ANALYSIS OF ALSELMANI SIGNALIZED INTERSECTION IN BENGHAZI - LIBYA

Thesis

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ABSTRACT

Intersections play an important role in the road network, where traffic flows in different directions converge. Because of their influence each other, disturbance of pedestrians and bicycle to vehicles, and the lost of green time for beginning and clearance and so on.

The spectacular growth of vehicles, as of the most convenient mode of travel has brought in its wake problems of frustration, accidents, delays, queues, congestion and environmental degradation. This has generated a new branch of the circulation of knowledge, known as traffic management. The argument is gaining importance as vehicles whose population is increasing in recent years, and traffic problems become more alarming. In many situations of daily life, it addresses the problem of providing services to the requirements of random. Consider the simple case of drivers who arrive at a junction with traffic lights, if the arrival rate is high and the rate of supply is unable to cope, it is inevitable that increases in the queue. Delay occurred as a result. In fact, queuing and delays to traffic at intersections, especially at the traffic light is a common problem in dealing with urban traffic. In this perspective, it is necessary to apply appropriate management of traffic at critical intersections to minimize the effect of delays and queues for the global system of circulation. Thus, knowledge on methods of traffic management is essential. In the recent years vehicle ownership and traffic volume in links increase dramatically due to the continuous high speed growth , which cause traffic congestion of different level in Bengazi city.. The objective of this study is to evaluate the performance of intersection and make alternative solutions recommendation. The analysis is done by evaluating the performance of intersection, including the initial conditions of intersection by using Indonesian Highway Capacity Manual (IHCM 1997).so the traffic flow, Then saturation flow rates and delays of the intersection Finally, some countermeasures for enhancing the capacities of the intersection are proposed including channelization of the traffic flow in the intersection, drawing the traffic marking and line in the intersection, strengthen traffic safety education, improving traffic control method, regulation of pedestrians and bicycle flows at the signalized intersection and improving the safety facility and capacity at the signalized intersections and so on. The control was changed from two-phase to four-phase control during , leading to excessive queuing in the area. The calculations for four phase control shows the reason why, the Intersection flow ratio increases , and the degree of saturation becomes higher in all approaches during the studied peak hour. The reason to change from two-phase to four-phase control was probably that the there were many traffic accidents in the intersection. An alternative action would have been to increase the inter-green periods between the phases, and to keep two-phase control which obviously results in a much higher intersection capacity and better level of performance level.The advantage of changed to four phase: inevitably leads to an increase of the cycle time and of the ratio of time allocated to switching between phases (Intergreen). Although this may be beneficial from the traffic safety point of view, it normally means that the overall capacity of the intersection is decreased.

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CHAPTER 1
INTRODUCTION

1.1 Background

Intersections play an important role in the road network, where traffic flows in different directions converge. Because of their influence each other, disturbance of pedestrians and bicycle to vehicles, and the lost of green time for beginning and clearance and so on , the capacity of intersections is much lower than that of their approach links. Thus, the intersections usually are the bottleneck of the network, the popular and immediate source of the traffic jam and traffic accidents. Benghazi City is the second largest city of Libya contains a lot of Beaches monuments and places and in holiday times crowded roads in the city because a lot of people come to spend the holiday in Benghazi. And especially the city is costal , tourist city and also close to the Seaport .

The increasing numbers of motor vehicles in Libya, all kinds of private cars, private transport, taxis and trucks and heavy equipment an indication of the severity of congestion on major roads between main and interior cities, there is continual congestion, traffic congestion is frequently a problem through the weekdays. Both geographically and administratively, the network started to suffer from various traffic problems. More and more policies and regulations are being developed , particularly in the field of traffic and transport.The spectacular growth of vehicles, as of the most convenient mode of travel has brought in its wake problems of frustration, accidents, delays, queues, congestion and environmental degradation. This has generated a new branch of the circulation of knowledge, known as traffic management. The argument is gaining importance as vehicles whose population is increasing in recent years, and traffic problems become more alarming. In many situations of daily life, it addresses the problem of providing services to the requirements of random. Consider the simple case of drivers who arrive at a junction with traffic lights, if the arrival rate is high and the rate of supply is unable to cope, it is inevitable that increases in the queue. Delay occurred as a result. In fact, queuing and delays to traffic at intersections, especially at the traffic light is a common problem in dealing with urban traffic. In this perspective, it is necessary to apply appropriate management of traffic at critical intersections to minimize the effect of delays and queues for the global system of circulation. Thus, knowledge on methods of traffic management is essential. In the recent years vehicle ownership and traffic volume in links increase dramatically due to the continuous high speed growth , which cause traffic congestion of different level in Benghazi city. Under this circumstance, velocity of vehicles drops largely and in the city the velocity in peak hour is even lower than 10 km/h. All of these already influence the normal performance of urban function, hamper the continuous and steady growth of the economy and affect residents' daily lives. It is also same for the developed country, the most. Traffic congestion happens at the intersection in Benghazi. First two main reasons causing the traffic congestion in Benghazi is as follows: there is an exclusive lane for right turn, right turn vehicle obscures the following vehicle and the capacity of the intersection is enough; Thus, it is significant to study the traffic flow characteristics at signalized intersections because the capacity of the intersection can be improved according to their Characteristics. Study on traffic flow characteristics at signalized intersections is one of most effective and immediate measure to enhance the capacity of road networks and relieve the congestion in the city. In this study, the traffic flow, Then saturation flow rates and delays of the intersection Finally, some countermeasures for enhancing the capacities of the intersection are proposed including channelization of the traffic flow in the intersection, drawing the traffic marking and line in the intersection, strengthen traffic safety education, improving traffic control method, regulation of pedestrians and bicycle flows at the signalized intersection and improving the safety facility and capacity at the signalized intersections and so on. The objective of the research is to evaluate the performance of intersection using the 1997 IHCM. The phases, cycle times, green times, amber and red lights for this intersection will be varied by existing traffic flow. The performance of this intersection will be expected to perform better, so the traffic jam such as delay and queuing could be reduced.

1.2 Problem Statement

As it was mentioned earlier in this chapter, the selected operating mode of traffic signal for a particular intersection is usually based on a measured or estimated peak traffic volume. This may provide efficient traffic flow during rush hours; however, operation during off-peak hours will be far from ideal. This happens because the main traffic parameter for signal design changes significantly. In most cases during the off-peak hours, the traffic volume decreases a lot and vehicle arrival patterns are random in nature. The conditions stated above make it hard or almost impossible to find a single optimal plan for signal timing. It can be even worse at an intersection where the traffic signal is coordinated with another traffic signal at different intersections. That particular problem can be very obvious in Benghazi Because it is a tourist and costal town and there is commercial seaport that is why the traffic congestion is common. The lack of optimized traffic control during off-peak hours can cause different issues such as travel delay and increased gas consumption for travelers who may be stuck at traffic lights on empty roads. But, most importantly, these conditions may cause safety issues – being stuck at the traffic light at night a driver may be robbed or experience other serious safety and security issues. Modern traffic signal systems researches have already come with a solution for that problem introducing several types of traffic signal operations including pre-timed, fully actuated, semi-actuated and flashing signal operating patterns. Improvements in off-peak traffic signal control can be beneficial for drives making it possible to get to their final destination during off-peak hours much faster. Importance Alsellmani Street does not lie in being a carrier route only but one of the key facilities which are located in the north and south of the intersection where there is a huge number means these facilities and thus witnessing the early morning hours and afternoon hours busiest great to get to these facilities .The east side is linked to the coastal road that connects the city and suburbs who have most of the city's population follow in terms of labor and universities, who is also witnessing at its busiest great morning and afternoon (the end of working hours).

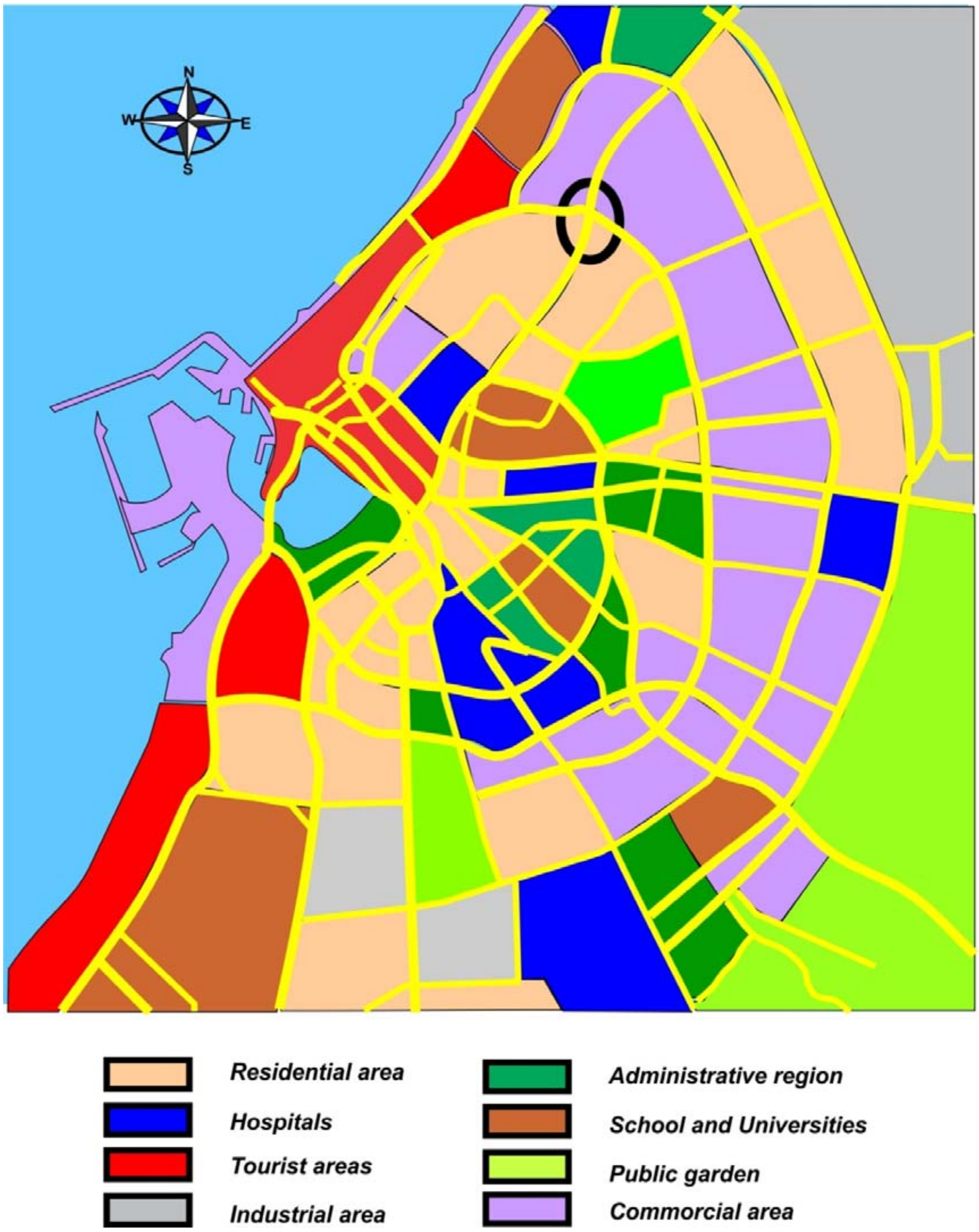


Figure 1.1 Land Use Map Of Alselmani Intersection

(Sorce: Atlas Book Of Benghazi(2010))

Figure 1.2 Alselmani Intersection

1.3 Objective Of Study

The objective of this study is to evaluate the performance of intersection and make alternative solutions recommendation. The analysis is done by evaluating the performance of intersection, including the initial conditions of intersection by using Indonesian Highway Capacity Manual (IHCM 1997).

1.4 Scope Of Study

This study focuses on the estimation of delays and queue lengths that result from the adoption of a signal control strategy at an intersection, Traffic delays and queues are principal performance measures of intersections. In the evaluation of the adequacy of cycle length, the obtained minimum delay being the foremost goal of the traffic engineers.

1.5 Organization Of Thesis

To facilitate reading and comprehension, the results of recent studies will need to be classified parts of the report follow a systematic study as follows:

A. CHAPTER I (Introduction)

Contains a description of the background research, the research aims and objectives, constraints and systematic problems of writing.

B. CHAPTER II (Review Books)

Contains a description of the basics of theories related to the trip of the traffic, the model and its characteristics, statistical methods, and previous studies of similar or similar has ever done.

C. CHAPTER III (Research Methodology)

Contains a description of the mindset of research, stages and procedures of research and analytical methods used.

D. CHAPTER IV (Data Analysis and Discussion)

This section contains a description of the data results of research on the trip model of traffic is accompanied by analysis and discussion of the nature and trends of the results of the study.

E. CHAPTER V (Conclusions and Recommendations)

This section contains a description of the conclusions can be drawn from the results of analysis of the results of research that has been done. Also presented suggestions for the application of research results in the field and to the possibility of further study.

CHAPTER 2
LITERATURE REVIEW

2.1 Signalized Intersection

As the number of vehicles in use increases in all developing countries the most severe congestion is being experienced in urban areas where the number of junctions severely restricts traffic flow. Urban Transport has many particular features in the form of journeys to work peak hour, congestion, traffic restrains measures, public transport priorities act. The usual solution to the problem of junction capacity has been either to make the area of the junction larger, or to adopt grade separation. The former is difficult to achieve in congested urban areas and the latter is extremely expensive. It is therefore important to investigate every possible means of increasing the capacity of the single level junction so that delays are reduced. Intersection is of the greatest importance in highway design because of their effect on the movements and safety of vehicular traffic flow. The actual place of intersection is determined by setting and design and the act of the intersection by the regulation and control of the traffic movement. Priority control of traffic at junction is one of the most widely used ways of resolving the conflict between merging and crossing vehicles. The universal adoption of the 'Give Way to traffic on the right' rule at roundabouts together with the use of 'Give Way' and 'Stop' control at junctions has considerably increased the number of occasions at which a driver has to merge or cross a major road traffic stream making use of gaps or lags in one or more conflicting stream. Normally traffic signal is introduced for one or more of the following reasons:

- To avoid blockage of an intersection by conflicting traffic stream, thus guaranteeing that a certain capacity can be maintained even during peak traffic conditions; (Proceedings of the Eastern Asia Society for Transportation Studies, Vol.4, October, 2003).
- To facilitate the crossing of major road by the vehicles and/or pedestrian from a minor road;
- To reduce the number of traffic accidents caused by collisions between vehicles in conflicting directions. Signalization by means of three-colored light (green, amber, red) is applied to separate of conflicting traffic movement in time. Three

basic mechanisms, which affect intersection operation and performance, should be well understood:

- Discharge headways at signalized intersections, and their relationship to lost times and saturation flow rates.
- Factor affecting right turn at a signalized intersection.
- Uses of demand volume.

The capacity of a traffic signal controlled intersection is limited by the capacities of the individual approaches to the intersection. There are two types of factor, which affect the capacity of approach, roadway and environment factor and traffic and control factor. The roadway and environmental factor that controls the capacity of an approach are the physical layout of the approach, in particular its width, the road along which left or right turning vehicle has to travel, and the gradient of the approach and its exit from the intersection. The capacity of an approach is measured independently of traffic and control factors and is expressed as the saturation flow. Saturation flow is defined as the maximum flow, expressed as equivalent passenger cars that can cross the stop line of the approach when there is a continuous green signal indication and a continuous queue of vehicles on the approach. Delay is used to define the level of service at signalized intersections, since delay not only indicates the amount of lost travel time and fuel consumption; it is also a measure of the frustration and discomfort of motorists. Delay, depends on the red time, which in turn depend on the length of the cycle. Reasonable levels of service can therefore be obtained for short cycle length, even though the (v/c) ratio is as high as 0.9. The LOS criteria are given in term of the average stopped delay per vehicle during an analysis period of 15 min. Level of service C described that level of operation at which delay per vehicle ranges from 15.1 to 25 Sec. At level of service C, many vehicles go through the intersection without stopping, but a significant number of vehicles are stopped. In addition, not all vehicles at an approach clear the intersection during a few cycles (cycle failure). The higher delay may be due to the significant number of vehicles arriving during the red phases (poor progression) and or relative long cycle lengths. Level of service D described that level of operation at which delay per vehicle ranges from 25.1 to 40 Sec. At level of service D, vehicles are stopped at the intersection, resulting in the longer delay. The number of individual cycles failing is now noticeable. The longer delay at this level of service is due to a combination of two or more several factors that include long cycle length, high (v/c) ratios, and unfavorable progression.

Level of service E describes that level of operation at which the delay per vehicle ranges from 40.1 to 60 Sec. At level of service E, individual cycle's frequency fails. This long delay, which is usually taken as the limit of accepted delay, generally indicates high (v/c) ratios, long cycle lengths, and poor progression Level of service F describes that level of operation at which the delay per vehicle is greater than 60 sec. This long delay is usually unacceptable to most motorists. At level of service F the phenomenon know as over saturation usually occurs that is arrival flow rates are greater than the capacity of the intersection. Long delay can also occur as a result of poor progression and long cycle length. Note that this level of service can occur when approaches have high (V/C) ratios, which are less than 1.00, but also have many individual cycles failing. Area Traffic Control: where the traffic is controlled at junctions in street network that the output at one junction are related in some way to the inputs at adjacent junctions. The interaction between individual components of the system affects the overall network performance. Linked and coordinated signal plans are mainly applicable to the principal linear routes. Methods of controlling traffic in the network are referred to as area traffic control and may also include a control mode, which is traffic responsive. Area traffic control first became feasible with the introduction of the electronic computer. Further analysis can be carried out and modifications introduced which further refine the choices, and improve sensitivity and performance. In dynamic control systems the greatest difficulty lies in describing, in a logical and mathematical way, the behavior of traffic and the rapid computation of optimum control policies. An important part of the communication system is the link between the detector and the central computer and the return link conveying instructions to the signal. The data transmission system usually operates through out-stations, connected to the traffic signal controllers, and collecting data from the individual detectors. This information before or after local processing is relayed over cables, usually rented telephone pairs, to the central controller. Control instruction is transmitted in a reverse direction to the local controller via the out station. Each part of the system has a unique address, which can be interrogated by the central controller either for data retrieval and resetting or for instruction and control operation. It is usually necessary to monitor thus key points which have a critical effect on the control of the whole system and closed-circuit television is often provided for this purposed.

2.2 Analysis Of Signalised Intersection By Using IHCM 1997

Indonesia Highway Capacity Manual is program for determination of signal timing, capacity and traffic performance (delay, queue length and proportion of stopping vehicles) for signalized intersection in urban and semi-urban areas. The traffic facilities capacity and traffic performance is primarily a function of geometric conditions and traffic demand. By means of the signal however, the planner can distribute capacity to different approaches through the green time allocated to each approach. In order to calculate capacity and traffic performance it is therefore necessary to first determine the signal phasing and timing, which is most appropriate for the studied condition. The methodology for analysis of signalized intersections described below is based on the following main principles:

a. Geometry:

The calculations are done separately for each approach. One intersection arm can consist of more than one approach, i.e. be divided into two or more sub-approaches. This is the case if the right turning and/or left turning movements received green signal in different phases (s) than the straight thought traffic, or if they are physically divided by a traffic island in the approach. For each approach or sub-approach the effective width (We) is determined with consideration to the layout of the entry and the exit and distribution of turning movements.

b. Traffic flow:

The calculation is performed on an hourly basis for one or more periods, e.g. based on peak hour design flow for morning, noon and afternoon traffic conditions. The traffic flow for each movement (left turning, straight through and right turning are converted from vehicles per hour to passenger car units (pcu) per car units (pcu) per hour using the following passenger car equivalent (pce) for protected and for opposed approach types

Vehicle Type	Pce for approach type	
	Protected	Opposed
Light Veh . (L V)	1.0	1.0
Heavy Veh.(HV)	1.3	1.3
Motorcycle(MC)	0.2	0.4

c. Basic model

The amounts of traffic that can pass through a signal controlled intersection from a given approach depends on the green time available to the traffic and on the maximum flow of vehicles pass the stop line during the green period. When the signal changes to green vehicles take some second to start and accelerate to normal speed. After a few second the queue discharges at constant rate called **saturation flow (S)**.

The saturation flow is the flow, which would be obtained if there was a continuous queue of vehicles and they were passed at green time, or the saturation flow is the maximum departure rate, which can be achieved when there is a queue. The saturation flow is generally expressed in vehicles per hour green time. **Figure2.1** could be seen that the average rate of flow is lower during few minutes because vehicles are accelerating to normal running speed. The capacity (C) of an approach to a signalized intersection can be expressed as follows:

$$C = S \times g/c \qquad (1)$$

Where:

C = Capacity (pcu/h)

S = Saturation flow, i.e. mean discharge rate from a queue in the approach during green signal (pcu/hg = pcu per hour of green)

g = Displayed Green Time

c = Cycle time, i.e. duration of a complete sequence of signal changes (i.e. between two consecutive starts of green in the phase).

