds of delay. This longest delay shown on the west approaches is due to the fact that most of vehicles restricted U and Left Turns from North approaches will use the west approaches through different routes and join the Corresponding destinations of U and Left turns from the North approaches.

Table 4-9: *Sebategna LC performance (Modification case)*

Performance Parameters	Approaches				Intersection
	South	East	North	West	
Delay (Sec.)	30.4	45.1	40.1	48.3	37.8
Queue length (m)	72	89	38	49	89
LOS	C	D	D	D	D

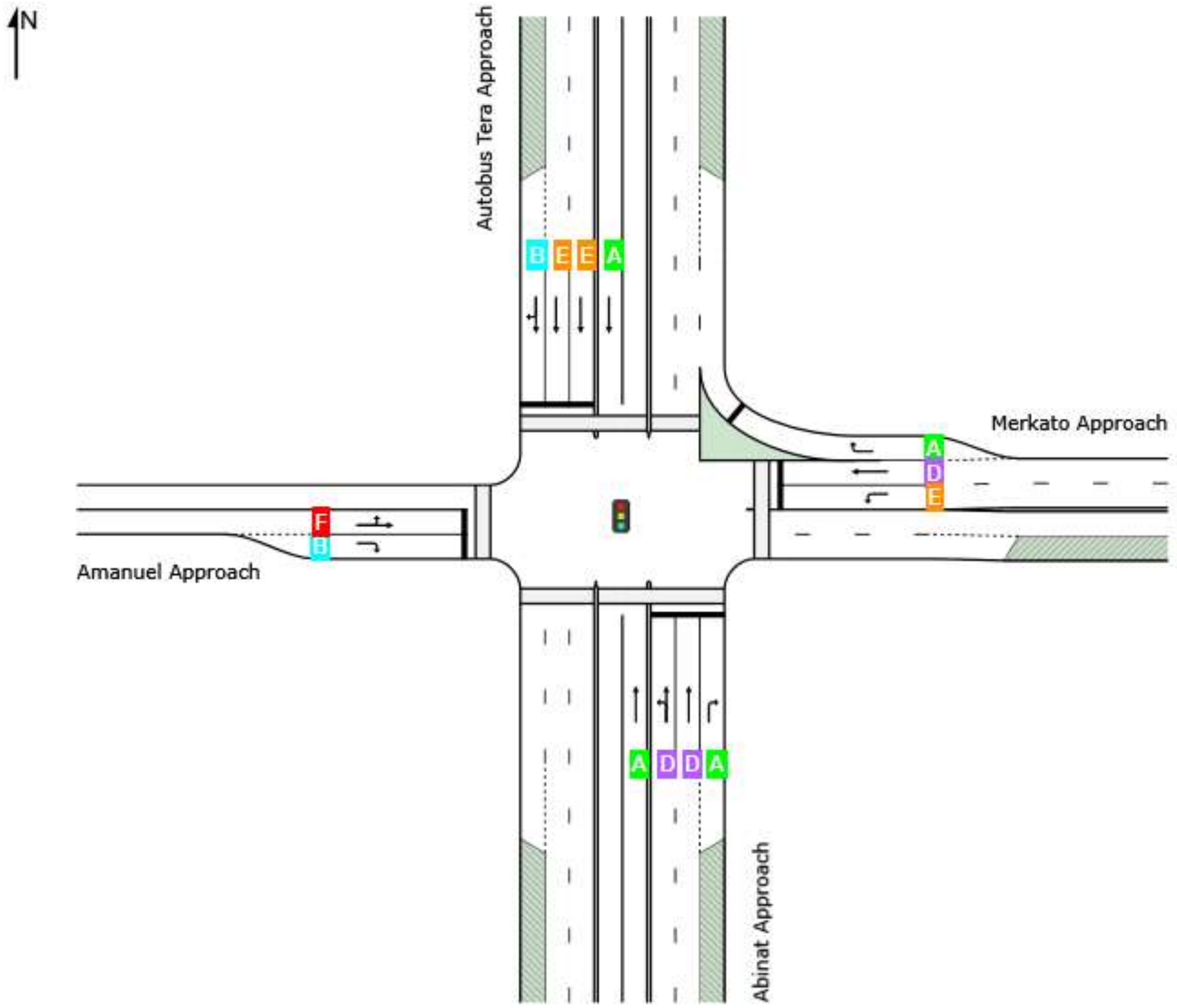


Figure 4-11: Sebategna LC performance (Modification case)

According to the analysis output, the longest queue (89m) will be on the East (Merkato approach) due to additional vehicles joined the left turns from the restricted U-turns of South (Abinat) approach as the optional routes. On the other hands, in the above model, allowing U-turns from any approaches will lead the intersection's performance to be at lower LOS (Less than LOS D).

The details of performance measures is given in table 4-10 for both Pedestrian and Vehicles modes.

Table 4-10: Performance measures for Modified Sebategna LC

Performance Measure	**Vehicles**		**Pedestrians**	
	Measures	**Unit**	**Measures**	**Unit**
Travel Speed (Average)	21	km/h	1.8	km/h
Travel Time (Total)	124	veh-h/h	205.2	ped-h/h
Percent Heavy Vehicles (Demand)	11.2	%		
Effective Intersection Capacity	2603	veh/h		
Control Delay (Total)	26.86	veh-h/h	126.47	ped-h/h
Control Delay (Average)	37.8	sec	43	sec
Control Delay (Worst Lane)	84.3	sec		
Control Delay (Worst Movement)	84.3	sec	43.8	sec
Intersection Level of Service (LOS)	LOS D		LOS E	

4.6.2 Intersection Modification for Adey Ababa LC

As observed in section 4.1, 4.2 and 4.2, the performance of Adey Ababa LC is under LOS F which is not desirable. As stated in the objective of this paper (section 1.4), to find the solution so that the LC would perform at least at LOS D. some of the possible options for the modification of level crossings were stated under section 4.5 and this section is the sup topics which describe one of the selected options for Adey Ababa LC.

Two modifications to the LC are proposed: Traffic Movement Management and Intersection Geometry Modification.

4.6.2.1 Traffic Movement management for Adey Ababa LC

- Restricting left turns Movements from West (Beretsige) and East (Addis Sefer) approaches,
- Restricting U-turn movements from all approaches,

These restricting Traffic movements can be diverted to different directions. The following are some of the optional possible route assignments for those restricted traffics.

U turns from the south approach can join the West approach with those left turn from the south approaches followed by either U-turn and right turn from the West approach or use the other optional available routes for their destination as shown in figure 4-12 below.

Figure 4-12: Possible routes for restricted Traffic (U-turn from East and South approaches and Left Turns from East approaches)

Similarly, as shown in figure 4-12 above, those vehicles with left and U-turn from the East approach can pass to the west approach with through traffic of east approach and after which can join these comes from east approach as Right turn movements.

Following the same fashion for the north approaches and West approaches the traffic movements can be managed. That traffic Left turn from the West approach will as through vehicle to the east and then U-turn from east and then right turn to their original destination.

The U turn from the North approach can be converted to left turn to join the east approach. Finally, these vehicles U-turn and then Right turn to their wishing Destination. Figure 4-13 below shows the possible routes. The U-Turns from East and West approaches can be managed outside of the Intersection as the road.

Figure 4-13: Possible routes for restricted Traffic (U-turn from west and North approaches and Left Turns from West approaches)

The flow of traffic will not remain the same if these traffic movement management is done. Hence, the traffic flow for the modification shall be re arranged for the analysis as shown in table.

Table 4-11: traffic Flow (Original Vs Modification, Adey Ababa LC)

Approaches	***Original Movements***				***Modification case***			
	RT	**THR**	**LT**	**U**	**RT**	**THR**	**LT**	**U-Turn**
South	49	428	70	49	49	428	119	0
East	94	135	89	0	684	135	0	0
North	121	621	186	251	121	621	437	0
West	28	118	359	0	28	477	0	0

In addition to the traffic volume, the pedestrian volume is included to the analysis since the modified intersections shall accommodate them.

4.6.2.2 Intersection Geometry Modification for Adey Ababa LC

Following the Traffic Flow direction diversions, the intersection was modelled in Sidra Intersection 8 without any geometry change with different allowable Cycle lengths. Any of the tried models did not shows the Intersection will perform at the minimum of LOS D. hence, only traffic movement direction change cannot be remedial measurements to increase the LOS to D. Considering the approaching traffic volume, the north approach needs additional short lane (By pass) for Right turns and the inner lane to be used for left turns only (Exclusive left turn lane). Similarly, those restricted U-turn from the south and Left turn form the East approaches would join the west approaches with single lane in each direction. This will result in high delay (the result shows the same) and additional short lane for the right turn shows a better performance of the Intersection. Figure 4-14 below shows the geometry of the proposed level crossing.

