ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF GRADUATE STUDIES
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Comparison of Alternative Junction Designs For Light Rail Transit Grade Crossings

Case Study of Sealite Mehret and CMC Roundabouts

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

By:

Rediet Firdu G/Tsadik

Advisor: Bikila Teklu Wodajo (PhD)

Co- Advisor: Yonas Minalu (M.Sc.)

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Addis Ababa, Ethiopia

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GSR/3518/07

The undersigned have examined the thesis entitled '**Comparison of Alternative Junction Designs for Light Rail Transit Grade Crossings**' presented by **Rediet Firdu G/Tsadik**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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I certify that research work titled “Comparison of Alternative Junction Designs for Light Rail Transit Grade Crossings” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ABSTRACT

Light rail transit is a relatively new mode of transport for Ethiopia. Hence, rigorous research needs to be done in order to identify and work out flaws in the system. Most of the road-rail intersections within Addis Ababa LRT corridors are built as grade separated intersections. However, certain intersections such as Sealite Mehret and CMC roundabout meet the LRT line at grade. Research done up to now on these areas, points out significant problems with respect to traffic congestion. The main causes of congestion in this area that are related to the LRT line are: frequent LRT crossing and interruption of traffic flow, high pedestrian volumes at stations near the roundabouts that impede entry and exit flows, lack of U turn areas along the LRT corridor, which transfers high volumes of U turning vehicles to the roundabouts, which in turn adds to congestion of the roundabouts. Performance evaluation of the roundabouts under current traffic conditions was done during peak and off – peak periods. Results indicate approach delays of over ten minutes for both locations during peak periods. However, during off – peak periods the roundabouts perform above level of service C.

The goal of this study is to propose alternative solutions to mitigate each cause of congestion and to see if adequate performance can be achieved without grade separation at the selected locations: Sealite Mehret and CMC roundabouts. In order to fulfill this goal necessary data were collected at the study areas. Then the alternatives were designed and modeled using VISSIM simulation software.

The alternatives considered are signalized intersection, signalized roundabout, installation of pedestrian signals and diverting U turns to other locations. Comparing these alternatives based on queue length, travel time, intersection delay and level of service under current traffic conditions, showed that the best alternatives for both locations are signalized roundabout and signalized intersection.

Up on comparing signalized roundabout and signalized intersection alternatives under forecasted traffic conditions, analysis results showed that both options have similar travel times and delay. Both alternatives are viable options. However, since signalized roundabout alternative provides the option of functioning as a normal roundabout during off peak periods, and reduces risk of right angle crashes, signalized roundabout is preferred.

Key words: Signalized roundabout, LRT grade crossing, VISSIM

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Acronyms

AACRA	Addis Ababa City Roads Authority
CBD	Central Business District
ERA	Ethiopian Road Authority
FHWA	Federal Highway Administration
HCM	Highway Capacity Manual
LOS	Level of service
LRT	Light rail transit
LRV	Light rail vehicle
MUTCD	Manual on Uniform Traffic Control Devices
PHF	Peak-hour factor
PCU	Passenger Car Unit
TRRL	Transport Research Record Laboratory
VISSIM	"Verkehr In Städten - SIMulationsmodell" (German for "Traffic in cities - simulation model")
v/c	Demand to capacity ratio
Xc	Demand to capacity ratio

CHAPTER ONE: INTRODUCTION

1.1. Background

Ethiopian Railway Corporation has launched a Light Rail Transit Project in the city of Addis Ababa in order to enhance urban mobility. The project has a total length of 34.25 km. The first 17.35Km runs from East to West direction stretching from Ministry of Defense to Ayat and the second 16.9Km line runs from North to South from Menelik Square to Kaliti. The two transit lines have a common track of about 2.8Km which stretches from Lideta to Meskel-Square. There are a total of 22 stations on the East-West line; one underground station, 16 at grade stations and 5 elevated stations in the East-West LRT system. [**1**]

Some of the advantages of light rail transit are safety, reliable travel times, lower life cycle cost due to lower maintenance, fuel and vehicle operating costs, increase in property value, environmental benefits such as lesser noise and air pollution and use of renewable energy as power supply. While the benefits of implementing an LRT system are numerous, if the system is not well integrated with the existing road network, negative impacts may be observed. Such as; additional delay around at grade intersections and LRT station, risks to safety of pedestrians and increased travel paths.

In the assessment of impact on traffic congestion due to the new LRT system, a study done in 2013 by Yilkal Endeshaw predicted that there would be an additional delay to the normal control delay at three junctions (Beshale/ Sealite Mehret , CMC and Ayat Roundabout, where the LRT crosses at-grade) to both left turn movements and through movement [**2**]. A subsequent study done in 2014 by Eskinder Nigussie at the same study areas showed that additional delay at roundabouts increases as LRT crossing frequency increases. Furthermore, pedestrians getting on and off the LRT vehicle at ground stations interrupted traffic movement. [**3**]

As mentioned above, previous studies pointed out that the introduction of the LRT line will cause additional delay at roundabouts. This study compares alternative junction designs with two roundabouts. The aim of this study is to evaluate the performance of selected roundabouts under existing traffic conditions and to compare them with alternative designs. The alternative designs that will be considered are signalized roundabout, signalized intersection, installation of pedestrian signals and diverting U turns.

1.2. Statement of the Problem

Light rail transit is a relatively new mode of transport for Ethiopia. Hence, rigorous research needs to be done in order to identify and work out flaws in the system. Research done up to now on at-grade LRT crossings, points out significant problems with respect to traffic congestion.

- CMC and Sealite Mehret roundabouts were performing at LOS F even before the introduction of the LRT line. [**2**]
- The introduction of the LRT line causes additional delay at roundabouts to adjacent traffic [**2**] which increases with increasing LRT crossing frequency. [**3**]

This shows that both roundabouts are performing at a low level of service. Therefore, it would be relevant to see how other types of at grade intersections would perform under the same conditions.

1.3. Objectives of the Study

1.3.1. General Objectives

The goal of this study is to compare alternative at grade junction designs and recommend the best solution for the study areas and similar areas with the same key characteristics: roundabouts with high traffic volumes, median LRT crossings and high pedestrian volumes.

1.3.2. Specific Objectives

The specific objectives of this research are:

- To evaluate the performance of selected roundabouts under existing conditions and identify the major causes of congestion.
- To evaluate performance of alternative solutions under current and forecasted traffic using VISSIM,
- To compare the alternatives solutions based on various performance measures and recommend the best alternative to the existing roundabouts.

1.4. Scope of the Study

This study only addresses at grade junction types suitable for the study area that are expected to improve current performance. It does not include comparison to grade separated junctions.

1.5. Relevance

- **Proposes alternative solutions to reduce traffic congestion at study areas**

Safe and reliable transportation is a key asset to the economy. Improving operation of intersections, improves the performance of the whole road network, which aids the development of the city. This study attempts to alleviate congestion at two roundabouts within the Addis Ababa East – West LRT corridor and further explores the causes of congestion. Based on the causes identified, simple and low cost solutions such as signalized roundabout, diverting U turns and installing pedestrian signals are proposed.

- **Modeling a relatively new type of design**

Signalized roundabout is a relatively new type of junction design. Previous studies on roundabouts are mainly based on single-lane and double-lane roundabouts. There is much literature concerning safety, operational analysis and environmental impacts of roundabouts. In contrast, very few publications are devoted to developing new-style traffic control solutions. [**4**] Furthermore, comparison studies involving signalized roundabouts are scarce. Therefore, any additional study focusing on signalized roundabout, will add to the current body of knowledge.

- **Provide some insight for future intersection designs with LRT lines**

This study will compare existing roundabouts to various alternatives in an attempt to find the better junction design to accommodate a median LRT grade crossing without causing excessive delay to vehicles. This could provide some indications for future intersections to be designed within LRT corridors, as LRT systems are only beginning to develop in Ethiopia.

1.6. Thesis Organization

This Thesis is organized in five chapters. The first chapter explains why this study needed to be done and what other significances it will have apart from addressing problems pointed out in previous studies. The second chapter summarizes previous literature relevant to this study. Chapter three details the research methods used, the data collected for this study and reports the details of the designs of the signalized roundabout and intersection that were compared to the existing roundabout. Chapter four presents the analysis carried out to compare the alternative designs. Chapter five concludes the thesis.

CHAPTER TWO: LITERATURE REVIEW

2.1. Introduction

The design of at-grade intersections near highway-railroad grade crossings is challenging because of the interaction between the two geometric features. Their designs can have a critical effect on safety and operation at both features. [**5**]While travelling at-grade in an urban environment, a Light Rail Transit (LRT) system needs to traverse road intersections of various complexities. These intersections, including roundabouts, are major hazardous locations in LRT networks. There are several documents related to the treatment of conventional LRT intersections for guaranteeing safety, but almost no reference to LRT roundabouts design, except for the French guidelines. Roundabout operation changes when an LRT system is implemented. A common way of including a modern LRT line in a roundabout is with the tracks running through its center, protected by traffic lights located before the points where the circular road crosses the LRT tracks. Other types of crossings near intersections are side and adjacent alignments. [**6**]

Previous studies done on at-grade intersections near highway-railroad grade crossings , such as roundabouts or signalized intersections indicate reduced LOS and increased delay to vehicular traffic due to an introduction of an LRT line. [**2**,**7**] While other studies have found some intersections worsen and others do not show significant change in delay and level of service. [**8**,**9**]

Studies have been done to reduce delay and improve LOS or safety at these locations, however, studies that suggest the best at grade junction for any type of at grade LRT crossing have not been found.

The first section of this chapter will describe the types of at grade intersections that can be alternatives to the study area. Section 2.3 will review comparative studies done on at grade junctions. The final section of this chapter will attempt to identify gaps in previous research and to show how this study would be relevant.

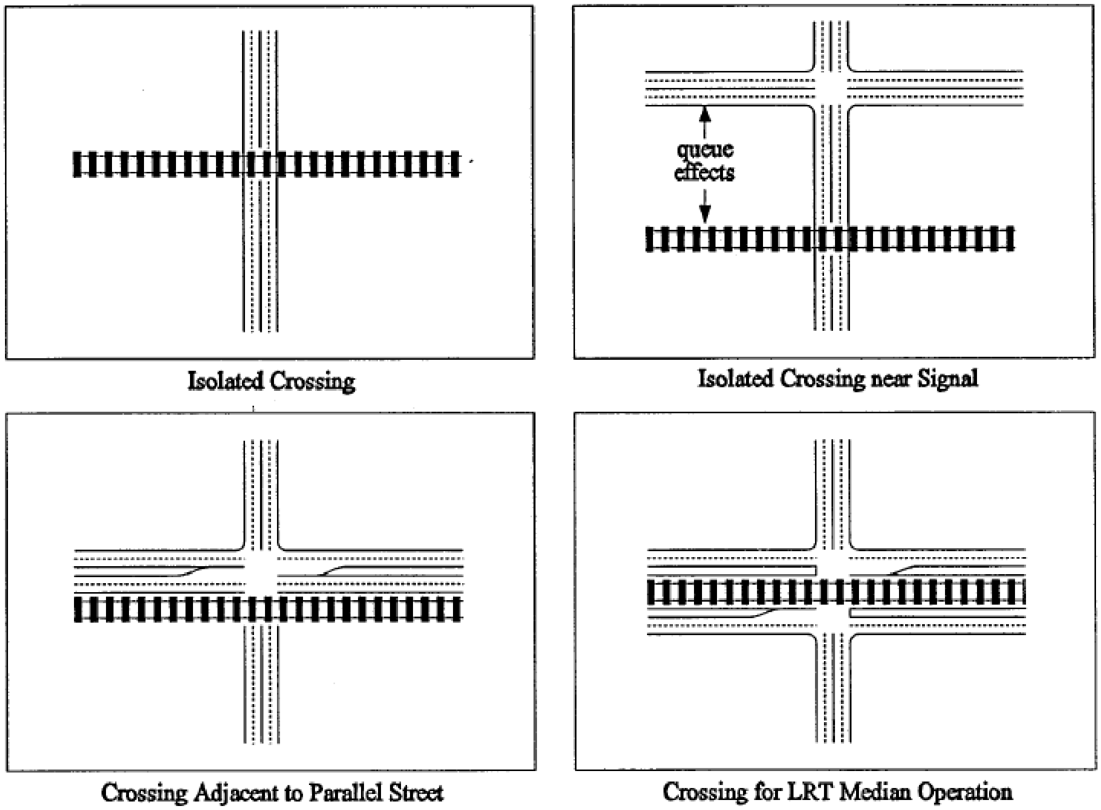


Figure 1 Types of LRT crossings [6]

2.2. Alternative junction designs for LRT grade crossings

2.2.1. Roundabouts

2.2.1.1. Introduction

Roundabouts are junctions with a one-way circulatory carriageway around a central island. Vehicles on the circulatory carriageway have priority over those approaching the roundabout. [**10**] Modern roundabouts are near-circular intersections at grade. They are an effective intersection type with fewer conflict points and lower speeds, and they provide for easier decision making than conventional intersections. They also require less maintenance than traffic signals. Well-designed roundabouts have been found to reduce crashes (especially fatal and severe injury collisions), traffic delays, fuel consumption, and air pollution. They also have a traffic-calming effect by reducing vehicle speeds using geometric design rather than relying solely on traffic control devices. [**11**]

Two basic operational and design principles define modern roundabouts [**12**]:

- Yield-at-entry where entering vehicles must yield to crossing pedestrians and to vehicles on the circulatory roadway of the roundabout and
- Deflection of entering traffic where entering traffic is deflected to the right by a central island on each approach to the roundabout.

Modern roundabouts range in size from mini-roundabouts with inscribed circle diameters as small as 50 ft (15.2 m), to compact roundabouts with inscribed circle diameters between 100 and 115 ft (30.5 and 35.1 m), to large roundabouts, often with multilane circulating roadways and more than four entries up to 500 ft (152.4 m) in diameter. [**12**] Mini-roundabouts are small single-lane roundabouts generally used in 25 mph or less urban/suburban environments. Because of this, mini-roundabouts are typically not suitable for use on higher-volume (greater than 6,000 AADT) state routes. A common application is to replace a stop-controlled or uncontrolled intersection with a mini-roundabout to reduce delay and increase capacity. [**11**] Mini-roundabouts are not well suited for high volumes of trucks, as trucks will occupy most of the intersection when turning. The central island of a mini-roundabout is typically a minimum of 4 m (13 ft) in diameter. [**13**]

Based on the number of circulating lanes roundabouts can be classified in to single and multiple lane roundabouts. Single-lane roundabouts have single-lane entries at all legs and one circulating lane. [**11**] The circulating carriageway width of single lane roundabouts should cater for the movement of the largest vehicle normally expected to use the roundabout. [**14**] Multilane roundabouts have at least one entry or exit with two or more lanes and more than one circulating lane. The operational practice for trucks negotiating roundabouts is to encroach on adjacent lanes. [**11**]

A roundabout brings together conflicting traffic streams, and allows the streams to safely merge and traverse the roundabout, and exit the streams to their desired directions. The geometric elements of the roundabout provide guidance to drivers approaching, entering, and traveling through a roundabout. [**13**] Roundabouts perform best at intersections with similar traffic volumes on each approach leg and at intersections with heavy left-turning volumes. Roundabouts reduce the severity and frequency of intersection crashes by the nature of their design. They also resolve vehicle conflicts by means of priority control; the key operational feature of roundabouts is that entering vehicles yield to the circulating traffic. Traffic interactions are based on gap acceptance; entering traffic must wait for a gap in the traffic stream to enter. [**12**]

Drivers approaching a roundabout must slow to a speed that will allow them to safely interact with other users of the roundabout, and to negotiate the roundabout. The width of the approach roadway, the curvature of the roadway, and the volume of traffic present on the approach govern this speed. As drivers approach the yield line, they must check for conflicting vehicles already on the circulating roadway and determine when it is safe and prudent to enter the circulating stream. The widths of the approach roadway and entry determine the number of vehicle streams that may form side by side at the yield line and govern the rate at which vehicles may enter the circulating roadway. The size of the inscribed circle affects the radius of the driver's path, which in turn determines the speed at which drivers travel on the roundabout. The width of the circulatory roadway determines the number of vehicles that may travel side by side on the roundabout.

Roundabouts eliminate left turns at intersections, which reduces the opportunity for crashes. Roundabouts contain only four merging conflict points, compared with 24 merging/crossing conflict points at intersections controlled by STOP signs or traffic signals. [**13**]

2.2.1.2. Capacity Models

Roundabout capacity is the main determinant parameter for the performance measure of many other parameters as delay and queue length. It is the maximum sustainable flow rate that can be achieved during a specific period under prevailing traffic, geometric and control conditions. Capacity is service rate and is different than maximum volume that an intersection can handle, which is the practical capacity under high demand volume; not under prevailing conditions. [**15**]

Roundabout capacity models used in different parts of the world can be grouped in to three types of models; Empirical regression models, Gap acceptance models and simulation based models. Empirical regression model, also known as conflicting volume model, predicts capacity by means of establishing a regression equation between entry capacity and circulating volume. The prediction is significant under saturated flow condition. [**16**] Many countries use empirical regression model, such as UK, Switzerland, Germany and France . In addition, Federal Highway Administration (FHWA) also proposed this type of model. [**17**] Akcelik stated that the limitation of this approach is that it would underestimate the capacity during low circulating flow and overestimate the capacity during high circulating flow. [**18**]

The gap-acceptance approach derives roundabout capacity by considering the driver's behavior that follows the offside priority rule. It is modeled as a series of T-intersections in which the entry flow is determined by the critical gap, follow-up time, circulating flow, and weaving section flow, and has no interactions among the approach flows. [**19**] The circulating streams have the absolute priority at modern roundabout, while the entry flows are similar to the minor-streams at TWSC intersection. Therefore, capacity model can be derived by means of gap acceptance theory. [**20**] The model has some key parameters which can describe microscopic traffic status, such as critical gap, follow-up time. Thus, the model can reflect the time-dependent characteristics of traffic flows and has more flexibility. Gap acceptance model is mainly used in Australia, Denmark and U.S.. [**17**]

Traffic flow modeling can generally be classified in to microscopic and macroscopic modeling. Macroscopic modeling looks at traffic flow from a global perspective, whereas microscopic modeling, as the term suggests, gives attention to the details of traffic flow and the interactions taking place within it. A microscopic model of traffic flow attempts to analyze the flow of traffic by modeling driver-driver and driver-road interactions within a traffic stream which respectively analyzes the interaction between a driver and another driver on road and of a single driver on the different features of a road. [**21**]The analysis tools for roundabout capacity are divided into two categories. The former is based on calculation models, such as aaSIDRA and RODEL, in which the roundabout is considered as an isolated geometric element. The latter are some software based on micro-simulation models, such as PARAMICS, SimTraffic and VISSIM. They can be used to simulate complex intersections, and the results will be more realistic and reliable. [**17**]
The maximum flow rate that can be accommodated at a roundabout entry depends mainly on the following factors; the circulating flow on the roundabout that conflicts with the entry flow, exiting flow and the geometric elements of the roundabout. When the circulating flow is low, drivers at the entry are able to enter the roundabout without any significant delay. The larger the gaps, i.e. the headways, in the circulating flow the more useful they are for the drivers entering the roundabout. In fact, more than one vehicle may enter in each gap. As the circulating flow increases, the size of the gaps in the circulating flow decreases, and the rate at which vehicles enter the roundabout decreases. The geometric elements of the roundabout also affect the rate of entry flow. The most important geometric elements are the width of the entry, the width of the circulatory roadway, or number of lanes around the central island. Two entry lanes permit nearly

twice the rate of entry flow as one lane. Wider circulatory roadways allow vehicles to travel alongside, or follow, each other in tighter bunches and so provide longer gaps between bunches of vehicles. The flare length also affects the capacity. The inscribed circle diameter and the entry angle have minor effects on capacity. [**15**]

2.2.1.3. Performance Analysis

The FHWA guide on roundabouts states three performance measures are typically used to estimate the performance of a given roundabout design: degree of saturation, delay, and queue length. Each measure provides a unique perspective on the quality of service at which a roundabout will perform under a given set of traffic and geometric conditions. Whenever possible, the analyst should estimate as many of these parameters as possible to obtain the broadest possible evaluation of the performance of a given roundabout design. In all cases, a capacity estimate must be obtained for an entry to the roundabout before a specific performance measure can be computed. [**13**]

2.2.1.4. Use of Roundabouts in LRT networks

I. Harmonization of Light Rail Transit and Principal Arterial Streets

Case Study on the Addis Ababa East-West LRT Line and Principal Arterial Streets

In order to assess impact on traffic congestion (the possible additional delay to be induced) at Intersections after the introduction of new Light Rail Transit System into existing Principal Arterial Streets, primary data was collected at four selected junctions of Addis Ababa E-W corridor where the LRT crosses at grade with the existing principal street intersections. Peak hour traffic data was collected and geometric elements of the study junctions was measured for the purpose. In the assessment of impact of traffic congestion due to the new LRT system, the result revealed that there is an additional delay to the normal control delay at the three junctions (Sealite Mehret Roundabout, CMC Roundabout and Ayat Roundabout) where the LRT crosses at-grade. This study predicted that with the existing geometric condition and projected traffic, in 2016 the left turn movements would face about 41.7 sec/vehicle of additional average delay after the introduction of Light Rail transit. On the other hand, the through traffic of North-South direction at these locations would experience more additional delays of about 47.7 sec/vehicle on average. At Bambis Intersection, since the LRT is separated from the city street traffic with

median curb stone and North-South Crossing is prohibited, additional delays are not observed. Instead, the through traffic of East-West direction at this junction is observed to experience less control delay than before due to the decrease in conflicts of North and South crossing traffic. [**2**]
From this study it can be understood that for the studied locations, a roundabout with an LRT line running through the median performs less than one without an LRT line, in terms of average delay.

II. Level of service analysis at light rail transit grade crossings under different operational conditions.

Case study of CMC and Sealite Mehret roundabouts

This study focuses on level of service analysis at LRT grade crossings under different operational conditions. Using VISSIM simulation software, CMC and Sealite Mehret roundabouts were modeled and evaluated under various LRT arrival frequencies as well as permitted and prohibited street parking considerations. Results showed that additional delay increases as LRT crossing frequency increases, LOS deteriorates. Furthermore, allowing street parking during rush hours increased traffic congestion and also that pedestrians crossing the street near ground stations interrupted traffic movement. [**3**]

III. Use and design of safe roundabouts in Light Rail Transit (LRT) networks

This study examines the use and design of safe roundabouts in Light Rail Transit (LRT) networks. It explains the roundabout operation changes when an LRT system is implemented. A common way of including a modern LRT line in a roundabout is with the tracks running through its center, protected by traffic lights located before the points where the circular road crosses the LRT tracks.
The way in which the usual LRT roundabout (with traffic lights) functions is as follows: the roundabout works conventionally when the light rail vehicle (LRV) is not present or approaching (priority for road vehicles that are on the roundabout). Nevertheless, traffic lights are provided in the circular road of the roundabout, immediately before the crossing of the tracks, which give priority to approaching or present LRVs. Therefore, road vehicle drivers will have the priority while running on the roundabout only when the LRV is not in the vicinity, but have to yield (stop before the traffic lights) if an LRV is present or approaching.

The main question that arises from the previous considerations is: When is it reasonable to use a roundabout in an LRT line? The generic answer is: do not use roundabouts as a general solution, but only when there are strong reasons that make this configuration more advisable than a conventional at-grade intersection controlled by traffic lights. These reasons are related to the movements that need to be allowed at the intersection, or to the configuration of the streets that converge on the roundabout.

- If left and/or U turn movements are transformed into a perpendicular crossing of the tracks, with a better visibility, as long as the roundabout is well designed.
- If the traffic light cycle for a conventional at-grade intersection would be complicated, whereas the solution of a roundabout is much simpler with traffic lights stopping road vehicles only when the LRV is present or approaching. [**22**]

2.2.2. Signalized Roundabouts

2.2.2.1. Introduction to signal control method at roundabouts

Signalization of roundabouts was first experimented with in 1959 in the UK to prevent circulating traffic from blocking entering traffic during peak periods. With the introduction of the offside-priority rule in roundabout operation in the mid-1960s, various operation and geometric layout improvements were implemented, usually aimed at smooth operation as well as improving the performance and capacity. Nevertheless, there were still problems arising from unbalanced entry flows, which in many cases resulted in long queues causing long delays and blocking back into preceding junctions. In more recent years, a number of studies have shown that the performance of some congested roundabouts can be improved with traffic signal control. Traffic signals installed on entry approaches and on the circulatory carriageway regulate the traffic flows rather than allowing certain movements to dominate under priority control. Signals are able to keep the circulatory traffic flow fluid and hence balance and improve the roundabout capacity. [**23**]

Although yield control of entries is the default at roundabouts, when necessary, traffic circles and roundabouts have been signalized by metering one or more entries, or signalizing the circulatory roadway at each entry. Roundabouts should never be planned for metering or signalization. However, unexpected demand may dictate the need after installation. [**13**]

In many cases, signals have been installed at existing roundabouts because of perceived operational problems. These can be summarized as aspects concerning delay, capacity, safety and convenience (specifically for pedestrians and cyclists).It should be noted that very small roundabouts (including mini and compact roundabouts as described in TD 16/07 *Geometric Design of Roundabouts* (HA, 2007)) are not suitable for signalization, but successful designs have been prepared for normal roundabouts with inscribed circle diameters as small as 50 meters. [**24**].

The US Federal Highway Administration (FHWA)guide describes three signalization alternatives to be considered should unexpected demand suggest the need for signals:

(1) Metering,

(2) Nearby pedestrian signals, and

(3) Full signalization of the circulatory roadway. [**25**].

The signals can be installed full time or part time. Where problems occur at roundabouts only under certain conditions, primarily at peak hours, it has been common to implement signal control on a part-time basis. These alternatives have their own advantages and disadvantages. Also what works for a certain roundabout might not work for another. The concept of a roundabout encompasses a wide range of junctions varying in size, complexity and traffic loading. When traffic signals are added, the number of design considerations increases enormously and no two signalized roundabouts will be identical. [**23**]

Even though signalized roundabouts have been used to tackle a range of issues such as improving capacity, reducing delay and accidents, improving accessibility to pedestrians and cyclists, literature that indicates their use at LRT grade crossings has not been found. However, a study done by Pratelli suggests the use of signals at roundabouts to deal with mobility problems at a BHLS (Buses with a High Level of Service) roundabout grade crossing. [**26**] The realization of a line with high LOS can be achieved in two different ways: facilities can be built in order to physically separate the path of the BHLS from that of private vehicles (i.e. underpasses, new bridges) so that the two opposite flows do not interfere with each other when crossing intersections; alternatively, traffic regulation systems can be used, providing signalized roundabouts, with traffic light priority to the bus, in which the crossing of the BHLS is scheduled on one of the branches of the roundabout to avoid blocking the overall running. [**26**] This paper showed that the use of traffic control systems to manage the roundabout intersections during the passage of BHLS; is capable of ensuring excellent levels of service in mixed traffic. The following are some of the studies done on signalized roundabouts.

I. A New Traffic-Signal Control for Modern Roundabouts: Method and Application

When the circulatory roadway of a roundabout has more than two lanes, the weaving and merging of vehicles cause large traffic and safety problems. In this paper, a new method of traffic-signal control for modern roundabouts is proposed to solve problems by eliminating the conflict points and weaving sections at a roundabout with different traffic-flow rates on each approach, which normally appear in the real world. A second stop line is set exclusively for the left-turn traffic on the circulatory roadway. It is beside the first stop line on the approach. Traffic signals are installed before each stop line to eliminate the conflicts between the traffic flows on the approaches and the left-turn traffic flows on the circulatory roadway. Left-turn vehicles on the circulatory roadway will stop before red signals to avoid weaving. Equations are derived to compute the signal cycle length and the green time for each traffic flow, considering the limited queue on the circulatory roadway. Capacity and delay are also formulated to evaluate the roundabout's performance. This traffic-signal control has a successful application of a roundabout in Xiamen, China, to solve the very serious traffic-congestion problem. The signal-timing scheme was computed with the proposed equations, as well as the capacity and delay. Pictures taken before and after the improvement show the operation. After the improvement, the roundabout capacity increases 72.1% and the average delay of each vehicle decreases by 20 s. [**27**]

II. Association of Signal-Controlled Method at Roundabout and Delay

The uncontrolled roundabout is only applicable to smaller vehicle flows. With increasing of roundabout flows, series of problems have occurred in respect of traffic jams and accidents. Arrangement of signal control will help to improve delay and safety at roundabout while differences exist between condition of adaptability and traffic efficiency under different signal controlling methods. It is on the basis of "The Secondary Control Theory on Left-turn at Circular Road" that this text is aimed at mainly introducing the two different control methods of "Control of Orderly Discharge Clockwise at Single Entrance Lanes" and "Control of Synergistic Discharge at Multi entrance Lanes". Everything is done in consideration of roundabout with different amount of entrance lanes and based on the control theory of traffic signal and the optimization method. In this foundation, by applying methods of numeric calculation and experimental traffic engineering, the authors analyzed the influence law on the average delay of vehicles caused by the signal cycle of time-space strategic parameters and radius of Central Island. The research result is useful when choosing signal control method of roundabout. [**28**]

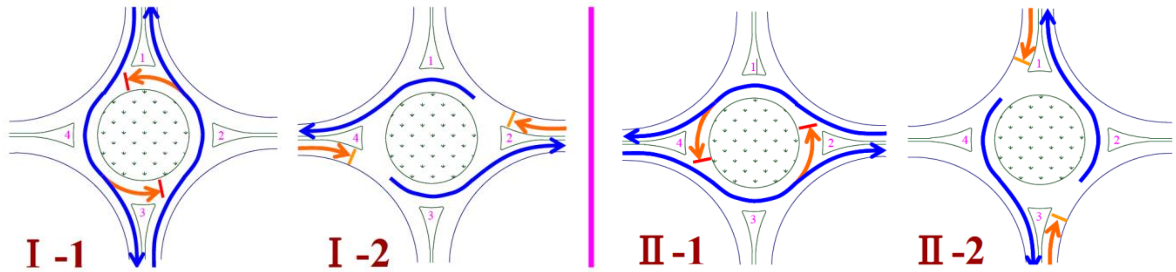


Figure 2 Schematic diagram for signalization of four leg roundabout

III. Multi-level traffic control at large four-leg roundabouts

Yield control and full signalization are typical traffic control solutions that can be used at large roundabouts. In the face of increasing congestion issues, it is preferred to use yield control during off-peak periods and full signalization during peak periods. To automatically accommodate time-varying vehicular demands, a multi-level traffic control (MTC) is developed to implement hybrid yield control and fully actuated control at large four-leg roundabouts. With new application of traffic control devices and traffic detection system, the right-of-way can be assigned to entering and circulating vehicles in three modes. The ‘all entering’ mode is equivalent to a yield control. The ‘no entering’ and ‘concurrent entering’ modes are equivalent to a fully actuated control. On the basis of time headways and occupancy times that are detected on the entry and circulatory roadways, the mode of right-of-way assignment can be changed in response to actual traffic conditions.

For a specific mode of right-of-way assignment, traffic signal operation is managed by some detectable traffic events that are happening. The results of the simulation experiments conducted by VISSIM indicated that: (i) MTC was stabilized at the ‘all entering’ mode during off-peak periods and at the ‘concurrent entering’ mode during peak periods; (ii) MTC would typically change the mode of right-of-way assignment according to actual traffic conditions as vehicular demands increased from off-peak to peak or decreased from peak to off-peak; and (iii) statistically speaking, MTC inherited the operational advantages of yield control and fully actuated control, and could be effective in improving the operational performance of large four-leg roundabouts for all hours of the day, regardless of the level of left-turn ratios. [**4**]

2.2.2.2. Studies on Capacity and Delay of Signalized Roundabouts

I. Development of Vehicle Delay and Queue Length Models for Adaptive Traffic Control at Signalized Roundabout

Vehicle delay and queue length models are important indexes to optimize signal timing plan for a signalized roundabout. However, at present much attention is paid on unsignalized roundabouts. In this paper, the authors first analyzed the impacts of phasing schemes on vehicle movements and brought forward two typical phasing schemes. The loop detector layout plan was established to detect vehicle volumes of different streams in real time. Then under each phasing scheme, the models for average vehicle delay and queue length were developed respectively. Finally, case study was conducted to evaluate the models using field data collected from a real signalized roundabout. The results show that: all precision errors are smaller than 15% and average precision errors are smaller than 10%. The developed two types of models can satisfy the requirements of engineering. [**29**]

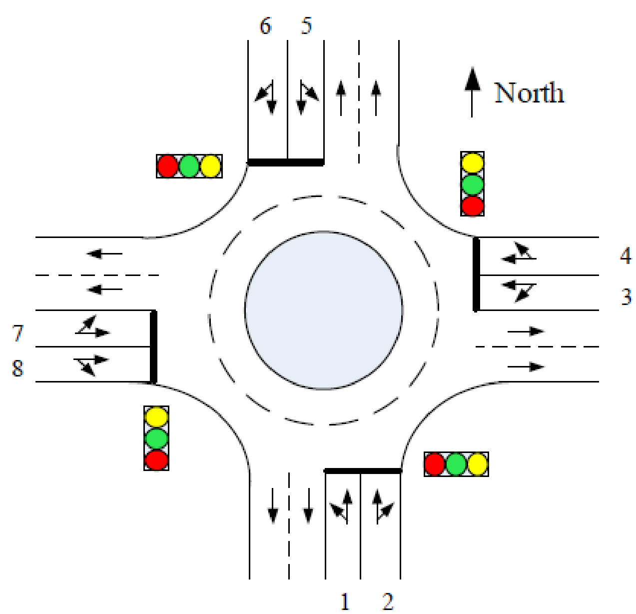


Figure 3 Signalized roundabout with four arms

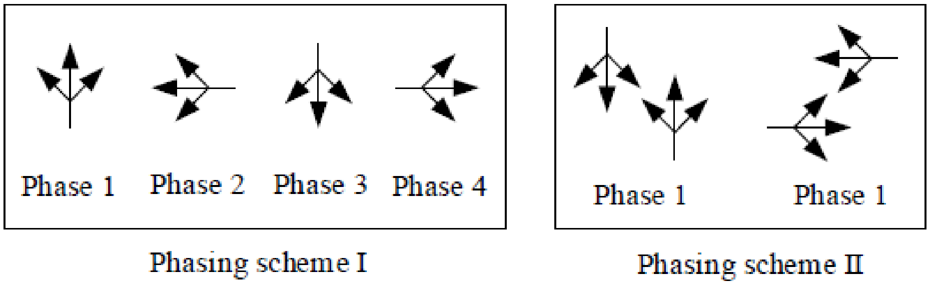


Figure 4 Two typical phasing schemes for the signalized roundabout with four arms

IV. Research on Capacity Model for Large Signalized Roundabouts

In order to calculate the capacity of signalized roundabout, this paper establishes the corresponding calculation model. Based on gap acceptance theory, the model defines that the maximum number of vehicles on the approach that can be imported into the island is the roundabout capacity. At the same time, some related parameters (such as cycle, green time) are considered to impact the model. At first, the paper analyses the difference of traffic flow characteristics between signalized roundabout and non-signalized roundabout, and according to the vehicles conflict traits, the movement process is divided into two stages. Then the new capacity model of signalized roundabout is put forward. Moreover analysis of the influence on capacity caused by different follow-up time, critical gap and green time are made, too. Finally, taking Changchun's signalized roundabout as an example, using the actual signal timing scheme and the data of the flow to verify the new capacity model,. It turns out that the results match the actual situation properly. In summary, a method of calculating the capacity of signalized roundabout is put forward, and proved to be practical in this paper. [**30**]

2.2.2.3. Use of Signalized Roundabouts in LRT networks

The integration of buses with a high level of service in the medium cities urban context

The realization of a line with high level of service can be achieved in two different ways: important facilities can be built in order to physically separate the path of the BHLS from that of private vehicles (i.e. underpasses, new bridges) so that the two opposite flows do not interfere each one with the other when crossing intersections; alternatively, traffic regulation systems can be used, providing signalized roundabouts, with traffic light priority to the bus, in which the crossing of the BHLS is scheduled on one of the branches of the roundabout to avoid to block the overall running, or in its central area. So, in order to design a signalized roundabout, first of all the branches lanes must be specialized in order to block only the maneuvers in conflict with the BHLS crossing.

In order to check the intersection operation and to verify that the private car traffic does not suffer excessive penalties two types of tests were performed. The first type concerns the checking of the roundabout in its conventional running, which was performed using the method of SETRA for big roundabouts with the external diameter greater than 40 m, and the CETUR method for smaller ones. The other type regards the checking of the traffic signal that was performed using the program HCS+.