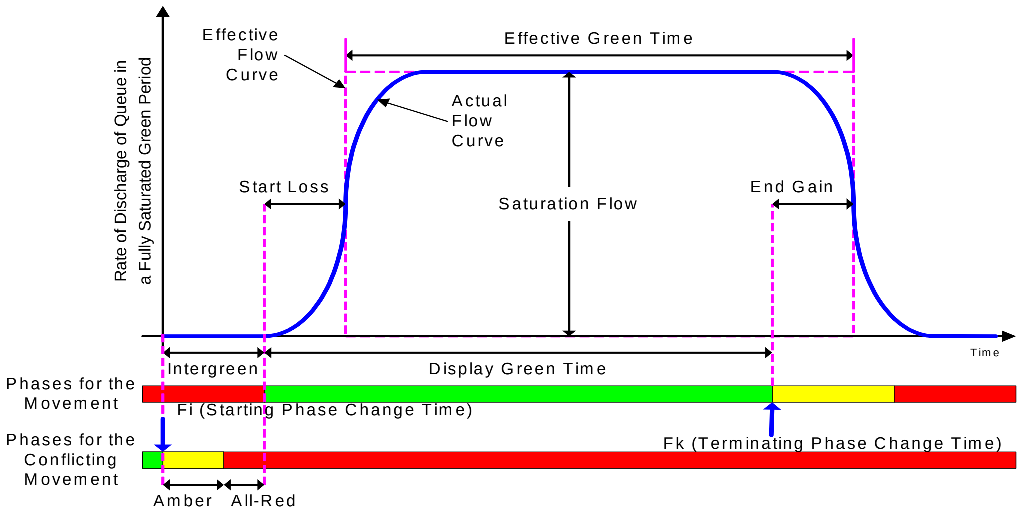


Figure 2.1 Basic model for saturation flow

Purpose of the Intergreen Period

- Warn discharging traffic that the phase is terminated. → Amber Period (for urban traffic signal in Indonesia is normally 3,0 sec) .
- Certify that the last vehicle in the green phase which is being terminated receives adequate time to evacuate the conflict zone before the first advancing vehicle in the next phase enters the same area. → All-Red Period.

All-Red Periods

When there are no vehicles at an intersection, the controller can be programmed for "red rest" operation, displaying the red indication to all approaches. The controller will provide an immediate green indication to the first phase where vehicle demand is sensed. After the demand is serviced, and assuming no other demand occurs, the intersection will return to red indications on all approaches.

Red revert timing is used with red rest to ensure that unreasonably short red intervals do not occur. Without some amount of red revert timing (usually between 2 and 6 seconds), the green indication could very quickly, and unexpectedly, return to a phase that had just turned red. This might surprise a driver since, once a signal turns red, most drivers expect to wait a certain amount of time before receiving the green again.

The saturation flow (S) can be expressed as a product between a base saturation flow (So) for a set of standard conditions, and adjustment factors (F) for deviation of the actual conditions from a set of pre-determined (ideal) conditions.

$$S = S_O \times F_{CS} \times F_{SP} \times F_G \times F_P \times F_{RT} \times F_{LT} \quad \text{pcu / hg} \qquad (2)$$

Where:

S = Saturation flow.

So = Base saturation flow.

F = Adjustment factors can be expressed as follows:

The City size correction factor F_{CS}

Table 2 : 1 City size correction factor F_{CS}

City population (M. inhabitants)	City size correction factor F_{CS}
> 3.0	1.05
1.0-3.0	1.00
0.3- 1.0	0.94
< 0.3	0.83

The Side friction correction factor F_{SF}

Table 2:2 Adjustment factor .for Road environment type and Side friction

Road environment	Side friction	Phase type	Ratio of unmotorised vehicles					
			0.0	0.05	0.10	0.15	0.20	0.25
Commercial	High	Opposed	0.93	0.88	0.84	0.79	0.74	0.70
		Protected	0.93	0.91	0.88	0.87	0.85	0.81

Road environment	Side friction	Phase type	Ratio of unmotorised vehicles					
			0.0	0.05	0.10	0.15	0.20	0.25
	Medium	Opposed	0.94	0.89	0.85	0.80	0.75	0.71
		Protected	0.94	0.92	0.89	0.88	0.86	0.82
	Low	Opposed	0.95	0.90	0.86	0.81	0.76	0.72
		Protected	0.95	0.93	0.90	0.89	0.87	0.83
Residential	High	Opposed	0.96	0.91	0.86	0.81	0.78	0.72
		Protected	0.96	0.94	0.92	0.89	0.86	0.84
	Medium	Opposed	0.97	0.92	0.87	0.82	0.79	0.73
		Protected	0.97	0.95	0.93	0.90	0.87	0.85
	Low	Opposed	0.98	0.93	0.88	0.83	0.80	0.74
		Protected	0.98	0.96	0.94	0.91	0.80	0.86
Restricted Ac-cess(RA)	High/Medium/Low	Opposed	1.00	0.95	0.90	0.85	0.80	0.75
		Protected	1.00	0.98	0.95	0.93	0.90	0.88

The Gradient correction factor F_G

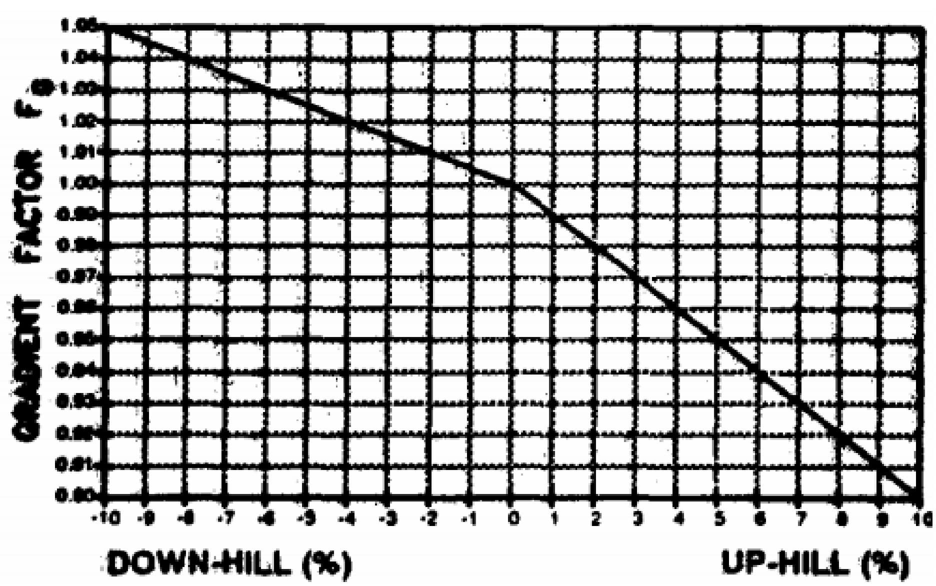


Figure 2:2 Correction factor for gradient F_G

The Right Turn correction factor

FRT

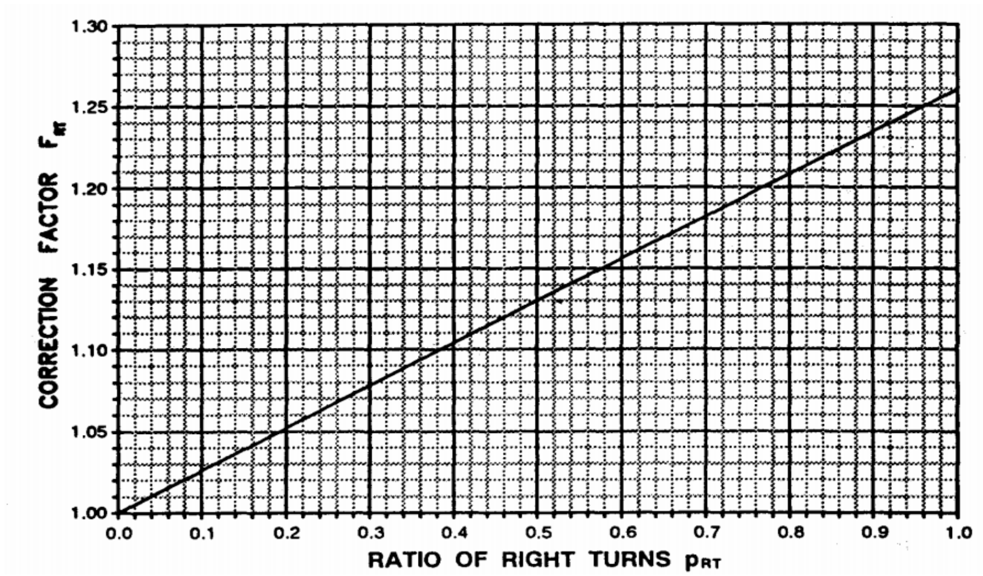


Figure 2:3 Correction factor for right turns FRT. (o n l y applicable for approach type P, two-way streets)

The Left turn correction factorFLT

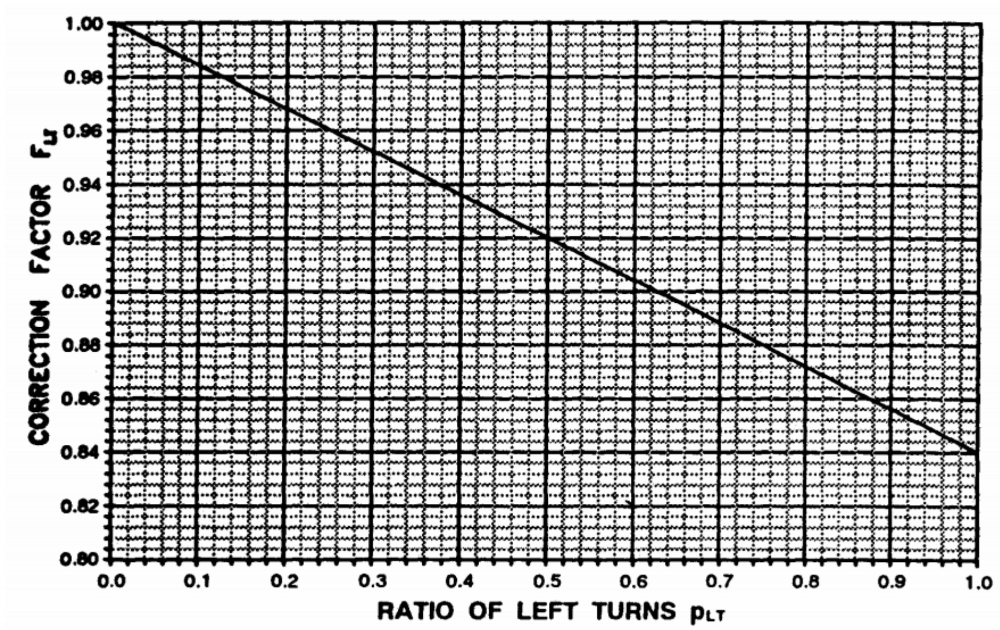


Figure 2:4 Correction factorfor effect of left turn FLT

For protected approaches P (protected discharge) the base saturation flows So is determined as a function of the effective approach width (We):

So = 600 x We pcu/hg, (see Figure 2.5)

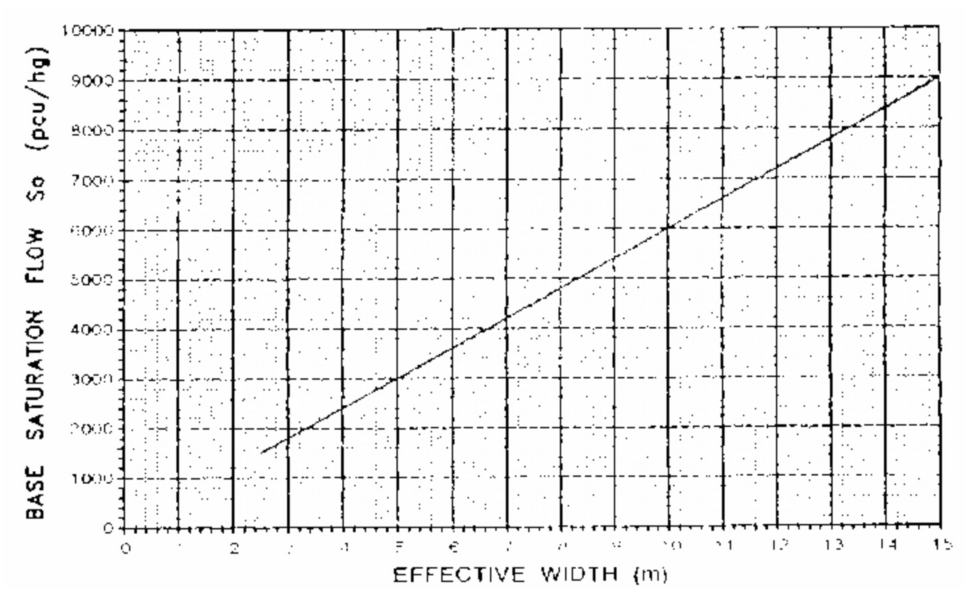


Figure 2.5 Base saturation flow for approach type P

For approach type 0 (opposed discharge), S_o is determined from Figure 2:6 (for approaches without separate left-turning lanes), and from Figure 2:7 (for approaches with separate left-turning lane) as a function of W_e Q_{LT} and Q_{LTO}

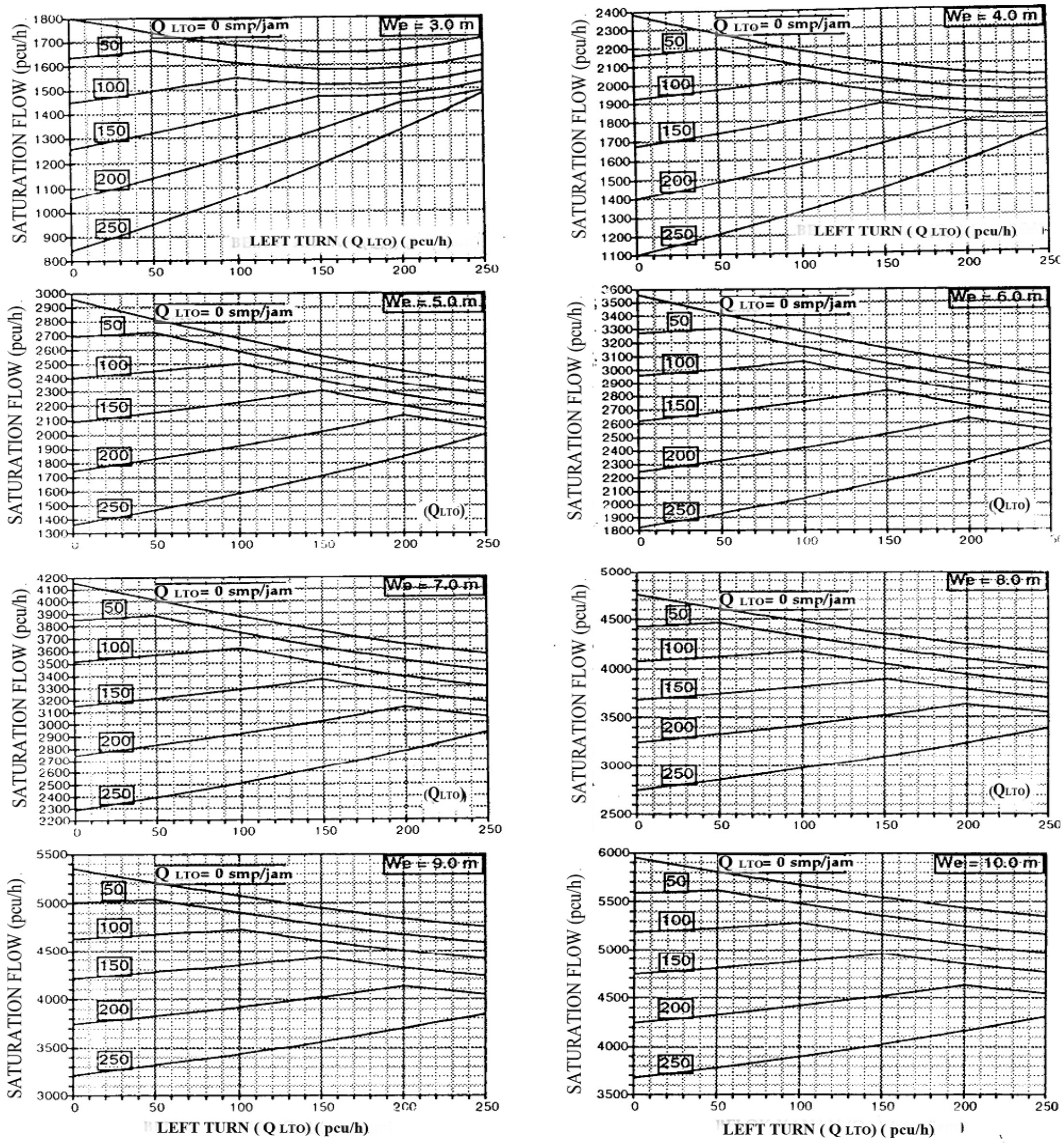


Figure 2:6 So for approaches type O without separate Left-turninglane

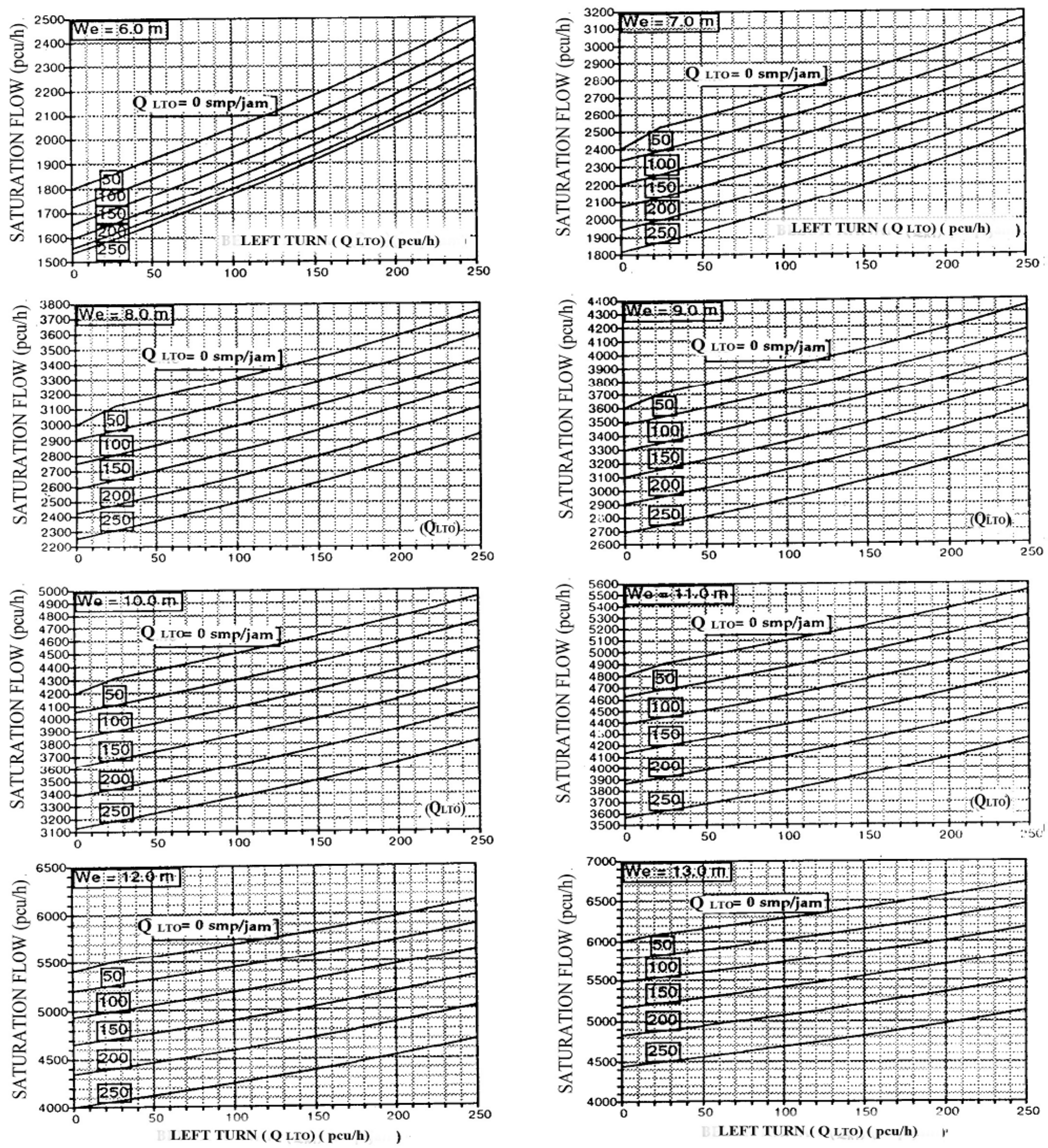


Figure 2.7 So for approach type O with separate Left-turning lane.

d. Signal Timing

Signal Timing the signal timing for fixed-time control conditions is determined based on the Webster (1966) method for minimization of overall vehicle delay in the intersection. First the cycle time (c) is determined, and after that the length of green (g) in each phase (i).

$$c = (1,5 \times LTI + 5) / (1 - FRcrit) \qquad (4)$$

Where:

c	=	Signal cycle time (Sec)
LTI	=	Lost time per cycle (Sec)
FR	=	Flow divided by saturation flow (Q/S)
FRcrit	=	The highest value of FR in all approaches being discharged in a signal phase
FRcrit	=	Intersection flow ratio = sum of FRcrit for all phases in the cycle

If the cycle time is shorter than this value there is a serious risk for over saturation of the intersection. Too long cycle time results in increased average delay of the traffic. If FRcrit is close to or greater than 1, the intersection is oversaturated and the formula will result in very high or negative cycle time values.

$$g = (c - LTI) \times FRcrit / (FRcrit) \qquad (5)$$

Where:

g = Displayed green time in phase I (Sec)

The performance of a signalized intersection is generally much more sensitive to errors in the green time distribution than to a too long cycle time. Even a small deviation from the green ratio (g/c) determined from equation (4) and (5) above result in high increase of the average delay in the intersection.

e. Capacity and degree of saturation

The approach capacity (C) is obtained by multiplication of the saturation flow with the green ratio (g/c) for each approach; see equation (1) above.

