HIGHWAY CAPACITY MANUAL

MALAYSIA

HIGHWAY PLANNING UNIT
MINISTRY OF WORKS
MALAYSIAN GOVERNMENT

ACKNOWLEDGEMENTS

The Highway Planning Unit (HPU), Ministry of Works, Malaysia, first embarked upon a pilot study in 1996 to initiate a drive towards producing a Malaysian Highway Capacity Manual. Since the pilot study, a Stage 1 Study was also completed in order to produce the first Malaysian Highway Capacity Manual. The manual must be accepted and be reliable source to the practitioners in the fields of transport and traffic planning and to achieve this, equation and figures in the manual must be continuously updated in order to have accurate equation and data. Therefore, it is the ultimate aim of the Stage 2 Study to further improve on the overall quality of the manual.

HPU would like to express their appreciation to the many agencies, firms and individuals who have supported and contributed to the study. In particular, HPU would like to recognize the efforts of the Consultant, Universiti Sains Malaysia who have consistently given their full co-operation and assistances to ensure the smooth completion and production of the first version of the Malaysian Highway Capacity Manual. In addition, the project team has been guided throughout the study by the Steering and Technical Committee meetings. HPU expresses its appreciation to all committee members for their participants and contributions.

Steering and Technical Committee Members

Advisor
Prof. Karl- Lennart Barg
Royal Institute of Technology, Sweden

i. Ir. Ooi Gin Hoay
Unit Perancang Ekonomi, Jabatan Perdana Menteri

ii. Puan Norhayati Hj. Ibrahim
Kementerian Kewangan Malaysia

iii. Ir. Dr. Leong Siew Mun
Dewan Bandaraya Kuala Lumpur

iv. Abdul Karim b. Mohd. Tahir
JKR Selangor

v. Ir. Dr. Nordin Yunus
JKR Kedah

vi. Ir. Alan Wong
JKR Sabah

vii. Professor Mohamed Rehan Karim
Jabatan Kejuruteraan Awam, Fakulti Kejuruteraan Universiti Malaya

viii. Professor Ir. Dr. Reza Atiq Abdullah O.K Rahmat
Jabatan Kejuruteraan dan Struktur, Universiti Kebangsaan Malaysia

ix. Professor Dr. Radin Umar b.Radin Sohadi
Jabatan Kejuruteraan Awam, Fakulti Kejuruteraan Universiti Putra Malaysia

x. En. Hussain Hamid
Jabatan Kejuruteraan Awam, Fakulti Kejuruteraan Universiti Putra Malaysia

xi. Prof. Madya Daniel Mohamed
Fakulti Kejuruteraan Awam, Universiti Teknologi Malaysia

TABLE OF CONTENTS

3.0 SIGNALISED INTERSECTIONS

4.0 UNSIGNALISED INTERSECTIONS

5.0 URBAN AND SUBURBAN ARTERIALS

LIST OF FIGURES / TABLES

FIGURES

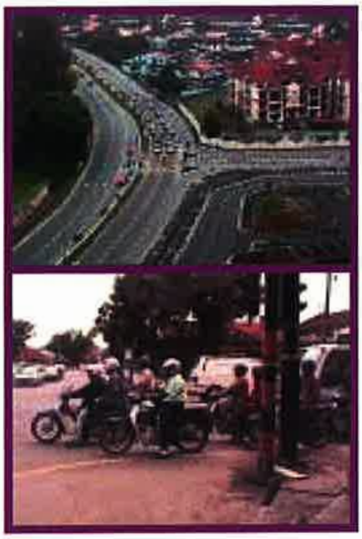
TABLES

CHAPTER 1

INTRODUCTION, CONCEPT AND APPLICATION

CHAPTER 1

1.0 INTRODUCTION, CONCEPT AND APPLICATIONS



CONTENTS

FIGURES

TABLES

1.1 INTRODUCTION

The Ministry of Works Malaysia has responded to the need of having a Malaysian Highway Capacity Manual (MHCM) by initiating two projects. The preceding project, entitled the "Traffic Study Malaysia" had concluded that the US Highway Capacity Manual (USHCM), which has been extensively used in the highway design standards for Malaysia may no longer be suitable. As a consequent, the first stage of the Malaysian Highway Capacity Study was carried out focussing on the urban and suburban facilities.

Preliminary studies on Malaysian travel behaviour showed that there are many distinct differences with the conditions elsewhere, hence advocating the move for having our own Malaysian Highway Capacity Manual, based on researches, carried out in Malaysia. Figure 1.1 illustrates the general approach being used in carrying out the study towards establishing the Malaysian Highway Capacity Manual.

As illustrated in Figure 1.1, the first stage in the development of the Malaysian Highway Capacity Manual is to provide justification that the proposed new manual is really needed for Malaysia. Therefore, the suitability and validity of the US HCM were checked based on Malaysia traffic condition. Due to the time constraint and a wide range of scope to be elaborated in the manual, there is a need to identify and determine which facilities are most important in order to fulfil the need of Malaysian road and travel behaviour.