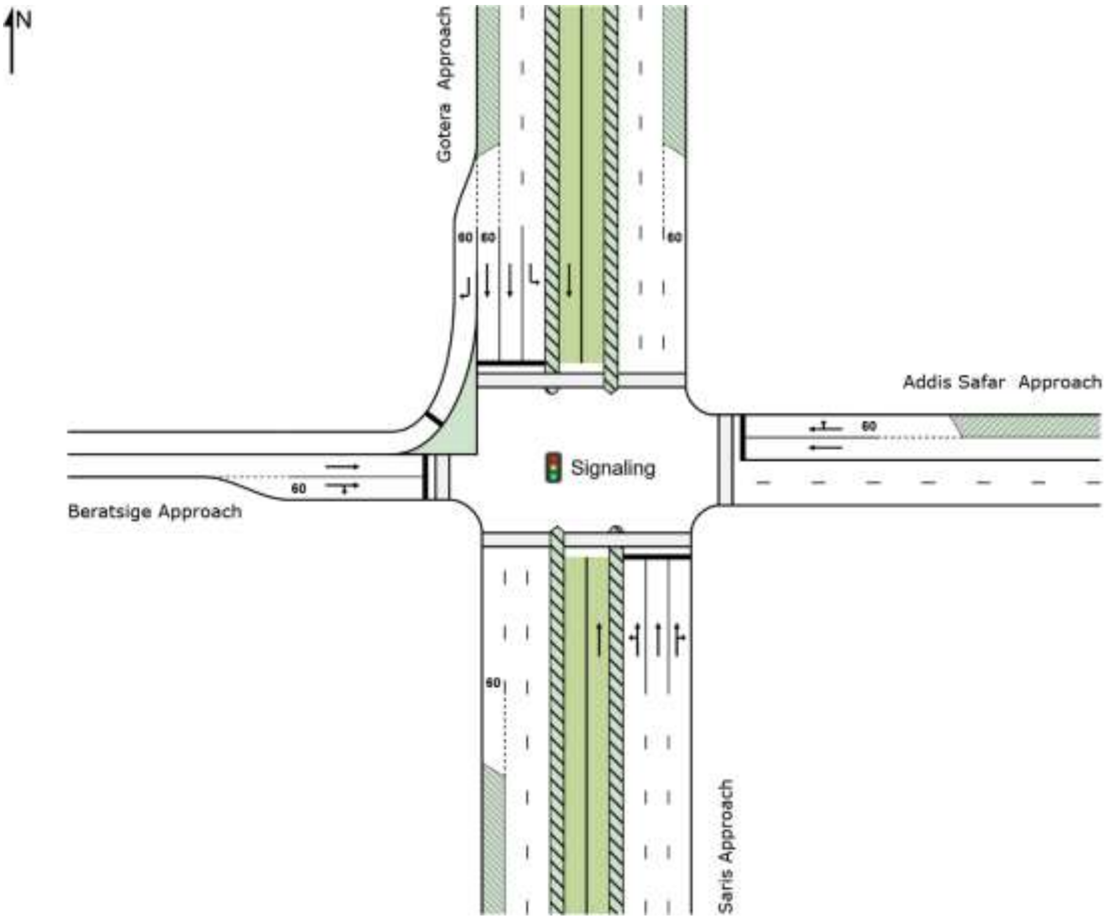


Figure 4-14: Intersection Geometry Modification (Adey Ababa), Sidra Intersection 8 out put

4.6.2.3 Intersection Controlling for Adey Ababa

As discussed in Section 3.5.2, the traffic condition at Adey Ababa Level Crossing warranty for the Signal. Thus, signaling design has been made with two Phases and Cycle Length has been selected with the highest performance (lowest average queue length and lowest delay) by iteration in the Sidra Intersection 8 software. Accordingly, 90 second of Cycle length has been selected. The Traffic in each phase movement is also shown in figure 4-15 below and the split of Time for each phase as shown in table 4-12.

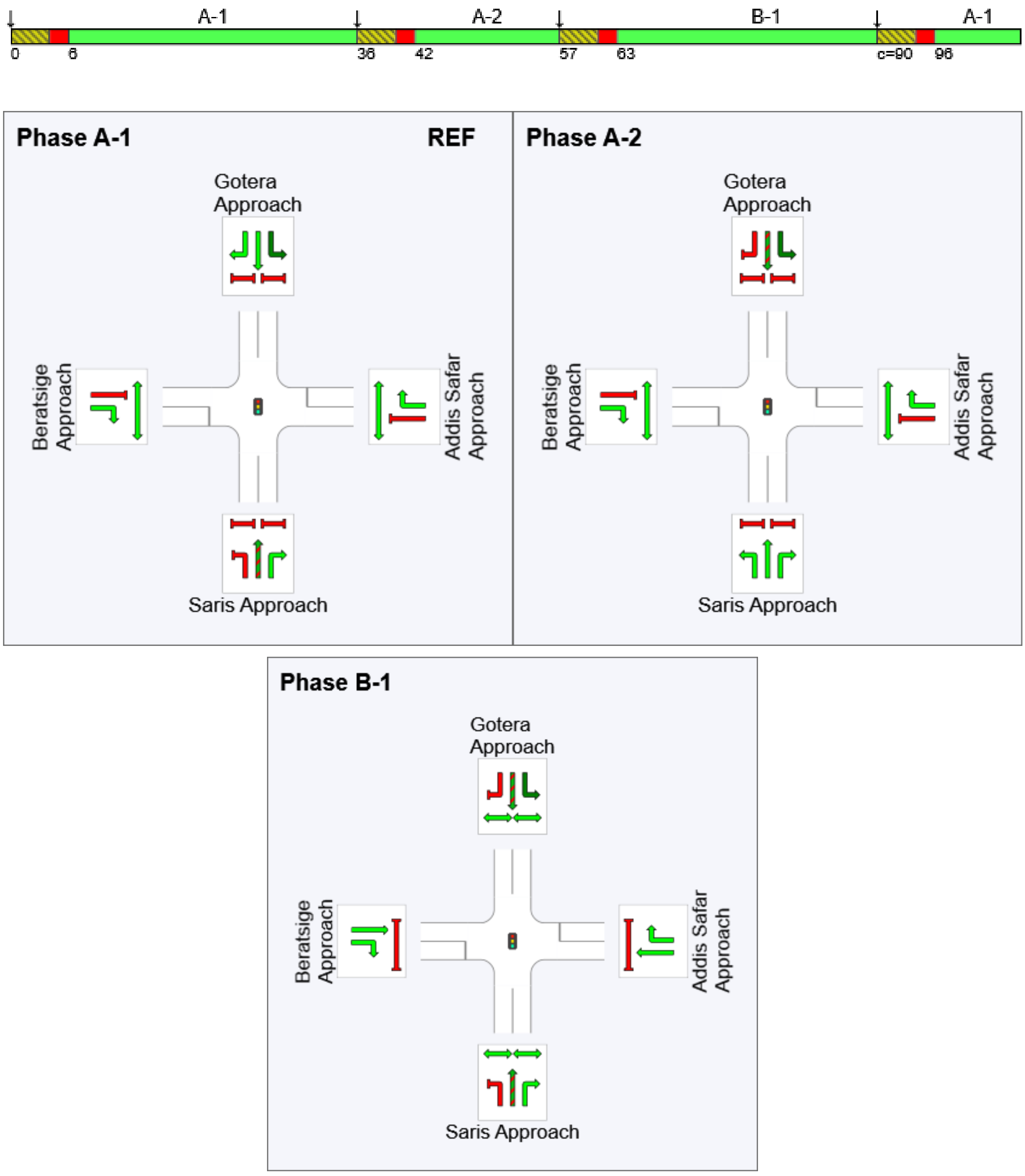


Figure 4-15: Traffic Movements in each phase (Adey Ababa LC), Sidra Intersection 8 out put

The detail of Timing in each phase is shown in table 4-12 below.

Table 4-12: Phase parameters (Adey Ababa LC)

Phase	**A-1**	**A-2**	**B-1**
Phase Change Time (sec)	0	41	64
Yellow Red	4sec	4sec	4sec
All read	2sec	2sec	2sec
Green Time (sec)	30	15	27
Phase Time (sec)	36	21	33
Phase Split	40%	23%	37%

4.6.2.4 Model Results of Adey Ababa Modified LC

The level Crossing with this modification (Traffic Movement Management and Intersection Geometry modification) was modelled in SIDRA Intersection 8 and analyzed for the proposed phases and Cycle Length. According to the result the Level crossing will perform at LOS of D (delay 40.2 seconds) if these modifications are incorporated. The performance of each approaching lane and Intersection is summarized in table 4-13 for the intersection and figure 4-16 for approaching Lane performance.

Table 4-13: Modified Adey Ababa LC performance

	Approaches				Intersection
	South	East	North	West	
Delay (Control)	90.9	6.9	43.3	27.5	40.2
Queue Average (m)	75	33	158	45	158
LOS	F	A	D	C	D