The degree of saturation (DS) is obtained as:

$$DS = Q/C = (Q \times c) / (S \times g) \qquad (6)$$

f. Traffic performance

Different measures of traffic performance can be determined based on the traffic flow (Q), degree of saturation (DS) and signal timing (c and g) as described below:

Queue Length

The average number of queuing puck at the beginning of green NQ is calculated as the number of pcu remain from the previous green phase NQ1 plus the number of pcu that arrive during the red phase (NQ2):

$$NQ = NQ1 + NQ2 \qquad (7)$$

With

$$NQ1 = 0.25 \text{ x } C \text{ x } \left[(DS-1) + \sqrt{((DS-1)2 + ((8x(DS-0.5))/C)} \right] \qquad (8)$$

If DS > 0.5, otherwise NQ 1 = 0

$$NQ2 = c \text{ x } \frac{1-GR}{1-GR \text{ X } DS} * \frac{Q}{3600} \qquad (9)$$

Where:

NQ1 = number of pcu that remain from the previous green phase

NQ2 = number of pcu that arrive during the red phase

DS = degree of saturation

GR = green ratio

c = cycle time

C = capacity (pcu/h) = saturation flow times the green ratio (S x GR)

Q = traffic flow in the approach (pcu/h).

For design purposes the manual includes provision for adjustment of this average value to a desired level of probability for overloading.

The resulting queue length QL is obtained by multiplication of NQ with the average area occupied per pcu (20 sqm) and division with the entry width.

$$QL = c \text{ x } \frac{NQ_{MAX} \text{ x } 20}{W_{ENTRY}} \quad (10)$$

Where:

NQ max = maximum number of queuing

Use Figure 2.5 below to adjust NQ with regard to the desired probability for overloading P_{OL}(%)

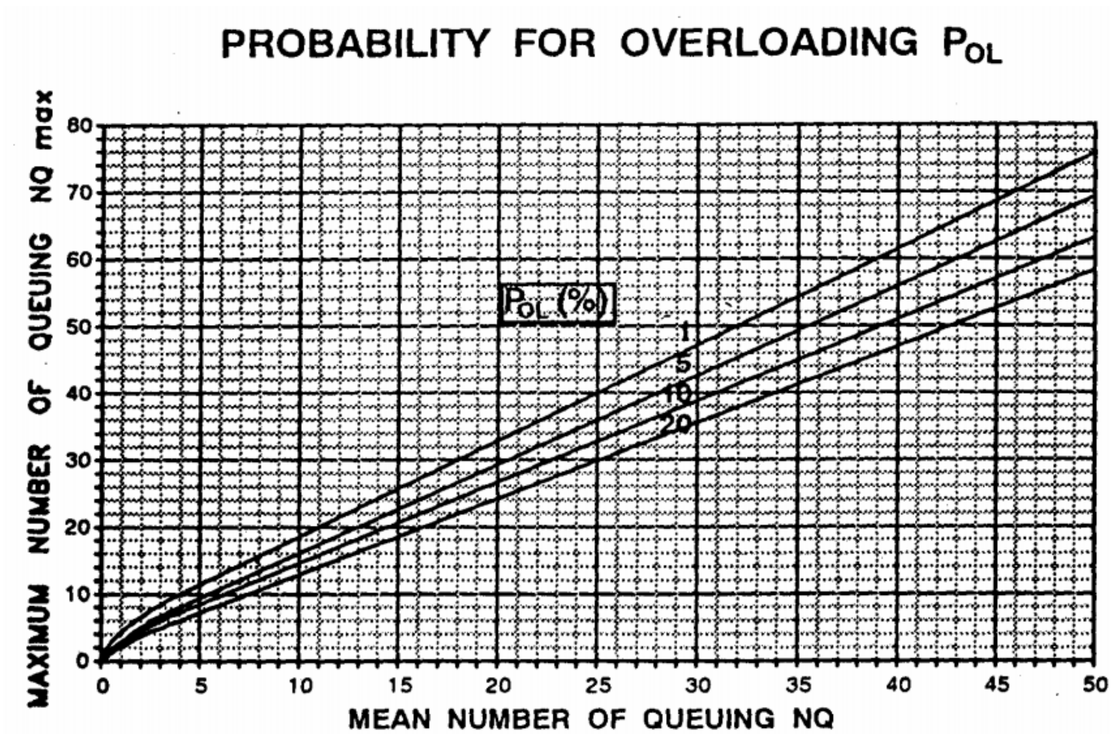


Figure 2.8 Calculation of no. of queuing pcu NQmax

Stop Rate

The stop rate (NS), i.e. the average number of stops per vehicle (including multiple stop in aqueue) before passing the intersection, is calculated as

$$NS = 0.9 * \frac{NQ}{Q*c} * 3600 \quad (11)$$

Where: c is the cycle time (Sec)

Q the traffic flow (pcu/h) in the studied approach.

Number of stopped vehicles Nsv

Nsv = Q x NS (pcu/ h) (12)

The proportion of stopped vehicles psv, i.e. the ratio of ratio of vehicles that have to stop because of the red signal before the intersection, is calculated as

$$psv = \min(NS, 1)$$

Where: NS is the stop rate in the approach.

Delay

Delay D at an intersection can occur for two reasons:

a. **Traffic Delay (DT)** due to traffic intersection with other movement in the junction
b. **Geometric Delay (DG)** due to deceleration and acceleration when making a turn in the intersection and/or when being stopped by the red light.

The average delay for an approach j is calculated as:

$$Dj = DTj + DGj \quad (13)$$

Where:

Dj = Mean delay for approach j (sec/pcu)
DTj = Mean traffic delay for approach j (sec/pcu)
DGj = Mean geometric delay for approach j (sec/pcu)

The average traffic delay for an approach j can be determined from the following formula (based on Akcelik, 1988).

$$DT = c * A + \frac{NQ_1 * 3600}{C} \quad (14)$$

Where:

DT = Mean traffic delay for approach j (sec/pcu)
c = Adjusted cycle time(sec)
A $= \frac{0.5 * (1 - GR)^2}{(1 - GR \times DS)}$
GR = Green ratio (g/c)

DS = Degree of Saturation

NQ1 = Number of pcu that remain from the previous green phase

C = Capacity (pcu/h)

Observe that the calculation result are not valid if the capacity of the intersection is influenced by "external" factors such as blocking of an exit due to downstream congestion, manual police control etc.

Average Geometric Delay

The average geometric delay for an approach j can be estimated as follows:

$$DGj = (1 - psv) \times pt \times 6 + (psv \times 4) \quad (15)$$

Where:

DGj = Mean geometric delay for approach j (sec/pcu)

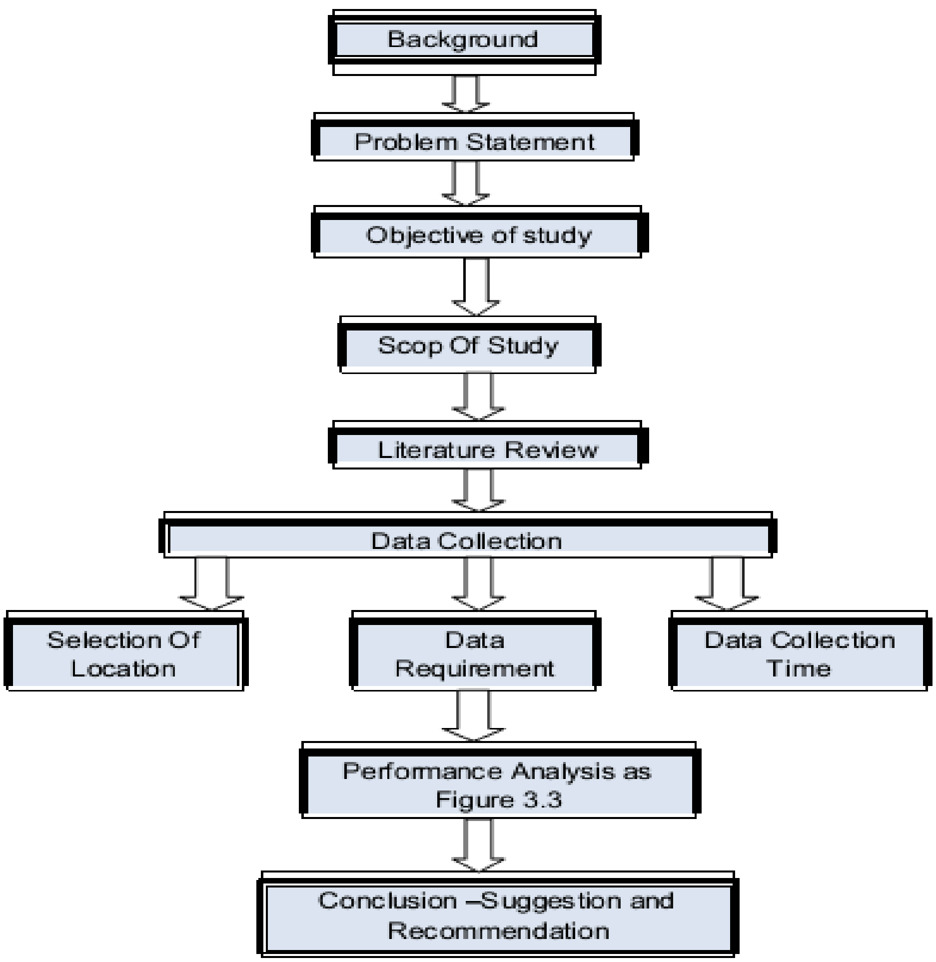
psv = Proportion of stopped vehicle in the approach

pt = Proportion of turning vehicle in the approach

CHAPTER 3
METHODOLOGY

3.1 Study Outline

Figure 3.1 presents the flow chart of an overview of the study. The objective of this research is to improve transportation planning level techniques for the assessment of the traffic signal coordination system on congestion. Specifically, This study aimed to analyze traffic flow system in a study area location to get green that optimized signal timing in that network. Study scope cover survey implementation traffic such as volume calculation saturated flow road and rate have been classified in peak hour.



Fiugre 3 . 1 Flow Chart for The Study

3.2 Selection Of Location:

Benghazi City contains a lot of Beaches monuments and places.In the holiday times the roads are in the city because a lot of people come to spend the holiday in Benghazi. And especially the city is costal , tourist city and also close to the Seaport . This study requires area where there are straight path with successive nodes. So the selection of the study area is based on the following criteria:

i. Roads with a variation of traffic volume.

ii. Good access and safety for the enumerators and equipment during the data collection process.

iii. Good overhead vantage points for video recording purposes.

Figure.3-2 Alselmani intersection in Benghazi city

3.3 Data Collection

3.3.1 Data Collection Equipment and Method:

For data collection at signalized intersections we need two cameras and trumpeter that Record data simultaneously but because of there was not enough equipment we decided to collect data in different time but with consideration almost situation in time and place of installing cameras. So the peak hour traffic flow in intersection morning peak , noon off peak hour and afternoon peak hour recorded. The location of the cameras was influenced by logistics such as visibility, building, etc. During the survey below data was collected for the video recording method, the video camera was set up at each junction and intersections at a distance of approximately 1.8 meters above the ground level over-looking the intersection. An external time device was attached to the camera to provide a permanent record. Efforts were made to position the camera so that all the vehicles on the observed roadways could be observed. However, this was a compromise between satisfying this requirement and the camera resolution

3.3.2 Data collection time:

The times for data collection during the morning peak , off peak hours and afternoon peak hours. Data were collected during weekday's periods in the evening at peak and Off peak hours. Field research was at one day starting in early 8 December 2012 Saturday in morning the period of time between the hours of 7:00 am to 8:30 am, afternoon in the time period between 13:30 pm to15: 00 pm , evening in the time period between 14:15 pm to 17:45 pm.

3.4 Problem Identification

Problems at an intersection are identified through a survey at congested signalized intersections, field investigations, and preliminary operational and safety analysis. To determine whether a problem exists, this information needs to be evaluated against defined goals or standards. A problem statement can be defined after a review of the established operational and safety criteria against the known characteristics of an intersection. In some cases, additional data may need to be collected to confirm that a problem exists

3.5 Data And Performance Analysis

Calculations in this study to evaluate the performance of signalized intersections are using the rules set by IHCM 1997. The existing data consist of geometric, traffic flow, side friction condition, the phase, green time. The result of calculation consists of capacity, degree of saturation, average no. of stops, mean average intersection delay and Almost all of the intersections show lower performance. Figure 3.3 below is a flow chart of procedures for calculating the performance analysis.

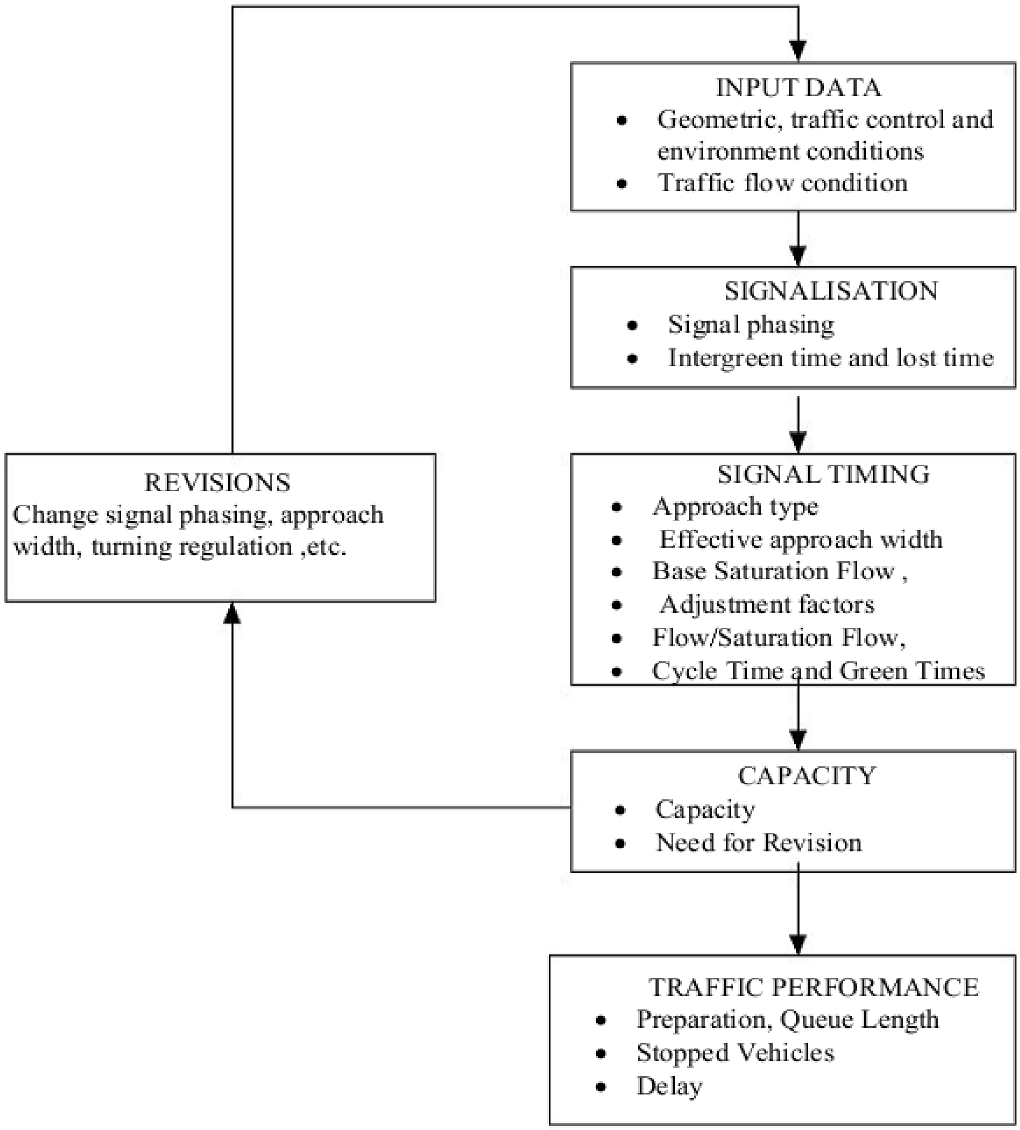


Figure 3.3 Flow chart of the performance analysis of signal intersections

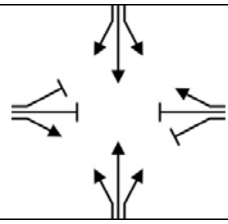
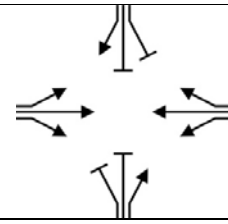
CHAPTER IV
ANALYSIS OF DATA AND RESULTS

4.1 Data geometric conditions of roads and intersections.

Data geometric condition of roads and intersections were obtained by direct measurement of the standard geometric parameters of road and intersection segments of roads and intersections affected by traffic expected.

Table 4.1:Geometry, traffic arrangements and environmental conditions for intersection

City Size	1.008700 million			
Approach Code	North	South	East	West
Median Y/N	Y	Y	Y	Y
Side Friction	L	L	L	L
Road Environment	COM	RES	COM	COM
Gradient %	0	0	0	0
Approach WA (m)	8	8	9	9
Entry WEntry (m)	5	5	6	6
RT on Red WRTOR (m)	3	3	3	3
Wexit (m)	5	5	5	5

Phase 1	Phase 2
	

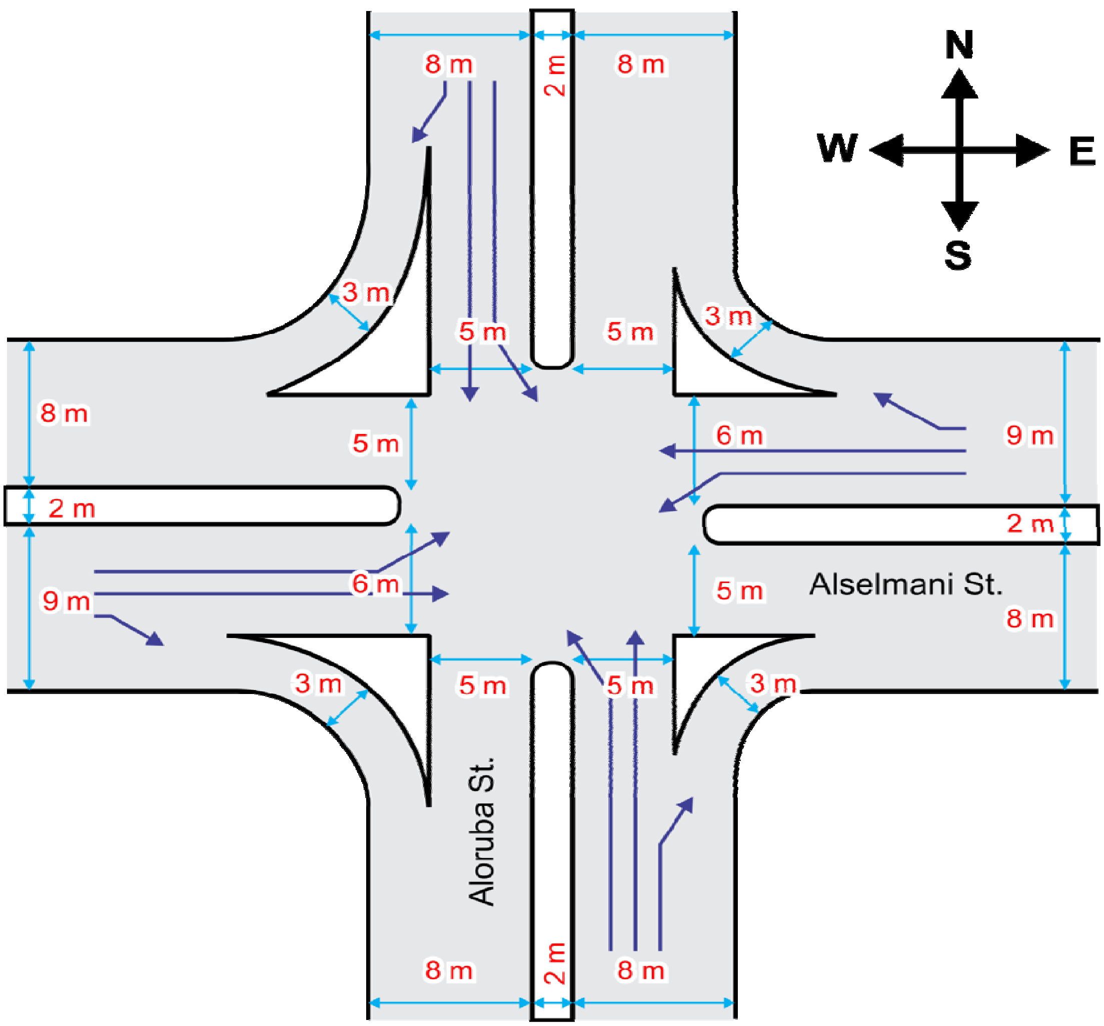


Figure 4.1 Geometric Condition Of Alselmani Intersection (Two Phase)

4.2 Data setting phase to the signalized intersections(existing condition).

Data on the phase settings for the traffic signalized intersection is obtained by direct measurement of the phase duration of traffic lights at the existing intersections.