This paper showed that the use of traffic control systems to manage the roundabout intersections during the passage of BHLS is capable of ensuring excellent levels of service in mixed traffic, which does not undergo excessive penalties, and allows creating lines characterized by high performance, because of low delays at signalized intersections thanks to the controlled priority. Moreover, by using traffic light control systems, physical devices to separate the two opposing flows are avoided. [**26**]

2.2.3. Signalized intersections

Intersections can be categorized into four major types; Simple, flared, channelized intersections and roundabouts. Simple intersections maintain the street’s typical cross-section and number of lanes throughout the intersection, on both the major and minor streets. Simple intersections are best-suited to locations where auxiliary (turning) lanes are not needed to achieve the desired level of-service, or are infeasible due to nearby constraints. Flared intersections expand the cross-section of the street (main, cross or both). The flaring is often done to accommodate a left-turn lane, so that left-turning bicycles and motor vehicles are removed from the through-traffic stream to increase capacity at high-volume locations, and safety on higher speed streets. Channelized intersections use pavement markings or raised islands to designate the intended vehicle paths. The most frequent use is for right turns, particularly when accompanied by an auxiliary right-turn lane. [**12**]

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 these results in a delay to vehicles in all streams, it is important that traffic signals be used only when necessary. The most important factor that determines the need for traffic signals at a particular intersection is the intersection's approach traffic volume, although other factors such as pedestrian volume and crash experience may also play a significant role. The Manual on Traffic Signal Design (MUTCD) describes eight warrants in detail, at least one of which should be satisfied for an intersection to be signalized. However, these warrants should be considered only as a guide; professional judgment based on experience also should be used to decide whether or not an intersection should be signalized.

Different manuals propose methods of signal timing. The NCHRP signal timing manual uses an outcome based process to time signals. The outcome based process is a modern approach to signal timing and reflects the complex nature of many signalized intersections and operating environments. It encourages practitioners to consider all system users (e.g., pedestrians, bicycles, motor vehicles, emergency vehicles, transit, and rail) and to establish user priorities by movement for each signalized intersection location. The focus is applying those elements to the selection of operational objectives that clearly define desired signal timing outcomes. In order to assess the effectiveness of a signal timing plan, performance measures must also be identified for each objective. Performance measures can be used to determine initial success, as well as to monitor and sustain desired outcomes throughout the life of a signal.

2.3. Comparison Studies of at grade junctions

2.3.1. Optimal Traffic Signal Timing Allocation for Urban Congested Roundabouts

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

This study compared three different at grade junction designs; roundabout, signalized intersection and signalized roundabout, at the three locations mentioned above. The study utilized spot speed study, traffic and pedestrian volume study and roundabout geometric study to model the study areas in VISSIM and VISTRO simulation software. Signal timing design and approaching saturation flow rates were computed by using HCM 2000.

The study found that Roundabouts have excess control delay and higher travel time relative to the alternatives. Alternative of converting the existing roundabout to signalized intersection and signalized roundabout reduce control delay by 34% and 47%, respectively, under base year conditions and 52% and 69% under forecasted condition for peak hour traffic flow. This study proposed that study area roundabouts should be supported by traffic signal timing (roundabout signalization) or should be demolished and reconstructed as signalized intersections. [**31**]

2.3.2. Efficiency of Roundabouts as Compared to Traffic Light Controlled Intersections in urban Road Networks

This study compared three at grade junctions; roundabout, roundabout with traffic lights (signalized roundabout) and signalized intersection using queue length as a performance measure under varying arrival rates. A Multi-stream Minimum Acceptable Space (MMAS) Cellular Automata (CA) model is used for the simulation of vehicular traffic at double-lane roundabouts and cross intersection. The study found that Roundabout without traffic light performs best when the arrival load is evenly distributed on each arm of the intersection; and the more unbalanced the arrival rates are at each arm of the roundabout, the less it performs and hence longer queue lengths. Roundabout with traffic light is operationally better than roundabout without traffic light installed at its entrance. However, for arrival rates below 0.1 vps, its performance is exactly the same when traffic light is absent. The average critical arrival rate for the best performance of a roundabout with traffic light is approximately 0.64 vps per lane provided there is equal distribution of arrivals per each arm. Below this arrival rate roundabout with traffic light is found to be efficient. However, when arrival rates exceed this critical value, its performance is compromised and a queue starts to build. Thus, a signalized intersection is a better choice than a roundabout if the average arrival rate per each lane exceeds about 0.64 vps which means an arrival rate of nearly 16 vehicles per each 25 seconds. The average critical arrival rate for a double-lane cross intersection is approximately 0.8 vps per lane. [**32**]

2.3.3. Evaluation of Roundabouts versus Signalized and Unsignalized Intersecions in Delaware

This study evaluates and compares a single-lane roundabout with an un-signalized (two way stop controlled) intersection and a signalized (pre-timed) intersection and recommends conditions under which the construction of a roundabout may be more appropriate for an intersection. The measures of effectiveness used for the comparisons were effective intersection capacity, major and minor road entry lane capacity, major and minor road average delay, major and minor road queue length, and emission rates (CO, NOX, HC, and CO2). These measures are provided by the aaSIDRA software package which could then be used to establish “thresholds” in the major road one-way volume. These volumes indicate the threshold values for which the roundabout performs better than the un-signalized and signalized intersections. Two single-lane roundabouts with medium to high traffic volumes were studied in order to obtain local driver gap acceptance characteristics for use in the SIDRA analyses.

The study considered two heavy vehicle percentage scenarios, 10% and 20%. Based on the weighted averages of the measures of effectiveness mentioned above, the study found that for10% heavy vehicles on both roads, the roundabout range of one way major road is between 150 and 850 vph. When heavy vehicles on the major road are increased to 20%, this range shifts slightly to between 140 and 840 vph. A signalized intersection is estimated to maintain high weighted measure of effectiveness for one way major road volumes of 840 vph and above. [**33**]

2.3.4. Unsignalized versus signalized roundabouts under critical traffic conditions: A quantitative comparison

This paper focuses on evaluating roundabout performance with or without signal control in critical traffic situations. The performance of two congested roundabouts was tested by gradually increasing traffic volumes until the first over saturation episode was reached on the most critical leg of the roundabout. The study found that at low traffic volumes, queuing delays are really negligible and an un-signalized roundabout performs better if compared to a signalized one; on the contrary, at high traffic volumes, a signalized roundabout produces more balanced and limited delays to the vehicle of all approaching legs, and could improve the overall capacity.

The study recommends that in order to reach the benefits of both signalized and un-signalized roundabouts, the signalized solution could be adopted during peak hours whilst the facility could regulate itself when charged by low traffic volumes. [**34**]

2.4. Gaps in Previous Research

Even though various studies have been done to compare at grade junction designs, there is limited research for junctions with a median LRT crossing and also limited research comparing signalized intersection and signalized roundabouts to conventional roundabouts. Therefore it is the objective of this study to do so. Previous studies done on the same locations have studied the effects of the LRT line on the existing roundabout and these studies have indicated that the introduction of the LRT line brings about additional delay and deteriorations in level of service [**2**,**3**]. This study focuses on attempting to reduce these effects by considering alternative at grade junction designs.

CHAPTER THREE: RESEARCH METHODOLOGY

3.1. Study Areas

The study areas are two locations within the Addis Ababa East – West LRT corridor, specifically; Sealite Mehert and CMC roundabouts. These roundabouts are both at grade with LRT lines passing through the center island.

The characteristics of these roundabouts are the following.

- ✓ Surrounded by commercial and residential areas
- ✓ At grade LRT crossing through Median
- ✓ LRT station located at median of one approach to the roundabout which acts as a bottleneck for exiting and entering vehicles (approximately 200m from Sealite Mehret and 125m from CMC roundabout yield line)
- ✓ High Passenger volumes boarding and exiting at the station.
- ✓ Nearby school on East approach of CMC roundabout which is a cause of road side parking, adds to traffic congestion.
- ✓ The two roundabouts are approximately 2.3kms apart and there are 4 LRT stations between the roundabouts.

The two roundabouts are going to be studied as two isolated roundabouts because there is a long distance between them, and traffic changes in one roundabout does not affect the other.

3.2. Study Design

During the initial stages of the study, the aim was to understand current performance of the roundabouts to create a baseline for the next stages. The subsequent stages involved comparison of alternative designs to the current roundabout. The alternatives and the existing roundabout were modeled using VISSIM simulation software and compared based on queue length, level of service, travel time and delay. The study framework will be as follows.

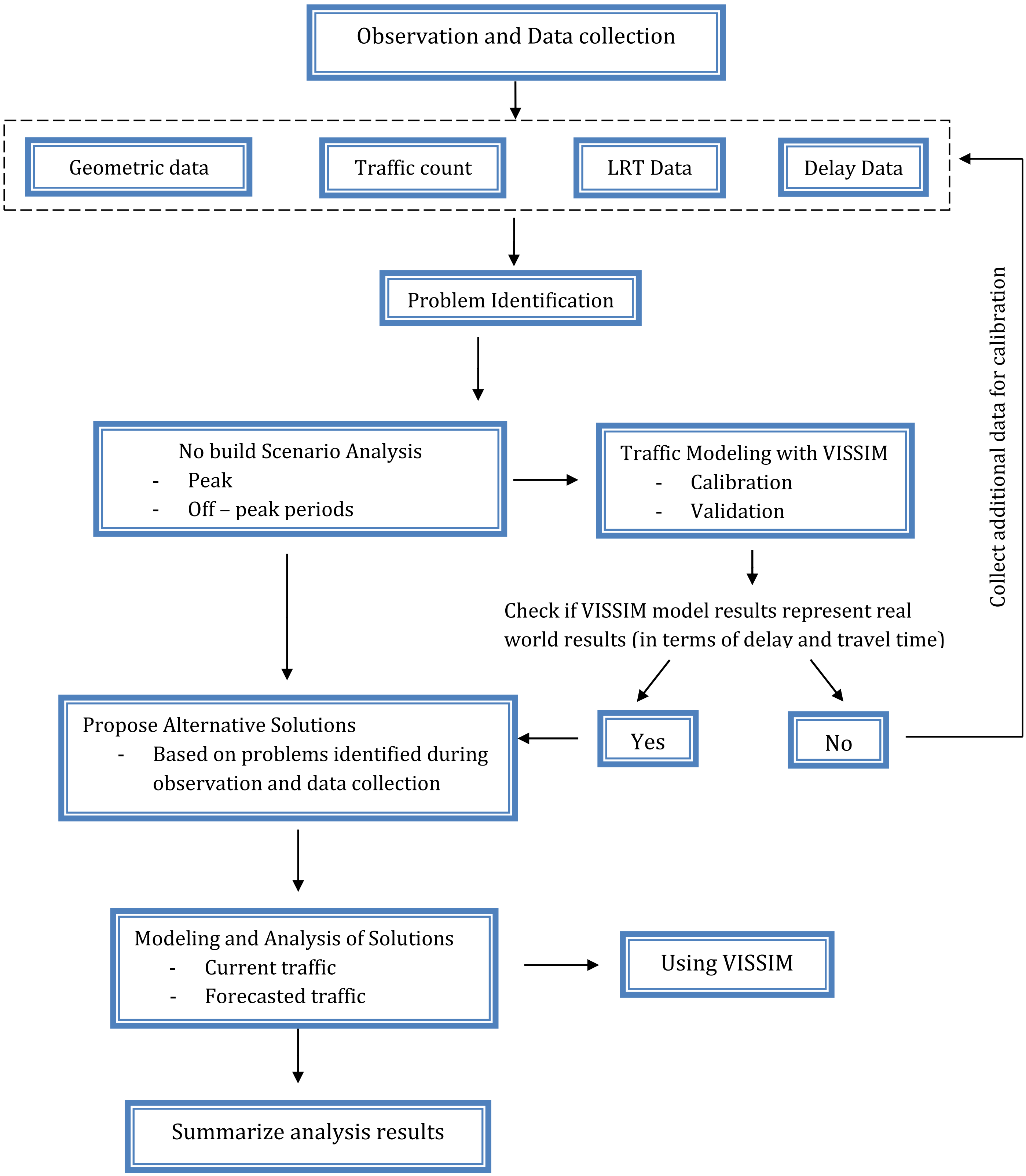


Figure 5 Study Design

3.3. Data collection

3.3.1. Geometric data

This study attempts to compare alternative junction designs with existing roundabouts in terms of average vehicle delay, queue length and LOS. As mentioned above the geometric elements of the roundabout affect the rate of entry flow and thus capacity. The most important geometric elements are the width of the entry, the width of the circulatory roadway, or number of lanes around the central island. [**15**].The method used is collection of secondary data in the form of design documents obtained from AACRA and Google Earth maps.

Table 1 Geometric Data [**35**]

Geometric Elements	Sealite Mehret roundabout	CMC roundabout
Number of legs	4	4
Number of lanes along Major road (East-West)	4	4
Number of lanes along Minor road (North- South)	2	2
Number of circulating lanes	4	4
Number of lanes at entry of roundabout from minor roads	3	2
Angle of intersection (degrees)	60	90
Width of Entry and exit lanes		
North Approach	3.7m	3.5m
South Approach	4.3m	4m
East Approach	3.5m	3.5m
West Approach	3.5m	3.5m
Width of circulating road	14m	14m
Shape of central island	elliptical	circular
Diameter of central island	60m, 100m	125m
Median width	11m	11m
Grade		
North Approach	-4%	1.5%
South Approach	7%	3%
East Approach	2%	1.5%
West Approach	2%	1.5%

3.3.2. Traffic Data

3.3.2.1. Traffic Operations

CMC roundabout is surrounded by residential and commercial areas, giving rise to traffic congestion especially during morning peak hours, as shown in Figure 6. There are four legs joining at the roundabout. Morning peak hour traffic mainly comes from Ayat and Summit approaches. While evening peak hour traffic mainly comes from Megenagna Approach. Even though there is significant traffic coming from and exiting to Summit leg, there are only two entry and exit lanes each. Furthermore, parking is permitted at entry and exit of every leg adding to the congestion of the area. Heavy parking is observed on Ayat entry approach, at a nearby school.

There is a median LRT line crossing the intersection in both directions and a station at 125m from CMC roundabout. High pedestrian volume is observed at the station with no pedestrian signal to facilitate traffic. Traffic police intervene at morning peak hours to reduce queuing of vehicles blocked by pedestrian traffic at the station. Illegal pedestrian crossing and U turns are also observed at the west (right side) LRT grade crossing.

Figure 6 Morning peak hour, CMC roundabout

During morning peak hours operating speed of vehicles is generally low. The Google traffic map below shows typical traffic on a Tuesday at 7:30 AM. The slowest part of the roundabout is the approach from Ayat and the exit leg to Megenagna. The red lines on the map mean highway traffic is moving at less than 40.2 Km per hour and could indicate an accident or congestion on that route. While the Red-black line on the map indicates extremely slow or stopped traffic. [**36**]

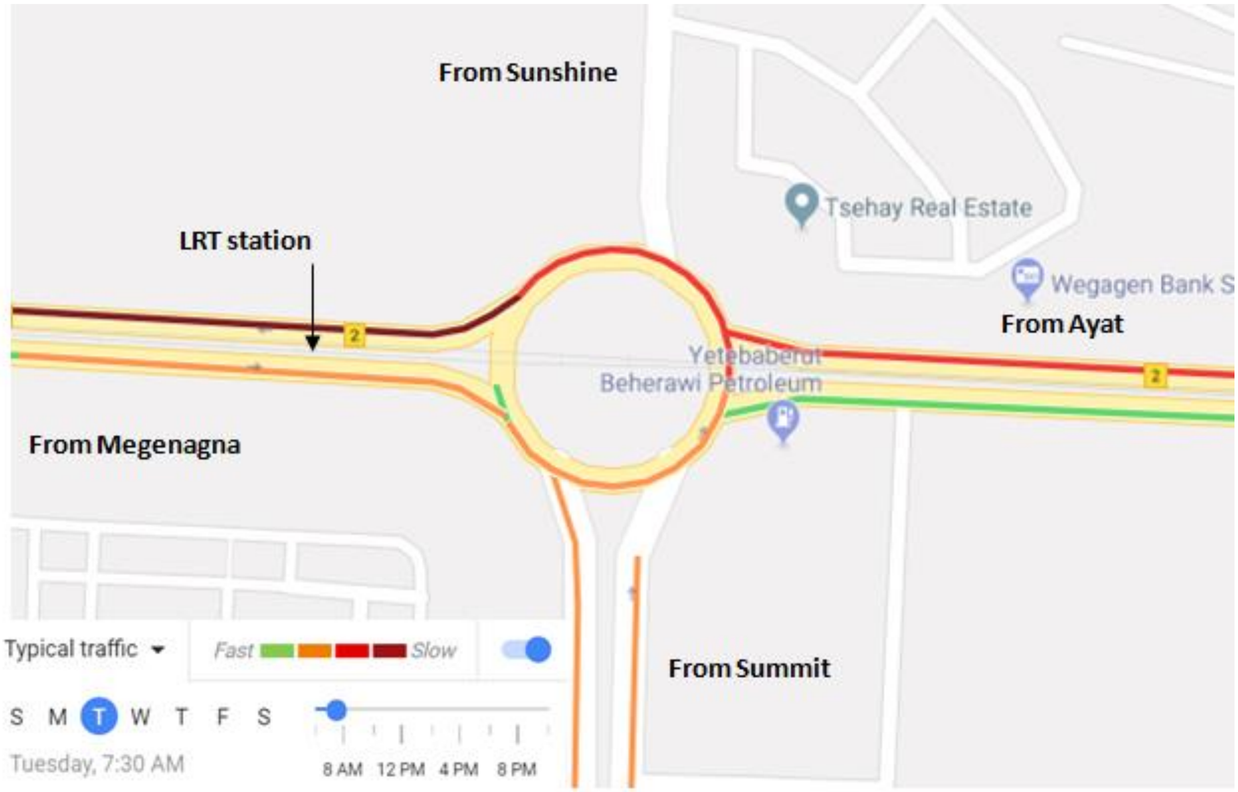


Figure 7 Typical Morning peak hour traffic speed, CMC roundabout [**36**]

Similar to CMC roundabout, Sealite Mehret roundabout is surrounded by residential and commercial areas. Traffic congestion is observed during morning, midday and evening peak hours. The roundabout has four legs. Heaviest traffic comes from CMC and Mebrat Hail approaches during morning peak hour. The roundabout gets highly congested without the intervention of traffic police. Parking is permitted on three approaches for public transportation.

Figure 8 Morning Peak hour, Sealite Mehret roundabout

Figure 9 Typical Morning traffic, Sealite Mehret roundabout

There is an LRT station approximately 200m from the roundabout. High pedestrian volume is observed at the station with no pedestrian signal to facilitate traffic. This significantly adds to the congestion of CMC approach marked in red-black line in Figure 9, which indicates extremely slow or stopped traffic. The Google traffic map (Figure 9) shows typical traffic on a Thursday at 8:35 AM.

3.3.2.2. Traffic Count

In order to compare different types of junction designs in terms of queue length and delay, inputs such as peak hour volume and turning movement ratios are necessary. Peak Hour Volume (PHV) is the maximum number of vehicles that pass a point on a highway during a period of 60 consecutive minutes. PHVs are used for design of the geometric characteristics of a highway, for example, number of lanes, intersection signalization, or channelization and Capacity analysis among others. [**37**]

The method used to collect peak hour volume data is traffic study with the help of video recording. For the purpose of collecting peak hour volume the two study areas were videotaped for duration of 6 hours during morning peak hour, midday off-peak hour and evening peak hours, on a typical week day (Tuesday and Wednesday) during good weather conditions. According to Handbook of Simplified Practice for Traffic Studies and MUTCD Intersection turning movement counts, count duration of 2hours is sufficient for turning movement studies. [**38**,**39**]

In order to determine the traffic variation of the study areas previous studies are viewed. A study done on the same study areas by Berhanu G/Yohannes, determines the AADT for seven days traversing through the roundabout. [**40**] From these AADT values, the directional design hour volume (DDHV) can be determined using the following equation.

$$DDHV = AADT * K * D \tag{1}$$

Where

K= proportion of daily traffic occurring during the peak hour, generally equals 0.12 for suburban and urban areas.

D= proportion of peak hour traffic traveling in the peak direction of flow.

The DDHV for the study areas are as follows, for K value of 0.12 and for D value of 1, since the AADT count was done separately at two directions on the roundabout.

Table 2 Directional design hour volumes (veh/hr) for Sealite Mehret and CMC roundabouts

Location	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday	Sunday
Sealite Mehret	1444.2	1436.64	1566.6	1544.52	1464.84	1310.28	1109.52
CMC	1030.56	1019.16	969.72	973.32	872.76	970.2	893.64

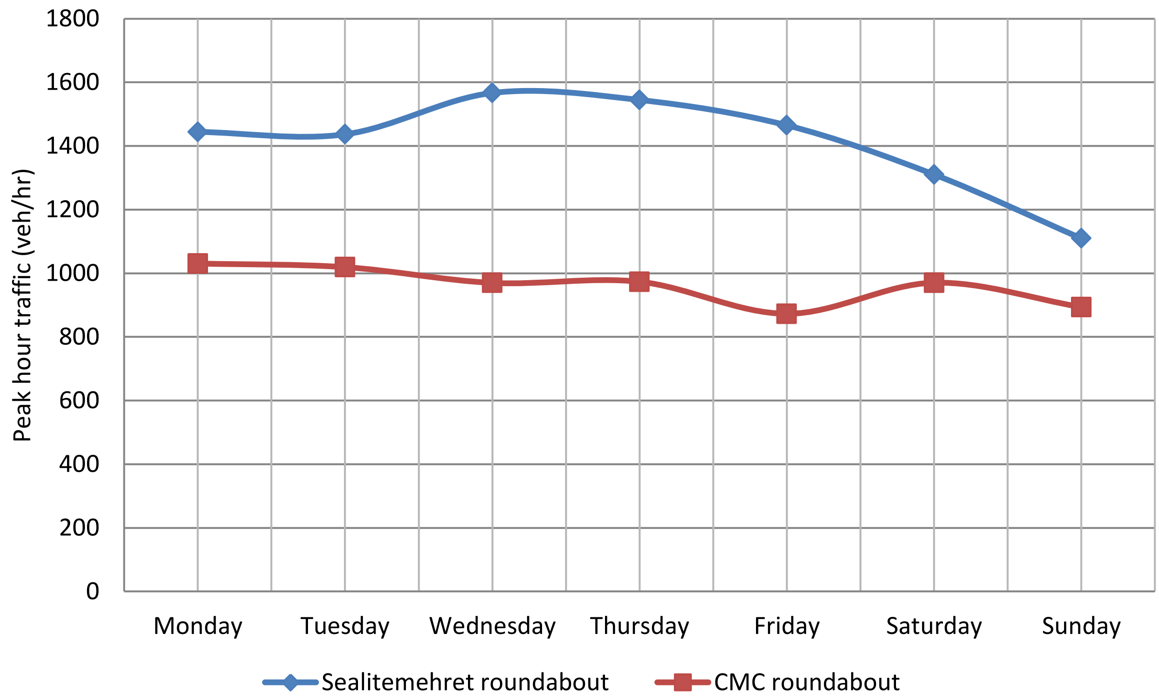


Figure 10 Daily Directional design hour volumes (veh/hr) for Sealite Mehret and CMC roundabouts

Assuming a normal distribution, minimum number of days that traffic must be counted is calculated below. To determine typical traffic of the area typical week days are considered: Tuesday, Wednesday and Thursday. The standard deviation of the peak hour traffic is first determined for these days, and then the minimum sample size is calculated for 95% confidence interval and an error of 10%.

$$N = \left(\frac{Z\sigma}{d}\right)^2 \qquad (2)$$

Where, N= minimum sample size

Z= number of standard deviations corresponding to the required confidence level, 1.96 for 95 percent confidence level.

σ= standard deviation

d= limit of acceptable error

Table 3 Minimum number of days to do traffic count

Location	Tuesday	Wednesday	Thursday	σ	d		N
Sealite Mehret	1436.64	1566.6	1544.52	69.54	10.00%	149.14	1
CMC	1019.16	969.72	973.32	27.56	10.00%	98.74	1

Therefore, peak hour traffic data must be collected for one day at both locations.

Directly counting each turning movement is difficult and time consuming. It is almost impossible to track every vehicle from entry to exit at such large roundabouts. Therefore, in order to determine turning movements, simultaneous equations were used. Counts were done at every entry and four points of the circulating carriageway. The exit volumes were determined from the entry and circulating counts. This leaves twelve knowns but sixteen unknowns, including U turns. Therefore, right turns, which are easier to track, needed to be counted first. Once the right turns were determined there were twelve unknowns left, which leads to the twelve simultaneous equations in Equations 3. The knwons for entry, exit and circulating counts were substituted then the equations were solved by converting to a 12X12 augmented matrix. The augmented matrices were solved using Scilab 6.0.1.

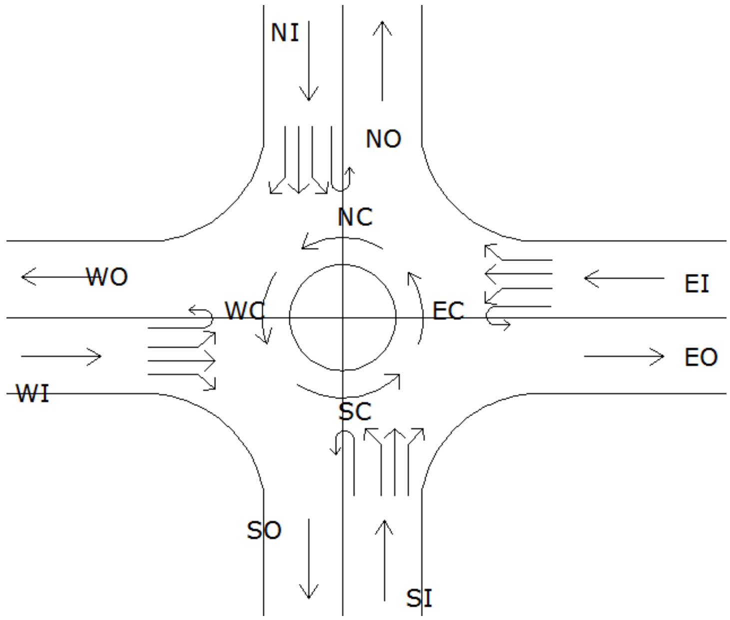


Figure 11 Entry, Exit and Circulating movements

L- left, R- right, T- through, U- U turn

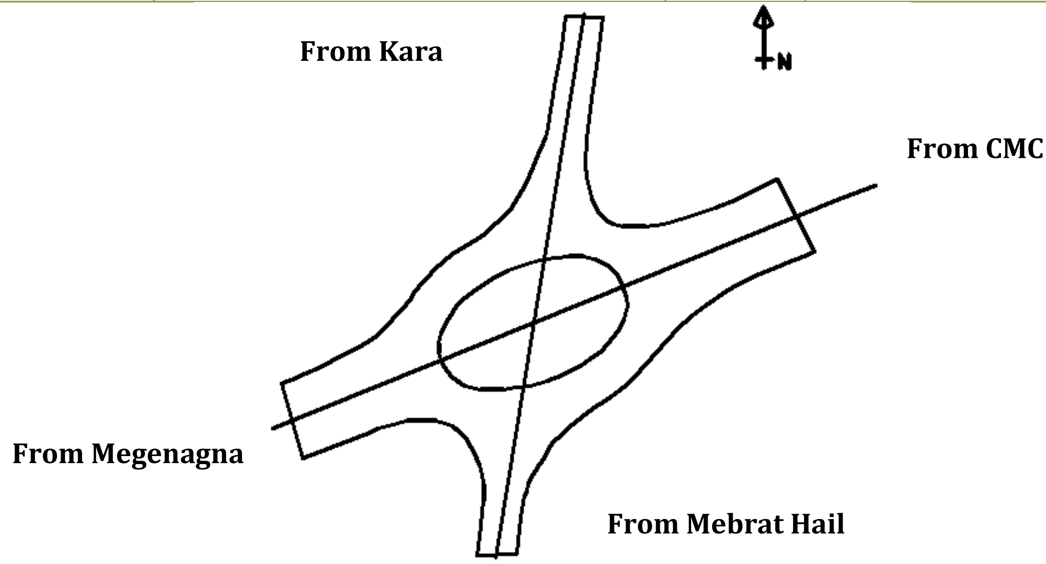
$$
\left.\begin{aligned}
EI &= EL + ER + ET + EU \\
EC &= WU + ST + WL + SL + NU + SU \\
EO &= SR + EU + WT + NL \\
WI &= WL + WR + WT + WU \\
WC &= NL + EL + NT + EU + NU + SU \\
WO &= NR + WU + SL + ET \\
NI &= NL + NR + NT + NU \\
NC &= EL + SL + WU + EU + ET + SU \\
NO &= ST + ER + WL + NU \\
SI &= SL + SR + ST + SU \\
SC &= WL + EU + NL + WT + WU + NU \\
SO &= WR + NT + EL + SU
\end{aligned}\right\} \quad (3)
$$

Accordingly, the following traffic data were collected. The largest traffic flow occurs during morning peak hour, from 7:00AM to 9:00AM for both locations, specifically from 7:15AM – 8:15AM for CMC and 7:45AM – 8:45AM for Sealite Mehret roundabout.

Table 4 Traffic count, Sealite Mehret Roundabout

Turning movements

General information		Site Information	
Date	13/6/2018	Location	Sealite Mehret Roundabout
Weather condition	Good	City	Addis Ababa , Ethiopia

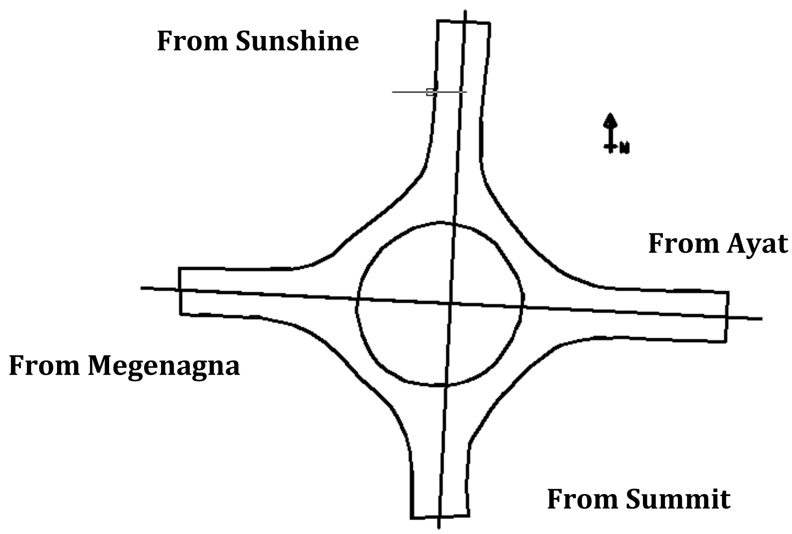


Count Period	Approach	Movement				Total (veh/hr)	veh/hr/lane	HV %
		Left	Through	Right	U turn			
Morning peak hour	CMC	1011	1126	477	86	2700	675	10%
7:00 AM - 9:00 AM	Megenagna	60	868	104	128	1160	290	7%
	Kara	68	128	1040	16	1252	626	3%
	Mebrat Hail	1080	156	224	180	1640	820	5%
Off peak hour	CMC	277	292	385	508	1462	365	7%
2:00 PM - 4:00 PM	Megenagna	215	492	862	600	2169	542	3%
	Kara	154	77	569	15	815	408	5%
	Mebrat Hail	38	77	723	11	849	425	8%
Evening peak hour	CMC	176	480	377	476	1509	377	8%
5:00 PM - 7:00 PM	Megenagna	396	708	936	432	2472	618	7%
	Kara	300	80	660	100	1140	570	4%
	Mebrat Hail	64	104	692	12	872	436	6%

Table 5 Traffic count, CMC roundabout

Turning movements

General information		Site Information	
Date	7/6/2018	Location	CMC Roundabout
Weather condition	Good	City	Addis Ababa , Ethiopia



Count Period	Approach	Movement				Total (veh/hr)	veh/hr/lane	HV %
		Left	Through	Right	U turn			
Morning peak hour	Ayat	373	1183	613	288	2457	614	11%
7:00 AM - 9:00 AM	Megenagna	22	202	498	823	1545	386	5%
	Sunshine	43	65	618	18	744	372	6%
	Summit	849	48	438	43	1378	689	6%
Off peak hour	Ayat	116	244	136	364	860	215	12%
2:00 PM - 4:00 PM	Megenagna	92	136	424	408	1060	265	7%
	Sunshine	36	44	332	10	422	211	10%
	Summit	196	48	300	64	608	304	12%
Evening peak hour	Ayat	32	252	108	628	1020	255	9%
5:00 PM - 7:00 PM	Megenagna	340	688	888	580	2496	624	6%
	Sunshine	84	108	376	20	588	294	4%
	Summit	224	120	692	16	1052	526	5%

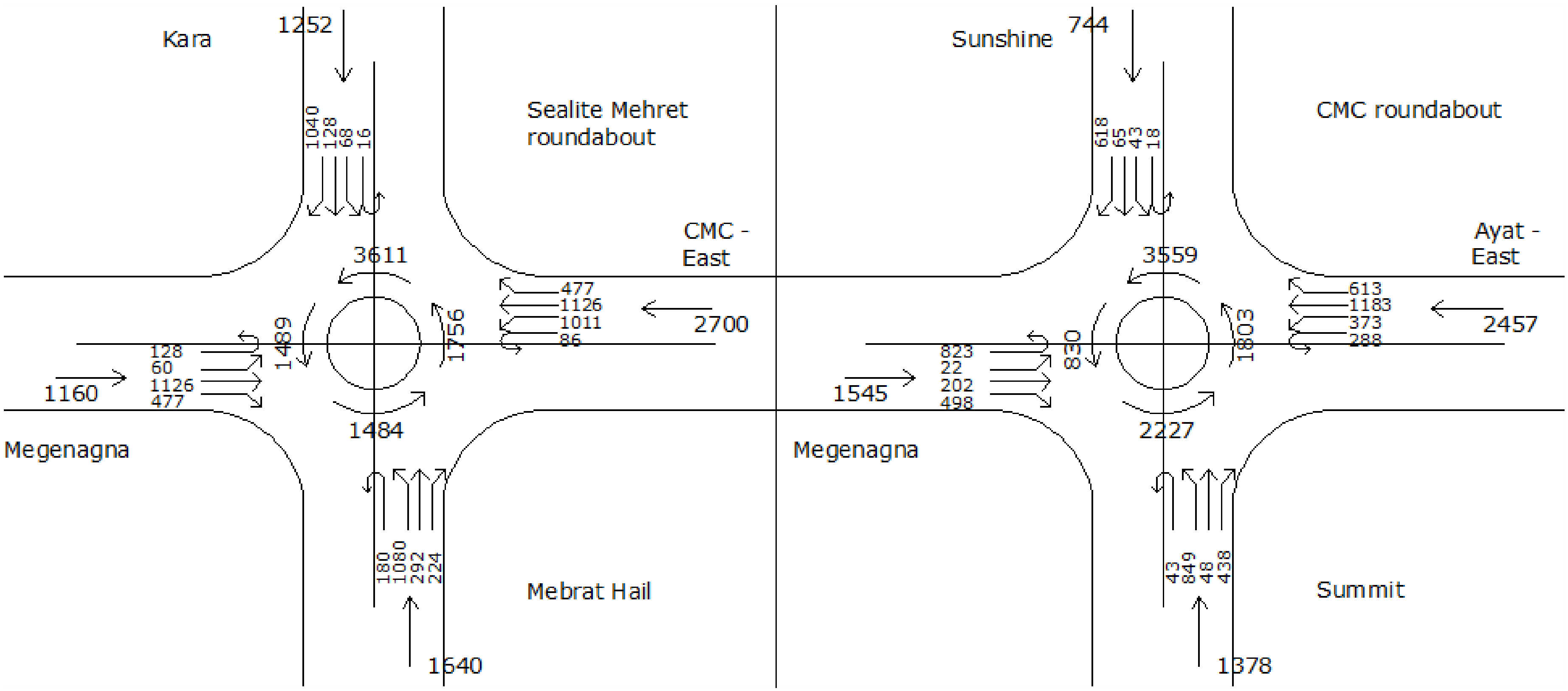


Figure 12 Vehicle Movement Distribution for morning peak hours

3.3.3. Speed Study

Speed studies were done for the study areas to estimate

- approach speeds along the major road during free flow conditions,
- circulating speeds around the roundabouts at peak and off-peak periods,
- Speeds at reduced speed areas of LRT grade crossings.

Vehicles cross the LRT line with reduced speeds. The road is built with a small bump at the LRT grade crossing. The speed data for these areas were collected during off peak hours.

The speed data were collected using a stopwatch as vehicles traverse a road segment of known length. The basic assumption made in determining the minimum sample size for speed studies is that the normal distribution describes the speed distribution over a given section of highway. [**37**] A minimum sample size of at least 30 is recommended. The standard deviation was first estimated from a small sample size of ten to determine minimum sample size for 95% confidence interval. The raw data is included in Appendix A.