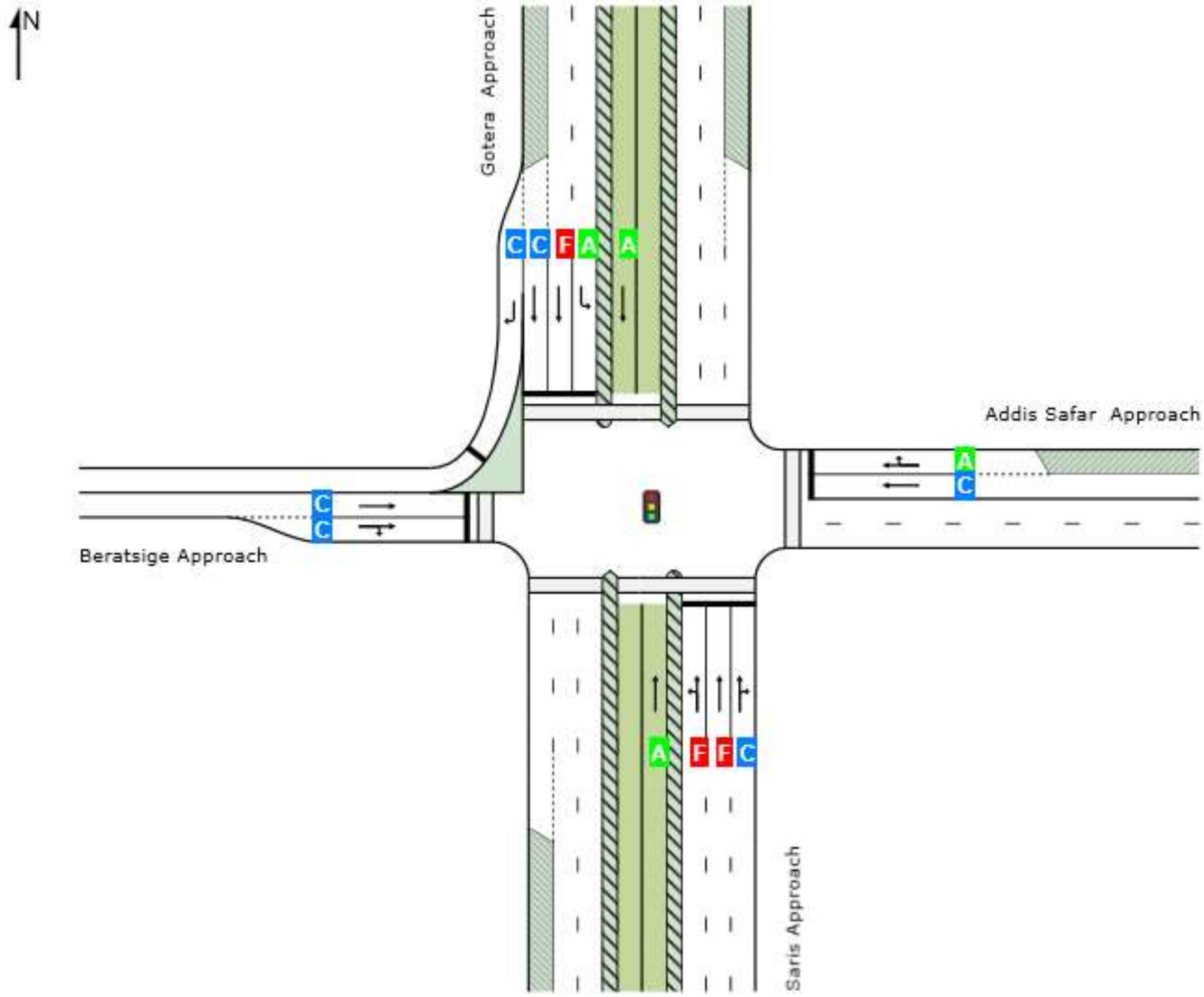


Figure 4-16: Lane and Intersection LOS (Modified Adey Ababa LC)

The West and East Approaches will perform at better LOS than the others as U and Left Turns are restricted from these approaches, whereas the South approaches will perform at lowest, LOS F (90.9 seconds of delay). The Northern approach with Highest traffic volume shows a better performance than the south approaches due to the fact the additional lane for Right turns is added as modifications. Generally, with this traffic movement modification and Intersection geometry modification, the Intersection will perform at LOS D (40.2 Seconds of delay).

The performance measures for each measure of Pedestrians and Vehicle requirement are Given in table 4-14 below.

Table 4-14: Performance measures of Modified Adey Ababa LC

Performance Measure	Vehicles		Pedestrians	
	Measures	Unit	Measures	Unit
Travel Speed (Average)	19.2	km/h	1.8	km/h
Travel Time (Total)	173.9	veh-h/h	244.3	ped-h/h
Percent Heavy Vehicles (Demand)	4.3	%		
Effective Intersection Capacity	3245	veh/h		
Control Delay (Total)	36.46	veh-h/h	150.77	ped-h/h
Control Delay (Average)	40.2	sec	43	Sec
Control Delay (Worst Lane)	107	sec		
Control Delay (Worst Movement)	106.7	sec	43	Sec
Intersection Level of Service (LOS)	LOS D		LOS E	

Allowing U Turns from any of the approaches and Left Turns from the Easting and Northing approaches without changes the Intersection geometry will lead to the lower performance. It is observed also, if the additional lane recommended to the north approach is omitted the Intersection will perform at LOS E (58.9 seconds/vehicle of delay) whereas the North approaches will perform at LOS D (86.3 seconds/vehicle of delay).

As all level crossings under this modification model is Controlled merely by the signal signs, the effect of Pedestrians' Volume is not such considerable on the Total delay of road Vehicles. Any Additional pedestrians greater than the given green time will lead to additional delay of Pedestrians rather than that of Roads' vehicles.

4.7 Validation of the Software and Models

In addition to the evidences cited in the literature review for the compatibility of Sidra Intersection plus 8, before selecting the software, a model for a known Traffic data have been modelled and the output result were compared with theoretical calculation of different delay models. The model was compared for intersection with 4 approaches (Figure 4-17).

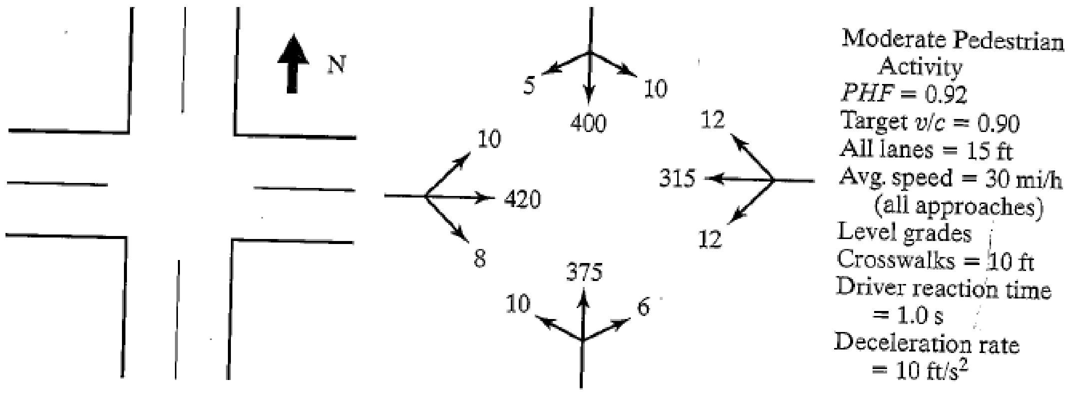
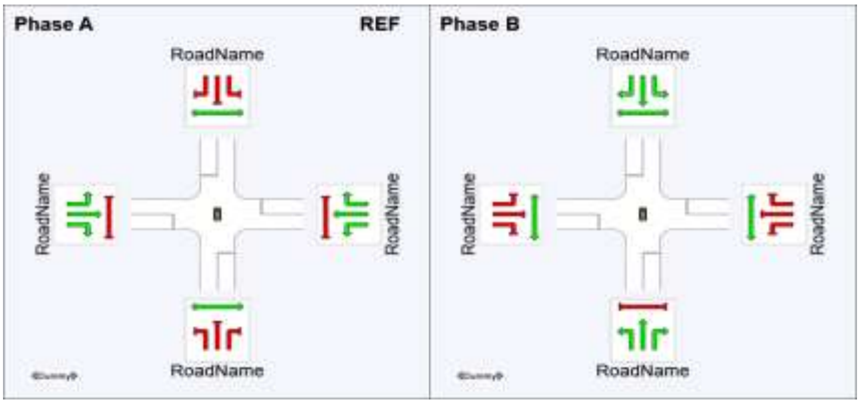


Figure 4-17: Data for Model validation ((Roses 2004)

The Intersection with Two phases of C=36seconds and green time as indicated in figure 4-178 below.



Phase	A	B
Phase Change Time (sec)	0	19
Green Time (sec)	13	12
Phase Split	52%	48%

Figure 4-18: Model for Software validation

the same parameters have been also used for theoretical Computation of delay (parameter selected for Performance measure) and the result have been presented as below in 4-19.

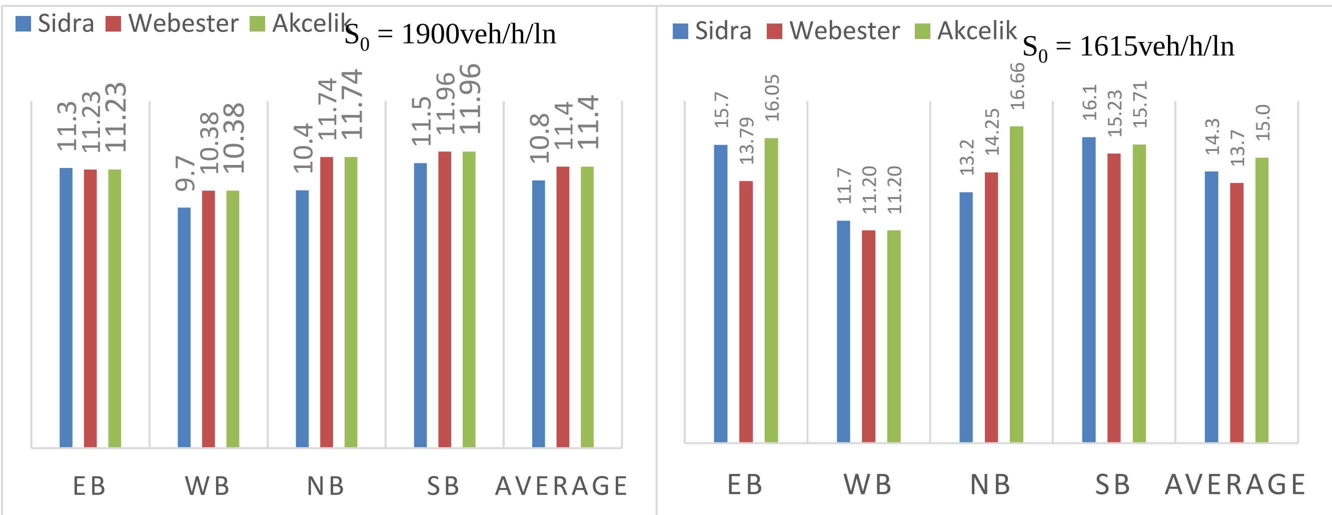


Figure 4-19: Sidra vs Theoretical models out put

As it can be seen from the output, almost Sidra output is the same with the theoretical models of Webster delay model and Akcelick Model with about 4.5% of difference at both base saturations.