Table 4.2: Phase time, green, and cycle time for intersection

Approach Code	Phase time		No. of phases	Cycle Time
	Green	Intergreen		
North	22	4	Phase (1)	54
South	22	4		54
East	24	4	Phase (2)	54
West	24	4		54

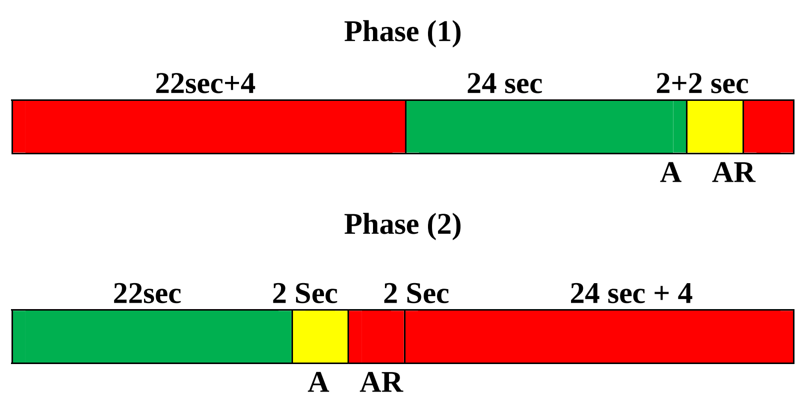


Table 4.3 The Results of existing condition (2 Phase)

Data		
Cycle Time (sec)		54
Green Time (N)		22
Green Time (S)		22
Green Time (E)		24
Green Time (W)		24
Intergreen (sec)		4
Queue Length (m)		87
Average.no of stop (sec/pcu)		1.03
Average Delay Sec/pcu	**N**	41.06
	S	18.30
	E	34.92
	W	22.47
Stop Rate Stop/pcu	**N**	1.24
	S	0.78
	E	1.14
	W	0.90
Average Intersection delay (sec/pcu)		64.30
Degree Saturation	**N**	0.925
	S	0.66
	E	0.92
	W	0.81

4.3 Data of volume traffic flow for the intersections.

Data volume of traffic crossing the road or obtained by surveying the traffic counting on roads and intersections. The implementation of a traffic count survey conducted on Saturday 8 December 2012 in morning the period of time between the hours of 7:00 am to 8:30 am, afternoon in the time period between 13:30 pm to15: 00 pm , evening in the time period between 14:15 pm to 17:45 pm.

Tables 4.4 The peak hours and off-peak hours

Morning Period 7:00 – 8:30 Am

Time	Direction		Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
7:00 -8:00	N	LT	91	74	3	3577
		ST	461	64	3	
		RT	131	52	0	
	S	LT	79	74	6	
		ST	205	93	1	
		RT	67	60	2	
	E	LT	153	114	7	
		ST	408	173	4	
		RT	174	117	3	
	W	LT	110	66	9	
		ST	391	145	12	
		RT	135	81	9	
7:15 -8:15	N	LT	103	70	5	3681
		ST	456	77	4	
		RT	142	44	0	
	S	LT	89	68	5	
		ST	225	102	4	
		RT	85	59	3	
	E	LT	168	120	8	
		ST	422	147	7	
		RT	191	99	4	
	W	LT	133	70	11	
		ST	402	125	8	
		RT	137	82	6	
7:30 -8:30	N	LT	**109**	**68**	**5**	**3780**
		ST	**465**	**78**	**3**	
		RT	**140**	**47**	**3**	
	S	LT	**104**	**62**	**4**	
		ST	**236**	**99**	**4**	
		RT	**88**	**55**	**3**	
	E	LT	**164**	**113**	**5**	
		ST	**443**	**150**	**5**	
		RT	**202**	**98**	**4**	
	W	LT	**145**	**84**	**10**	
		ST	**413**	**132**	**6**	
		RT	**137**	**90**	**6**	

Afternoon Period 13:30 – 15:00 Pm

Time	Direction		Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
13:30 – 14:30	N	LT	**107**	**77**	**6**	**3750**
		ST	**331**	**88**	**4**	
		RT	**129**	**86**	**3**	
	S	LT	**107**	**74**	**4**	
		ST	**368**	**84**	**1**	
		RT	**124**	**100**	**4**	
	E	LT	**134**	**89**	**4**	
		ST	**382**	**180**	**6**	
		RT	**130**	**107**	**11**	
	W	LT	**158**	**58**	**10**	
		ST	**412**	**133**	**3**	
		RT	**136**	**95**	**5**	
13:45 -14:45	N	LT	113	82	3	3630
		ST	349	89	1	
		RT	117	95	2	
	S	LT	109	70	5	
		ST	385	75	2	
		RT	122	101	1	
	E	LT	136	77	2	
		ST	351	170	4	
		RT	139	102	10	
	W	LT	133	45	11	
		ST	398	120	3	
		RT	126	79	3	
14:00 – 15:00	N	LT	111	94	5	3532
		ST	343	92	1	
		RT	110	103	0	
	S	LT	105	70	4	
		ST	351	83	4	
		RT	120	84	1	
	E	LT	130	74	1	
		ST	369	157	3	
		RT	146	116	7	
	W	LT	122	45	9	
		ST	369	105	3	
		RT	110	82	3	

Evening Period : 16:15 – 17:45 Pm

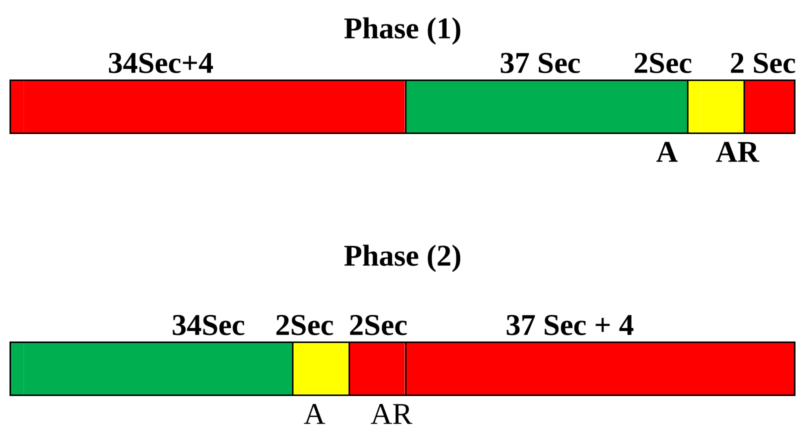
Time	Direction		Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
16:15-17:15	N	LT	97	63	6	2742
		ST	274	69	5	
		RT	88	82	3	
	S	LT	79	65	6	
		ST	237	98	3	
		RT	80	65	2	
	E	LT	80	70	6	
		ST	159	63	6	
		RT	105	68	6	
	W	LT	115	56	9	
		ST	267	126	8	
		RT	145	71	10	
16:30-17:30	N	LT	**116**	**70**	**4**	**2763**
		ST	**284**	**74**	**4**	
		RT	**101**	**82**	**3**	
	S	LT	**89**	**67**	**5**	
		ST	**215**	**97**	**4**	
		RT	**88**	**64**	**3**	
	E	LT	**96**	**82**	**7**	
		ST	**186**	**79**	**7**	
		RT	**127**	**68**	**7**	
	W	LT	**106**	**60**	**11**	
		ST	**228**	**116**	**7**	
		RT	**137**	**62**	**7**	
16:45-17:45	N	LT	104	63	6	2530
		ST	248	67	4	
		RT	94	78	3	
	S	LT	104	64	4	
		ST	165	89	4	
		RT	81	55	3	
	E	LT	119	91	5	
		ST	185	92	5	
		RT	145	77	7	
	W	LT	70	56	10	
		ST	168	86	4	
		RT	117	50	7	

Due to the similarity of data traffic volume were taken in the morning peak hour, and then multiply the volume of traffic into (0.75, 0.50, 0.25) to get a low traffic volume.

4.4 The Calculation Of Morning Peak Hour(7:30 – 8:30)

a. Very High Traffic Flow

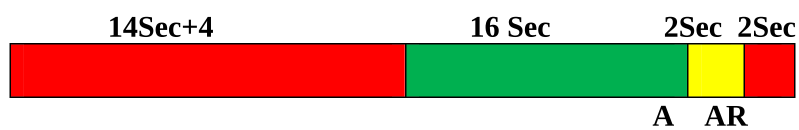
Approach Code	Phase time		No. of phases	Cycle Time
	Green	Intergreen		
North	34	4	phase 1	79
South	34	4		79
East	37	4	Phase 2	79
West	37	4		79



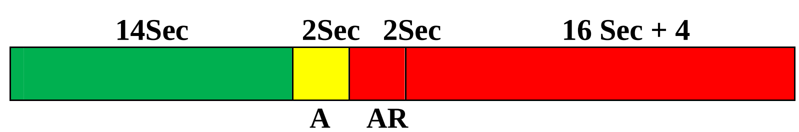
b. High Traffic Flow (traffic volume x 0.75)

Approach Code	Phase time		No. of phases	Cycle Time
	Green	Intergreen		
North	14	4	phase 1	38
South	14	4		38
East	16	4	Phase 2	38
West	16	4		38

Phase (1)



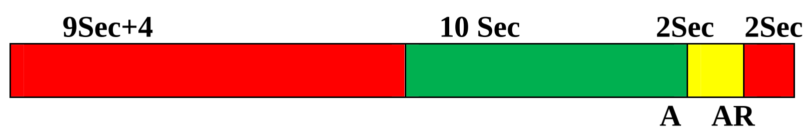
Phase (2)



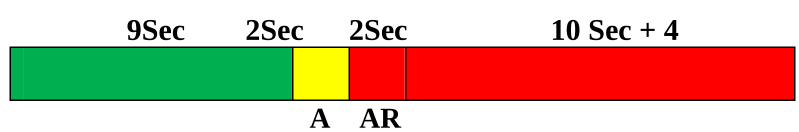
c. Low Traffic Flow (traffic volume x 0,50)

Approach Code	Phase time		No. of phases	Cycle Time
	Green	Intergreen		
North	9	4	phase 1	27
South	9	4		27
East	10	4	Phase 2	27
West	10	4		27

Phase (1)



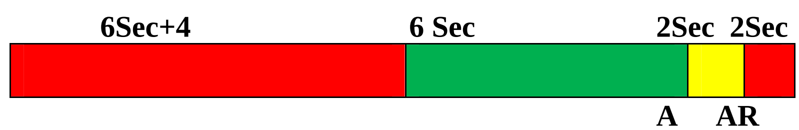
Phase (2)



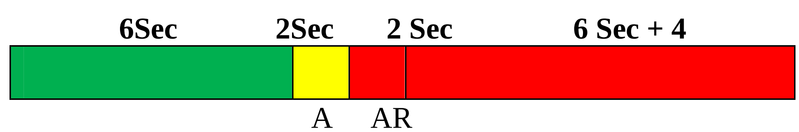
d. Very Low Traffic Flow (traffic volume x 0.25)

Approach Code	Phase time		No. of phases	Cycle Time
	Green	Intergreen		
North	6	4	phase 1	20
South	6	4		20
East	6	4	Phase 2	20
West	6	4		20

Phase (1)



Phase (2)

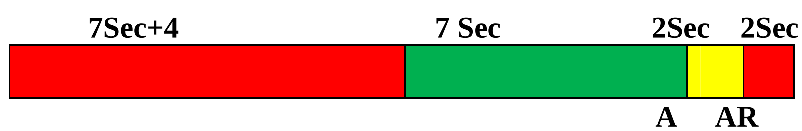


Note:

The minimum green time based on (IHCM 1997) is 7 Sec so the cycle time for very low traffic flow become as:

Approach Code	**Phase time**		**No. of phases**	**Cycle Time**
	Green	**Intergreen**		
North	7	4	phase 1	22
South	7	4		22
East	7	4	Phase 2	22
West	7	4		22

Phase (1)



Phase (2)

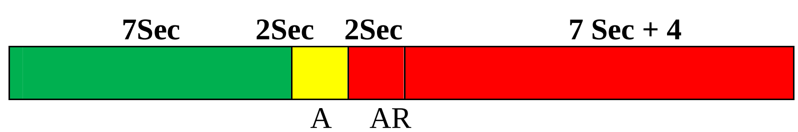


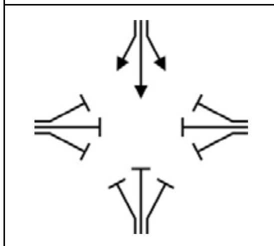
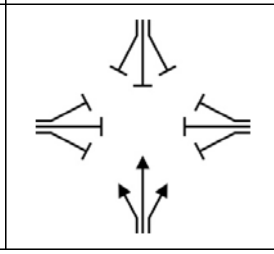
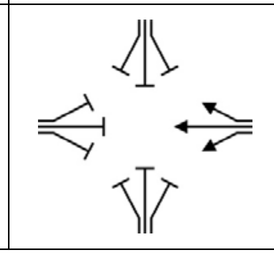
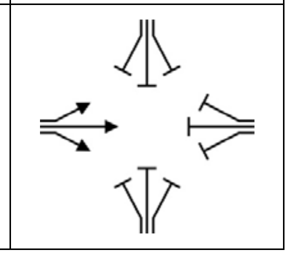
Table 4.5: The Results of Intersection (2 Phase)

Data		**Traffic Condition**				
		Very High	**High**	**Low**	**Very Low**	
Cycle Time (sec)		79	38	27	20	22
Green Time (N, S)		34	14	9	6(min 7)	7
Green Time (E, W)		37	16	10	6(min 7)	7
Intergreen (sec)		4	4	4	4	4
Queue Length (m)		108	42	20	11	11
Average.no of stop (sec/pcu)		0.81	0.81	0.68	0.68	0.68
Average Delay Sec/pcu	**N**	36.1	17.1	9.75	8.42	8.60
	S	22.5	12.45	9.32	8.31	8.50
	E	32.1	14.67	9.32	8.53	8.70
	W	25.12	14.73	9.1	8.46	8.60
Stop Rate Stop/pcu	**N**	0.96	0.88	0.70	0.68	0.66
	S	0.72	0.70	0.66	0.66	0.65
	E	0.92	0.81	0.68	0.68	0.68
	W	0.80	0.8	0.66	0.68	0.66
Average Intersection delay (sec/pcu)		29.47	14.89	9.4	8.45	8.61
Degree Saturation	**N**	0.87	0.71	0.46	0.25	0.23
	S	0.68	0.50	0.32	0.17	0.16
	E	0.87	0.69	0.46	0.26	0.25
	W	0.77	0.67	0.42	0.24	0.23

Changed signal phasing

An alternative phasing scheme with separate phase for left-turning traffic might be appropriate. Introduction of separate phases for left- turning traffic may have to accompanied with widening measures as well.

Geometric Condition Of Alselmani Intersection (Four Phase type 1)

Phase 1	Phase 2	Phase 3	Phase 4
			

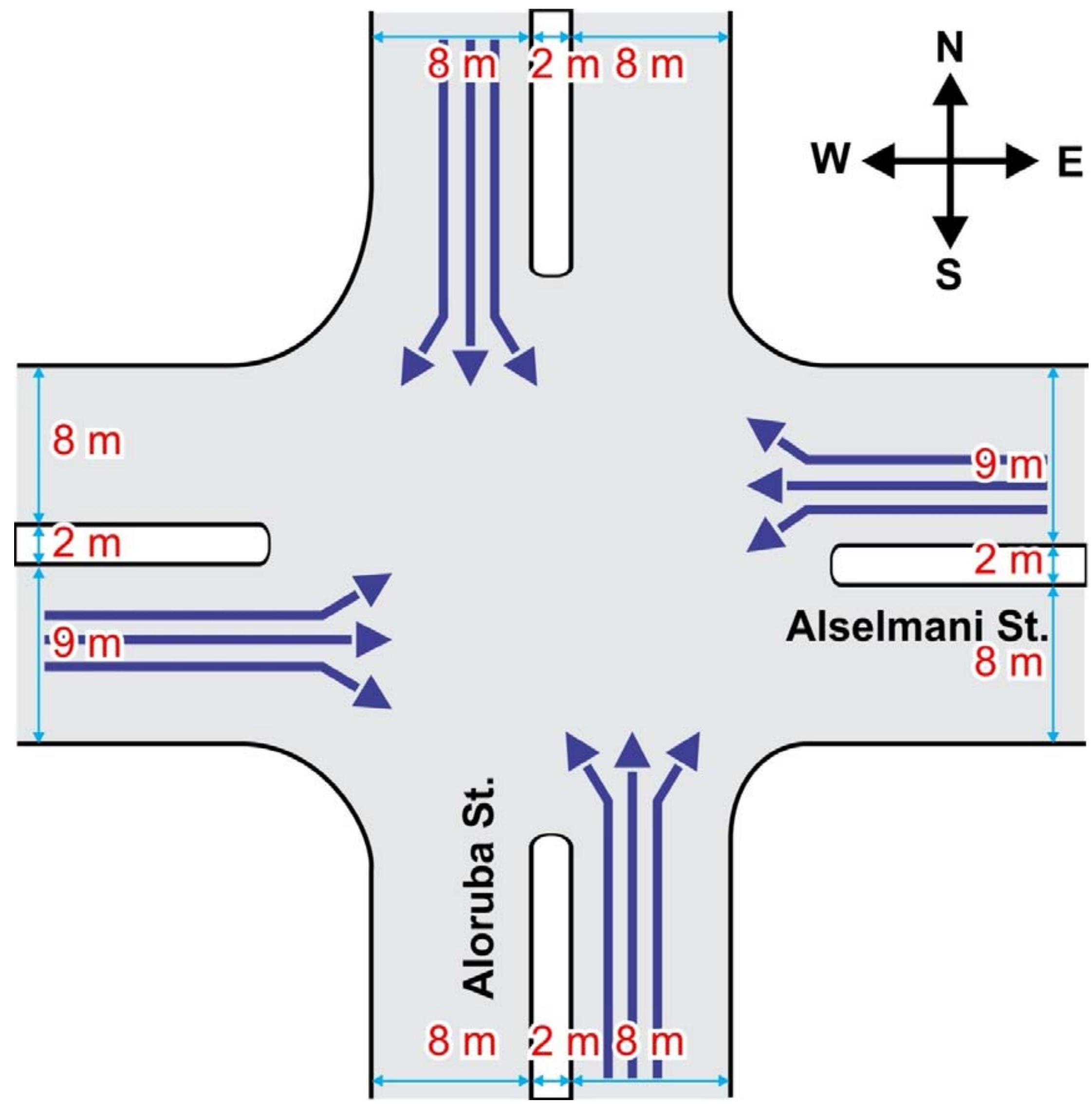
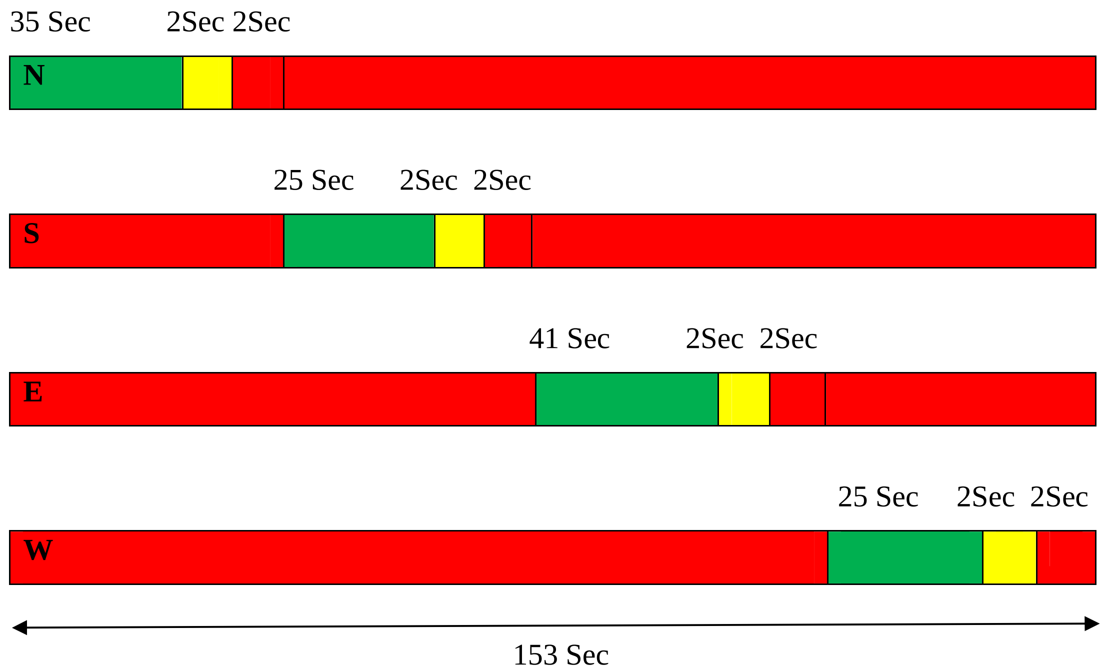


