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

Performance Evaluation of Selected Intersections in Bahir Dar City

By
Nurhussien Hassen Belay

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By

Nurhussien Hassen Belay

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Advisor

Bikila Teklu (PHD)

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Approved by:

Name and signature of member of the examining board

Name	Signature	Date
1. ______________________ (Advisor)	______________ (Signature)	____________ (Date)
2. ______________________ (External Examiner)	______________ (Signature)	____________ (Date)
3. ______________________ (Internal Examiner)	______________ (Signature)	____________ (Date)
4. ______________________ (Chairman)	______________ (Signature)	____________ (Date)

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Abstract

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

In Bahir Dar city most of the intersections are closely spaced with high pedestrian, Cyclists, Bajaj and other motorized vehicular traffics so that their performance is highly affected by it. The objective of this study is to evaluate the performance of roadway intersections in Bahir dar city and to study the special effect of pedestrians, cyclists and bajaj in the performance of intersections. Traffic and geometric data's for the selected intersections have been collected and SIDRA (Signalized and Unsignalized Intersection Design and Research Aid) intersection software is used for analysis and evaluation of the performance of intersections. And the result of this research shows high magnitude of degree of saturation, delay and queue length and low magnitude of capacity for the selected intersections in the city. In general major intersections in the city have bad level of services (poor performances). And volume of pedestrians and Bajaj traffics are the main causes for this. In Bahir dar as well as in other big cities of Ethiopia's regional states, number of Bajaj's, Bicycles and Motor bicycles should be controlled and the number other public transports like buses and minibuses should be increased to improve the performance of major intersections in these cities. Besides, parking areas for vehicles and a separate way for pedestrian, like an over pass, to cross the road at the intersection areas should be prepared to increase the performance of the intersections in these cities.

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1 Introduction

1.1 Background

Traffic congestion on urban roads results in tremendous economic loss, additional delay and user cost. Intersections play an important role in the road network, where traffic flows in different directions converge. Because of disturbance of pedestrians, mixed traffic and lost green time for beginning and clearance etc., capacity or the maximum rate of flow at which persons or vehicles can reasonably expected to traverse an intersection is much lower than their approach links. Thus, intersections are usually the bottleneck of the network and are the greatest and immediate source of traffic accidents. Hence, level of service at intersection significantly affects the overall level of service of the road. The critical aspect of increasing capacity of any road lies in increasing capacity of the intersection. Traffic signals, roundabouts, stop and yield controls are commonly used in several at grade junction in urban areas to maximize traffic efficiency and safety by separating conflicting traffic movements in time [1]. And Intersection performance measures (capacity, degree of saturation, delay, queue length, fuel consumption, operating cost and emission etc.) used to evaluate the intersection performance [2] appropriately at these days based on different manuals such as USA HCM, Austria road and UK TRL etc. depending on the driver behavior and road way condition and by using different software packages like SIDRA, ARCADY, and RODEL. [3, 4].

Capacity which is the maximum sustainable flow rate that can be achieved during a specified time period under a given (prevailing) road, traffic and control conditions and degree of Saturation or the ratio of arrival (demand) flow rate to capacity during a given flow period are relevant measure of performance of intersections. Delay which is additional travel time experienced by a vehicle or pedestrian with reference to a base travel time is also one of the measure performances of intersections. Among others back of queue is a more useful performance measure of intersections because it is relevant to the design of appropriate queuing space, e.g. for short lane design to avoid queue spillback into adjacent lanes, for phasing design to avoid blockage of upstream signals in paired intersection situations, and for signal coordination offset design to prevent interruption of platoons by downstream queue. In addition to these, estimation of operating cost, fuel consumption and pollutant emissions for evaluating intersection also important for design, operations and planning purposes in traffic management as well as in improving the performance of intersections in urban areas[5].

Bahr Dar is a capital city of Amhara region and 3rd largest city in Ethiopia with traffic composition of pedestrian, bicycle, and motorized vehicles such as bajaj, taxi, bus, truck etc. The city has several straight roadways with a number of different intersection controls such as roundabouts, traffic signals, stop and yield control intersections. Like all others the performance of intersections in the city would be evaluated in terms of capacity, degree of saturation, average queue length, maximum queue length, delay, operating cost, fuel consumption and emission of pollutants.

1.2 Statement of the problem

Most of the researches related to performance evaluation of intersection in Ethiopia are focused only on highly congested junction areas in Addis Ababa. And there is no sufficient study on the performance of intersections in other cities of the country to handle the problem of congestion, delay, accident and emissions in the intersection areas before they become congested like in Addis Ababa. And studies on the performance of intersection for Bahir Dar city will be important to maximize traffic efficiency and safety in the intersection areas in the future for the city as well as for other cities which have similar traffic and topography conditions such as Hawassa, Mekelle, and Adama etc.

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

1.3 Objective

1.3.1 General objective

The general objective of this study is to evaluate the performance (capacity, degree of saturation, delay, queue length, and level of service) of roadway intersections in Bahir Dar city at different years.

1.3.2 Specific objective

Specific objectives of this study are:

- To evaluate the performance of selected major intersections in the city at study (base) year.
- To predict the performance of intersections for those which have good performance at study year by forecasting the traffic.
- To study the special effect of pedestrians, cyclists and Bajaj in the performance of intersections.
- Depending on the result of analysis to give recommendation in designing as well as improving the performance of intersections for the future in the city.

1.4 Methodology

The methodology followed to achieve the above objective started by intensive review of literatures associated with the title of the research and by correlating it with the real ground conditions. And then field observation have been made on the current road and intersection conditions of Bahir dar city to select intersections for the study. The third step followed to reach on the research objective is giving training for data collection and traffic counts of pedestrians, cyclists, and motorized vehicles and measurement of geometric elements of the selected intersections in the city have been conducted. Lastly SIDRA 5.1 developed by the Australian Road Research Board (ARRB) and Akcelik Associates with Microsoft excel is used for analysis and evaluation of intersection performance based on USA HCM and Austria road manuals.

1.5 Application of the result

The performance of intersections in Bahir Dar city is not studied until now and the result of this research will be important for designing and improving the performance of intersections in the future for the city. And also it will be important for design, operations and planning purposes in traffic management in the city as well as for other cities which have similar traffic and topographic conditions in Ethiopia in general.

2 Literature review

2.1 Intersection traffic control

Intersection is an area shared by two or more roads. This area is designated for the vehicles to turn to different directions to reach their desired destinations. Its main function is to guide vehicles to their respective directions. Traffic intersections are complex locations on any highway. This is because vehicles moving in different direction want to occupy same space at the same time. In addition, the pedestrians also seek same space for crossing. Both from accident and capacity perspective, the study of intersection are very important especially in the case of urban scenario [6]. At most intersection therefore traffic control measures are necessary to assign the right way [7].Types of intersection traffic control include: Basic rules of the road, give way control, two-way stop control, and all-way stop control, channelization islands, roundabouts, traffic signals, Grade separation [6].

2.1.1 Basic rules of the road

Where sufficient visibility is provided in low volume situations, some intersections operate effectively without formalized traffic control. In these cases, normal right of way rules apply [7].

2.1.2 Give way control, two-way stop control, and all-way stop control

The GIVE WAY control requires the driver in the minor road to slow down to a minimum speed and allow the vehicle on the major road to proceed. Two ways stop control requires the vehicle drivers on the minor streets should see that the conflicts are avoided. Finally an all-way stop control is usually used when it is difficult to differentiate between the major and minor roads in an intersection. In such a case, STOP sign is placed on all the approaches to the intersection and the driver on all the approaches are required to stop the vehicle. The vehicle at the right side will get priority over the left approach [6].

2.1.3 Channelized intersections

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. At skewed intersections, channelization islands are often used to delineate right turns, even in the absence of auxiliary right turn lanes. At intersections located on a curve, divisional islands can help direct drivers to and through the intersection. At large intersections, short median islands can be used effectively for pedestrian refuge **[7].**

2.1.4 Roundabouts

The roundabout is a channelized intersection with one-way traffic flow circulating around a central island. All traffic-through as well as turning enters this one-way flow. Although usually circular in shape, the central island of a roundabout can be oval or irregularly shaped. Roundabouts can be appropriate design alternative to both stop-controlled and signal-controlled intersections, as they have fewer conflict points than traditional intersections (eight versus 32, respectively) [7].Roundabout is Suitable for relatively balanced approach volumes, safer for vehicular travel, can result in less delay and emissions, can accommodate aesthetic treatments, lower injury and fatality rates, less suitable for high volume/multilane approaches, less intuitive for pedestrians/bicycle lists than other intersection type [8].

2.1.5 Traffic signals

Control using traffic signal is based on time sharing approach. At a given time, with the help of appropriate signals, certain traffic movements are restricted where as certain other movements are permitted to pass through the intersection. Two or more phases may be provided depending upon the traffic conditions of the intersection. The signals can operate in several modes. Most common are fixed time signals and vehicle actuated signals [6].Traffic signals are suitable for high volume intersections, allows protected pedestrian movements, accommodates unbalanced approach volumes, lower construction cost, can have high amounts of stopped time and delay (congestion), higher potential for rare end collision accidents, multiple lanes for pedestrians to cross [8].

2.1.6 Grade separated intersections

Intersections are generally classified as at-grade intersections and grade-separated intersections. In at-grade intersections, all roadways join or cross at the same vertical level. Grade separated intersections allows the traffic to cross at different vertical levels. Sometimes the topography itself may be helpful in constructing such intersections. Otherwise, the initial construction cost required will be very high. Therefore, they are usually constructed on high speed facilities like expressways, freeways etc. This type of intersection increases the road capacity because vehicles can flow with high speed and accident potential is also reduced due to vertical separation of traffic. Different types of grade-separators for this type of intersection are flyovers and interchange. Flyovers itself are subdivided into overpass and underpass. When two roads cross at a point, if the road having major traffic is elevated to a higher grade for further movement of traffic, then such structures are called overpass. Otherwise, if the major road is depressed to a lower level to cross another by means of an under bridge or tunnel, it is called under-pass. Interchange is a system where traffic between two or more roadways flows at different levels in the grade separated junctions. Common types of interchange include trumpet interchange, diamond interchange, and cloverleaf interchange [6].

2.1.6.1 Trumpet interchange

Trumpet interchange is a popular form of three leg interchange. If one of the legs of the interchange meets a highway at some angle but does not cross it, then the interchange is called trumpet interchange [6].

2.1.6.2 Diamond interchange

Diamond interchange is a popular form of four-leg interchange found in the urban locations where major and minor roads crosses. The important feature of this interchange is that it can be designed even if the major road is relatively narrow [6].

2.1.6.3 Clover leaf interchange

It is also a four leg interchange and is used when two highways of high volume and speed intersect each other with considerable turning movements. The main advantage of cloverleaf intersection is that it provides complete separation of traffic. In addition, high speed at intersections can be achieved. However, the disadvantage is that large area of land is required [6].

2.2 Intersection performance measures

It is said that “If you cannot tell how your system performed yesterday, you cannot expect to manage it today”. Transportation system performance measures constitute an invaluable source of information for decisions related to infrastructure resource allocation, investment plan monitoring and project evaluation. The advent of Intelligent Transportation Systems has further increased the significance of obtaining timely and accurate transportation performance measures, which can be used for either optimizing traffic management strategies or informing travelers with respect to their optimal travel paths. The challenge for transportation system has been diverted from developing basic infrastructure to managing and operating the existing transportation resources and delivering better services to road travelers under various conditions. Performance measurement becomes the critical tool to meet such challenges. Intersection performance measures are used to determine or evaluate the service quality provided by the existing road facilities and the control plans at an intersection. Intersection measures usually can be calculated or estimated based on traffic data collected from the subject intersection and its surrounding intersections [9]. Capacity, degree of saturation, delay, queue length, and level of service are the main performance measures of intersections.

2.2.1 Capacity and degree of saturation

Capacity is the maximum sustainable flow rate that can be achieved during a specified time period under given (prevailing) road, traffic and control conditions [2]. The proviso "prevailing conditions" is important since capacity is not a constant value, but varies as a function of traffic flow levels. Capacity represents the service rate (queue clearance rate) in the performance (delay, queue length, stop rate) functions, and therefore is relevant to both under saturated and over saturated conditions. Conceptually, this is different from the maximum volume that the intersection can handle which is the practical capacity (based on the a target degree of saturation) under increased demand volumes, not the capacity under prevailing condition [10]. The basic equation for the calculation of capacity, which is applicable to both signalized and unsignalized intersections, is:

$$Q = su \qquad (1)$$

Where Q = capacity (veh/h), u = proportion of time when the vehicles can depart from the queue (signals are green or gaps are available in the opposing stream) and s = saturation (queue discharge) flow rate (veh/h).

For signalized intersections, u is the green time ratio, u = g / c, where g = effective green time (s) and c = cycle time (s).For gap-acceptance processes at roundabouts and sign-controlled intersections, u is the unblocked time ratio related to average durations of block and unblock periods in the opposing stream [11].

Saturation flow rate is the maximum flow rate that can be sustained when there is a queue and the vehicles can depart from the queue, i.e. signals are not red or the gaps in the opposing stream are not too short. Saturation flow rate corresponds to a queue discharge headway which represents the minimum headway between vehicles that is achieved while they are departing from the queue:

$$h_s = 3600/s \qquad (2)$$

h_s = queue discharge (saturation) headway (seconds) and s = saturation flow rate (veh/h). For example, a saturation flow rate of s = 1800 veh/h corresponds to a saturation headway of = 2.0 seconds. The gap-acceptance method uses the follow-up headway (t_f) as the queue discharge (saturation) headway ($t_f = h_s$). The follow-up headway corresponds to a saturation flow rate which is the maximum gap-acceptance capacity that can be achieved when the opposing flow is close to zero [11].

$$s = 3600/t_f \qquad (3)$$

Where s=saturation flow rate veh/h and t_f =follow-up headway as a queue discharge (saturation) headway (seconds)

2.2.1.1 Capacity modeling issues

Important issues in modeling capacity, relevant to all types of intersection, are the level of aggregation in terms of individual lanes, lane groups and approaches. Different methods have been used to measure and model capacities in terms of level of aggregation:

1) Lane-by-lane analysis as in the SIDRA INTERSECTION software

2) Analysis by lane groups (movements combined according to shared lanes) as in the Highway Capacity Manual, and

3) Analysis by total approach flows, i.e. all movements in all approach lanes combined, as in the TRL (UK) method for capacity analysis

The lane-by-lane method simplifies the analysis method and introduces improved accuracy levels in capacity and performance prediction by allowing improved spatial (geometric) modeling of all types of intersection. This method allows modeling of unequal lane utilization which is an important factor that affects the capacity and performance of intersection [5, 10, and 11].

SIDRA standard model, HCM 2010 model and UK TRL model are the three well known analytical models of capacity. Both SIDRA standard model and HCM 2010 model are implemented in SIDRA INTERSECTION software and there capacity estimation basis on gap-acceptance theory. And UK TRL (linear regression) model implemented on ARCADY and RODEL software [5].

2.2.1.2 Gap acceptance capacity model

Gap-acceptance capacity models apply to the analysis of minor movements at two-way stop and give-way (yield) sign-controlled intersections, entry streams at roundabouts and opposed (permitted) turns at signalized intersections. The same modeling principles apply to all these cases with different model parameters representing the intersection geometry, control and driver behavior at different traffic facilities [12].

The saturation flow rate for a gap-acceptance process is the maximum capacity that can be achieved when the opposing flow is close to zero. The capacity is reduced from this value with increased opposing flow rates, due to the decreased value of unblocked time ratio as seen from Figure 2-1.While the follow-up headway determines the capacity value at low opposing flow rates directly, the critical gap parameter affects the u parameter (the proportion of time when the vehicles can depart from the queue) in Equation (1) with lower values of u resulting from larger values of critical gap (hence lower capacity) for a given opposing flow rate [11]. This is also depicted in Figure 2-1.

The estimation of arrival headways is fundamental to the modeling of gap acceptance processes for estimating capacities of sign-controlled traffic streams, roundabout entry streams and filter (permitted) turns at signalized intersections .Some of arrival headways distribution are exponential arrival headway distribution models known as negative exponential (M1), shifted negative exponential (M2) and bunched exponential (M3). The bunched exponential distribution of arrival headways (M3) .The negative and shifted negative exponential distributions (M1 and M2) are extensively discussed and used in the literature as models of random arrivals. The shifted negative exponential model (M2) is normally used for single-lane traffic only. The bunched exponential distribution (M3) offers improved accuracy in the prediction of small arrival headways (up to about 12 seconds), which is important for most urban traffic analysis applications.M1 and M2 model assumes random arrivals with no bunching in contrast with the bunched headways model used by the M3 model. The assumption of no bunching cannot be supported especially at high opposing flow rates where vehicles are highly bunched. Another aspect of the M1 model is that it is not sensitive to the number of opposing movement lanes, which is a shortcoming since the same opposing flow rate in more lanes means better gap-acceptance opportunities [12].

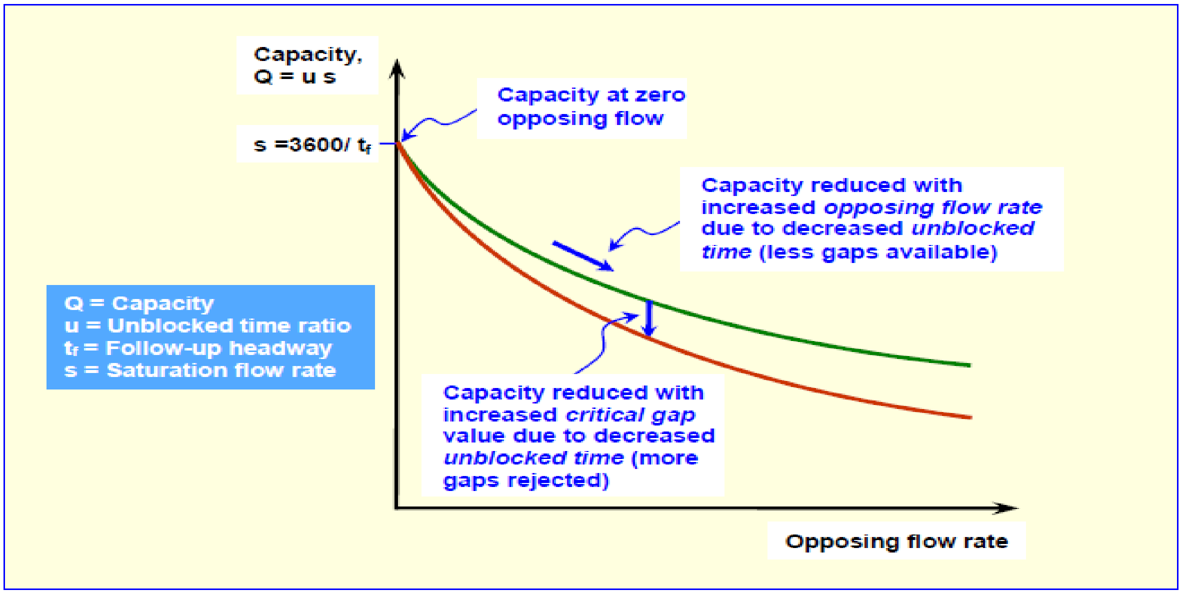

Figure 2.1 Gap-acceptances capacity [11]

Bunching model

An important parameter in a bunched exponential model for estimating headway distribution of vehicles in a traffic stream is the proportion of unbunched (or bunched) vehicles. This parameter is used in gap-acceptance as well as performance models (delay, queue length.). For the bunched exponential model, the following model is used for estimating the proportion free (unbunched) vehicles in a traffic stream:

$\varphi = (1 - \Delta q) / [1 - (1 - k_d) \Delta q]$ ……………………subject to $1.0 \geq \varphi \geq 0.10$………..………(4)

Where:

φ = proportion unbunched (free),

Δ = minimum intrabunch headway in the traffic stream (seconds),

q = arrival flow rate (veh/s or pcu/s), and

k_d = bunching delay parameter (a constant).

The minimum value of $\varphi_{min} = 0.10$ is used for computational reasons.

The model given by Equation (4) was developed by considering a fundamental relationship between travel delay parameter in Akçelik's speed-flow function and the bunching delay obtained through the bunching model to determine vehicle headway distributions. Parameters k_d and Δ in the two models are given in Table 2-1. Parameters for uninterrupted traffic streams are used (i) for major (opposing) traffic streams at traffic signals and two-way sign-controlled intersections in gap-acceptance analysis, and (ii) for entry streams at two-way sign-controlled intersections and roundabouts in performance analysis (parameter values for 1-lane case are used in this case [5,12].

Table 2-1 Parameter values for estimating the proportion of free (unbunched) vehicles [5, 12]

Total number of lanes	Uninterrupted traffic streams				Roundabout circulating streams			
	Δ	3600/Δ	b	k_d	Δ	3600/Δ	b	k_d
1	1.8	2000	0.5	0.20	2.0	1800	2.5	2.2
2	0.9	4000	0.3	0.20	1.0	3600	2.5	2.2
> 2	0.6	6000	0.7	0.30	0.8	4500	2.5	2.2

All capacity and performance calculations are carried out for individual lanes of entry (minor) movements, but traffic in all lanes of the major (conflicting) movement is treated together as one stream. When there are several conflicting (higher priority) streams at sign-controlled and signalized intersections, all conflicting streams are combined as one stream. The resulting total opposing flow rate (q_m) may be expressed in passenger car units (pcu/s) allowing for the effect of heavy vehicles in the opposing stream(s) [12].

Gap-acceptance capacity model for unsignalized intersections was derived using a traffic signal analogy concept. The signal analogy concept treats block and unblock periods in a priority (major) traffic stream (as defined in the traditional gap acceptance modeling) as red and green periods in a way similar to the modeling of signal-controlled traffic streams. And from this analogy unblocked time (u) ratio is given by:

$$u = g/c = (1-\Delta_m q_m + 0.5\varphi_m q_m t_f)\, e^{(-\lambda\,(tc-\Delta m))} \qquad (5)$$

Where: λ=model parameter and calculated by:

$\lambda = \varphi_m q_m / (1-\Delta_m q_m)$ …………………………. subjected to $q_m \leq 0.98/\Delta_m$……..………..….… (6)

Entry stream saturation flow rate, s (veh/h): $s=3600/t_f$

Gap-acceptance capacity (Q_g) (veh/h):

$Q_g = su = (3600/t_f)\, u = (3600/t_f)\,(1-\Delta_m q_m + 0.5\varphi_m q_m t_f)\, e^{(-\lambda(t_c-\Delta_m))}$…………..……….……………(7)

Entry stream capacity (veh/h):

$Q = \max(Q_g, Q_m)$………….……...………...….………………………………………….. (8)

Where: Q_m is the minimum capacity (veh/h) given by:

$Q_m = \min(q_e, 60n_m)$……..……..………...…………………………………………… (9)

Where: q_e is the entry stream flow rate (veh/h), and n_m is the minimum number of entry stream vehicles that can depart under heavy major stream flow conditions (veh / min).When there are several opposing (higher priority) streams, the total major stream flow (q_m) is calculated as the sum of all conflicting stream flows and parameters Δ_m and φ_m are determined accordingly. And this gap acceptance capacity model is referred to as the Akçelik M3D model and it is implemented in SIDRA standard model. HCM 2010 model uses M_1 arrival headway distribution model in its gap-acceptance capacity estimation [12].

In all gap-acceptance cases, SIDRA INTERSECTION determines the effective number of lanes considering all opposing movements before selecting appropriate parameters from Table 2-1.For each opposed movement at a sign-controlled intersection, all opposing movements are combined together as one multi-lane stream and the intra-bunch headway and proportion bunched are determined accordingly [5].

2.2.2 Delay

Delay is the additional travel time experienced by a vehicle or pedestrian with reference to a base travel time (e.g. free-flow travel time).The delay to a vehicle which decelerates from the approach cruise speed to a full stop (due to a reason such as a red signal, a queue ahead, or lack of an acceptable gap), waits and then accelerates to the exit cruise speed is considered to include the delay due to a deceleration from the approach cruise speed down to an approach negotiation speed and then to zero speed, idling time, acceleration to an exit negotiation speed along the negotiation distance, travelling the rest of the negotiation distance (if any) at the constant exit negotiation speed, and then acceleration to the exit cruise speed. This delay is the intersection control delay (overall delay with geometric delay) [5] and it is shown in the Figure 2-2.In the figure definition of control delay, geometric delay, stop-line delay, and stopped delay experienced by a turning vehicle at an intersection is shown.

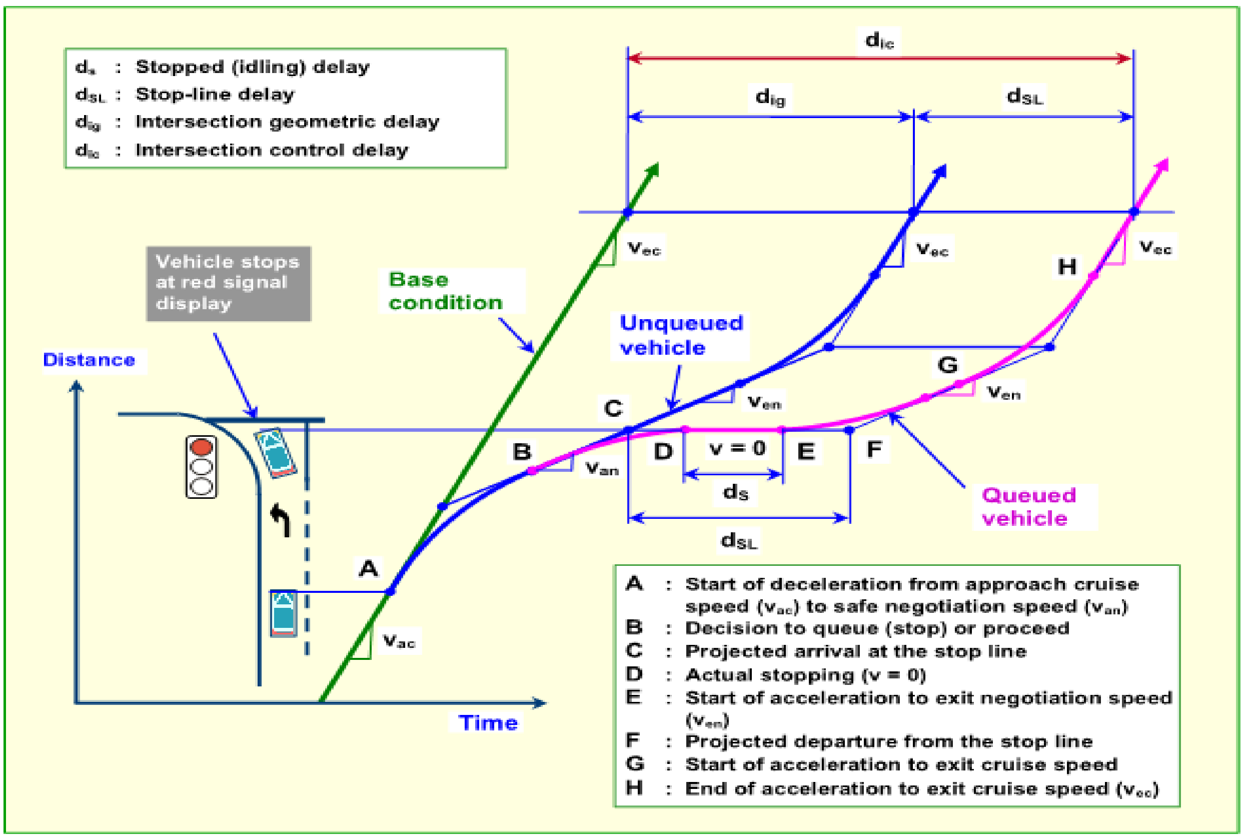

Figure 2.2 Control delay, geometric delay, stop-line delay, and stopped delay [5]

2.2.2.1 Delay at signalized intersection

2.2.2.1.1 Types of delay

In analytic models for predicting delay, there are three distinct components of delay. These are uniform, random, overflow delays.

1. Uniform delay

Uniform delay is the delay based on an assumption of uniform arrivals and stable flow with no individual cycle failures. Figure 2-3, shows stable flow throughout the period depicted. No signal cycle fails here, i.e., no vehicles are forced to wait for more than one green phase to be discharged. During every green phase, the departure function catches up with the arrival function. Total aggregate delay during this period is the total of all the triangular areas between the arrival and departure curves. This type of delay is known as uniform delay [13].