The validation of models for Sebategna and Adey Ababa LC have been modelled in similar way and selected for Validation. As most of the available theoretical models are valid only for the Vehicular cases (no theoretical models for inclusive of LRV vehicles at Level crossings) other than models in software for simulations, the case of Without LRT was taken as a base model and validated then which is extended to use for other scenarios.

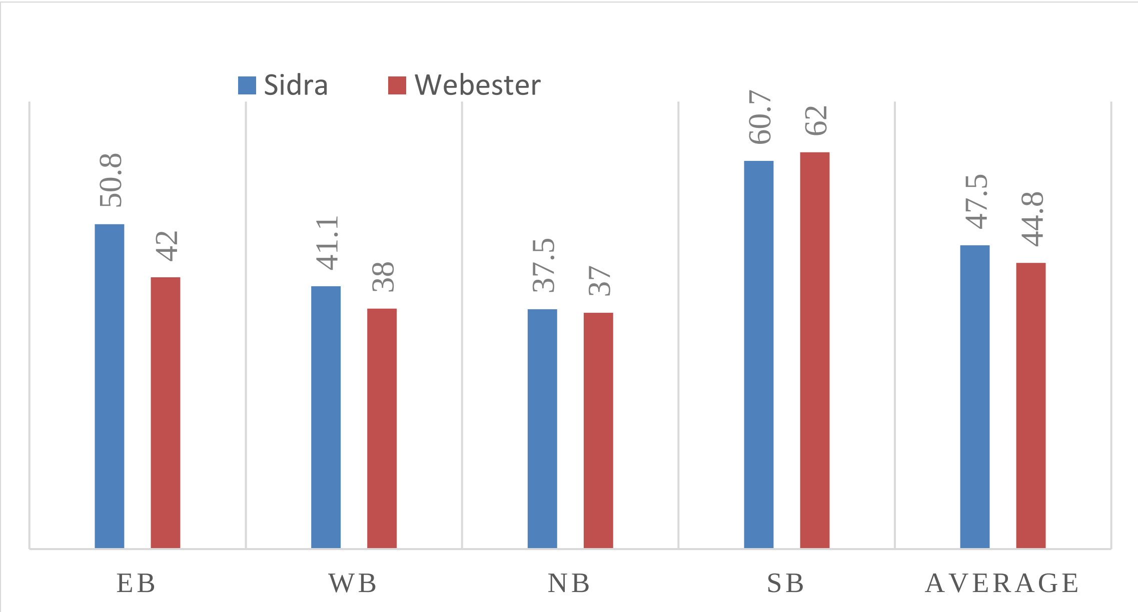


Figure 4-20: Model Validation (Adey Ababa LC, Without LRT, Signal Model)

Almost both models give the same result by which Sidra Outputs slightly greater than the Theoretical calculation (6% difference on average). Thus, it can be summarized that sidra intersection software can be used for analysis of signalized intersection.

CHAPTER 5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Addis Ababa Light rail transportation is one of the public transports owned by the government. This light rail lines are running from south to north and from east to west in between roads from both sides (left and right). At the intersections, specially, around the Level crossings (at grade intersection) the congestion of the traffic vehicles is experienced since the Light rail is start operation. Some of the researchers were interested in conducting the researches with different titles on Addis Ababa light rail transportation (AALRT) starting from its construction stage.

Currently, AALRT is serving with head time of average at about 15 minutes. At this service time, the congestion at the level crossings, especially at Adey Ababa and Sebategna calls attention of that what would happen if the AALRT starts service at the design head time of minutes. Hence, in order to get the answer for such like question relating to the frequency of Light rail vehicle (LRV) and its effects on the adjacent roads at the level crossings, the performance evaluation of level crossing is one of the methods for the enquiry.

At the selected level crossings, Adey Ababa and Sebategna level crossings, the mechanism of traffic control is not defined as it is subjective to traffic police who is in power to control the movement of the traffic flow at the intersection. Even though, the mechanism seams with the Yield control (Give away) sometimes it is similar to all-way stop.

The level crossings were modelled in three types; Yield sigh control, signaling with existing traffic and geometry condition and Signaling with some modification of traffic's movements and geometry changes.

Yield sign model of level crossings, at existing traffic and geometry conditions, shows that both pedestrians and Light rail vehicles has considerable effects on the Level crossing performance. Linear regression model from varying Pedestrians' Volume and Light Rail vehicle's head time shows that increasing both the LRV frequency and Pedestrians' Volume results in increasing of total delay at level crossing (reduction of Level crossing performance).

Analysis from modelling of LC signalizing with existing traffic and Geometric Conditions shows that the presence of light rail vehicle at the level crossing will not lead to excessive delays on the adjacent traffic movements which is confidential for AALRT to plan their operation to the designed head time (6minutes). Moreover, the presence of LRV will favor the reduction of delays at the level crossing by providing additional free movement time for parallel vehicles moving with LRT line.

Additionally, signaling these level crossings will lead to better performance (lowers the average control delay) of level crossings. In signalizing, the effect of Pedestrians' Volume is not severe as of Yield Sign control. only about 18.4sec/veh and 23.4 sec/veh of additional delay were observed for Adey Ababa and Sebategna LCs due to presence of pedestrians in the signaling model of existing traffic and intersection geometry condition. Any additional of pedestrian's' volume than the capacity of allowed by the signal, will result in additional delay of Pedestrians' itself than adding of delay to Road's vehicles. It is observed that no additional delay on roads' vehicle due to pedestrians greater than 250 from each approach.

The Level crossings has been checked if they could perform at least at LOS D if minor geometry modification is done and/ or Traffic movement management at the intersection is re arranged. The result from Sidra Intersection 8 analysis shows:

At Adey Ababa level crossing: - If geometry modification for Right turning and traffic movement for left and U-turns are restricted and merged with right turn and then left turn from the Right-Hand Minor approaching road, particularly for North and South approach, the level crossing will perform at LOS D (average delay of 40.2 seconds/vehicles) With pedestrian performance LOS E (43 seconds/pedestrians delay).

Similarly, Sebategna Level crossing, if restriction of U-turns from all approaches and Left turns from North approaches are done with geometry modification (additional short lane from east approach to be used by Right Turn traffics), the level crossing will perform at LOS D (average delay of 37.8 seconds/vehicles) With pedestrian performance LOS E (43.8 seconds/pedestrians delay).

The findings of this paper have been checked with related recent papers at the same locations (at Adey Ababa and Sebategna Level crossings). Two of Recent Papers, (Robel Desta, Daric Tesfaye and Ja´nos To´th 2021) and (Demelash Berhane Mengesha et al.: 2019) have been selected for the relationship of the results or findings.

According to (Robel Desta, Daric Tesfaye and Ja´nos To´th 2021), vehicular delay at Sebategna level crossings would increase if there is no light rail vehicle and the installation of traffic signal would not improve the performance of the installation. On the other hand, in this paper it is found that, the installation of Traffic signal would increase the performance of the level crossings as it will allow pattern vehicular and pedestrian movements and the effect of presence of Light rail vehicles is less considerable in the intersection's delay.

According to (Demelash Berhane Mengesha et al.: 2019), the existence of the pedestrian adds greater severity on the vehicular delay than the presence of light rail vehicles. Similar conclusion has been obtained from this paper, even though there is difference in modelling methodology, Traffic Volume values, performance measures values, and analyzing methods. However, the effect of Pedestrians is different from model to model. Signalizing the level crossings will lowers the effect of total roads' vehicle delay.

5.2 Recommendation

General Recommendations:

- Level crossing shall be furniture with important intersection markings.
- Parking near by the level crossings (approaching and at the exit) from all directions shall be protected so that the performance of the Level crossings will be increased.
- Road side Vender which reduces the movement of the pedestrians and blocks the walkway shall be protected from the side roads which in turn increase the space for the pedestrians.
- The level crossing shall be converted to signal control mechanism so that delay will be minimized than the Yield control methods.
- Traffic managements at level crossings including U-turn restriction from all approaches of both level crossings, left turn from East and West approach of Adey Ababa level crossings; and left turn restrict from North approach of Sebategna Level crossing followed by facilitating the routes for these restricted traffic movement directions shall be facilitated. A
- Additional short lane for Right Turns forms East approaches of Sebategna Level crossing and from north approach of Adey Ababa, since the restricted U-turns and Left Turns will be diverted to right Turn or to other direction so that the performance will increase.
- Facilitating the pedestrians' crossings: at both level crossings, as there is no pedestrian crossings for long sections, the pedestrians are concentrated at these level crossings. Distributing these pedestrians to different places by providing pedestrians' crossing facilities (e.g., Pedestrians Bridges) at some acceptable interval so that the performance of Level Crossings will increase as their effect is sounder than the presence of Light Rail Vehicle.

- Addis Ababa Light Rail Transit can increase its operation frequency to the designed head way (6 minutes of frequency) as the additional delay due to rail vehicle is less than 6 seconds/Vehicles.

For Researchers and Designers:

- The research can be extended by designing the Signal with different signal timing and different software, and different traffic Composition and intersections geometry modifications.