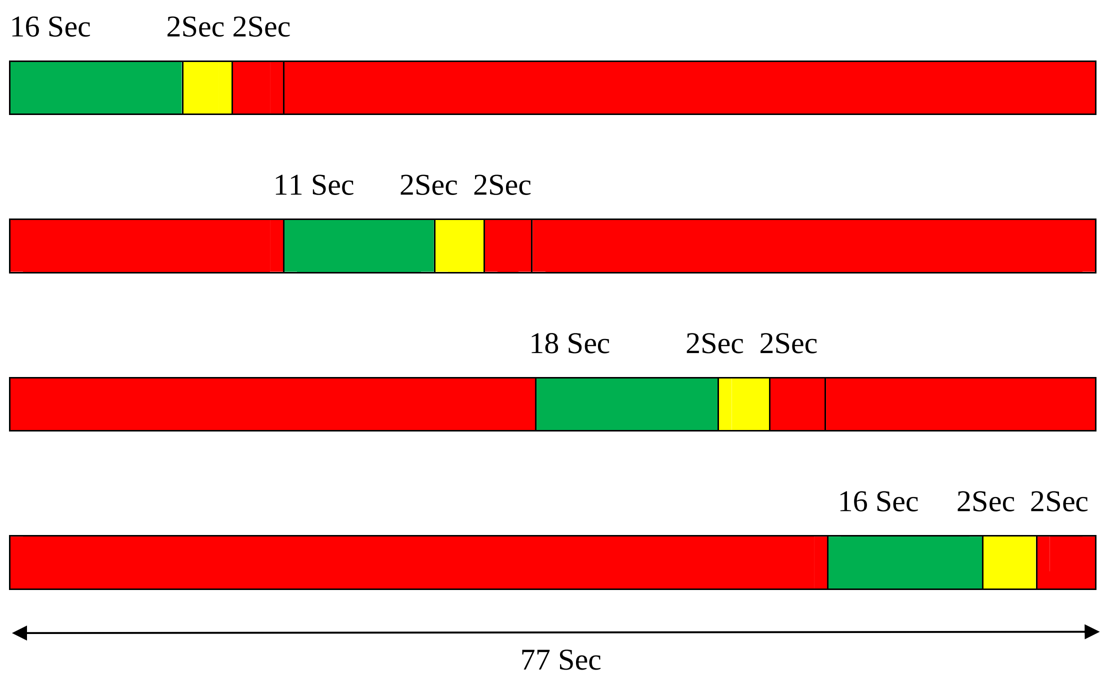
Figure 4.2 Geometric Condition Of Alselmani Intersection (Four Phase)

The Sequence Diagram For Four Phase

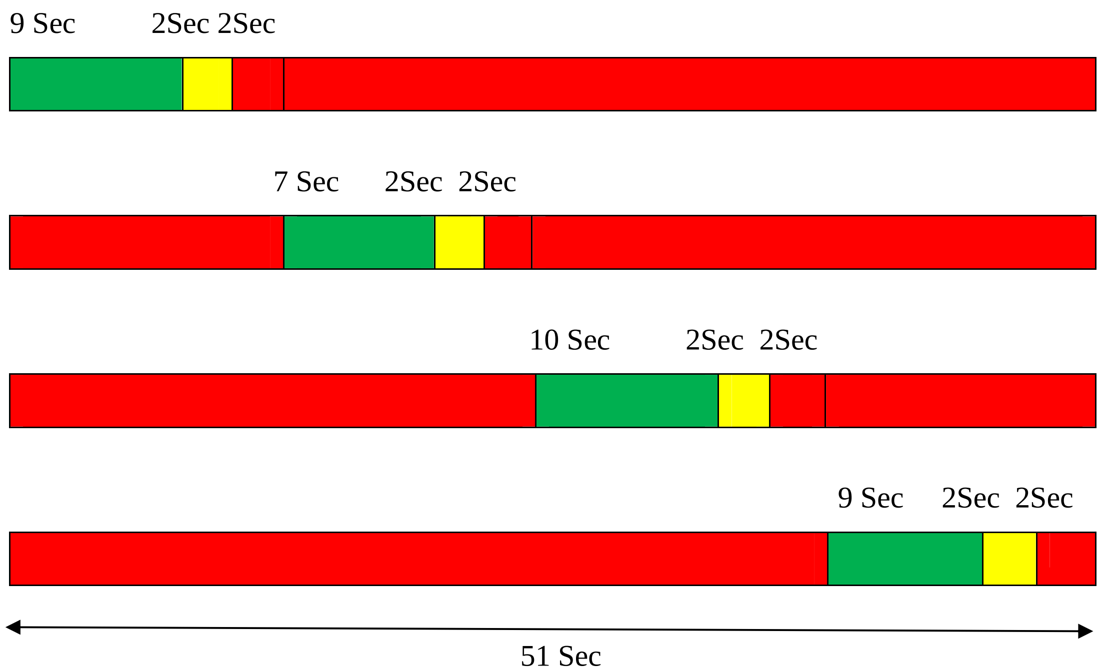
1. **Very High Trffic Flow**



2. **High Trffic Flow**



3. Low Trffic Flow



a. Very Low Trffic Flow

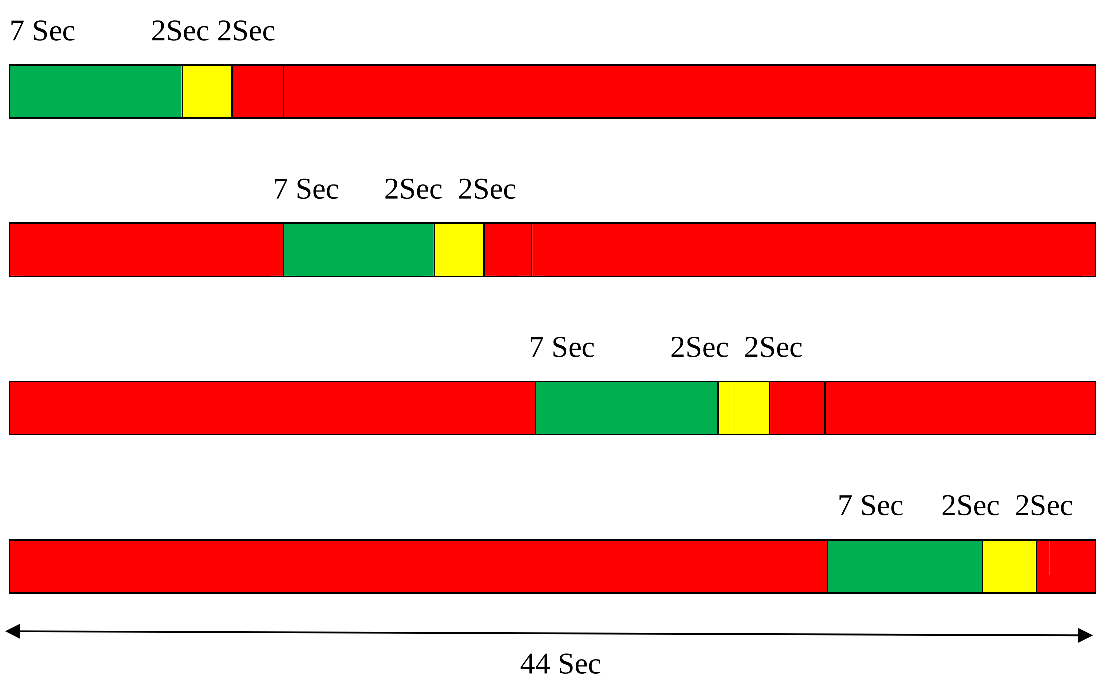


Table 4.6 The Results of Intersection (4 Phase)

Type(1)

Data		**Traffic Condition**				
		Very High	**High**	**Low**	**Very Low**	
Cycle Time (sec)		153	77	51	36	44
Green Time (N)		35	16	9	5(min=7)	7
Green Time (S)		25	11	7	4(min= 7)	7
Green Time (E)		41	18	10	6(min=7)	7
Green Time (W)		35	16	9	5(min=7)	7
Intergreen (sec)		4	4	4	4	4
Queue Length (m)		173	67	31	16	17
Average.no of stop (sec/pcu)		0.95	0.93	0.86	0.81	0.81
Average Delay Sec/pcu	**N**	73.29	38.62	23.96	17.61	19.88
	S	85.5	43.91	24.91	18.35	19.59
	E	66.96	37.30	23.82	16.59	20.07
	W	61.52	36.34	24.00	17.63	19.84
Stop Rate Stop/pcu	**N**	0.96	0.94	0.86	0.82	0.80
	S	1.00	0.98	0.87	0.83	0.79
	E	0.93	0.91	0.87	0.79	0.81
	W	0.93	0.91	0.86	0.82	0.79
Average Intersection delay (sec/pcu)		70.25	38.52	24.10	17.42	19.88
Degree Saturation	**N**	0.90	0.77	0.60	0.38	0.33
	S	0.91	0.80	0.56	0.34	0.24
	E	0.90	0.80	0.64	0.31	0.38
	W	0.91	0.77	0.61	0.38	0.33

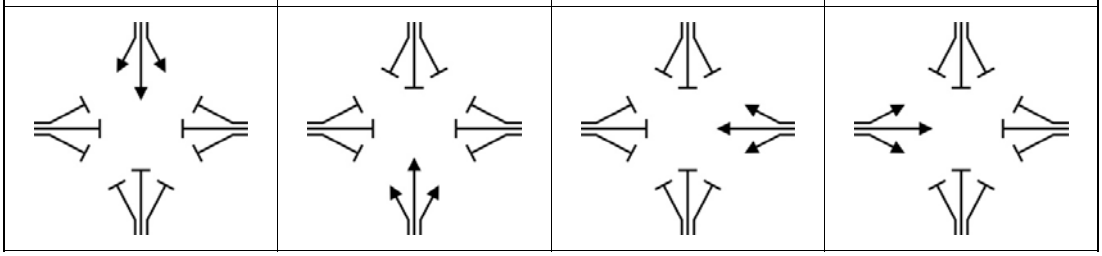
Comment:

Form the results that the Intersection Flow Ratio IFR = 0.808, leading

to a calculated cycle time for minimum delay of 153 sec. The existing timing can therefore be expected to lead to long queues. finally the level of performance with the new signal timing. As can be seen, the intersection is still heavily congested with queue-lengths in the order of 173 m, average proportion of stopped vehicles 0.93, and average delay 70.25 sec.

Geometric Condition Of Alselmani Intersection (Four Phase type 2)

Phase 1	Phase 2	Phase 3	Phase 4



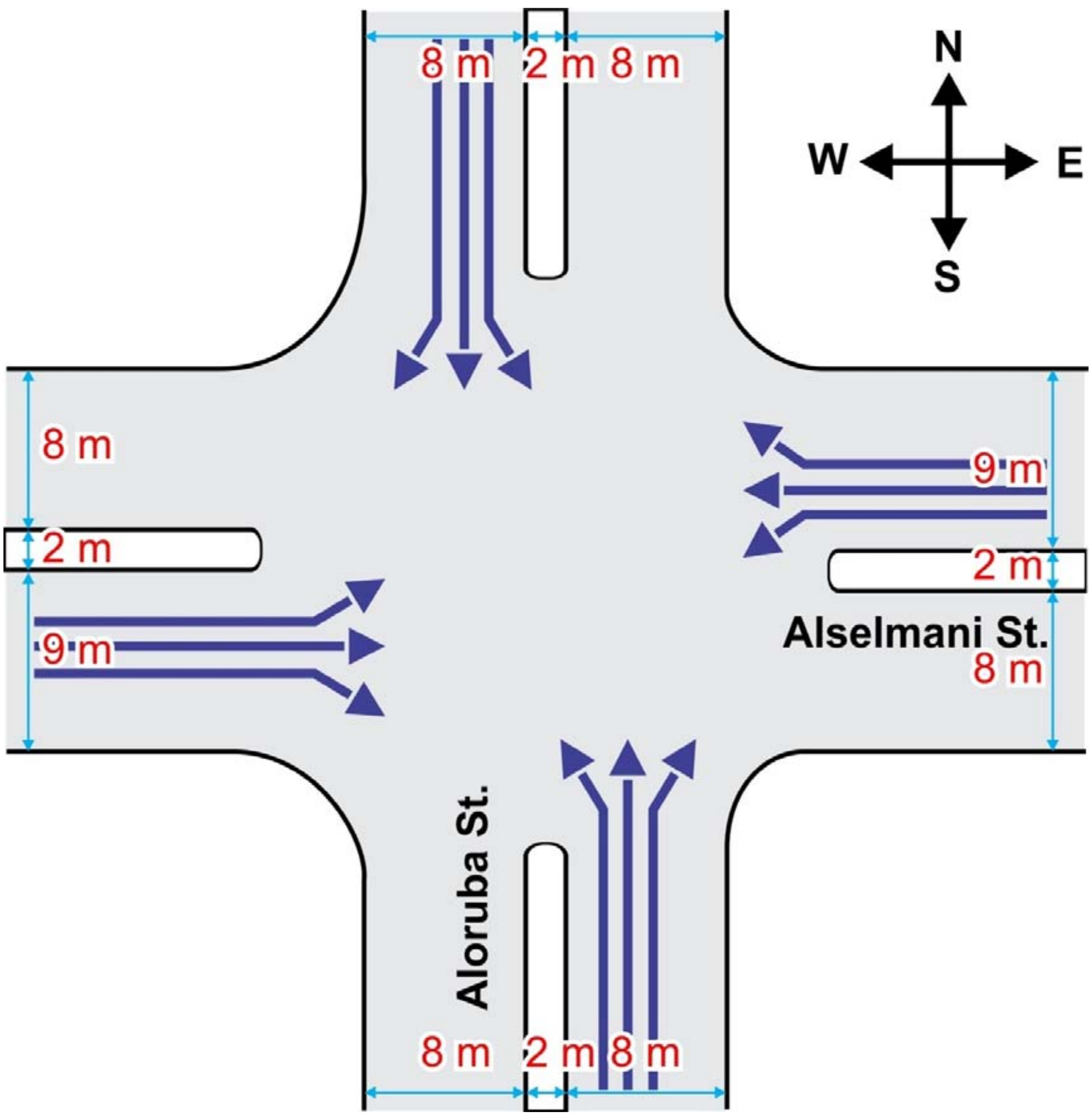


Figure 4.3 Geometric Condition Of Alselmani Intersection (Four Phase)

Table 4.7 The Results of Intersection (4 Phase)

Type(2)

Data		Traffic Condition			
		Very High	High	Low	Very Low
Cycle Time (sec)		175	78	53	45
Green Time (N)		51	20	11	7
Green Time (S)		22	9	7	7
Green Time (E)		57	22	12	8
Green Time (W)		29	11	7	7
Intergreen (sec)		4	4	4	4
Queue Length (m)		264	87	44	23
Average.no of stop (sec/pcu)		0.95	0.92	0.85	0.85
Average Delay Sec/pcu	N	135.35	53.90	24.93	19.76
	S	106.85	47.03	24.7	20.67
	E	118.65	56.09	28.02	19.97
	W	85.38	42.85	24.94	19.81
Stop Rate Stop/pcu	N	1.20	1.12	0.88	0.81
	S	1.05	1.03	0.886	0.79
	E	1.12	1.15	0.92	0.80
	W	0.94	0.98	0.83	0.79
Average Intersection delay (sec/pcu)		81	37.46	24.2	19.2
Degree Saturation	N	0.91	0.78	0.64	0.43
	S	0.83	0.68	0.43	0.28
	E	0.925	0.81	0.67	0.42
	W	0.81	0.70	0.58	0.37

Comments:

The control was changed from two-phase to four-phase control during , leading to excessive queuing in the area. The calculations for four phase control shows the reason why, the Intersection flow ratio increases , and the degree of saturation becomes higher in all approaches during the studied peak hour. The reason to change from two-phase to four-phase control was probably that the there were many traffic accidents in the intersection. An alternative action would have been to increase the inter-green periods between the phases, and to keep two-phase control which obviously results in a much higher intersection capacity and better level of performance level.

The advantage of changed to four phase: inevitably leads to an increase of the cycle time and of the ratio of time allocated to switching between phases (Intergreen). Although this may be beneficial from the traffic safety point of view, it normally means that the overall capacity of the intersection is decreased.

CHAPTER V
CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

All of the data inserted and analyzed by using IHCM. The result of calculation consists of a number of phases, cycle length, green time, average intersection delay and average no. of stops. The intersections show the low performance in existing data case by the highest numbers in average intersection delay and average no. of stops.

The performance of the intersections will be expected to perform better by changing the phases from two phase to four phase which were calculated using IHCM for Alselmani intersection, so the change from two phase control to four phase control gave an increase of the cycle time and of the ratio of time allocated to switching between phases (Intergreen). Although this may be beneficial from the traffic safety point of view, it normally means that the overall capacity of the intersection is decreased.

5.2 Suggestion And Recommendation

a. Implementation of traffic management such as improved geometric (without widening the means) and movement arrangements need to be done because it will provide significant performance improvements at the intersection signalized intersection and intersection-existing
b. It is recommended the study area is extended to conclude for more intersections and network in order to observe the role of a coordination system for traffic performance on network to avoid the many problems such as congestion.
c. The last recommendation is to change from private transportation to public transportation to get rid of congestion and blocking traffic on the roads.

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APPENDIX A

This appendix includes the Survey traffic data flow at the Intersection

1. North

a. Morning period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
7:00 -7:15	LT ST RT	18 112 32	20 11 19	0 1 0	38 124 51
7:15 -7:30	LT ST RT	21 119 29	12 19 10	2 2 0	35 140 39
7:30 -7:45	LT ST RT	26 104 31	22 15 15	1 0 0	49 119 46
7:45 -8:00	LT ST RT	26 126 39	20 19 8	1 0 0	47 145 47
8:00 -8:15	LT ST RT	30 116 43	16 24 11	1 2 0	47 142 54
8:15 -8:30	LT ST RT	27 119 27	10 20 13	1 1 3	38 140 43

b. Afternoon period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
13:30 – 13:45	LT ST RT	24 80 39	12 16 20	3 3 1	39 99 60
13:45 -14:00	LT ST RT	26 95 31	17 21 17	0 0 0	43 116 48
14:00 – 14:15	LT ST RT	34 83 32	20 20 26	2 0 0	56 103 58
14:15 -14:30	LT ST RT	23 73 27	28 31 23	1 1 2	52 105 52
14:30 -14:45	LT ST RT	30 98 27	17 17 29	0 0 0	47 115 56
14:45- 15:00	LT ST RT	24 89 24	29 24 25	2 0 0	55 116 49

Evening Period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
16:15 – 16:30	LT ST RT	14 43 14	10 12 16	3 2 1	27 57 31
16:30 -16:45	LT ST RT	36 77 31	16 21 17	0 1 0	52 99 48
16:45 – 17:00	LT ST RT	24 92 26	20 25 26	2 1 0	46 118 52
17:00 -17:15	LT ST RT	23 62 17	17 11 23	1 1 2	41 74 42
17:15 -17:30	LT ST RT	33 53 27	17 17 16	1 1 1	51 71 44
17:30- 17:45	LT ST RT	24 41 24	9 14 13	2 1 0	35 56 37

2. South

a. Morning period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
7:00 -7:15	LT ST RT	15 42 14	19 20 13	2 0 1	36 62 28
7:15 -7:30	LT ST RT	19 50 13	19 24 16	1 0 0	39 74 29
7:30 -7:45	LT ST RT	19 52 14	20 22 15	3 1 1	42 75 30
7:45 -8:00	LT ST RT	26 61 26	16 27 16	0 2 0	52 80 42
8:00 -8:15	LT ST RT	25 62 32	13 29 12	1 1 2	39 92 46
8:15 -8:30	LT ST RT	34 61 16	13 21 12	0 0 0	47 82 28

b. Afternoon period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
13:30 – 13:45	LT	22	20	0	42
	ST	90	32	0	122
	RT	35	17	3	55
13:45 -14:00	LT	32	18	1	51
	ST	106	15	0	121
	RT	35	34	0	69
14:00 – 14:15	LT	25	18	2	45
	ST	89	15	1	105
	RT	22	34	1	57
14:15 -14:30	LT	28	18	1	47
	ST	83	22	0	105
	RT	32	15	0	47
14:30 -14:45	LT	24	16	1	41
	ST	107	23	1	131
	RT	33	18	0	51
14:45- 15:00	LT	28	18	0	46
	ST	72	23	2	97
	RT	33	17	0	50

c. Evening Period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
16:15 -16:30	LT	15	11	2	28
	ST	52	20	0	72
	RT	14	13	1	28
16:30 -16:45	LT	19	16	1	36
	ST	71	29	0	100
	RT	23	21	0	44
16:45 -17:00	LT	19	12	3	34
	ST	53	22	1	76
	RT	14	15	1	30
17:00 -17:15	LT	26	26	0	52
	ST	61	27	2	90
	RT	29	16	0	45
17:15 -17:30	LT	25	13	1	39
	ST	30	19	1	50
	RT	22	12	2	36
17:30 -17:45	LT	34	13	0	47
	ST	21	21	0	42
	RT	16	12	0	28