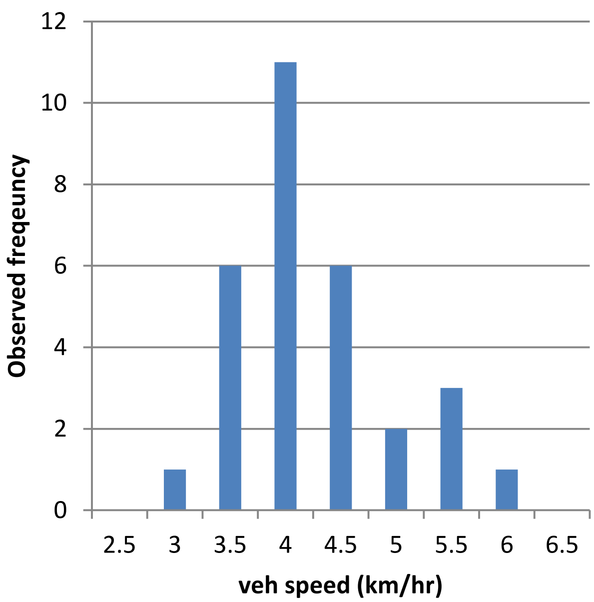


Figure 13 Histogram of Peak hour circulating speeds, CMC roundabout

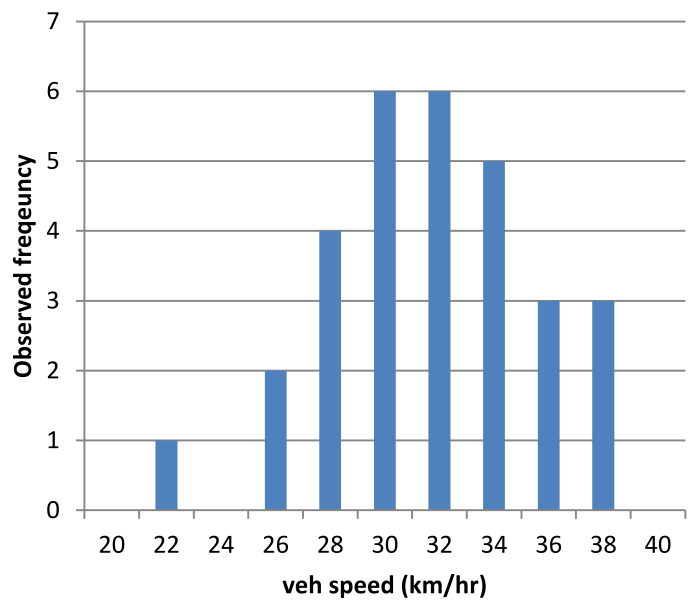


Figure 14 Histogram of Off-Peak hour circulating speeds, CMC roundabout

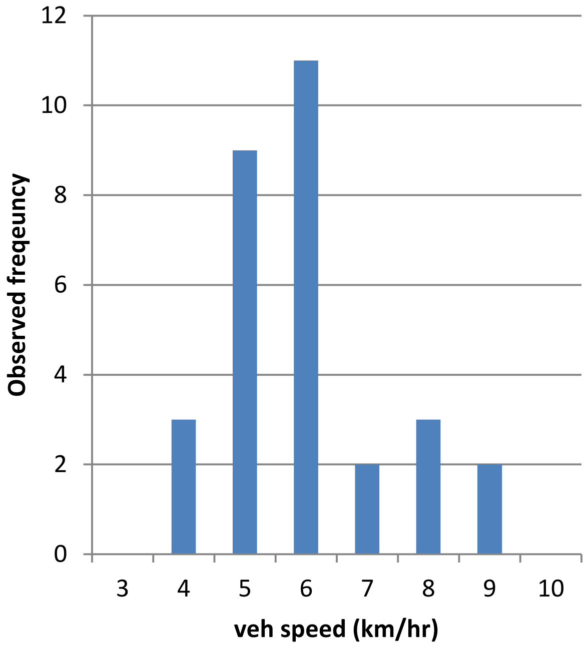


Figure 15 Histogram of Speeds at level crossings

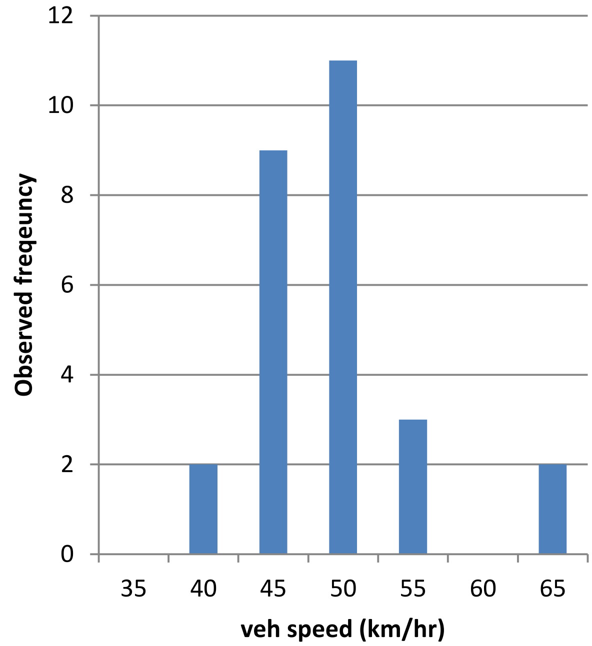


Figure 16 Approach Speeds along major road

3.3.4. Travel time and Delay

Travel time studies involve recording the time it takes vehicles to traverse a specified length of roadway. Travel time data is often reported in terms of delay (travel time in excess of free-flow, unimpeded travel time) or of average speeds in links or zones. Delay is considered to be excess time spent in the segment above what would be spent if travel were free-flow. The base free flow speed may be determined empirically, using the 50th percentile speed of low volume traffic (<1300 pc ph Pl), or may be taken as either the posted limit or some reasonable lower figure. [**41**] The method used to conduct travel time study is average speed technique. This technique involves driving the test car along the length of the test section at a speed that, in the opinion of the driver, is the average speed of the traffic stream. [**37**] Assuming normal distribution for travel times, the test run is repeated for the minimum number of times determined by minimum sample size equation (Equation 2). Travel time is recorded as the test vehicle traverses the test section. For both roundabouts the test section is selected to be long enough to include the end of queues, for CMC the test section is 0.8 km long and for Sealite Mehret it is 1km.

Table 6 Travel time for CMC roundabout

			No.	Speed (km/hr)	Travel time (min)
Standard deviation	2.21				
Acceptable error (min)	2.0		1	3.15	15.2
Minimum sample size	5		2	3.52	13.6
Average (min)	12.33		3	3.70	13.0
Median (min)	12.62		4	3.92	12.3
Maximum (min)	15.25		5	4.36	11.0
Minimum (min)	8.85		6	5.42	8.8

Table 7 Travel time for Sealite Mehret roundabout

			No.	Speed (km/hr)	Travel time (min)
Standard deviation	3.39				
Acceptable error (min)	3.00		1	4.58	13.1
Minimum sample size	5		2	8.11	7.4
Average (min)	12.02		3	4.2	14.3
Median (min)	13.30		4	7.32	8.2
Maximum (min)	15.60		5	3.85	15.6
Minimum (min)	7.40		6	4.44	13.5

Table 8 Delay at CMC and Sealite Mehret roundabouts

Roundabout	**Segment**	**Distance**	**Free flow speed (km/hr)**	**Ideal travel time (min)**	**Total**	**Median travel time (min)**	**Delay (min)**
Sealite Mehret	Entry to roundabout	0.63	48	0.79	1.34	13.1	11.76
	Circulating lane	0.15	31	0.29			
	Exit from roundabout	0.22	50	0.26			
CMC	Entry to roundabout	0.61	48	0.76	1.40	12.62	11.22
	Circulating lane	0.225	31	0.44			
	Exit from roundabout	0.165	50	0.20			

3.3.5. Pedestrian Data

Pedestrian volume counts are necessary because in urban roundabouts, pedestrian crosswalks at legs reduce entry and exit capacities in proportion to the value of the pedestrian flow. [**42**] These counts can be used for crash analysis, capacity analysis, and determining minimum signal timings at signalized intersections. [**37**]

Pedestrian volume counts can be done for the same duration as peak hour counts, in 15min intervals. [**39**] Thus, for this study counts were done for 2 hours during the morning peak period. It was observed that a few pedestrians jaywalked to reduce travelled distance, however these were negligible compared to the whole and therefore have not been included in the simulation model. The largest number of pedestrians is observed at the LRT stations, that are close to the roundabouts and affect entry and exit flows to and from the roundabouts.

Table 9 Pedestrian volumes

Approach	**Pedestrians in both directions (ped/min)**	
	Sealite Mehret roundabout	CMC roundabout
North	4	1
South	5	3
East	33	0
West	0	17

3.3.6. LRT data

The main data collected concerning the LRT line are listed below. These data have been collected in order to build a representative model of the study areas. The geometric data were measured from Google Earth maps and the rest of the data were collected on site during peak hour and off-peak periods. The following are the LRT data collected at the study areas.

- Geometric Data
- LRT crossing frequency
- Size of LRT vehicles
- Speed of LRT vehicles
- Time difference between LRT vehicles in opposite directions
- Advance warning time before LRT vehicle arrival
- Actual vehicle waiting time before LRT vehicle arrival

Before the LRT vehicle arrives, it emits warning sounds so that vehicles clear the grade crossing. However, vehicles do not obey these warnings. As shown in Table 12 there is a significant difference between the time before which vehicles should clear the grade crossing and the time they actually do. There are also gates at the grade crossings, but these gates no longer function.

Table 10 Geometric Data, measured from Google Earth Maps

LRT Data	
Location or LRT line	Median of Major road
Number of LRT Lines in each direction	1
Width of LRT corridor	11m
Width of LRT crossing	
Sealite Mehret Roundabout	20m
CMC roundabout	20m
Distance of nearby stations from	
Sealite Mehret Roundabout	200m
CMC roundabout	125m

Table 11 LRT crossing frequency during peak hour

Direction	Frequency (min)	speed (Km/hr)	Length of LRV (m)
West - East		22.50	60
	9.52	22.50	60
	12.30	27.00	30
	33.43	28.42	60
	12.40	25.71	60
	15.63	23.48	60
Median	12.40	24.60	
East West		20.00	60
	15.73	33.75	60
	11.50	23.48	60
	14.50	25.71	60

	14.83	23.48	30
	15.73	23.48	60
Median	14.83	23.48	

With 95% confidence, The LRT vehicle crosses the intersection in east – west direction, approximately every 15min ± 2min, with a speed of 24km/hr. The same is observed during off-peak periods. The time difference between the arrivals of LRT vehicles in opposite directions is approximately 7 min ± 2min, with 95% confidence. The same is observed during off peak periods.

Table 12 Time difference between LRT vehicles in opposite directions during peak hour

Direction	Arrival (time stamp on video , min)	time difference (min)
E	2.62	1.10
W	3.72	8.42
E	12.13	7.32
W	19.45	4.98
E	24.43	6.52
W	30.95	
W	45.45	12.42
E	57.87	2.42
W	60.28	9.98
E	70.27	5.75
W	76.02	9.88
E	85.90	

Table 13 Advance warning time before LRT vehicle arrival during peak period

No.	Alarm sound (time stamp on video, min)	Arrival (min)	Advance warning (min)
1	1.35	3.23	1.88
2	10.90	12.27	1.37
3	18.38	19.90	1.52
4	23.22	24.87	1.65
5	29.95	31.90	1.95
6	44.48	45.90	1.42
7	56.60	58.73	2.13
8	59.28	60.57	1.28
9	74.75	76.03	1.28

Table 14 Advance warning time before LRT vehicle arrival during off peak period

No.	Alarm sound (time stamp on video, min)	Arrival (min)	Advance warning (min)
1	9.23	10.38	1.15
2	24.27	25.72	1.45
3	28.95	30.02	1.07
4	38.07	39.03	0.97
5	46.48	47.38	0.90
6	54.28	55.57	1.28

Advance warning sounds are emitted by the LRT vehicle approximately 1min before it reaches the roundabout during off- peak periods and approximately 1.6 min during peak periods. However vehicles ignore this warning and continue to enter the grade crossing until about 9 seconds before the LRT vehicle arrives.

Table 15 Actual vehicle waiting time before LRT vehicle arrival

No.	Vehicle stopping time (time stamp on video, min)	LRV arrival (min)	waiting time(sec)
1	2.55	2.62	4
2	2.77	3.02	15
3	12.03	12.13	6
4	19.63	19.72	5
5	30.75	30.95	12
6	31.25	31.33	5
8	45.65	45.80	9
9	57.73	57.87	8
10	58.10	58.18	5
11	60.00	60.28	17
12	60.52	60.67	9
13	70.13	70.27	8
14	70.37	70.62	15
15	75.88	76.02	8
16	76.25	76.40	9
17	85.75	85.90	9
18	86.12	86.28	10

Vehicles stop entering the grade crossing approximately 9s ± 2s before the LRT vehicle arrives.

3.4. Problem Identification

Based on observation and data collection three major problems are identified at the study areas.

- Excessive delay to vehicles and queue formations

This could be due to a number of reasons. There are high volumes of vehicles coming from all directions at both roundabouts; therefore, the capacity of the roundabouts might have been exceeded. Another cause for the congestion could be excessive parking along entry and exits of the roundabouts by public transportation and private vehicles near schools. There are also LRT stations near both roundabouts with high pedestrian volumes boarding and alighting from the LRT. This continuously interrupts traffic flow. Another problem observed at the study area is the interlocking of vehicle paths at weaving sections when an LRV crosses the intersection. Furthermore, lack of U turn areas along the LRT corridor, which transfers high volumes of U turning vehicles to the roundabouts, adds to congestion of the roundabouts.

- Delay to LRT vehicle

Even though the LRV emits warning sounds well in advance of its arrival, vehicles fail to clear the intersection in time. Since the gates at the LRT grade crossing do not function, vehicles continue to enter until seconds before the LRV arrives. This not only causes delay to the LRT, it also compromises the safety of vehicles.

- Lack of safety of pedestrians and vehicles

Illegal pedestrian crossings and U turns take place at the roundabouts, to avoid traveling long paths and congestion.

Table 16 Problems identified at study areas

NO.	Problem	Cause	Alternative Solution
1.	Interlocking of through vehicle paths from major road (E-W) with those from minor road.	Presence of at – grade LRT crossing (E-W)	Separate movements so that major road through movements occur simultaneously with LRT crossing and minor road through movements occur after the LRV has left the intersection by using signals - Signalized Intersection - Signalized Roundabout
2.	High U turn volumes at roundabouts, increased delay to U turns and other	Lack of and closing of U turn areas along major	Reduce traffic congestion by diverting U turns to minor roads and opening closed U turn area near

	movements	road	CMC roundabout
3.	Partially blocked entry and exit from roundabouts, resulting in excessive delay to vehicles.	Presence of LRT stations near roundabouts with high pedestrian volumes	Fairly allocate time to pedestrians and vehicles by installing pedestrian signals.
4.	Delay experienced by LRT vehicle	Vehicles do not clear the grade crossing in time	Remove conflicting movements by proper allocation of green time and full LRT priority.

3.5. Traffic modeling using VISSIM

One of the many uses of VISSIM micro-simulation software is comparison of different junction geometries. The alternative designs of this study have been modeled according to the PTV VISSIM 9 user manual. [**43**] The major steps followed during traffic modeling of the roundabouts are shown in Table 17.

Table 17 VISSIM Modeling steps

<table>
<tr><th colspan="2">1. Building the Network</th></tr>
<tr><td>• Geometry and right of way
- links and connectors
- conflict areas
• Vehicle Inputs
- Approach Volumes
- Vehicle Composition
- Vehicle routes
• Pedestrian Inputs
- Pedestrian Volumes
- Pedestrian routes, crossings</td><td>• LRT inputs
- Geometry
- LRT routes
- Vehicle type, length , speed
- Departure times and crossing frequency
- Priority</td></tr>
<tr><th colspan="2">2. Model Calibration</th></tr>
<tr><td colspan="2">• Approach speeds
• Reduced speed areas
• Circulating speeds</td></tr>
<tr><th colspan="2">3. Validation</th></tr>
<tr><td colspan="2">• Using travel time and Delay</td></tr>
</table>

3.5.1. Building the network

3.5.1.1. Background image

The background images along with scales were obtained from Google Earth 2017. VISSIM enables the user to insert and scale background images, so that the models built can be as much like the existing geometries as possible. The following is the background image used to build the model of Sealite Mehret roundabout.

Figure 17 Background image for Sealite Mehret

3.5.1.2. Links and Connectors

The geometry of the roundabout is built using links and connectors. Links represent the approaches to the intersection while connectors connect the different approaches according to the movements that need to be made. Links have attributes such as width, number of lanes, gradient and assumed speed, that enable the user to model existing conditions accurately.

3.5.1.3. Right of way assignment using conflict areas

Conflict areas need to be defined at entrances to roundabouts to assign the right of way to the circulating traffic, so that when vehicles reach the circulating carriage way they will have to stop, give priority to the circulating traffic and wait for an acceptable gap. This feature was also used at pedestrian crossings. VISSIM user manual advises using conflict areas instead of priority rules to model the right of way at intersections. Conflict areas are displayed automatically, are easier to edit, and lead to a "more intelligent" driving behavior during simulation.

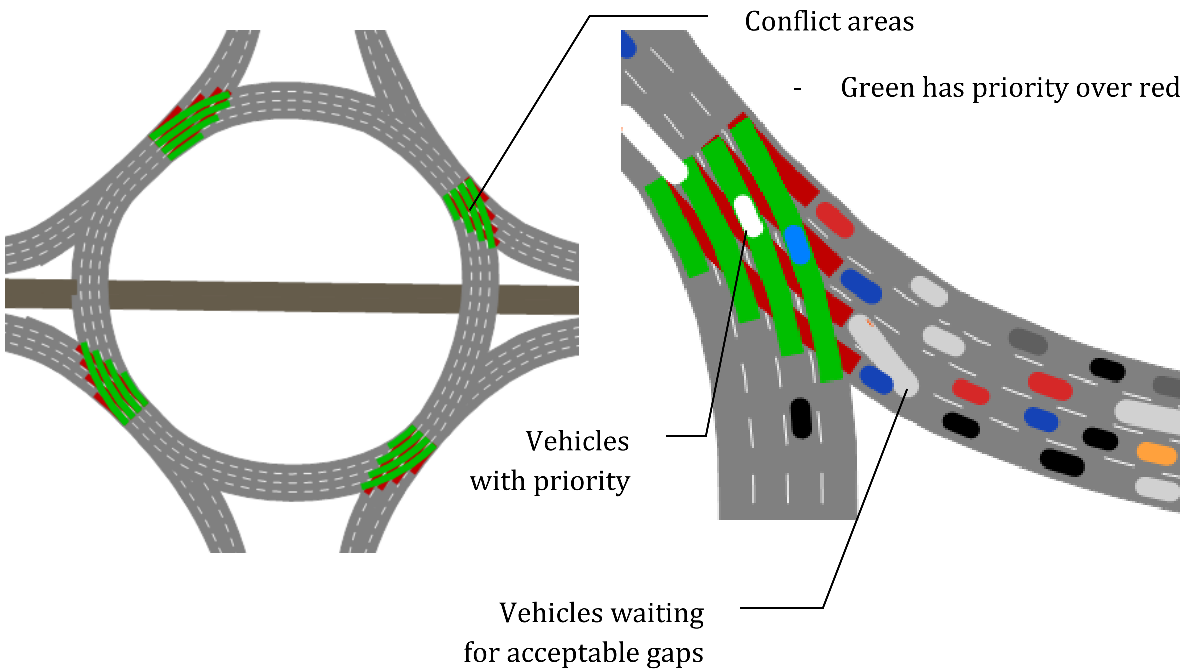


Figure 18 Conflict areas

3.5.1.4. Vehicle Inputs

Vehicles move in the network using a traffic flow model. The quality of the traffic flow model is essential for the quality of the simulation. In contrast to simpler models in which a largely constant speed and a deterministic car following logic are provided, VISSIM uses the psycho-physical perception model developed by Wiedemann (1974). The basic concept of this model is that the driver of a faster moving vehicle starts to decelerate as he reaches his individual perception threshold to a slower moving vehicle. Since he cannot exactly determine the speed of that vehicle, his speed will fall below that vehicle's speed until he starts to slightly accelerate again after reaching another perception threshold. There is a slight and steady acceleration and deceleration. The different driver behavior is taken into consideration by VISSIM with distribution functions of the speed and distance behavior. [**43**]

Total number of vehicles arriving at each approach is input into the network based on traffic counts. Two types of vehicles have been used, Car and HGV. The first represents passenger cars and the second represents heavy vehicles. The turning movement ratios are modeled using vehicle routes.

3.5.1.5. Pedestrian Input

Pedestrians have been included in the model according to the count results presented in data collection section. Pedestrian crossings were built with priority to pedestrians. Modeling of pedestrians also involves defining their routes, which in this case is simple since the only routes relevant to this study are at the pedestrian crossings. Four pedestrian types are included in VISSIM, man, woman, woman and child, and wheelchair User. Default values are also provided in order to describe walking behaviors according to social force model [**43**]. Pedestrian areas and lanes are drawn so that pedestrians can follow a certain route crossing from one pedestrian area to another.

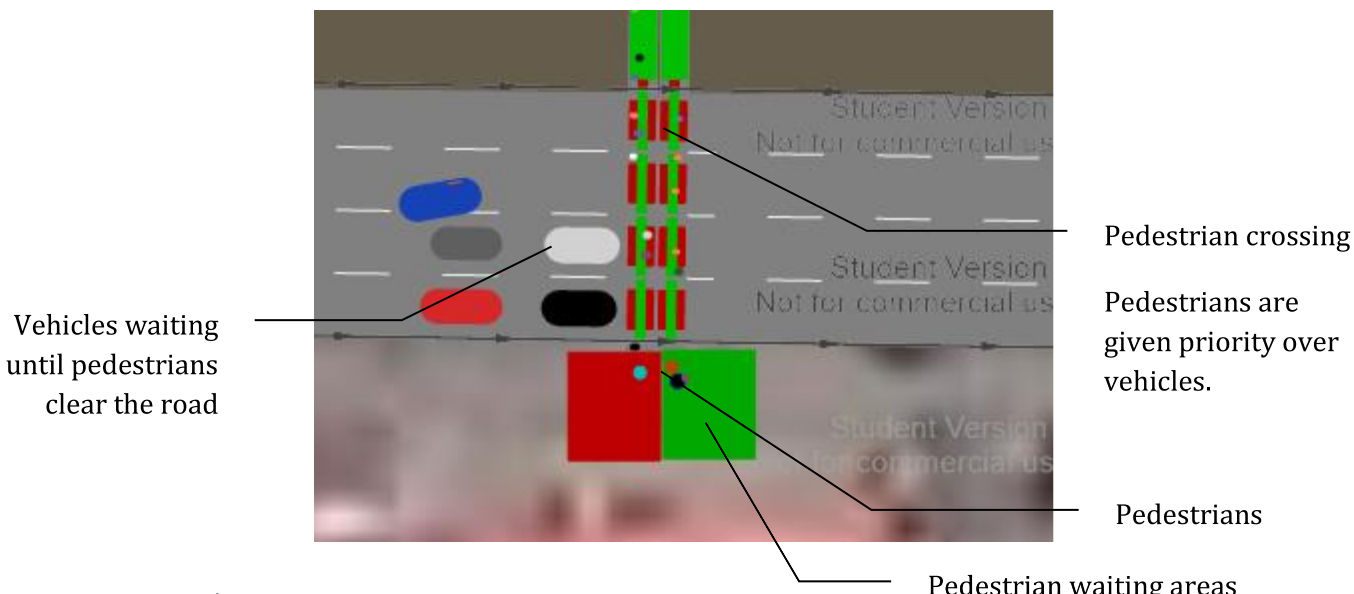


Figure 19 Pedestrian crossing in VISSIM

3.5.1.6. Modeling LRT line

Modeling the LRT line included,

- Building LRT lanes in both directions and assigning routes
- Specifying the LRT vehicle type, length and speed
- Adding departure times and crossing frequency
- Assigning priority to LRT vehicle at grade crossings

The geometry of the LRT line was built according to the data collected with width of 5.5m in each direction and 20m wide grade crossing. The speed of the LRT vehicle is 25km/hr when it crosses the intersection and the length is typically 60m during peak hour. The LRT vehicle arrives every 15 min during peak and off peak periods. Therefore departure times and rate of departure was assigned using this frequency. The LRT has full priority at the study areas. It emits warning sounds when it is more than 1 min away or at a speed of 25km/hr, when it is 416m away. The gates do not function so vehicles actually stop and give priority when the LRT vehicle is only 9s away or 63m away. The distance used during modeling on VISSIM is 63m. As soon as the LRT vehicle crosses a mark 63m away from the roundabout vehicles stop and wait for it to pass until it clears the grade crossing. The following picture shows the priority given to the LRT vehicle on VISSIM.

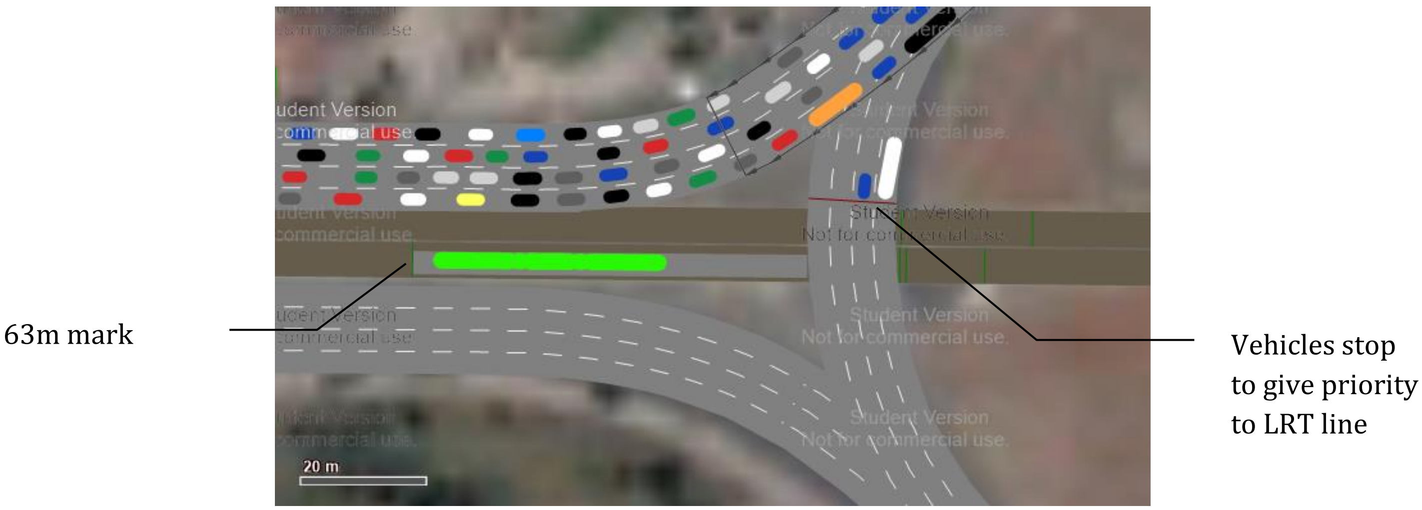


Figure 20 LRT priority

3.5.2. Signal Control

For the signalized intersection and signalized roundabout this feature is used. Signal control in VISSIM includes defining signal groups or phases, creating a signal program and placing signal heads at the proper locations.

3.5.3. Simulation

After necessary requirements such as setting up of Nodes and Vehicle travel time measurement locations are finished the simulation is conducted. The simulation lasts for about 600 simulation seconds or 10min.Various outputs can be specified according to the user’s requirement. After the simulation, the results are collected and analyzed.

3.6. Model Calibration and validation

Model calibration is the process of quantifying model parameters using real world data so that the model can realistically represent the traffic environment being analyzed. Vehicle characteristics and driver characteristics are the key parameters, which may be site specific and require calibration. Driver behavior is not homogeneous, and thus different drivers may behave differently given the same traffic conditions. Most of the microscopic models represent stochastic or random driver behavior by taking statistical distributions of behavior related parameters such as desired free-flow speed, queue discharge headway, acceptable gaps for lane changing and car following and driver response to advance information and warning signs. [**44**]

Model validation is the process of comparing model results with corresponding data observed in the field and ensuring that such results realistically represent the real world system. Only measurable data collected in the field can be used to validate model results. The most commonly used data for validation are queue length, travel time, delay, speed, density and throughput. [**44**]

For this study the main data used for calibration are;

- Approach free flow speeds
- Circulating speeds at roundabouts during free flow conditions
- Speeds at reduced speed areas at LRT grade crossings.

Approach free flow speeds were modeled using desired speed decision and assumed link speed attributes. While circulating speed and speeds at LRT grade crossings were modeled using reduced speed area attributes of VISSIM.

Model validation was done using delay and travel time. Travel time was determined as the time it takes for a vehicle to pass the intersection during peak hour. The distance the vehicle traverses in VISSIM is taken to be equal to the distance used during delay study, which is 800m for CMC and 1000m for Sealite Mehret. Using link segment results in VISSIM which gives speed of each link in 10m segments, travel time was calculated. Then delay was determined by subtracting ideal travel time. The simulation outputs used to determine travel time are attached in Appendix

B. Figures 21- 24 show the speed- distance and travel time- distance relationships of the roundabouts.

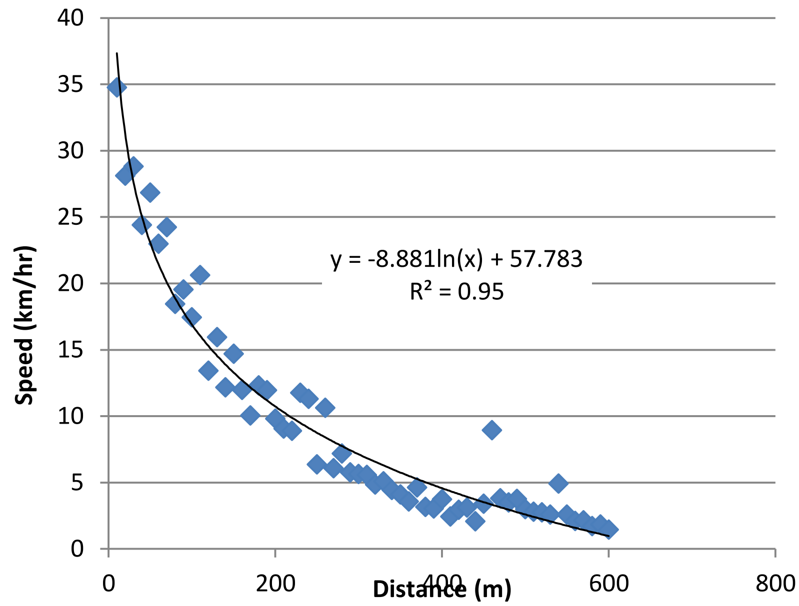


Figure 21 VISSIM Approach speeds of Sealite Mehret roundabout during peak hour

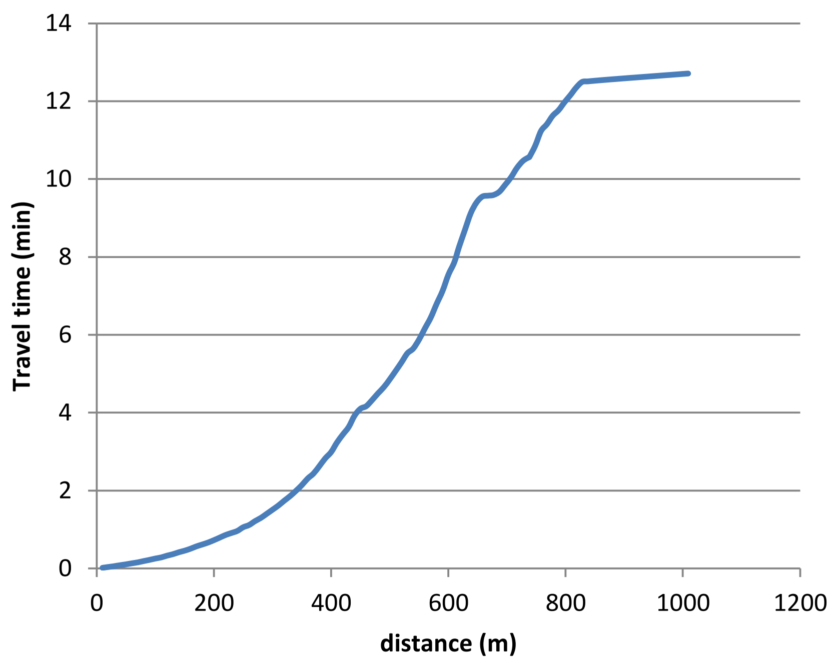


Figure 22 Distance - travel time plot for Sealite Mehret roundabout

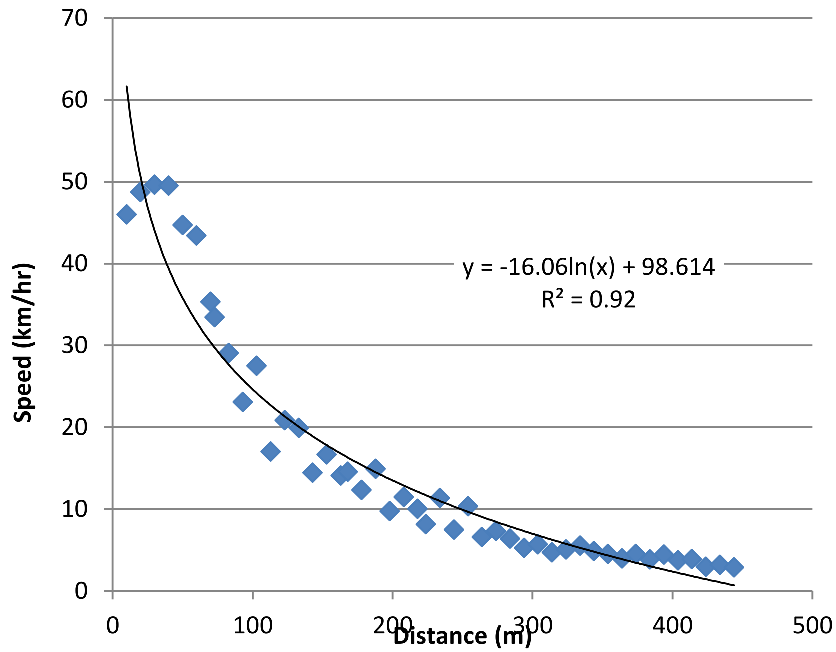


Figure 23 VISSIM Approach speeds of CMC roundabout during peak hour

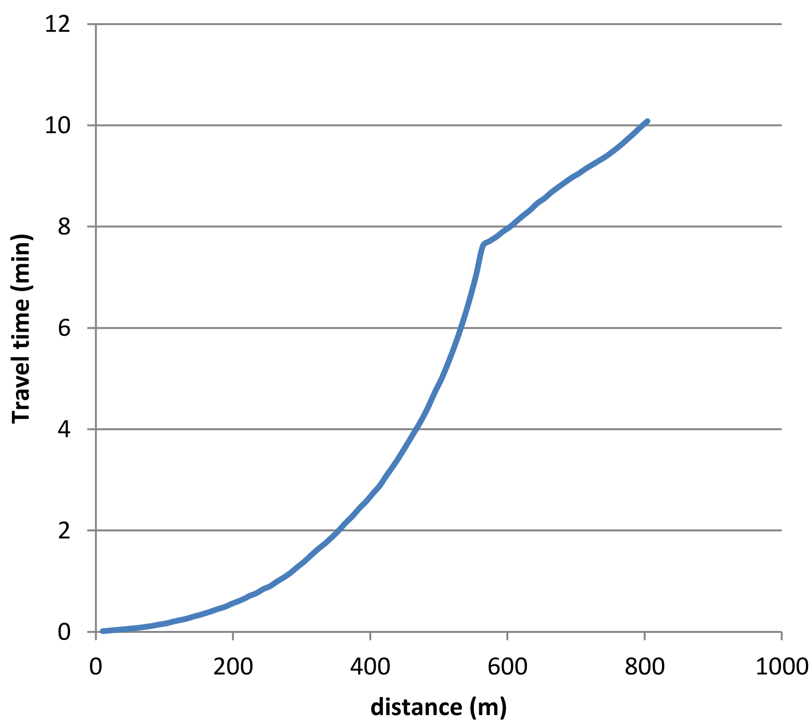


Figure 24 Distance - travel time plot for Sealite Mehret roundabout

As shown in Figure 24 total travel time to cross Sealite Mehret roundabout is 12.71min in VISSIM simulation. Ideal travel time to traverse the same distance is 1.34min (Table 8); therefore delay along the east-west route during peak hour is 11.37 min. The travel time data collected at the roundabout gave an average travel time of 13.1 min and a delay of 11.76 min. Therefore, this indicates the VISSIM model represents real-world conditions closely.

The total travel time to cross CMC roundabout in VISSIM simulation is 10.96 min. Subtracting ideal travel time of 1.40min gives delay of 9.16 min. The real-world travel time data collected at CMC is 12.62 min. This results in a delay of 11.22min with acceptable error of 2min. The delay is expected to be 9 to 13min. This shows that the delay predicted by the VISSIM model matches real-world delay of the approach. Furthermore, the following images from VISSIM and the study area show similarities in the way vehicles queue around the roundabout during morning peak hour.