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ANNEXIS

1. Traffic Volume Data

1.1. Sebategna LC Traffic Volume

1.1.1. Sebategna LC Traffic Volume at Morning

F) Abinat Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers		
Direction: Abinat to Merkato (Right Turn)												
2:00 - 2:15	27	2	2	5	9	44	-	-	4	1	-	92
2:15 - 2:30	26	1	3	2	18	37	-	-	2	-	-	89
2:30 - 2:45	23	6	2	4	17	44	-	-	3	1	1	100
2:45 - 3:00	27	4	1	4	10	47	-	-	2	-	-	93
Direction: Abinat to Autobus Tera (Through)												
2:00 - 2:15	43	7	2	5	2	50	-	-	2	-	1	110
2:15 - 2:30	54	7	5	8	2	47	-	1	1	-	-	122
2:30 - 2:45	54	10	4	10	5	23	-	1	1	1	-	107
2:45 - 3:00	54	8	7	3	7	62	-	1	1	-	-	141
Direction: Abinat to Amanuel (Left Turn)												
2:00 - 2:15	8	2	2	5	2	2	-	-	2	-	-	20
2:15 - 2:30	8	3	2	3	1	11	-	-	2	-	1	28
2:30 - 2:45	14	4	1	4	3	8	-	-	2	-	-	34
2:45 - 3:00	16	2	2	4	1	8	-	-	1	-	-	34
Direction: Abinat to Abinat (U-Turn)												
2:00 - 2:15	12	3	1	2	1	3	-	-	2	-	-	22
2:15 - 2:30	10	2	1	3	1	15	-	-	4	1	-	34
2:30 - 2:45	15	3	1	1	1	15	-	-	2	-	-	37
2:45 - 3:00	12	5	1	3	2	14	-	-	3	-	-	

G) Merkato Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: **Merkato to Autobus Tera (Right Turn)**												
2:00 - 2:15	9	2	6	9	5	5	-	-	2	-	-	36
2:15 - 2:30	12	3	5	9	3	5	-	-	3	1	-	39
2:30 - 2:45	15	2	4	6	2	6	-	-	2	-	-	35
2:45 - 3:00	13	2	4	6	1	7	-	-	2	-	-	34
Direction: **Merkato to Amanuel (Through)**												
2:00 - 2:15	10	2	3	2	2	10	-	-	1	-	-	29
2:15 - 2:30	6	2	2	3	2	9	-	-	2	-	1	26
2:30 - 2:45	9	2	2	2	1	10	-	-	1	1	-	26
2:45 - 3:00	8	2	3	3	2	8	-	-	1	-	-	25
Direction: **Merkato to Abinat (Left Turn)**												
2:00 - 2:15	19	3	7	5	9	30	-	-	2	-	-	74
2:15 - 2:30	17	3	6	5	8	27	-	1	3	1	-	67
2:30 - 2:45	18	5	3	4	8	33	-	-	5	-	1	75
2:45 - 3:00	14	3	3	4	11	34	-	1	3	-	-	71
Direction: **Merkato to Merkato (U-Turn)**												
2:00 - 2:15	1	1	1	1	1	1	-	-	1	-	-	5
2:15 - 2:30	1	1	1	1	-	3	-	-	1	-	-	6
2:30 - 2:45	2	1	1	1	1	2	-	-	2	-	-	7
2:45 - 3:00	1	1	-	2	-	2	-	-	1	-	-	

H) Amanuel Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: **Amanuel to Abinat (Right Turn)**												
2:00 - 2:15	10	2	2	2	1	9	-	-	2	-	-	27
2:15 - 2:30	10	3	1	2	1	6	-	-	2	-	-	22
2:30 - 2:45	10	4	1	2	-	6	-	-	1	-	-	22
2:45 - 3:00	6	2	1	2	-	4	-	-	1	-	-	16
Direction: **Amanuel to Merkato (Through)**												
2:00 - 2:15	15	1	1	6	1	23	-	-	1	-	-	47
2:15 - 2:30	20	1	1	2	3	20	-	-	2	-	-	48
2:30 - 2:45	9	1	1	2	1	14	-	-	1	-	-	29
2:45 - 3:00	8	1	1	4	-	13	-	-	1	-	-	26
Direction: **Amanuel to Autobus Tera (Left Turn)**												
2:00 - 2:15	14	1	1	2	1	8	-	-	2	-	-	28
2:15 - 2:30	21	1	2	1	1	14	-	-	1	-	-	39
2:30 - 2:45	12	1	1	1	1	9	-	-	1	-	-	26
2:45 - 3:00	10	1	1	1	1	13	-	-	1	-	-	27
Direction: **Amanuel to Amanuel (U-Turn)**												
2:00 - 2:15	-	1	-	-	-	-	-	-	-	-	-	1
2:15 - 2:30	1	-	-	1	-	1	-	-	1	-	-	2
2:30 - 2:45	1	-	-	-	-	-	-	-	-	-	-	1
2:45 - 3:00	-	-	-	-	-	1	-	-	-	-	-	1

I) Autobus Tera Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: **Autobus Tera to Amanuel (Right Turn)**												
2:00 - 2:15	6	2	2	2	2	6	-	-	2	-	-	21
2:15 - 2:30	2	1	1	2	1	3	-	-	1	-	1	11
2:30 - 2:45	4	2	1	2	2	5	-	-	2	-	-	17
2:45 - 3:00	6	3	3	1	1	4	-	-	2	-	-	18
Direction: **Autobus Tera to Abinat (Through)**												
2:00 - 2:15	29	7	5	5	8	30	-	-	1	-	-	84
2:15 - 2:30	21	6	3	10	5	34	-	1	1	-	1	80
2:30 - 2:45	24	6	5	7	6	31	-	-	3	-	-	81
2:45 - 3:00	37	10	6	5	3	33	-	1	2	-	-	95
Direction: **Autobus Tera to Merkato (Left Turn)**												
2:00 - 2:15	18	2	2	3	2	10	-	-	1	-	-	37
2:15 - 2:30	19	5	4	4	2	18	-	-	2	-	-	52
2:30 - 2:45	17	2	5	2	3	8	-	-	1	-	-	37
2:45 - 3:00	15	1	2	6	3	20	-	-	3	-	-	48
Direction: **Autobus Tera to Autobus Tera (U-Turn)**												
2:00 - 2:15	7	2	2	2	2	13	-	-	2	-	-	28
2:15 - 2:30	8	1	1	1	1	8	-	-	2	-	-	21
2:30 - 2:45	5	1	2	2	1	5	-	-	1	-	-	16
2:45 - 3:00	5	2	1	2	2	7	-	-	3	-	-	19

1.1.2. Sebategna LC Traffic Volume at Mid-day

A) Abinat Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Abinat to Merkato (Right Turn)												
6:00 - 6:15	15	7	5	8	9	29	-	-	2	-	-	73
6 :15 - 6:30	18	6	5	6	13	24	-	-	2	1	-	72
6 :00 - 6:45	12	9	2	3	6	23	-	-	2	-	-	54
6 :45 - 7:00	13	3	2	7	6	16	-	-	1	-	-	47
Direction: Abinat to Autobus Tera (Through)												
6:00 - 6:15	37	11	8	11	4	46	-	1	4	-	-	121
6 :15 - 6:30	42	8	4	10	2	53	-	1	6	-	-	125
6 :00 - 6:45	32	8	5	11	6	34	-	-	3	-	-	98
6 :45 - 7:00	22	9	3	11	4	29	-	1	2	-	-	78
Direction: Abinat to Amanuel (Left Turn)												
6:00 - 6:15	5	1	2	2	1	2	-	-	1	-	-	13
6 :15 - 6:30	5	2	2	1	1	3	-	-	2	1	-	14
6 :00 - 6:45	5	2	2	4	2	2	-	-	2	-	-	18
6 :45 - 7:00	3	2	1	4	1	3	-	-	1	-	-	14
Direction: Abinat to Abinat (U-Turn)												
6:00 - 6:15	8	2	2	2	2	5	-	-	2	-	-	21
6 :15 - 6:30	13	3	3	3	2	6	-	-	2	-	-	30
6 :00 - 6:45	9	2	2	1	1	6	-	-	2	-	-	21
6 :45 - 7:00	6	2	2	2	2	5	-	-	1	-	-	

B) Merkato Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Merkato to Autobus Tera (Right Turn)												
6:00 - 6:15	10	3	4	4	5	7	-	-	2	-	-	34
6 :15 - 6:30	7	1	3	4	3	8	-	-	1	1	-	26
6 :00 - 6:45	8	3	2	6	2	11	-	-	3	-	-	34
6 :45 - 7:00	12	2	1	5	3	6	-	-	1	-	-	29
Direction: **Merkato to Amanuel (Through)**												
6:00 - 6:15	7	2	2	1	-	9	-	-	1	-	-	21
6 :15 - 6:30	4	1	1	2	1	9	-	-	1	-	-	18
6 :00 - 6:45	3	2	4	2	1	5	-	-	1	-	-	17
6 :45 - 7:00	4	1	1	1	1	6	-	-	2	-	-	15
Direction: **Merkato to Abinat (Left Turn)**												
6:00 - 6:15	19	5	5	3	5	23	-	-	3	-	-	62
6 :15 - 6:30	21	4	4	6	9	19	-	-	2	-	-	64
6 :00 - 6:45	14	2	5	4	2	22	-	-	3	1	-	51
6 :45 - 7:00	20	2	7	3	6	23	-	-	4	-	-	64
Direction: **Merkato to Merkato (U-Turn)**												
6:00 - 6:15	1	1	-	1	1	3	-	-	2	-	-	7
6 :15 - 6:30	1	1	-	1	-	1	-	-	1	-	-	4
6 :00 - 6:45	2	-	1	1	-	3	-	-	1	-	-	8
6 :45 - 7:00	1	1	-	1	1	2	-	-	2	-	-	