3. East:

a. Morning period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
7:00 -7:15	LT	30	21	1	52
	ST	102	60	1	163
	RT	29	29	0	58
7:15 -7:30	LT	39	25	4	68
	ST	100	35	2	137
	RT	32	27	0	59
7:30 -7:45	LT	42	36	2	80
	ST	96	39	1	136
	RT	52	33	1	86
7:45 -8:00	LT	42	32	0	74
	ST	110	39	3	152
	RT	61	28	2	91
8:00 -8:15	LT	45	27	2	74
	ST	116	34	1	151
	RT	46	11	1	58
8:15 -8:30	LT	35	18	1	54
	ST	121	38	0	159
	RT	43	26	0	69

b. Afternoon period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
13:30 – 13:45	LT	28	30	2	60
	ST	119	39	2	160
	RT	30	32	1	63
13:45 -14:00	LT	42	18	1	61
	ST	89	49	1	139
	RT	33	26	3	62
14:00 – 14:15	LT	25	16	0	41
	ST	101	52	2	155
	RT	30	31	4	72
14:15 -14:30	LT	39	25	1	65
	ST	73	40	1	114
	RT	37	18	3	58
14:30 -14:45	LT	30	18	0	48
	ST	88	29	0	117
	RT	39	27	0	66
14:45- 15:00	LT	36	15	0	51
	ST	107	36	0	143
	RT	40	40	0	80

c. Evening Period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
16:15 -16:30	LT	19	15	1	35
	ST	30	18	0	48
	RT	21	11	0	32
16:30 -16:45	LT	17	9	3	29
	ST	47	15	2	64
	RT	18	17	0	35
16:45 -17:00	LT	22	26	2	50
	ST	36	11	1	48
	RT	30	22	4	56
17:00 -17:15	LT	22	20	0	42
	ST	46	19	3	68
	RT	36	18	2	56
17:15 -17:30	LT	35	27	2	64
	ST	57	34	1	92
	RT	43	11	1	55
17:30 -17:45	LT	40	18	1	59
	ST	46	28	0	74
	RT	36	26	0	62

4. West

a. Morning period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
7:00 -7:15	LT	24	10	1	35
	ST	98	49	4	151
	RT	26	20	3	49
7:15 -7:30	LT	28	13	2	43
	ST	102	30	2	134
	RT	39	23	2	64
7:30 -7:45	LT	30	13	1	44
	ST	88	30	4	122
	RT	46	24	2	72
7:45 -8:00	LT	28	30	5	63
	ST	103	36	2	141
	RT	24	14	2	40
8:00 -8:15	LT	47	14	3	64
	ST	109	29	0	138
	RT	28	21	0	49
8:15 -8:30	LT	40	27	1	68
	ST	113	37	0	150
	RT	39	31	2	72

b. Afternoon period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
13:30 – 13:45	LT	43	24	4	71
	ST	92	40	2	134
	RT	35	35	3	73
13:45 -14:00	LT	40	11	3	54
	ST	129	45	0	174
	RT	42	22	2	68
14:00 – 14:15	LT	45	14	1	60
	ST	101	35	1	137
	RT	32	23	0	55
14:15 -14:30	LT	30	9	2	41
	ST	90	13	0	103
	RT	27	15	0	42
14:30 -14:45	LT	18	11	5	34
	ST	78	27	2	107
	RT	25	19	1	45
14:45- 15:00	LT	29	11	1	41
	ST	100	30	0	130
	RT	26	25	2	53

Evening Period

Time	Direction	Light Vehicles (LV)	Heavy Vehicle (HV)	Motorcycles (MC)	Total
16:15 -16:30	LT	26	10	1	37
	ST	68	29	1	98
	RT	36	20	3	59
16:30 -16:45	LT	48	13	2	63
	ST	85	41	3	129
	RT	39	23	2	64
16:45 -17:00	LT	30	13	1	44
	ST	68	30	2	100
	RT	46	14	2	62
17:00 -17:15	LT	11	20	5	36
	ST	46	26	2	74
	RT	24	14	3	41
17:15 -17:30	LT	17	14	3	34
	ST	29	19	0	48
	RT	28	11	0	39
17:30 -17:45	LT	12	9	1	22
	ST	25	11	0	36
	RT	19	11	2	32

APPENDIX B

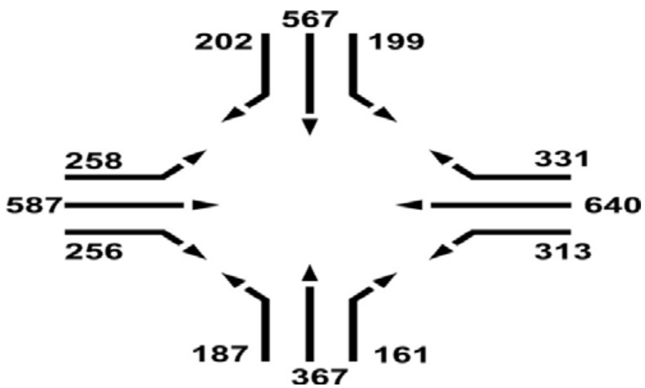
Form USIG

This appendix includes the signalized calculation

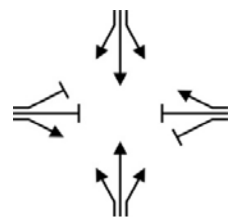
Calculations Of Existing Condition

Appr code	Dir.	TRAFFIC FLOW MOTORISED VEHICLES (MV)								
		Light Vehicles (LV)		Heavy Vehicles (HV)		Motorcycles (MC)		Total Motor vehicles MV	Ratio of turning	
		pce protected = 1.0 pce opposed = 1.0		pce protected = 1.3 pce opposed = 1.3		pce protected = 0.2 pce opposed = 0.4				
		veh/h	pcu/h Opp	veh/h	pcu/h Opp	veh/h	pcu/h Opp	pcu/h Opp	p LT Eq.(13)	p RT Eq.(14)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
N	LT/LTOR	109	109	68	88	5	2	199	0.21	
	ST	465	465	78	101	3	1	567		
	RT	140	140	47	61	3	1	202		0.21
	TOTAL							968		
S	LT/LTOR	104	104	62	81	4	2	187	0.26	
	ST	236	236	99	129	4	2	367		
	RT	88	88	55	72	3	1	161		0.23
	TOTAL							715		
E	LT/LTOR	164	164	113	147	5	2	313	0.24	
	ST	443	443	150	195	5	2	640		
	RT	202	202	98	127	4	2	331		0.26
	TOTAL							1284		
W	LT/LTOR	145	145	84	109	10	4	258	0.23	
	ST	413	413	132	172	6	2	587		
	RT	137	137	90	117	6	2	256		0.23
	TOTAL							1101		

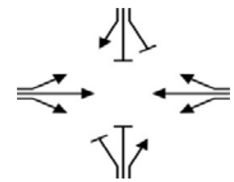
Traffic Flow Distribution



Phase (1)



Phase(2)



Appr Code	Green In Phase No	Appr type	Ratio of turning vehicles			RT-flow pcu/h		Eff width (m)	Saturation flow pcu/h						Traffic Flow pcu/h	Flow ratio FR	Phase Ratio	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
						Own dir	Oppos Dir		Base Value pcu/h	Adjustment factors All appr type				Adjusted Value pcu/h						
										City size	Side friction	Gradient	Parking							
			P_{LTOR}	P_{LT}	P_{RT}	Q_{LT}	Q_{LTO}	W_e	S_o Fig. 2.3 Fig. 2.4	F_{CS}	F_{SF}	F_G	F_P	S Eq.(2)	Q	Q/S	PR= FRcrit/IFR	g Eq.(5)	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	o		0.21	0.21	199	187	5	2140	1.00	0.95	1.00	1.00	2033	766	0.38	0.48	22	828	0.925
S	1	o		0.26	0.23	187	199	5	2120	1.00	0.98	1.00	1.00	2078	554	0.27		22	847	0.66
E	2	o		0.24	0.26	313	258	6	2460	1.00	0.95	1.00	1.00	2337	953	0.41	0.52	24	1039	0.92
W	2	o		0.23	0.23	258	313	6	2460	1.00	0.95	1.00	1.00	2337	845	0.37		24	1039	0.81
Total lost time				Unadjustment cycle time			C_{US} (sec) Eq.(2								IFR=					
LTI (sec) 8 Sec				Adjustment cycle time			C (sec) Eq.(3			54					$\sum FR_{CRIT}$					

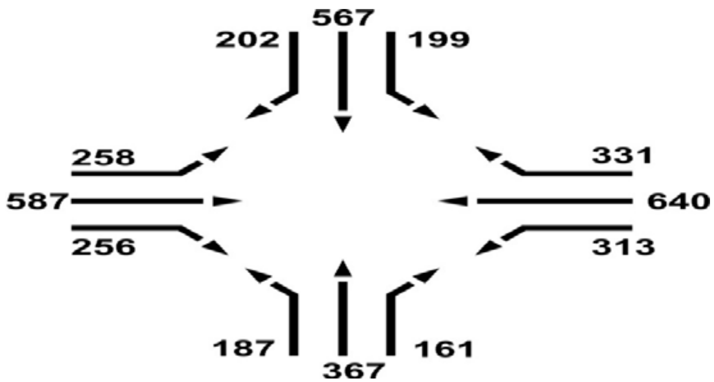
Approach Code (1)	Traffic flow pcu/h Q (2)	Capacity pcu/h C (3)	Degree of Saturation DS = Q/C (4)	Green Ratio GR = g/c (5)	No. of queuing vehicles (pcu)				Queue length (m) QL Eq.(10) (10)	Stop rate stops/pcu NS Eq.(11) (11)	No. of Stops pcu/h N_{SV} Eq.(12) (12)	Delay			
					NQ_1 Eq.(8) (6)	NQ_2 Eq.(9) (7)	Total NQ_1+NQ_2 NQ Eq.(7) (8)	NQ_{MAX} Fig.2.5 (9)				Average Traffic delay sec/pcu DT Eq.(14) (13)	Average Geometric delay sec/pcu DG Eq.(15) (14)	Average delay sec/pcu D = DT+DG (13)+(14) (15)	Total Delay Pcu.sec D x Q (2)+(15) (16)
N	766	828	0.925	0.41	4.89	10.92	15.81	21.5	86	1.24	950	36.40	4.66	41.06	31452
S	554	847	0.66	0.41	0.47	6.60	7.07	12	48	0.78	424	14.88	3.42	18.30	9955
E	953	1039	0.92	0.44	4.72	13.45	18.17	26	87	1.14	1086	30.58	4.34	34.92	33279
W	845	1039	0.81	0.44	1.61	11.03	12.64	19	63	0.90	761	18.73	3.74	22.47	18987
ROTR (all)															
Flow adj. Qadj :									Total:		3221	Total:			167317
Total flow Qtot :	3118								Average no. of stops/pcu		1.03	Average Intersection Delay (sec / pcu)			64.30

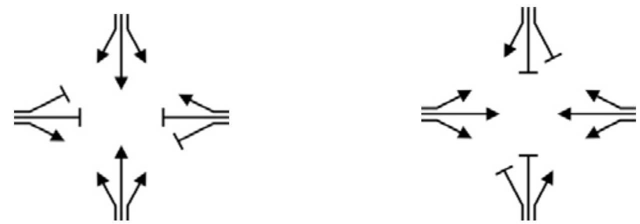
Calculations Of Two Phase

1. Very High Traffic Flow

Appr code	Dir.	TRAFFIC FLOW MOTORISED VEHICLES (MV)								
		Light Vehicles (LV) pce protected = 1.0 pce opposed = 1.0		Heavy Vehicles (HV) pce protected = 1.3 pce opposed = 1.3		Motorcycles (MC) pce protected = 0.2 pce opposed = 0.4		Total Motor vehicles MV	Ratio of turning	
		veh/h	pcu/h Opp	veh/h	pcu/h Opp	veh/h	pcu/h Opp	pcu/h Opp	p LT Eq.(13)	p RT Eq.(14)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
N	LT/LTOR	109	109	68	88	5	2	199	0.21	
	ST	465	465	78	101	3	1	567		
	RT	140	140	47	61	3	1	202		0.21
	TOTAL							968		
S	LT/LTOR	104	104	62	81	4	2	187	0.26	
	ST	236	236	99	129	4	2	367		
	RT	88	88	55	72	3	1	161		0.23
	TOTAL							715		
E	LT/LTOR	164	164	113	147	5	2	313	0.24	
	ST	443	443	150	195	5	2	640		
	RT	202	202	98	127	4	2	331		0.26
	TOTAL							1284		
W	LT/LTOR	145	145	84	109	10	4	258	0.23	
	ST	413	413	132	172	6	2	587		
	RT	137	137	90	117	6	2	256		0.23
	TOTAL							1101		

Traffic Flow Distribution **Phase (1)** **Phase(2)**





Appr Code	Green In Phase No	Appr type	Ratio of turning vehicles			RT-flow pcu/h		Eff Width (m)	Saturation flow pcu/h						Traffic Flow pcu/h	Flow Ratio FR	Phase Ratio	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
						Own Dir	Oppos Dir		Base Value pcu/h	Adjustment factors				Adjusted Value pcu/h						
										All appr type										
										City size	Side friction	Gradient	Parking							
			P_{LTOR}	P_{LT}	P_{RT}	Q_{LT}	Q_{LTO}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	S	Q	Q/S	PR= FRcrit/IFR	g	C	Q/C
									Fig. 2.3 Fig. 2.4					Eq.(2)				Eq.(5)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	o		0.21	0.21	199	187	5	2140	1.00	0.95	1.00	1.00	2033	766	0.38	0.48	35	878	0.87
S	1	o		0.26	0.23	187	199	5	2120	1.00	0.98	1.00	1.00	2078	554	0.27		35	898	0.62
E	2	O		0.24	0.26	313	258	6	2460	1.00	0.95	1.00	1.00	2337	953	0.41	0.52	38	1096	0.87
W	2	o		0.23	0.23	258	313	6	2460	1.00	0.95	1.00	1.00	2337	845	0.37		38	1096	0.77

Total lost time		Unadjustment cycle time	C_{US} (sec)	81
LTI (sec) 8 Sec		Adjustment cycle time	C (sec)	81

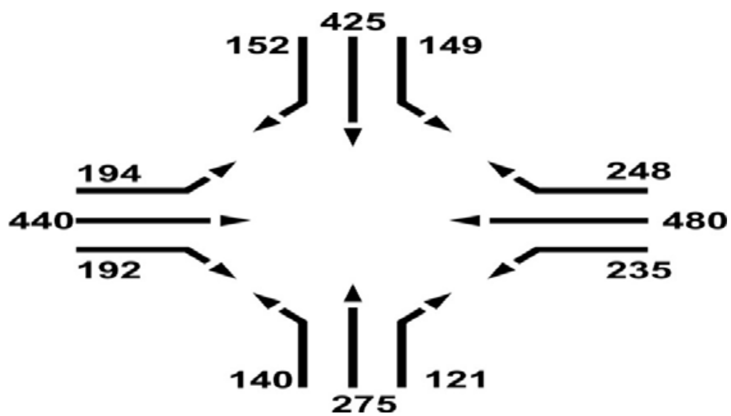
IFR= $\sum FR_{CRIT}$	0.79

Approach Code	Traffic flow pcu/h	Capacity pcu/h	Degree of Saturation	Green Ratio	No. of queuing vehicles (pcu)				Queue length	Stop rate	No. of Stops	Delay			
			DS	GR	NQ_1	NQ_2	Total	NQ_{MAX}	(m)	stops/pcu	pcu/h	Average Traffic delay sec/pcu	Average Geometric delay sec/pcu	Average delay sec/pcu	Total Delay Pcu.sec
	Q	C	= Q/C	= g/c			NQ_1+NQ_2 NQ		QL	NS	N_{SV}	DT	DG	D = DT+DG	D x Q
					Eq.(8)	Eq.(9)	Eq.(7)	Fig.2.5	Eq.(10)	Eq.(11)	Eq.(12)	Eq.(14)	Eq.(15)	(13)+(14)	(2)+(15)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	766	878	0.87	0.43	2.72	15.7	18.42	27	108	0.96	735	32.2	3.89	36.09	27645
S	554	898	0.62	0.43	0.32	9.7	10.02	16	64	0.72	399	19.22	3.27	22.49	12459
E	953	1096	0.87	0.47	2.74	19.22	21.96	32	107	0.92	877	28.25	3.80	32.05	30544
W	845	1096	0.77	0.47	1.16	15.8	16.96	26	87	0.80	676	21.64	3.48	25.12	21226
ROTR (all)	950														
Flow adj. Qadj :										Total:	2687			Total:	91874
Total flow Qtot :	3118									Average no. of stops/pcu	0.86			Average intersection delay (sec/pcu)	29.47

2. High Traffic Flow

Appr code (1)	Dir. (2)	Total Motor vehicles MV pcu/h Opp (9)	Ratio of turning p_{LT} Eq.(13) (10)	p_{RT} Eq.(14) (11)
N	LT/LTOR	149	0.21	
	ST	425		
	RT	152		0.21
	TOTAL	726		
S	LT/LTOR	140	0.26	
	ST	275		
	RT	121		0.23
	TOTAL	536		
E	LT/LTOR	235	0.24	
	ST	480		
	RT	248		0.26
	TOTAL	963		
W	LT/LTOR	194	0.23	
	ST	440		
	RT	192		0.23
	TOTAL	826		

Traffic Flow Distribution



Appr Code	Green In Phase No	Appr type	Ratio of turning vehicles			RT-flow pcu/h		Eff Width (m)	Saturation flow pcu/h						Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit / IFR	Green Time Sec	Capacity pcu/h S x g/c	Degree Of saturation
						Own dir	Oppos Dir		Base Value pcu/h	Adjustment factors All appr type				Adjusted Value pcu/h						
										City size	Side friction	Gradient	Parking							
			P_{LTOR}	P_{LT}	P_{RT}	Q_{LT}	Q_{LTO}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	O		0.21	0.21	149	140	5	2320	1.00	0.95	1.00	1.00	2204	574	0.26	0.47	14	812	0.71
S	1	O		0.26	0.23	140	149	5	2300	1.00	0.98	1.00	1.00	2254	415	0.18		14	830	0.50
E	2	O		0.24	0.26	235	194	6	2580	1.00	0.95	1.00	1.00	2451	715	0.29	0.53	16	1032	0.69
W	2	O		0.23	0.23	194	235	6	2350	1.00	0.95	1.00	1.00	2233	634	0.28		16	940	0.67

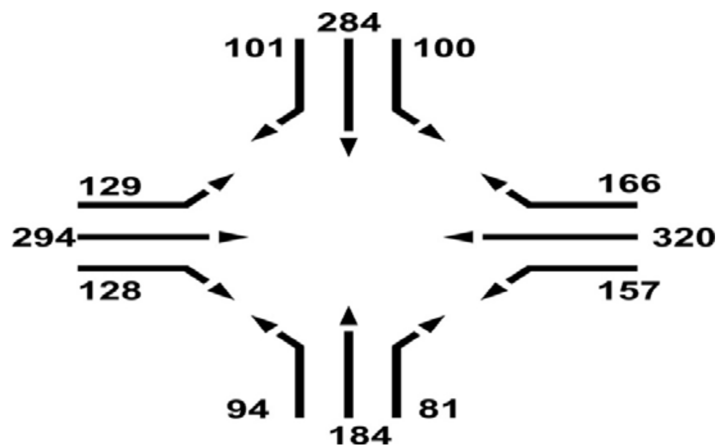
Total lost time		Unadjustment cycle time	C_{US} (sec)	38	IFR= $\sum FR_{CRIT}$	0.55
LTI (sec) 8 Sec		Adjustment cycle tim	C (sec)	38		

Approach Code	Traffic Flow	Capacity pcu/h	Degree of Saturation	Green ratio	No. of queuing vehicles (pcu)				Queue length	Stop rate	No. of Stops	Delay			
	pcu/h		DS	GR	NQ_1	NQ_2	Total	NQ_{MAX}	(m)	stops/pcu	pcu/h	Average Traffic delay sec/pcu	Average Geometric delay sec/pcu	Average delay sec/pcu	Total Delay Pcu.sec
	Q	C	= Q/C	= g/c			NQ_1+NQ_2 NQ		QL	NS	N_{SV}	DT	DG	D = DT+DG	D x Q
					Eq.(8)	Eq.(9)	Eq.(7)	Fig.2:5	Eq.(10)	Eq.(11)	Eq.(12)	Eq.(14)	Eq.(15s)	(13)+(14)	(2)+(15)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	574	812	0.71	0.37	0.72	0502	5.92	10.6	42	0.88	505	13.4	3.67	17.1	9815
S	415	830	0.50	0.37	0	3.4	3.4	7	28	0.70	291	9.25	3.2	12.45	5167
E	715	1032	0.69	0.42	0.61	6.2	6.81	11.7	39	0.81	579	11.13	3.54	14.67	10489
W	634	940	0.67	0.42	0.61	5.4	6.01	10	33	0.81	514	11.23	3.5	14.73	9339
ROTR (all)	713														
Flow adj. Qadj :										Total:	1889			Total:	34810
Total flow Qtot :	2338									Average no. of stops/pcu	0.81			Average intersection delay (sec/pcu)	14.89

3. Low Traffic Flow

Appr code	Dir.	Total Motor vehicles MV	Ratio of turning	
		pcu/h	p_{LT}	p_{RT}
		Opp	Eq.(13)	Eq.(14)
(1)	(2)	(9)	(10)	(11)
N	LT/LTOR	100	0.21	
	ST	284		
	RT	101		0.21
	TOTAL	485		
S	LT/LTOR	94	0.26	
	ST	184		
	RT	81		0.23
	TOTAL	359		
E	LT/LTOR	157	0.24	
	ST	320		
	RT	166		0.26
	TOTAL	643		
W	LT/LTOR	129	0.23	
	ST	294		
	RT	128		0.23
	TOTAL	551		