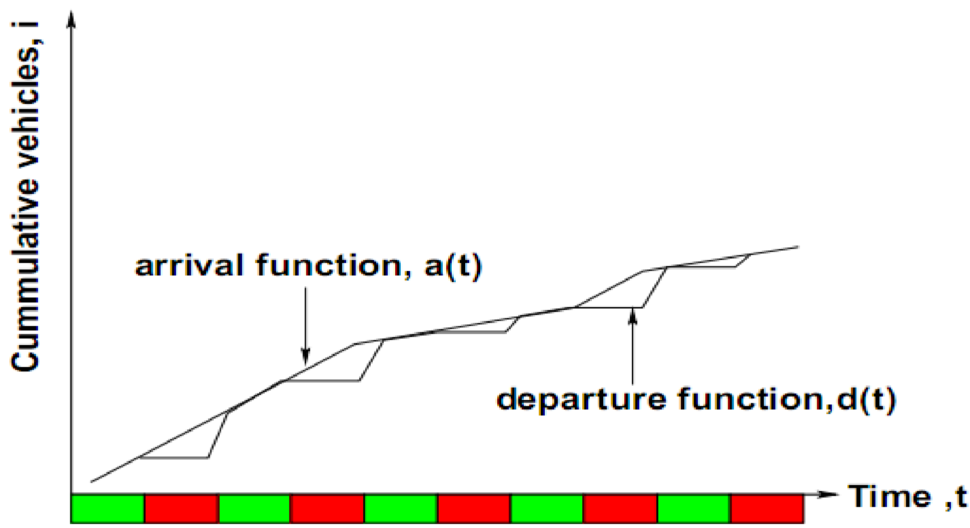

Figure 2.3 Uniform delay [13]

Illustration of one cycle uniform delay

All analytic models of delay begin with a plot of cumulative vehicles arriving and departing vs. time at a given signal location. Figure 2-4 shows a plot of total vehicle verses time. Two curves are shown: (a) a plot of arriving vehicles and (b) a plot of departing vehicles. The time axis is divided into periods of effective green and effective red [13].

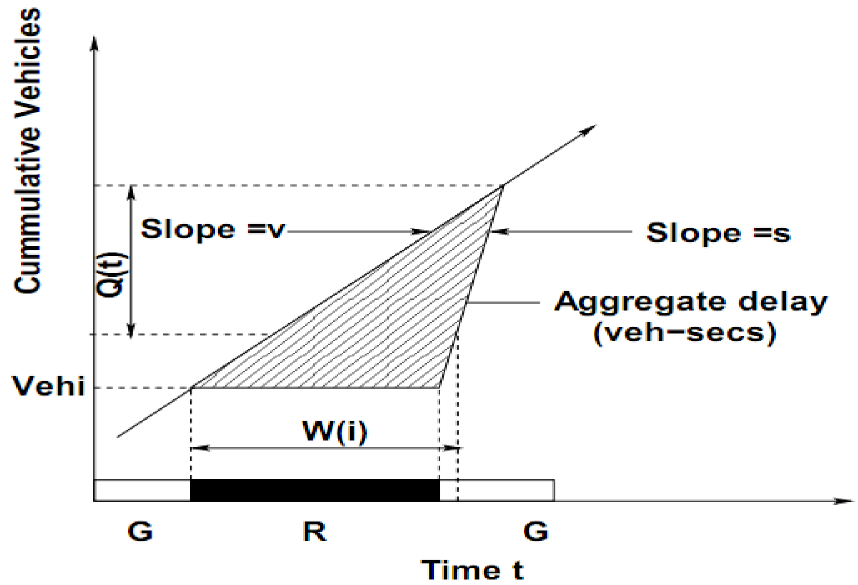

Figure 2.4 Illustration of delay, waiting time and queue length [13]

Vehicles are assumed to arrive at a uniform rate of flow of v vehicles per unit time. This is shown by the constant slope of the arrival curve. Uniform arrivals assume that the inter-vehicle arrival time between vehicles is a constant. Assuming no preexisting queue, arriving vehicles depart instantaneously when the signal is green (i.e., the departure curve is same as the arrival curve). When the red phase begins, vehicles begin to queue, as none are being discharged. Thus, the departure curve is parallel to the x-axis during the red interval. When the next effective green begins, vehicles queued during the red interval depart from the intersection, at a rate called saturation flow rate, s, in veh/s. For stable operations, depicted here, the departure curve catches up with the arrival curve before the next red interval begins (i.e., there is no residual queue left at the end of the effective green) [13]. In this simple model:

- The total time that any vehicle i spends waiting in the queue, W(i), is given by the horizontal time-scale difference between the time of arrival and the time of departure

- The total number of vehicles queued at any time t, Q(t), is the vertical vehicle-scale difference between the number of vehicles that have arrived and the number of vehicles that have departed

- The aggregate delay for all vehicles passing through the signal is the area between the arrival and departure curves (vehicles X time)

2. Random delay

Random delay is the additional delay, above and beyond uniform delay, because flow is randomly distributed rather than uniform at isolated intersections. In Figure 2-5 some of the signal phases fail. At the end of the second and third green intervals, some vehicles are not served (i.e., they must wait for a second green interval to depart the intersection). By the time the entire period ends, however, the departure function has caught up with the arrival function and there is no residual queue left unserved. This case represents a situation in which the overall period of analysis is stable (i.e., total demand does not exceed total capacity) [13].

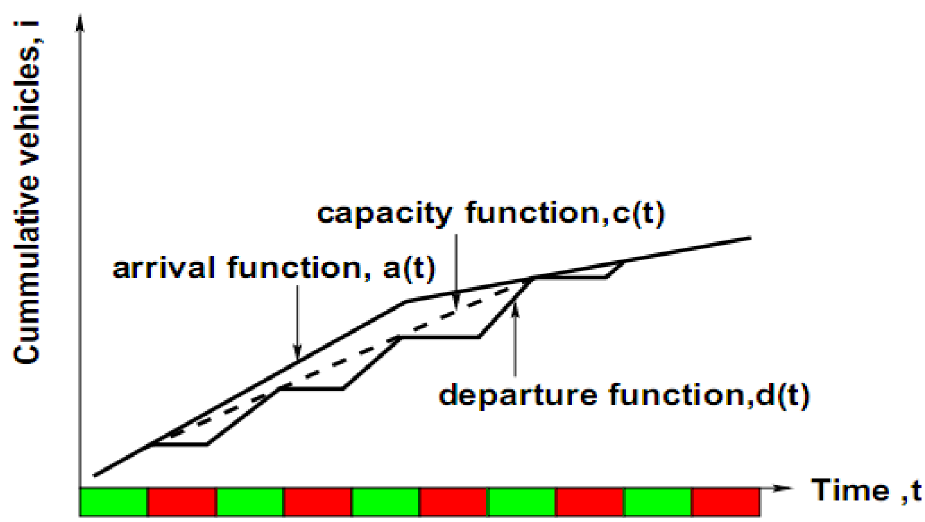

Figure 2.5 Random delay [13]

3. Overflow delay

Overflow delay is the additional delay that occurs when the capacity of an individual phase or series of phases is less than the demand or arrival flow rate. Figure 2-6 shows the worst possible case, every green interval fails for a significant period of time, and the residual, or unserved, queue of vehicles continues to grow throughout the analysis period [13].

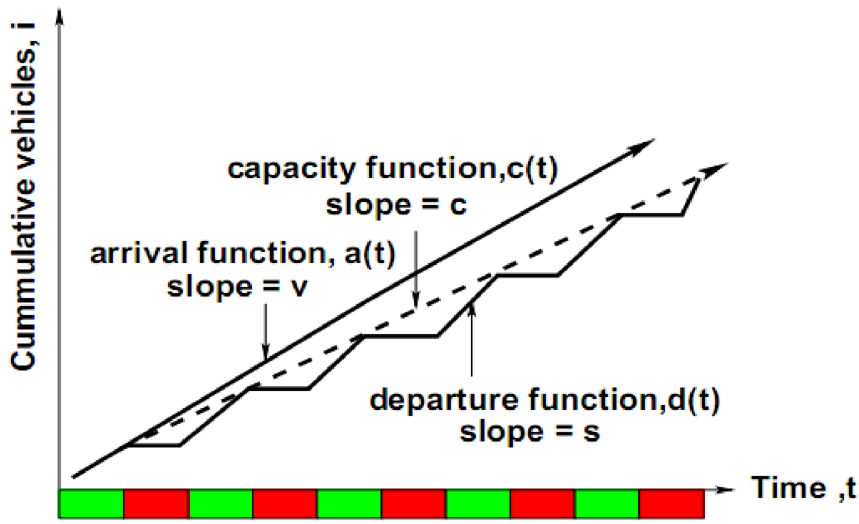

Figure 2.6 Overflow delay [13]

1) Webster's delay model

a) Uniform delay model

UD=C/2[(1 – g/C) ^{2}/ (1 – (g/C) X)]…………………………………………………………… (10)

Where:

C = cycle length, s

X=v/c ratio

g=effective green time, sec

b) Random delay model

RD = X^2/ (2v (1 − X))…………………………………………………………………………… (11)

The formula was found to somewhat overestimate delay. Webster proposed that total delay is the sum of uniform delay and random delay, and it can be estimated as,

D = 0.90(UD + RD)…………………………………………………………………....... (12)

c) Overflow delay model

$OD = (T/2)/(X - 1)$…………………………………………………………………………… (13)

Where,

T= analysis period, h

Inconsistencies between Random and Overflow delay occurs when the X is in the vicinity of 1.0. Most studies show that uniform delay model holds well in the range $X< 0.85$. In this range true value of random delay is minimum and there is no overflow delay. Also overflow delay model holds well in the range $X>1.15$. The inconsistency occurs in the range $0.85 <X <1.15$; here both the models are not accurate. Much of the more recent work in delay modeling involves attempts to bridge this gap, creating a model that closely follows the uniform delay model at low X ratios, and approaches the theoretical overflow delay model at high X ratios, producing ”reasonable” delay estimates in between. The most commonly used model for bridging this gap was developed by Akcelik for the Australian Road Research Board's signalized intersection [13].

2) Akcelik delay Model

To address the above said problem Akcelik proposed a delay model. In his delay model, overflow component is given by,

$OD = (cT/4)\ [(X - 1) + ((X - 1)^2 + 12(X - X0)/cT)^{1/2})]$……………………………………………… (14)

For $x > X_0$, $OD = 0$……………………………………………………………………………….. (15)

For $X \leq X_0$, $X_0 = 0.67 + sg/600$………………………………………………………………. (16)

Where,

T= analysis period, h

X=v/c ratio

c= capacity, veh/hour

s= saturation flow rate, veh/sg (vehicles per second of green) g=effective green time, sec

3) HCM 2000 delay model

The delay model incorporated into the HCM 2000 includes the uniform delay model, a version of Akcelik's overflow delay model, and a term covering delay from an existing or residual queue at the beginning of the analysis period. The delay is given as,

$d = d_1PF + d_2 + d_3$………………………………………………………………….… (17)

$d_1 = c/2[(1 – g/c)^2/ (1 – (\min (1, X) (g/c))]$………………………………………...… (19)

$d_2 = 900T [(X − 1) + ((X − 1)^2 + 8klX/cT)^{1/2}]$…………………………………….......... (20)

Where, d = control delay, s/veh

d_1 = uniform delay component, s/veh

PF = progression adjustment factor

d_2 = overflow delay component, s/veh

d_3 = delay due to pre-existing queue, s/veh

T = analysis period, h

X = v/c ratio

C = cycle length, s

k = incremental delay factor for actuated controller settings; 0.50 for all pre-timed controllers

l = upstream filtering/metering adjustment factor; 1.0 for all individual intersection analyses

2.2.2.2 Delay models for unsignalized intersections

Akgelik - Troutbeck, SIDRA and HCM 2000 delay model are some of the models for estimating delay to sign-controlled (two-way stop or give-way) and roundabout intersections. Akgelik - Troutbeck model (1991) biases on a delay model originally proposed by Troutbeck (1986, 1989) and is derived by an extension of the simple queuing theory method using a minimum delay parameter based on gap-acceptance modeling; and the delay model used in the SIDRA 5 package which is based on gap-acceptance, queuing theory and overflow queue [14]. HCM 2000 delay model equation is shown below.

1) HCM 2000 delay model

For roundabouts delays can be predicted in a manner similar to that used for stop-controlled and signal-controlled intersections. Equation 21 shows the model that should be used to estimate control delay for each lane of an approach of a roundabout.

$$d=3600/c +900\, T[X-1+ ((X-1)^2+ ((3600/c)\, X)/450T)^{1/2}] \quad (21)$$

Where:

d = control delay, sec/veh;

c = capacity of subject lane, veh/h; and

T = time period, h (T=1 for 1-hr analysis, T=0.25 for 15-min analysis).

Equation 21 is the same as that for stop-controlled intersections except that it does not include the “+ 5s” term. This modification is necessary to account for the yield control on the subject entry, which does not require drivers to come to a complete stop if there is no conflicting traffic.

The HCM delay models (TRB 2000, 2010a) do not include geometric delays although HCM qualifies delay equations as estimating control delay. SIDRA INTERSECTION standard delay models include geometric delay as an explicit additional term. SIDRA INTERSECTION output includes geometric delays estimated using the SIDRA standard methods for all types of intersection even when the HCM Delay Formula option is selected. These geometric delays are not added to delay values obtained using the HCM delay equations when the HCM Delay Formula option is used [5].

2.2.3 Queue length

Two types of queue length are used by different guides and software packages, namely the back of queue and the cycle-average queue. A percentile queue length is a value below which the specified percentage of the average queue length values observed for individual cycles fall. For example, the 95th percentile queue length is the value below which 95 per cent of all observed cycle queue lengths fall, or 5 per cent of all observed queue lengths exceed. With a cycle time of 120 s, a 30-min peak (analysis) period would have 15 cycles. This would mean that the queue length would be larger than the 95th percentile value in 15 x 0.05 = 0.75 cycles during the analysis period (i.e. exceeded approximately once). The 70th, 85th, 90th, 98th and 98th percentile queue lengths are useful for the design of queue spaces (turn bays and parking bans).The choice of a percentile queue should be made by the user according to the local design practices. Use of the 95th percentile value of the back of queue is generally considered to be a good choice for design purposes [5, 15].

The cycle-average queue is the average value of the number of vehicles in the queue during each cycle. This would be based on a queue count recorded frequently, e.g. every 5 seconds. The cycle-average queue length incorporates all queue states including zero queues observed towards the end of the cycle. The back of queue is the maximum extent of the queue that occurs once each cycle, usually during green time or unblocks time. Zero queue states are not relevant to the back of queue. The back of queue is a more useful performance measure since it is relevant to the design of appropriate queuing space, e.g. for short lane design to avoid queue spillback into adjacent lanes, for phasing design to avoid blockage of upstream signals in paired intersection situations, and for signal coordination offset design to prevent interruption of platoons by downstream queue [5, 15].

Delay and back of queue are not always consistent in terms of magnitude. Low average delay can be associated with a long back of queue as a result of a high arrival flow rate, large green time ratio (relatively short red period) and low degree of saturation. In this case, a large proportion of vehicles may be undelayed, and therefore the cycle-average queue could be small. In contrast, the case of short back of queue may be associated with a large average delay as a result of a low arrival flow rate, small green time ratio (relatively long red period) and a high degree of saturation [16].

The traditional gap-acceptance and queuing theory models do not give sufficient information for the design of roundabouts and sign-controlled intersections since they predict cycle-average queue lengths rather than the back of queue. The models used in SIDRA INTERSECTION fill the gap in modeling queue length for unsignalized intersections. A gap-acceptance cycle consists of a block period and an unblock period, i.e. vehicles waiting due to lack of an acceptable gap, then departing when an acceptable gap occurs, similar to a signal cycle that consists of a red period and a green period and changing the Gap-Acceptance Capacity model will affect some parameters in the back of queue model [5].

2.2.4 Level of service

Level-of-service is one measure of user satisfaction with an intersection. For all users, level-of-service is linked to average delay [7].

2.2.4.1 Vehicles level of service

There are different methods for level of service definitions to vehicles such as delay (HCM 2000), Delay & v/c (HCM 2010), and delay & degree of Saturation method.

1) Delay (HCM 2000) method

This method is based on HCM 2000 (TRB 2000) and uses delay only for LOS determination for vehicles. The LOS thresholds are given in Table 2-2. The method is used as the default for the Standard Left, Standard Right and New Zealand models in SIDRA INTERSECTION. For these models, the Roundabout LOS Method is "Same as Signalized Intersections" (applying the delay thresholds given under "Signals" in Table 2-2). Thus, the roundabout LOS method is unchanged for these models in Version 5.1 compared with previous versions [5].

Table 2-2 Delay (HCM 2000) method for LOS definitions (for vehicles) [5]

Level of Service	Control delay per vehicle in seconds (d)		
	Signals (*SIDRA standard default for roundabouts*)	"SIDRA Roundabout LOS" option	Sign Control
A	d ≤ 10	d ≤ 10	d ≤ 10
B	10 < d ≤ 20	10 < d ≤ 20	10 < d ≤ 15
C	20 < d ≤ 35	20 < d ≤ 35	15 < d ≤ 25
D	35 < d ≤ **55**	35 < d ≤ **50**	25 < d ≤ **35**
E	**55** < d ≤ 80	**50** < d ≤ 70	**35** < d ≤ 50
F	80 < d	70 < d	50 < d

2) Delay & v/c (HCM 2010) method

This is the LOS method for vehicles introduced in HCM 2010 (TRB 2010a). It offers an important variation on the Delay (HCM 2000) method in using both the average control delay and the v/c (demand volume / capacity) ratio, or degree of saturation for LOS determination. It uses delay thresholds which are the same as in the Delay (HCM 2000) method, but assigns LOS F when v/c > 1.0 (oversaturated conditions) irrespective of delay, as seen in Table 2-3. This method replaces the Delay (HCM) & Degree of Saturation method which was available in earlier versions of SIDRA INTERSECTION. The earlier method used the LOS criteria and thresholds for vehicles given in Table 2-4. It was based on a proposal by Berry (1987). The Delay & v/c (HCM 2010) method could be considered to be a simplified version of the earlier method used in SIDRA INTERSECTION. Both methods give LOS F for oversaturated conditions (v/c > 1.0) but the Delay (HCM) & Degree of Saturation had more subtle conditions that could lead to LOS D or E for degrees of saturation (v/c ratios) close to 1.0 [5].

Table 2-3 Delay & v/c (HCM 2010) method for LOS definitions (for vehicles) [5]

Level of Service for v/c ≤ 1.0	Average delay per vehicle in seconds (d)			Level of Service for v/c > 1.0
	Signals	"SIDRA Roundabout LOS" option	Sign Control (HCM 2010 default for roundabouts)	All Intersection Types
A	d ≤ 10	d ≤ 10	d ≤ 10	F
B	10 < d ≤ 20	10 < d ≤ 20	10 < d ≤ 15	F
C	20 < d ≤ 35	20 < d ≤ 35	15 < d ≤ 25	F
D	35 < d ≤ **55**	35 < d ≤ **50**	25 < d ≤ **35**	F
E	**55** < d ≤ 80	**50** < d ≤ 70	**35** < d ≤ 50	F
F	80 < d	70 < d	50 < d	F

Table 2-4 Delay & Degree of Saturation method for LOS definitions (for vehicles) [5]

Level of Service	Control delay per vehicle in seconds (d)			Degree of saturation (v/c ratio) (x)
	Signals	"SIDRA Roundabout LOS" option	Sign Control	
A	d ≤ 10	d ≤ 10	d ≤ 10	0 < x ≤ 0.85
B	10 < d ≤ 20	10 < d ≤ 20	10 < d ≤ 15	0 < x ≤ 0.85
C	20 < d ≤ 35	20 < d ≤ 35	15 < d ≤ 25	0 < x ≤ 0.85
D	35 < d ≤ 55	30 < d ≤ 50	25 < d ≤ 35	0 < x ≤ 0.85
	0 < d ≤ **55**	0 < d ≤ **50**	0 < d ≤ **35**	0.85 < x ≤ **0.95**
E	**55** < d ≤ 80	**50** < d ≤ 70	**35** < d ≤ 50	0 < x ≤ 0.95
	0 < d ≤ 80	0 < d ≤ 70	0 < d ≤ 50	0.95 < x ≤ 1.00
F	80 < d	70 < d	50 < d	**1.00 < x**

2.2.4.2 Pedestrian level of service

The criteria used for pedestrian level of service as well as the guide for the risk-taking behavior of pedestrians given in Table 2-5 are based on HCM 2000 (TRB 2000) Chapter 18, Exhibit 18-9 for signalized intersections and Exhibit 18-13 for unsignalized intersections in HCM 2000 Chapter 18 (Pedestrians).The LOS criteria and thresholds given in Table 2-5 apply for all SIDRA INTERSECTION models irrespective of the LOS Method selected for vehicles [5].

Table 2-5 Level of Service method for pedestrians (based on delay only) [5]

Level of Service	Average delay per pedestrian in seconds (d)		Likelihood of risk-taking behaviour
	Signals	Unsignalised Intersections	
A	$d \leq 10$	$d \leq 5$	Low
B	$10 < d \leq 20$	$5 < d \leq 10$	-
C	$20 < d \leq 30$	$10 < d \leq 20$	Moderate
D	$30 < d \leq 40$	$20 < d \leq 30$	-
E	$40 < d \leq 60$	$30 < d \leq 45$	High
F	$60 < d$	$45 < d$	Very High

2.2.5 Pedestrian saturation flow rate and performance measures

The pedestrian saturation flow rate represents a number of pedestrians in each row of the queue (with increasing pedestrian volumes, pedestrians tend to queue side by side in increasing numbers, thus increasing the saturation flow rate). The pedestrian saturation flow rate (s_p in ped/h) can be estimated from:

$$s_p = 3600\ n_p / h_{sp1} \qquad (22)$$

Where n_p is the number of pedestrians in a row in the queue, and h_{sp1} is the queue discharge headway between two pedestrian rows (seconds):

$$h_{sp1} = 0.8 + L_{hjp} / 1.5 \qquad (23)$$

Where L_{hjp} is the pedestrian queue spacing (default value is 1.0 m, or 3 ft for the HCM version with US Customary Units). For pedestrian queue length estimation, SIDRA INTERSECTION estimates the number of pedestrians in a row ("number of pedestrian lanes") from:

$$n_p = s_p h_{sp1} / 3600 \qquad (24)$$

SIDRA INTERSECTION estimates performance measures (delay, queue length and effective stop rate) for vehicles, pedestrians and all persons separately. Various total values given in persons per hour combining the results for vehicles and pedestrians (e.g. total delay, total effective stops, etc.) are calculated using the Vehicle Occupancy (persons/vehicle) factor. The total vehicle and total pedestrian values are not added together directly, and the total values for the intersection (as experienced by both vehicles and pedestrians) are given in terms of total person values only. The average intersection values are also based on persons, i.e. determined by dividing the total intersection value (e.g. total delay in person-hours per hour) by the total intersection flow in persons/h [5].

2.3 Intersection performance evaluation models

There are different micro simulation and analytical models for evaluation of intersection performance in different software packages. Micro simulation software includes PARAMICS (UK), AIMSUN (Spain), VISSIM (Germany), SIMTRAFFIC, CORSIM. Issues arose in practice with this micro simulation software: data hungry, user specialization, slow for large applications, animation implying unjustified accuracy; and model quality (calibration difficulties and benchmarking) [17].SIDRA standard model, HCM 2010 model and UK TRL model are the three well known analytical models. Both SIDRA standard model and HCM 2010 model are implemented in SIDRA INTERSECTION software and basis on gap-acceptance theory. And UK TRL (linear regression) model implemented on ARCADY and RODEL software. Detail comparison of the three analytical models [3, 4, and5] is shown in the Table 2-6.

Table 2-6 Comparison of SIDRA Standard, HCM 2010 and UK TRL models [3, 4, and5]

Model Feature	***SIDRA Standard* Model**	***HCM 2010* Model**	***UK TRL* model**
Methodology	Based-on gap-acceptance theory with empirical (regression) equations to model gap-acceptance parameters.	Empirical (exponential regression) capacity model with clear basis in gap-acceptance theory.	Empirical (linear regression) capacity model with no stated theoretical basis.
Individual Entry and Circulating Lanes	***Lane-based*** model: Capacity and performance of individual entry lanes are modeled.	***Lane-based*** model: Capacity and performance of individual entry lanes are modeled.	***Approach-based*** model: All lanes aggregated and capacity and performance modeled for the approach as whole.
	Variations in lane disciplines (exclusive and shared lanes, slip and continuous lanes) can be modeled.	Variations in lane disciplines (exclusive and shared lanes, slip and continuous lanes) can be modeled.	Variations in lane disciplines (exclusive and shared lanes, slip and continuous lanes) cannot be modeled.
	Dominant and subdominant entry lanes identified.	Dominant and subdominant entry lanes identified.	Entry lanes not identified.
	Number of circulating lanes affects capacity.	Number of circulating lanes affects capacity.	Number of circulating lanes does not affect capacity.
	Circulating lane flow rates used allowing for unbalanced flows. Amount of queuing before entering circulating stream affects capacity.	Total circulating flow rate used. Circulating lane flows not used.	Total circulating flow used. Circulating lane flows not used.
	Assumes bunched arrival headways for the circulating stream. Proportion bunched modeled. Extra bunching to model upstream signal effects allowed.	Assumes random arrival headways for the circulating stream. Effect of upstream signals modeled as an extension in SIDRA INTERSECTION software.	No explicit assumptions about circulating stream headways. Not applicable.
	A proportion of exiting flow can be added to circulating flow as opposing flow.	Not applicable.	*Not known to the author.*
Lane Utilization for Multilane Approaches	Entry lane flow rates are calculated.	Entry lane flow rates are calculated.	No lane flow details.
	De facto exclusive lanes are identified.	De facto exclusive lanes are identified.	De facto exclusive lanes cannot be identified.
	Unequal lane use can be modeled by specifying lane utilization ratios.	Unequal lane use can be modeled by specifying lane volume percentages.	Unequal lane use cannot be modeled.
Volume / Capacity Ratio	v/c ratio (degree of saturation) for a multilane approach represents the *critical lane* value.	v/c ratio for a multilane approach represents the *critical lane* value.	Only the average v/c ratio for the approach is available. This will underestimate the higher v/c ratio of the critical lane unless equal lane use exists.

Model Feature	*SIDRA Standard* Model	*HCM 2010* Model	*UK TRL* model
Driver Behavior Parameters	Gap-acceptance parameters (Follow-up Headway, Critical Gap), entry lane-use model, circulating stream bunching represent driver behavior. Driver response times determined.	Gap-acceptance parameters (Follow-up Headway, Critical Gap), entry lane-use model, circulating stream bunching represent driver behavior.	No direct representation of any aspect of driver behavior. Capacity is sensitive to the circulating flow rate only.
	Follow-up Headway and Critical Gap depend on roundabout geometry.	Follow-up Headway, Critical Gap values are constant.	Not applicable.
	Follow-up Headway and Critical Gap values are reduced (more aggressive driver behavior) with increased circulating flow rates.	Follow-up Headway, Critical Gap values are constant.	Not applicable.
	Priority sharing and priority emphasis effects are included in the model.	Not applicable.	Not applicable.
Roundabout Geometry Parameters (list of geometry parameters affecting capacity) Differences in sensitivities indicated.	Average entry lane width	Not used	Total entry width
	Number of entry lanes	Number of entry lanes	Not used
	Number of circulating lanes	Number of circulating lanes	Not used
	Inscribed diameter With increased inscribed diameter: capacity increases and then decreases for very large roundabouts.	Not used	Inscribed diameter With increased inscribed diameter: capacity increases with increasing inscribed diameter; capacity does not decrease for very large roundabouts.
	Entry radius With increased entry radius: the capacity at zero circulating flow increases (more capacity), and the slope of the capacity curve decreases (more capacity); capacity remains same if the capacity at zero circulating flow is user-specified.	Not used	Entry radius With increased entry radius: the capacity at zero circulating flow increases (more capacity), and the slope of the capacity curve also increases (less capacity); capacity decreases if the capacity at zero circulating flow is user-specified.
	Entry angle With decreased entry angle: the capacity at zero circulating flow increases (more capacity), and the slope of the capacity curve decreases (more capacity); capacity remains same if the capacity at zero circulating flow is user-specified.	Not used	Entry angle With decreased entry angle: the capacity at zero circulating flow increases (more capacity), and the slope of the capacity curve also increases (less capacity); capacity decreases if the capacity at zero circulating flow is user-specified.
	Approach short lane capacity and overflow into adjacent lane modeled using gap-acceptance cycles and back of queue modeling.	Short lanes modeled as an extension in SIDRA INTERSECTION software.	Approach flaring (Approach half width and Flare length). Interpolation for lane width between single and multilane approach values problematic.
	Number of exit lanes and exit short lanes (merge lanes) modeled through effect on upstream approach lane use (increased v/c ratio due to lane	Not applicable.	Not applicable.