C) Amanuel Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Amanuel to Abinat (Right Turn)												
6:00 - 6:15	12	2	2	2	-	5	-	-	1	-	-	23
6 :15 - 6:30	8	3	-	2	1	2	-	-	2	-	-	17
6 :00 - 6:45	6	3	2	2	1	6	-	-	1	-	-	19
6 :45 - 7:00	7	2	-	4	1	2	-	-	1	-	-	15
Direction: **Amanuel to Merkato (Through)**												
6:00 - 6:15	5	3	2	2	1	7	-	-	2	-	-	20
6 :15 - 6:30	3	1	4	2	1	8	-	-	1	-	-	18
6 :00 - 6:45	9	2	2	1	1	9	-	-	2	-	-	24
6 :45 - 7:00	3	1	2	2	1	6	-	-	1	-	-	15
Direction: **Amanuel to Autobus Tera (Left Turn)**												
6:00 - 6:15	5	2	1	2	1	6	-	-	3	-	-	18
6 :15 - 6:30	1	1	1	2	1	3	-	-	2	-	-	10
6 :00 - 6:45	3	1	-	2	1	8	-	-	2	-	-	16
6 :45 - 7:00	2	2	1	2	1	5	-	-	1	-	-	13
Direction: **Amanuel to Amanuel (U-Turn)**												
6:00 - 6:15	1	-	-	1	-	1	-	-	-	-	-	3
6 :15 - 6:30	-	1	-	-	1	1	-	-	1	-	-	3
6 :00 - 6:45	1	-	-	1	-	-	-	-	1	-	-	2
6 :45 - 7:00	1	-	-	-	-	1	-	-	1	-	-	2

D) Autobus Tera Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: **Autobus Tera to Amanuel (Right Turn)**												
6:00 - 6:15	4	1	3	3	2	5	-	-	2	-	-	20
6 :15 - 6:30	5	2	3	5	1	3	-	-	1	-	-	19
6 :00 - 6:45	5	1	2	5	1	3	-	-	2	-	-	17
6 :45 - 7:00	4	3	2	8	2	3	-	-	1	-	-	22
Direction: **Autobus Tera to Abinat (Through)**												
6:00 - 6:15	40	13	12	10	6	61	-	1	3	-	-	145
6 :15 - 6:30	59	13	8	6	3	60	-	1	2	-	-	150
6 :00 - 6:45	75	14	9	7	4	61	-	1	4	-	-	173
6 :45 - 7:00	51	10	12	13	3	60	-	1	4	-	-	151
Direction: **Autobus Tera to Merkato (Left Turn)**												
6:00 - 6:15	14	2	7	6	2	20	-	-	6	-	-	55
6 :15 - 6:30	16	1	8	8	1	14	-	-	4	-	-	50
6 :00 - 6:45	9	2	6	10	2	18	-	-	3	-	-	49
6 :45 - 7:00	10	3	6	11	3	16	-	-	5	-	-	53
Direction: **Autobus Tera to Autobus Tera (U-Turn)**												
6:00 - 6:15	5	1	2	3	1	7	-	-	1	-	-	19
6 :15 - 6:30	6	2	2	5	1	4	-	-	2	-	-	21
6 :00 - 6:45	3	1	2	2	1	5	-	-	2	-	-	15
6 :45 - 7:00	2	2	2	6	1	6	-	-	2	-	-	19

1.1.3. Sebategna LC Traffic Volume at Evening

A) Abinat Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Merkato (Right Turn)												
11:00 - 11:15	18	4	2	7	10	31	-	-	3	-	-	73
11:15 - 11:30	21	5	2	3	12	34	-	-	4	1	-	80
11:30 - 11:45	11	6	1	2	6	19	-	-	3	1	-	47
11:45 - 12:00	13	5	1	4	7	22	-	-	3	-	-	52
Direction: Abinat to Autobus Tera (Through)												
11:00 - 11:15	39	15	5	2	7	46	-	1	6	-	-	119
11:15 - 11:30	32	10	2	1	9	39	-	1	3	1	-	95
11:30 - 11:45	31	9	5	3	6	30	-	-	4	-	-	86
11:45 - 12:00	22	3	-	2	5	26	-	1	2	-	1	61
Direction: Abinat to Amanuel (Left Turn)												
11:00 - 11:15	4	1	1	1	1	2	-	-	2	-	-	10
11:15 - 11:30	3	1	2	2	1	3	-	-	3	-	-	14
11:30 - 11:45	2	1	1	2	1	2	-	-	3	-	-	11
11:45 - 12:00	2	2	1	2	1	1	-	-	2	-	-	10
Direction: Abinat to Abinat (U-Turn)												
11:00 - 11:15	6	5	1	2	2	5	-	-	3	-	-	23
11:15 - 11:30	8	5	1	1	1	4	-	-	2	-	-	20
11:30 - 11:45	8	1	1	1	1	3	-	-	3	-	-	18
11:45 - 12:00	7	2	1	2	2	4	-	-	1	-	-	18

B) Merkato Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: **Merkato to Autobus Tera (Right Turn)**												
11:00 - 11:15	9	3	1	1	3	2	-	-	2	-	-	19
11:15 - 11:30	12	3	-	1	2	5	-	-	1	-	-	22
11:30 - 11:45	10	3	-	1	3	2	-	-	2	-	-	19
11:45 - 12:00	10	2	1	-	3	3	-	-	1	-	-	19
Direction: **Merkato to Amanuel (Through)**												
11:00 - 11:15	9	3	1	1	1	7	-	-	2	-	-	21
11:15 - 11:30	7	1	1	-	1	8	-	-	-	-	-	17
11:30 - 11:45	4	3	1	1	1	6	-	-	1	-	-	15
11:45 - 12:00	8	2	-	-	-	6	-	-	-	-	-	16
Direction: **Merkato to Abinat (Left Turn)**												
11:00 - 11:15	15	4	1	2	7	17	-	-	1	-	-	47
11:15 - 11:30	28	2	1	1	7	22	-	-	2	-	-	62
11:30 - 11:45	27	3	1	-	7	14	-	-	2	-	-	52
11:45 - 12:00	24	2	-	1	7	14	-	-	2	-	-	50
Direction: **Merkato to Merkato (U-Turn)**												
11:00 - 11:15	3	1	-	-	1	2	-	-	2	-	-	8
11:15 - 11:30	4	1	1	-	-	4	-	-	2	-	-	11
11:30 - 11:45	1	1	-	1	-	5	-	-	1	-	-	8
11:45 - 12:00	3	1	-	1	1	3	-	-	1	-	-	9

C) Amanuel Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: **Amanuel to Abinat (Right Turn)**												
11:00 - 11:15	12	1	1	1	1	9	-	-	2	-	-	26
11:15 - 11:30	12	4	1	-	1	1	-	-	2	-	1	19
11:30 - 11:45	14	2	-	1	1	7	-	-	2	-	-	25
11:45 - 12:00	6	1	1	1	-	2	-	-	1	-	-	11
Direction: **Amanuel to Merkato (Through)**												
11:00 - 11:15	4	2	1	1	1	8	-	-	2	-	-	17
11:15 - 11:30	6	2	2	1	1	6	-	-	2	-	-	17
11:30 - 11:45	3	1	1	1	1	6	-	-	1	-	-	12
11:45 - 12:00	7	1	1	1	1	8	-	-	2	-	-	19
Direction: **Amanuel to Autobus Tera (Left Turn)**												
11:00 - 11:15	6	2	1	1	1	5	-	-	2	-	-	16
11:15 - 11:30	5	1	1	1	2	9	-	-	1	-	-	18
11:30 - 11:45	4	1	1	-	1	4	-	-	2	-	-	12
11:45 - 12:00	3	2	1	-	1	3	-	-	1	-	-	9
Direction: **Amanuel to Amanuel (U-Turn)**												
11:00 - 11:15	1	1	-	-	-	-	-	-	1	-	-	2
11:15 - 11:30	1	1	-	1	-	-	-	-	1	-	-	2
11:30 - 11:45	1	-	-	-	-	-	-	-	-	-	-	1
11:45 - 12:00	1	1	-	-	-	-	-	-	1	-	-	2

D) Autobus Tera Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: **Autobus Tera to Amanuel (Right Turn)**												
11:00 - 11:15	7	2	-	1	1	3	-	-	5	-	-	18
11:15 - 11:30	8	2	-	1	3	3	-	-	5	-	-	20
11:30 - 11:45	9	3	1	1	3	4	-	-	4	-	-	23
11:45 - 12:00	6	4	-	1	1	2	-	-	4	-	-	17
Direction: **Autobus Tera to Abinat (Through)**												
11:00 - 11:15	71	19	1	1	9	99	-	1	5	-	-	205
11:15 - 11:30	70	11	5	3	8	59	-	1	4	-	-	160
11:30 - 11:45	51	10	2	2	6	65	-	1	3	-	-	138
11:45 - 12:00	55	10	1	1	7	54	1	-	5	-	-	131
Direction: **Autobus Tera to Merkato (Left Turn)**												
11:00 - 11:15	14	3	1	1	4	13	-	-	4	-	-	38
11:15 - 11:30	13	3	-	1	1	10	-	-	2	-	-	29
11:30 - 11:45	7	2	1	1	4	9	-	-	4	-	-	27
11:45 - 12:00	7	2	1	1	2	12	-	-	4	-	-	28
Direction: **Autobus Tera to Autobus Tera (U-Turn)**												
11:00 - 11:15	2	1	-	-	-	12	-	-	2	-	-	16
11:15 - 11:30	4	1	-	1	3	7	-	-	2	-	-	17
11:30 - 11:45	3	1	-	-	1	9	-	-	2	-	-	15
11:45 - 12:00	4	1	1	-	2	6	-	-	2	-	-	14