Table 18 Approach Delay and Travel time VISSIM versus Real - world data

Location	Data	Travel time (min)	Delay (min)	Error (min)
Sealite Mehret	VISSIM	12.69	11.35	0.61
	Real- world	13.3	11.96	
CMC	VISSIM	10.96	9.16	2.06
	Real- world	12.62	11.22	

Figure 25 Morning peak hour VISSIM simulation image of CMC roundabout

Figure 26 Morning peak hour image of CMC roundabout

3.7. Alternative solutions

This section presents alternative solutions that can be implemented at the study areas. These solutions are proposed based on the problems identified. All of the alternatives presented are relatively low cost solutions that aim to investigate if satisfactory junction performance can be achieved at LRT crossings without grade separation.

3.7.1. Installation of pedestrian signals

There are LRT stations located near the roundabouts. These stations have high pedestrian volumes boarding and alighting from the LRT. Since the stations are located at the median of the major road, pedestrians cross the road in both directions, and significantly affect traffic flow. This can be seen on the traffic operation maps of Figure 7 and 9. Figure 26 shows an image taken at CMC roundabout during morning peak hour. The station near CMC is located on the west approach coming from Megenana, approximately 125m from the roundabout. Morning peak hour flow is mainly east to west and one of the problems at this location is observed to be exiting the roundabout at west approach.

At Sealite Mehret roundabout, the station is located on the east approach coming from CMC approximately 200m from the roundabout. The problem here is entering the roundabout. Since there is a continuous flow of pedestrians, vehicles do not find a gap to cross, hence long queue lengths are observed near LRT stations. If pedestrian signals were to be installed at these locations time can be allocated to both pedestrians and vehicles in a reasonable manner. This is expected to improve pedestrian safety as well as reduce delay to vehicles.

Pedestrian signals are installed on CMC approach of VISSIM model, which is the approach with the highest pedestrian volume. Green time provided for pedestrians is calculated to exceed Minimum pedestrian green time requirement. The signal program for the pedestrian signal is shown in Figure 27 and 28.

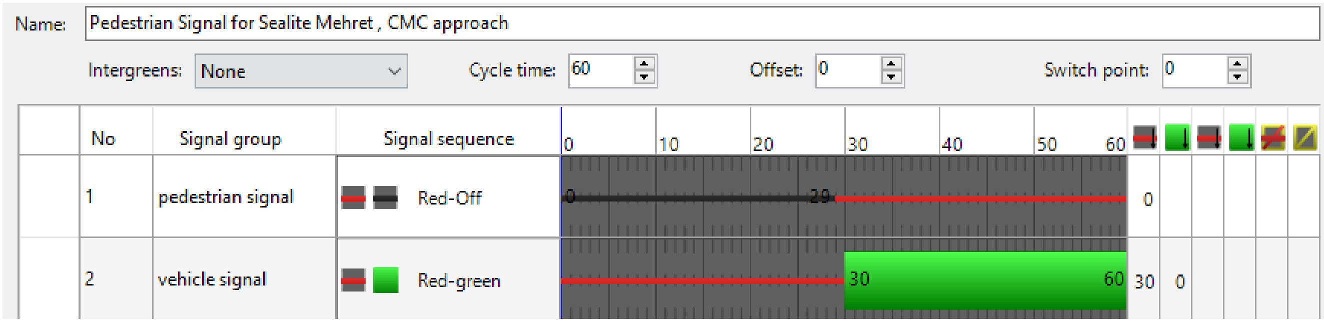


Figure 27 Pedestrian signal program for Sealite Mehret

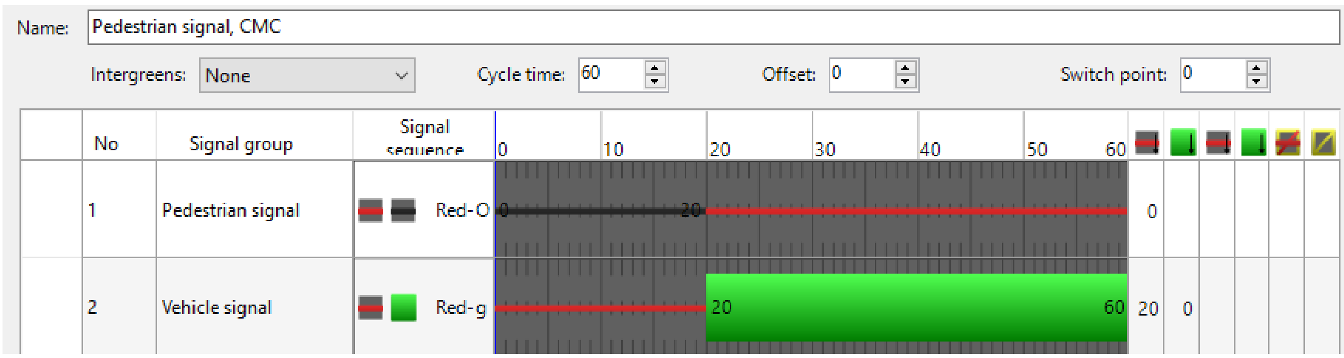


Figure 28 Pedestrian signal program for CMC

Figure 29 U turn location near CMC roundabout

3.7.1. Redirecting U- turns away from the roundabouts

As shown in section 3.3.2.2., Traffic count done for CMC intersection indicates significant U turns from Megenagna or CMC direction (823 veh/hr) and from Ayat direction (288 veh/hr), during morning peak hour. These U turns can be made at a location near St. Micheal Station, located about 1.15km from the roundabout on west (Megenagna) Approach. This area is currently closed because of the undue delay it was causing to west bound vehicles. Since the U turn was just next to an LRT station, west bound vehicles have to stop at the station in order to let pedestrians cross (Figure 29). This creates a long gap for the U turns to enter continuously. Since the U turn volume at this location is very high, the delay introduced to west bound vehicles is significant. This is why the U turn area was closed. But if signals are installed at the U turn and at the LRT station's pedestrian crossing, time can be allocated to all users efficiently. U turns from other approaches can first be directed to take a right turn to the adjacent minor road, make a U turn on the minor road and then make a left turn.

3.7.2. Installation of Traffic Signals

Before installing traffic signals, the U turns need to be redirected as explained above. Therefore, the signal timing designs are done based on this assumption. The traffic volumes have been rearranged accordingly. This section shows the geometric and signal timing design of CMC and Sealite Mehret signalized intersection.

3.7.2.1. Geometric Design

The aim of the Geometric Design is to modify the existing roundabout to a signalized intersection, under the existing space constraints. Therefore, the following decisions have been made with that in mind, to provide the best possible design. To carry out the design AACRA and AASHTO design manuals were used.The following modifications were made to the existing roundabout when converting to signalized intersection.

- ✓ Auxiliary lanes

There are high left turn volumes coming from minor roads in to the major road (East- West direction) at both CMC and Sealite Mehret. Therefore, exclusive left turn lanes were added on both approaches of the minor road. No geometric changes have been made to the major road due to space constraints.

✓ Turning roadways

A turning roadway should be designed to provide at least the minimum size island and the minimum width of roadway. The turning roadway should be wide enough to permit the right and left wheel tracks of a selected vehicle to be within the edges of the traveled way by about 0.6 m on each side. Generally, the turning roadway width should not be less than 4.2 m. When the turning roadway is designed for a semitrailer combination, a much wider roadway is needed. (40) The desired width recommended by AASHTO is 5.4m, wide enough for heavy vehicles but not too wide to encourage passenger cars to use it as a double lane. The site conditions permit a turning road way of 5.4m for both locations. For the left turns a control radius of 33m.

✓ Islands

AASHTO geometric design manual states that the smallest curbed corner island normally should have an area of approximately 5 m^2 for urban and 7 m^2 for rural intersections. However, 9 m^2 is preferable for both. Accordingly, corner triangular islands should not be less than about 3.5 m^2, and preferably 4.5 m^2, on a side after the rounding of corners. However there is no maximum limit. Thus the corner islands for the study areas have been designed to exceed the minimum limit, and in such a way that the right turn roadway is as far removed from the intersection as possible.

✓ Median openings

For any three- or four-leg intersection on a divided highway, the length of median opening should be as great as the width of crossroad traveled way plus shoulders. Where the crossroad is a divided highway, the length of opening should be at least equal to the width of the crossroad traveled ways plus that of the median. Median openings that are too wide, however, may lead to inefficient traffic signal operations. The median openings of the signalized intersection alternatives were designed according to this guideline and according to the requirement of left turning vehicles.

The figures 30-33 show the geometric design of the intersections.

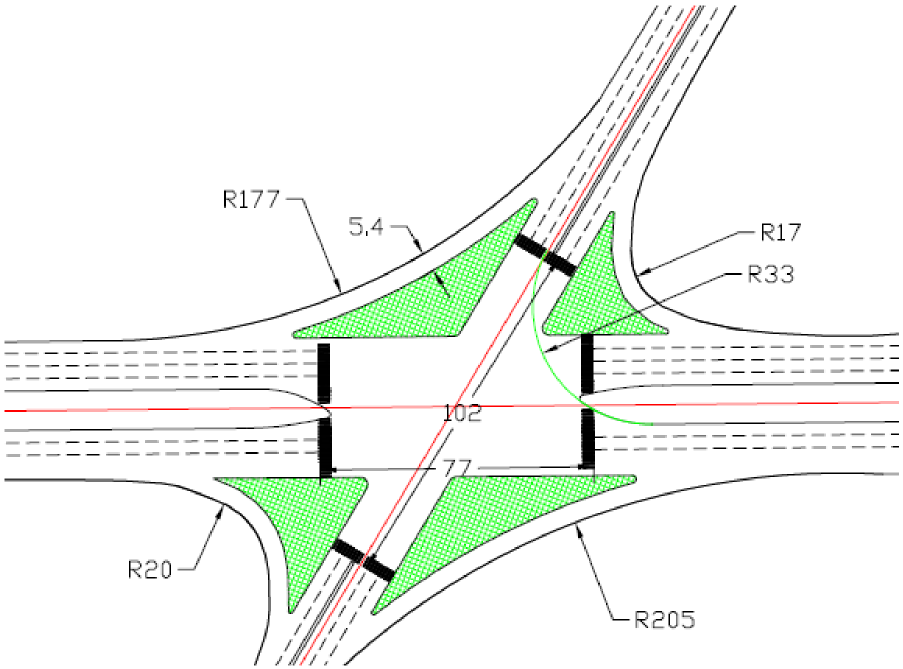


Figure 30 Geometric design of Sealite Mehret Intersection

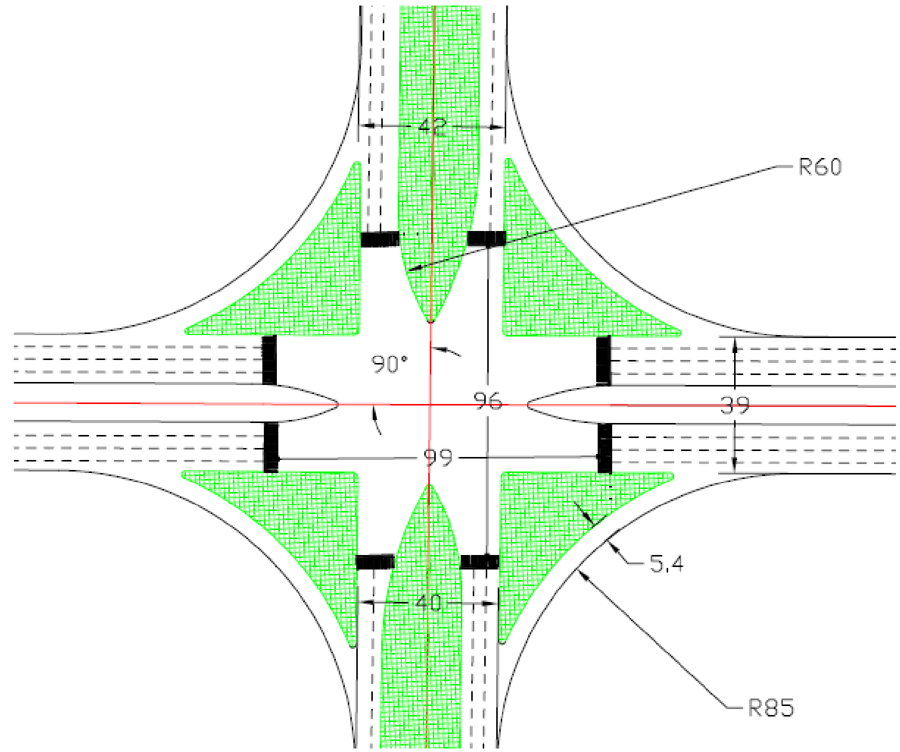


Figure 31 Geometric design of CMC intersection

Figure 32 Geometry of Signalized Intersection compared to existing roundabout, Sealite Mehret

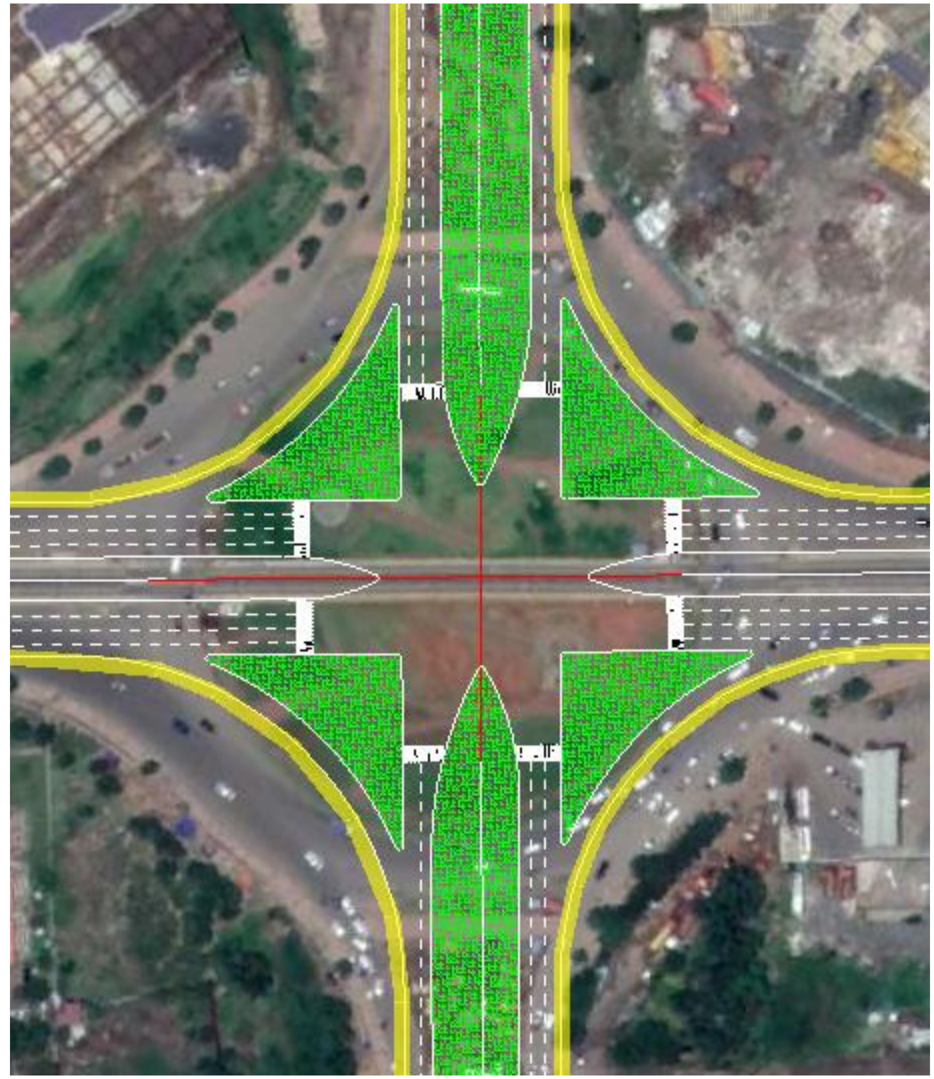


Figure 33 Geometry of Signalized Intersection compared to existing roundabout, CMC

3.7.2.2. Signal Timing Design for CMC

I. Signalized Intersection

To design the signal two manuals were viewed, the National Cooperative Highway Research Program (NCHRP) Report 812, Signal timing manual and the Highway Capacity Manual. This section is a report of the steps followed to determine signal timing requirements.

The design for the operation of a traffic signal is a complex process involving three important decisions: type of signal controller to be used, phase plan to be adopted, and allocation of green time among the various phases. [**45**]

II. Type of Signal controller

There are three types of traffic signal controllers: pretimed, fully actuated and semiactuated. Pretimed controllers have a preset sequence of phases displayed in repetitive order. [**44**] A major disadvantage of fixed or pretimed signals is that they cannot adjust themselves to handle fluctuating volumes. When the fluctuation of traffic volumes warrants it, a vehicle-actuated signal is used. These signals are capable of adjusting themselves. When such a signal is used, vehicles arriving at the intersection are registered by detectors which transmit this information to a controller. The controller then adjusts the phase lengths to meet the requirements of the prevailing traffic condition. [**37**]

The study areas experience excessive delays during morning and evening peak hours. The alternative of signalized intersection is proposed for these periods. At these peak hours, fluctuation of traffic volumes is not expected. Therefore, the type of signal controller that is selected for the study areas is pretimed signal controller.

III. Determination of left turn treatment

According to HCM 2000 left turns need to be protected if,

1. There is more than one left turn lane on the approach.
2. There are more than 240 veh/hr (unadjusted) turning left.
3. The cross product of the unadjusted left turn and opposing main line volumes exceeds the following minimum values.

Table 19 Minimum Cross- Product Values for Recommending Left Turn Protection

Number of through lanes	Minimum Cross- Product
1	50,000
2	90,000
3	110,000

If the U turns coming from CMC are made at a different location, Summit left turn will be equal to,

$$SL = 1764 - 846 = 900$$

Table 20 Left turn treatment

Approach	**Movement**			**Number of Opposing Lanes**	**Vo/No**	**VLT *(Vo/No)**	**Protection of left turns**	**Reason**
	L	T	R					
Ayat	373	1183	613	2	101	37673	Yes	VLT> 240 Veh/hr
CMC	22	202	498	2	592	13013	No	product < 90,000
Sunshine	331	126	618	1	109	36079	Yes	VLT> 240 Veh/hr
Summit	900	109	438	1	126	113,400	Yes	Product > 50,000

Where V_{LT}= left-turn flow rate, veh/hr

V_0= opposing through movement, veh/hr

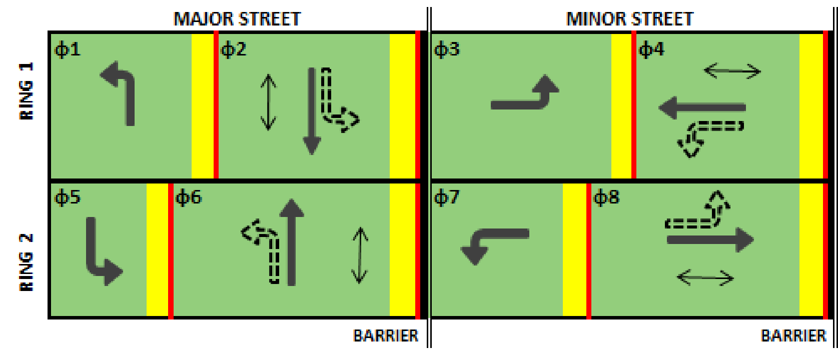
N_0 = number of lanes for opposing through

Left turns on all approaches need to be protected except for CMC approach, which has a low left turn volume.

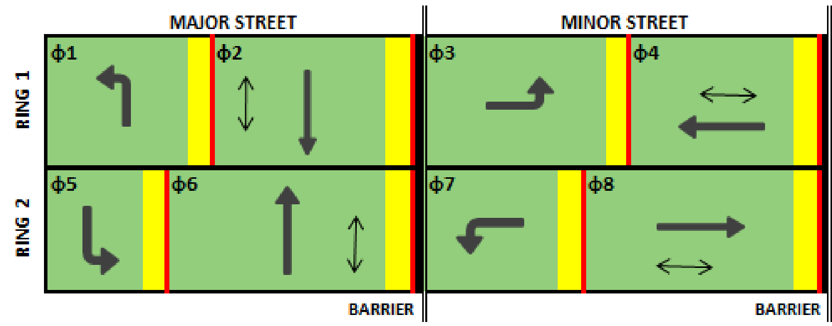
IV. Phase plans

The most critical aspect of any design of signal timing is the selection of an appropriate phase plan. The phase plan comprises the number of phases to be used and the sequence in which they are implemented. As a general guideline, simple two-phased control should be used unless conditions dictate the need for additional phases. Because the change interval between phases contributes to lost time in the cycle, as the number of phase's increases, the percentage of the cycle made up of lost time generally also increases. [**44**]

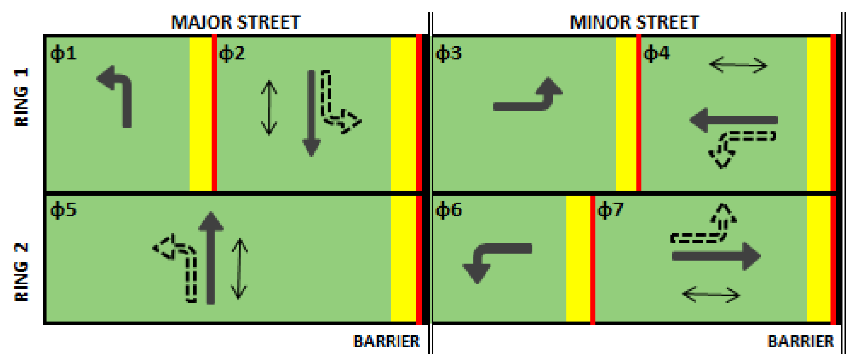
1.
Protected - Permitted Left turn Phasing [**44**] [**46**]



2.
Protected Left turn Phasing [**44**] [**46**]



3.
Additional Phasing Options with no protection for Left turn from CMC



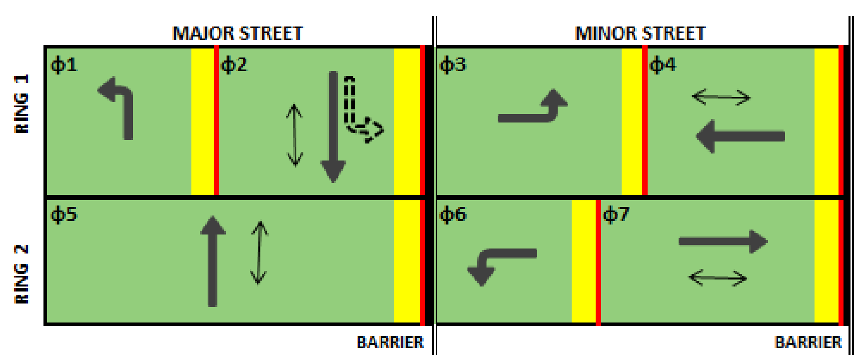


Figure 34 Alternative Phase plans

HCM and NCHRP Report 812 include different common phase plans with two phase and multiphase control. For CMC intersection, three left turns need to be protected. Keeping this in mind the phase plans in Figure 34 are expected to be suitable. These phase plans will be compared in order to find the best phase plan.

V. Phase sequence

Regardless of the type of left-turn phasing that is applied, it may be advantageous under certain conditions to change the sequence in which left-turn phases are served (relative to the through phases). There are three sequence options available: lead-lead, lag-lag, and lead-lag.

The most common left-turn phase sequence is the lead-lead sequence, which starts opposing left-turn phases prior to the through phases. The advantages of this sequence option include the following:

- Users react quickly to the leading green arrow indication.
- It minimizes conflicts between left-turn and through movements on the same approach,
- It gives unused time to the through movements, [**46**] if a dual ring controller is used.

For these reasons, the lead-lead sequence is selected.

VI. Calculation of Yellow Interval

The Institute of Transportation Engineers (ITE) offers the following equation for computing the yellow change interval [**46**]. The equation calculates the time required for a driver to make a decision to come to a safe stop or proceed. This is the minimum time to eliminate a dilemma zone, which exists if the yellow is too short.

$$y = t + \frac{1.47S_{85}}{2a + 64.4 * 0.01G} \tag{4}$$

Where, 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/s^2 (commonly 10 ft/s^2)

G= grade of approach, %

64.4= twice the acceleration rate due to gravity (32.2 ft/s^2)

- The speed limit of the approach road is 50kph (31.07 mph).
- The grades of the approaches to CMC intersection are 1.5% and 3%.

$$y = 1 + \frac{1.47 * 31.07}{2 * 10 + 64.4 * 0.01 * 1.5} = 3.53\ s$$

Yellow Interval of 4s will be used.

VII. Lost time per cycle

The total lost time in the signal can be obtained from Exhibit 10-17 of the Highway capacity manual (Table 21) on the basis of whether left turns are protected or permitted for the major street and the minor street. For CMC both major and minor street left turns are protected. Therefore, the lost time will equal 16 sec.

Table 21 Default lost time per cycle by left phase type (from HCM 2000, Exhibit 10 -17)

Major Street	Minor Street	Number of Phases	L(s)
Protected	Protected	4	16
Protected	Permitted	3	12
Permitted	Protected	3	12
Permitted	Permitted	2	8

VIII. Lane Assignments of the Approaches

The geometric change made to CMC roundabout approaches, is the addition of left turn lanes along the minor road and exclusive right turn lanes on all approaches.

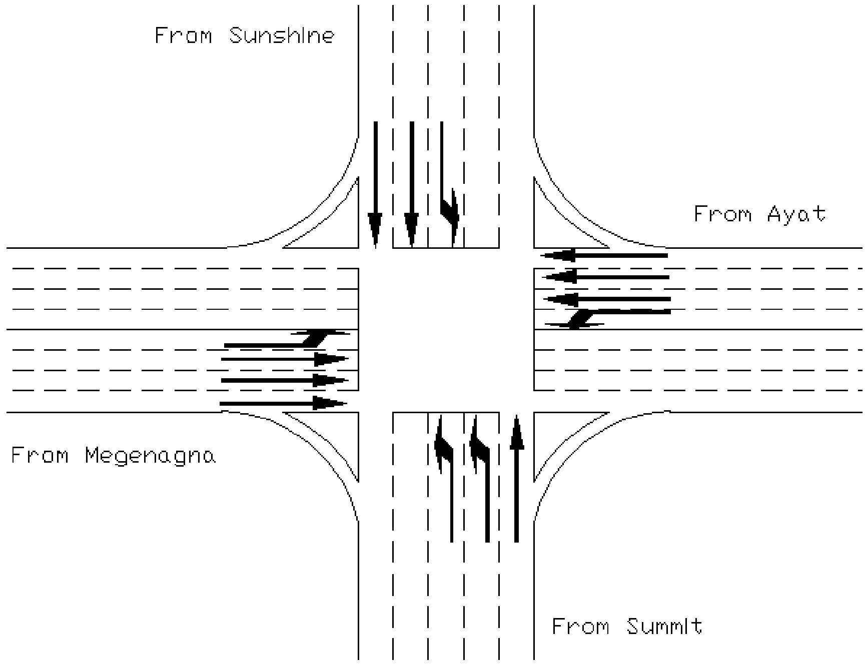


Figure 35 Lane Assignment, CMC signalized intersection

IX. Through Car equivalents

Left turn equivalency factors are determined below using Exhibit C16-3 of the HCM 2000, included in Appendix C. All left turns except for CMC are protected in all phasing alternatives considered. But since right turning vehicles have an exclusive lane that allows them to bypass the signalized intersection right turn equivalency factors are not needed. Heavy vehicle adjustments have also been made using a factor of 2.

Table 22 Demand per lane

Approach	Maneuver		Number of left turn lanes	Number of through lanes	left turn veh/hr/lane	through veh/hr/lane
	Left	Through				
Ayat	414	1313	1	3	414	438
CMC	23	212	1	3	23	71
Sunshine	366	134	1	2	366	67
Summit	900	126	2	1	450	126

Table 23 through car equivalents if left turns are permitted or protected

Approach	Left turn	opposing flow	type of left turn lane	ELT protected	ELT permitted	Through Car Equivalent	
						If protected	If permitted
Ayat	414	71	Exclusive	1.05	1.39	435	575
CMC	23	438	Exclusive	1.05	1.98	24	46
Sunshine	366	126	Exclusive	1.05	1.49	384	545
Summit	450	67	Exclusive	1.05	1.40	473	630

X. Cycle length

Cycle length is determined using the following equations from HCM 2000.

$$C = \frac{LXi}{\left[Xi - \sum_i \left(\frac{v}{s}\right)_{ci}\right]} \tag{5}$$

Where

C = cycle length (s);

L = Lost time per cycle (s);

X_C = critical v/c ratio for the intersection

X_i = v/c ratio for lane group i (note that target v/c ratio is a user- specified input with respect to this procedure; default suggested is 0.90);

$(v/s)_i$ = flow ratio for lane group i;

Si = saturation flow rate for lane group i (veh/h); and

Gi = effective green time for lane group i (s).

The shortest cycle length that will avoid oversaturation may be computed by Equation B16-5 of HCM 2000 using critical volume to capacity ratio Xc = 1.00

$$C_{min} = \frac{LX_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]} \tag{6}$$

To determine cycle length, flow ratios must first be calculated.

Flow ratio for lane group i, $(v/s)_i$

The flow ratio is the ratio of the actual flow rate to the saturation flow rate for a lane group at an intersection. The saturation flow rate for each lane group is computed according to Equation 16-4 of the Highway capacity Manual. [**44**] The formulae of the adjustment factors are included in Appendix C.

$$s = s_o N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb} \quad (7)$$

Where

s	= saturation flow rate for subject lane group, expressed as a total for all lanes in lane - group (veh/h);
s_o	= base saturation flow rate per lane (pc/h/ln);
N	= number of lanes in lane group;
f_w	= adjustment factor for lane width;
f_{HV}	= adjustment factor for heavy vehicles in traffic stream;
f_g	= adjustment factor for approach grade;
f_p	= adjustment factor for existence of a parking lane and parking activity adjacent to lane group;
f_{bb}	= adjustment factor for blocking effect of local buses that stop within intersection area;
f_a	= adjustment factor for area type;
f_{LU}	= adjustment factor for lane utilization;
f_{LT}	= adjustment factor for left turns in lane group;
f_{RT}	= adjustment factor for right turns in lane group;
f_{Lpb}	= pedestrian adjustment factor for left-turn movements; and
f_{Rpb}	= pedestrian – bicycle adjustment factor for right- turn movements.

All left lanes are exclusive left turn lanes and are therefore considered to be a single lane group. Using the base saturation flow rate of 1,900 passenger cars per hour per lane (pc/h/ln) provided in HCM and using the adjustment factors calculated and included in Table 24, saturation flow rate has been determined. The adjustments have been made based on the lane width, grade, and heavy vehicle data collected.

Adjustment factor for existence of a parking , blocking effect of local buses, area type, pedestrian adjustment , factor pedestrian – bicycle adjustment factor are assumed to be 1, since parking is assumed to be prohibited near the signalized intersection, there are no local buses that might block the area and no cyclists in significant numbers.

Table 24 Demand and Saturation Flow rates for CMC

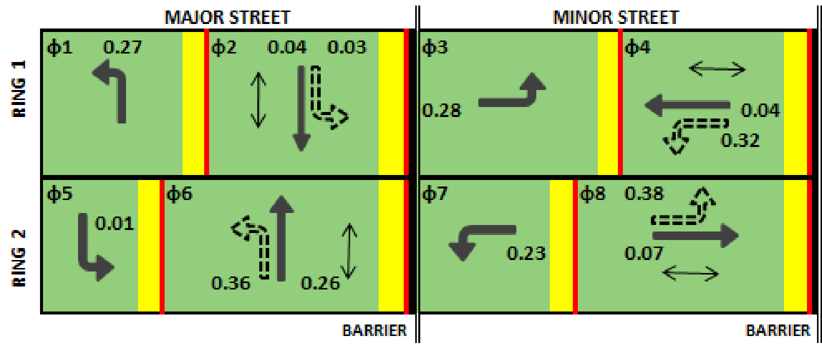
Approach	Movement	f_w	f_{HV}	fg	f_{LU}	f_{RT}	f_{LT}	s	v	v/s	v	v/s
Ayat	L	0.989	0.901	0.993	1	1	0.95	1596	435	0.27	575	0.36
	T						1	1680	438	0.26	438	0.26
CMC	L	0.989	0.952	0.993	1	1	0.95	1687	24	0.01	46	0.03
	T						1	1776	71	0.04	71	0.04
Sunshine	L	1	0.943	0.993	1	1	0.95	1690	384	0.23	545	0.32
	T						1	1779	67	0.04	67	0.04
Summit	L	1	0.943	0.985	1	1	0.95	1677	473	0.28	630	0.38
	T						1	1766	126	0.07	126	0.07

Critical v/s ratio

The critical phase volume is the volume for the movement that requires the greenest time during the phase. If two opposing lefts are moving during the same phase, the critical lane volume is the higher-volume left turn. [**37**]

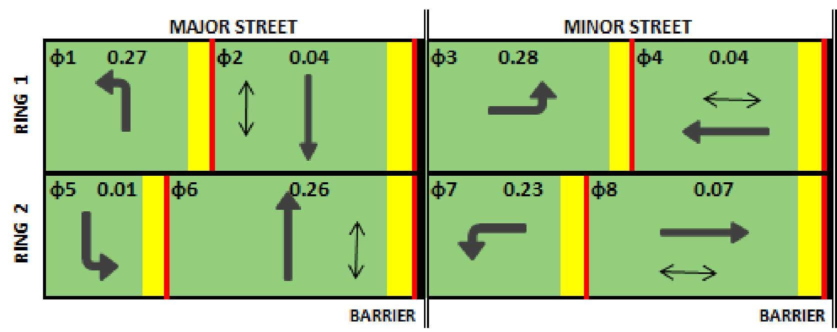
Four alternative phase plans are compared to arrive at the best phase plan. The critical v/s ratio computations for the alternatives are as follows.

Alternative 1: Protected - Permitted Left turn Phasing



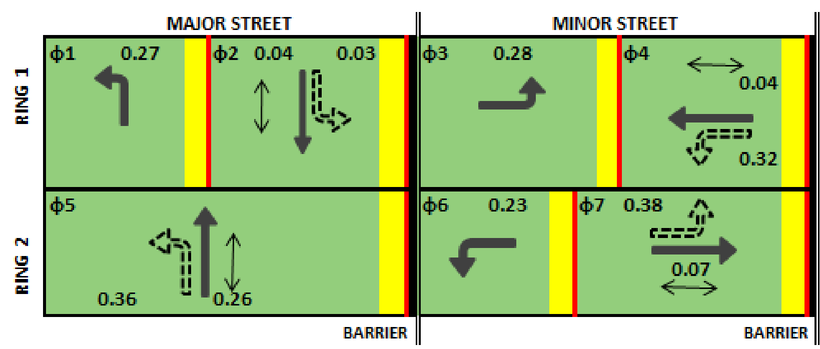
	Major street v/s	Minor street v/s	
Ring 1	0.31	0.6	
Ring 2	0.37	0.61	
Max.	0.37	0.61	**sum=0.98**

Alternative 2: Protected Left turn Phasing



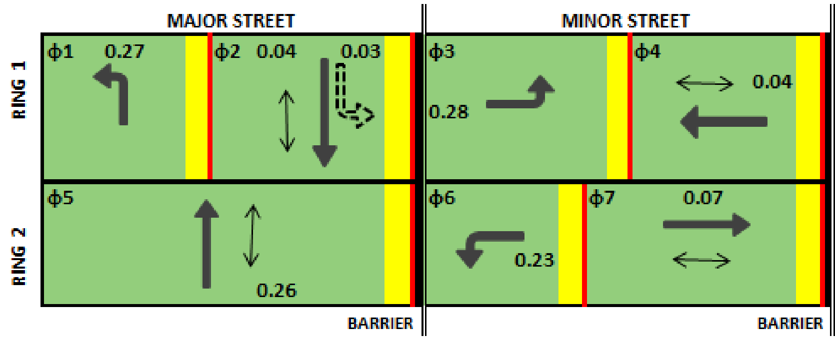
	Major street v/s	Minor street v/s	
Ring 1	0.31	0.32	
Ring 2	0.27	0.3	
Max.	0.31	0.32	**sum=0.63**

Alternative 3:



	Major street v/s	Minor street v/s	
Ring 1	0.31	0.6	
Ring 2	0.36	0.61	
Max.	0.36	0.61	**sum=0.97**

Alternative 4:



	Major street v/s	Minor street v/s	
Ring 1	0.31	0.32	
Ring 2	0.26	0.3	
Max.	0.31	0.32	**sum=0.63**

Alternative 2 and 4 have better v/s ratios. Alternative 4 is selected, since green time for phase 5 in alternative 2 is going to be too short with v/s ratio of 0.01.

Minimum cycle length, Xc= 1

$$C_{min} = \frac{LX_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]}$$

$$C_{min} = \frac{4 * 4 * 1}{[1 - 0.63]} = 43.24\ s$$

Desirable cycle length, Xc=0.75

$$C_{des} = \frac{LX}{\left[X - \sum_i \left(\frac{v}{s}\right)_{ci}\right]}$$

$$C_{des} = \frac{4 * 4 * 0.75}{[0.75 - 0.63]} = 100\ s$$

Cycle length of 100s is selected.