Traffic Flow Distribution



Appr Code	Green In Phase no	Appr Type	Ratio of turning vehicles			RT-flow pcu/h		Eff Width (m)	Saturation flow pcu/h						Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit	Green Time Sec	Capacity pcu/h S x g/c	Degree Of saturation
						Own dir	Oppos Dir		Base Value pcu/h	Adjustment factors All appr type				Adjusted Value pcu/h						
										City size	Side Friction	Gradient	Parking							
			P_{LTOR}	P_{LT}	P_{RT}	Q_{LT}	Q_{LTO}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	S	Q	Q/S	IFR	G	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	T (12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	O		0.21	0.21	100	94	5	2510	1.00	0.95	1.00	1.00	2385	384	0.16	0.47	9	826	0.46
S	1	O		0.26	0.23	94	100	5	2500	1.00	0.98	1.00	1.00	2450	278	0.11		9	848	0.33
E	2	O		0.24	0.26	157	129	6	2850	1.00	0.95	1.00	1.00	2708	477	0.18	0.53	10	1042	0.46
W	2	O		023	0.23	129	157	6	2760	1.00	0.95	1.00	1.00	2622	423	0.16		10	1008	0.42

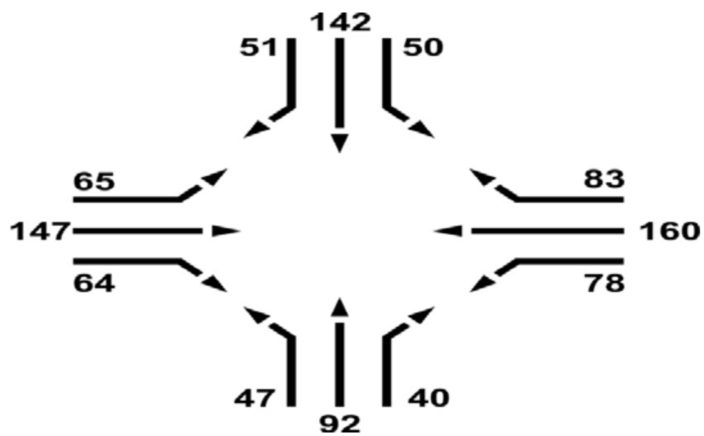
Total lost time		Unadjustment cycle time	C_{US} (sec)	26	IFR= $\sum FR_{CRIT}$	0.34
LTI (sec) 8 Sec		Adjustment cycle time	C (sec)	27		

Approach Code	Traffic Flow pcu/h Q	Capacity pcu/h C	Degree of Saturation DS = Q/C	Green ratio GR = g/c	No. of queuing vehicles (pcu)				Queue length (m) QL	Stop rate stops/pcu NS	No. of Stops pcu/h N_{SV}	Delay			
					NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}				Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG (13)+(14)	Total Delay Pcu.sec D x Q (2)+(15)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	384	826	0.46	0.35	0	2.15	2.15	5	20	0.70	269	6.55	3.20	9.75	3744
S	278	848	0.33	0.35	0	1.47	1.47	3.5	14	0.66	184	6.21	3.11	9.32	2591
E	477	1042	0.46	0.38	0	2.60	2.60	6	20	0.68	324	6.1	3.22	9.32	4446
W	423	1008	0.42	0.38	0	2.25	2.25	5.8	19	0.66	279	5.95	3.11	9.10	3849
ROTR (all)	476														
Flow adj. Qadj :										Total:	1056			Total:	14630
Total flow Qtot :	1562									Average no. of stops/pcu	0.68			Average intersection delay (sec/pcu)	9.40

4. Very Low Traffic Flow

Appr code	Dir.	Total Motor vehicles MV	Ratio of turning	
		pcu/h	p_{LT}	p_{RT}
		Opp	Eq.(13)	Eq.(14)
(1)	(2)	(9)	(10)	(11)
N	LT/LTOR	50	0.21	
	ST	142		
	RT	51		0.21
	TOTAL	243		
S	LT/LTOR	47	0.26	
	ST	92		
	RT	40		0.23
	TOTAL	179		
E	LT/LTOR	78	0.24	
	ST	160		
	RT	83		0.26
	TOTAL	321		
W	LT/LTOR	65	0.23	
	ST	147		
	RT	64		0.23
	TOTAL	276		

Traffic Flow Distribution



Appr Code	Green In Phase No	Appr type	Ratio of turning vehicles			RT-flow pcu/h		Eff width (m)	Saturation flow pcu/h						Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit / IFR	Green Time Sec	Capacity pcu/h S x g/c	Degree Of saturation
						Own Dir	Oppos Dir		Base Value pcu/h	Adjustment factors (All appr type)				Adjusted Value pcu/h						
			P_{LTOR}	P_{LT}	P_{RT}	Q_{LT}	Q_{LTO}	W_e	S_o	City size F_{CS}	Side Friction F_{SF}	Gradient F_G	Parking F_P	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	O		0.21	0.21	50	47	5	2720	1.00	0.95	1.00	1.00	2584	192	0.074	0.49	6	775	0.25
S	1	O		0.26	0.23	47	50	5	2710	1.00	0.98	1.00	1.00	2656	139	0.052		6	797	0.17
E	2	O		0.24	0.26	78	65	6	3200	1.00	0.95	1.00	1.00	3040	238	0.078	0.51	6	912	0.26
W	2	O		0.23	0.23	65	78	6	3100	1.00	0.95	1.00	1.00	2945	212	0.072		6	884	0.24
Total lost time				Unadjustment cycle time					C_{US} (sec)	20					IFR=	0.152				

LTI (sec) 8 Sec		Adjustment cycle time C (sec)	20		$\sum FR_{CRIT}$	

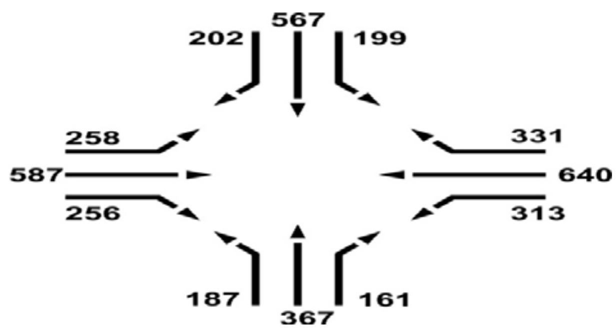
Approach Code	Traffic flow pcu/h	Capacity pcu/h	Degree of Saturation DS	Green Ratio GR	No. of queuing vehicles (pcu)				Queue length (m)	Stop rate stops/pcu	No. of Stops pcu/h	Delay			
					NQ_1	NQ_2	Total NQ_1+NQ_2	NQ_{MAX}				Average Traffic delay sec/pcu	Average Geometric delay sec/pcu	Average delay sec/pcu	Total Delay Pcu.sec
	Q	C	= Q/C	= g/c			NQ		QL	NS	N_{SV}	DT	DG	D = DT+DG (13)+(14)	D x Q (2)+(15)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	192	775	0.25	0.30	0	0.81	0.81	2.8	11	0.68	131	5.3	3.12	8.42	1617
S	139	797	0.17	0.30	0	0.57	0.57	2	8	0.66	92	5.2	3.11	8.31	1155
E	238	912	0.26	0.30	0	1.00	1.00	3	10	0.68	162	5.31	3.22	8.53	2030
W	212	884	0.24	0.30	0	0.89	0.89	2.9	10	0.68	144	5.3	3.16	8.46	1794
ROTR (all)	238														
Flow adj. Qadj :										Total:	529			Total:	6596
Total flow Qtot :	781									Average no. of stops/pcu	0.68			Average intersection delay (sec/pcu)	8.45

The Calculations Of Four Phase (Type 1)

1. Very High Traffic Flow

Appr code	Dir.	TRAFFIC FLOW MOTORISED VEHICLES (MV)								
		Light Vehicles (LV) pce protected = 1.0 pce opposed = 1.0		Heavy Vehicles (HV) pce protected = 1.3 pce opposed = 1.3		Motorcycles (MC) pce protected = 0.2 pce opposed = 0.4		Total Motor vehicles MV	Ratio of turning	
		veh/h	pcu/h Opp	veh/h	pcu/h Opp	veh/h	pcu/h Opp	pcu/h Opp	p_{LT} Eq.(13)	p_{RT} Eq.(14)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
N	LT/LTOR	109	109	68	88	5	2	199	0.21	
	ST	465	465	78	101	3	1	567		
	RT	140	140	47	61	3	1	202		0.21
	TOTAL							968		
S	LT/LTOR	104	104	62	81	4	2	187	0.26	
	ST	236	236	99	129	4	2	367		
	RT	88	88	55	72	3	1	161		0.23
	TOTAL							715		
E	LT/LTOR	164	164	113	147	5	2	313	0.24	
	ST	443	443	150	195	5	2	640		
	RT	202	202	98	127	4	2	331		0.26
	TOTAL							1284		
W	LT/LTOR	145	145	84	109	10	4	258	0.23	
	ST	413	413	132	172	6	2	587		
	RT	137	137	90	117	6	2	256		0.23
	TOTAL							1101		

Traffic Flow Distribution

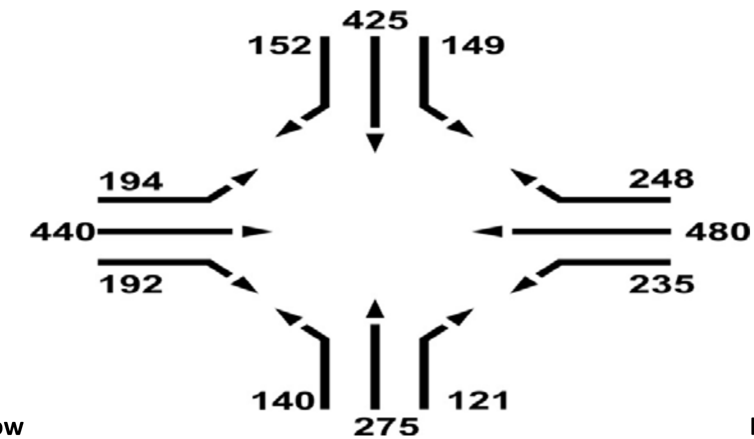


Appr Code	Green In Phase no	Appr type	Ratio of turning vehicles			Eff width (m)	Base Value pcu/h	Saturation flow pcu/h: Adjustment factors: All appr type: City size	Side Friction	Gradient	Parking	Only type p: Right Turns	Lift Turns	Adjusted Value pcu/h	Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit / IFR	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
			P_{LTOR}	P_{LT}	P_{RT}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	F_{RT}	F_{LT}	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N		P		0.21	0.21	8	4800	1.00	0.95	1.00	1.00	1.054	0.966	4646	968	0.208	0.257	35	1070	0.9
S		P		0.26	0.23	8	4800	1.00	0.98	1.00	1.00	1.059	0.958	4778	715	0.149	0.185	25	786	0.91
E		P		0.24	0.26	9	5400	1.00	0.95	1.00	1.00	1.067	0.962	5267	1284	0.24	0.297	41	1421	0.90
W		p		0.23	0.23	9	5400	1.00	0.95	1.00	1.00	1.058	0.963	5232	1101	0.21	0.26	35	1205	0.91
Total lost time LTI (sec)		16 Sec	Unadjustment cycle time C_{US} (sec)						151						IFR=					
			Adjustment cycle time C (sec)						153						$\sum FR_{CRIT}$	0.808				

Approach Code	Traffic flow pcu/h Q	Capacity pcu/h C	Degree of Saturation DS = Q/C	Green Ratio GR = g/c	No. of queuing vehicles (pcu)				Queue length (m) QL	Stop rate stops/pcu NS	No. of Stops pcu/h N_{SV}	Delay			
					NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}				Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG	Total Delay Pcu.sec D x Q
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	968	1070	0.90	0.23	3.74	39.68	43.42	60	150	0.96	929	69.40	3.89	73.29	70945
S	715	786	0.91	0.16	4.10	29.68	33.78	48	120	1.00	715	81.50	4.00	85.5	61133
E	1284	1421	0.90	0.27	3.80	52.3	56.10	78	173	0.93	1194	63.13	3.83	66.96	85977
W	1101	1205	0.91	0.23	4.23	45.3	49.53	69	153	0.93	1024	57.70	3.82	61.52	67734
ROTR (all)															
Flow adj. Qadj :										Total:	3862			Total	285789
Total flow Qtot :	4068									Average no. of stops/pcu	0.95			Average intersection delay (sec/pcu)	70.25

2. High Traffic Flow

App. code	Dir.	Total Motor vehicles MV	Ratio of turning	
		pcu/h	p_{LT}	p_{RT}
		Opp	Eq.(13)	Eq.(14)
(1)	(2)	(9)	(10)	(11)
N	LT/LTOR	149	0.21	
	ST	425		
	RT	152		0.21
	TOTAL	726		
S	LT/LTOR	140	0.26	
	ST	275		
	RT	121		0.23
	TOTAL	536		
E	LT/LTOR	235	0.24	
	ST	480		
	RT	248		0.26
	TOTAL	963		
W	LT/LTOR	194	0.23	
	ST	440		
	RT	192		0.23
	TOTAL	826		



Traffic Flow Distribution

Appr Code	Green In Phase no	Appr type	Ratio of turning vehicles			Eff width (m)	Base Value pcu/h	Saturation flow pcu/h							Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
								Adjustment factors						Adjusted Value pcu/h						
								All appr type				Only type p								
								City size	Side Friction	Gradient	Parking	Right Turns	Lift Turns							
			P_{LTOR}	P_{LT}	P_{RT}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	F_{RT}	F_{LT}	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N		P		0.21	0.21	8	4800	1.00	0.95	1.00	1.00	1.054	0.966	4646	726	0.16	0.26	16	965	0.75
S		P		0.26	0.23	8	4800	1.00	0.98	1.00	1.00	1.059	0.958	4778	536	0.11	0.18	11	683	0.78
E		P		0.24	0.26	9	5400	1.00	0.95	1.00	1.00	1.067	0.962	5267	961	0.19	0.30	18	1231	0.78
W		p		0.23	0.23	9	5400	1.00	0.95	1.00	1.00	1.058	0.963	5232	823	0.16	0.26	16	1087	0.76

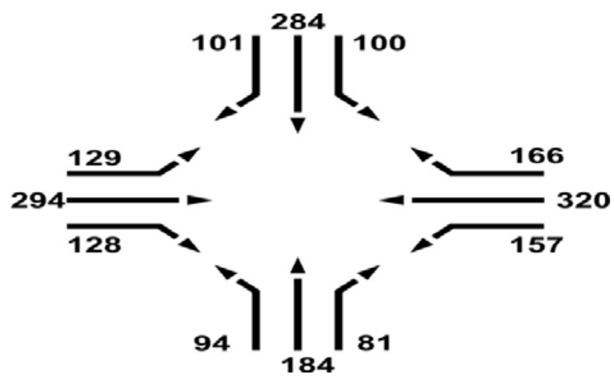
Total lost time LTI (sec)	16 Sec	Unadjustment cycle time C_{US} (sec)	76						IFR=						
		Adjustment cycle time C (sec)	77						$\sum FR_{CRIT}$	0.62					

Approach Code	Traffic flow pcu/h Q	Capacity pcu/h C	Degree of Saturation DS = Q/C	Green Ratio GR = g/c	No. of queuing vehicles (pcu) NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}	Queue length (m) QL	Stop rate stops/pcu NS	No. of Stops pcu/h N_{SV}	Delay: Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG (13)+(14)	Total Delay Pcu.sec D x Q (2)+(15)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	726	965	0.75	0.21	1.61	14.63	16.24	24	60	0.94	682	34.78	3.84	38.62	28038
S	536	683	0.78	0.14	1.47	11.10	12.57	20	50	0.99	531	39.94	3.97	43.91	23536
E	961	1231	0.78	0.23	1.48	19.40	20.88	30	67	0.91	875	33.52	3.78	37.30	35845
W	823	1087	0.76	0.21	1.16	16.60	17.76	26	58	0.91	749	32.58	3.76	36.34	29908
ROTR (all)	3046														
Flow adj. Qadj :										Total:	2837			Total:	117327
Total flow Qtot :	3046									Average no. of stops/pcu	0.93			Average intersection delay (sec/pcu)	38.52

3. Low Traffic Flow

Appr code	Dir.	Total Motor vehicles MV	Ratio of turning	
		pcu/h	p_{LT}	p_{RT}
		Opp	Eq.(13)	Eq.(14)
	(2)	(9)	(10)	(11)
N	LT/LTOR	100	0.21	
	ST	284		
	RT	101		0.21
	TOTAL	485		
S	LT/LTOR	94	0.26	
	ST	184		
	RT	81		0.23
	TOTAL	359		
E	LT/LTOR	157	0.24	
	ST	320		
	RT	166		0.26
	TOTAL	643		
W	LT/LTOR	129	0.23	
	ST	294		
	RT	128		0.23
	TOTAL	551		

Traffic Flow Distribution



Appr Code	Green In Phase no	Appr type	Ratio of turning vehicles			Eff width (m)	Base Value pcu/h	Saturation flow pcu/h — Adjustment factors — All appr type: City size	Side Friction	Gradient	Parking	Only type p: Right Turns	Lift Turns	Adjusted Value pcu/h	Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
			P_{LTOR}	P_{LT}	P_{RT}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	F_{RT}	F_{LT}	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N		P		0.21	0.21	8	4800	1.00	0.95	1.00	1.00	1.054	0.966	4646	485	0.11	0.261	9	805	0.6
S		P		0.26	0.23	8	4800	1.00	0.98	1.00	1.00	1.059	0.958	4778	359	0.082	0.195	7	646	0.56
E		P		0.24	0.26	9	5400	1.00	0.95	1.00	1.00	1.067	0.962	5267	643	0.125	0.30	10	1006	0.64
W		p		0.23	0.23	9	5400	1.00	0.95	1.00	1.00	1.058	0.963	5232	551	0.11	0.261	9	905	0.61

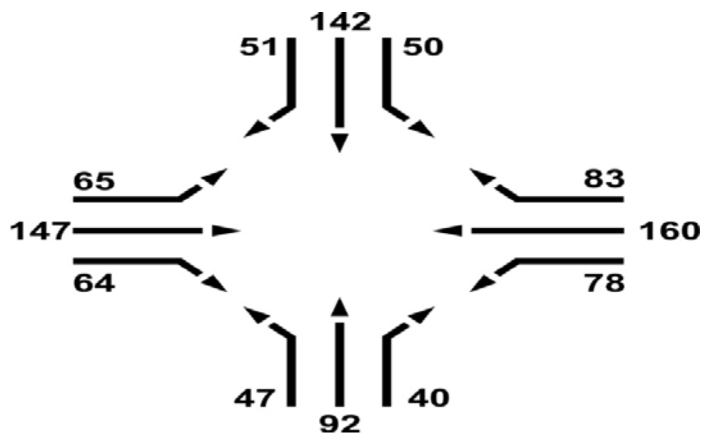
Total lost time LTI (sec)	16 Sec	Unadjustment cycle time C_{US} (sec)	50					IFR=					
		Adjustment cycle time C (sec)	51					$\sum FR_{CRIT}$	0.421				

Approach Code	Traffic Flow pcu/h	Capacity pcu/h	Degree of Saturation	Green Ratio	No. of queuing vehicles (pcu)				Queue length	Stop rate	No. of Stops	Delay			
			DS = Q/C	GR = g/c	NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}	(m) QL	stops/pcu NS	pcu/h N_{SV}	Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG (13)+(14)	Total Delay Pcu.sec D x Q (2)+(15)
	Q	C													
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	485	805	0.6	0.18	0.25	6.32	6.57	12	20	0.86	417	20.34	3.62	23.96	11621
S	359	646	0.56	0.14	0.14	4.80	4.94	9	23	0.87	314	21.24	3.67	24.91	8943
E	643	1006	0.64	0.20	0.40	8.36	8.76	14	31	0.867	556	20.15	3.67	23.82	15316
W	551	905	0.61	0.18	0.28	7.19	7.47	13	29	0.86	474	20.37	3.63	24	13224
ROTR (all)															
Flow adj. Qadj :										Total:	1761			Total:	49104
Total flow Qtot :	2038									Average no. of stops/pcu	0.864			Average intersection delay (sec/pcu)	24.10

4. Very Low Traffic Flow

Appr code	Dir.	Total Motor vehicles MV pcu/h	Ratio of turning	
		Opp	p_{LT} Eq.(13)	p_{RT} Eq.(14)
(1)	(2)	(9)	(10)	(11)
N	LT/LTOR	50	0.21	
	ST	142		
	RT	51		0.21
	TOTAL	243		
S	LT/LTOR	47	0.26	
	ST	92		
	RT	40		0.23
	TOTAL	179		
E	LT/LTOR	78	0.24	
	ST	160		
	RT	83		0.26
	TOTAL	321		
W	LT/LTOR	65	0.23	
	ST	147		
N	RT	64		0.23
	TOTAL	276		

Traffic Flow Distribution



Appr Code	Green In Phase no	Appr type	Ratio of turning vehicles			Eff width (m)	Base Value pcu/h	Saturation flow pcu/h — Adjustment factors: All appr type				Only type p		Adjusted Value pcu/h	Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
								City size	Side Friction	Gradient	Parking	Right Turns	Lift Turns							
			P_{LTOR}	P_{LT}	P_{RT}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	F_{RT}	F_{LT}	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N		P		0.21	0.21	8	4800	1.00	0.95	1.00	1.00	1.054	0.966	4646	243	0.052	0.26	5	645	0.38
S		P		0.26	0.23	8	4800	1.00	0.98	1.00	1.00	1.059	0.958	4778	179	0.037	0.18	4	531	0.24
E		P		0.24	0.26	9	5400	1.00	0.95	1.00	1.00	1.067	0.962	5267	321	0.061	0.30	6	1024	0.38