1.2. Adey Ababa LC Traffic Volume

1.2.1. Adey Ababa LC Traffic Volume at Morning

A) Addis Sefer Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Addis Sefer to Gotera (Right Turn)												
2:00 - 2:15	15	3	-	2	-	-	-	-	4	-	1	24
2:15 - 2:30	19	3	-	1	-	1	-	-	2	-	-	25
2:30 - 2:45	15	4	1	1	-	2	-	-	2	-	1	24
2:45 - 3:00	21	2	-	1	-	-	-	-	2	-	-	25
Direction: **Addis Sefer to Beretsige (Through)**												
2:00 - 2:15	32	3	-	1	-	-	-	-	1	-	1	36
2:15 - 2:30	25	6	1	2	1	-	-	-	1	1	1	37
2:30 - 2:45	26	4	-	1	-	-	-	-	3	1	1	34
2:45 - 3:00	37	4	1	2	-	-	-	-	3	1	1	47
Direction: **Addis Sefer to Saris (Left Turn)**												
2:00 - 2:15	17	2	1	2	-	1	-	-	-	-	-	22
2:15 - 2:30	17	7	1	1	-	-	-	-	2	-	-	27
2:30 - 2:45	19	11	-	1	-	1	-	-	3	-	-	34
2:45 - 3:00	24	4	-	2	-	-	-	-	2	-	-	31
Direction: **Addis Sefer to Addis Sefer (U-Turn)**												
2:00 - 2:15	-	-	-	-	-	-	-	-	-	-	-	-
2:15 - 2:30	-	-	-	-	-	-	-	-	-	-	-	-
2:30 - 2:45	-	-	-	-	-	-	-	-	-	-	-	-
2:45 - 3:00	-	-	-	-	-	-	-	-	-	-	-	-

B) Gotera Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Gotera to Beretsige(Right Turn)												
2:00 - 2:15	19	5	1	3	-	1	-	-	-	-	-	28
2:15 - 2:30	20	6	-	1	-	-	-	-	-	1	-	27
2:30 - 2:45	16	4	-	1	-	-	-	-	1	1	-	21
2:45 - 3:00	18	7	-	1	-	-	-	-	1	1	1	28
Direction: Gotera to Saris **(Through)**												
2:00 - 2:15	94	38	1	6	6	3	-	1	3	-	-	150
2:15 - 2:30	90	38	3	7	9	3	-	2	2	-	2	153
2:30 - 2:45	87	36	4	7	7	2	-	1	4	-	1	146
2:45 - 3:00	93	38	3	8	7	2	-	2	3	-	1	154
Direction: **Gotera to Addis Sefer (Left Turn)**												
2:00 - 2:15	24	6	1	-	-	-	-	-	1	-	-	32
2:15 - 2:30	21	7	-	-	-	-	-	-	2	-	-	29
2:30 - 2:45	25	6	1	1	-	-	-	-	1	-	-	32
2:45 - 3:00	16	8	-	-	-	-	-	-	3	-	-	26
Direction: **Gotera to Gotera (U-Turn)**												
2:00 - 2:15	18	14	-	3	-	-	-	-	1	-	-	35
2:15 - 2:30	30	20	-	1	-	1	-	-	2	-	-	53
2:30 - 2:45	32	20	-	1	-	1	-	-	1	-	1	55
2:45 - 3:00	42	15	-	3	-	1	-	-	2	-	-	61

C) Beretsige Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
2:00 - 2:15	4	1	-	1	-	-	-	-	-	-	-	5
2:15 - 2:30	3	2	-	-	-	-	-	-	-	-	-	5
2:30 - 2:45	3	1	1	-	-	-	-	-	-	1	-	5
2:45 - 3:00	3	1	-	-	-	-	-	-	-	-	-	4
Direction: Beretsige to Addis Sefer **(Through)**												
2:00 - 2:15	17	2	-	2	-	-	-	-	2	-	1	23
2:15 - 2:30	11	1	-	1	-	-	-	-	-	1	-	13
2:30 - 2:45	19	3	-	2	-	-	-	-	1	-	-	24
2:45 - 3:00	21	1	1	1	-	-	-	-	3	-	-	26
Direction: **Beretsige to Gotera (Left Turn)**												
2:00 - 2:15	90	11	1	5	1	1	-	-	6	-	2	114
2:15 - 2:30	93	19	-	5	-	-	-	-	2	-	1	119
2:30 - 2:45	88	15	1	3	-	1	-	-	3	-	-	110
2:45 - 3:00	85	10	2	2	-	-	-	-	4	-	-	102
Direction: **Beretsige to Beretsige(U-Turn)**												
2:00 - 2:15	-	-	-	-	-	-	-	-	-	-	-	-
2:15 - 2:30	-	-	-	-	-	-	-	-	-	-	-	-
2:30 - 2:45	-	-	-	-	-	-	-	-	-	-	-	-
2:45 - 3:00	-	-	-	-	-	-	-	-	-	-	-	-

D) Saris Approach

Direction: S**aris to Addis Sefer (Right Turn)**												
Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
2:00 - 2:15	2	1	-	1	2	-	-	-	1	-	-	6
2:15 - 2:30	2	2	-	-	-	-	-	-	-	-	-	3
2:30 - 2:45	1	1	-	-	-	1	-	-	-	-	-	3
2:45 - 3:00	3	-	-	-	-	-	-	-	-	-	-	3
Direction: Saris to Gotera **(Through)**												
2:00 - 2:15	55	28	1	5	5	1	-	2	3	-	1	99
2:15 - 2:30	56	28	5	4	2	4	-	2	1	1	-	101
2:30 - 2:45	52	24	2	7	4	3	-	2	1	-	1	94
2:45 - 3:00	53	19	2	6	3	3	-	1	3	-	3	90
Direction: **Saris to Beretsige (Left Turn)**												
2:00 - 2:15	8	2	-	-	-	-	-	-	-	-	-	10
2:15 - 2:30	7	2	-	1	-	-	-	-	1	1	-	11
2:30 - 2:45	5	4	1	2	-	-	-	-	-	-	-	11
2:45 - 3:00	9	2	1	2	-	-	-	-	1	-	-	14
Direction: **Saris to Addis Saris (U-Turn)**												
2:00 - 2:15	11	19	-	1	4	1	-	-	1	-	-	35
2:15 - 2:30	10	28	-	-	3	2	-	-	-	-	-	43
2:30 - 2:45	14	23	-	1	2	2	-	-	-	2	-	42
2:45 - 3:00	15	20	1	1	1	3	-	-	-	1	-	40

1.2.2. Adey Ababa LC Traffic Volume at Mid-Day

A) Addis Sefer Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
6:30 - 6:45	17	4	-	2	-	-	-	-	4	-	-	26
6:45 - 7:00	22	2	-	1	-	-	-	-	1	-	-	25
7:00 - 7:15	20	3	1	-	-	-	-	-	1	-	-	24
7:15 - 7:30	15	3	-	1	-	-	-	-	2	-	-	20
Direction: **Addis Sefer to Beretsige(Through)**												
6:30 - 6:45	30	5	-	-	-	-	-	-	3	-	-	37
6:45 - 7:00	21	5	1	1	-	-	-	-	2	1	-	30
7:00 - 7:15	22	4	1	1	-	-	-	-	1	1	-	29
7:15 - 7:30	31	4	1	1	-	-	-	-	3	1	1	40
Direction: **Addis Sefer to Saris (Left Turn)**												
6:30 - 6:45	19	1	-	-	-	-	-	-	1	1	-	22
6:45 - 7:00	18	4	-	2	-	-	-	-	1	-	-	25
7:00 - 7:15	17	4	-	3	-	-	-	-	2	-	1	26
7:15 - 7:30	14	1	1	1	-	-	-	-	1	-	-	17
Direction: **Addis Sefer to Addis Sefer (U-Turn)**												
6:30 - 6:45	-	-	-	-	-	-	-	-	-	-	-	-
6:45 - 7:00	-	-	-	-	-	-	-	-	-	-	-	-
7:00 - 7:15	-	-	-	-	-	-	-	-	-	-	-	-
7:15 - 7:30	-	-	-	-	-	-	-	-	-	-	-	-

B) Gotera Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Gotera to Beretsige(Right Turn)												
6:30 - 6:45	20	7	1	3	-	-	-	-	1	-	1	31
6:45 - 7:00	23	5	1	1	-	-	-	-	1	-	-	30
7:00 - 7:15	22	8	1	1	-	-	-	-	1	-	-	32
7:15 - 7:30	20	6	-	1	-	-	-	-	2	-	1	29
Direction: Gotera to Saris (Through)												
6:30 - 6:45	97	32	7	7	6	2	-	2	6	-	-	156
6:45 - 7:00	94	29	12	7	5	2	-	1	6	-	1	155
7:00 - 7:15	90	29	10	11	10	3	-	1	9	-	-	162
7:15 - 7:30	94	32	4	4	5	2	-	1	7	-	1	149
Direction: Gotera to Addis Sefer (Left Turn)												
6:30 - 6:45	27	6	-	1	-	-	-	-	3	-	-	35
6:45 - 7:00	54	6	1	2	1	-	-	-	1	-	-	63
7:00 - 7:15	31	7	1	2	-	-	-	-	3	-	-	42
7:15 - 7:30	36	6	-	2	-	-	-	-	3	-	-	46
Direction: Gotera to Gotera (U-Turn)												
6:30 - 6:45	49	19	-	3	-	-	-	-	3	-	-	73
6:45 - 7:00	34	19	1	2	-	1	-	-	2	-	-	58
7:00 - 7:15	35	19	1	2	1	-	-	-	1	-	1	59
7:15 - 7:30	40	19	1	1	-	-	-	-	2	-	-	62