Allocation of green time

The total effective green time available per cycle is given by [**37**],

$$Gte = C - L \tag{8}$$

Effective green time for each phase,

$$Ge = \left(\frac{v}{s}\right)_i / \sum_i \left(\frac{v}{s}\right)_{ci} * Gte \tag{9}$$

Actual green time,

$$Gai = Gte + L - y \tag{10}$$

Table 25 Green times for CMC signalized intersection

Phase	1	2	3	4	5	6	7
Actual green time	31	8	33	12	39	30	15

Minimum Green time

At an intersection where a significant number of pedestrians cross, it is necessary to provide a minimum green time that will allow the pedestrians to safely cross the intersection. [**37**] If pedestrian timing requirements exist, the minimum green time for the phase is estimated by Equation 16-2 of the HCM 2000 or local practices. [**45**]

$$Gp = 3.2 + \frac{L}{Sp} + \left(\frac{0.81Nped}{We}\right) \quad \text{For We} > 3.0\text{m} \tag{11}$$

$$Gp = 3.2 + \frac{L}{Sp} + (0.27Nped) \ \text{For We} \leq 3.0\text{m}$$

Where,

Gp	=	minimum green time (s)
L	=	crosswalk length (m)
Sp	=	average speed of pedestrians (m/s), 1.2 m/s [**44**]
W_E	=	effective crosswalk width (m)
3.2	=	pedestrian start-up time (s), and
Nped	=	number of pedestrians crossing during an interval (p)

Using this equation from the HCM 2000 the following minimum pedestrian green times were determined and compared to the sum of amber and green times of concurrent phases.

Table 26 Pedestrian Minimum Green

location	Approach	Pedestrian volume in Pedestrian interval	Cross Walk length, L	Phase Number	Vehicle green + Amber (s)	Pedestrian minimum green (s)	Requirement check
CMC	Ayat	0	14	Φ 7	19	15	Sufficient
	CMC	5	14	Φ 4	16	16	Sufficient
	Sunshine	1	10.8	Φ 5	43	12	Sufficient
	Summit	1	10.8	Φ 2	12	12	Sufficient

In addition, the green times have been checked against typical values for minimum green to satisfy driver expectancy according to NCHRP report 812. (Table 63, Appendix C)

3.7.2.3. Determination of Control Delay

The total control delay for lane group *i* is given as [**44**]

$$d_l = d_{1i}PF + d_{2i} + d_{3i} \tag{12}$$

Where,

dl =the average control delay per vehicle for a given lane group

PF =uniform delay adjustment factor for quality of progression

d1i = uniform control delay component assuming uniform arrival

d2i = incremental delay component for lane group *i*, no residual demand at the start of the analysis period

d3i = residual delay for lane group *i*

Uniform Delay

$$d_{1i} = 0.50C * \frac{\left(1 - \frac{g_i}{C}\right)^2}{1 - \left(\frac{g_i}{C}\right)[min(X_i, 1.0)]} \tag{13}$$

Where,

dli =uniform delay (sec/vehicle) for lane group *i*

C = cycle length (sec)

gi = effective green time for lane group *i* (sec)

Xi = (*v*/*c*) ratio for lane group *i*

Incremental Delay

$$d_{2i} = 900T\left[(X_i - 1) + \sqrt{((X_i - 1)^2 + \left(\frac{8k_iI_iX_i}{C_iT}\right))}\right] \tag{14}$$

Where,

d2i = incremental delay (sec/vehicle) for lane group *i*

ci =capacity of lane group *i* (veh/h)

T = duration of analysis period (hr)

ki = incremental delay factor that is dependent on controller settings

Ii = upstream filtering metering adjustment factor accounts for the effect of filtered arrivals from upstream signals (for isolated intersections, *I*=1)

Xi =*v*/*c* ratio for lane group *i*

Residual Demand Delay

$$d_{3i} = \frac{(1800Q_{bi}(1 + u_i) * t_i)}{c_iT} \tag{15}$$

Where,

Qbi = initial unmet demand at the start of period *Ti* vehicles for lane group *i*

ci = adjusted lane group capacity veh/h

T = duration of analysis period (h)

ti = duration of unmet demand in *T* for lane group *i* (h)

ui = delay parameter for lane group *i*

Using these equations total control delay for each phase and average control delay

C= 100s

k= 0.5 for pretimed signals. [**37**]

I= 1 for isolated intersections. [**37**]

PF= 1 for isolated intersections. (Arrival type 3) [**37**]

T= 10 min (Set to be similar to VISSIM simulation period in order to compare results)

Table 27 Calculation of Control Delay

phase	gi	V	s	ci	V/s	Xi	d1i	d2i	d3i	di	di*vi
1	31	435	1596	495	0.27	0.88	32.72	2.36	0.00	35.08	15261.42
2	8	71	1776	142	0.04	0.50	44.08	1.19	0.00	45.27	3214.44
3	33	473	1677	553	0.28	0.85	31.26	1.75	0.00	33.01	15613.00
4	12	67	1779	213	0.04	0.31	40.24	0.37	0.00	40.60	2720.25
5	39	438	1680	655	0.26	0.67	25.17	0.52	0.00	25.69	11251.86
6	30	384	1690	507	0.23	0.76	31.70	1.04	0.00	32.74	12572.45
7	15	126	1766	265	0.07	0.48	38.90	0.58	0.00	39.48	4974.89
	Σ	1994								Σ	65608.30

Intersection Delay,

$$d_I = \frac{\sum_{A=1}^{A_n} d_A V_A}{\sum_{A}^{A_n} V_A} = \frac{65608.30}{1994} = 32.90\ sec/veh \tag{16}$$

3.7.2.4. Signal Program

The figures 36 and 37 display the ring diagram and signal program for CMC Intersection.

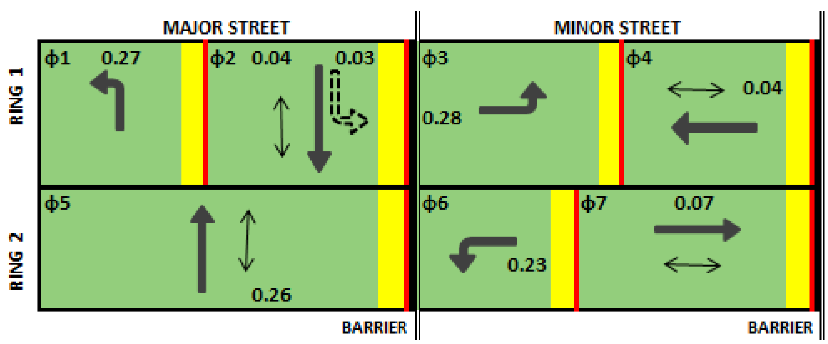


Figure 36 Ring diagram for CMC signalized intersection

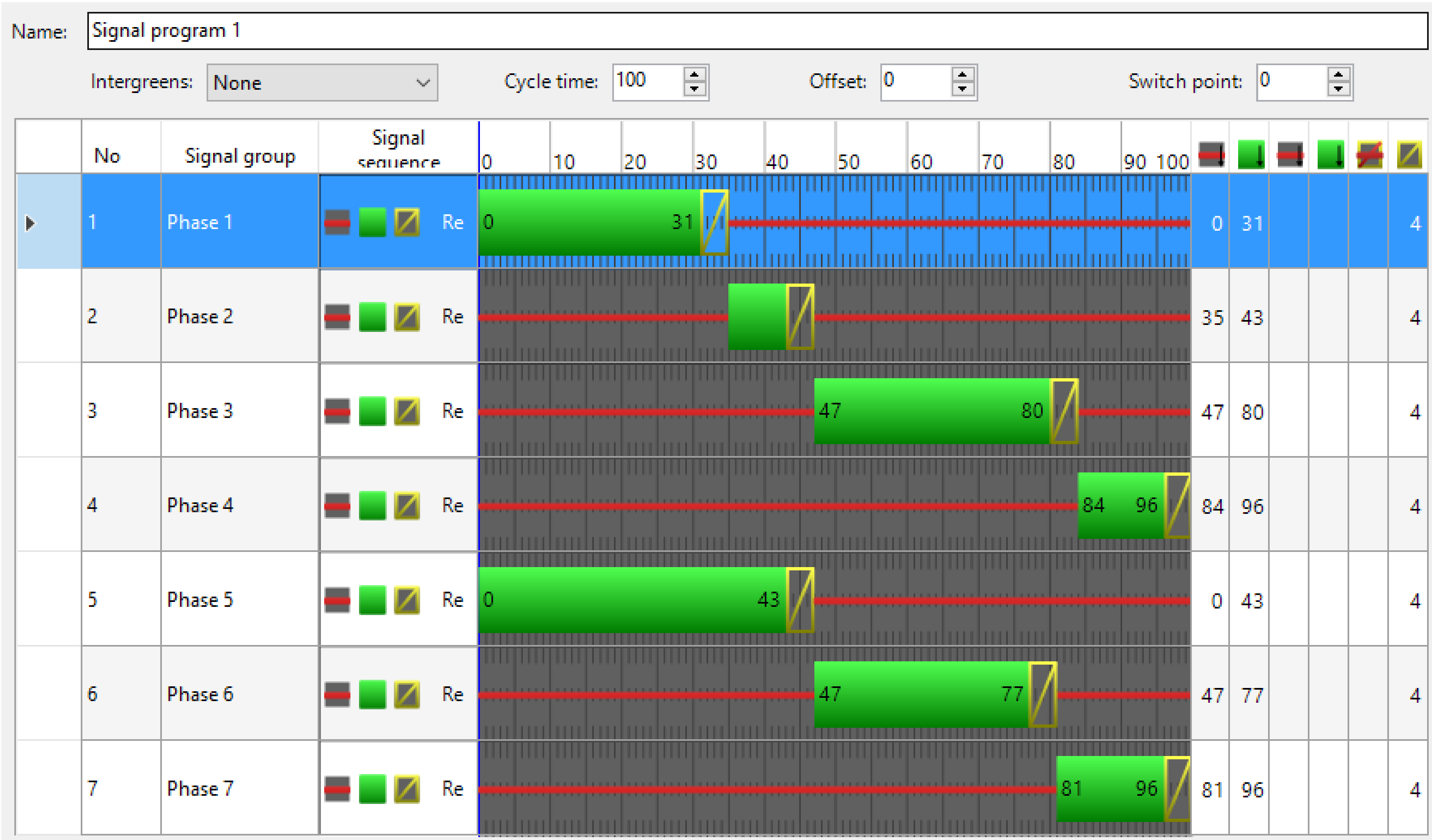


Figure 37 Signal Program for CMC signalized intersection

3.7.2.5. Signal Timing Design for Sealite Mehret Signalized Intersection

I. Type of Signal controller

Similar to CMC intersection, the signalized intersection is designed for peak hour conditions. At these peak hours, fluctuation of traffic volumes is not expected. Therefore, the type of signal controller that is selected for the study areas is pretimed signal controller.

II. Determination of left turn treatment

Table 28 Left turn treatment

Approach	Movement			Number of Opposing Lanes	Vo/No	VLT *(Vo/No)	Protection of left turns	Reason
	L	T	R					
CMC	1011	1126	563	2	434	438907	Yes	VLT> 240 Veh/hr
Megenagna	60	868	232	2	563	33774	No	product < 90,000
Kara	154	324	1040	1	352	54208	Yes	Product > 50,000
Mebrat Hail	1208	352	224	1	324	391392	Yes	Product > 50,000

Where V_{LT}= left-turn flow rate, veh/hr

V_0= opposing through movement, veh/hr

N_0 = number of lanes for opposing through

Left turns on all approaches need to be protected except for Megenagna approach, which has a low left turn volume.

III. Phase plans

Similar to CMC intersection, three left turns need to be protected. The left turn that does not need to be protected is from west approach, just like CMC. Therefore, the same phase plans that were initially considered for CMC (Figure 34) will also be considered for Sealite Mehret.

IV. Phase sequence

For the same reasons explained in section 3.7.2.2. V, the type of sequence that is used here is lead- lead sequence.

V. Calculation of Yellow Interval

The yellow interval for Sealite Mehret Intersection is calculated as follows.

$$y = t + \frac{1.47 S_{85}}{2a + 64.4 * 0.01G}$$

- The speed limit of the approach road is 50kph (31.07 mph).
- The lowest grade of all approaches to Sealite Mehret intersection is -4%.

$$y = 1 + \frac{1.47 * 31.07}{2 * 10 + 64.4 * 0.01 * -4} = 3.62\ s$$

Yellow Interval of 4s will be used.

VI. Lost time per cycle

The total lost time in the signal can be obtained from Exhibit 10-17, (Table 21) of the Highway capacity manual on the basis of whether left turns are protected or permitted for the major street and the minor street. For Sealite Mehret both major and minor street left turns are protected. Therefore, the lost time will equal 16 sec.

VII. Lane Assignments of the Approaches

The geometric change made to Sealite Mehret roundabout approaches, is the addition of exclusive right turn lanes on all approaches and left turn lanes along the minor road. Lane assignments have been made according to the left turn and through movement volumes. Two left turns have been assigned to CMC and Mebrat Hail approach since these are the largest left turn volumes.

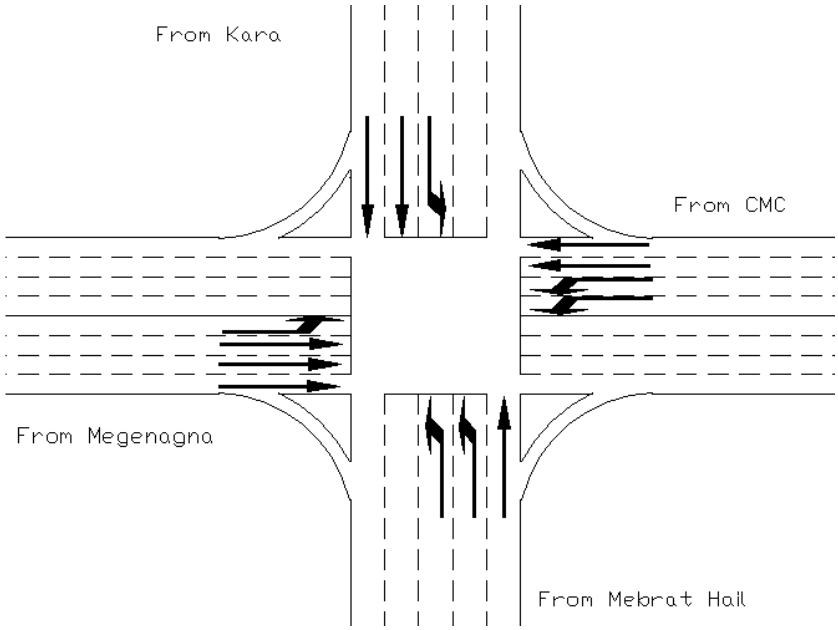


Figure 38 Lane assignment , Sealite Mehret Signalized Intersection

VIII. Through Car equivalents

Left turn equivalency factors are determined below using Exhibit C16-3 of the HCM 2000 (Appendix C). All left turns except for Megenagna are protected in all phasing alternatives considered. But since right turning vehicles have an exclusive lane that allows them to bypass the signalized intersection, right turn equivalency factors are not needed. Heavy vehicle adjustments have also been made using a factor of 2.

Table 29 Through Car Equivalents

Approach	Left turn	opposing flow	type of left turn lane	ELT		if protected	if permitted
				protected	permitted		
CMC	556	310	Exclusive	1	2	584	985
Megenagna	64	619	Exclusive	1	2	67	151
Kara	159	163	Exclusive	1	2	167	294
Mebrat Hail	637	66	Exclusive	1	2	668	987

IX. Cycle length

Cycle length is determined using the following equations from HCM 2000.The shortest cycle length that will avoid oversaturation may be computed by Equation B16-5 of HCM 2000 using Xc= 1.00

$$C_{min} = \frac{LX_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]}$$

To determine cycle length , flow ratios must first be calculated.

Flow ratio for lane group i ,(v/s)$_i$

Using the base saturation flow rate of 1,900 passenger cars per hour per lane (pc/h/ln) provided in HCM and using the adjustment factors calculated and included in Table 30, saturation flow rate has been determined. The adjustments have been made based on the lane width, grade, and heavy vehicle data collected.

Adjustment factor for existence of a parking , blocking effect of local buses, area type, pedestrian adjustment , factor pedestrian – bicycle adjustment factor are assumed to be 1, since parking is assumed to be prohibited near the signalized intersection, there are no local buses that might block the area and no cyclists in significant numbers.

Table 30 Demand and Saturation Flow rates for Sealite Mehret

Approach	Movement	f_w	f_{HV}	fg	f_{LU}	f_{RT}	f_{LT}	s	v	v/s	v	v/s
CMC	L	0.989	0.909	0.990	1	1	0.95	1697	584	0.34	985	0.58
	T						1	1786	619	0.35	619	0.35
Megenagna	L	0.989	0.935	0.990	1	1	0.95	1744	67	0.04	151	0.09
	T						1	1836	310	0.17	310	0.17
Mebrat Hail	L	1.000	0.971	0.965	1	1	0.95	1766	668	0.38	987	0.56
	T						1	1859	163	0.09	163	0.09
Kara	L	1.000	0.952	1.020	1	1	0.95	1831	167	0.09	294	0.16
	T						1	1928	66	0.03	66	0.03

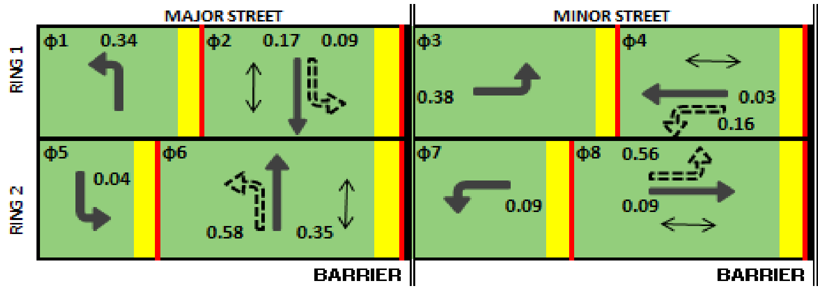
Critical v/s ratio

Four alternative phase plans are compared to arrive at the best phase plan. The critical v/s ratio computations for the alternatives are as follows.

Alternative 1:

Protected - Permitted

Left turn Phasing

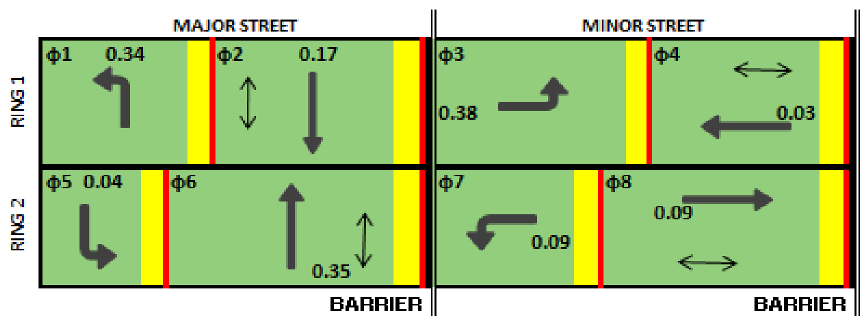


	Major street v/s	Minor street v/s	
Ring 1	0.51	0.54	
Ring 2	0.62	0.65	
Max.	0.62	0.65	**sum=1.27**

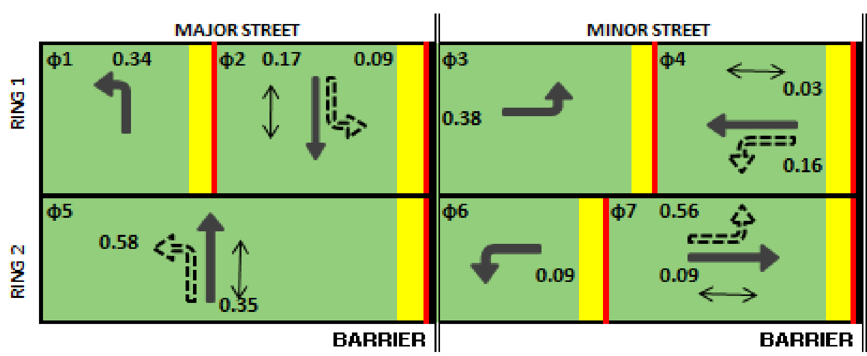
Alternative 2:

Protected Left turn

Phasing



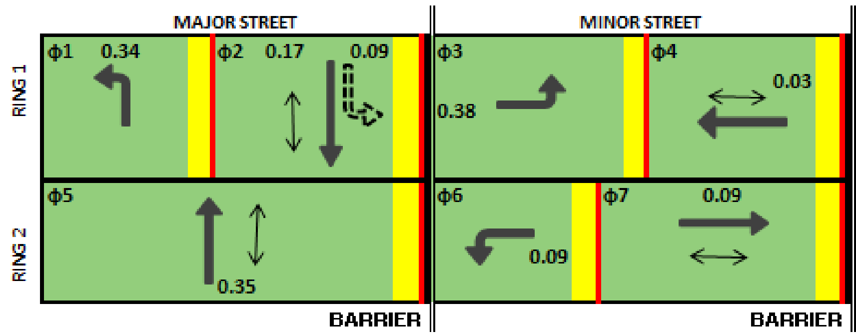
	Major street v/s	Minor street v/s	
Ring 1	0.51	0.41	
Ring 2	0.39	0.18	
Max.	0.51	0.41	**sum=0.92**



Alternative 3:

	Major street v/s	Minor street v/s	
Ring 1	0.51	0.54	
Ring 2	0.58	0.65	
Max.	0.58	0.65	**sum=1.23**

Alternative 4:



	Major street v/s	Minor street v/s	
Ring 1	0.51	0.41	
Ring 2	0.35	0.29	
Max.	0.51	0.41	**sum=0.92**

Alternative 2 and 4 have critical sums of 0.92. Since the left turn volume of phase 5 in alternative 2 is very small and the green time is going to be very short, protecting this left turn phase will bring unnecessary delay in lost times. Therefore alternative 4 is selected.

$$C_{min} = \frac{LX_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]}$$

$$C_{min} = \frac{4 * 4 * 1}{[1 - 0.92]} = 200s$$

Allocation of green time

The total effective green time available per cycle is given by [**37**], $Gte = C - L$

Effective green time for each phase, $Ge = \left(\frac{v}{s}\right)_i / \sum_i \left(\frac{v}{s}\right)_{ci} * Gte$

Actual green time, $Gai = Gte + L - y$

Table 31 Green times for Sealite Mehret Signalized Intersection

Phase	1	2	3	4	5	6	7
Actual green time	69	34	75	7	103	41	41

Minimum Green time

Using Equation 11, the following minimum pedestrian green times were determined and compared to the sum of amber and green times of concurrent phases.

Green plus Amber time of phase four needs to be increased to 16s in order to meet pedestrian requirement. Therefore 5s is added to the total cycle length, increasing it to 205s.

Table 32 Pedestrian Minimum Green

location	Approach	Pedestrian volume in Pedestrian interval	Cross Walk length, L	Phase Number	Vehicle green + Amber (s)	Pedestrian minimum green (s)	Requirement check
Sealite Mehret	CMC	0	14	Φ 7	45	15	sufficient
	Megenagna	4	14	Φ 4	11	16	insufficient
	Kara	2	10.8	Φ 5	107	13	sufficient
	Mebrat Hail	2	10.8	Φ 2	38	13	sufficient

Table 33 Adjusted green time

Phase	1	2	3	4	5	6	7
Actual green time	69	34	75	12	103	43	43

X. Determination of Control Delay

Total control delay for each phase and average control delay is determined below.

C= 205s,

k= 0.5 for pretimed signals, I= 1 for isolated intersections. [**37**]

PF= 1 for isolated intersections. (Arrival type 3) [**37**]

T= 10 min (Set to be similar to VISSIM simulation period in order to compare results)

Table 34 Calculation of Control Delay for Sealite Mehret Signalized Intersection

phase	gi	V	s	ci	V/s	Xi	d1i	d2i	d3i	di	di*vi
1	69	584	1697	570	0.34	1.03	68.08	14.06	15.00	97.14	56730.48
2	34	310	1836	310	0.17	1.00	85.64	12.86	0.00	98.50	30499.98
3	74	668	1766	654	0.38	1.02	65.50	12.98	13.60	92.08	61537.61
4	12	66	1928	116	0.03	0.57	93.99	1.93	0.00	95.92	6331.00
5	103	619	1786	916	0.35	0.68	38.67	0.39	0.00	39.06	24182.57
6	43	167	1831	394	0.09	0.42	70.26	0.32	0.00	70.58	11782.86
7	43	163	1859	400	0.09	0.41	70.00	0.29	0.00	70.29	11457.36
	Σ	2577								Σ	202521.86

Intersection delay,

$$d_I = \frac{\sum_{A=1}^{A_n} d_A V_A}{\sum_{A}^{A_n} V_A} = \frac{202521.89}{2577} = 78.58\ sec/veh$$

XI. Signal Program

Figures 39 and 40, display the ring diagram and signal program for Sealite Mehret Intersection.

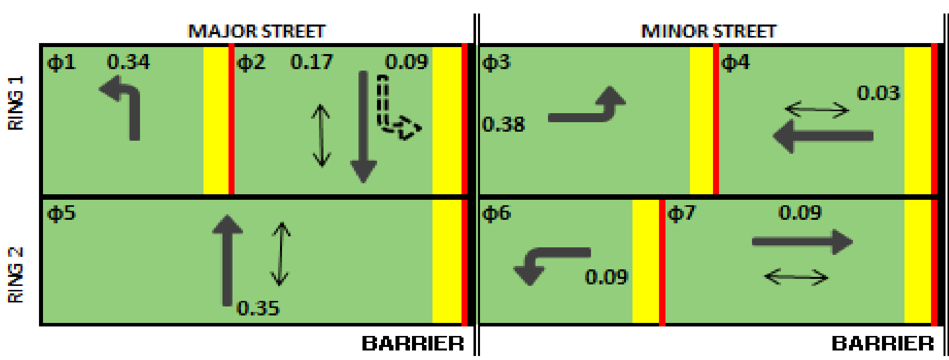


Figure 39 Ring diagram for Sealite Mehret Signalized Intersection

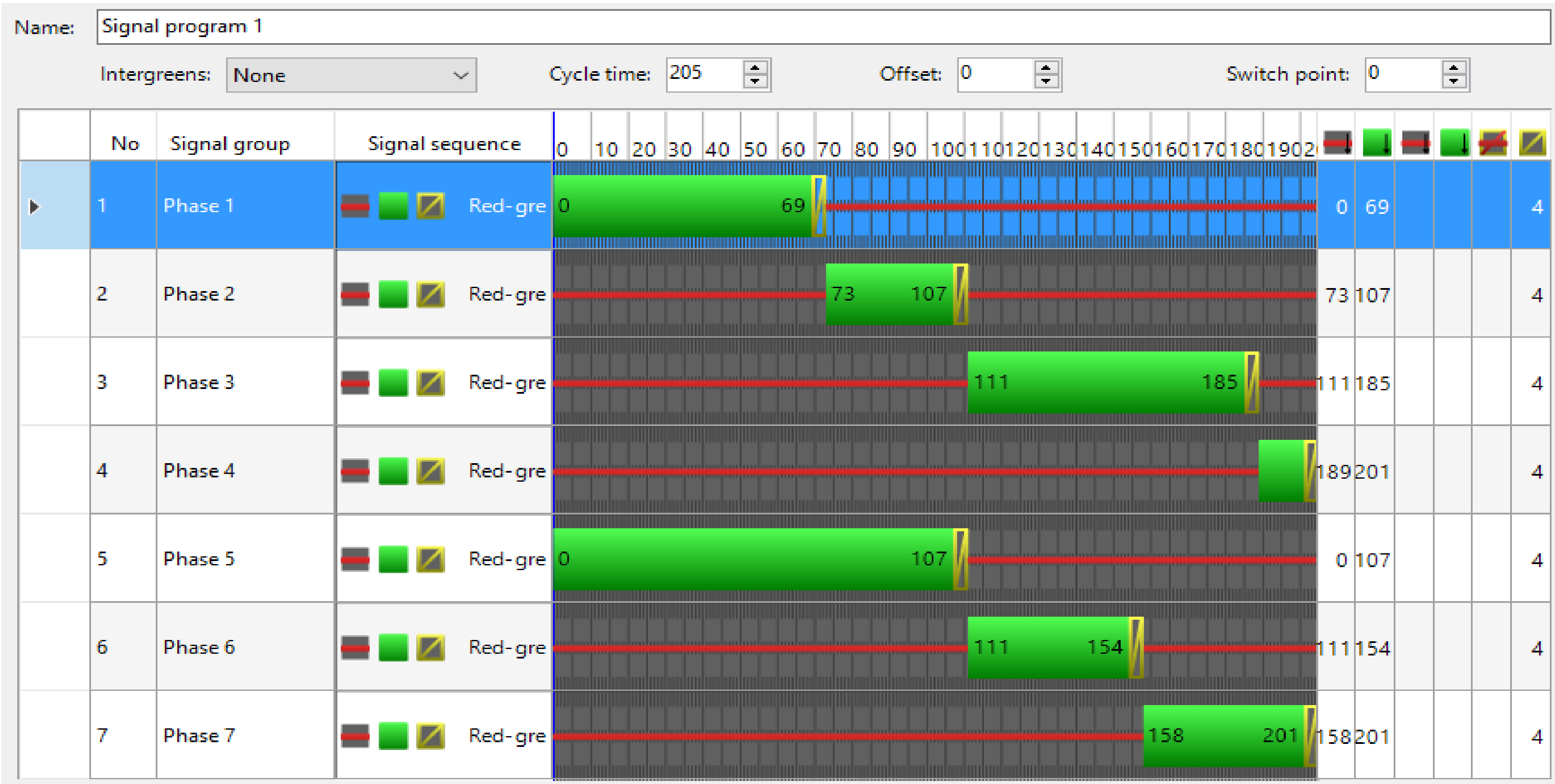


Figure 40 Signal Program of Sealite Mehret Intersection

3.7.2.6. Synchronizing Signal program with LRT arrival

The LRT line crosses the roundabouts through the median of the east-west corridor (major road). The signals of the signalized intersection and signalized roundabout alternatives should be timed to give the LRT full priority. If the LRT arrives during major road through movements there will be no conflict between the LRT and vehicles. Left turns from the major road will wait until the LRT passes. The LRT will take 12.24 sec to cross Sealite Mehret Signalized intersection, traveling at a speed of 25km/hr. For CMC it takes 14.4 sec, traveling at the same speed. Left turns from the major road will have to wait for these durations until the LRT clears the intersection.

If however, the LRT arrives during minor road phases, it will not only interrupt traffic flow but it will also compromise safety. Therefore, the signal should be timed in such a way that the LRT will not arrive during minor road phases, during which vehicles cross the LRT line. This is difficult to accomplish with pretimed signals, without affecting LRT schedule or causing unnecessary delay to vehicles by adding extra time so that LRT arrival will be synchronous to the cycle. On the other hand, if fully actuated signals are used, a detector can be placed along the LRT line to detect its arrival. If the LRT is going to arrive during undesirable phases, the green time for desirable phases can be extended or the green time for undesirable phases can be shortened. The LRT will have full priority and the signal will be designed to modify itself according to the requirements of the LRT and vehicles. This type of adaptive signal control has been found to improve transit travel time and corridor performance. [**47**] [**48**]

As mentioned above, the type of signal control used for this study is pretimed signal. Figures 41 and 42 show the LRT arrival with respect to the pretimed signal cycles of both study areas. The VISSIM simulation lasts for 600s, for the license used for this study. For this simulation period the LRT arrives during desirable phases or major road phase (shown in green), for both locations. The cycle lengths of both study areas are suitable for the simulation period of this study. However, this does not mean the LRT will always arrive during desirable phases. The solution for this problem is using fully actuated signals. These types of signals perform better than pretimed signals with respect to transit priority and reduced delay to vehicles. [**47**] If this study establishes that converting existing roundabouts to signalized intersections with pretimed -

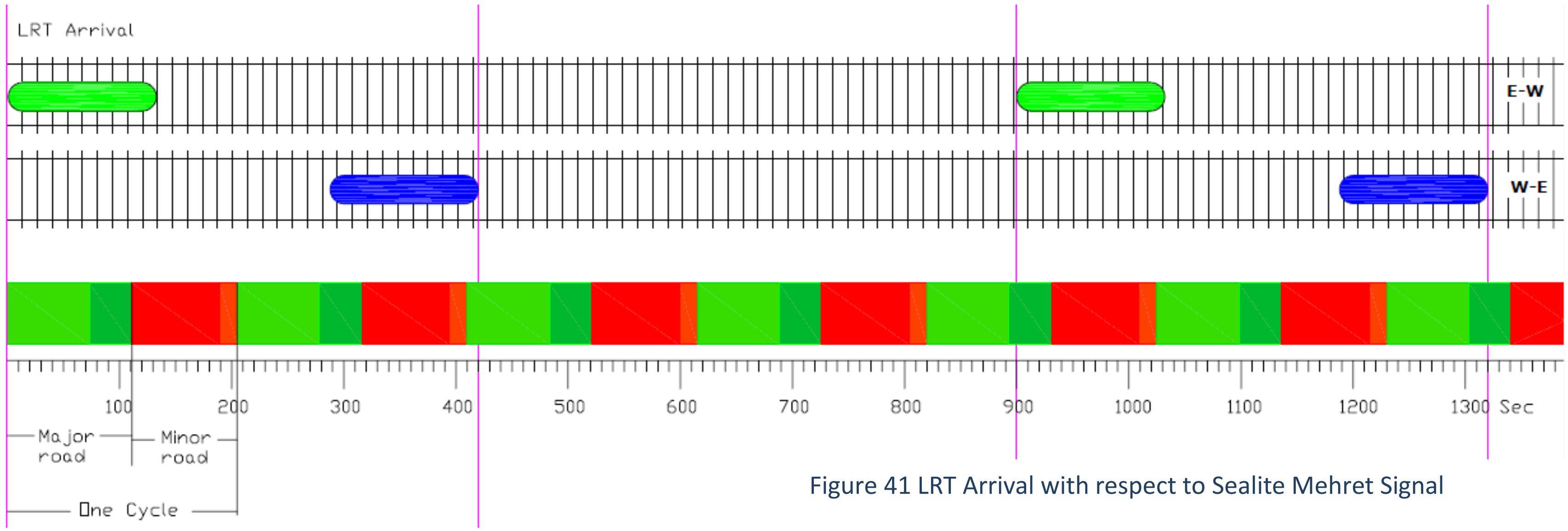


Figure 41 LRT Arrival with respect to Sealite Mehret Signal

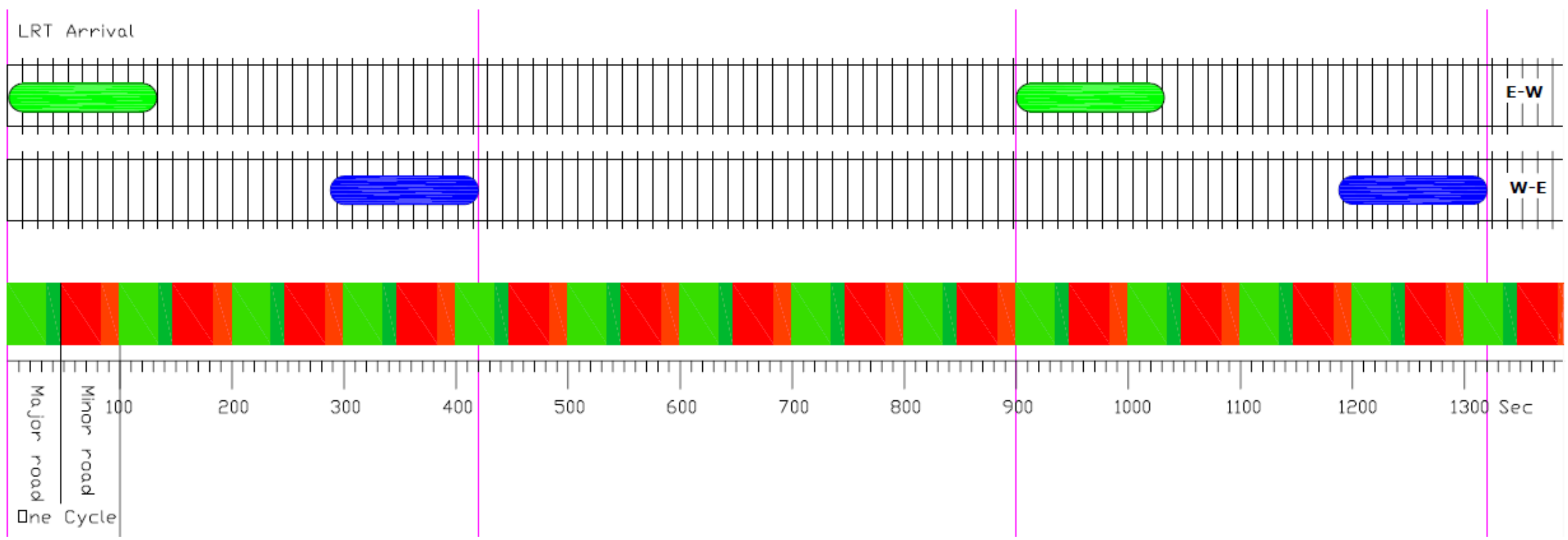


Figure 42 LRT Arrival with respect to CMC signal

signals improve performance, further studies can be done to compare pretimed signals to fully actuated signals for the study areas.