Model Feature	***SIDRA Standard* Model**	***HCM 2010* Model**	***UK TRL* model**
Unbalanced Flows	Capacity is sensitive to Origin-Destination demand flow pattern, lane use and level of queuing on approaches. Roundabout modeled with high level of interaction between traffic using all intersection approaches (O-D Factor method used).	No sensitivity to Origin-Destination flow patterns. Roundabout modelled as a series of T-intersections. O-D Factor method offered as an extension in SIDRA INTERSECTION software.	No sensitivity to O-D flow patterns. Roundabout modeled as a series of T-intersections.
	Adjustment options exist for high Entry Flow / Circulating Flow ratio (increased entry capacity at very low circulating flow rates due to increased driver aggressiveness level).	Adjustment options for high Entry Flow / Circulating Flow ratio offered as an extension in SIDRA INTERSECTION software.	Not applicable.
Heavy Vehicles	Circulating flow rate is increased for heavy vehicles in the circulating stream. Follow-up Headway and Critical Gap values are increased for heavy vehicles in the entry lane.	Capacity is decreased for heavy vehicles directly.	*Not known to the author.*
Model Calibration	Intersection-level or approach-level calibration using Environment Factor. A general value of 1.2 used for US conditions. Movement-level calibration using Follow-up Headway and Critical Gap parameters.	Method described to calibrate the model parameters using known Follow-up Headway and Critical Gap values.	The capacity at zero circulating flow (y-intercept) value of the linear regression capacity function can be adjusted. Problematic since the capacity decreases with improved geometry (increased entry radius, decreased entry angle, increase entry width, increased flare length) if the capacity at zero circulating flow is user-specified.
	Sensitivity analysis facility is available for driver behavior and roundabout geometry parameters.	Offered as an extension in SIDRA INTERSECTION software.	*Not known to the author.*
Level of Service	Uses HCM and additional level of service methods (options for alternative LOS methods including HCM 2010 and HCM 2000 methods, ICU method, etc.); the *LOS Target* parameter to specify acceptable LOS levels for different intersection types.	HCM 2010 LOS methods define different LOS thresholds for signalized intersections and all unsignalized intersections.	*Not known to the author.*
	Roundabout LOS options ("Same as Sign Control", "Same as Signalised Intersections" and "SIDRA Roundabout LOS") available; uses "Same as Signalised Intersections" as default.	Same LOS thresholds for roundabouts and sign-controlled intersections.	

Model Feature	*SIDRA Standard* Model	*HCM 2010* Model	*UK TRL* model
Drive Cycles	Detailed drive-cycle model (cruise, decelerate, idle, accelerate) of movements through the intersection. determined for queued and unqueued vehicles (light and heavy vehicles separately) for each lane. Negotiation radius, speed and distance calculated (used for geometric delay, fuel consumption, emissions and operating cost).	Aggregate model.	Aggregate model.
Delay, Queue and Stops	The gap-acceptance cycles are identified for modelling delay, back of queue, stop rate, proportion queued, etc. for each lane (as well as capacity). *Geometric delay* determined. *Back of queue* is important for modeling short lane capacities and blocking of upstream intersections. *Percentile queue* values (not a single value) and *probability of blockage* of upstream lanes calculated.	Simple queuing theory for delay *and cycle-average queue*. *Geometric delay* not determined. 95th percentile queue only for unsignalized intersections. No back of queue model for unsignalized intersections.	Simple queuing theory for delay and cycle-average queue. *Geometric delay* ? No back of queue model.
Fuel Consumption, Emissions and Operating Cost	Detailed vehicle power-based model using drive cycle information derived for queued and unqueued vehicles in each lane. Light and heavy vehicles modelled separately. Drive cycle model incorporating acceleration - deceleration models are important for geometric delay, fuel consumption, emissions and operating cost.	Not available.	Not available.

2.4 Factors affecting intersection performance

Intersection means an area where two highways or more meet or cross each other .These intersections are very crucial on highways as they control the efficiency , safety and capacity of the vehicles and others user of the road facility [18]. The following features affects intersection performance;

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

- Turning volumes
- Horizontal curve radii
- sight distance
- proximity of adjacent intersections
- Types of adjacent intersections [19]

2.4.1 Factors affecting intersection performance for different road users

Intersections should accommodate all users of the facility, including vehicles, bicyclists, pedestrians and transit [19].

Pedestrians

Key elements affecting intersection performance for pedestrians are: (1) amount of right-of-way provided for the pedestrian including both sidewalk and crosswalk width, accuracy of slopes and cross slopes on curb cut ramps and walkways, audible and/or tactile cues for people with limited sight, and absence of obstacles in accessible path; (2) crossing distance and resulting duration of exposure to conflicts with motor vehicle and bicycle traffic; (3) volume of conflicting traffic; and (4) speed and visibility of approaching traffic [20].

Bicyclists

Key elements affecting intersection performance for bicycles are: (1) degree to which pavement is shared or used exclusively by bicycles; (2) relationship between turning and through movements for motor vehicles and bicycles; (3) traffic control for bicycles; (4) differential in speed between motor vehicle and bicycle traffic; and (5) visibility of the bicyclist [20].

Motor vehicles

Key elements affecting intersection performance for motor vehicles are: (1) type of traffic control; (2) vehicular capacity of the intersection, determined primarily from the number of lanes and traffic control (although there are other factors); (3) ability to make turning movements; (4) visibility of approaching and crossing pedestrians and bicycles; and (5) speed and visibility of approaching and crossing motor vehicles [20].

Transit

When transit operations involve buses, they share the same key characteristics as vehicles. In addition, transit operations may involve a transit stop at an intersection area, and influence pedestrian, bicycle, and motor vehicle flow and safety [20].

Human factors common to all road users

The task of traveling on the roadway system, whether by motor vehicle, bicycle, or foot, primarily involves searching for, finding, understanding, and applying information, as well as reacting to the appearance of unanticipated information. Once found and understood, the relevance of this information must be assessed and decisions and actions taken in response. This activity is cyclic, often occurring many times per second in complex, demanding environments. The capabilities of human vision, information processing, and memory all affect a road user's ability to use an intersection, and these may affect the likelihood of user error [21].

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3 Research methodology

3.1 General

This thesis has been compiled by intensive review of literatures related with the title of the research and by associating it with the real ground conditions. After thorough study of literatures on the topic, field observation have been made on the current road and intersection conditions of Bahir Dar city to select intersections for the study. Geometric data, site plane and other necessary information about the intersections have been collected from concerned bodies in the city. By giving the training for data collection traffic counts of pedestrians, cyclists, and motorized vehicles and measurement of geometric elements of the selected intersections in the city have been conducted. An interview to the traffic polices, road users and to different organization have done on the issues concerning about the performance of intersections in the city. Historical traffic or traffic growth rate data of the city had been collected to forecast the traffic for prediction of intersection performance. The Signalized and Unsignalized Intersection Design and Research Aid (SIDRA 5.1) developed by the Australian Road Research Board (ARRB) and Akcelik Associates with Microsoft excel is used for analysis and evaluation of intersection performance based on USA HCM and Austria road manuals; depending on the driver behavior and road way, traffic and geometric condition at the intersection areas in the city.

3.2 Data collection

To select intersections for the study and to decide on the peak hour, field observation have been made on the current roads of Bahir dar city. Since traffic and geometric data's are too important to achieve the objective of this thesis, it was found necessary to collect this data using skilled persons and by assigning them at the intersections. And yet, since it was not easy to get skilled manpower for the task, high school students has trained at reasonable expense.

As a result, after being skillfully trained, the trainees were able how to fill the general information on appropriate forms for vehicle traffic volume and pedestrian counts and how to measure geometric elements of the intersections.

3.2.1 Traffic data collection

In order to encompass a good image about the site and the flow condition, repeated visual visits and information from traffic polices are taken. From these repeated visual visits a decision on the peak flow period time is arrived. And the traffic volume is clearly higher in the nightfall than at the morning times. And from among the night fall times the time from 5:00-6:15 is peak flow period and to determine the peak hour exactly the data are collected at 15 minute intervals. Since Both vehicle and pedestrian traffics have a great influence on the performance of intersections and that why traffic count is made for both vehicle and pedestrian traffics on Thursday. Four junctions which have high traffic volume are selected for the study and these are closely spaced traffic signal and roundabout of Goirgis's, roundabout at papyrus and the junction with no traffic control system at Kucht lelmat building. Except roundabout at Goirgis with three legs, all are four leg intersections.

In Bahir dar city there is high volume of bicycles, Bajaj and pedestrian traffics compared with other vehicular traffics in divergence to most areas of the country. And hence bajaj, bicycles, pedestrians, light (cars, taxi, minibus, pickups) and heavy vehicles (bus, trucks, truck and trailer are counted separately as well and this are shown in the Appendix A. For analysis and design of intersections such a mixed traffic should be converted into passenger cars. To change all vehicle into a passenger cars (light vehicles in this case) passenger car unit (pcu) for each vehicles is needed. Actually pcu value is depends on the characteristics of vehicle, traffic stream, roadway, environment and control and climatic conditions. But there is scarce research that specifies pcu values of Bahir dar as well as for Ethiopia as a whole. And to adopt passenger car unit it becomes a must to consider recently done researches on pcu for Tamale in Ghana (Charles Anum Adams,M. ,Abdul Muhsin Zambangand ,Richter Opoku – Boahen on May 2014) (Table 3.1) and Passenger car unit values recommended by the Indian Road Congress (IRC) (Table 3.2) which have mixed traffic and high proportion of motorcycles and tricycles (Bajaji) in their traffic composition like in Bahir dar.

Table 3-1 PCU values for Barclays bank and Agric intersections in Tamale, Ghana

Intersection	Motorcycle	Tricycle	Car/Taxis	Bus/Truck
Barclays bank	0.30	0.75	1.00	1.52
Agric	0.38	0.67	1.00	1.68

Table 3-2 PCU values recommended by the Indian Road Congress (IRC)

Vehicle Type	Passenger Car Units
Motorcycle	0.5
Tricycles	0.7
Buses	2.0

From Table 3.1 pcu for motorcycle to Baclays bank and Agric intersections are 0.3, 0.38 respectively and from Table 3.2 pcu value of 0.5 for motorcycle is recommended by IRC. And in this paper motorcycle and bicycles are counted in one and it will reduce somehow pcu value that only motorcycle will have. By considering this and the recommendation of IRC and research on pcu values for Tamale in Ghana pcu of 0.33 is taken for bicycles and motorcycle in this paper.

PCU of tricycles for Baclays bank and Agric intersections from Table 3.1 are 0.75 and 0.67 respectively and from Table 3.2 pcu value of 0.7 for tricycles is recommended by IRC. Here tricycles include motorized (Bajaj) and non-motorized tricycles that used for goods transport. But non-motorized tricycles are not common in Bahir dar, only motorized tricycle (Bajaj) exist in the city. And only motorized tricycles (Bajaj) will have lesser pcu value than motorized and non-motorized tricycles will have. Therefore by considering this condition and recommended pcu values for Tamale in Ghana and IRC for this paper pcu value of 0.5 is taken for Bajaj's.

Proportion of heavy vehicles relative to other vehicular traffic in Bahir dar city is very small and Bus and medium Trucks are the major compositions of heavy vehicles in the city. There are many literatures that recommend pcu value for heavy vehicles. And most of these literatures recommend pcu value for bus and medium trucks in the vicinity of 2. And in this paper pcu value of 2 is taken for heavy vehicles. But pcu unit value of heavy vehicle is used directly in SIDRA intersection as input data and the software will change heavy vehicles into passenger cars by itself. For other vehicles (bicycles, motorcycles and Bajaj in this paper) the user should have to change the vehicles into passenger car by using pcu to feed the vehicles volume data into the software.

To determine the peak hour average number of vehicles (in pcu) and pedestrians of all intersections in each 15 minute intervals of peak flow period are and it is shown in Table 3.3 and Figure 3.1 below.

Table 3-3 Average number of vehicles (in pcu) and pedestrians for each 15 minute intervals of peak flow period in all four intersections

Time	Average number of Vehicles	Average number of pedstrians
5:00-5:15	309	1030
5:15-5:30	341	1150
5:30-5:45	363	1180
5:45-6:00	392	1263
6:00-6:15	303	911

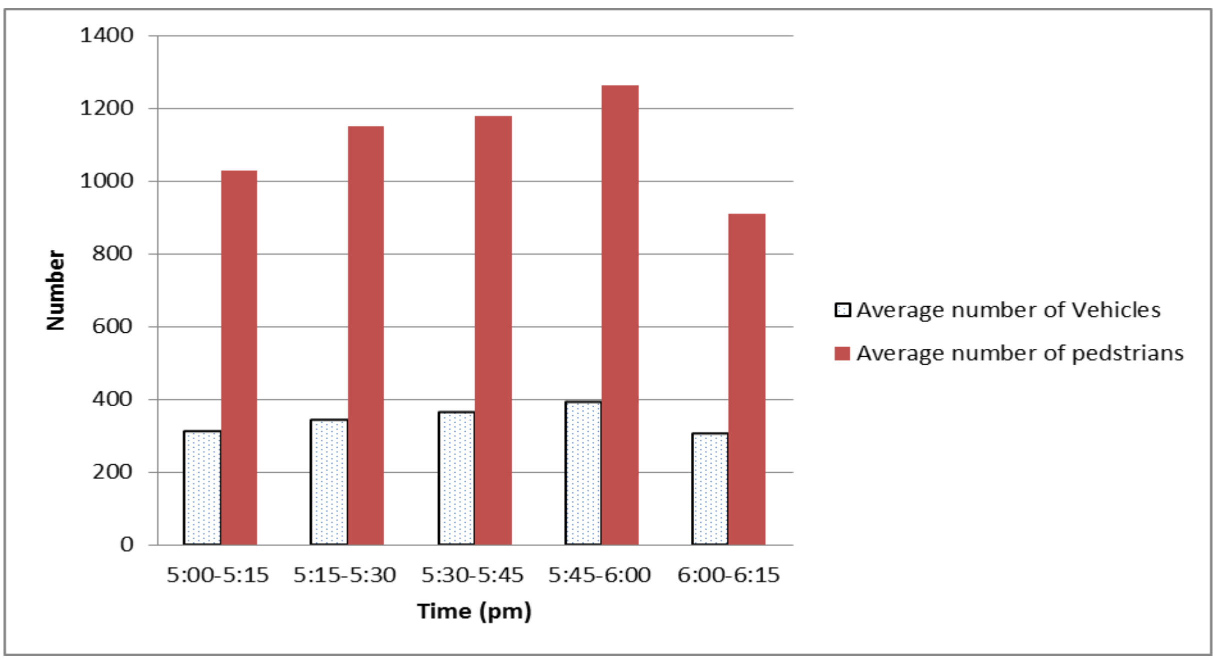


Figure 3.1 Average number of vehicles (in pcu) and pedestrians for each 15 minute interval in all four intersections

A Figure 3.1 show the time from 5:45-6:00 pm is the 15 minute interval which has higher traffic volume than other 15 minute intervals of peak flow period (5:00-6:15 pm) and both vehicles and pedestrians traffics decreases in both directions from this time. In addition to this the traffic volume for the two end 15 minute intervals (5:00-6:00 and 6:00-6:15 pm) is low relative to other intermediate 15 minute interval traffic volumes and logically this indicates that the traffic volume could not be higher in number than from the selected range of peak flow period time. And it assures the decision made on peak flow period determination by the visual visit is correct. And from among the two ends 15 minute intervals time interval from 6:00-6:15 pm has lower vehicle as well as pedestrian traffics. Therefore the time from 5:00-6:00 pm is taken as peak hour for the area and used in this paper. And in this peak hour number of vehicles and pedestrians for each intersection from their approach legs are shown in the Table 3.4.

Table 3-4 Number of vehicles and pedestrians for each intersections from their approach legs in the peak 1 hour

Time (PM)	Bicycles and motor bicycles (A)			Bajaj (B)			Light vehicles (C)			Heavy vehicles (D)			Totoal vehicles (pcu) (A/3+B/2+C+D)			Percentage of Heavy vehicles	Pedestrians
To	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R		
Georgis Round about vehicle & pedestrian traffics																	
From Hospital-Abyssinia bank approach leg																	
5:00-6:00		101	236		178	414		85	199		7	17		215	501	3.4	1096
Poli-Georgis approach leg																	
5:00-6:00	187	80		174	74		305	131		34	15		489	209		7.0	1400
Gebya-protection house approach leg																	
5:00-6:00	114		105	321		308	101		109	6		5	305		304	1.8	810
Georgics trafic signal vehicle & pedestrian traffics																	
From Hospital-kersima approach leg																	
5:00-6:00	37	124	45	173	472	141	58	192	70	4	12	4	161	481	160	2.5	494
Kebele 5-Ethiopia hotel approach leg																	
5:00-6:00	26	52	29	15	28	11	10	25	12	3	6	3	29	61	30	9	1797
Gebya-Ethiopia hotel approach leg entry vehicle & pedestrian traffics																	
5:00-6:00	43	144	53	163	445	134	47	158	58	3	8	3	146	437	145	1.9	670
Mesgid -Zmamnesh building approach leg entry vehicle & pedestrian traffics																	
Total	41	97	47	29	57	24	20	40	23	2	4	2	50	104	52	3.80	980
Papirus Roundabout vehicle & pedestrian traffics																	
Giorgis-central approach leg																	
5:00-6:00	90	52	116	292	167	375	87	50	112	6	3	7	268	153	345	2.1	3236
Azwa- Jinad approach leg entry vehicle & pedestrian traffics																	
5:00-6:00	161	125	72	224	174	99	111	86	49	27	21	12	304	236	135	8.9	2284
Addis amba-central caffe approach leg entry vehicle & pedestrian traffics																	
5:00-6:00	36	64	82	110	193	248	39	68	87	9	15	19	114	200	257	7.5	2003
Kebt tera-papirus approach leg entry vehicle & pedestrian traffics																	
5:00-6:00	60	68	43	112	128	80	29	33	21	9	10	6	114	130	81	7.7	1472
Kucht junction vehicle & pedestrian traffics																	
Dashn bank-kucht approach leg entry vehicle & pedestrian traffics																	
5:00-6:00		250			627			146			19			562		3.4	828
Azwa-commercial bank approach leg entry vehicle & pedestrian traffics																	
5:00-6:00		265			399			271			45			604		7.5	1414

3.2.2 Geometric data collection

For performance evaluation of intersections in addition to traffic data's geometric data are also necessary and these data's are collected and are showed as the figure below for Papyrus Roundabout and geometric data of other intersections are shown in the Appendix B.

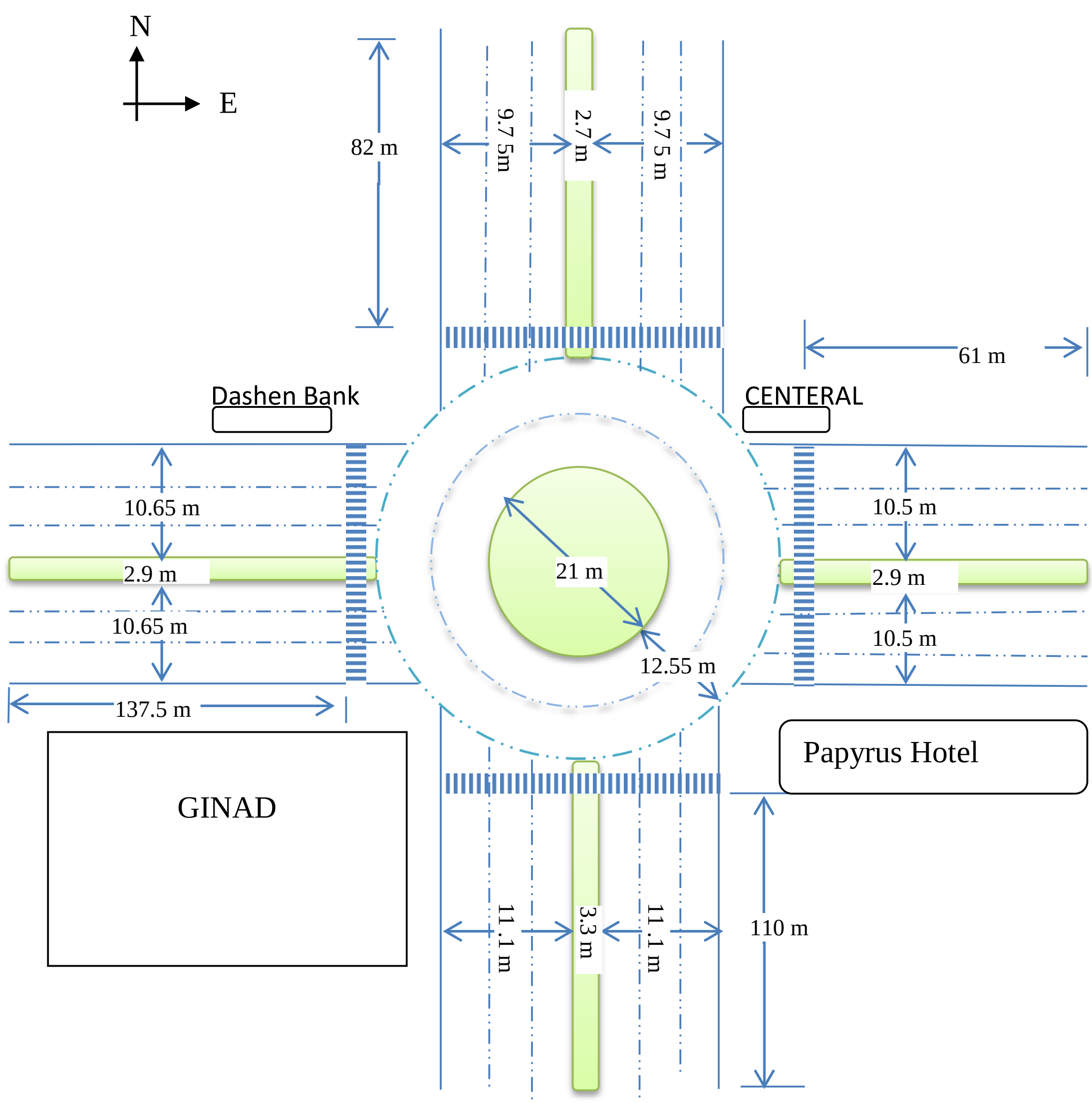


Figure 3.2 Papyrus Roundabout intersection

4 Analysis and discussion

4.1 Introduction

Existing intersection analysis models fall generally into two categories which are Empirical models and Analytical models. Empirical models rely on field data to develop relationships between geometric design features and performance measures such as capacity and delay. Instead Analytical models are based on the concept of gap acceptance theory. The choice of an analysis approach depends on the calibration data available. For this specific research we used the analytical model of intersection analysis. This is done through the use of SIDRA INTERSECTION version 5.1 software. SIDRA INTERSECTION used to:

- Analyse a large number of intersection types including signalised intersections (fixed-time / pre timed and actuated), signalised pedestrian crossings, single point interchanges (signalised), roundabouts, roundabout metering, two-way stop sign control, all-way stop sign control, and give-way / yield sign-control;
- Obtain estimates of capacity and performance characteristics such as delay, queue length, stop rate as well as operating cost, fuel consumption and pollutant emissions for all intersection types;
- Analyse many design alternatives to optimise the intersection geometry, signal phasing and timings specifying different strategies for optimisation;
- Handle intersections with up to 8 legs, each with one-way or two-way traffic, one-lane or multi- lane approaches, and short lanes, slip lanes, continuous lanes and turn bans as relevant;
- Determine signal timings (fixed-time / pre timed and actuated) for any intersection geometry allowing for simple as well as complex phasing arrangements;
- Carry out a design life analysis to assess impact of traffic growth;
- Carry out a parameter sensitivity analysis for calibration, optimisation, evaluation and geometric design purposes;
- Design intersection geometry including lane use arrangements taking advantage of the unique lane-by-lane analysis method of SIDRA INTERSECTION;

- Design short lane lengths (turn bays, lanes with parking upstream, and loss of a lane at the exit side);
- Analyse effects of heavy vehicles on intersection performance;
- Analyse complicated cases of shared lanes and opposed turns (e.g. permissive and protected phases, slip lanes, turns on red);
- Analyse oversaturated conditions making use of the time-dependent delay, queue length and stop rate models used in SIDRA INTERSECTION.

In using SIDRA INTERSECTION, you can:

- Prepare data and inspect output with ease due to the graphical nature of SIDRA INTERSECTION input and output, and using extensive templates supplied with the software;
- Obtain output including capacity, timing and performance results reported for individual lanes, individual movements (or lane groups), movement groupings (such as vehicles and pedestrians), and for the intersection as a whole;
- Control the amount of output by selecting individual output tables, with options for summary and full output;
- Set your own options, create and use customised models (defaults system) and customised templates;
- In your reports, present your data and results in picture and graphs form;
- Carry out sensitivity analyses to evaluate the impact of changes on parameters representing intersection geometry and driver behaviour;
- Calibrate the parameters of the operating cost model for your local condition allowing for factors such as the value of time and resource cost of fuel.

4.2 Models, criteria's or values used for analysis

In addition to traffic and geometric data's for performance evaluation of intersections model, criteria or value used for analysis of each performance measures (saturation flow, capacity, delay, queue length, and level of service) should be specified. Model, criteria or value used for:

I. Saturation flow

Saturation flow rate which is the maximum flow rate that can be sustained when there is a queue and the vehicles can depart from the queue, i.e. signals are not red or the gaps in the opposing stream are not too short will be calculated by determining minimum headway between vehicles. But for this paper saturation flow is taken by considering SIDRAs recommendation for intersections environmental conditions.

SIDRA INTERSECTION uses basic saturation flow values, s_b, in through car units per hour (tcu/h) as a starting point for saturation flow estimation for vehicle movements. Table 4.1 gives two different basic saturation flows for two environment classes (area types) representing two different sets of road and traffic conditions. The definitions of the two environment classes (Ideal and Average to Poor) are based on past and recent research on queue discharge characteristics at signalised intersections (Akçelik 1981, Akçelik, Besley and Roper 1999, Akçelik and Besley 2002, TRB 2000).

Table 4-1 Environment classes and default values of basic saturation flows in through car units per hour

Environment class (area type)	Definition	Basic saturation flow, s_b (tcu/h)	
		SIDRA INTERSECTION standard	HCM versions
1 (Ideal)	Near ideal conditions for free movement of vehicles on both approach and exit sides indicated by good intersection geometry, long distances to upstream and downstream intersections, good visibility, small numbers of pedestrians, and little interference due to loading and unloading of goods vehicles, buses or parking turnover.	**1950**	**1900**
2 (Average to Poor)	Average to poor conditions indicated by adequate to poor intersection geometry, usually closely-spaced intersection environment, possibly poor visibility, moderate to large numbers of pedestrians, and interference from standing vehicles, loading and unloading of goods vehicles, buses, parking turnover, and vehicles entering and leaving premises.	1800	1710

Average to poor conditions indicated by adequate to poor intersection geometry, usually closely-spaced intersection environment, possibly poor visibility, moderate to large numbers of pedestrians, and interference from standing vehicles, loading and unloading of goods vehicles, buses, parking turnover, and vehicles entering and leaving premises are all this are physical characteristics of intersections in Bahir Dar city. But numbers of pedestrians is too large relative to vehicular traffics to account for this the saturation flow for intersections in Bahir dar city is taken by deducting 50 from SIDRA INTERSECTION standard value (1800 tcu/h) which is 1750 tcu/h.