C) Beretsige Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Beretsige to Saris (Right Turn)												
6:30 - 6:45	6	2	1	-	-	-	-	-	1	-	-	9
6:45 - 7:00	4	2	-	-	-	1	-	-	-	-	-	6
7:00 - 7:15	3	1	1	1	-	-	-	-	-	-	-	5
7:15 - 7:30	6	1	1	2	-	-	-	-	-	1	-	9
Direction: Beretsige to Addis Sefer (Through)												
6:30 - 6:45	17	4	-	2	-	-	-	-	4	1	1	28
6:45 - 7:00	27	4	-	1	-	-	-	-	3	-	-	34
7:00 - 7:15	19	3	-	1	-	-	-	-	3	-	-	26
7:15 - 7:30	22	4	-	1	-	-	-	-	3	-	-	30
Direction: Beretsige to Gotera (Left Turn)												
6:30 - 6:45	70	15	-	2	-	-	-	-	3	1	2	92
6:45 - 7:00	73	15	1	3	-	1	-	-	4	-	-	95
7:00 - 7:15	63	11	1	5	-	-	-	-	4	-	-	83
7:15 - 7:30	69	13	1	3	-	1	-	-	4	-	-	90
Direction: Beretsige to Beretsige(U-Turn)												
6:30 - 6:45	-	-	-	-	-	-	-	-	-	-	-	-
6:45 - 7:00	-	-	-	-	-	-	-	-	-	-	-	-
7:00 - 7:15	-	-	-	-	-	-	-	-	-	-	-	-
7:15 - 7:30	-	-	-	-	-	-	-	-	-	-	-	-

D) Saris Approach

Direction: Saris to Addis Sefer (Right Turn)												
Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
6:30 - 6:45	7	5	-	1	-	-	-	-	-	-	-	12
6:45 - 7:00	9	2	-	-	-	-	1	-	-	-	-	11
7:00 - 7:15	7	5	-	2	-	-	-	-	1	-	-	14
7:15 - 7:30	8	3	-	1	-	1	-	-	1	-	-	12
Direction: Saris to Gotera (Through)												
6:30 - 6:45	78	24	4	4	3	2	-	2	13	1	1	129
6:45 - 7:00	66	15	2	5	4	2	-	2	7	1	2	101
7:00 - 7:15	58	16	5	4	3	1	-	1	10	-	-	95
7:15 - 7:30	66	21	5	4	2	2	-	1	4	1	1	104
Direction: Saris to Beretsige (Left Turn)												
6:30 - 6:45	10	5	-	1	-	1	-	-	3	-	1	20
6:45 - 7:00	11	2	-	-	1	-	-	-	2	-	-	15
7:00 - 7:15	15	1	-	1	-	-	-	-	1	-	1	18
7:15 - 7:30	13	2	-	1	-	-	-	-	2	-	2	18
Direction: Saris to Addis Saris (U-Turn)												
6:30 - 6:45	20	22	-	3	1	2	-	-	1	-	1	49
6:45 - 7:00	18	20	-	1	1	3	-	-	1	1	-	44
7:00 - 7:15	10	14	1	1	1	3	-	-	-	-	-	29
7:15 - 7:30	16	12	-	2	-	3	-	-	1	1	-	34

1.2.3. Adey Ababa LC Traffic Volume at Evening

A) Addis Sefer Approach

Direction: **Addis Sefer to Gotera (Right Turn)**												
Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
11:00 -11:15	21	5	-	-	-	-	-	-	3	-	-	29
11:15 -11:30	21	4	-	-	-	-	-	-	2	1	-	28
11:00 -11:45	16	3	-	-	-	-	-	-	1	-	-	20
11:45 -12:00	19	6	-	-	-	-	-	-	1	1	-	26
Direction: **Addis Sefer to Beretsige(Through)**												
11:00 -11:15	18	5	-	1	-	-	-	-	4	-	1	28
11:15 -11:30	27	3	1	1	-	-	-	-	2	-	-	33
11:00 -11:45	20	6	1	1	-	-	-	-	3	-	-	29
11:45 -12:00	16	2	1	-	-	-	-	-	3	-	-	21
Direction: **Addis Sefer to Saris (Left Turn)**												
11:00 -11:15	20	5	-	-	-	-	-	-	-	1	-	26
11:15 -11:30	22	2	-	1	-	-	-	-	3	-	-	27
11:00 -11:45	16	5	-	1	-	1	-	-	2	-	-	25
11:45 -12:00	21	1	-	1	-	-	-	-	-	-	1	23
Direction: **Addis Sefer to Addis Sefer (U-Turn)**												
11:00 -11:15	-	-	-	-	-	-	-	-	-	-	-	-
11:15 -11:30	-	-	-	-	-	-	-	-	-	-	-	-
11:00 -11:45	-	-	-	-	-	-	-	-	-	-	-	-
11:45 -12:00	-	-	-	-	-	-	-	-	-	-	-	-

B) Gotera Approach

Direction: Gotera to Beretsige(Right Turn)												
Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
11:00 -11:15	18	4	-	-	-	-	-	-	1	-	-	22
11:15 -11:30	17	6	-	-	-	-	-	-	-	-	-	23
11:00 -11:45	19	6	-	-	-	1	-	-	-	-	-	26
11:45 -12:00	23	3	-	-	-	-	-	1	-	-	-	26
Direction: Gotera to Saris (Through)												
11:00 -11:15	120	38	2	2	7	3	-	1	7	-	-	179
11:15 -11:30	131	22	1	2	7	4	-	1	4	-	1	171
11:00 -11:45	135	21	1	1	7	2	-	-	6	-	1	172
11:45 -12:00	99	15	3	2	8	3	-	2	3	1	1	135
Direction: Gotera to Addis Sefer (Left Turn)												
11:00 -11:15	25	8	-	-	1	2	-	-	2	1	-	37
11:15 -11:30	29	4	-	1	1	-	-	-	3	1	-	37
11:00 -11:45	31	5	-	1	-	1	-	-	1	-	-	38
11:45 -12:00	20	4	1	2	-	2	-	-	1	-	-	28
Direction: Gotera to Gotera (U-Turn)												
11:00 -11:15	35	21	-	1	-	-	-	-	1	-	-	57
11:15 -11:30	33	25	-	1	1	-	-	-	-	-	-	59
11:00 -11:45	36	22	-	1	-	-	-	-	1	-	-	59
11:45 -12:00	31	23	-	-	1	1	-	-	2	2	-	57

C) Beretsige Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Beretsige to Saris (Right Turn)												
11:00 -11:15	9	3	-	-	-	-	-	-	-	1	1	13
11:15 -11:30	4	2	-	-	-	-	-	-	-	-	-	6
11:00 -11:45	12	4	-	-	-	-	-	-	-	1	-	17
11:45 -12:00	11	3	-	1	-	-	-	-	-	-	-	14
Direction: Beretsige to Addis Sefer (Through)												
11:00 -11:15	22	2	1	1	-	-	-	-	4	1	1	31
11:15 -11:30	21	2	1	-	-	-	-	-	3	-	2	28
11:00 -11:45	24	3	-	-	-	-	-	-	2	1	1	30
11:45 -12:00	21	5	-	-	-	1	-	-	2	1	1	29
Direction: Beretsige to Gotera (Left Turn)												
11:00 -11:15	56	7	1	2	-	3	-	-	4	-	-	72
11:15 -11:30	59	10	-	1	1	-	-	-	1	-	1	71
11:00 -11:45	42	6	-	-	1	-	-	-	2	-	1	51
11:45 -12:00	71	11	2	2	-	-	-	-	4	-	-	88
Direction: Beretsige to Beretsige(U-Turn)												
11:00 -11:15	-	-	-	-	-	-	-	-	-	-	-	-
11:15 -11:30	-	-	-	-	-	-	-	-	-	-	-	-
11:00 -11:45	-	-	-	-	-	-	-	-	-	-	-	-
11:45 -12:00	-	-	-	-	-	-	-	-	-	-	-	-

D) Saris Approach

Time	Car	Taxi	Truck	Small Truck	Bus	Small Bus/Mini Van	BRT	LRT	Two Wheelers	Three Wheelers	Cyclist	Total
Direction: Saris to Addis Sefer (Right Turn)												
11:00 -11:15	8	4	-	1	-	-	-	-	-	-	-	12
11:15 -11:30	7	3	-	-	-	1	-	-	-	-	-	10
11:00 -11:45	8	4	-	-	-	-	-	-	-	-	-	12
11:45 -12:00	6	2	-	-	1	2	-	-	1	-	-	11
Direction: Saris to Gotera (Through)												
11:00 -11:15	63	24	1	3	6	6	-	1	13	-	1	116
11:15 -11:30	50	13	1	2	10	3	-	1	8	-	1	87
11:00 -11:45	65	23	1	1	11	2	-	1	2	1	-	104
11:45 -12:00	57	13	3	2	5	9	-	-	2	-	-	89
Direction: Saris to Beretsige (Left Turn)												
11:00 -11:15	11	2	1	1	-	1	-	-	4	1	-	19
11:15 -11:30	14	4	1	1	-	1	-	-	1	1	1	22
11:00 -11:45	12	4	1	-	-	2	-	-	1	-	-	18
11:45 -12:00	11	2	-	-	1	1	-	-	2	1	-	16
Direction: Saris to Addis Saris (U-Turn)												
11:00 -11:15	25	19	-	2	1	1	-	-	-	-	-	46
11:15 -11:30	30	14	-	1	1	1	-	-	1	1	1	47
11:00 -11:45	19	21	1	2	2	2	-	-	1	-	1	46
11:45 -12:00	20	12	-	2	3	2	-	-	1	2	1	40