W		p		0.23	0.23	9	5400	1.00	0.95	1.00	1.00	1.058	0.963	5232	275	0.053	0.26	5	727	0.33
Total lost time LTI (sec)		16 Sec	Unadjustment cycle time C_{US} (sec)						36						IFR=					
			Adjustment cycle time C (sec)						36						$\sum FR_{CRIT}$	0.203				

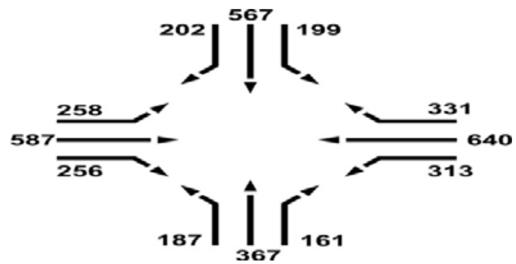
Approach Code	Traffic Flow pcu/h	Capacity pcu/h	Degree of Saturation	Green Ratio	No. of queuing vehicles (pcu)				Queue length (m)	Stop rate stops/pcu	No. of Stops pcu/h	Delay			
			DS = Q/C	GR = g/c	NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}				Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG (13)+(14)	Total Delay Pcu.sec D x Q (2)+(15)
	Q	C							QL	NS	N_{SV}				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	243	645	0.38	0.14	0	2.21	2.21	5.5	14	0.82	199	14.10	3.51	17.61	4279
S	179	531	0.24	0.11	0	1.65	1.65	4	10	0.83	149	14.80	3.55	18.35	3285
E	321	1024	0.38	0.17	0	2.81	2.81	7	16	0.79	254	13.10	3.49	16.59	5325
W	275	727	0.33	0.14	0	2.50	2.50	6	13	0.82	226	14.10	3.53	17.63	4848
ROTR (all)															
Flow adj. Qadj :										Total:	828			Total:	17737
Total flow Qtot :	1018									Average no. of stops/pcu	0.81			Average intersection delay (sec/pcu)	17.42

The Calculations Of Four Phase (Type 2)

1. Very High Traffic Flow

Appr code	Dir.	TRAFFIC FLOW MOTORISED VEHICLES (MV)								
		Light Vehicles (LV) pce protected = 1.0 pce opposed = 1.0		Heavy Vehicles (HV) pce protected = 1.3 pce opposed = 1.3		Motorcycles (MC) pce protected = 0.2 pce opposed = 0.4		Total Motor vehicles MV	Ratio of turning	
		veh/h	pcu/h Opp	veh/h	pcu/h Opp	veh/h	pcu/h Opp	pcu/h Opp	p_{LT} Eq.(13)	p_{RT} Eq.(14)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
N	LT/LTOR	109	109	68	88	5	2	199	0.21	
	ST	465	465	78	101	3	1	567		
	RT	140	140	47	61	3	1	202		0.21
	TOTAL							968		
S	LT/LTOR	104	104	62	81	4	2	187	0.26	
	ST	236	236	99	129	4	2	367		
	RT	88	88	55	72	3	1	161		0.23
	TOTAL							715		
E	LT/LTOR	164	164	113	147	5	2	313	0.24	
	ST	443	443	150	195	5	2	640		
	RT	202	202	98	127	4	2	331		0.26
	TOTAL							1284		
W	LT/LTOR	145	145	84	109	10	4	258	0.23	
	ST	413	413	132	172	6	2	587		
	RT	137	137	90	117	6	2	256		0.23
	TOTAL							1101		

Traffic Flow Distribution



Appr Code	Green In Phase no	Appr type	Ratio of turning vehicles			Eff width (m)	Base Value pcu/h	Saturation flow pcu/h – Adjustment factors – All appr type: City Size	Side Friction	Gradient	Parking	Only type p: Right Turns	Lift Turns	Adjusted Value pcu/h	Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
			P_{LTOR}	P_{LT}	P_{RT}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	F_{RT}	F_{LT}	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	P				5	3000	1.00	0.95	1.00	1.00	1.054	0.9664	2903	769	0.265	0.32	51	846	0.91
S		p				5	3000	1.00	0.98	1.00	1.00	1.0598	0.9584	2986	528	0.177		51	870	0.61
N	2	P				3	1800	1.00	0.95	1.00	1.00	1.054	0.9664	1741	199	0.114	0.14	22	219	0.91
S		p				3	1800	1.00	0.98	1.00	1.00	1.0598	0.9584	1792	187	0.104		22	225	0.83
E	3	p				5.5	3300	1.00	0.95	1.00	1.00	1.0676	0.9616	3218	971	0.302	0.36	57	1048	0.926
W		P				5.5	3300	1.00	0.95	1.00	1.00	1.0589	0.9632	3197	843	0.264		57	1041	0.81
E	4	p				3.5	2100	1.00	0.95	1.00	1.00	1.0676	0.9616	2048	313	0.153	0.18	29	339	0.92
W		p				3.5	2100	1.00	0.95	1.00	1.00	1.0589	0.9632	2035	258	0.126		29	337	0.77

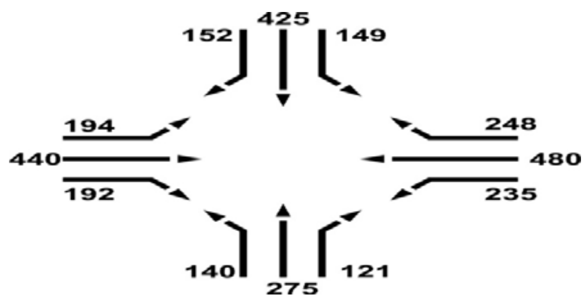
Total lost time LTI (sec)	16 Sec	Unadjustment cycle time C_{US} (sec)	175						IFR=					
		Adjustment cycle time C (sec)	175						$\sum FR_{CRIT}$	0.834				

Approach Code	Traffic flow pcu/h Q	Capacity pcu/h C	Degree of Saturation DS = Q/C	Green Ratio GR = g/c	No. of queuing vehicles (pcu)				Queue length (m) QL	Stop rate stops/pcu NS	No. of Stops pcu/h N_{SV}	Delay			
					NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}				Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG	Total Delay Pcu.sec D x Q
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	769	846	0.91	0.29	4.11	36.1	40.21	66	264	0.97	746	77.41	3.92	81.33	62543
S	528	870	0.61	0.29	0.28	22.14	22.42	32	128	0.79	417	54.75	3.45	58.2	30730
N	199	219	0.91	0.13	3.39	9.54	12.93	20	133	1.20	239	130.8	4.55	135.35	26935
S	187	225	0.83	0.13	1.78	8.87	10.65	17	113	1.05	196	102.72	4.13	106.85	19981
E	971	1048	0.926	0.32	5.03	45.61	50.64	70	255	0.965	937	74.78	3.91	78.69	76408
W	843	1041	0.81	0.32	1.61	37.62	39.23	54	196	0.86	725	60.18	3.63	63.81	53792
E	313	339	0.92	0.17	4.04	14.97	19.01	28	160	1.12	351	114.36	4.29	118.65	37137
W	258	337	0.77	0.17	1.14	11.98	13.12	20	114	0.94	243	81.54	3.84	85.38	22028
ROTR (all)															
Flow adj. Qadj :										Total:	3854			Total	329554
Total flow Qtot :	4068									Average no. of stops/pcu	0.95			Average intersection delay (sec/pcu)	81

2. High Traffic Flow

App. code	Dir.	Total Motor vehicles MV	Ratio of turning	
		pcu/h	p_{LT}	p_{RT}
		Opp	Eq.(13)	Eq.(14)
(1)	(2)	(9)	(10)	(11)
N	LT/LTOR	149	0.21	
	ST	425		
	RT	152		0.21
	TOTAL	726		
S	LT/LTOR	140	0.26	
	ST	275		
	RT	121		0.23
	TOTAL	536		
E	LT/LTOR	235	0.24	
	ST	480		
	RT	248		0.26
	TOTAL	963		
W	LT/LTOR	194	0.23	
	ST	440		
	RT	192		0.23
	TOTAL	826		

Traffic Flow Distribution



Appr Code (1)	Green In Phase no (2)	Appr type (3)	Ratio of turning vehicles P_{LTOR} (4)	P_{LT} (5)	P_{RT} (6)	Eff width (m) W_e (7)	Base Value pcu/h S_o (8)	Saturation flow pcu/h — Adjustment factors — All appr type — City Size F_{CS} (9)	Side Friction F_{SF} (10)	Gradient F_G (11)	Parking F_P (12)	Only type p — Right Turns F_{RT} (13)	Lift Turns F_{LT} (14)	Adjusted Value pcu/h S (15)	Traffic Flow pcu/h Q (16)	Flow ratio FR Q/S (17)	Phase Ratio PR = FRcrit / IFR (18)	Green Time Sec g (19)	Capacity pcu/h S x g/c C (20)	Degree Of Saturation Q/C (21)
N	1	P				5	3000	1.00	0.95	1.00	1.00	1.054	0.9664	2903	577	0.199	0.32	20	744	0.78
S		p				5	3000	1.00	0.98	1.00	1.00	1.0598	0.9584	2986	396	0.133		20	766	0.52
N	2	P				3	1800	1.00	0.95	1.00	1.00	1.054	0.9664	1741	149	0.086	0.14	9	201	0.74
S		p				3	1800	1.00	0.98	1.00	1.00	1.0598	0.9584	1792	140	0.071		9	207	0.68
E	3	p				5.5	3300	1.00	0.95	1.00	1.00	1.0676	0.9616	3218	728	0.226	0.361	22	908	0.80
W		P				5.5	3300	1.00	0.95	1.00	1.00	1.0589	0.9632	3197	632	0.198		22	902	0.70
E	4	p				3.5	2100	1.00	0.95	1.00	1.00	1.0676	0.9616	2048	235	0.115	0.184	11	289	0.81

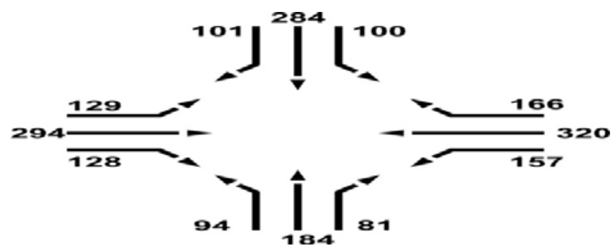
W		p				3.5	2100	1.00	0.95	1.00	1.00	1.0589	0.9632	2035	194	0.095		11	287	0.68
Total lost time LTI (sec)		16 Sec	Unadjustment cycle time C_{US} (sec)						78						IFR=					
			Adjustment cycle time C (sec)						78						$\sum FR_{CRIT}$	0.626				

Approach Code	Traffic flow pcu/h Q	Capacity pcu/h C	Degree of Saturation DS = Q/C	Green Ratio GR = g/c	No. of queuing vehicles (pcu)				Queue length (m) QL	Stop rate stops/pcu NS	No. of Stops pcu/h N_{SV}	Delay			
					NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}				Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG	Total Delay Pcu.sec D x Q
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	577	744	0.78	0.26	1.25	11.60	12.85	20	80	0.925	534	32.84	3.79	36.63	21136
S	396	766	0.52	0.26	0.042	7.34	7.38	12	48	0.77	305	24.89	3.40	28.29	11203
N	149	201	0.74	0.11	0.89	3.13	4.02	8	53	1.12	167	49.57	4.33	53.90	8031
S	140	207	0.68	0.11	0.55	2.92	3.47	7	47	1.03	144	42.95	4.08	47.03	6584
E	728	908	0.80	0.28	1.48	14.64	16.12	24	87	0.92	670	31.92	3.80	35.72	26004
W	632	902	0.70	0.28	0.66	12.26	12.92	20	73	0.85	537	27.78	3.61	31.39	19838
E	235	289	0.81	0.14	1.54	4.94	6.48	11	63	1.15	270	51.72	4.37	56.09	13181
W	194	287	0.68	0.14	0.56	4.00	4.56	8.5	49	0.98	190	38.90	3.95	42.85	8313
ROTR (all)															
Flow adj. Qadj :											Total: 2817			Total	114290
Total flow Qtot :	3051									Average no. of stops/pcu	0.92	Average intersection delay (sec/pcu)			37.46

3. Low Traffic Flow

Appr code	Dir.	Total Motor vehicles MV	Ratio of turning	
		pcu/h	p LT	p RT
		Opp	Eq.(13)	Eq.(14)
	(2)	(9)	(10)	(11)
N	LT/LTOR	100	0.21	
	ST	284		
	RT	101		0.21
	TOTAL	485		
S	LT/LTOR	94	0.26	
	ST	184		
	RT	81		0.23
	TOTAL	359		
E	LT/LTOR	157	0.24	
	ST	320		
	RT	166		0.26
	TOTAL	643		
W	LT/LTOR	129	0.23	
	ST	294		
	RT	128		0.23
	TOTAL	551		

Traffic Flow Distribution



Appr Code	Green In Phase no	Appr type	Ratio of turning vehicles			Eff width (m)	Base Value pcu/h	Saturation flow pcu/h						Adjusted Value pcu/h	Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
								Adjustment factors												
								All appr type				Only type p								
								City Size	Side Friction	Gradient	Parking	Right Turns	Lift Turns							
			P_{LTOR}	P_{LT}	P_{RT}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	F_{RT}	F_{LT}	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	P				5	3000	1.00	0.95	1.00	1.00	1.054	0.9664	2903	385	0.133	0.32	11	603	0.64
S		p				5	3000	1.00	0.98	1.00	1.00	1.0598	0.9584	2986	265	0.089		11	620	0.43
N	2	P				3	1800	1.00	0.95	1.00	1.00	1.054	0.9664	1741	100	0.057	0.14	7	230	0.43
S		p				3	1800	1.00	0.98	1.00	1.00	1.0598	0.9584	1792	94	0.052		7	237	0.40
E	3	p				5.5	3300	1.00	0.95	1.00	1.00	1.0676	0.9616	3218	486	0.151	0.36	12	729	0.67
W		P				5.5	3300	1.00	0.95	1.00	1.00	1.0589	0.9632	3197	422	0.132		12	724	0.58
E	4	p				3.5	2100	1.00	0.95	1.00	1.00	1.0676	0.9616	2048	157	0.077	0.18	7	270	0.58
W		p				3.5	2100	1.00	0.95	1.00	1.00	1.0589	0.9632	2035	129	0.065		7	269	0.48

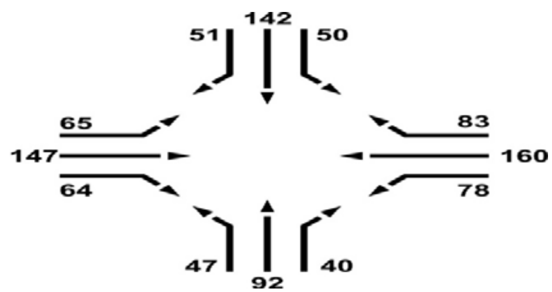
Total lost time LTI (sec)	16 Sec	Unadjustment cycle time C_{US} (sec)	50						IFR=					
		Adjustment cycle time C (sec)	53						$\sum FR_{CRIT}$	0.418				

Approach Code	Traffic flow pcu/h Q	Capacity pcu/h C	Degree of Saturation DS = Q/C	Green Ratio GR = g/c	No. of queuing vehicles (pcu) NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}	Queue length (m) QL	Stop rate stops/pcu NS	No. of Stops pcu/h N_{SV}	Delay Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG	Total Delay Pcu.sec D x Q
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	385	603	0.64	0.21	0.39	5.17	5.56	10	40	0.88	339	21.26	3.67	24.93	9598
S	265	620	0.43	0.21	0	3.39	3.39	7	28	0.78	207	18.18	3.42	21.6	5724
N	100	230	0.43	0.13	0	1.36	1.36	3.8	25	0.83	83	21.25	3.53	24.78	2478
S	94	237	0.40	0.13	0	1.27	1.27	3.2	21	0.826	78	21.16	3.54	24.7	2322
E	486	729	0.67	0.23	0.51	6.51	7.02	12	44	0.88	428	21.09	3.71	24.8	12053
W	422	724	0.58	0.23	0.19	5.52	5.71	10	36	0.83	350	19.08	3.55	22.63	9550
E	157	270	0.58	0.13	0.19	2.17	2.36	6	34	0.92	144	24.22	3.80	28.02	4399
W	129	269	0.48	0.13	0	1.76	1.76	4.3	25	0.83	107	21.39	3.55	24.94	3217
ROTR (all)															
Flow adj. Qadj :										Total:	1736			Total	49341
Total flow Qtot :	2038									Average no. of stops/pcu	0.85			Average intersection delay (sec/pcu)	24.2

4. Very Low Traffic Flow

Appr code	Dir.	Total Motor vehicles MV pcu/h	Ratio of turning	
(1)	(2)	Opp (9)	p_{LT} Eq.(13) (10)	p_{RT} Eq.(14) (11)
N	LT/LTOR	50	0.21	
	ST	142		
	RT	51		0.21
	TOTAL	243		
S	LT/LTOR	47	0.26	
	ST	92		
	RT	40		0.23
	TOTAL	179		
E	LT/LTOR	78	0.24	
	ST	160		
	RT	83		0.26
	TOTAL	321		
W	LT/LTOR	65	0.23	
	ST	147		
N	RT	64		0.23
	TOTAL	276		

Traffic Flow Distribution



Appr Code	Green In Phase no	Appr type	Ratio of turning vehicles			Eff width (m)	Base Value pcu/h	Saturation flow pcu/h							Traffic Flow pcu/h	Flow ratio FR	Phase Ratio PR = FRcrit	Green Time Sec	Capacity pcu/h S x g/c	Degree Of Saturation
								Adjustment factors						Adjusted Value pcu/h						
								All appr type				Only type p								
								City Size	Side Friction	Gradient	Parking	Right Turns	Lift Turns							
			P_{LTOR}	P_{LT}	P_{RT}	W_e	S_o	F_{CS}	F_{SF}	F_G	F_P	F_{RT}	F_{LT}	S	Q	Q/S	IFR	g	C	Q/C
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
N	1	P				5	3000	1.00	0.95	1.00	1.00	1.054	0.9664	2903	193	0.066	0.32	7	452	0.43
S		p				5	3000	1.00	0.98	1.00	1.00	1.0598	0.9584	2986	132	0.044		7	464	0.28
N	2	P				3	1800	1.00	0.95	1.00	1.00	1.054	0.9664	1741	51	0.029	0.14	7	271	0.19
S		p				3	1800	1.00	0.98	1.00	1.00	1.0598	0.9584	1792	47	0.026		7	279	0.17
E	3	p				5.5	3300	1.00	0.95	1.00	1.00	1.0676	0.9616	3218	243	0.076	0.36	8	572	0.42
W		P				5.5	3300	1.00	0.95	1.00	1.00	1.0589	0.9632	3197	211	0.066		8	568	0.37
E	4	p				3.5	2100	1.00	0.95	1.00	1.00	1.0676	0.9616	2048	78	0.038	0.18	7	319	0.24

W		p				3.5	2100	1.00	0.95	1.00	1.00	1.0589	0.9632	2035	64	0.031		7	317	0.20
Total lost time LTI (sec)		16 Sec	Unadjustment cycle time C_{US} (sec)						37						IFR=					
			Adjustment cycle time C (sec)						45						$\sum FR_{CRIT}$	0.209				

Approach Code	Traffic flow pcu/h Q	Capacity pcu/h C	Degree of Saturation DS = Q/C	Green Ratio GR = g/c	No. of queuing vehicles (pcu)				Queue length (m) QL	Stop rate stops/pcu NS	No. of Stops pcu/h N_{SV}	Delay			
					NQ_1	NQ_2	Total NQ_1+NQ_2 NQ	NQ_{MAX}				Average Traffic delay sec/pcu DT	Average Geometric delay sec/pcu DG	Average delay sec/pcu D = DT+DG	Total Delay Pcu.sec D x Q
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
N	193	452	0.43	0.16	0	2.18	2.18	5	20	0.81	156	5.91	3.48	9.39	1812
S	132	464	0.28	0.16	0	1.45	1.45	3	12	0.79	104	16.62	3.45	20.07	2649
N	51	271	0.19	0.16	0	0.55	0.55	2	13	0.776	40	16.37	3.39	19.76	1008
S	47	279	0.17	0.16	0	0.51	0.51	1.9	13	0.78	37	16.31	3.42	19.73	927
E	243	572	0.42	0.18	0	2.69	2.69	6.3	23	0.80	194	16.37	3.51	19.88	4831
W	211	568	0.37	0.18	0	2.32	2.32	5	18	0.79	167	16.21	3.42	19.63	4142
E	78	319	0.24	0.16	0	0.85	0.85	2.3	13	0.78	61	16.51	3.46	19.97	1558
W	64	317	0.20	0.16	0	0.69	0.69	2.1	12	0.776	50	16.40	3.41	19.81	1268
ROTR (all)															
Flow adj. Qadj :										Total:	809			Total	18195
Total flow Qtot :	948									Average no. of stops/pcu	0.85			Average intersection delay (sec/pcu)	19.2