3.7.3. Converting the existing roundabouts to Signalized roundabouts

Similarly to signalized intersections, the U turns are diverted before converting the roundabouts. This section includes the geometric and signal timing design of the two roundabouts.

3.7.3.1. Geometric design of Signalized Roundabout

This is a relatively new type of design compared to conventional roundabouts and signalized intersection, therefore, not many design manuals are available. The manuals most used in similar studies and are easily available are the Design Manual for Roads and Bridges (DMRB), the Geometric Layout of Signal controlled Junctions and Signalized Roundabouts [**10**] and the Local transport note 1/09, Signal controlled roundabouts [**24**]. The later incorporates the previous one; therefore the design recommendations are not at all disparate.

The DMRB states that the following geometric modifications can be made to roundabouts when converting them to signalized roundabouts. Geometric modifications may be required to roundabouts in order to:

- ✓ increase capacity on external approaches;
- ✓ increase capacity on the circulatory carriageway;
- ✓ accommodate queues at internal stop lines on the circulatory carriageway;
- ✓ improve forward visibility and inter-visibility in the junction inter-visibility zone;
- ✓ provide alignment improvements and lane control measures;
- ✓ Provide specific measures for pedestrians, cyclists and buses.

These modifications may take the form of:

- ✓ additional nearside or offside approach lanes;
- ✓ additional internal approach lanes within the circulatory carriageway;
- ✓ increasing the size of splitter islands in order to achieve longer internal approaches;
- ✓ Segregated facilities or signaled crossing facilities.

Due to the space constraints at Sealite Mehret, no changes have been made to the existing geometry. Instead signal heads were added at the existing yield lines. On the other hand, there is sufficient space at CMC to add nearside and offside approach lanes along minor road. This modification is the only geometric modification to the existing roundabout.

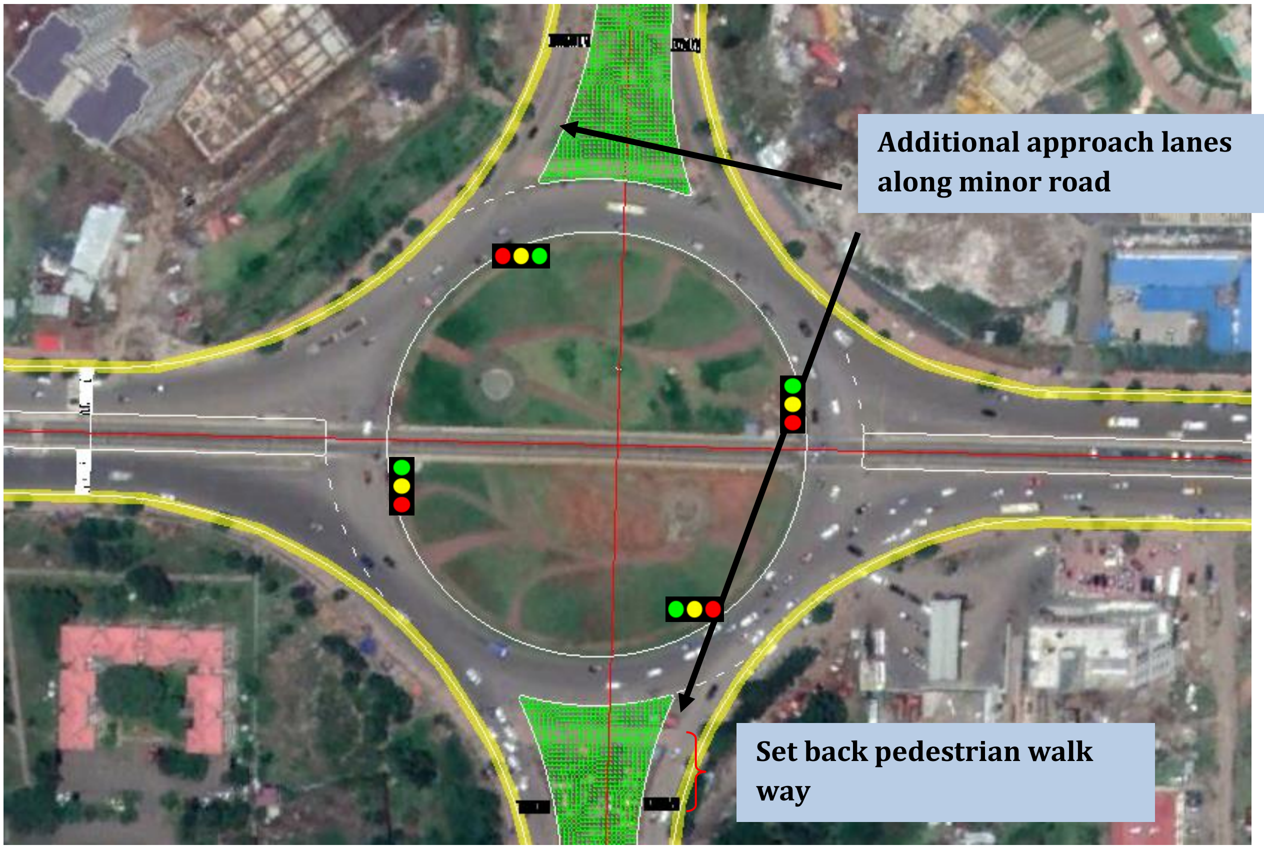


Figure 43 Modified Geometry of CMC Signalized roundabout

3.7.3.2. Signal timing design for Signalized roundabouts

Signal timing manuals for signalized roundabouts have not been found. Therefore, the signalized roundabouts are first going to be assumed to function as signalized intersections. Based on this assumption the signal timing is going to be designed using HCM method. The main difference here, however is that since the existence of a center island separates the movements, implementing a two phase signal will not compromise safety as adversely as a signalized intersection. There will be no need to protect left turns in order to prevent right angle crashes because left turns merge with conflicting through vehicles. Keeping this in mind, the phases for the signalized roundabouts have been reduced to two and the signal cycle lengths and green times have been calculated as follows. The green and amber times calculated were modeled in VISSIM then adjusted if necessary to meet site conditions.

XII. Phase plans

For the reasons explained above two phase plan is selected for the study areas. As a general guideline, simple two-phased control should be used unless conditions dictate the need for additional phases. Because the change interval between phases contributes to lost time in the cycle, as the number of phases increases, the percentage of the cycle made up of lost time generally also increases. [**44**]

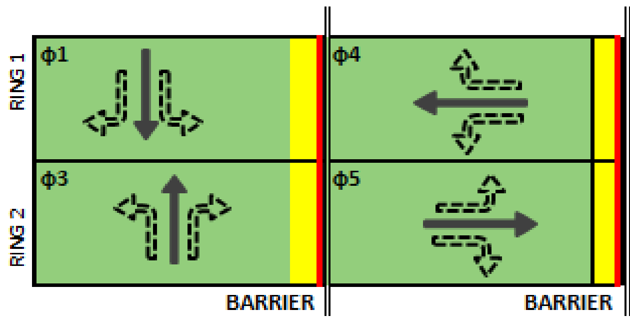


Figure 44 Two phased plan

XIII. Calculation of Yellow Interval

Yellow interval is determined using the following equation.

$$y = t + \frac{1.47S_{85}}{2a + 64.4 * 0.01G}$$

The speed limit of the approach roads to both study areas is 50kph and the grades remain the same (1.5% for CMC and -4% for Sealite Mehret roundabout), therefore a yellow interval of 4s will be used.

XIV. Lost time per cycle

According to Table 21, if left turns of both major and minor street are permitted, default lost time per cycle will be equal to 8s.

XV. Lane assignment of the approaches

There are no geometric changes made to Sealite Mehret roundabout but exclusive left turn lanes have been added along the minor roads of CMC roundabout. Accordingly, the following are the lane assignments that are used for the study areas. The U turns are going to be diverted in the same manner as for the signalized intersections.

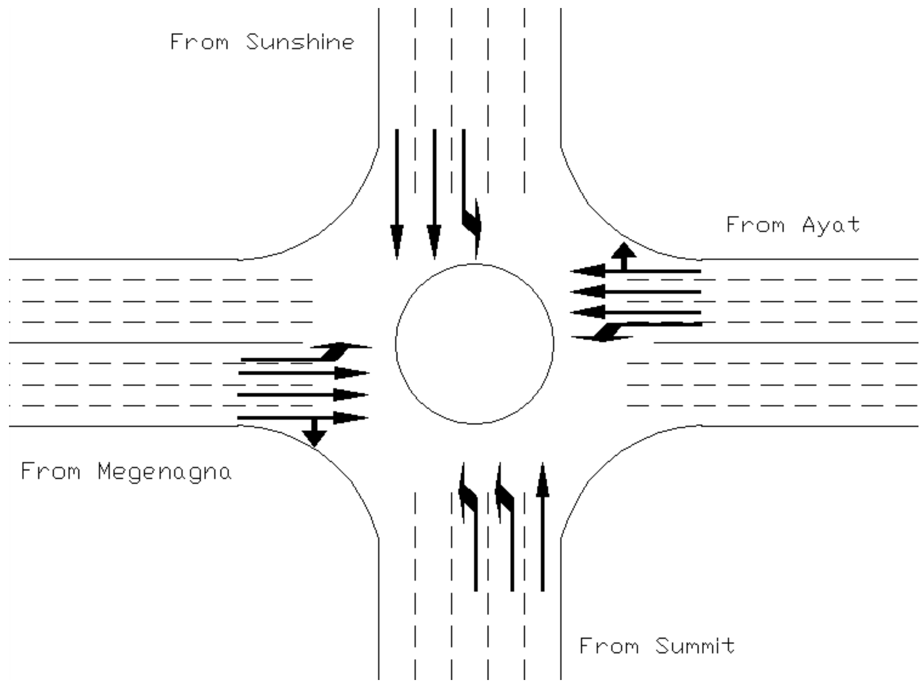


Figure 45 Lane Assignment of CMC signalized roundabout

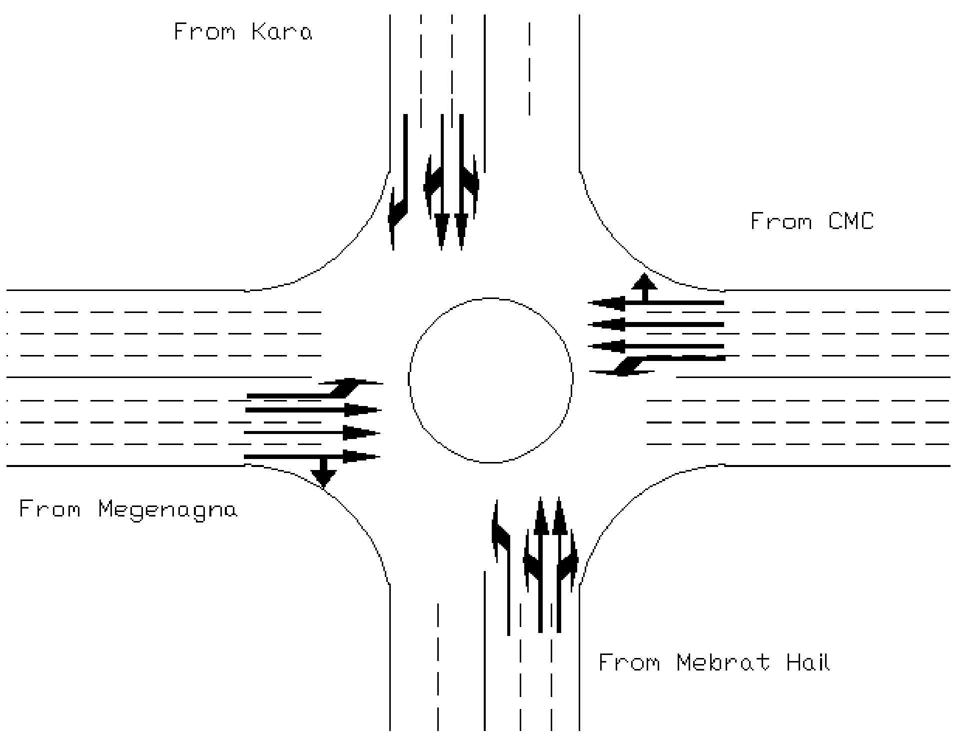


Figure 46 Lane assignment of Sealite Mehret signalized roundabout

Step 5: Cycle length

Cycle length is determined using Equation 5. First flow ratios are calculated for each lane group.

Table 35 Flow ratios for Sealite Mehret Signalized roundabout

Volume adjustment and saturation flow rate												
Volume adjustment												
	CMC			Megenagna			Mebrat Hail			Kara		
	L	T	R	L	T	R	L	T	R	L	T	R
Volume,V	1011	1126	563	60	868	232	604	352	224	154	324	520
No. Lanes	1	2	1	1	2	1	1	2	0	0	2	1
Lane group	L	T	R	L	T	R	L	LTR			LTR	R
Adj. flow	1011	1126	563	60	868	232	604	1180			998	520
Prop. LT	1.000			1.000			1.000	0.512			0.154	
Prop. RT			1.000			1.000		0.190			0.521	1.000
Saturation flow rate												
	CMC			Megenagna			Mebrat Hail			Kara		
Lane group	L	T	R	L	T	R		LTR			LTR	
So	1900	1900	1900	1900	1900	1900	1900	1900			1900	1900
Lanes	1	2	1	1	2	1	1	2	0	0	2	1
fw	0.989	0.989	0.989	0.989	0.989	0.989	1.044	1.044			1.044	1.044
fHV	0.909	0.909	0.909	0.935	0.935	0.935	0.971	0.971			0.952	0.952
fg	0.990	0.990	0.990	0.99	0.99	0.99	0.954	0.954			1.020	1.02
flu	1.000	1.000	1.000	1.000	1.000	1.000	1	0.977			0.960	1
fRT	1.000	1.000	1.000	1.000	1.000	1.000	1	0.972			0.922	1
fLT	1.000	1.000	1.000	1.000	1.000	1.000	1	0.975			0.992	1
s	1691	3382	1691	1739	3479	1739	1838	3401			3382	1926
Flow ratios (V/s)	**0.60**	**0.33**	**0.33**	**0.03**	**0.25**	**0.13**	**0.33**	**0.35**			**0.30**	**0.27**

Table 36 Flow ratios for CMC Signalized roundabout

Volume adjustment and saturation flow rate												
Volume adjustment												
	Ayat			CMC			Sunshine			Summit		
	L	T	R	L	T	R	L	T	R	L	T	R
Volume,V	373	1183	901	22	202	498	331	126	618	846	109	438
No. Lanes	1	2	1	1	2	1	0	2	0	0	2	0
Lane group	L	T	R	L	T	R		LTR			LTR	
Adj. flow	373	1183	613	22	202	498		1075			2219	
Prop. LT	1.000			1.000			0.308			0.602		
Prop. RT			1.000			1.000			0.575			0.197
Saturation flow rate												
	Ayat			CMC			Sunshine			Summit		
Lane group	L	T	R	L	T	R		LTR			LTR	
So	1900	1900	1900	1900	1900	1900		1900			1900	
Lanes	1	2	1	1	2	1	0	2	0	0	2	0
fw	0.989	0.989	0.989	0.989	0.989	0.989		1.044			1.044	
fHV	0.901	0.901	0.901	0.952	0.952	0.952		0.943			0.952	
fg	0.993	0.993	0.993	0.993	0.993	0.993		0.993			1.020	
flu	1.000	1.000	1.000	1.000	1.000	1.000		0.870			0.831	
fRT	1.000	1.000	1.000	1.000	1.000	1.000		0.914			0.952	
fLT	1.000	1.000	1.000	1.000	1.000	1.000		0.985			0.971	
s	1681	3362	1681	1776	3553	1776		2908			2959	
Flow ratios (V/s)	**0.22**	**0.35**	**0.54**	**0.01**	**0.06**	**0.28**		**0.37**			**0.46**	

- **Critical V/s ratio**

For both study areas two phased plans are selected. The critical v/s ratios are calculated as follows.

1. Critical V/s ratio for CMC

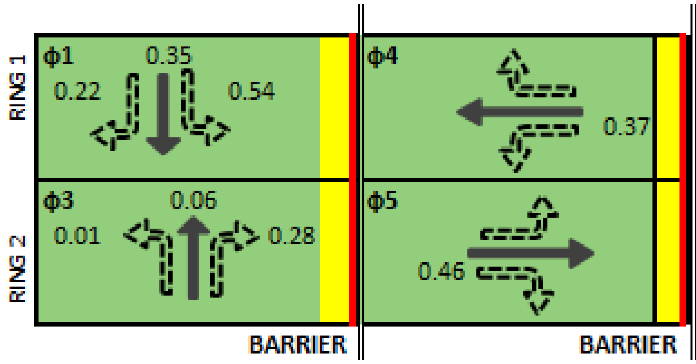


Figure 47 V/s ratios for CMC signalized roundabout

Sum of Critical V/s ratio = 0.54 + 0.46 = 1.00

$$C_{min} = \frac{LX_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]}$$

For C= 160s, $X = \frac{C-L}{C} = \left(\frac{160-8}{160}\right) = 0.95$

Cycle length of 160s is applied. The green times are allocated based on v/s ratios as follows. The green times for phase 1 and 3 are equal and so are the green times for phase 4 and 5.

$$Gte = C - L = 160 - 8 = 152s$$

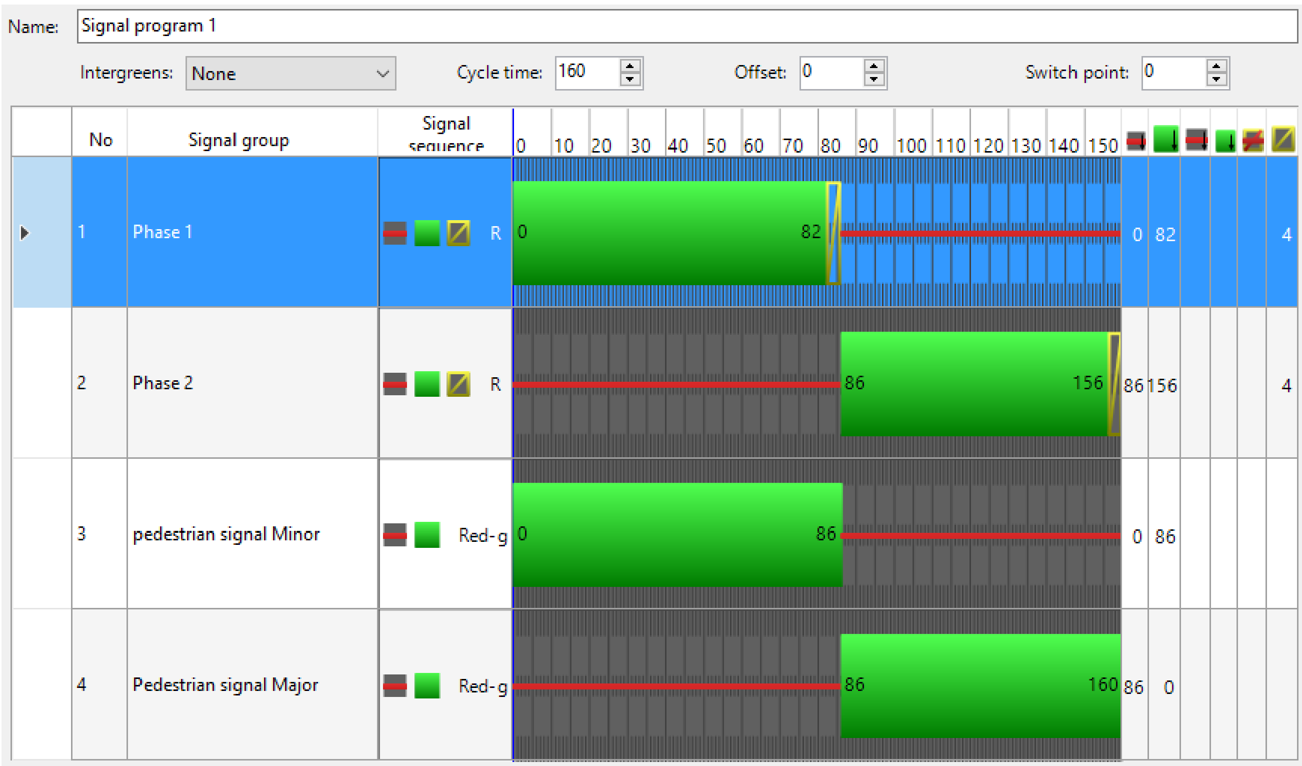
$$g1 = \frac{0.54}{1.00} * 152s = 82\ s\ ,\ g2 = \frac{0.46}{1.00} * 152s = 70\ s$$

Table 37 Pedestrian green times for CMC signalized roundabout

location	Approach	Pedestrian volume in ped interval	Cross Walk length, L	Vehicle green + Amber (s)	Pedestrian minimum green (s)	Requirement check
CMC	Ayat	0	14	70	15	sufficient
	CMC	20	14	70	20	sufficient
	Sunshine	1	10.8	82	13	sufficient
	Summit	4	10.8	82	13	sufficient

The green times provided are sufficient for pedestrians crossing the road.

Table 38 Signal program for CMC signalized roundabout



2. Critical V/s ratio for Sealite Mehret

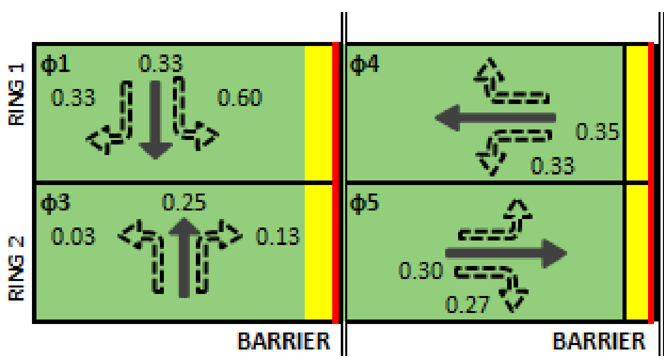


Figure 48 V/s ratios for Sealite Mehret signalized roundabout

Sum of Critical V/s ratio = 0.60 + 0.35 = 0.95

$$C_{min} = \frac{2 * 4 * 1}{[1 - 0.95]} = 160s$$

$$Gte = C - L = 160 - 8 = 152s$$

$$g1 = \frac{0.60}{0.95} * 152 = 96\ s\ ,\ g2 = \frac{0.35}{0.95} * 152 = 56\ s$$

Table 39 Pedestrian green times for Sealite Mehret signalized roundabout

location	Approach	Pedestrian volume in ped interval	Cross Walk length, L	Vehicle green + Amber (s)	Pedestrian minimum green (s)	Requirement check
Sealite Mehret	CMC	0	14	56	15	sufficient
	Megenagna	16	14	56	19	sufficient
	Mebrat Hail	2	10.8	96	13	sufficient
	Kara	5	10.8	96	13	sufficient

The green times provided meet pedestrian requirement.

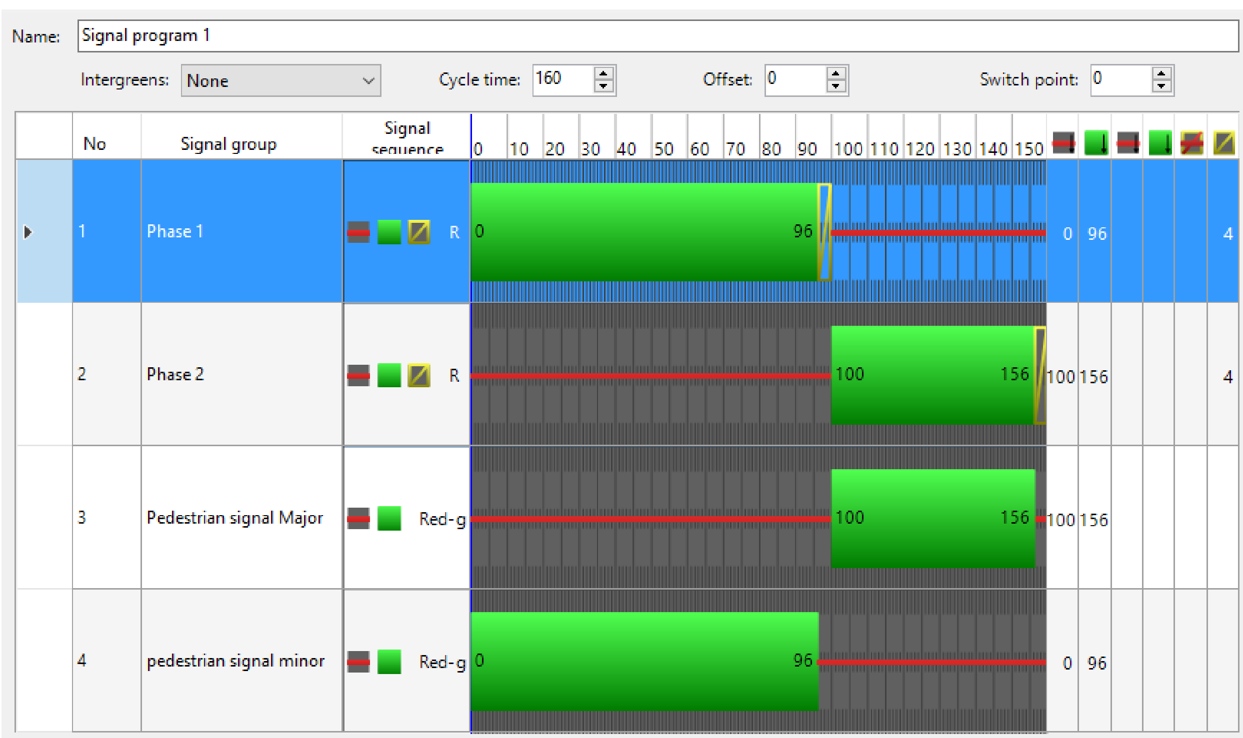


Figure 49 Signal program for Sealite Mehret signalized roundabout

3.8. Performance evaluation measures

The HCM states that each facility type that has a defined method for assessing capacity and level of service also has performance measures that can be calculated. These measures reflect the operating conditions of a facility, given a set of roadway, traffic, and control conditions. Travel speed and density on freeways, delay at signalized intersections, and walking speed for pedestrians are examples of performance measures that characterize flow conditions on a facility. One or more of the stated performance measures serves as the primary determinant of level of service. This LOS-determining parameter is called the service measure for each facility type. The service measure for signalized intersection is delay. It is used to determine level of service. However, HCM does not include a method for estimating performance measures for roundabouts. [**44**]

The FHWA guide on roundabouts states three performance measures are typically used to estimate the performance of a given roundabout design: degree of saturation, delay, and queue length. Each measure provides a unique perspective on the quality of service at which a roundabout will perform under a given set of traffic and geometric conditions. [**13**] The main performance evaluation measures used to compare the alternatives are queue length, intersection delay, travel time and level of service.

CHAPTER FOUR: RESULTS AND DISCUSSION

For the two study areas: Sealite Mehret and CMC roundabout, the alternative solutions presented in chapter three were modeled using VISSIM software. This chapter includes the analysis results and discussion. The performance measures used are average and maximum queue length, intersection delay, travel time along major road and level of service. First, the performance of each solution is evaluated. Then the alternatives are compared. Finally, the performance of the better alternatives is checked for forecasted traffic.

4.1. Performance Evaluation of alternatives to Sealite Mehret Roundabout

4.1.1. No- build alternative

Table 40 shows the performance evaluation results of Sealite Mehret roundabout under existing conditions. The Intersection level of service is F. Maximum queue lengths can reach to as high as 449.5m and average delay of an approach can be as high as 10.08 min/veh. The largest delay is recorded on the east approach which is the approach coming from CMC. Delay for the whole intersection is 6.64 min.

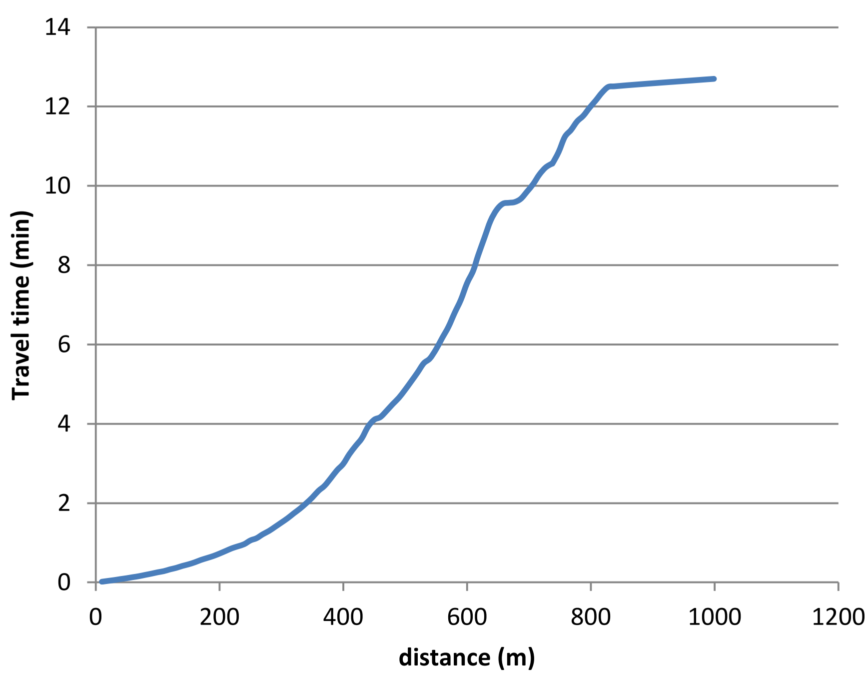


Figure 50 Distance - travel time plot of Sealite Mehret roundabout

Table 40 Performance Evaluation results of Sealite Mehret - Existing roundabout

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (min/veh)	Approach delay (min/veh)	Intersection delay (min/veh)	Intersection LOS
Sealite Mehret Roundabout	CMC	L	108.14	395.28	1011	12.70	10.08	6.64	F
		T			1126	8.42			
		R			477	8.17			
		U			86	11.58			
	Kara	L	99.65	285.63	868	3.69	3.56		
		T			60	3.70			
		R			104	1.40			
		U			128	4.41			
	Mebrat Hail	L	149.97	449.49	128	2.66	2.42		
		T			68	4.47			
		R			1040	2.22			
		U			16	4.67			
	Megenagna	L	10.95	70.21	156	5.72	6.39		
		T			1080	6.49			
		R			224	5.64			
		U			180	7.32			

4.1.2. Installation of Pedestrian Signals

Installing pedestrian signals at Sealite Mehret improves flow along CMC approach. Figure 51 shows the distance - travel time plot for the major road during peak hour. Total travel time is reduced to 7.16min; however, this is not sufficient to provide adequate level of service.

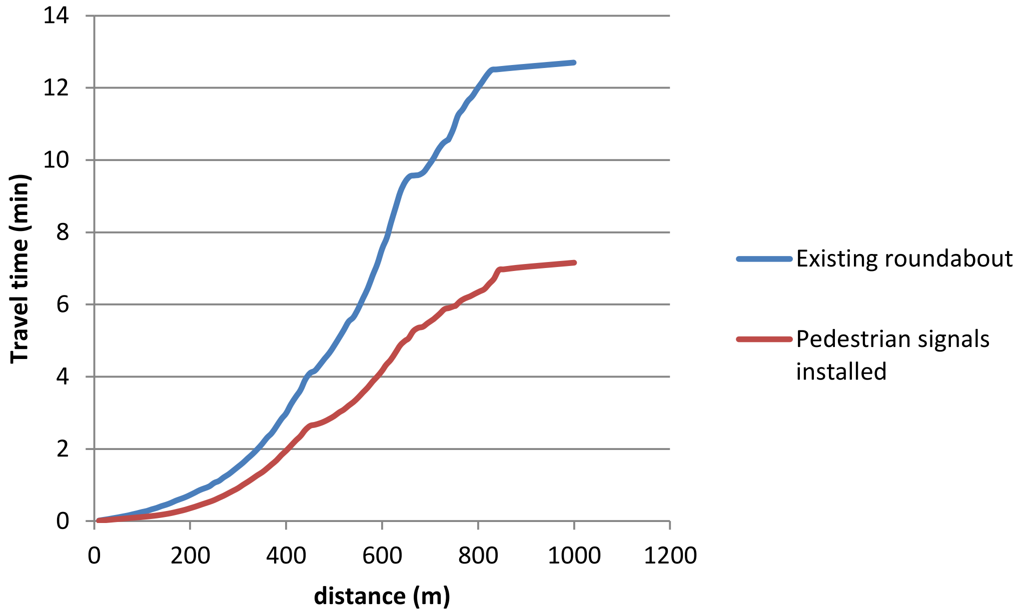


Figure 51 Distance - travel time plot of Sealite Mehret roundabout with pedestrian signals

4.1.3. Installation of Traffic Signals

Table 41 shows the performance evaluation results of Sealite Mehret roundabout if it was converted to a signalized intersection. The Intersection level of service is F. Intersection delay is 69.63 sec/veh. This is close to the delay estimated using HCM equations in Chapter three, which is 78.58sec/veh. Travel time along the major road headed west is 1.64 min. The travel time needed to traverse the same distance is 12.15min with the existing roundabout. Therefore, if traffic signals are installed, travel time will be reduced by 86.50%.

Table 41 Performance Evaluation results of Sealite Mehret - Signalized Intersection

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (sec/veh)	Approach delay (sec/veh)	Intersection delay (sec/veh)	Intersection LOS
Signalized Intersectio	CMC	L	130.79	236.35	1011	151.55	90.54	69.63	F
		R			563	20.61			
		T			1126	70.73			
		L	6.76	21.65	68	57.10	14.97		

	Kara	R			1040	3.04			
		T			128	89.48			
	Mebrat Hail	L	33.69	126.06	1080	53.96	57.45		
		R			224	76.37			
		T			156	54.46			
	Megenagna	L	38.10	86.93	60	103.35	94.55		
		R			232	5.28			
		T			868	117.80			

4.1.4. Converting the existing roundabouts to Signalized roundabouts

Table 42 shows the performance evaluation results of Sealite Mehret roundabout if it was converted to a signalized roundabout. The Intersection level of service is C. Intersection delay is 23.13 sec/veh. Travel time along the major road headed west is 1.09 min. The travel time needed to traverse the same distance is 12.15min with the existing roundabout. Therefore, if the existing roundabout is converted to a signalized roundabout, travel time will be reduced by 91.03%.

Table 42 Performance Evaluation results of Sealite Mehret - Signalized roundabout

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (sec/veh)	Approach delay (sec/veh)	Intersection delay (sec/veh)	Intersection LOS
Signalized Roundabout	CMC	L	32.61	178.68	1011	28.18	22.63	23.13	C
		R			563	3.86			
		T			1126	27.03			
	Kara	L	3.52	19.31	68	1.83	53.70		
		R			1040	58.25			
		T			128	44.26			
	Mebrat Hail	L	5.93	62.49	1080	0.86	5.59		
		R			224	16.82			
		T			156	22.25			
	Megenagna	L	6.41	44.49	60	32.39	13.78		
		R			232	0.32			
		T			868	16.09			

4.1.5. Comparisons between Alternatives under current traffic conditions – Sealite Mehret

4.1.5.1. Intersection and approach delay

The following chart shows that the alternative that results in the most reduction in intersection delay is the signalized roundabout, by 94.2%. The signalized intersection also shows significant reduction in intersection delay, by 82.5%.

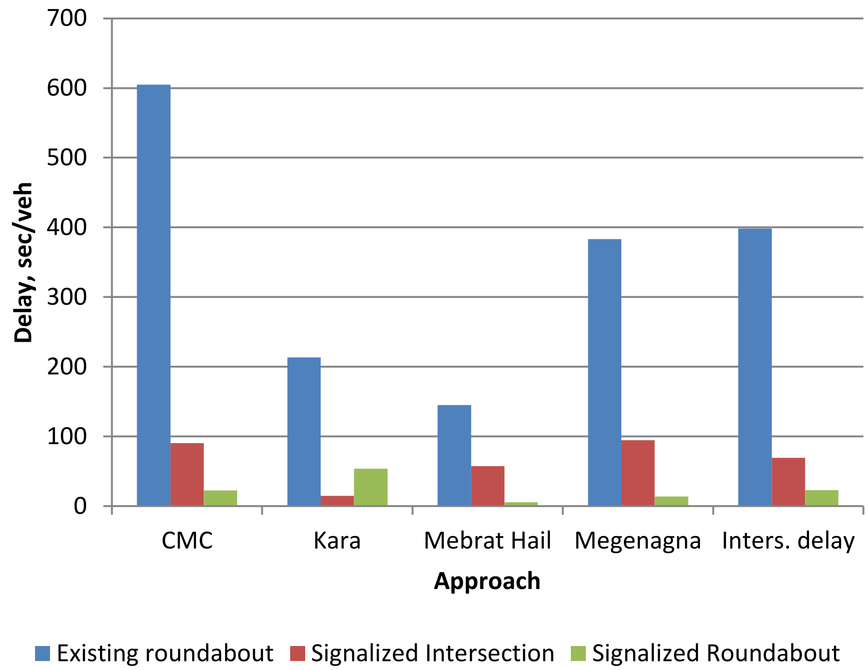


Figure 52 Intersection and approach delays, Sealite Mehret

4.1.5.2. Travel time along major road during morning peak hour

The alternatives that provide the least travel time along the major road headed west are signalized intersection and signalized roundabout with 1.64min and 1.09min, respectively. The addition of pedestrian signals also reduces travel time along the major road. However, it is not within acceptable limits.