II. Capacity

Capacity which is the maximum sustainable flow rate that can be achieved during a specified time period under given (prevailing) road, traffic and control conditions will be calculated for both signalized and unsignalized intersections by the basic equation, $Q=su$,Where Q = capacity (veh/h), u = proportion of time when the vehicles can depart from the queue (signals are green or gaps are available in the opposing stream) and s = saturation (queue discharge) flow rate (veh/h).For signals u value will be calculated by $u=g/c$ but to determine this value for sign-controlled traffic streams, roundabout entry streams and filter (permitted) turns at signalized intersections gap acceptance theory is used in the paper. To estimate the capacity by using this theory determination of arrival headways distribution is fundamental and some of arrival headways distributions are exponential arrival headway distribution models known as negative exponential (M1), shifted negative exponential (M2) and bunched exponential (M3).M1 and M2 model assumes random arrivals with no bunching in contrast with the bunched headways model used by the M3 model. The assumption of no bunching cannot be supported especially at high opposing flow rates where vehicles are highly bunched. M1 and M2 model is normally used for single-lane traffic only. The bunched exponential distribution (M3) offers improved accuracy in the prediction of small arrival headways (up to about 12 seconds), which is important for most urban traffic analysis applications. And this M3 model is selected for calculation of capacity by gap acceptance theory in this paper. This model and other parameters which are important for calculation of gap acceptance capacity and general equations for this gap acceptance capacity itself are described in this paper in section 2.2.1.1 from Equation 4- Equation 9. And in general this gap acceptance capacity model is referred to as the Akçelik M3D model and used in the paper for determination of capacity for sign-controlled traffic streams, roundabout entry streams and filter (permitted) turns at signalized intersections.

III. Delay

Webster's, Akcelik and HCM 2000 delay model are some of the models that used for the determination of delay in signalized intersections. Webster's proposed models for uniform, random, and over flow delays separately and most studies shows uniform delay model holds well in the range X(v/c) < 0.85 and overflow delay model holds well in the range X>1.15 but inconsistency occurs in the range 0.85 <X <1.15; here both random, and over flow delay models are not accurate. To address the above said problem Akcelik proposed an overflow delay model. And HCM 2000 delay model includes the uniform delay model, a version of Akcelik's overflow delay model, and a term covering delay from an existing or residual queue at the beginning of the analysis period and that is why HCM 2000 delay model is used for determination of delay for signalized intersection in this paper. HCM 2000 delay model is also adopted for unsignalised intersections in this paper although Akgelik - Troutbeck, SIDRA delay models are also used to calculate delay for this intersection. Generally HCM 2000 delay model give accurate result than other models of delay and it selected for analysis of intersections in this paper. The detail equations for all models are shown in section 2.2.2 from Equation 10- Equation 21.

IV. Queue length

A percentile queue length is a value below which the specified percentage of the average queue length values observed for individual cycles fall. The 70th, 85th, 90th, 98th and 98th percentile queue lengths are useful for the design of queue spaces (turn bays and parking bans). Use of the 95th percentile value of the back of queue is generally considered to be a good choice for design purposes. And 95th percentile value is used in this paper.

V. Vehicles level of service

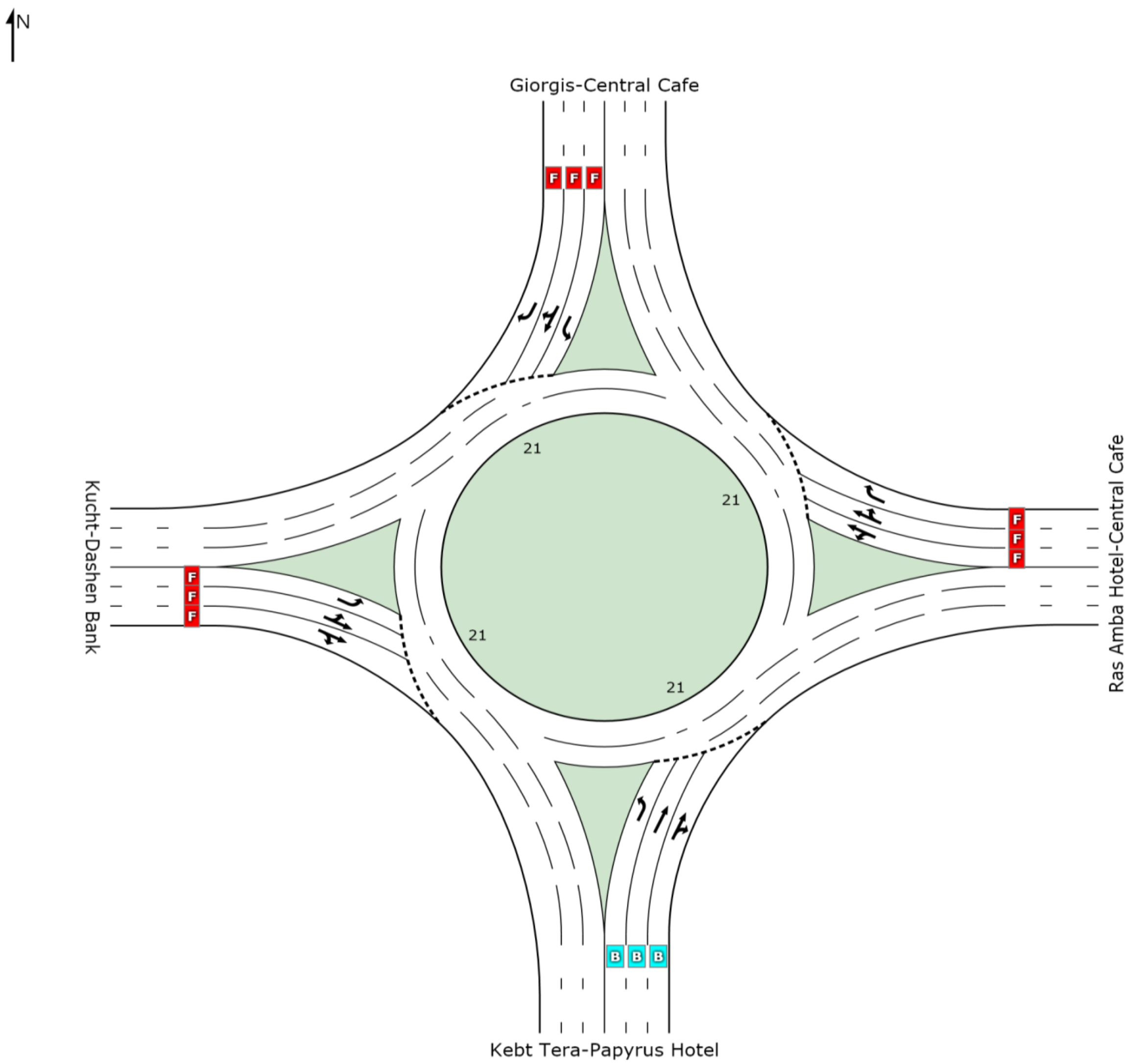
There are different methods for level of service definitions to vehicles such as delay (HCM 2000), Delay & v/c (HCM 2010), and delay & degree of Saturation method. Delay (HCM 2000) method is based on HCM 2000 (TRB 2000) and uses delay only for LOS determination for vehicles. Delay & v/c (HCM 2010) uses delay thresholds which are the same as in the Delay (HCM 2000) method, but assigns LOS F when v/c > 1.0 (oversaturated conditions) irrespective of delay. Delay (HCM) & Degree of Saturation had more indirect conditions that could lead to LOS D or E for degrees of saturation (v/c ratios) close to 1.0. The Delay & v/c (HCM 2010) method is considered as a simplified method and in this paper this method is used for determination of level of services. Details of all the methods and criteria they use are shown in section 2.2.4.1 of this paper.

Intersection performance analysis for selected intersections by using SIDRA INTERSECTION are done and shown as follows.

4.3 Papyrus roundabout performance analysis

In the analysis of papyrus intersection performance by using SIDRAINTERSECTION 5.1 input and output data's are shown in Appendix C and the level of service for the roundabout is shown below in the Figure 4.1.

Level of service



	South	East	North	West	Intersection
LOS	B	F	F	F	F

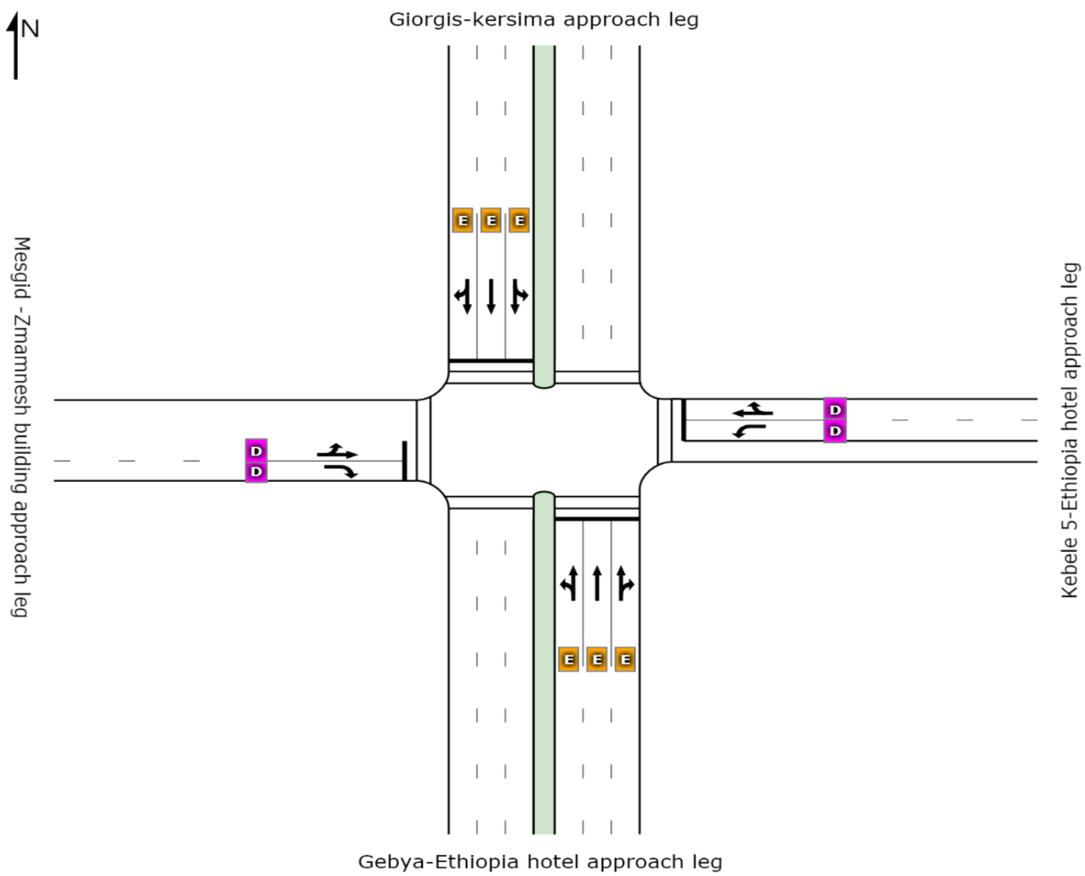
Figure 4.1 Level of service of papyrus intersection

Generally capacity, degree of saturation, delay, queue length, and level of service are the main performance measures of intersections. Analysis result of Papyrus roundabout intersection shows high values of degree of saturation, delay and queue length and low values of capacity for the three approaches of the intersection and in effect level of service of all the three approaches are F but Kebttera-Papyrus approach of the intersection has good performance measure values and its level of service is B. And when considering traffic demand of this approach it has low vehicle as well as pedestrian traffic relative to other three approaches of the intersection.

4.4 Performance analysis of traffic signal near Giorgis

In the analysis of intersection performance of traffic signal by using SIDRAINTERSECTION 5.1 input and output data's are shown in Appendix D. And the level of service for Giorgis traffic signal is shown below in the Figure 4.2.

Level of service of traffic signal



	South	East	North	West	Intersection
LOS	E	D	E	D	E

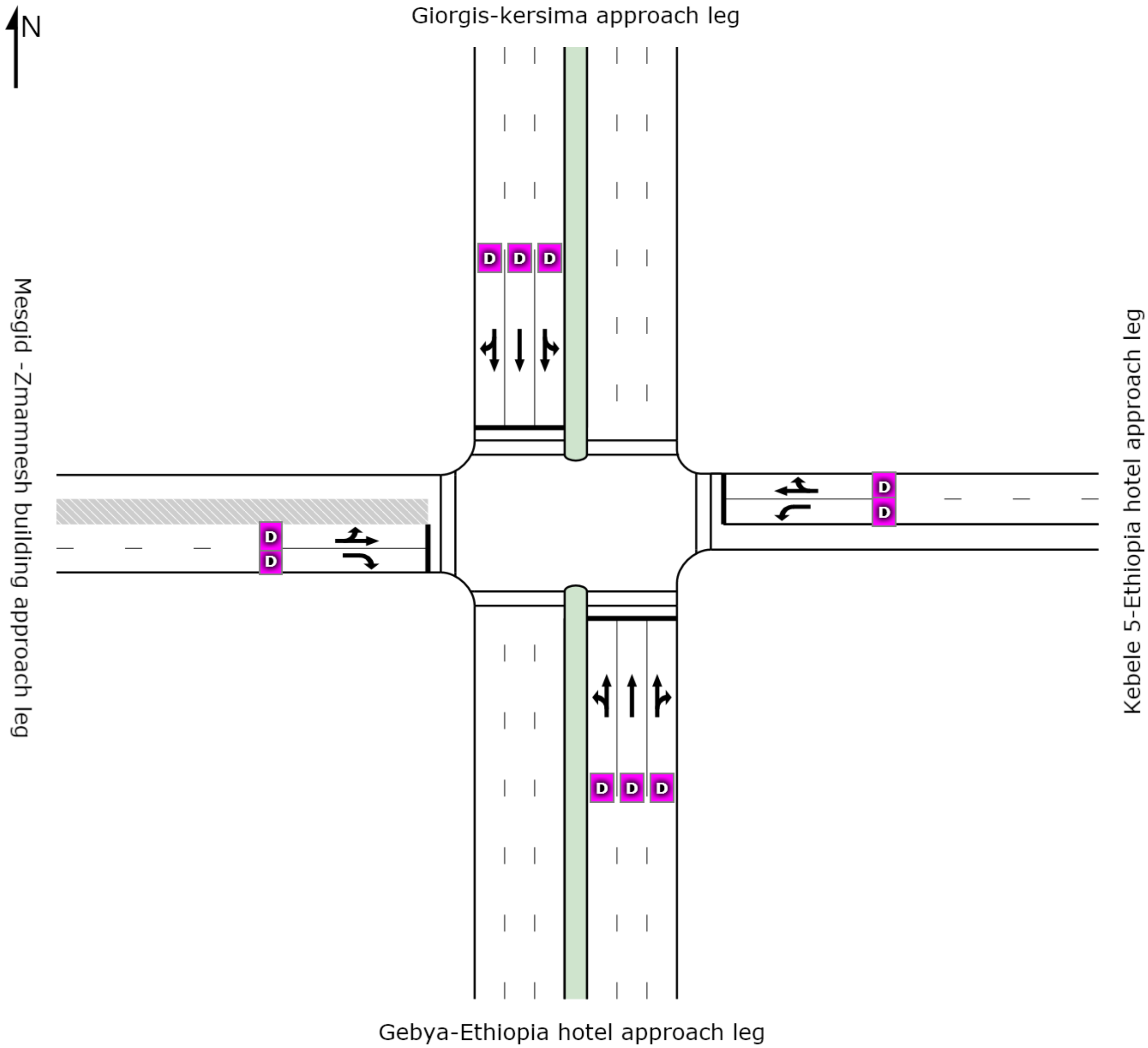
Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.
LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).
HCM Delay Model used. Geometric Delay not included.

Figure 4.2 Level of service of traffic signal near giorgis

In traffic signal near Giorgis there is high vehicle traffic in the major road and high pedestrian traffic crossing the minor approach roads and values of intersection performance measures such as degree of saturation, delay and queue length are high for the major approach roads as a result level of service for this approach as well as for the intersection as a whole is E. But equal phase times are given to major and minor approach roads and to check whether the performance of the intersection could improve or not three seconds are added on the phase times of the major road by deducting on the phase times of minor road and the result of the analysis is shown below. And from the result the level of service of the intersection has improved from level of service E to D.

4.4.1 Analysis results of traffic signal near Giorgis for improved phases

Level of service of traffic signal for improved phases



	South	East	North	West	Intersection
LOS	D	D	D	D	D

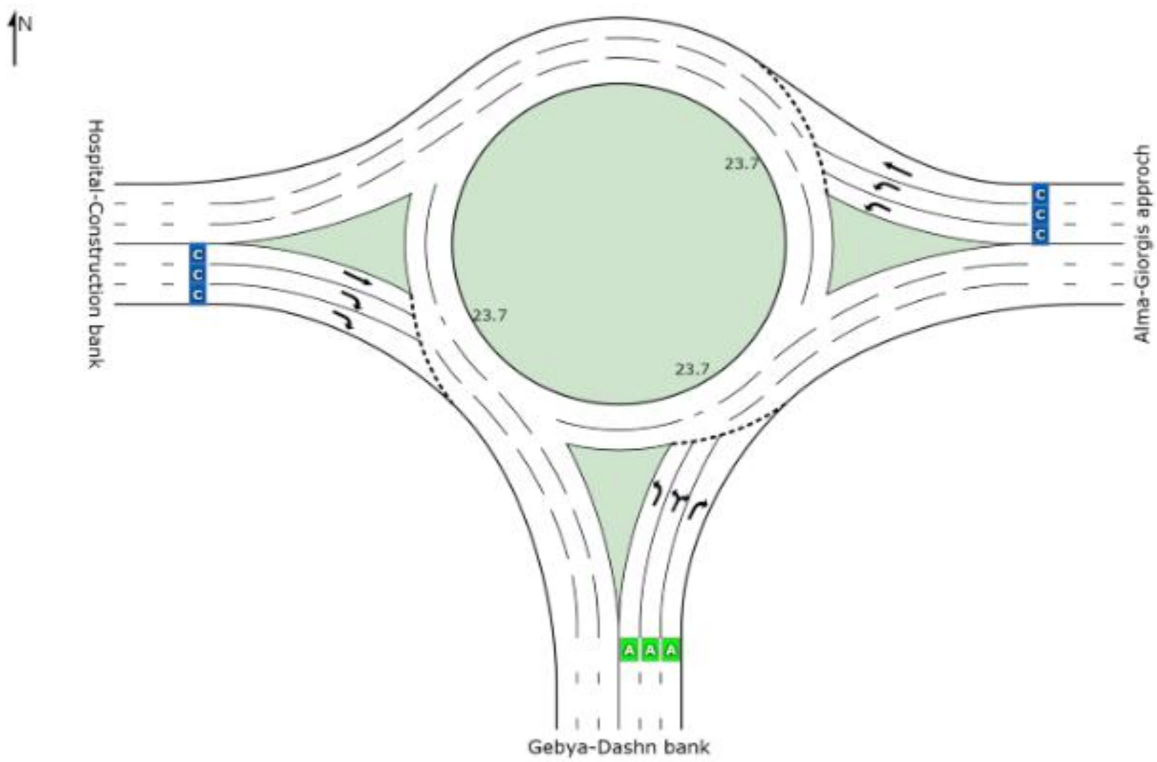
Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.
LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).
HCM Delay Model used. Geometric Delay not included.

Figure 4.3 Level of service of traffic signal near giorgis for improved phase times

4.5 Giorgis roundabout performance analysis

In the analysis of Giorgis intersection performance by using SIDRAINTERSECTION 5.1 input and output data's are shown in Appendix E and the level of service for the roundabout is shown below in the Figure 4.4.

Level of service of Giorgis roundabout



	South	East	West	Intersection
LOS	A	C	C	B

Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Roundabout LOS Method: Same as Signalised Intersections.
Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.
LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).
HCM Delay Model used. Geometric Delay not included.

Figure 4.4 Level of service of giorgis roundabout

All the performance measure values for this intersection are good in general especially for Gebaya-Dashn bank approach leg. The values of degree of saturation, delay and queue length for this approach are very low and its level of service is A. when we see the input data's, the number of pedestrians crossing this approach is small compared to other two approaches which has level of service C. Since level of service of the intersection is B, design life analysis is done to check the performance of intersection for the next 20 years and all the result are shown in the Figure 4.6 below. From the result of the analysis level of service of the intersection will to change to D after 4 years and values of degree of saturation, delay and queue length will increase rapidly after 3 years.

Level of service of Giorgis roundabout after four years

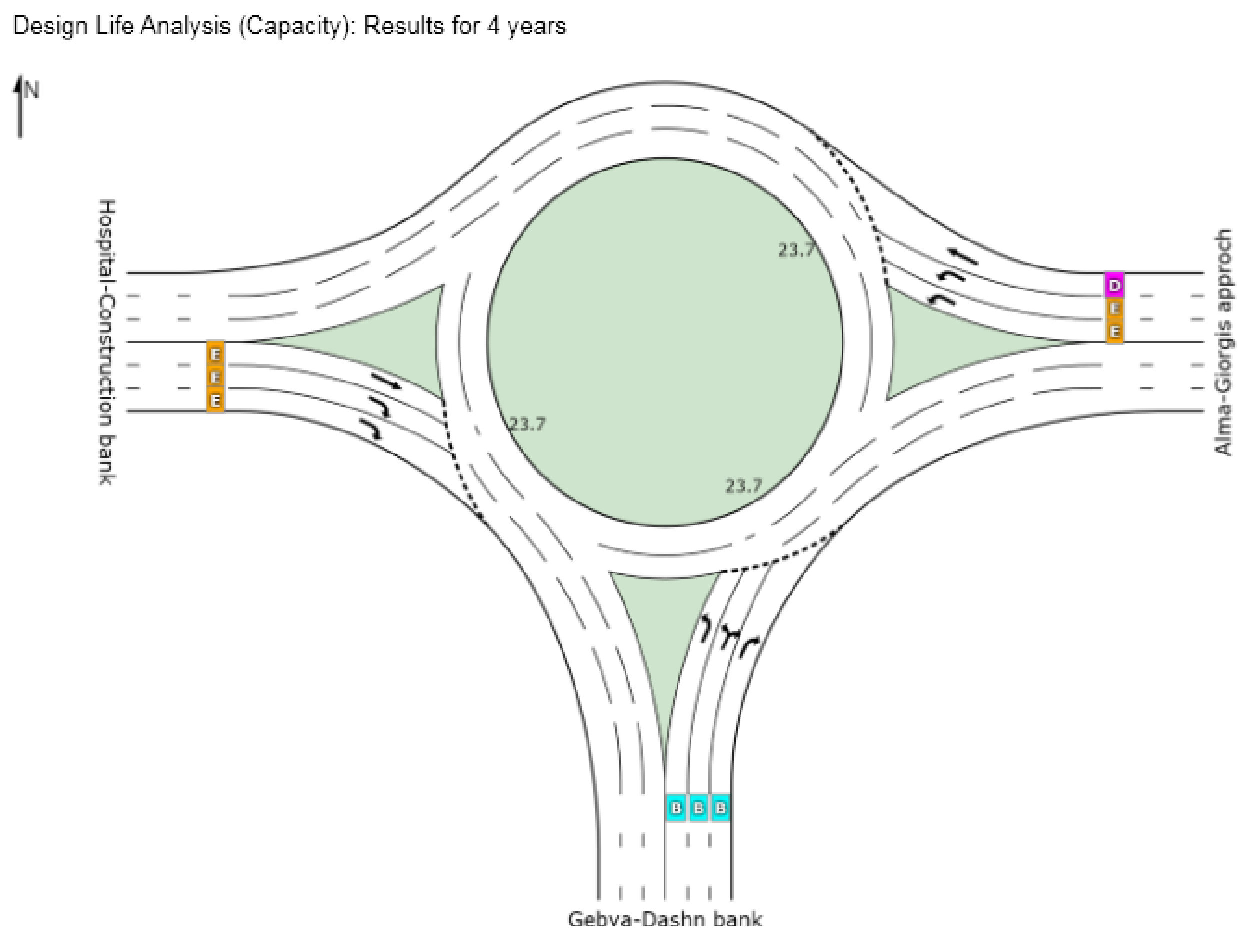


Figure 4.5 Level of service of giorgis roundabout after four years

Graph that shows performance of Giorgis roundabout in its design life

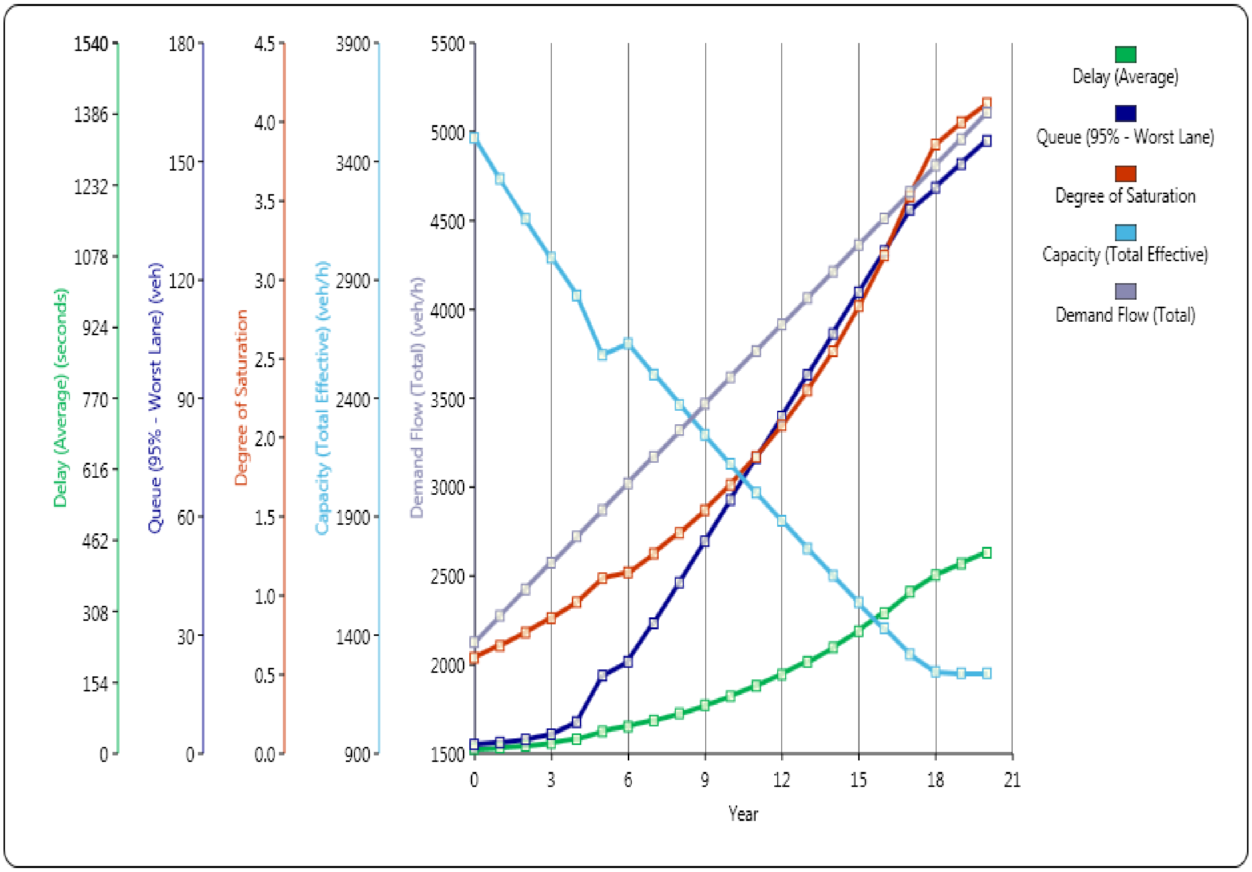
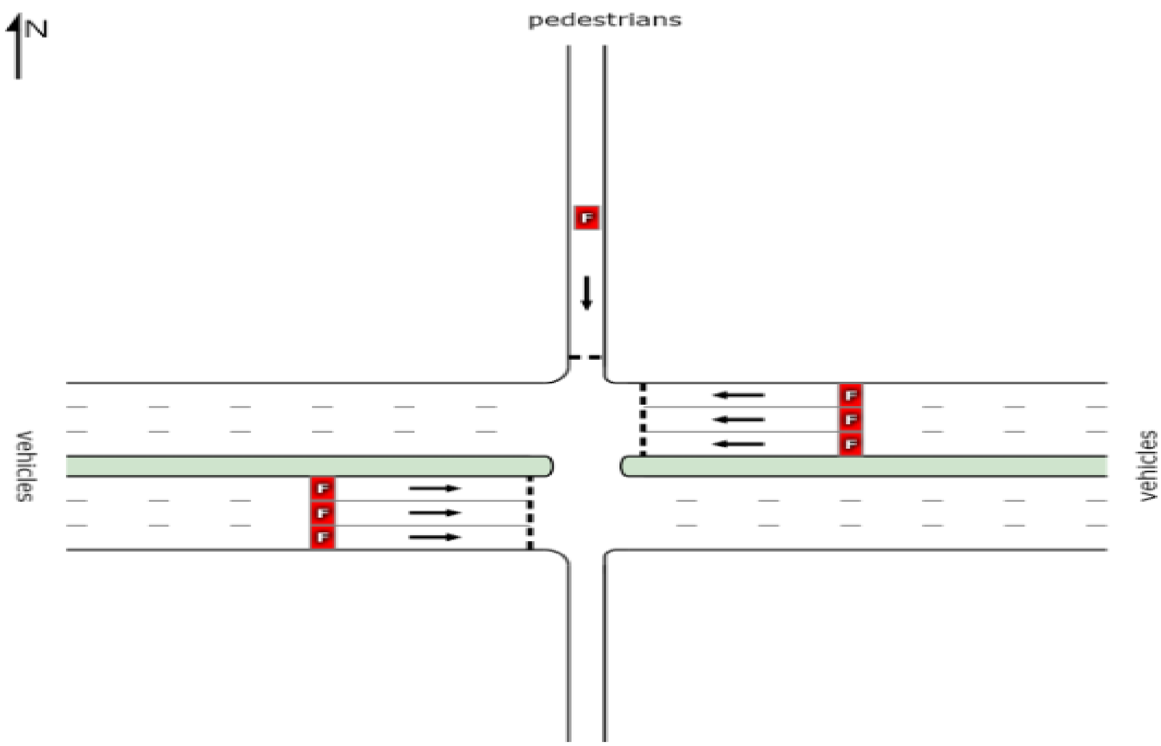


Figure 4.6 Graph that shows performance of giorgis roundabout in its design life

4.6 Performance analysis of Kucht junction

In the analysis of Kucht intersection performance by using SIDRAINTERSECTION 5.1 input and output data's are shown in Appendix F and the level of service for the junction is shown below in the Figure 4.7**.**

Level of service



	South	East	North	West	Intersection
LOS	NA	F	F	F	NA

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.

LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).

Minor Road Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).

NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.

SIDRA Standard Delay Model used.

Figure 4.7 Level of service of Kucht junction

The performance evaluation of Kucht junction shows as its level of service is F and from input data's the reason for this is the number of pedestrians crossing the junction are very high.

4.7 Proportion of each vehicle types and pedestrians

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

Table 4-2 Total number of each vehicle type and pedestrians traffic for each intersection in a hour

Intersection	Bicycles and motor bicycles	Bajaj	Light vehicles	Heavy vehicles	Total vehicles	Pedestrians
Papyrus Roundabout	969	2201	771	144	4085	8995
Giorgis Traffic signal	743	1696	711	53	3203	3941
Giorgis Roundabout	823	1469	930	84	3306	3306
Kucht Junction	515	1026	417	64	2022	2242
Average of all intersections	763	1598	707	86	3154	4621

Proportion of each vehicle types and pedestrians is determined by first by calculating the average of each vehicle types for all intersections to represent the whole intersections. And by dividing this average value of each vehicle type to the total vehicles average proportion of each vehicle type is determined for all intersections and it is shown in the Figure 4.8.Proportion of total vehicles and pedestrians is also determined by using this average values and this is shown in the Figure 4.9.

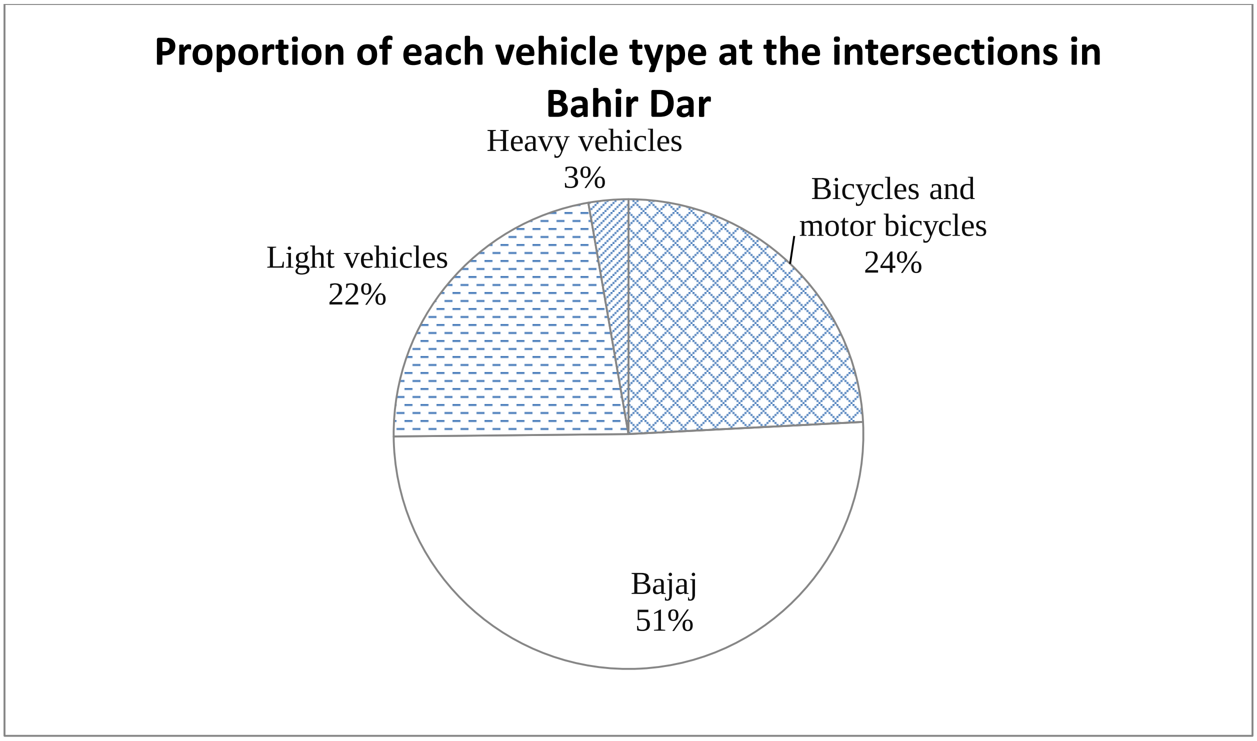


Figure 4.8 Proportion of each vehicle type at the intersections in Bahir Dar city

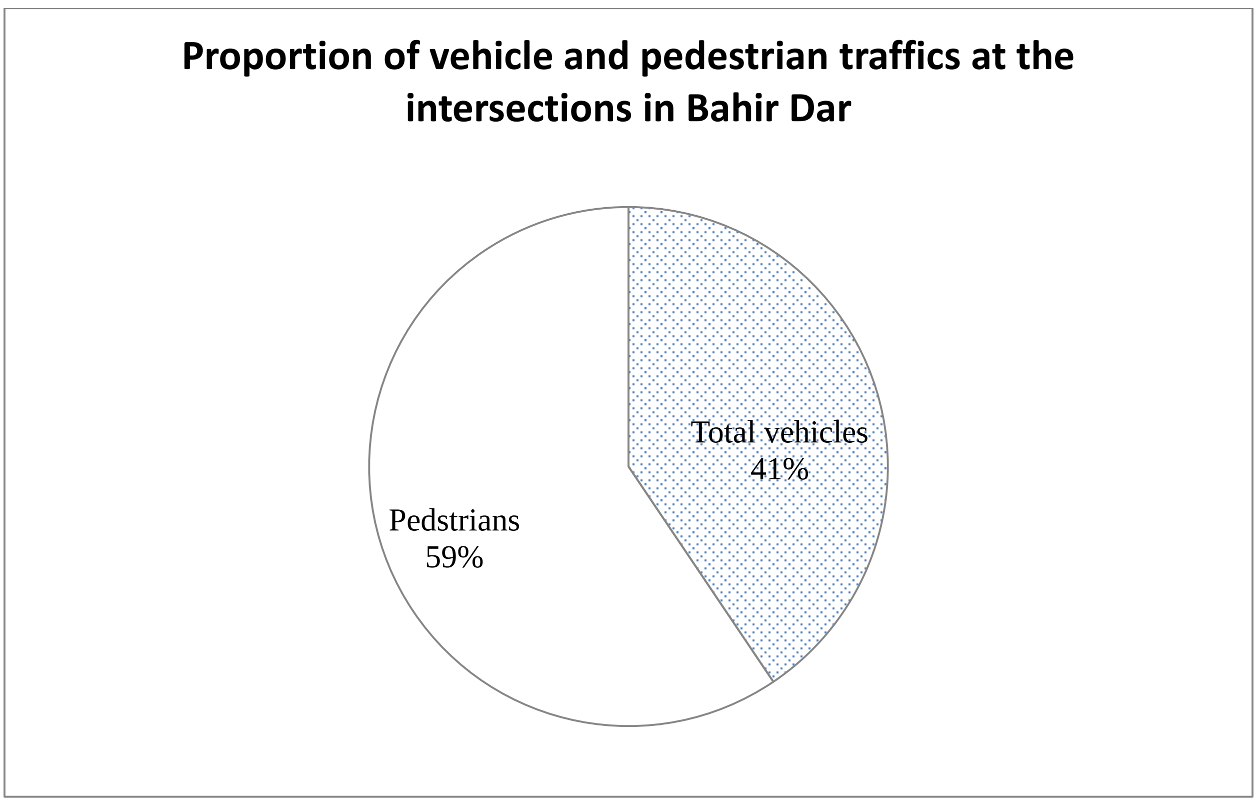


Figure 4.9 Proportion of vehicle and pedestrian traffics at the intersections in Bahir Dar

5 Conclusions and recommendations

5.1 Conclusions

Almost all analysis result of selected major intersections in Bahir dar city shows high values of degree of saturation, delay and queue length and low values of capacity in general and in effect bad level of service of intersections. And all this results revealed as the intersections have poor performances as a results congestion, tremendous economic loss, additional delay and user cost rises from day to day.

Proportion of pedestrian’s traffic at the major intersection areas in Bahir dar city is 59% of the entire traffic and it is the root cause for the intersections to have poor performance. And among the total vehicular traffics Bajaj’s, Bicycles and Motor bicycles covers 75% of the whole vehicular traffics at the intersection areas in the city and they have also obviously a great influence on the performance of intersections. Furthermore the performance of the intersections is highly affected by parking lanes at the intersections approaches. In addition to this, intersections are closely spaced in the city and it has also its own effect on the performance of the intersections. Some of the intersections especially traffic signals have a design problem and the performance of intersection is also affected by it.

5.2 Recommendations

In Bahir dar as well as in other big cities of Ethiopia which has similar traffic compositions, number of Bajaj’s, Bicycles and Motor bicycles should be controlled and the number of other public transports like buses and minibuses should be increased to improve the performance of major intersections in these cities. Besides to this, as much as possible parking areas for vehicles and a separate way for pedestrian, like an over pass, to cross the road at the intersection areas should be prepared to increase the performance of the intersections in these cities. Moreover intersections should be designed properly specially traffic signals in these cities for the intersection to perform as well. Since in Bahir dar city most of the intersections are closely spaced, coordinated traffic signals should be installed to reduce the dalliance at each junctions and to improve the performance of the intersections as a whole. Saturation flow and passenger car equivalent determination for the city are an optional study areas that needs further researches.

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Appendix A-Traffic data

Table A-1 Papyrus Roundabout vehicle & pedestrian traffics from each approach leg

Time (PM)	Bicycles and motor bicycles (A)			Bajaj (B)			Light vehicles (C)			Heavy vehicles (D)			Totoal vehicles (pcu) (A/3+B/2+C+D)			Percent age of Heavy vehicles	Pedestrians
To	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R		
From Giorgis-central approach leg																	
5:00-5:15	22	12	28	61	35	78	17	10	22	1	1	2	56	32	72	2.5	715
5:15-5:30	15	9	20	60	34	77	19	11	25	0	0	0	55	31	71	0.6	864
5:30-5:45	28	16	36	67	38	86	21	12	27	1	1	1	65	37	83	1.6	835
5:45-6:00	26	15	33	104	60	134	29	17	38	3	2	4	93	53	119	3.0	822
6:00-6:15	19	11	24	55	32	71	20	11	26	1	1	2	55	32	71	2.5	705
From Azwa- Jinad approach leg entry vehicle & pedestrian traffics																	
5:00-5:15	39	30	17	6	5	3	20	15	9	7	5	3	43	33	19	15.8	512
5:15-5:30	36	28	16	63	49	28	24	19	11	5	4	2	73	57	32	7.4	514
5:30-5:45	43	34	19	69	54	31	39	30	17	6	5	3	94	73	42	6.3	554
5:45-6:00	43	33	19	86	67	38	28	22	13	9	7	4	94	73	42	9.5	704
6:00-6:15	38	30	17	21	16	9	15	12	7	5	4	2	43	34	19	11.5	506
From Addis amba-central caffe approach leg entry vehicle & pedestrian traffics																	
5:00-5:15	8	13	17	25	44	57	10	17	22	3	5	6	27	48	62	9.5	419
5:15-5:30	10	18	23	29	50	65	9	16	21	3	5	7	30	53	68	10.0	553
5:30-5:45	10	18	23	26	45	58	10	18	23	1	2	3	28	48	62	4.3	493
5:45-6:00	8	15	19	30	53	68	10	17	22	2	3	4	29	51	66	6.2	538
6:00-6:15	6	11	14	29	51	65	8	15	19	3	5	6	28	49	63	10.1	294
From Kebt tera-papirus approach leg entry vehicle & pedestrian traffics																	
5:00-5:15	13	15	9	25	29	18	6	6	4	1	1	1	24	27	17	4.5	282
5:15-5:30	13	15	10	25	28	18	8	9	6	2	2	1	26	30	19	6.7	391
5:30-5:45	18	20	13	33	38	24	10	11	7	4	4	3	36	41	26	9.8	361
5:45-6:00	16	18	11	29	34	21	6	7	4	2	3	2	28	32	20	8.6	438
6:00-6:15	12	13	8	26	30	19	6	7	5	2	2	1	25	28	18	7.0	201

Table A-2 Gorgis traffic signal vehicle & pedestrian traffics from each approach leg

Time	Bicycles and motor bicycles (A)			Bajaj (B)			Light vehicles (C)			Heavy vehicles (D)			Totoal vehicles (pcu) (A/3+B/2+C+D)			Percent age of Heavy vehicles	Pedestrians
To	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R		
From Hospital-kersima approach leg																	
5:00-5:15	7	25	9	40	109	33	14	47	17	1	2	1	37	112	37	2.1	195
5:15-5:30	10	32	12	47	129	39	13	42	15	1	2	1	40	120	39	2.0	76
5:30-5:45	12	39	14	41	113	34	14	47	17	1	3	1	40	119	40	2.5	134
5:45-6:00	8	28	10	44	121	36	17	56	21	2	4	1	43	130	43	3.2	89
6:00-6:15	8	25	9	39	105	32	13	43	16	1	2	1	35	106	35	2.3	75
Fromo Kebele 5-Ethiopia hotel approach leg																	
5:00-5:15	6	12	7	3	5	2	2	5	2	1	1	1	6	12	6	8.3	501
5:15-5:30	7	13	7	4	8	3	3	7	3	1	1	1	8	16	8	6.4	389
5:30-5:45	7	14	8	4	8	3	3	7	3	1	2	1	8	17	8	12.0	438
5:45-6:00	7	13	7	4	8	3	2	6	3	1	2	1	7	16	8	9.8	469
6:00-6:15	6	12	7	4	7	3	2	5	2	1	1	1	6	13	6	7.7	391
From Gebya-Ethiopia hotel approach leg entry vehicle & pedestrian traffics																	
5:00-5:15	7	23	8	47	128	38	11	35	13	1	2	1	37	109	35	1.7	144
5:15-5:30	14	47	17	41	112	33	12	40	15	1	2	1	38	113	38	1.6	158
5:30-5:45	12	41	15	38	103	31	10	35	13	1	2	1	34	102	34	1.8	171
5:45-6:00	10	34	12	38	103	31	14	48	18	1	3	1	38	114	38	2.6	197
6:00-6:15	10	32	12	36	99	30	10	34	13	1	2	1	32	96	32	1.9	149
From Mesgid -Zmamnesh building approach leg entry vehicle & pedestrian traffics																	
5:00-5:15	9	22	10	7	14	6	4	7	4	1	1	0	11	22	11	5.5	192
5:15-5:30	11	27	13	8	16	7	7	14	8	0	1	0	15	32	16	2.3	261
5:30-5:45	12	29	14	8	15	6	4	9	5	1	1	1	13	27	13	3.8	286
5:45-6:00	8	20	10	6	12	5	5	10	6	1	1	1	11	24	12	4.2	241
6:00-6:15	8	19	9	7	13	5	5	10	6	0	1	0	11	23	11	2.2	202

Table A-3 Gorgis Roundabout vehicle & pedestrian traffics from each approach leg

Time (PM)	Bicycles and motor bicycles (A)			Bajaj (B)			Light vehicles (C)			Heavy vehicles (D)			Totoal vehicles (pcu) (A/3+B/2+C+D)			Percent age of Heavy vehicles	Pedestri ans
To	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R		
From Hospital-Abyssinia bank approach leg																	
5:00-5:15		20	46		46	106		23	53		2	4		54	126	3.3	248
5:15-5:30		25	57		41	95		21	49		1	2		51	118	1.8	259
5:30-5:45		32	74		54	125		21	48		3	6		61	142	4.4	289
5:45-6:00		25	59		38	88		21	48		2	4		50	116	3.6	300
6:00-6:15		19	44		38	90		17	39		1	2		43	100	2.1	245
From Poli-Georgis approach leg																	
5:00-5:15	43	19		51	22		75	32		11	5		126	54		9	278
5:15-5:30	60	26		48	20		73	31		10	4		126	54		8	374
5:30-5:45	30	13		36	16		80	34		6	3		114	49		6	388
5:45-6:00	53	23		39	17		78	33		7	3		122	52		6	360
6:00-6:15	25	11		39	17		72	31		8	3		107	46		7	269
From Gebya-protection house approach leg																	
5:00-5:15	28		25	67		65	24		25	1		1	67		67	1.5	176
5:15-5:30	27		25	88		84	24		27	2		2	79		79	2.5	204
5:30-5:45	32		30	79		75	24		27	2		1	76		76	2.0	220
5:45-6:00	27		25	87		84	28		31	1		1	82		82	1.2	210
6:00-6:15	31		28	75		72	31		34	1		1	80		80	1.2	171

Table A-4 Kucht junction vehicle & pedestrian traffics from each approach leg

Time (PM)	Bicycles and motor bicycles (A)			Bajaj (B)			Light vehicles (C)			Heavy vehicles (D)			Totoal vehicles (pcu) (A/3+B/2+C+D)			Percent age of Heavy vehicles	Pedestri ans
To	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R		
From Dashn bank-kucht approach leg entry vehicle & pedestrian traffics																	
5:00-5:15		82			144			33			2			134		1.5	179
5:15-5:30		65			179			26			7			144		4.9	216
5:30-5:45		34			162			40			5			137		3.6	225
5:45-6:00		69			142			47			5			146		3.4	208
6:00-6:15		69			142			47			5			146		3.4	168
From Azwa-commercial bank approach leg entry vehicle & pedestrian traffics																	
5:00-5:15		76			105			63			10			151		6.6	277
5:15-5:30		60			74			60			13			130		10.0	339
5:30-5:45		69			105			80			11			167		6.6	324
5:45-6:00		60			115			68			11			157		7.0	474
6:00-6:15		63			88			68			7			140		5.0	268