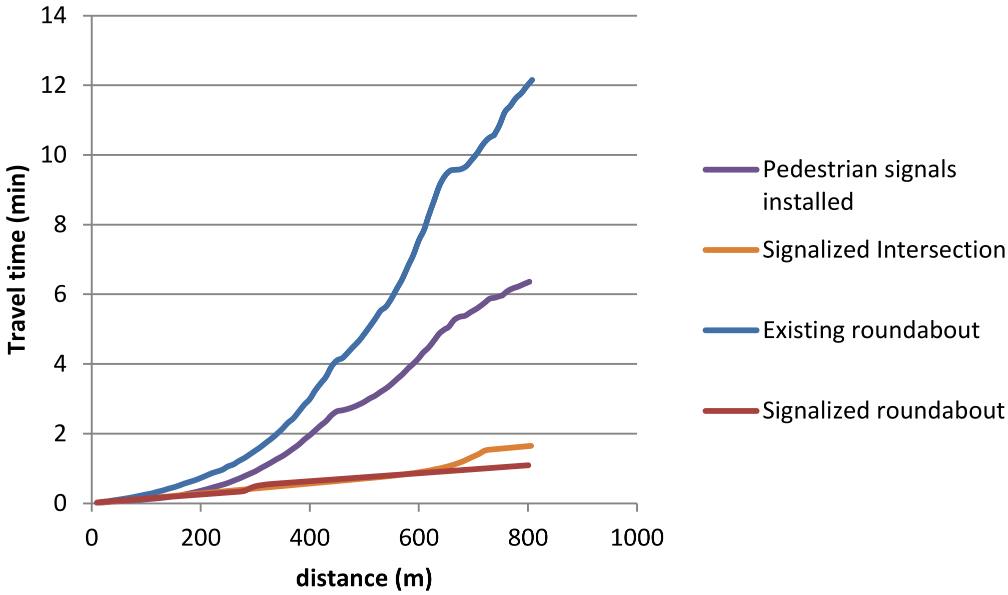


Figure 53 Travel time along major road headed west

4.1.5.3. Average and maximum queue lengths

Analysis of simulation results shows that the alternative signalized roundabout has lower average and maximum queue lengths on all approaches. Similar results are observed for the alternative of signalized intersection, except for the average queue length of CMC approach, which shows an increase.

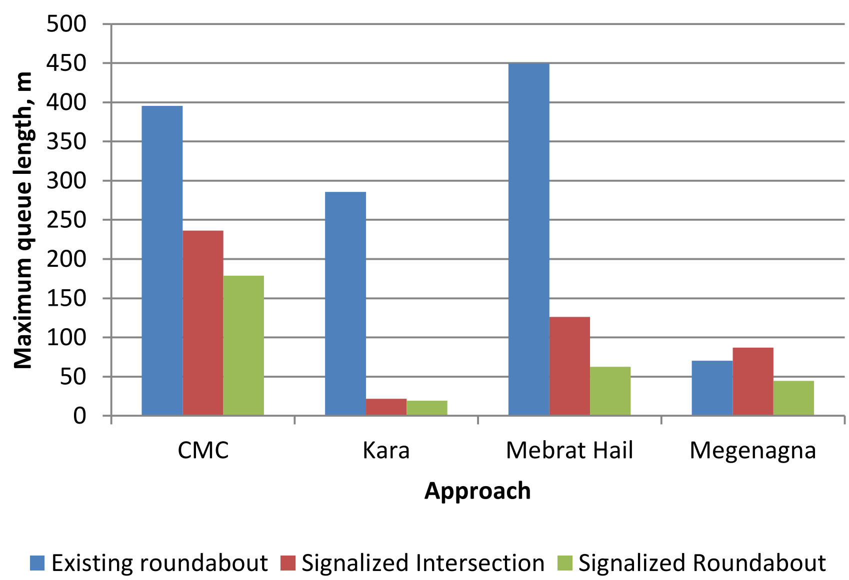


Figure 54 Maximum queue lengths per approach, Sealite Mehret

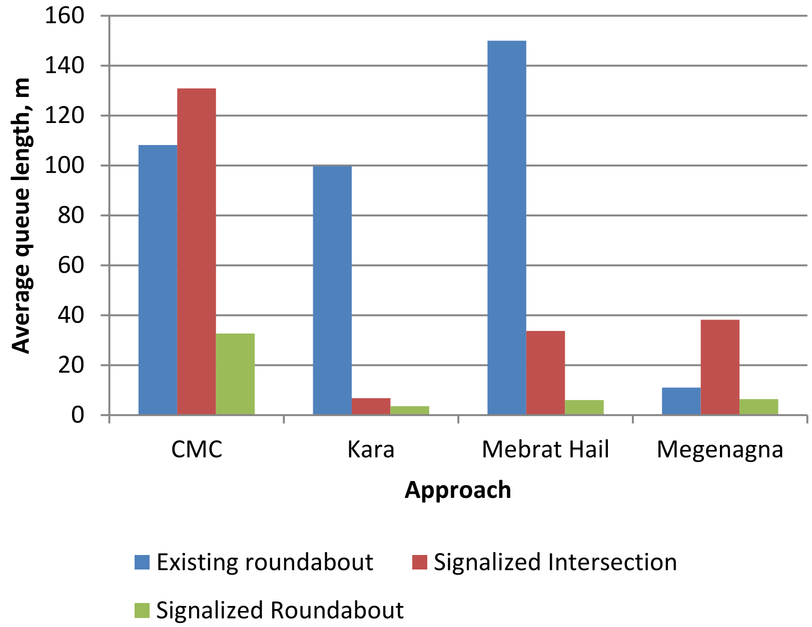


Figure 55 Average queue lengths per approach, Sealite Mehret

Analysis results show that, for existing traffic conditions converting Sealite Mehret roundabout to a signalized intersection or signalized roundabout will result in an improved performance of the intersection. The best alternative is signalized roundabout with intersection delay of 23.13 sec/veh and LOS C. The performance of the two alternatives will be evaluated under forecasted traffic in section 4.2.7.

4.2. Performance Evaluation of alternatives TO CMC Roundabout

4.2.1. No- build alternative

Table 43 shows the performance evaluation results of CMC roundabout under existing conditions. The Intersection level of service is F. Maximum queue lengths can reach to as high as 510.21m and average delay of an approach can be as high as 9.18min/veh. The largest delay is observed on the east approach which is the approach coming from Ayat. Delay for the whole intersection is 5.75 min.

Table 43 Performance Evaluation results of CMC - Existing roundabout

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (min/veh)	Approach delay (min/veh)	Intersection delay (min/veh)	Intersection LOS
CMC Roundabout	Ayat	T	180.15	444.35	1183	10.96	9.18	5.75	F
		L			373	8.11			
		R			613	6.58			
		U			288	8.79			
	CMC	T	31.97	66.00	202	2.18	3.69		
		L			22	3.34			
		R			498	1.53			
		U			823	5.38			
	Sunshine	T	234.27	510.21	65	2.40	3.08		
		L			43	2.90			
		R			618	3.14			
		U			18	4.06			
	Summit	T	22.77	110.93	48	2.54	3.40		
		L			849	4.59			
		R			438	1.22			
		U			43	3.14			

4.2.2. Installation of Pedestrian Signals

Installing pedestrian signals at CMC improves flow along the major road and reduces intersection delay by 66.1%.

Table 44 Distance - travel time plot of CMC roundabout with pedestrian signals

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (min/veh)	Approach delay (min/veh)	Intersection delay (min/veh)	Intersection LOS
CMC Roundabout with pedestrian signals	Ayat	T	184.06	369.85	1183	3.30	3.13	1.95	F
		L			373	3.21			
		R			613	2.88			
		U			288	2.87			
	CMC	T	0.00	0.00	202	1.03	1.12		
		L			22	0.99			
		R			498	0.89			
		U			823	1.28			
	Sunshine	T	0.45	12.94	65	1.18	1.12		
		L			43	0.90			
		R			618	1.13			
		U			18	0.92			
	Summit	T	0.96	30.69	48	1.45	1.22		
		L			849	1.32			
		R			438	1.00			
		U			43	1.26			

4.2.3. Redirecting U- turns away from the roundabouts

If U turns are directed away from the roundabout as explained in Chapter 3, performance of CMC roundabout will improve. Intersection delay will be reduced by 82.6%.

Table 45 Performance evaluation results of CMC, diverted U turns

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (min/veh)	Approach delay (min/veh)	Intersection delay (min/veh)	Intersection LOS
CMC Roundabout, diverted U turns	Ayat	T	181.85	393.79	1183	3.12	3.08	1.62	F
		L			373	3.34			
		R			613	2.91			
		U			288	2.96			
	CMC	T	0.00	0.00	202	1.62	1.42		
		L			22	2.09			
		R			498	0.97			
	Sunshine	T	2.51	69.14	65	4.07	3.54		
		L			43	4.47			
		R			618	3.59			
		U			18	0.22			
	Summit	T	6.17	122.74	48	3.29	3.00		
		L			849	3.32			
		R			438	2.63			
		U			43	0.46			

4.2.4. Installation of Traffic Signals

Table 46 shows the performance evaluation results of CMC roundabout if it was converted to a signalized intersection. The Intersection level of service is C. Intersection delay is 28.97 sec/veh. This is close to the delay estimated using HCM equations in Chapter three, which is 32.90 sec/veh. Travel time along the major road headed west is 1.28 min. The travel time needed to traverse the same distance is 10.96 min with the existing roundabout. Therefore, if traffic signals are installed, travel time will be reduced by 88.32%.

Table 46 Performance Evaluation results of CMC - Signalized Intersection

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (sec/veh)	Approach delay (sec/veh)	Intersection delay (sec/veh)	Intersection LOS
CMC Signalized Intersection	Ayat	L	17.29	123.29	373	28.77	26.10	28.97	C
		R			613	6.70			
		T			1183	35.19			
	CMC	L	17.57	96.58	22	32.86	45.59		
		R			202	2.24			
		T			498	64.91			
	Sunshine	L	7.56	109.86	43	31.91	23.69		
		R			618	1.13			
		T			48	31.83			
	Summit	L	4.32	38.13	849	39.23	27.47		
		R			438	2.10			
		T			48	50.88			

4.2.5. Converting the existing roundabouts to Signalized roundabouts

Table 47 shows the performance evaluation results of CMC roundabout if it was converted to a signalized roundabout. The Intersection level of service is C. Intersection delay is 22.08 sec/veh. Travel time along the major road headed west is 1.30 min. The travel time needed to traverse the same distance is 10.96 min with the existing roundabout. Therefore, if the existing roundabout is converted to a signalized roundabout, travel time will be reduced by 88.14%.

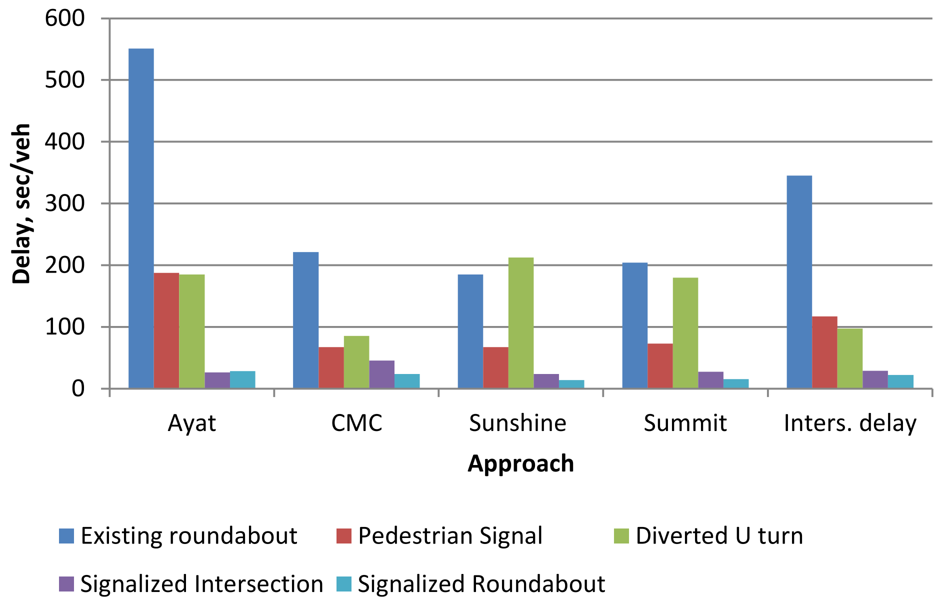
Table 47 Performance Evaluation results of CMC - Signalized roundabout

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (sec/veh)	Approach delay (sec/veh)	Intersection delay (sec/veh)	Intersection LOS
CMC Signalized Roundabout	Ayat	L	43.25	263.19	373	34.05	28.44	22.08	C
		R			613	2.41			
		T			1183	39.92			
	CMC	L	17.21	148.09	22	41.43	23.56		
		R			202	37.25			
		T			498	12.64			
	Sunshine	L	41.42	295.72	43	76.49	14.00		
		R			618	1.76			
		T			48	16.97			
	Summit	L	4.66	52.91	849	19.58	15.22		
		R			438	5.61			
		T			48	25.72			

4.2.6. Comparisons between Alternatives under current traffic conditions – CMC

4.2.6.1. Intersection and approach delay

The following chart shows that the alternative that results in the most reduction in intersection delay is the signalized roundabout, by 93.6%. The signalized intersection also shows significant reduction in intersection delay, by 91.6%.



Intersection and approach delays, CMC

4.2.6.2. Travel time along major road during morning peak hour

The alternatives that provide the least travel time along the major road headed west are signalized intersection and signalized roundabout with 1.28 min and 1.30 min, respectively. The addition of pedestrian signals reduces travel time to 3.13 min, while diverting U turns reduces travel time to 3.08 min along the same direction. Travel time with existing conditions is 10.96 min. Figure 56 compares the travel times of the alternatives with that of the existing roundabout.

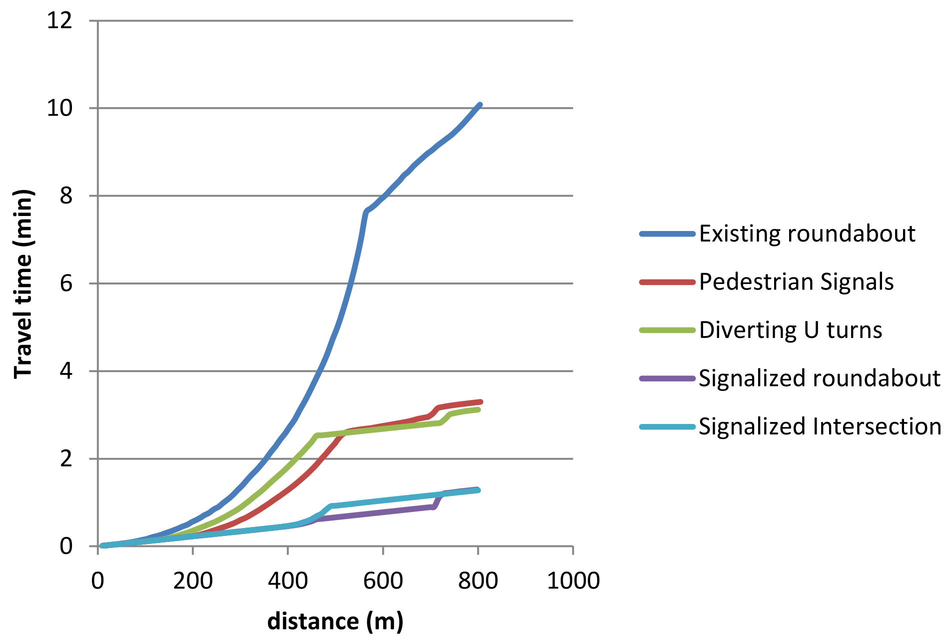


Figure 56 Travel times along major road, CMC

4.2.6.3. Average and maximum queue lengths

The following charts compare the average and maximum queue lengths of the existing roundabout with the alternatives. All alternatives show reduced queue lengths on most approaches.

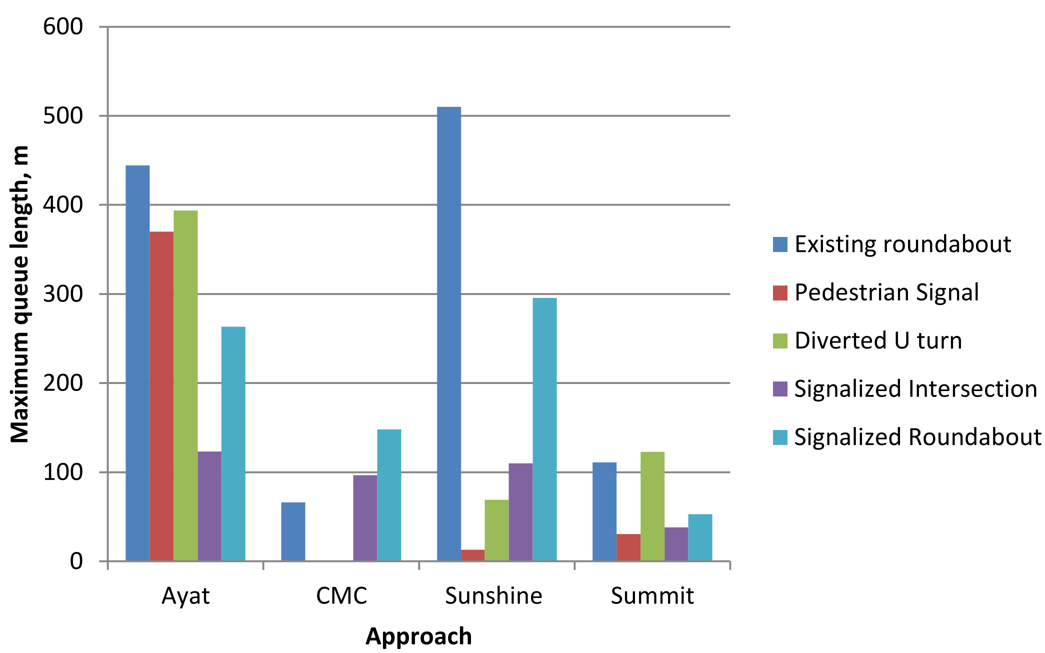


Figure 57 Maximum queue lengths on approaches, CMC

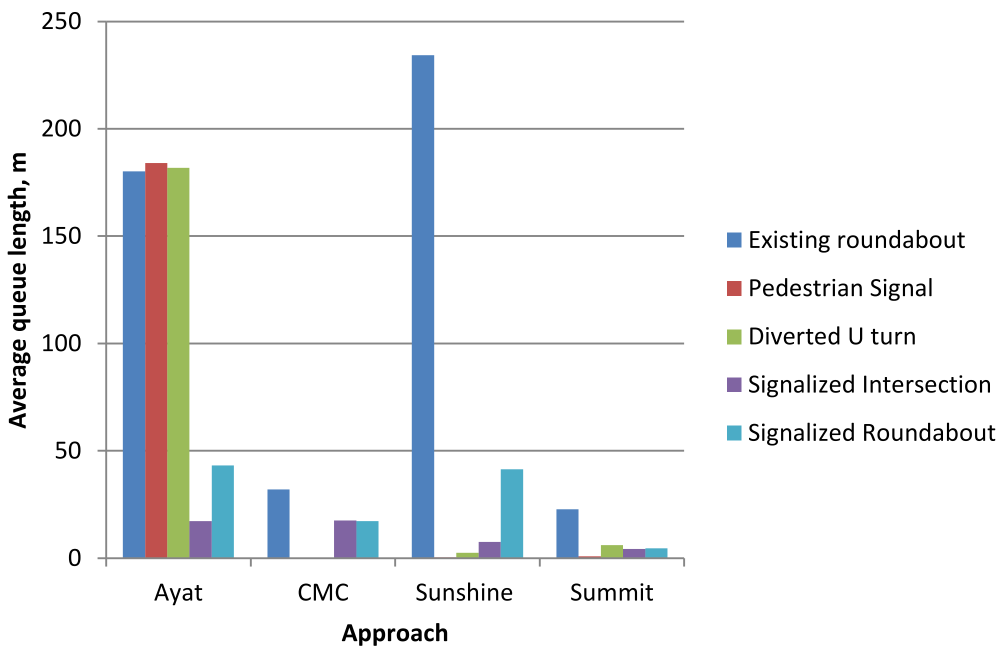


Figure 58 Average queue lengths on approaches, CMC

Analysis results show that, for existing traffic conditions the best alternatives are converting the roundabout to signalized intersection or signalized roundabout. Both alternatives result in acceptable LOS of C and intersection delay less than 30 sec/veh. The performance of these alternatives will be evaluated under forecasted traffic conditions in the next section.

4.2.7. Performance Evaluation of roundabouts during off-peak period

Based on the data collected during off – peak periods (2:00 PM- 4:00 PM) for both locations, performance evaluation was carried out. The image in figure 59 shows what traffic flow is like during off – peak periods at CMC roundabout. The traffic flow for this period was modeled using VISSIM and simulation results indicate a LOS of A and an intersection delay of 5.01 sec/veh for CMC roundabout. At Sealite Mehret roundabout heavier traffic flow is recorded and the level of service during off peak periods is C with intersection delay of 29.23 sec/veh. These results show that the roundabouts are performing at acceptable level of services during off – peak periods and do not require any modifications. Table 48 and 49 show the performance evaluation results of the roundabouts.

Table 48 Performance Evaluation of CMC roundabout during off peak periods

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (sec/veh)	Approach delay (sec/veh)	Intersection delay (sec/veh)	Intersection LOS
CMC Roundabout	Ayat	L	1.53	17.95	116	7.36	5.62	5.01	A
		R			136	3.07			
		T			244	5.46			
		U			364	6.12			
	Sunshine	L	0.23	11.84	92	8.93	5.71		
		R			424	5.94			
		T			136	2.63			
		U			408	5.78			
	Summit	L	0.52	18.02	196	6.02	4.21		
		R			300	2.46			
		T			48	2.90			
		U			64	7.89			
	CMC	L	0.03	5.43	36	7.11	3.14		
		R			332	2.31			
		T			44	5.47			
		U			10	6.21			

Figure 59 Off - peak traffic of CMC roundabout

Table 49 Performance Evaluation of Sealite Mehret roundabout during off peak periods

Intersection	Approach	Movement	Average queue length (m)	Maximum queue length(m)	Volume (veh/hr)	Delay (sec/veh)	Approach delay (sec/veh)	Intersection delay (sec/veh)	Intersection LOS
Sealite Mehret Roundabout	Mebrat Hail	L	64.67	167.74	277	82.40	51.24	29.23	C
		R			385	56.84			
		T			292	50.02			
		U			508	30.71			
	Kara	L	11.22	64.88	215	55.98	20.67		
		R			862	22.15			
		T			492	13.80			
		U			600	11.53			
	CMC	L	11.90	48.31	154	10.66	24.76		
		R			569	23.21			
		T			77	53.09			
		U			15	82.96			
	Megenagna	L	21.84	93.13	38	26.17	17.50		
		R			723	14.02			
		T			77	44.59			
		U			11	26.49			

4.2.8. Comparison of alternatives under forecasted traffic

The best alternatives for both study areas, under current traffic conditions are signalized intersection and signalized roundabout. In this section the performance of these alternatives is going to be evaluated for future traffic. The traffic growth rates that are used were prepared by CORE Consulting Engineers PLC based on demand elasticity and GDP growth rate. [**2**] For the years from 2018-2020 an average growth rate of 7.2% is used for passenger cars, while a factor of 8% is used for heavy vehicles. For the years of 2020-2033 an average growth rate of 6.6% is used for passenger cars, while a factor of 6.8% is used for heavy vehicles. For pedestrian traffic, growth rate of 3.8% is used, which is the annual population growth factor according to the Ethiopian Statistics Agency. Traffic is forecasted for the next 15 years. Tables 50 to 57 show the forecasted traffic volumes of Sealite Mehret and CMC roundabouts.

Table 50 CMC roundabout, 2 years forecasted traffic

CMC Roundabout				**2 years**		
Count Period: Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	Ayat	48	152	79	37	315
	Megenagna	1	12	31	51	95
	Sunshine	3	4	42	1	50
	Summit	62	3	32	3	100
passenger cars	Ayat	381	1210	627	295	2513
	Megenagna	24	220	542	896	1682
	Sunshine	47	70	669	19	806
	Summit	915	52	472	46	1485
Total	Ayat	429	1362	706	332	2828
	Megenagna	25	232	573	947	1777
	Sunshine	49	75	711	21	856
	Summit	977	55	504	49	1585

Table 51 CMC roundabout, 5 years forecasted traffic

CMC Roundabout				**5 years**		
Count Period : Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	Ayat	66	211	109	51	438
	Megenagna	2	17	43	70	132
	Sunshine	4	6	58	2	69
	Summit	86	5	44	4	139
passenger cars	Ayat	530	1681	871	409	3492
	Megenagna	33	306	753	1245	2337
	Sunshine	65	98	930	27	1120
	Summit	1271	72	656	64	2063
Total	Ayat	597	1892	980	461	3930
	Megenagna	35	323	796	1315	2469
	Sunshine	69	104	988	29	1189
	Summit	1357	77	700	69	2202

Table 52 CMC roundabout, 10 years forecasted traffic

CMC Roundabout					**10 years**	
Count Period: Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	Ayat	92	293	152	71	609
	Megenagna	3	24	59	98	183
	Sunshine	6	8	80	2	97
	Summit	119	7	61	6	193
passenger cars	Ayat	737	2336	1210	569	4852
	Megenagna	46	425	1047	1730	3247
	Sunshine	90	136	1292	38	1556
	Summit	1767	100	911	89	2867
Total	Ayat	829	2629	1362	640	5460
	Megenagna	49	449	1106	1827	3431
	Sunshine	95	144	1372	40	1652
	Summit	1885	107	973	95	3060

Table 53 CMC roundabout, 15 years forecasted traffic

CMC Roundabout					**15 years**	
Count Period: Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	Ayat	113	357	185	87	741
	Megenagna	3	29	72	119	223
	Sunshine	7	10	98	3	118
	Summit	145	8	75	7	235
passenger cars	Ayat	897	2846	1475	693	5910
	Megenagna	56	517	1275	2107	3956
	Sunshine	110	166	1574	46	1895
	Summit	2152	122	1110	109	3493
Total	Ayat	1010	3203	1660	780	6652
	Megenagna	60	546	1347	2226	4179
	Sunshine	116	176	1672	49	2013
	Summit	2297	130	1185	116	3728

Table 54 Sealite Meret roundabout, 2 years forecasted traffic

Sealite Mehret Roundabout				**2 years**		
Count Period: Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	CMC	118	131	56	10	315
	Megenagna	5	71	9	10	95
	Kara	3	5	42	1	50
	Mebrat Hail	66	10	14	11	100
passenger cars	CMC	1046	1164	493	89	2793
	Megenagna	64	927	111	137	1239
	Kara	75	142	1154	18	1390
	Mebrat Hail	1176	170	244	196	1786
Total	CMC	1164	1296	549	99	3107
	Megenagna	69	999	120	147	1334
	Kara	78	147	1196	18	1440
	Mebrat Hail	1242	179	258	207	1886

Table 55 Sealite Meret roundabout, 5 years forecasted traffic

Sealite Mehret Roundabout				**5 years**		
Count Period: Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	CMC	164	182	77	14	438
	Megenagna	7	99	12	15	132
	Kara	4	7	58	1	69
	Mebrat Hail	91	13	19	15	139
passenger cars	CMC	1453	1618	686	123	3880
	Megenagna	89	1289	154	190	1722
	Kara	105	197	1604	25	1931
	Mebrat Hail	1634	236	339	272	2482
Total	CMC	1617	1800	763	137	4318
	Megenagna	96	1387	166	205	1854
	Kara	109	204	1662	26	2000
	Mebrat Hail	1726	249	358	288	2621

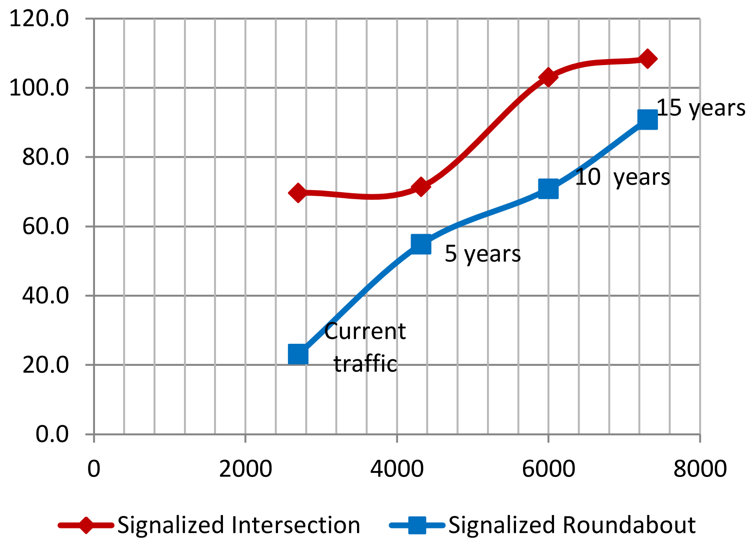
Table 56 Sealite Meret roundabout, 10 years forecasted traffic

Sealite Mehret Roundabout				**10 years**		
Count Period : Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	CMC	228	254	107	19	608
	Megenagna	9	137	16	20	183
	Kara	5	10	80	1	97
	Mebrat Hail	127	18	26	21	193
passenger cars	CMC	2019	2248	953	171	5391
	Megenagna	124	1791	215	264	2393
	Kara	146	274	2228	34	2683
	Mebrat Hail	2271	328	471	378	3449
Total	CMC	2247	2502	1060	191	6000
	Megenagna	133	1928	231	284	2576
	Kara	151	284	2309	36	2779
	Mebrat Hail	2398	346	497	400	3642

Table 57 Sealite Meret roundabout, 15 years forecasted traffic

Sealite Mehret Roundabout				**15 years**		
Count Period : Morning peak hour	Approach	Movement				Total (veh/hr)
		Left	Through	Right	U turn	
Heavy vehicles	CMC	277	309	131	24	741
	Megenagna	12	167	20	25	223
	Kara	6	12	98	2	118
	Mebrat Hail	155	22	32	26	235
passenger cars	CMC	2460	2739	1160	209	6568
	Megenagna	151	2181	261	322	2915
	Kara	177	334	2715	42	3268
	Mebrat Hail	2766	400	574	461	4201
Total	CMC	2737	3047	1291	232	7309
	Megenagna	162	2349	281	346	3139
	Kara	184	346	2812	43	3386
	Mebrat Hail	2921	422	606	487	4436

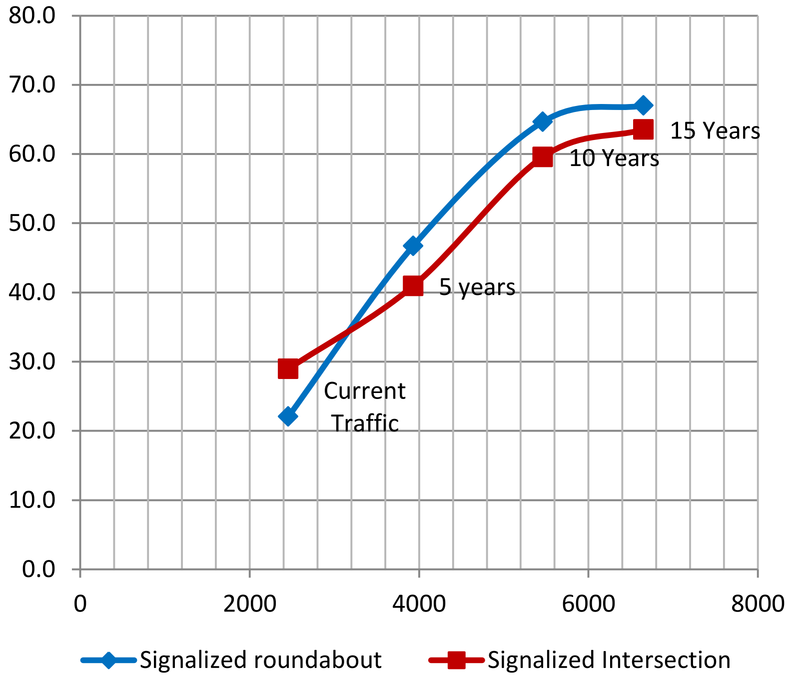
The signalized roundabout and signalized intersection alternatives were evaluated for both locations, under 5, 10 and 15 years forecasted traffic using VISSIM. Comparisons were done based on Intersection delay and travel time along major road headed west, which is the main direction of flow during morning peak hour. The analysis results are discussed below.



Year	VOL (veh/hr)	Intersection delay (sec/veh)	
		SR	SI
0	2700	23.1	69.6
5	4318	54.8	71.3
10	6000	70.8	103.0
15	7309	90.8	108.3

Figure 60 Comparison of Alternatives under forecasted traffic, Sealite Mehret

Comparison of alternatives under forecasted traffic for Sealite Mehret roundabout based on intersection delay (Figure 60) shows that the delay of signalized roundabout alternative is less than that of signalized intersection alternative for 5, 10 and 15 years forecast. The intersection delay for 15 years forecast is 90.8 sec/veh for signalized roundabout and 108.3 sec/veh for signalized intersection. Travel time comparisons (Figure 66) show that both options provide similar travel times with differences less than 30 sec. Travel time for 15 year forecast is 3.64min for signalized roundabout option and 3.61 min for signalized intersection option along the major road. The better alternative based on intersection delay for Sealite Mehret roundabout is signalized roundabout, but both options are viable solutions.



Year	VOL (veh/hr)	Intersection delay (sec/veh)	
		SR	SI
0	2457	22.1	29.0
5	3930	46.7	40.9
10	5460	64.7	59.6
15	6652	67.0	63.5

SR – Signalized roundabout
SI – Signalized Intersection

Figure 61 Comparison of alternatives under forecasted traffic, CMC

The results of the alternatives for CMC roundabout are different when it comes to intersection delay. The intersection delay of signalized roundabout (Figure 61) is less than signalized intersection option for the current traffic but it is greater for 5,10 and 15 years forecast by 5 seconds on average. Delay for 15 year forecast is 67.0 sec/veh for signalized roundabout and 63.5 sec/veh for signalized intersection option. Travel time comparisons (Figure 67) show that both options have similar travel times differing by less than 30 seconds. Travel time for 15 year forecast is 3.05min for signalized roundabout and 3.26min for signalized intersection. Based on these results it can be concluded that both options are viable. However, since signalized roundabout alternative provides the option of functioning as a normal roundabout during off peak periods, and reduces risk of right angle crashes, signalized roundabout is preferred.

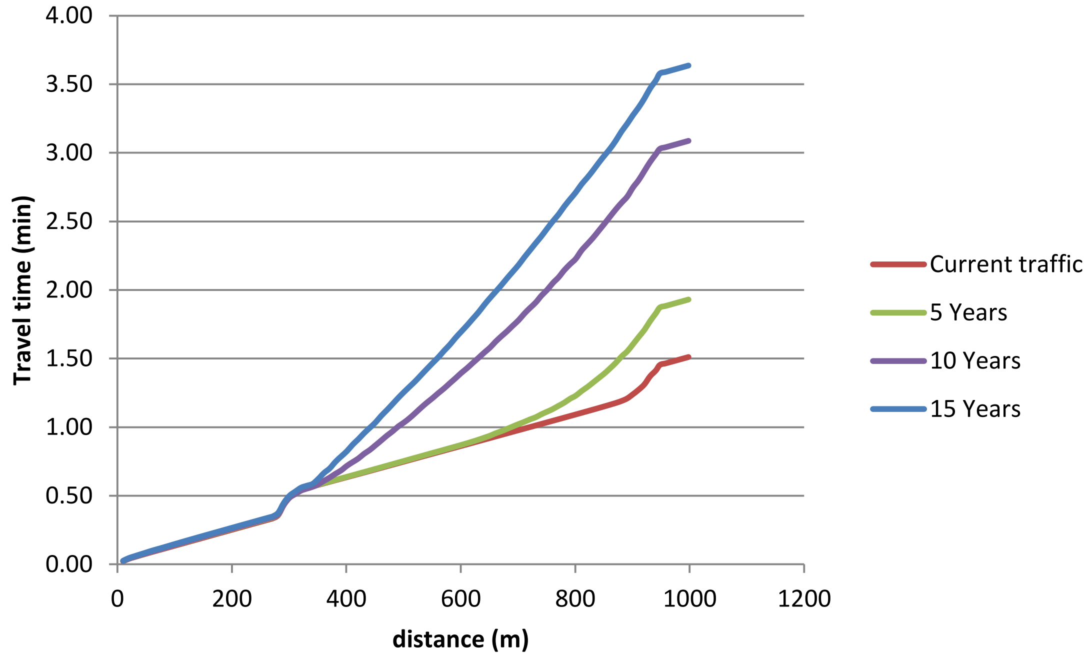


Figure 62 Travel times of Sealite Mehret Signalized roundabout under forecasted traffic

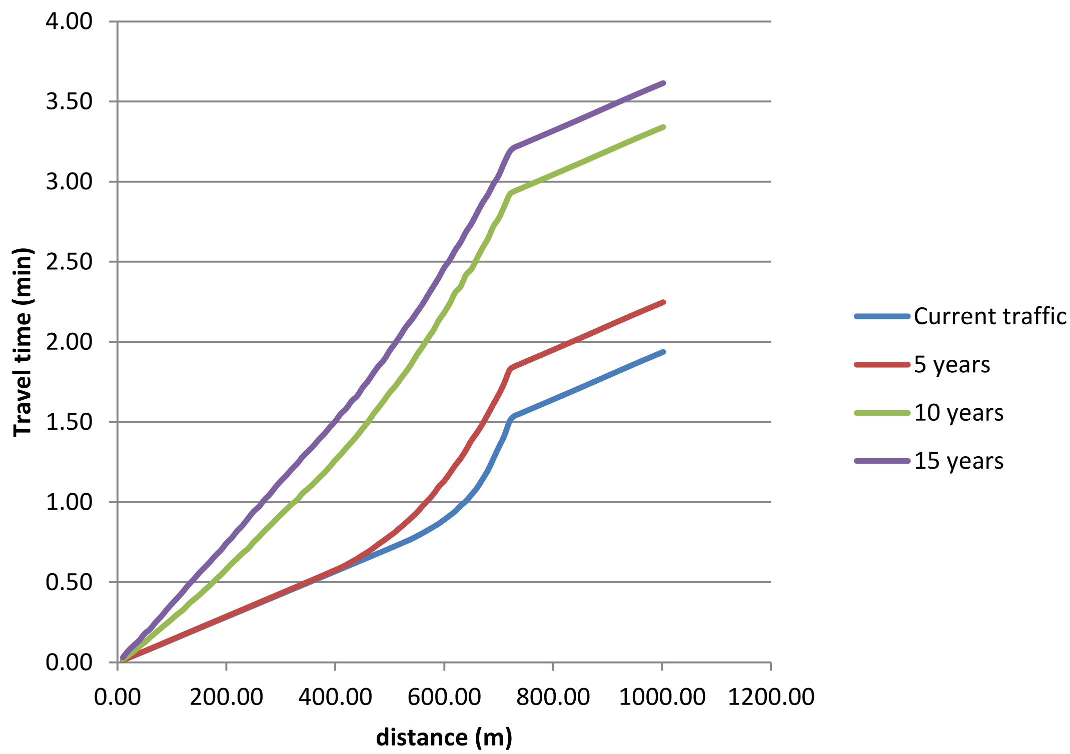


Figure 63 Travel times of Sealite Mehret Signalized Intersection under forecasted traffic

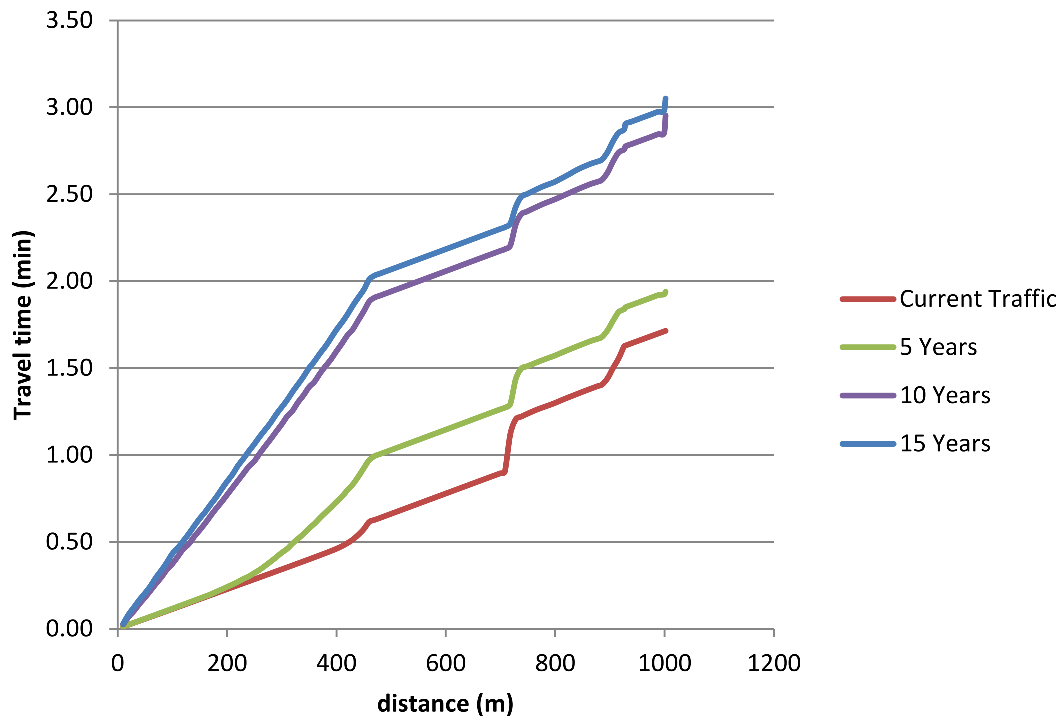


Figure 64 Travel times of CMC Signalized roundabout under forecasted traffic

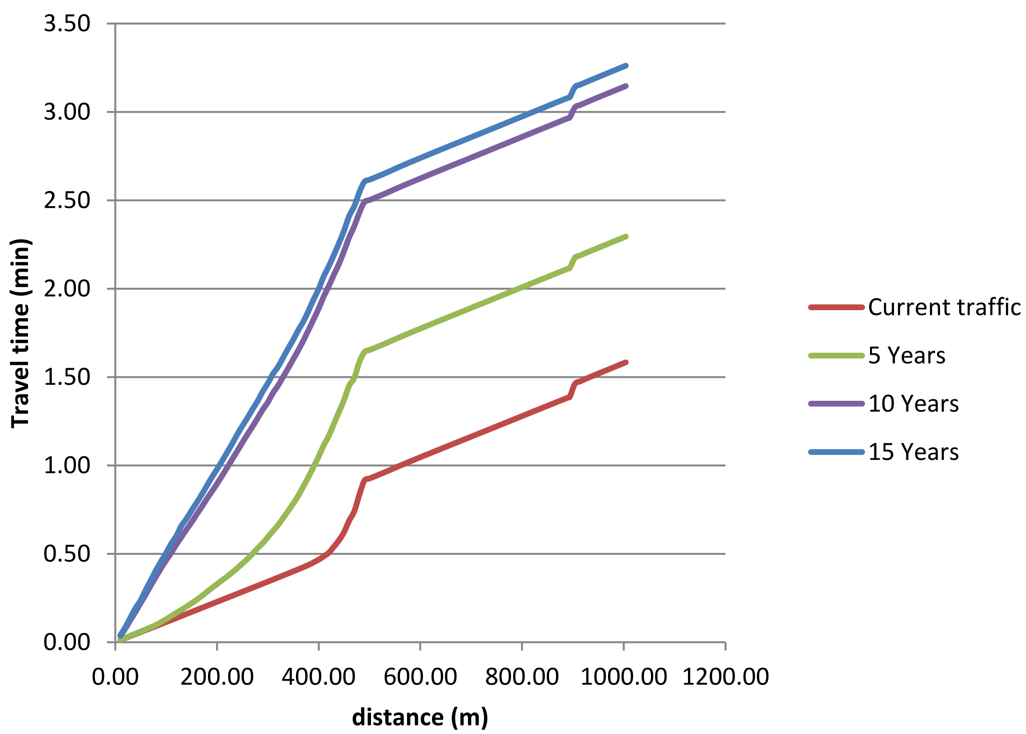


Figure 65 Travel times of CMC Signalized Intersection under forecasted traffic

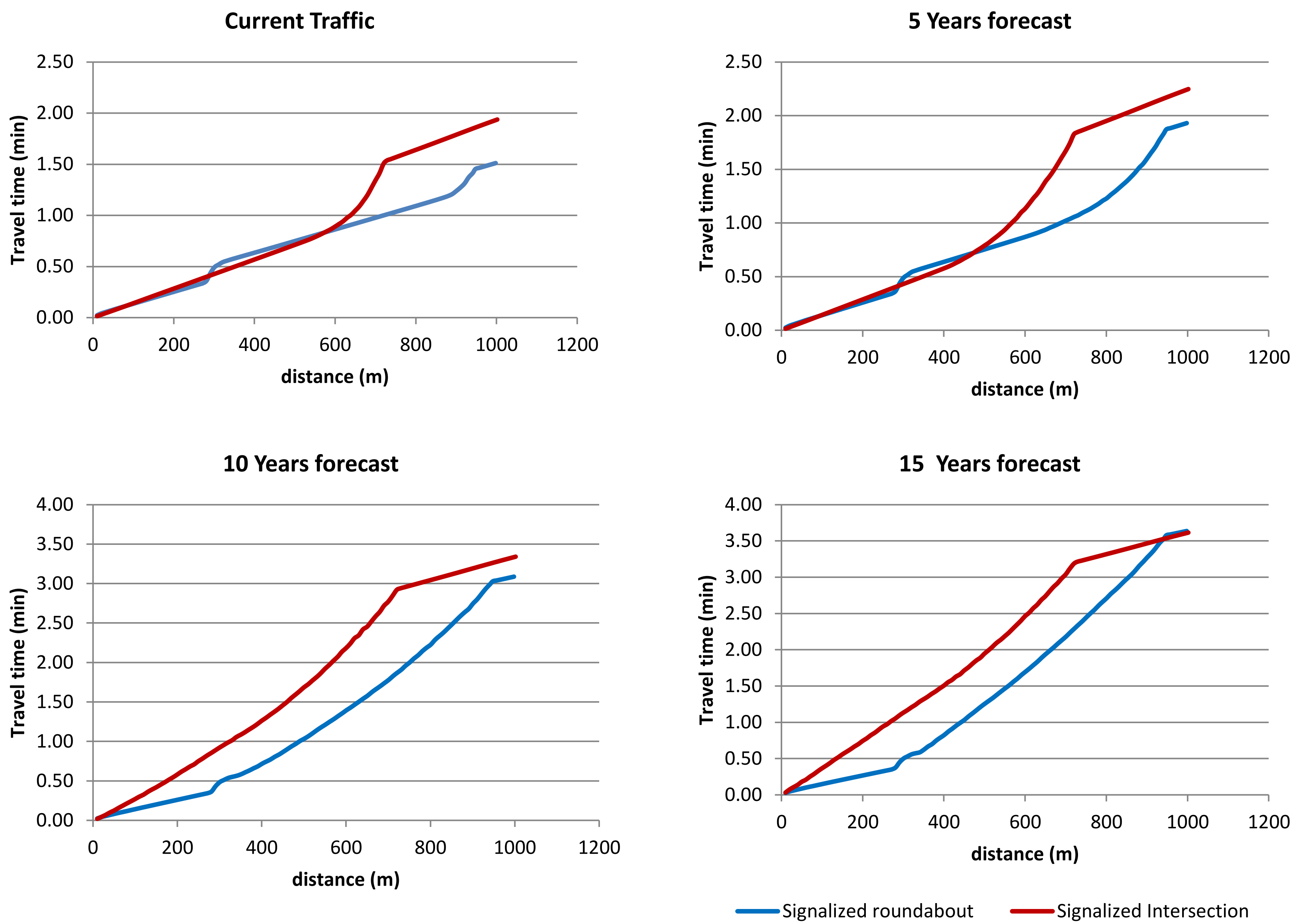


Figure 66 Travel times of Sealite Mehret Signalized roundabout under forecasted traffic, Signalized roundabout versus signalized intersection

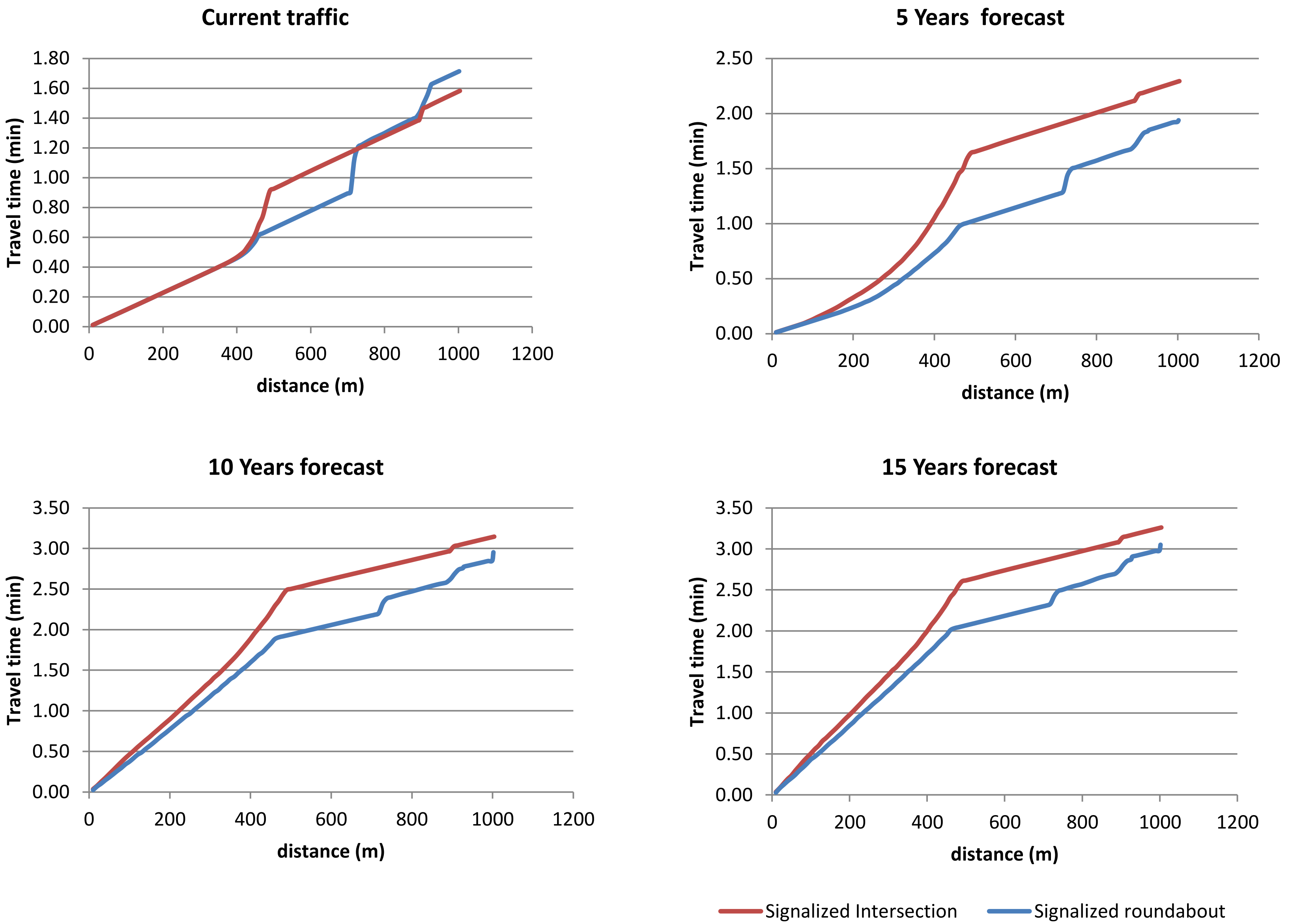


Figure 67 Travel times of CMC Signalized roundabout under forecasted traffic, Signalized roundabout versus signalized intersection

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

Previous studies done on the same locations have studied the effects of the LRT line on the existing roundabout and these studies have indicated that the introduction of the LRT line brings about additional delay and deteriorations in level of service [**2**,**3**]. This study focuses on mitigating this effect by considering alternative at grade junction designs.

The study areas were at two locations within the Addis Ababa East – West LRT corridor specifically, Sealite Mehert and CMC roundabouts. These roundabouts are both at grade with LRT lines passing through the center island.

During the initial stages of the study, the aim was to understand current performance of the roundabouts to create a baseline for the next stages. The subsequent stages involved comparison of alternative designs to the existing roundabouts. The alternative designs considered are signalized intersection, signalized roundabout, installation of pedestrian signals and diverting U turns to other locations. The alternatives and the existing roundabout were modeled using VISSIM simulation software and compared based on queue length, level of service, travel time and intersection delay under current and future traffic conditions.

It was found that the best alternatives for both study areas, under current traffic conditions are signalized intersection and signalized roundabout. Up on comparing these alternatives under forecasted traffic conditions, analysis results showed that both options have similar travel times and delay. Both alternatives are viable options. However, since signalized roundabout alternative provides the option of functioning as a normal roundabout during off peak periods, and reduces risk of right angle crashes, signalized roundabout is preferred.

This study identifies major problems associated with the East – West LRT line and attempts to propose solutions to each problem and establishes that signalized roundabouts can be a solid and even a better alternative to signalized intersections, especially at intersections with low off – peak volumes.

Recommendations

It is the recommendation of this study, that in order to improve the current performance of the existing Sealite Meheret and CMC roundabouts, signals should be added. The conventional roundabouts can be converted to signalized roundabouts; this would be a less costly alternative than converting to signalized intersections or grade separation.

Future Research can focus on

- Comparison of conventional roundabout, signalized intersection and signalized roundabouts at LRT grade crossings in terms of safety.
- Further improving the performance of the signalized alternatives by considering actuated signals and transit signal priority.

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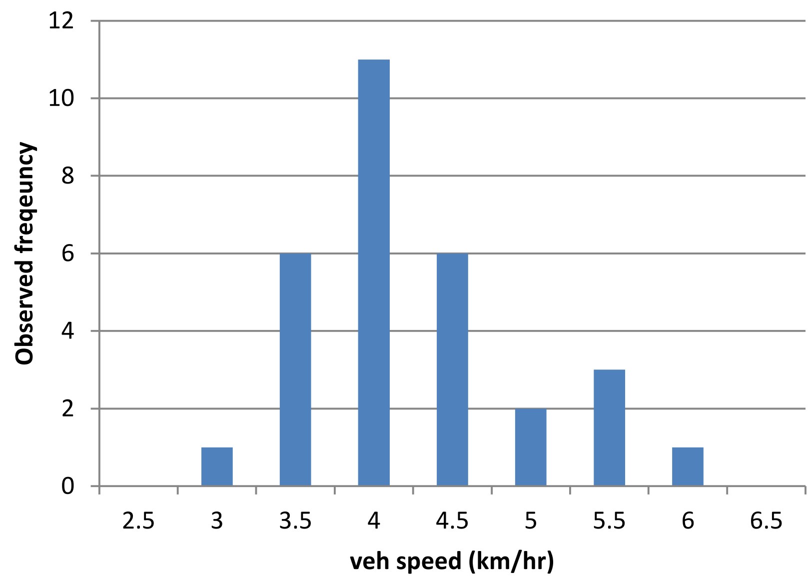
APPENDIX A RAW DATA

Table 58 Peak hour circulating speeds, CMC roundabout

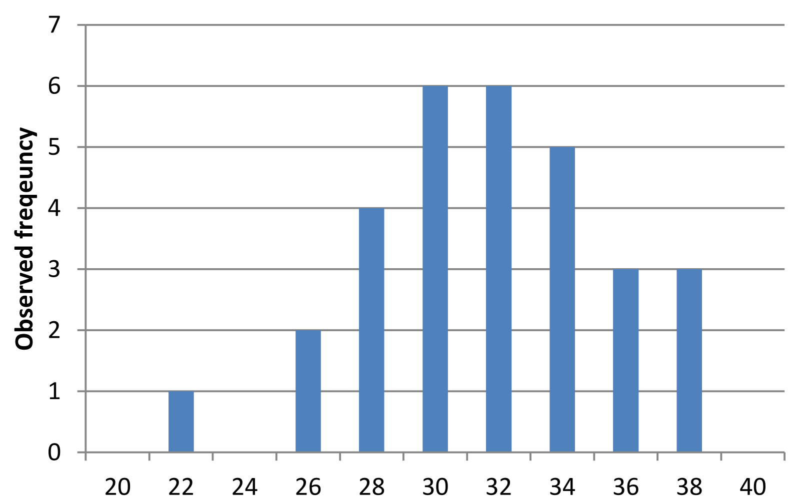
No	Speed (km/hr)	No	Speed (km/hr)
1	2.77	16	3.33
2	2.98	17	3.58
3	3.33	18	3.93
4	4.62	19	3.47
5	3.60	20	2.67
6	4.05	21	2.47
7	3.18	22	4.12
8	3.80	23	4.15
9	3.53	24	4.25
10	2.42	25	4.33
11	3.07	26	3.42
12	2.45	27	3.64
13	3.17	28	3.66
14	3.62	29	3.68
15	2.82	30	3.69



Standard deviation	0.72	Median speed (km/hr)	3.76
Sample size	2	Maximum speed (km/hr)	5.54
Average speed (km/hr)	3.98	Minimum speed (km/hr)	2.90

Table 59 Off-Peak hour circulating speeds, CMC roundabout

No	Speed (km/hr)	No	Speed (km/hr)
1	24.88	16	30.79
2	24.38	17	26.14
3	27.2	18	25.76
4	26.18	19	22.54
5	26.52	20	21.91
6	32.22	21	22.58
7	36.53	22	27.99
8	24.91	23	21.77



9	28.78	24	28.42					
10	28.52	25	33.2					
11	25.26	26	30.69					
12	23.1	27	30.13					
13	21.37	28	26.36		Standard deviation	3.92	Median speed (km/hr)	30.56
14	27.49	29	26.98		Sample size	15	Maximum speed (km/hr)	37.57
15	24.72	30	23.93		Average speed (km/hr)	30.56	Minimum speed (km/hr)	21.98

Table 60 Speeds at level crossings

No	Speed (km/hr)	No	Speed (km/hr)					
1	7.71	16	3.86					
2	7.71	17	4.91					
3	9.00	18	4.15					
4	6.00	19	3.60					
5	6.00	20	4.91					
6	4.91	21	3.38					
7	9.00	22	4.50					
8	5.40	23	5.40					
9	5.40	24	4.91					
10	5.40	25	4.91					
11	7.71	26	6.75					
12	5.40	27	6.00					
13	4.91	28	4.91		Standard deviation	1.42	Median speed (km/hr)	5.40
14	6.00	29	5.40		Sample size	8	Maximum speed (km/hr)	9.00
15	6.00	30	6.75		Average speed (km/hr)	5.70	Minimum speed (km/hr)	3.38

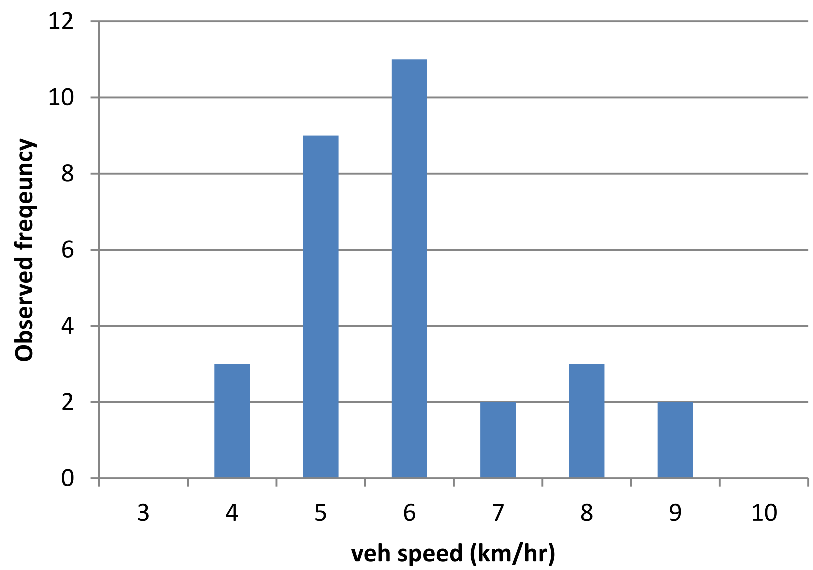
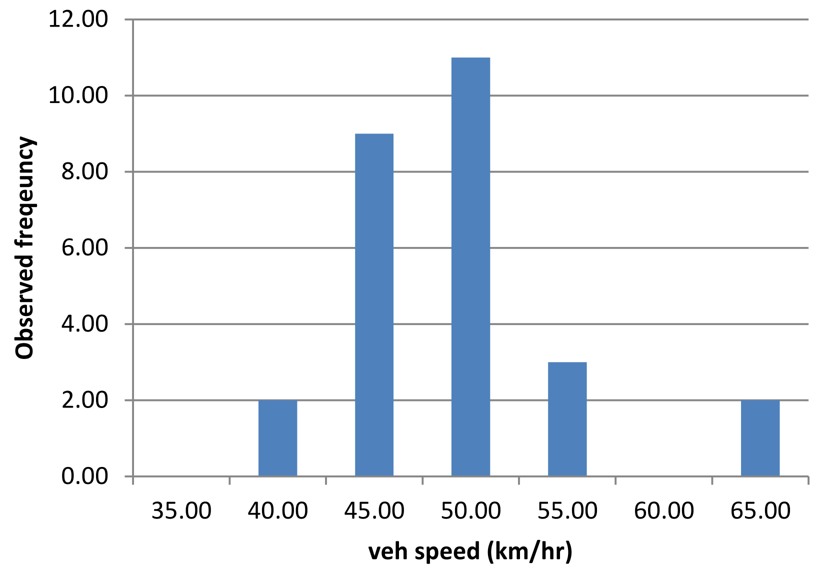


Table 61 Approach Speeds along major road

No	Speed (km/hr)	No	Speed (km/hr)
1	40.50	16	44.18
2	44.18	17	48.60
3	44.18	18	44.18
4	48.60	19	54.00
5	40.50	20	69.43
6	48.60	21	69.43
7	40.50	22	48.00
8	37.38	23	48.60
9	54.00	24	48.60
10	54.00	25	48.60
11	46.29	26	44.18
12	36.00	27	60.75
13	46.29	28	69.43
14	64.80	29	48.60
15	46.29	30	44.18



Standard deviation	7.40	Median speed (km/hr)	48.30
Sample size	23	Maximum speed (km/hr)	69.43
Average speed (km/hr)	44.56	Minimum speed (km/hr)	36.00

APPENDIX B SIMULATION OUTPUT USED TO DETERMINE TRAVEL TIME

VISSIM Simulation output used to determine travel time for Sealite Mehret roundabout

Table 62 Simulation output

Sim. run	Link Distance	Cumulative distance	Density	Relative delay	Speed (km/hr)	Time (min)	cumulative time	Volume
1	10	10	77.17	33.56%	34.75	0.017	0.017	2682.03
1	10	20	94.39	46.55%	28.10	0.021	0.039	2652.21
1	10	30	90.83	45.47%	28.81	0.021	0.059	2617.15
1	10	40	105.82	53.98%	24.39	0.025	0.084	2581.19
1	10	50	94.86	49.43%	26.83	0.022	0.106	2545.40
1	10	60	109.28	56.66%	22.98	0.026	0.133	2511.46
1	10	70	102.48	54.19%	24.23	0.025	0.157	2483.23
1	10	80	132.68	65.03%	18.44	0.033	0.19	2446.61

1	10	90	123.34	62.85%	19.52	0.031	0.221	2408.17
1	10	100	135.90	66.73%	17.44	0.034	0.255	2370.13
1	10	110	113.58	60.59%	20.61	0.029	0.284	2340.68
1	10	120	171.81	74.30%	13.42	0.045	0.329	2305.14
1	10	130	142.37	69.45%	15.94	0.038	0.366	2269.20
1	10	140	183.78	76.72%	12.15	0.049	0.416	2232.80
1	10	150	149.85	71.85%	14.68	0.041	0.457	2200.28
1	10	160	180.81	77.02%	11.98	0.05	0.507	2165.58
1	10	170	212.06	80.72%	10.04	0.06	0.567	2129.07
1	10	180	170.23	76.37%	12.31	0.049	0.615	2095.59
1	10	190	172.63	77.08%	11.95	0.05	0.665	2063.75
1	10	200	207.21	81.24%	9.79	0.061	0.727	2028.89
1	10	210	219.64	82.64%	9.06	0.066	0.793	1990.13
1	10	220	220.00	82.95%	8.89	0.067	0.86	1955.70
1	10	230	164.12	77.46%	11.75	0.051	0.912	1927.86
1	10	240	168.28	78.34%	11.29	0.053	0.965	1899.85
1	10	250	293.31	87.80%	6.36	0.094	1.059	1865.80
1	10	260	172.30	79.64%	10.63	0.056	1.115	1831.30
1	10	270	295.71	88.37%	6.07	0.099	1.214	1794.71
1	10	280	245.64	86.27%	7.17	0.084	1.298	1761.83
1	10	290	300.36	89.01%	5.75	0.104	1.402	1725.63
1	10	300	300.43	89.22%	5.63	0.106	1.509	1692.72
1	10	310	297.85	89.33%	5.57	0.108	1.617	1659.98
1	10	320	335.87	90.72%	4.84	0.124	1.74	1626.65
1	10	330	313.74	90.27%	5.07	0.118	1.859	1592.23
1	10	340	351.25	91.52%	4.43	0.135	1.994	1555.44
1	10	350	371.69	92.19%	4.09	0.147	2.141	1518.71
1	10	360	415.28	93.19%	3.56	0.168	2.309	1480.22
1	10	370	313.29	91.21%	4.62	0.13	2.439	1447.03
1	10	380	448.91	94.03%	3.14	0.191	2.63	1410.64
1	10	390	456.36	94.26%	3.03	0.198	2.828	1381.55
1	10	400	359.37	92.91%	3.74	0.16	2.989	1345.00
1	10	410	537.95	93.88%	2.43	0.247	3.236	1307.08
1	10	420	433.40	90.85%	2.93	0.205	3.441	1269.08
1	10	430	399.20	90.30%	3.10	0.193	3.634	1239.16
1	10	440	585.09	93.59%	2.06	0.291	3.925	1205.01
1	10	450	342.75	89.41%	3.40	0.177	4.102	1165.12
1	10	460	128.88	72.86%	8.94	0.067	4.169	1151.80
1	10	470	294.39	90.13%	3.81	0.157	4.326	1122.48
1	10	480	311.53	92.05%	3.50	0.172	4.498	1089.28
1	10	490	280.99	92.18%	3.76	0.16	4.657	1056.98

1	10	500	341.77	94.09%	2.99	0.2	4.858	1022.87
1	10	510	352.34	94.58%	2.80	0.214	5.072	986.24
1	10	520	346.12	94.72%	2.74	0.219	5.291	946.72
1	10	530	355.09	95.03%	2.58	0.233	5.524	914.53
1	10	540	180.25	90.53%	4.92	0.122	5.646	887.13
1	10	550	334.93	95.06%	2.57	0.233	5.88	861.80
1	10	560	398.10	95.98%	2.09	0.286	6.166	833.86
1	10	570	371.91	95.88%	2.15	0.279	6.445	800.54
1	10	580	448.15	96.74%	1.71	0.352	6.797	764.18
1	10	590	325.87	96.51%	1.83	0.328	7.124	596.46
1	10	600	493.55	97.27%	1.43	0.418	7.543	708.16
1	10	610	354.51	96.41%	1.89	0.317	7.859	671.20
1	8	618	521.93	97.65%	1.24	0.388	8.247	646.38
1	10	628	332.90	96.94%	1.37	0.437	8.684	456.78
1	10	638	421.55	95.80%	1.38	0.433	9.118	583.50
1	10	648	260.03	93.29%	2.14	0.28	9.398	557.69
1	10	658	194.72	88.40%	3.93	0.153	9.55	765.58
1	10	668	6.36	17.92%	25.47	0.024	9.574	162.00
1	9	677	5.26	4.31%	30.78	0.018	9.591	162.00
1	10	687	134.64	79.88%	7.25	0.083	9.674	975.92
1	10	697	289.37	89.99%	3.31	0.181	9.855	956.94
1	10	707	297.00	90.83%	3.12	0.192	10.047	928.09
1	10	717	355.79	93.10%	2.51	0.239	10.287	891.51
1	10	727	253.22	90.99%	3.41	0.176	10.463	863.19
1	10	737	135.63	84.02%	6.22	0.096	10.559	844.23
1	1	738	28.99	28.10%	28.98	0.002	10.561	840.00
1	10	748	203.83	94.12%	2.06	0.291	10.852	420.56
1	10	758	336.54	95.83%	1.58	0.381	11.233	530.48
1	10	768	149.05	89.34%	3.43	0.175	11.408	511.44
1	10	778	299.11	92.19%	2.74	0.219	11.627	818.44
1	10	788	307.12	89.15%	4.23	0.142	11.769	1298.11
1	10	798	423.81	92.69%	2.97	0.202	11.971	1260.17
1	10	808	371.76	92.12%	3.29	0.182	12.153	1224.52
1	10	818	386.47	93.06%	3.07	0.196	12.349	1184.84
1	10	828	227.45	91.21%	4.16	0.144	12.493	945.45
1	10	838	22.69	24.84%	37.28	0.016	12.509	846.00
1	10	848	19.54	15.29%	43.30	0.014	12.523	846.00
1	10	858	6.80	11.25%	45.98	0.013	12.536	312.50
1	10	868	17.65	8.10%	47.92	0.013	12.548	846.00
1	10	878	16.97	4.83%	49.85	0.012	12.56	846.00
1	10	888	16.90	4.47%	50.07	0.012	12.572	846.00

1	10	898	16.39	1.88%	51.62	0.012	12.584	846.00
1	1	899	16.26	1.34%	52.04	0.001	12.585	846.00
1	10	909	16.17	1.11%	52.32	0.011	12.597	846.00
1	10	919	16.06	0.75%	52.68	0.011	12.608	846.00
1	10	929	15.99	0.55%	52.89	0.011	12.619	846.00
1	10	939	15.96	0.41%	53.00	0.011	12.631	846.00
1	10	949	15.95	0.34%	53.04	0.011	12.642	846.00
1	10	959	15.95	0.28%	53.05	0.011	12.653	846.00
1	10	969	15.96	0.26%	53.01	0.011	12.665	846.00
1	10	979	15.98	0.28%	52.93	0.011	12.676	846.00
1	10	989	16.01	0.31%	52.86	0.011	12.687	846.00
1	***10***	***999***	***16.02***	***0.29%***	***52.80***	***0.011***	***12.699***	***846.00***

APPENDIX C TABLES FROM NCHRP REPORT 812 AND HCM 2000

Table 63 Through - Car equivalents for permitted left turns

EXHIBIT C16-3. THROUGH-CAR EQUIVALENTS, E_{L1}, FOR PERMITTED LEFT TURNS

Type of Left-Turn Lane	Effective Opposing Flow, $v_{oe} = v_o/f_{LUo}$						
	1	200	400	600	800	1000	1200[a]
Shared	1.4	1.7	2.1	2.5	3.1	3.7	4.5
Exclusive	1.3	1.6	1.9	2.3	2.8	3.3	4.0

Table 64 Adjustment factors [44]

EXHIBIT 16-7. ADJUSTMENT FACTORS FOR SATURATION FLOW RATE[a]

Factor	Formula	Definition of Variables	Notes
Lane width	$f_w = 1 + \frac{(W - 3.6)}{9}$	W = lane width (m)	$W \geq 2.4$ If W > 4.8, a two-lane analysis may be considered
Heavy vehicles	$f_{HV} = \frac{100}{100 + \% HV(E_T - 1)}$	% HV = % heavy vehicles for lane group volume	E_T = 2.0 pc/HV
Grade	$f_g = 1 - \frac{\% G}{200}$	% G = % 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	$0 \leq N_m \leq 180$ $f_p \geq 0.050$ 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 = 1.000 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 Exhibit C16-1, Appendix C, for nonprotected phasing alternatives
Right turns	Exclusive lane: $f_{RT} = 0.85$ Shared lane: $f_{RT} = 1.0 - (0.15)P_{RT}$	P_{RT} = proportion of RTs in lane group	$f_{RT} \geq 0.050$

	Single lane: $f_{RT} = 1.0 - (0.135)P_{RT}$		
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 LT green P_{RT} = proportion of RTs in lane group P_{RTA} = proportion of RT protected green over total RT green	Refer to Appendix D for step-by-step procedure

Table 65 Typical values for minimum green to satisfy driver expectancy

Phase Type	Facility Type	Minimum Green (Seconds)
Through	Major Arterial (> 40 mph)	10 to 15
	Major Arterial (≤ 40 mph)	7 to 15
	Minor Arterial	4 to 10
	Collector, Local, or Driveway	2 to 10
Left Turn	Any	2 to 5