Appendix B-Geometric data

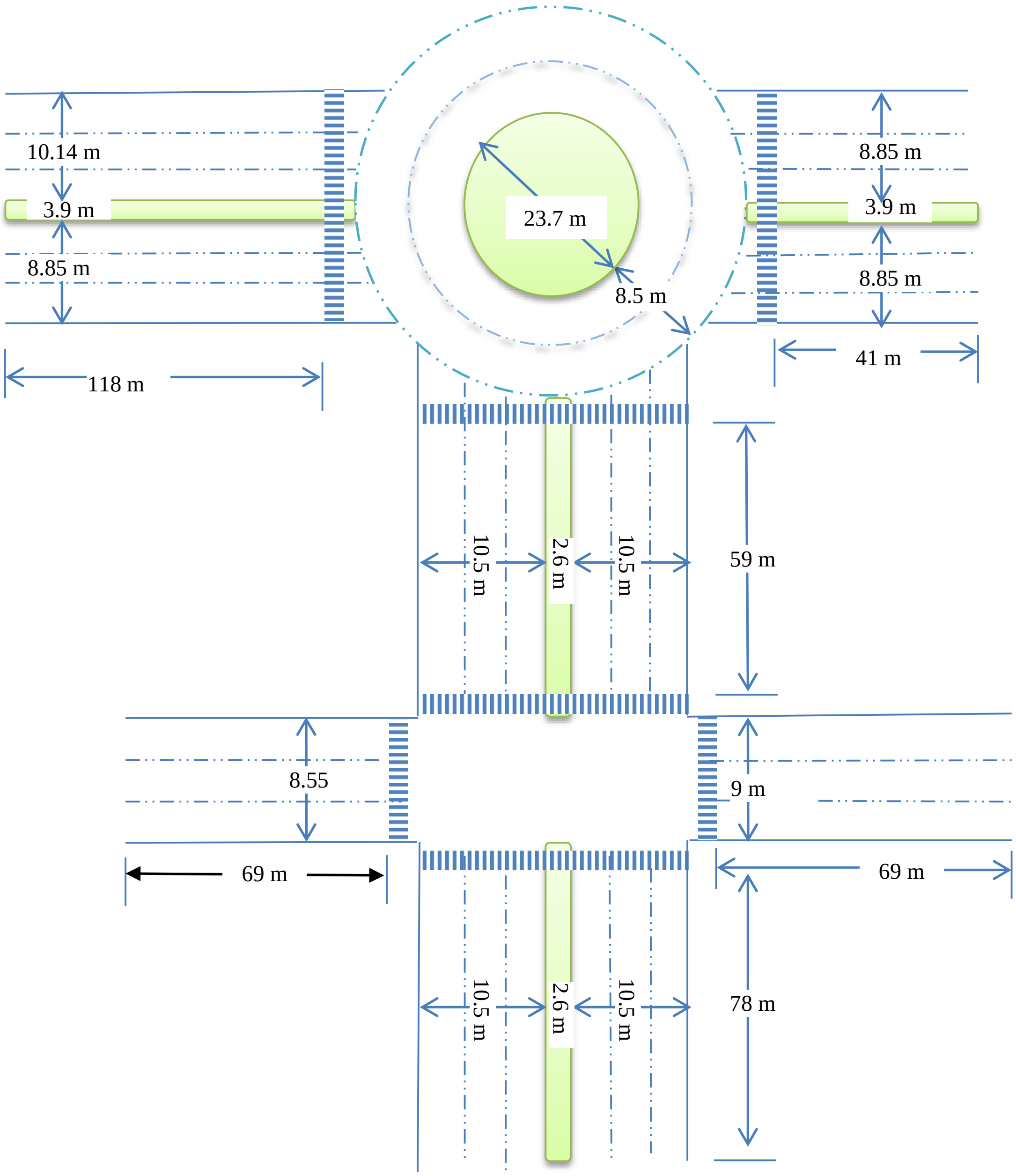


Figure B.1 Giorgis roundabout and intersection at Zmamnesh building

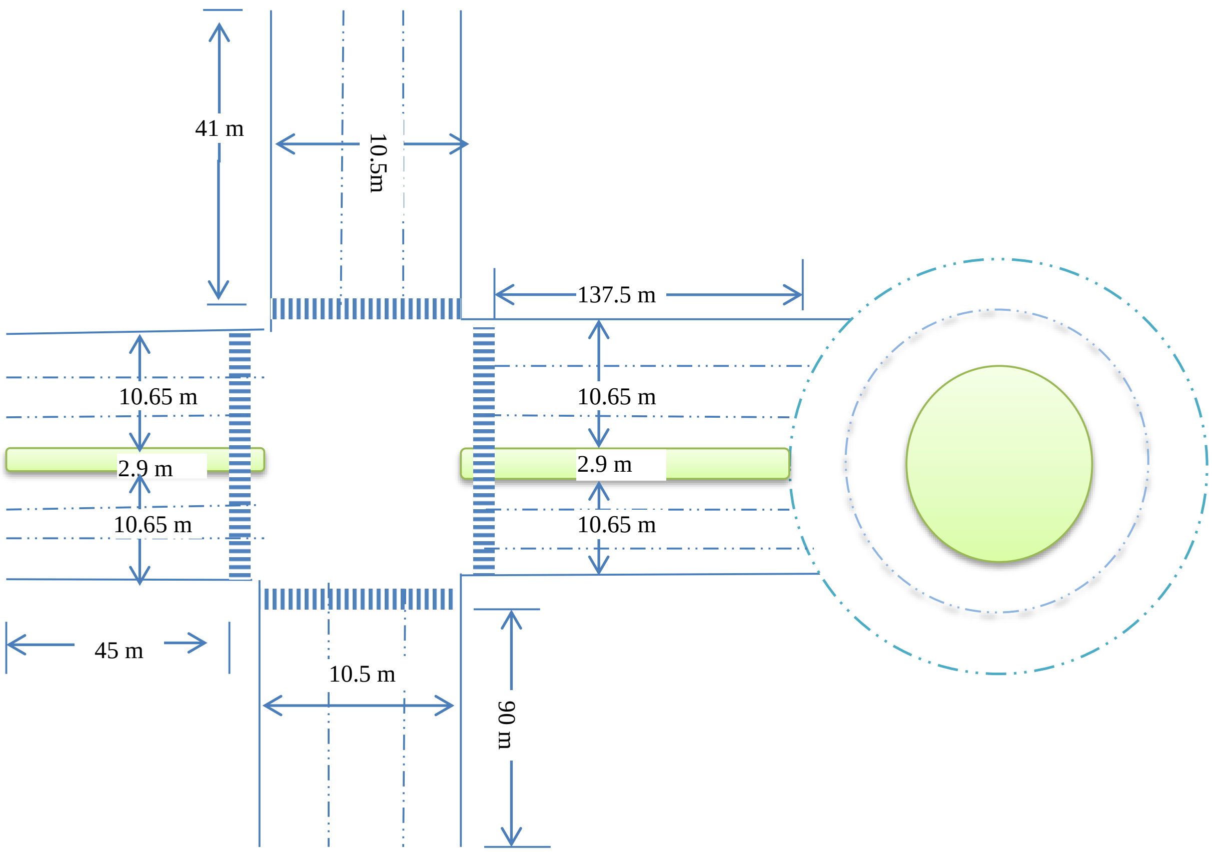


Figure B.2 Intersection at kucht building

Appendix C-Papyrus roundabout performance analysis

Layout of the intersection

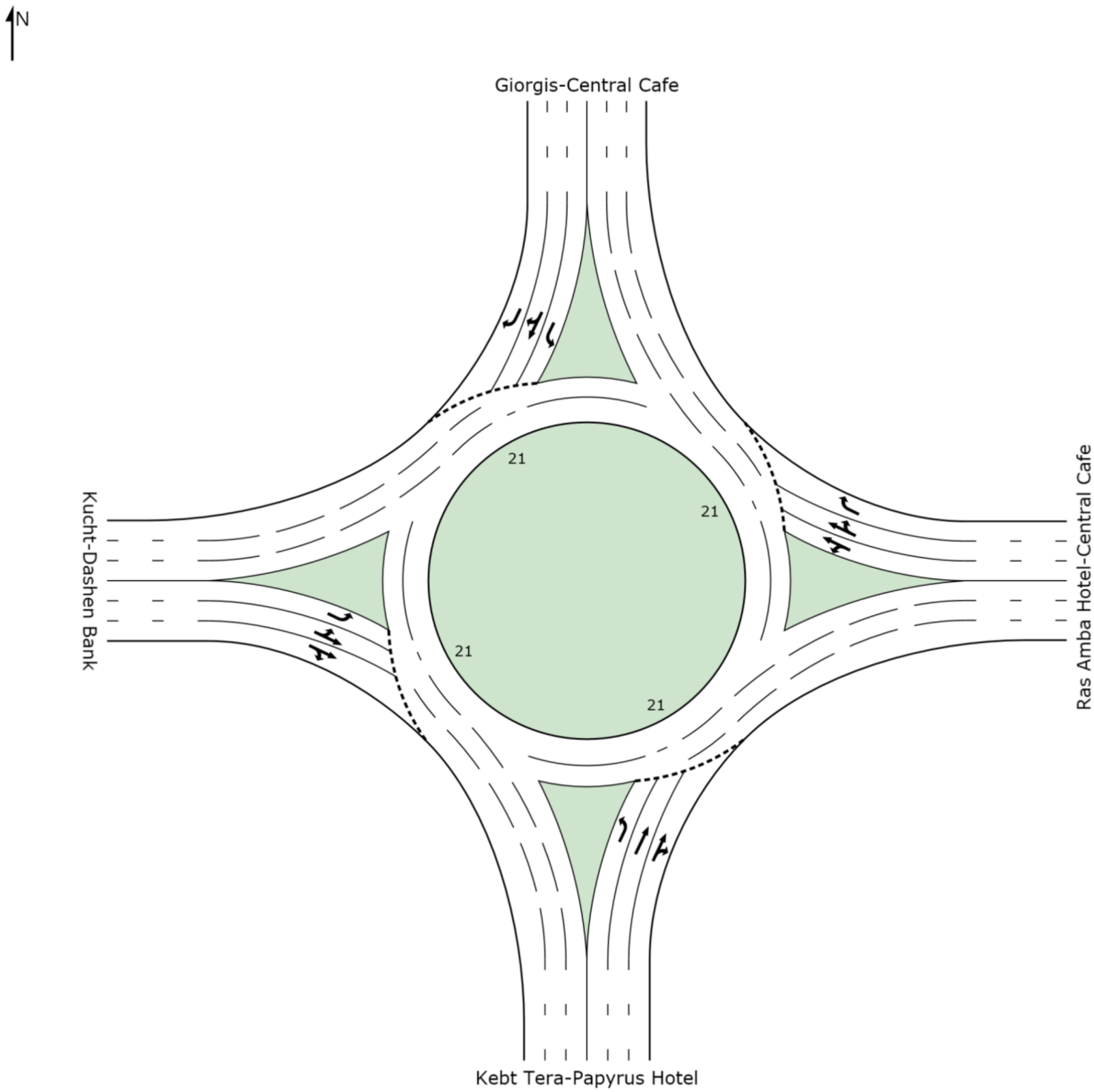


Figure C.1 Layout of papyrus roundabout

Movement IDs

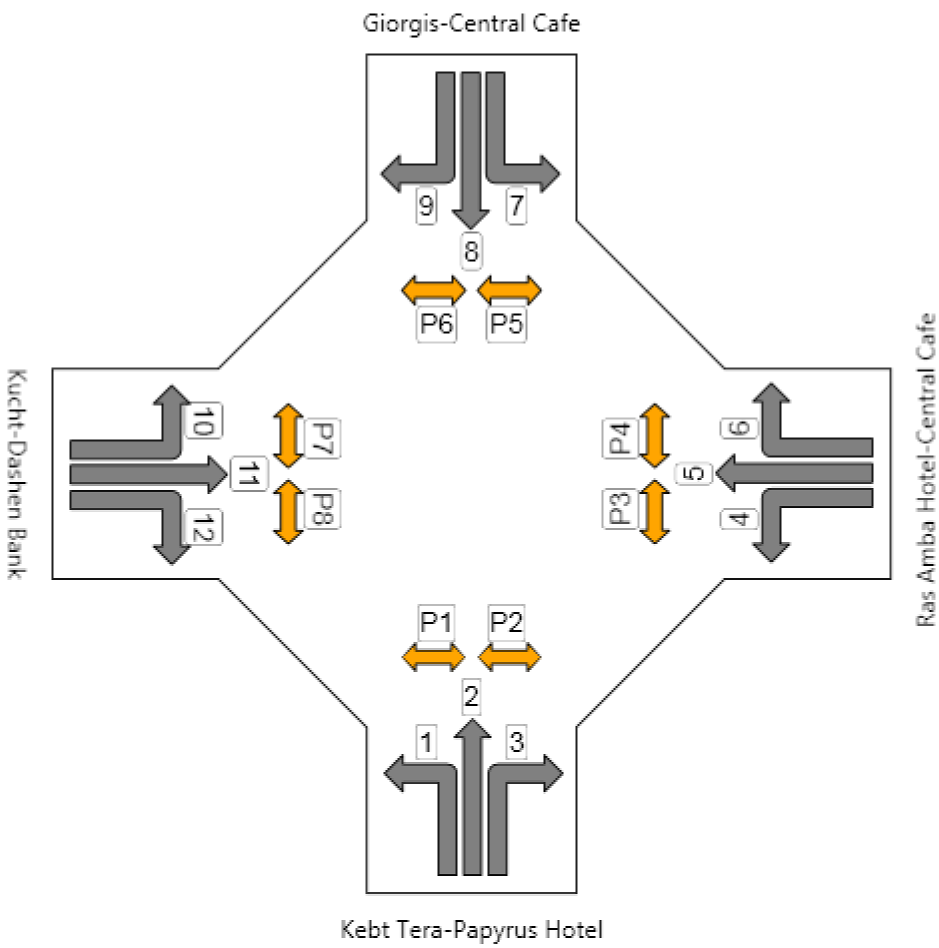


Figure C.2 Papyrus roundabout movement IDs

Table C-1 Summarized input data for analysis of papyrus roundabout

Papyrus Roundabout

Intersection Parameters	
Title	Papyrus Roundabout
Intersection ID	1
Unit Time (for volumes)	60 minutes
Peak Flow Period (for performance)	15 minutes

Geometry - Approach Data

Location	Name	Type	No. of App. Lanes	No. of Exit Lanes	Median Width m	Extra Bunching %
South	Kebt Tera-Papyrus Hotel	Two-way	3	3	3.30	0.0
East	Ras Amba Hotel-Central Cafe	Two-way	3	3	2.90	0.0
North	Giorgis-Central Cafe	Two-way	3	3	2.70	0.0
West	Kucht-Dashen Bank	Two-way	3	3	2.90	0.0

Geometry - Roundabout Data

Location	Name	Island Diameter m	Circ. Width m	Circ. Lanes	Entry Radius m	Entry Angle degrees	Env. Factor	Entry/Circ. Flow Adjust.
South	Kebt Tera-Papyrus Hotel	21.00	12.55	2	20.0	30.0	1.3000	Medium
East	Ras Amba Hotel-Central Cafe	21.00	12.55	2	20.0	30.0	1.3000	Medium
North	Giorgis-Central Cafe	21.00	12.55	2	20.0	30.0	1.3000	Medium
West	Kucht-Dashen Bank	21.00	12.55	2	20.0	30.0	1.3000	Medium

Geometry - Approach Lane Data

Lane Number	Lane Type	Lane Discip.	Basic Satn Flow tcu/h
South	Kebt Tera-Papyrus Hotel		
App. Lane 1	Normal	L	1750
App. Lane 2	Normal	T	1750
App. Lane 3	Normal	TR	1750
East	Ras Amba Hotel-Central Cafe		
App. Lane 1	Normal	LT	1750
App. Lane 2	Normal	TR	1750
App. Lane 3	Normal	R	1750
North	Giorgis-Central Cafe		
App. Lane 1	Normal	L	1750
App. Lane 2	Normal	TR	1750
App. Lane 3	Normal	R	1750
West	Kucht-Dashen Bank		
App. Lane 1	Normal	L	1750
App. Lane 2	Normal	LT	1750
App. Lane 3	Normal	TR	1750

Geometry - Approach & Exit Lane Data				
Lane Number	Lane Width m	Lane Length m	Grade %	SL Type
South Kebt Tera-Papyrus Hotel				
App. Lane 1	3.70	110.0	0.00	–
App. Lane 2	3.70	110.0	0.00	–
App. Lane 3	3.70	110.0	0.00	–
Exit Lane 1	3.70	110.0	0.00	–
Exit Lane 2	3.70	110.0	0.00	–
Exit Lane 3	3.70	110.0	0.00	–
East Ras Amba Hotel-Central Cafe				
App. Lane 1	3.50	61.0	0.00	–
App. Lane 2	3.50	61.0	0.00	–
App. Lane 3	3.50	61.0	0.00	–
Exit Lane 1	3.50	61.0	0.00	–
Exit Lane 2	3.50	61.0	0.00	–
Exit Lane 3	3.50	61.0	0.00	–
North Giorgis-Central Cafe				
App. Lane 1	3.25	82.0	0.00	–
App. Lane 2	3.25	82.0	0.00	–
App. Lane 3	3.25	82.0	0.00	–
Exit Lane 1	3.25	82.0	0.00	–
Exit Lane 2	3.25	82.0	0.00	–
Exit Lane 3	3.25	82.0	0.00	–
West Kucht-Dashen Bank				
App. Lane 1	3.55	137.5	0.00	–
App. Lane 2	3.55	137.5	0.00	–
App. Lane 3	3.55	137.5	0.00	–
Exit Lane 1	3.55	137.5	0.00	–
Exit Lane 2	3.55	137.5	0.00	–
Exit Lane 3	3.55	137.5	0.00	–

Lanes are numbered from left to right in the direction of travel.

Geometry - Movement Definitions

To Approach	Movement Banned	Turn Desig.
From: South	Kebt Tera-Papyrus Hotel	
South	Yes	–
West	No	L
North	No	T
East	No	R
From: East	Ras Amba Hotel-Central Cafe	
East	Yes	–
South	No	L
West	No	T
North	No	R
From: North	Giorgis-Central Cafe	
North	Yes	–
East	No	L
South	No	T
West	No	R
From: West	Kucht-Dashen Bank	
West	Yes	–
North	No	L
East	No	T
South	No	R

Volumes

To Approach	Total veh	HV %	Peak Flow Factor %	Vehicle Occupancy pers/veh	Flow Scale %	Growth Rate %/year
From: South	Kebt Tera-Papyrus Hotel					
West	114.0	7.70	95.0	5.00	100.00	7.00
North	130.0	7.70	95.0	5.00	100.00	7.00
East	81.0	7.70	95.0	5.00	100.00	7.00
From: East	Ras Amba Hotel-Central Cafe					
South	114.0	7.50	95.0	5.00	100.00	7.00
West	200.0	7.50	95.0	5.00	100.00	7.00
North	257.0	7.50	95.0	5.00	100.00	7.00
From: North	Giorgis-Central Cafe					
East	268.0	2.10	95.0	5.00	100.00	7.00
South	153.0	2.10	95.0	5.00	100.00	7.00
West	345.0	2.10	95.0	5.00	100.00	7.00
From: West	Kucht-Dashen Bank					
North	304.0	8.90	95.0	5.00	100.00	7.00
East	236.0	8.90	95.0	5.00	100.00	7.00
South	135.0	8.90	95.0	5.00	100.00	7.00

Path Data

To Approach		App. Cruise Speed km/h	Exit Cruise Speed km/h	App. Trav. Distance m
From: South	Kebt Tera-Papyrus Hotel			
West		40.0	40.0	110.0
North		40.0	40.0	110.0
East		40.0	40.0	110.0
From: East	Ras Amba Hotel-Central Cafe			
South		40.0	40.0	61.0
West		40.0	40.0	61.0
North		40.0	40.0	61.0
From: North	Giorgis-Central Cafe			
East		40.0	40.0	82.0
South		40.0	40.0	82.0
West		40.0	40.0	82.0
From: West	Kucht-Dashen Bank			
North		40.0	40.0	137.5
East		40.0	40.0	137.5
South		40.0	40.0	137.5

Movement Data - General

Turn	Mov. ID	Queue Space LV m	Queue Space HV m	Vehicle Length LV m	Vehicle Length HV m	HVE	P.Deg. Satn	Movement Type	Movement Control
South	Kebt Tera-Papyrus Hotel								
L	1	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
T	2	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
R	3	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
East	Ras Amba Hotel-Central Cafe								
L	4	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
T	5	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
R	6	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
North	Giorgis-Central Cafe								
L	7	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
T	8	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
R	9	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
West	Kucht-Dashen Bank								
L	10	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
T	11	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
R	12	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–

Movement Type and Control parameters are set automaticly from Approach Control and Lane Type data in the Geometry dialog.

Pedestrians

Mov. ID	Volume ped	Peak Flow %	Flow Scale %	Growth Rate %/year	Crossing Distance m	App. Trav. Distance m	Downst. Distance m	Walking Speed m/sec	Queue Space m	P.Deg. Satn
South	Kebt Tera-Papyrus Hotel									
P1	1472.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–
P2	1472.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–
East	Ras Amba Hotel-Central Cafe									
P3	2003.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–
P4	2003.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–
North	Giorgis-Central Cafe									
P5	3236.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–
P6	3236.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–
West	Kucht-Dashen Bank									
P7	2284.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–
P8	2284.0	95.0	100.00	2.00	–	10.0	10.0	1.40	1.00	–

Model Settings - Options	
General Options	
Level of Service Method	Delay & v/c (HCM 2010)
Level of Service Target	LOS D
Performance Measure	Delay
Percentile Queue	95 %
Hours per Year	3160 h
Gap Acceptance	
HV Method for Gap-Acceptance	Include HV Effect if above 5 per cent
Gap-Acceptance Capacity	SIDRA Standard (Akçelik M3D)
HCM Delay Formula	Yes
Downstream Short Lane Model	
Minimum Downstream Utilisation Ratio	20 %
Minimum Downstream Distance	30 m
Distance for Full Lane Utilisation	200 m
Calibration Parameter	1.2

Model Settings - Roundabouts	
Roundabout Model Options	
Capacity Model	SIDRA Standard
LOS Method	Same as Signalised Intersections
US HCM 2010 Roundabout Model	
Include Origin-Destination Pattern Effects	–
Factor for Parameter A	–
Factor for Parameter B	–
Other Roundabout Models	
FHWA 2000	No
Use Urban Compact Roundabout	–
HCM 2000	No
NAASRA 1986	No

Site Properties	
Site (Intersection) Type	Roundabout
Model Name	Standard Right
Drive Rule	Right-hand side of the road
New Zealand Rule	No
HCM Version	No
Units	Metric

Table C-2 Papyrus roundabout performance analysis result (output)

Lane Use and Performance

	Demand Flows L veh/h	Demand Flows T veh/h	Demand Flows R veh/h	Total veh/h	HV %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Lane Length m
South: Kebt Tera-Papyrus Hotel													
Lane 1	120	0	0	120	7.7	365	0.329	100	16.3	LOS B	0.7	5.2	110
Lane 2	0	111	0	111	7.7	341	0.325	100	17.2	LOS B	0.7	5.1	110
Lane 3	0	26	85	111	7.7	341	0.325	100	17.2	LOS B	0.7	5.1	110
Approach	120	137	85	342	7.7		0.329		16.9	LOS B	0.7	5.2	
East: Ras Amba Hotel-Central Cafe													
Lane 1	120	77	0	197	7.5	219	0.901	100	83.4	LOS F	2.4	17.7	61
Lane 2	0	134	74	207	7.5	230	0.901	100	80.5	LOS F	2.4	17.9	61
Lane 3	0	0	197	197	7.5	219	0.901	100	83.4	LOS F	2.4	17.7	61
Approach	120	211	271	601	7.5		0.901		82.4	LOS F	2.4	17.9	
North: Giorgis-Central Cafe													
Lane 1	282	0	0	282	2.1	150[2]	1.881	100	471.2	LOS F	45.3	322.7	82
Lane 2	0	161	101	262	2.1	150[2]	1.747	100	414.3	LOS F	38.9	277.2	82
Lane 3	0	0	262	262	2.1	150[2]	1.747	100	414.3	LOS F	38.9	277.2	82
Approach	282	161	363	806	2.1		1.881		434.2	LOS F	45.3	322.7	
West: Kucht-Dashen Bank													
Lane 1	234	0	0	234	8.9	172	1.366	100	248.8	LOS F	23.8	179.3	138
Lane 2	86	156	0	242	8.9	177	1.366	100	246.8	LOS F	24.5	184.4	138
Lane 3	0	92	142	234	8.9	172	1.366	100	248.8	LOS F	23.8	179.3	138
Approach	320	248	142	711	8.9		1.366		248.1	LOS F	24.5	184.4	
Intersection				2460	6.2		1.881		236.5	LOS F	45.3	322.7	

Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Roundabout LOS Method: Same as Signalised Intersections.
Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.
LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).
Roundabout Capacity Model: SIDRA Standard.
HCM Delay Model used. Geometric Delay not included.

2 Minimum Capacity

Intersection Performance - Hourly Values		
Performance Measure	**Vehicles**	**Persons**
Demand Flows (Total)	2460 veh/h	31234 pers/h
Percent Heavy Vehicles	6.2 %	
Degree of Saturation	1.881	
Practical Spare Capacity	-54.8 %	
Effective Intersection Capacity	1308 veh/h	
Control Delay (Total)	161.58 veh-h/h	807.90 pers-h/h
Control Delay (Average)	236.5 sec	93.1 sec
Control Delay (Worst Lane)	471.2 sec	
Control Delay (Worst Movement)	471.2 sec	471.2 sec
Geometric Delay (Average)	3.3 sec	
Stop-Line Delay (Average)	236.5 sec	
Intersection Level of Service (LOS)	LOS F	
95% Back of Queue - Vehicles (Worst Lane)	45.3 veh	
95% Back of Queue - Distance (Worst Lane)	322.7 m	
Total Effective Stops	11187 veh/h	55933 pers/h
Effective Stop Rate	4.55 per veh	1.79 per pers
Proportion Queued	0.84	0.33
Performance Index	316.0	316.0
Travel Distance (Total)	400.7 veh-km/h	2003.7 pers-km/h
Travel Distance (Average)	163 m	64 m
Travel Time (Total)	173.9 veh-h/h	869.3 pers-h/h
Travel Time (Average)	254.4 sec	100.2 sec
Travel Speed	2.3 km/h	2.3 km/h

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Signalised Intersections.

Intersection LOS value for Vehicles is based on average delay for all vehicle movements.

Roundabout Capacity Model: SIDRA Standard.

HCM Delay Model used. Geometric Delay not included.

Intersection Performance - Annual Values		
Performance Measure	**Vehicles**	**Persons**
Demand Flows (Total)	7,773,600 veh/y	98,699,440 pers/y
Delay	510,590 veh-h/y	2,552,949 pers-h/y
Effective Stops	35,349,660 veh/y	176,748,300 pers/y
Travel Distance	1,266,341 veh-km/y	6,331,706 pers-km/y
Travel Time	549,399 veh-h/y	2,746,994 pers-h/y

Appendix D-Analysis of traffic signal near Giorgis

Layout of the intersection

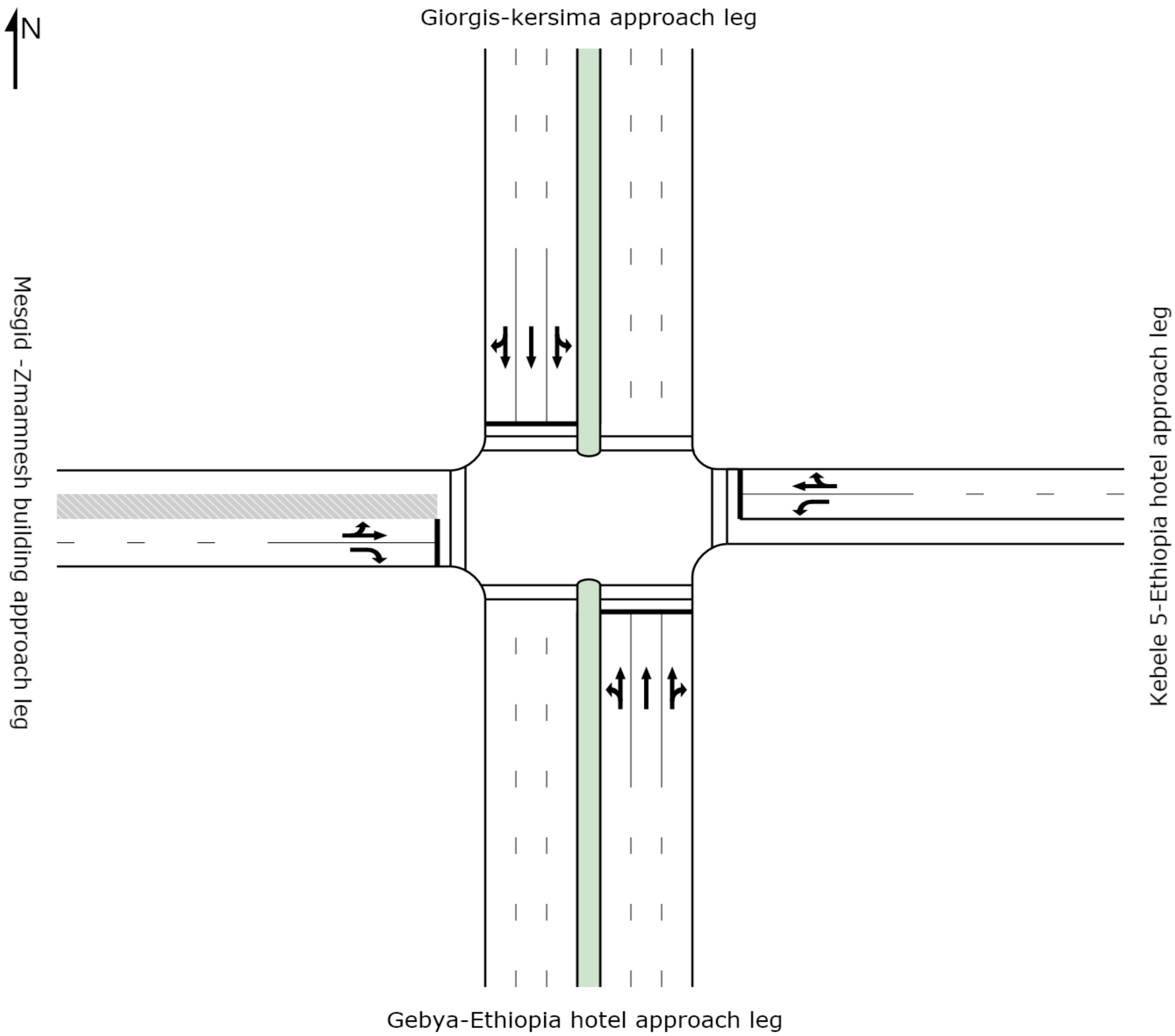


Figure D.1 Layout of traffic signal near giorgis

Movement IDs

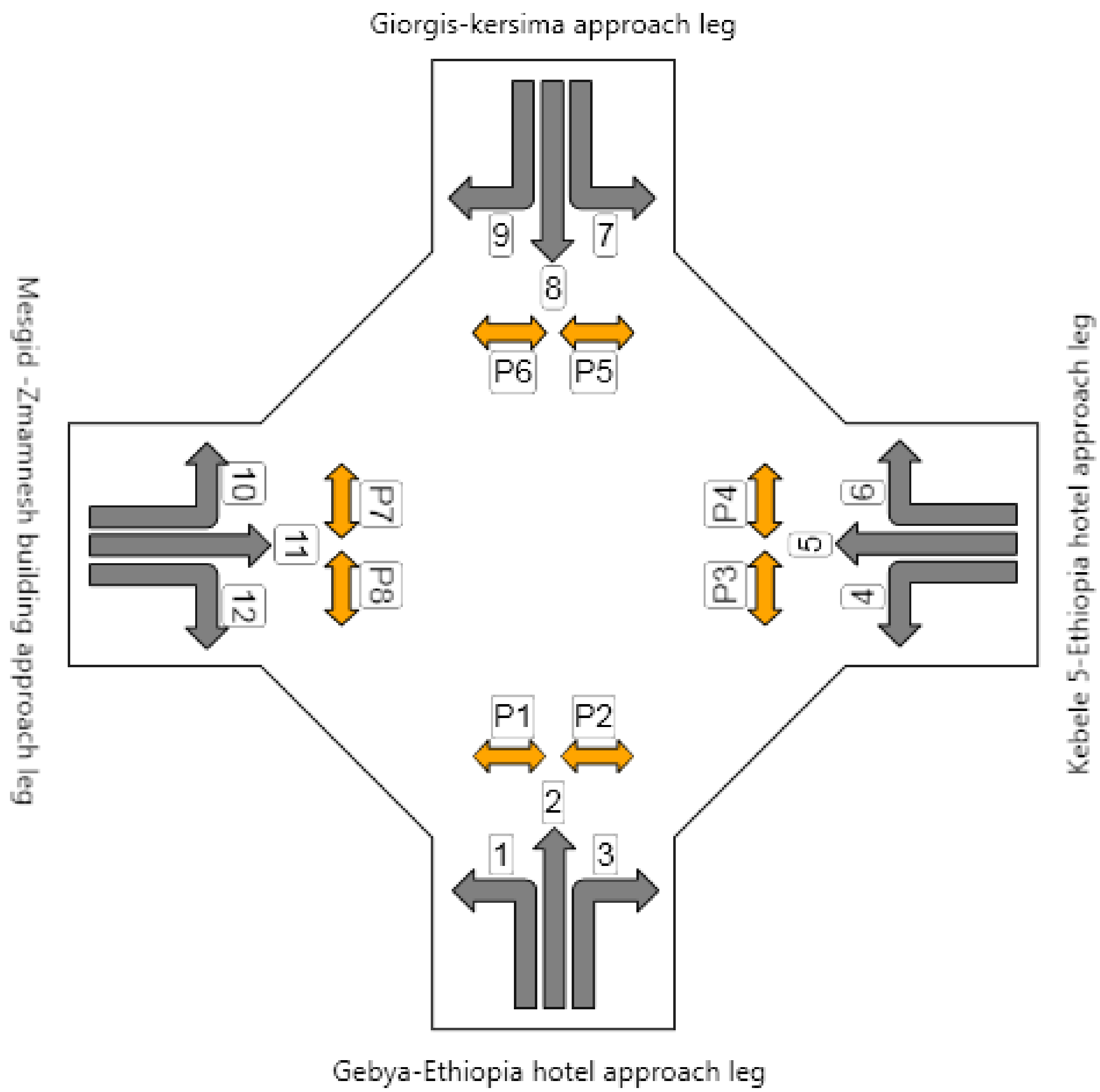


Figure D.2 Traffic signal near giorgis movement IDs

Table D-1 Summarized input data for analysis of traffic signal near Giorgis

Intersection Parameters	
Title	Traffic signal near giorgis
Intersection ID	2
Unit Time (for volumes)	60 minutes
Peak Flow Period (for performance)	15 minutes
Signal Analysis Method	Fixed Time

Geometry - Approach Data						
Location	Name	Type	No. of App. Lanes	No. of Exit Lanes	Median Width m	Extra Bunching %
South	Gebya-Ethiopia hotel approach leg	Two-way	3	3	2.60	0.0
East	Kebele 5-Ethiopia hotel approach leg	Two-way	2	1	-	0.0
North	Giorgis-kersima approach leg	Two-way	3	3	2.60	0.0
West	Mesgid -Zmamnesh building approach leg	Two-way	2	1	-	0.0

Geometry - Approach Lane Data - Signalised			
Lane Number	Lane Type	Lane Discip.	Basic Satn Flow tcu/h
South	Gebya-Ethiopia hotel approach leg		
App. Lane 1	Normal	LT	1750
App. Lane 2	Normal	T	1750
App. Lane 3	Normal	TR	1750
East	Kebele 5-Ethiopia hotel approach leg		
App. Lane 1	Normal	L	1750
App. Lane 2	Normal	TR	1750
North	Giorgis-kersima approach leg		
App. Lane 1	Normal	LT	1750
App. Lane 2	Normal	T	1750
App. Lane 3	Normal	TR	1750
West	Mesgid -Zmamnesh building approach leg		
App. Lane 1	Normal	LT	1750
App. Lane 2	Normal	R	1750

Geometry - Approach & Exit Lane Data					
Lane Number		Lane Width m	Lane Length m	Grade %	SL Type
South	Gebya-Ethiopia hotel approach leg				
App. Lane 1		3.50	78.0	0.00	–
App. Lane 2		3.50	78.0	0.00	–
App. Lane 3		3.50	78.0	0.00	–
Exit Lane 1		3.50	78.0	0.00	–
Exit Lane 2		3.50	78.0	0.00	–
Exit Lane 3		3.50	78.0	0.00	–
East	Kebele 5-Ethiopia hotel approach leg				
App. Lane 1		3.00	69.0	0.00	–
App. Lane 2		3.00	69.0	0.00	–
Exit Lane 1		3.00	69.0	0.00	–
North	Giorgis-kersima approach leg				
App. Lane 1		3.50	59.0	0.00	–
App. Lane 2		3.50	59.0	0.00	–
App. Lane 3		3.50	59.0	0.00	–
Exit Lane 1		3.50	59.0	0.00	–
Exit Lane 2		3.50	59.0	0.00	–
Exit Lane 3		3.50	59.0	0.00	–
West	Mesgid -Zmamnesh building approach leg				
App. Lane 1		2.85	69.0	0.00	–
App. Lane 2		2.85	69.0	0.00	–
Exit Lane 1		2.85	69.0	0.00	–

Lanes are numbered from left to right in the direction of travel.

Geometry - Movement Definitions		
To Approach	Movement Banned	Turn Desig.
From: South	Gebya-Ethiopia hotel approach leg	
South	Yes	–
West	No	L
North	No	T
East	No	R
From: East	Kebele 5-Ethiopia hotel approach leg	
East	Yes	–
South	No	L
West	No	T
North	No	R
From: North	Giorgis-kersima approach leg	
North	Yes	–
East	No	L
South	No	T
West	No	R
From: West	Mesgid -Zmamnesh building approach leg	
West	Yes	–
North	No	L
East	No	T
South	No	R

Volumes						
To Approach	Total veh	HV %	Peak Flow Factor %	Vehicle Occupancy pers/veh	Flow Scale %	Growth Rate %/year
From: South	Gebya-Ethiopia hotel approach leg					
West	146.0	1.90	95.0	5.00	100.00	7.00
North	437.0	1.90	95.0	5.00	100.00	7.00
East	146.0	1.90	95.0	5.00	100.00	7.00
From: East	Kebele 5-Ethiopia hotel approach leg					
South	30.0	9.00	95.0	5.00	100.00	7.00
West	61.0	9.00	95.0	5.00	100.00	7.00
North	30.0	9.00	95.0	5.00	100.00	7.00
From: North	Giorgis-kersima approach leg					
East	160.0	2.50	95.0	5.00	100.00	7.00
South	481.0	2.50	95.0	5.00	100.00	7.00
West	160.0	2.50	95.0	5.00	100.00	7.00
From: West	Mesgid -Zmamnesh building approach leg					
North	52.0	3.80	95.0	5.00	100.00	7.00
East	104.0	3.80	95.0	5.00	100.00	7.00
South	52.0	3.80	95.0	5.00	100.00	7.00

Path Data

To Approach	App. Cruise Speed km/h	Exit Cruise Speed km/h	App. Trav. Distance m
From: South	Gebya-Ethiopia hotel approach leg		
West	40.0	40.0	78.0
North	40.0	40.0	78.0
East	40.0	40.0	78.0
From: East	Kebele 5-Ethiopia hotel approach leg		
South	40.0	40.0	69.0
West	40.0	40.0	69.0
North	40.0	40.0	69.0
From: North	Giorgis-kersima approach leg		
East	40.0	40.0	59.0
South	40.0	40.0	59.0
West	40.0	40.0	59.0
From: West	Mesgid -Zmamnesh building approach leg		
North	40.0	40.0	69.0
East	40.0	40.0	69.0
South	40.0	40.0	69.0

Movement Data - General

Turn	Mov. ID	Queue Space LV m	Queue Space HV m	Vehicle Length LV m	Vehicle Length HV m	HVE	P.Deg. Satn	Movement Type	Movement Control
South	Gebya-Ethiopia hotel approach leg								
L	1	7.00	13.00	4.50	10.00	2.00	–	Normal	–
T	2	7.00	13.00	4.50	10.00	2.00	–	Normal	–
R	3	7.00	13.00	4.50	10.00	2.00	–	Normal	–
East	Kebele 5-Ethiopia hotel approach leg								
L	4	7.00	13.00	4.50	10.00	2.00	–	Normal	–
T	5	7.00	13.00	4.50	10.00	2.00	–	Normal	–
R	6	7.00	13.00	4.50	10.00	2.00	–	Normal	–
North	Giorgis-kersima approach leg								
L	7	7.00	13.00	4.50	10.00	2.00	–	Normal	–
T	8	7.00	13.00	4.50	10.00	2.00	–	Normal	–
R	9	7.00	13.00	4.50	10.00	2.00	–	Normal	–
West	Mesgid -Zmamnesh building approach leg								
L	10	7.00	13.00	4.50	10.00	2.00	–	Normal	–
T	11	7.00	13.00	4.50	10.00	2.00	–	Normal	–
R	12	7.00	13.00	4.50	10.00	2.00	–	Normal	–

Movement Data - Signalised

Turn	Mov. ID	Signal Coord. Type	PG %	Non-Actuated	Turn On Red	Turn Adjustment Type	Radius m	Pedestrian Effect Method	St. Loss sec
South	Gebya-Ethiopia hotel approach leg								
L	1	3	–	–	–	Normal	–	St. Loss	0
T	2	3	–	–	–	Normal	–	–	–
R	3	3	–	–	No	Normal	–	St. Loss	0
East	Kebele 5-Ethiopia hotel approach leg								
L	4	3	–	–	–	Normal	–	St. Loss	0
T	5	3	–	–	–	Normal	–	–	–
R	6	3	–	–	No	Normal	–	St. Loss	0
North	Giorgis-kersima approach leg								
L	7	3	–	–	–	Normal	–	St. Loss	0
T	8	3	–	–	–	Normal	–	–	–
R	9	3	–	–	No	Normal	–	St. Loss	0
West	Mesgid -Zmamnesh building approach leg								
L	10	3	–	–	–	Normal	–	St. Loss	0
T	11	3	–	–	–	Normal	–	–	–
R	12	3	–	–	No	Normal	–	St. Loss	0

Priorities

Opposed Movement	Opposing Movements South	South East	East	North East	North	North West	West	South West
South	Gebya-Ethiopia hotel approach leg							
L	–	–	–	–	–	–	P7	–
R	–	–	P3	–	–	–	–	–
East	Kebele 5-Ethiopia hotel approach leg							
L	P1	–	–	–	–	–	T	–
R	–	–	–	–	P5	–	–	–
North	Giorgis-kersima approach leg							
L	–	–	P3	–	–	–	–	–
R	–	–	–	–	–	–	P7	–
West	Mesgid -Zmamnesh building approach leg							
L	–	–	–	–	P5	–	–	–
R	P1	–	–	–	–	–	–	–

Gap Acceptance

Movement		Critical Gap sec	Follow-up Headway sec	End Departures veh	Exiting Flow Effect %
East	Kebele 5-Ethiopia hotel approach leg				
	L	4.000	2.000	2.20	0

Pedestrians

Mov. ID	Volume ped	Peak Flow %	Flow Scale %	Growth Rate %/year	Crossing Distance m	App. Trav. Distance m	Downst. Distance m	Walking Speed m/sec	Queue Space m	P.Deg. Satn	Satn Flow ped/h
South	Gebya-Ethiopia hotel approach leg										
P1	670.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000
P2	670.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000
East	Kebele 5-Ethiopia hotel approach leg										
P3	1797.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000
P4	1797.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000
North	Giorgis-kersima approach leg										
P5	494.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000
P6	494.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000
West	Mesgid -Zmamnesh building approach leg										
P7	980.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000
P8	980.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-	12000

Phasing Data

Current Sequence: Split Phasing

Name	Phase Time sec	Yellow Time sec	All-Red Time sec	Dummy Movement Parameters Specified	Min Green sec	Max Green sec	Movements Running in Phase S	SE	E	NE	N	NW	W	SW
A	30	4	2	No	-	-	LTR	-	P3,P4	-	-	-	-	-
B	30	4	2	No	-	-	-	-	-	-	LTR	-	P7,P8	-
C	30	4	2	No	-	-	P1,P2	-	-	-	-	-	LTR	-
D	30	4	2	No	-	-	-	-	LTR	-	P5,P6	-	-	-

Sequence Data	
Current Sequence	Split Phasing
Cycle Time Option	User-Given Phase Times
Green Split Option	
Green Split Priority	No

Movement Timing Data – Pedestrians

Current Sequence: Split Phasing

Mov. ID	Min Green sec	Max Green sec	Crossing Speed m/sec	Min Walk Time sec	Min Clearance Time sec	Clearance Time Overlap sec	Start Loss sec	End Gain sec
South	Gebya-Ethiopia hotel approach leg							
P1	–	–	1.40	5	5	2	2	3
P2	–	–	1.40	5	5	2	2	3
East	Kebele 5-Ethiopia hotel approach leg							
P3	–	–	1.40	5	5	2	2	3
P4	–	–	1.40	5	5	2	2	3
North	Giorgis-kersima approach leg							
P5	–	–	1.40	5	5	2	2	3
P6	–	–	1.40	5	5	2	2	3
West	Mesgid -Zmamnesh building approach leg							
P7	–	–	1.40	5	5	2	2	3
P8	–	–	1.40	5	5	2	2	3

Movement Timing Data – Vehicles

Current Sequence: Split Phasing

Turn	Mov. ID	Start Loss sec	End Gain sec	Min Green sec	Max Green sec
South	Gebya-Ethiopia hotel approach leg				
L	1	3	3	–	–
T	2	3	3	–	–
R	3	3	3	–	–
East	Kebele 5-Ethiopia hotel approach leg				
L	4	3	3	–	–
T	5	3	3	–	–
R	6	3	3	–	–
North	Giorgis-kersima approach leg				
L	7	3	3	–	–
T	8	3	3	–	–
R	9	3	3	–	–
West	Mesgid -Zmamnesh building approach leg				
L	10	3	3	–	–
T	11	3	3	–	–
R	12	3	3	–	–

Model Settings - Options	
General Options	
Level of Service Method	Delay & v/c (HCM 2010)
Level of Service Target	LOS D
Performance Measure	Delay
Percentile Queue	95 %
Hours per Year	3160 h
Gap Acceptance	
HV Method for Gap-Acceptance	Include HV Effect if above 5 per cent
Gap-Acceptance Capacity	SIDRA Standard (Akçelik M3D)
HCM Delay Formula	Yes
HCM Queue Formula	Yes
Downstream Short Lane Model	
Minimum Downstream Utilisation Ratio	20 %
Minimum Downstream Distance	30 m
Distance for Full Lane Utilisation	200 m
Calibration Parameter	1.2

Site Properties	
Site (Intersection) Type	Signals
Model Name	Standard Right
Drive Rule	Right-hand side of the road
New Zealand Rule	No
HCM Version	No
Units	Metric

Analysis result (output) of traffic signal near Giorgis

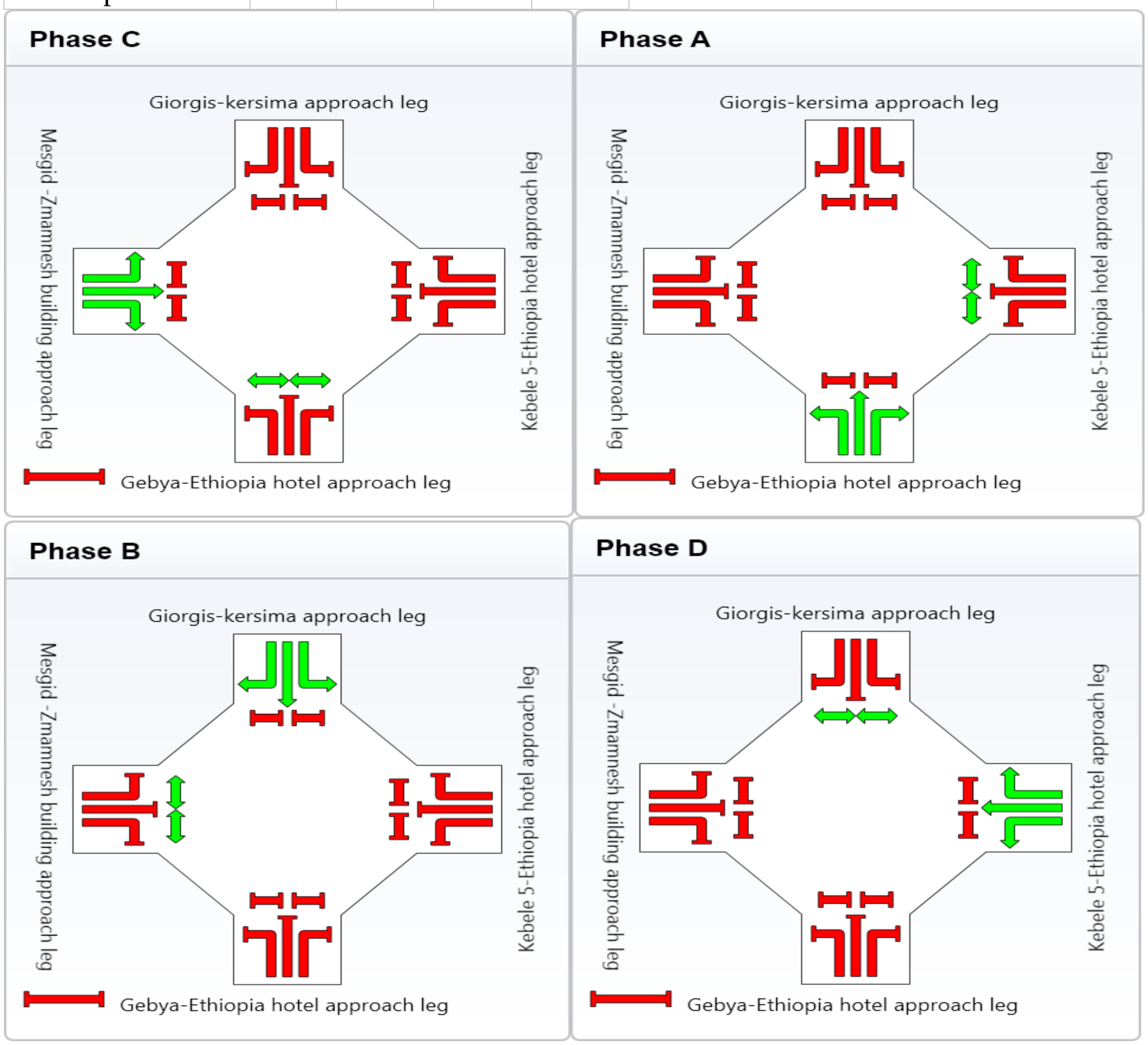
Phase timing results

Signals - Fixed Time Cycle Time = 120 seconds (User-Given Phase Times)

Sequence: Split Phasing

Table D-2 Phase timing results of traffic signal near giorgis

Phase	A	B	C	D
Green Time (sec)	24	24	24	24
Yellow Time (sec)	4	4	4	4
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	30	30	30	30
Phase Split	25 %	25 %	25 %	25 %



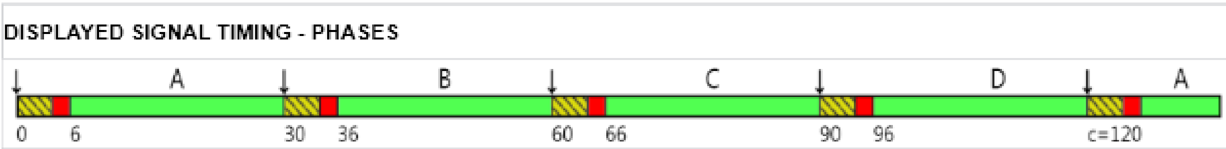


Figure D.3 Traffic signal Phases

Table D-3 Analysis result of traffic signal near giorgis

Lane Use and Performance

	Demand Flows L veh/h	Demand Flows T veh/h	Demand Flows R veh/h	Total veh/h	HV %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Lane Length m
South: Gebya-Ethiopia hotel approach leg													
Lane 1	154	99	0	253	1.9	339	0.747	100	59.1	LOS E	15.7	111.6	78
Lane 2	0	261	0	261	1.9	349	0.747	100	58.8	LOS E	16.2	114.9	78
Lane 3	0	99	154	253	1.9	339	0.747	100	59.1	LOS E	15.7	111.6	78
Approach	154	460	154	767	1.9		0.747		59.0	LOS E	16.2	114.9	
East: Kebele 5-Ethiopia hotel approach leg													
Lane 1	32	0	0	32	9.0	308	0.102	100	39.9	LOS D	1.5	11.4	69
Lane 2	0	64	32	96	9.0	320	0.300	100	43.2	LOS D	4.9	36.7	69
Approach	32	64	32	127	9.0		0.300		42.4	LOS D	4.9	36.7	
North: Giorgis-kersima approach leg													
Lane 1	168	110	0	278	2.5	337	0.824	100	66.0	LOS E	18.3	130.8	59
Lane 2	0	287	0	287	2.5	348	0.824	100	65.5	LOS E	18.8	134.7	59
Lane 3	0	110	168	278	2.5	337	0.824	100	66.0	LOS E	18.3	130.8	59
Approach	168	506	168	843	2.5		0.824		65.8	LOS E	18.8	134.7	
West: Mesgid -Zmamnesh building approach leg													
Lane 1	55	109	0	164	3.8	328	0.501	100	48.1	LOS D	9.0	64.8	69
Lane 2	0	0	55	55	3.8	317	0.173	100	41.0	LOS D	2.7	19.4	69
Approach	55	109	55	219	3.8		0.501		46.3	LOS D	9.0	64.8	
Intersection				1957	2.8		0.824		59.4	LOS E	18.8	134.7	

Intersection Performance - Hourly Values			
Performance Measure	**Vehicles**	**Pedestrians**	**Persons**
Demand Flows (Total)	1957 veh/h	8298 ped/h	18082 pers/h
Percent Heavy Vehicles	2.8 %		
Degree of Saturation	0.824	0.860	
Practical Spare Capacity	9.2 %		
Effective Intersection Capacity	2275 veh/h		
Control Delay (Total)	32.30 veh-h/h	94.51 ped-h/h	256.01 pers-h/h
Control Delay (Average)	59.4 sec	41.0 sec	51.0 sec
Control Delay (Worst Lane)	66.0 sec		
Control Delay (Worst Movement)	66.0 sec	43.4 sec	66.0 sec
Geometric Delay (Average)	1.8 sec		
Stop-Line Delay (Average)	59.4 sec		
Intersection Level of Service (LOS)	LOS E	LOS E	
95% Back of Queue - Vehicles (Worst Lane)	18.8 veh		
95% Back of Queue - Distance (Worst Lane)	134.7 m		
Total Effective Stops	1759 veh/h	6858 ped/h	15651 pers/h
Effective Stop Rate	0.90 per veh	0.83 per ped	0.87 per pers
Proportion Queued	0.98	0.83	0.91
Performance Index	122.0	177.4	299.4
Travel Distance (Total)	230.4 veh-km/h	225.8 ped-km/h	1377.6 pers-km/h
Travel Distance (Average)	118 m	27 m	76 m
Travel Time (Total)	39.8 veh-h/h	139.3 ped-h/h	338.3 pers-h/h
Travel Time (Average)	73.2 sec	60.4 sec	67.3 sec
Travel Speed	5.8 km/h	1.6 km/h	4.1 km/h

Intersection Performance - Annual Values			
Performance Measure	**Vehicles**	**Pedestrians**	**Persons**
Demand Flows (Total)	6,183,621 veh/y	26,221,680 ped/y	57,139,790 pers/y
Delay	102,072 veh-h/y	298,642 ped-h/y	809,004 pers-h/y
Effective Stops	5,556,887 veh/y	21,672,440 ped/y	49,456,870 pers/y
Travel Distance	727,940 veh-km/y	713,637 ped-km/y	4,353,340 pers-km/y
Travel Time	125,739 veh-h/y	440,237 ped-h/y	1,068,934 pers-h/y

Analysis results of traffic signal near Giorgis for improved phases

Improved phase time results

Signals - Fixed Time Cycle Time = 120 seconds (User-Given Phase Times)

Sequence: Split Phasing

Table D-4 Improved phase time results of traffic signal near giorgis

Phase Timing Results

Phase	A	B	C	D
Green Time (sec)	27	27	21	21
Yellow Time (sec)	4	4	4	4
All-Red Time (sec)	2	2	2	2
Phase Time (sec)	33	33	27	27
Phase Split	28 %	28 %	23 %	23 %

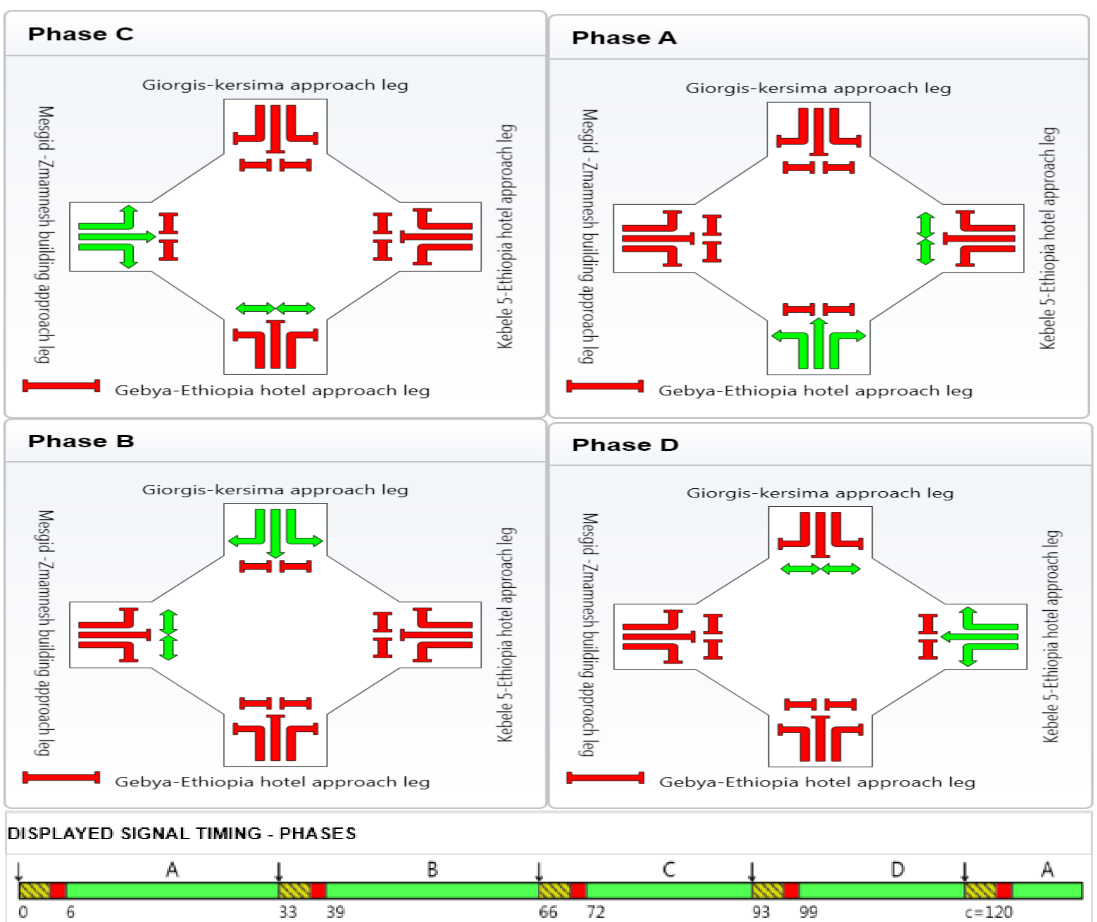


Figure D.4 Improved phase time results of traffic signal near giorgis

Table D-5 Analysis result of traffic signal near giorgis for improved phase times

Lane Use and Performance

	Demand Flows L veh/h	Demand Flows T veh/h	Demand Flows R veh/h	Total veh/h	HV %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Lane Length m
South: Gebya-Ethiopia hotel approach leg													
Lane 1	154	99	0	253	1.9	381	0.664	100	51.2	LOS D	14.7	104.4	78
Lane 2	0	261	0	261	1.9	393	0.664	100	50.9	LOS D	15.1	107.5	78
Lane 3	0	99	154	253	1.9	381	0.664	100	51.2	LOS D	14.7	104.4	78
Approach	154	460	154	767	1.9		0.664		51.1	LOS D	15.1	107.5	
East: Kebele 5-Ethiopia hotel approach leg													
Lane 1	32	0	0	32	9.0	270	0.117	100	42.6	LOS D	1.6	11.8	69
Lane 2	0	64	32	96	9.0	280	0.343	100	46.8	LOS D	5.1	38.1	69
Approach	32	64	32	127	9.0		0.343		45.7	LOS D	5.1	38.1	
North: Giorgis-kersima approach leg													
Lane 1	168	110	0	278	2.5	380	0.733	100	55.0	LOS D	16.8	120.4	59
Lane 2	0	287	0	287	2.5	391	0.733	100	54.7	LOS D	17.3	124.0	59
Lane 3	0	110	168	278	2.5	380	0.733	100	55.0	LOS D	16.8	120.4	59
Approach	168	506	168	843	2.5		0.733		54.9	LOS D	17.3	124.0	
West: Mesgid -Zmamnesh building approach leg													
Lane 1	55	109	0	164	3.8	287	0.572	100	53.4	LOS D	9.4	68.0	69
Lane 2	0	0	55	55	3.8	277	0.197	100	43.9	LOS D	2.8	20.0	69
Approach	55	109	55	219	3.8		0.572		51.1	LOS D	9.4	68.0	
Intersection				1957	2.8		0.733		52.4	LOS D	17.3	124.0	

Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.
LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).
HCM Delay Model used. Geometric Delay not included.

Intersection Performance - Hourly Values

Performance Measure	Vehicles	Pedestrians
Demand Flows (Total)	1957 veh/h	8298 ped/h
Percent Heavy Vehicles	2.8 %	
Degree of Saturation	0.733	0.757
Practical Spare Capacity	22.8 %	
Effective Intersection Capacity	2586 veh/h	
Control Delay (Total)	28.47 veh-h/h	92.35 ped-h/h
Control Delay (Average)	52.4 sec	40.1 sec
Control Delay (Worst Lane)	55.0 sec	
Control Delay (Worst Movement)	55.0 sec	45.9 sec
Geometric Delay (Average)	1.8 sec	
Stop-Line Delay (Average)	52.4 sec	
Intersection Level of Service (LOS)	LOS D	LOS E
95% Back of Queue - Vehicles (Worst Lane)	17.3 veh	
95% Back of Queue - Distance (Worst Lane)	124.0 m	
Total Effective Stops	1638 veh/h	6773 ped/h
Effective Stop Rate	0.84 per veh	0.82 per ped
Proportion Queued	0.97	0.82
Performance Index	113.3	174.8
Travel Distance (Total)	230.4 veh-km/h	225.8 ped-km/h
Travel Distance (Average)	118 m	27 m
Travel Time (Total)	35.9 veh-h/h	137.2 ped-h/h
Travel Time (Average)	66.1 sec	59.5 sec
Travel Speed	6.4 km/h	1.6 km/h

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Intersection LOS value for Vehicles is based on average delay for all vehicle movements.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

HCM Delay Model used. Geometric Delay not included.

Intersection Performance - Annual Values

Performance Measure	Vehicles	Pedestrians
Demand Flows (Total)	6,183,621 veh/y	26,221,680 ped/y
Delay	89,965 veh-h/y	291,823 ped-h/y
Effective Stops	5,177,124 veh/y	21,404,000 ped/y
Travel Distance	727,940 veh-km/y	713,637 ped-km/y
Travel Time	113,598 veh-h/y	433,417 ped-h/y

Appendix E-Giorgis roundabout performance analysis

Layout of` the intersection

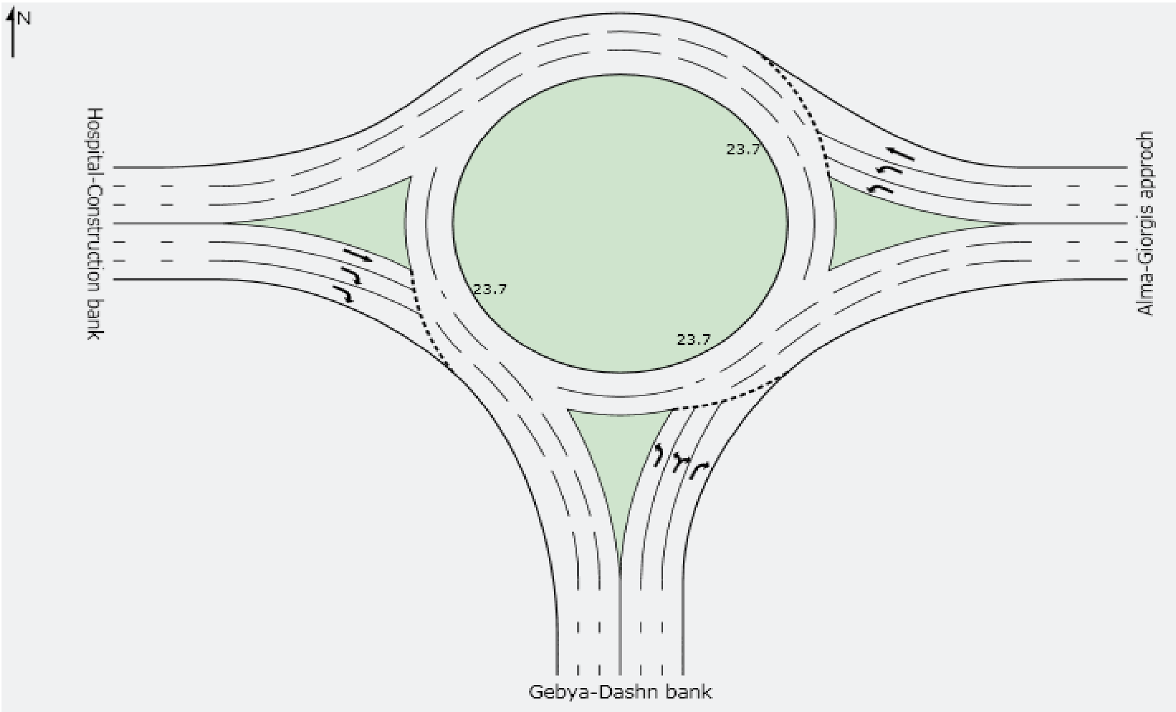


Figure E.1 Giorgis roundabout intersection layout

Movement IDs

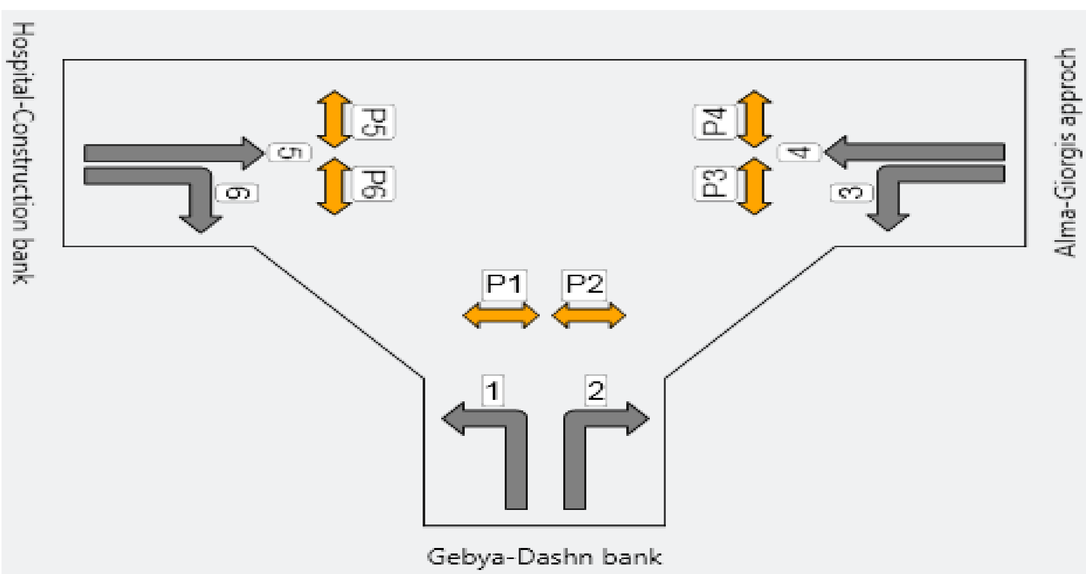


Figure E.2 Giorgis roundabout movement IDs

Table E-1 Summarized input data for analysis of Giorgis roundabout

Intersection Parameters	
Title	Giorgis Roundabout
Intersection ID	3
Unit Time (for volumes)	60 minutes
Peak Flow Period (for performance)	15 minutes

Geometry - Approach Data

Location	Name	Type	No. of App. Lanes	No. of Exit Lanes	Median Width m	Extra Bunching %
South	Gebya-Dashn bank	Two-way	3	3	2.60	0.0
East	Alma-Giorgis approch	Two-way	3	3	3.90	0.0
West	Hospital-Construction bank	Two-way	3	3	3.90	0.0

Geometry - Roundabout Data

Location	Name	Island Diameter m	Circ. Width m	Circ. Lanes	Entry Radius m	Entry Angle degrees	Env. Factor	Entry/Circ. Flow Adjust.
South	Gebya-Dashn bank	23.70	8.50	2	20.0	30.0	1.3000	Medium
East	Alma-Giorgis approch	23.70	8.50	2	20.0	30.0	1.3000	Medium
West	Hospital-Construction bank	23.70	8.50	2	20.0	30.0	1.3000	Medium

Geometry - Approach Lane Data

Lane Number	Lane Type	Lane Discip.	Basic Satn Flow tcu/h
South	Gebya-Dashn bank		
App. Lane 1	Normal	L	1750
App. Lane 2	Normal	LR	1750
App. Lane 3	Normal	R	1750
East	Alma-Giorgis approch		
App. Lane 1	Normal	L	1750
App. Lane 2	Normal	L	1750
App. Lane 3	Normal	T	1750
West	Hospital-Construction bank		
App. Lane 1	Normal	T	1750
App. Lane 2	Normal	R	1750
App. Lane 3	Normal	R	1750

Geometry - Approach & Exit Lane Data

Lane Number	Lane Width m	Lane Length m	Grade %	SL Type
South	Gebya-Dashn bank			
App. Lane 1	3.50	59.0	0.00	–
App. Lane 2	3.50	59.0	0.00	–
App. Lane 3	3.50	59.0	0.00	–
Exit Lane 1	3.50	59.0	0.00	–
Exit Lane 2	3.50	59.0	0.00	–
Exit Lane 3	3.50	59.0	0.00	–
East	Alma-Giorgis approch			
App. Lane 1	2.95	41.0	0.00	–
App. Lane 2	2.95	41.0	0.00	–
App. Lane 3	2.95	41.0	0.00	–
Exit Lane 1	2.95	41.0	0.00	–
Exit Lane 2	2.95	41.0	0.00	–
Exit Lane 3	2.95	41.0	0.00	–
West	Hospital-Construction bank			
App. Lane 1	2.95	118.0	0.00	–
App. Lane 2	2.95	118.0	0.00	–
App. Lane 3	2.95	118.0	0.00	–
Exit Lane 1	3.38	118.0	0.00	–
Exit Lane 2	3.38	118.0	0.00	–
Exit Lane 3	3.38	118.0	0.00	–

Geometry - Movement Definitions

To Approach		Movement Banned	Turn Desig.
From: South	Gebya-Dashn bank		
South		Yes	–
West		No	L
East		No	R
From: East	Alma-Giorgis approch		
East		Yes	–
South		No	L
West		No	T
From: West	Hospital-Construction bank		
West		Yes	–
East		No	T
South		No	R

Volumes

To Approach		Total veh	HV %	Peak Flow Factor %	Vehicle Occupancy pers/veh	Flow Scale %	Growth Rate %/year
From: South	Gebya-Dashn bank						
West		304.0	1.80	95.0	5.00	100.00	7.00
East		304.0	1.80	95.0	5.00	100.00	7.00
From: East	Alma-Giorgis approch						
South		489.0	7.00	95.0	5.00	100.00	7.00
West		209.0	7.00	95.0	5.00	100.00	7.00
From: West	Hospital-Construction bank						
East		215.0	3.40	95.0	5.00	100.00	7.00
South		501.0	3.40	95.0	5.00	100.00	7.00

Path Data

To Approach	App. Cruise Speed km/h	Exit Cruise Speed km/h	App. Trav. Distance m
From: South	Gebya-Dashn bank		
West	40.0	40.0	59.0
East	40.0	40.0	59.0
From: East	Alma-Giorgis approch		
South	40.0	40.0	41.0
West	40.0	40.0	41.0
From: West	Hospital-Construction bank		
East	40.0	40.0	118.0
South	40.0	40.0	118.0

Movement Data - General

Turn	Mov. ID	Queue Space LV m	Queue Space HV m	Vehicle Length LV m	Vehicle Length HV m	HVE	P.Deg. Satn	Movement Type	Movement Control
South	Gebya-Dashn bank								
L	1	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
R	2	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
East	Alma-Giorgis approch								
L	3	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
T	4	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
West	Hospital-Construction bank								
T	5	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–
R	6	7.00	13.00	4.50	10.00	2.00	–	*Normal*	–

Movement Type and Control parameters are set automaticly from Approach Control and Lane Type data in the Geometry dialog.

Pedestrians

Mov. ID	Volume ped	Peak Flow %	Flow Scale %	Growth Rate %/year	Crossing Distance m	App. Trav. Distance m	Downst. Distance m	Walking Speed m/sec	Queue Space m	P.Deg. Satn
South	Gebya-Dashn bank									
P1	810.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-
P2	810.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-
East	Alma-Giorgis approch									
P3	1400.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-
P4	1400.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-
West	Hospital-Construction bank									
P5	1096.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-
P6	1096.0	95.0	100.00	2.00	-	10.0	10.0	1.40	1.00	-

Model Settings - Options

General Options	
Level of Service Method	Delay & v/c (HCM 2010)
Level of Service Target	LOS D
Performance Measure	Delay
Percentile Queue	95 %
Hours per Year	3160 h
Gap Acceptance	
HV Method for Gap-Acceptance	Include HV Effect for all percentages
Gap-Acceptance Capacity	SIDRA Standard (Akçelik M3D)
HCM Delay Formula	Yes
Downstream Short Lane Model	
Minimum Downstream Utilisation Ratio	20 %
Minimum Downstream Distance	30 m
Distance for Full Lane Utilisation	200 m
Calibration Parameter	1.2

Model Settings - Roundabouts	
Roundabout Model Options	
Capacity Model	SIDRA Standard
LOS Method	Same as Signalised Intersections
US HCM 2010 Roundabout Model	
Include Origin-Destination Pattern Effects	–
Factor for Parameter A	–
Factor for Parameter B	–
Other Roundabout Models	
FHWA 2000	No
Use Urban Compact Roundabout	–
HCM 2000	No
NAASRA 1986	No

Site Properties	
Site (Intersection) Type	Roundabout
Model Name	Standard Right
Drive Rule	Right-hand side of the road
New Zealand Rule	No
HCM Version	No
Units	Metric

Table E-2 Analysis result of giorgis roundabout

Lane Use and Performance

	Demand Flows L veh/h	Demand Flows T veh/h	Demand Flows R veh/h	Total veh/h	HV %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Lane Length m
South: Gebya-Dashn bank													
Lane 1	213	0	0	213	1.8	696	0.306	100	9.0	LOS A	1.2	8.4	59
Lane 2	107	0	107	213	1.8	696	0.306	100	9.0	LOS A	1.2	8.4	59
Lane 3	0	0	213	213	1.8	696	0.306	100	9.0	LOS A	1.2	8.4	59
Approach	320	0	320	640	1.8		0.306		9.0	LOS A	1.2	8.4	
East: Alma-Giorgis approch													
Lane 1	257	0	0	257	7.0	424	0.607	100	23.8	LOS C	2.0	14.8	41
Lane 2	257	0	0	257	7.0	424	0.607	100	23.8	LOS C	2.0	14.8	41
Lane 3	0	220	0	220	7.0	408	0.539	100	21.4	LOS C	1.6	11.8	41
Approach	515	220	0	735	7.0		0.607		23.1	LOS C	2.0	14.8	
West: Hospital-Construction bank													
Lane 1	0	226	0	226	3.4	411	0.550	100	21.7	LOS C	2.0	14.4	118
Lane 2	0	0	260	260	3.4	427	0.608	100	23.7	LOS C	2.4	17.2	118
Lane 3	0	0	268	268	3.4	440	0.608	100	23.1	LOS C	2.4	17.3	118
Approach	0	226	527	754	3.4		0.608		22.9	LOS C	2.4	17.3	
Intersection				2128	4.2		0.608		18.8	LOS B	2.4	17.3	

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Signalised Intersections.

Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.

LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).

Intersection and Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).

Roundabout Capacity Model: SIDRA Standard.

HCM Delay Model used. Geometric Delay not included.

Intersection Performance - Hourly Values

Performance Measure	Vehicles	Persons
Demand Flows (Total)	2128 veh/h	17604 pers/h
Percent Heavy Vehicles	4.2 %	
Degree of Saturation	0.608	
Practical Spare Capacity	39.7 %	
Effective Intersection Capacity	3499 veh/h	
Control Delay (Total)	11.10 veh-h/h	55.48 pers-h/h
Control Delay (Average)	18.8 sec	11.3 sec
Control Delay (Worst Lane)	23.8 sec	
Control Delay (Worst Movement)	23.8 sec	23.8 sec
Geometric Delay (Average)	3.6 sec	
Stop-Line Delay (Average)	18.8 sec	
Intersection Level of Service (LOS)	LOS B	
95% Back of Queue - Vehicles (Worst Lane)	2.4 veh	
95% Back of Queue - Distance (Worst Lane)	17.3 m	
Total Effective Stops	1556 veh/h	7780 pers/h
Effective Stop Rate	0.73 per veh	0.44 per pers
Proportion Queued	0.58	0.35
Performance Index	33.7	33.7
Travel Distance (Total)	301.5 veh-km/h	1507.5 pers-km/h
Travel Distance (Average)	142 m	86 m
Travel Time (Total)	20.7 veh-h/h	103.6 pers-h/h
Travel Time (Average)	35.1 sec	21.2 sec
Travel Speed	14.5 km/h	14.5 km/h

Intersection Performance - Annual Values

Performance Measure	Vehicles	Persons
Demand Flows (Total)	6,725,811 veh/y	55,628,970 pers/y
Delay	35,061 veh-h/y	175,304 pers-h/y
Effective Stops	4,916,978 veh/y	24,584,890 pers/y
Travel Distance	952,744 veh-km/y	4,763,719 pers-km/y
Travel Time	65,504 veh-h/y	327,520 pers-h/y

Appendix F-Performance analysis of Kucht junction

Layout of the intersection

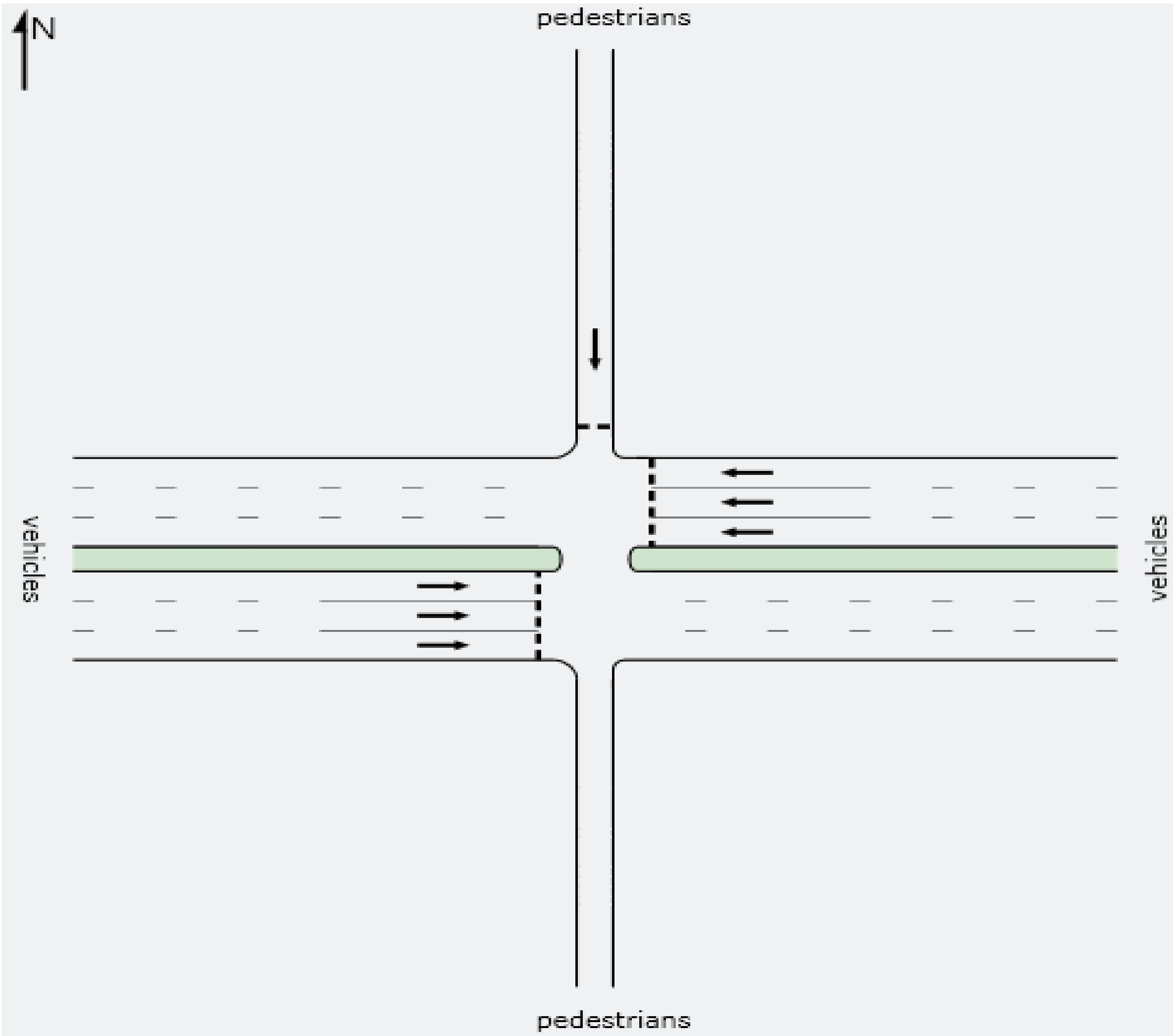


Figure F.1 Layout of Kucht junction

Table F-1 Summarized input data for analysis of Kucht junction

Intersection Parameters	
Title	Kucht Unsignalised pedestrian (Zebra) crossing across two-way road
Intersection ID	4
Unit Time (for volumes)	60 minutes
Peak Flow Period (for performance)	15 minutes

Geometry - Approach Data

Location	Name	Type	No. of App. Lanes	No. of Exit Lanes	Median Width m	Extra Bunching %
South	pedestrians	One-way Exit	0	1	–	0.0
East	vehicles	Two-way	3	3	2.90	0.0
North	pedestrians	One-way Approach	1	0	–	0.0
West	vehicles	Two-way	3	3	2.90	0.0

Geometry - Approach Lane Data

Lane Number	Lane Type	Lane Discip.	Basic Satn Flow tcu/h
East vehicles			
App. Lane 1	Normal	T	1750
App. Lane 2	Normal	T	1750
App. Lane 3	Normal	T	1750
North pedestrians			
App. Lane 1	Normal	T	1750
West vehicles			
App. Lane 1	Normal	T	1750
App. Lane 2	Normal	T	1750
App. Lane 3	Normal	T	1750

Geometry - Approach & Exit Lane Data

Lane Number		Lane Width m	Lane Length m	Grade %	SL Type
South	pedestrians				
Exit Lane 1		3.55	90.0	0.00	–
East	vehicles				
App. Lane 1		3.55	137.5	0.00	–
App. Lane 2		3.55	137.5	0.00	–
App. Lane 3		3.55	137.5	0.00	–
Exit Lane 1		3.55	137.5	0.00	–
Exit Lane 2		3.55	137.5	0.00	–
Exit Lane 3		3.55	137.5	0.00	–
North	pedestrians				
App. Lane 1		3.55	41.0	0.00	–
West	vehicles				
App. Lane 1		3.55	45.0	0.00	–
App. Lane 2		3.55	45.0	0.00	–
App. Lane 3		3.55	45.0	0.00	–
Exit Lane 1		3.55	45.0	0.00	–
Exit Lane 2		3.55	45.0	0.00	–
Exit Lane 3		3.55	45.0	0.00	–

Lanes are numbered from left to right in the direction of travel.

Geometry - Movement Definitions

To Approach		Movement Banned	Turn Desig.
From: East	vehicles		
East		Yes	–
South		Yes	–
West		No	T
From: North	pedestrians		
East		Yes	–
South		No	T
West		Yes	–
From: West	vehicles		
West		Yes	–
East		No	T
South		Yes	–

Volumes

To Approach	Total veh	HV %	Peak Flow Factor %	Vehicle Occupancy pers/veh	Flow Scale %	Growth Rate %/year
From: East	vehicles					
West	562.0	3.40	95.0	5.00	100.00	7.00
From: North	pedestrians					
South	2242.0	0.00	95.0	5.00	100.00	2.00
From: West	vehicles					
East	604.0	7.50	95.0	5.00	100.00	7.00

Path Data

To Approach	App. Cruise Speed km/h	Exit Cruise Speed km/h	App. Trav. Distance m
From: East	vehicles		
West	40.0	40.0	137.5
From: North	pedestrians		
South	4.0	4.0	10.0
From: West	vehicles		
East	40.0	40.0	45.0

Movement Data - General

Turn	Mov. ID	Queue Space LV m	Queue Space HV m	Vehicle Length LV m	Vehicle Length HV m	HVE	P.Deg. Satn	Movement Type	Movement Control
East	vehicles								
T	v2	7.00	13.00	4.50	10.00	2.00	–	*Normal*	*Giveway*
North	pedestrians								
T	p	1.00	1.00	0.50	0.50	2.00	–	*Normal*	*Giveway*
West	vehicles								
T	v1	7.00	13.00	4.50	10.00	2.00	–	*Normal*	*Giveway*

Movement Type and Control parameters are set automaticly from Approach Control and Lane Type data in the Geometry dialog.

Priorities

Opposed Movement	Opposing Movements South	South East	East	North East	North	North West	West	South West
East	vehicles							
T	-	-	-	-	T	-	-	-
North	pedestrians							
T	-	-	-	-	-	-	-	-
West	vehicles							
T	-	-	-	-	T	-	-	-

Model Settings - Options

General Options	
Level of Service Method	Delay & v/c (HCM 2010)
Level of Service Target	LOS D
Performance Measure	Delay
Percentile Queue	95 %
Hours per Year	3160 h
Gap Acceptance	
HV Method for Gap-Acceptance	Include HV Effect if above 5 per cent
Gap-Acceptance Capacity	SIDRA Standard (Akçelik M3D)
HCM Delay Formula	No
Downstream Short Lane Model	
Minimum Downstream Utilisation Ratio	20 %
Minimum Downstream Distance	30 m
Distance for Full Lane Utilisation	200 m
Calibration Parameter	1.2

Site Properties

Site (Intersection) Type	Giveway / Yield (Two-Way)
Model Name	Standard Right
Drive Rule	Right-hand side of the road
New Zealand Rule	No
HCM Version	No
Units	Metric

Table F-2 Performance analysis results of intersection at Kucht building

Lane Use and Performance

	Demand Flows L veh/h	Demand Flows T veh/h	Demand Flows R veh/h	Total veh/h	HV %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance m	Lane Length m
East: vehicles													
Lane 1	0	197	0	197	3.4	60[2]	3.287	100	1103.1	LOS F	50.8	366.1	138
Lane 2	0	197	0	197	3.4	60[2]	3.287	100	1103.1	LOS F	50.8	366.1	138
Lane 3	0	197	0	197	3.4	60[2]	3.287	100	1103.1	LOS F	50.8	366.1	138
Approach	0	592	0	592	3.4		3.287		1103.1	LOS F	50.8	366.1	
North: pedestrians													
Lane 1	0	2360	0	2360	0.0	1750[2]	1.349	100	156.9	LOS F	191.1	191.1	41
Approach	0	2360	0	2360	0.0		1.349		156.9	LOS F	191.1	191.1	
West: vehicles													
Lane 1	0	212	0	212	7.5	60[2]	3.532	100	1211.5	LOS F	55.9	416.5	45
Lane 2	0	212	0	212	7.5	60[2]	3.532	100	1211.5	LOS F	55.9	416.5	45
Lane 3	0	212	0	212	7.5	60[2]	3.532	100	1211.5	LOS F	55.9	416.5	45
Approach	0	636	0	636	7.5		3.532		1211.5	LOS F	55.9	416.5	
Intersection				3587	1.9		3.532		499.8	NA	191.1	416.5	

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Lane LOS values are based on average delay and v/c ratio (degree of saturation) per lane.

LOS F will result if v/c > irrespective of lane delay value (does not apply for approaches and intersection).

Minor Road Approach LOS values are based on average delay for all lanes (v/c not used as specified in HCM 2010).

NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.

SIDRA Standard Delay Model used.

2 Minimum Capacity

Intersection Performance - Hourly Values		
Performance Measure	**Vehicles**	**Persons**
Demand Flows (Total)	3587 veh/h	17937 pers/h
Percent Heavy Vehicles	1.9 %	
Degree of Saturation	3.532	
Practical Spare Capacity	-77.4 %	
Effective Intersection Capacity	1016 veh/h	
Control Delay (Total)	498.05 veh-h/h	2490.25 pers-h/h
Control Delay (Average)	499.8 sec	499.8 sec
Control Delay (Worst Lane)	1211.5 sec	
Control Delay (Worst Movement)	1211.5 sec	1211.5 sec
Geometric Delay (Average)	1.1 sec	
Stop-Line Delay (Average)	498.7 sec	
Intersection Level of Service (LOS)	NA	
95% Back of Queue - Vehicles (Worst Lane)	191.1 veh	
95% Back of Queue - Distance (Worst Lane)	416.5 m	
Total Effective Stops	3624 veh/h	18118 pers/h
Effective Stop Rate	1.01 per veh	1.01 per pers
Proportion Queued	1.00	1.00
Performance Index	743.0	743.0
Travel Distance (Total)	230.2 veh-km/h	1150.8 pers-km/h
Travel Distance (Average)	64 m	64 m
Travel Time (Total)	517.1 veh-h/h	2585.3 pers-h/h
Travel Time (Average)	518.9 sec	518.9 sec
Travel Speed	0.4 km/h	0.4 km/h

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

NA: Intersection LOS for Vehicles is Not Applicable for two-way sign control since the average intersection delay is not a good LOS measure due to zero delays associated with major road movements.

SIDRA Standard Delay Model used.

Intersection Performance - Annual Values		
Performance Measure	**Vehicles**	**Persons**
Demand Flows (Total)	11,336,090 veh/y	56,680,430 pers/y
Delay	1,573,838 veh-h/y	7,869,192 pers-h/y
Effective Stops	11,450,560 veh/y	57,252,790 pers/y
Travel Distance	727,275 veh-km/y	3,636,374 pers-km/y
Travel Time	1,633,882 veh-h/y	8,169,410 pers-h/y