As stated earlier, the US HCM is not suitable to be used in Malaysia. Due to this reason, Malaysian Highway Capacity Manual has come out with its own formulation and correction factors based on an intensive research conducted in various states throughout Malaysia. However, it is a continuous effort in improving and updating the manual based on academic and research findings. It is important to keep on improving the manual because it has a significant impact in the design and traffic specification.

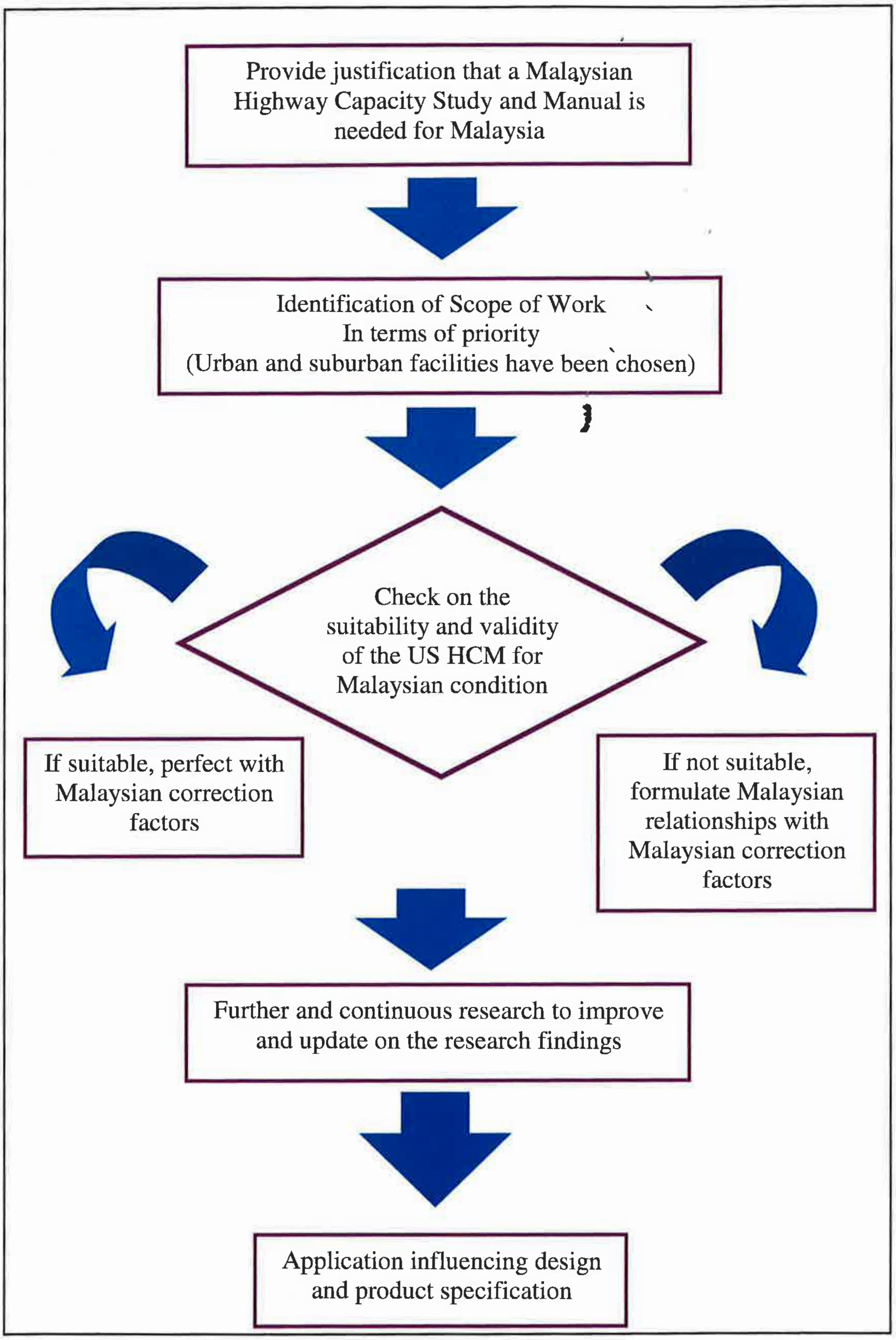


Figure 1.1 Approaches for The Malaysian Highway Capacity Study

1.1.1 IMPORTANCE OF CAPACITY

The capacity of signalised intersection, unsignalised intersection and suburban arterial indicates the ability of the facilities to accommodate a moving stream of people or vehicles. It is a measure of the supply side of any transportation facilities.

Accurate estimation of the capacity of transportation facilities are needed for most traffic engineering and transportation planning decisions and actions.

Capacity analysis is a set of procedures for estimating the traffic-carrying ability of facilities over the range of defined operational conditions. It provides tools to assess facilities and to plan and design improved facilities. It also estimates the maximum amount of traffic that a facility can accommodate while maintaining its prescribed level of operation.

The following four primary traffic engineering activities depend on capacity analysis:

1. When new facilities are planned or existing facilities are to be expanded, their size in terms of width or number of lanes must be determined.
2. When existing facilities are considered for upgrading, either by widening or by traffic operational changes, their operational characteristics and service levels must be assessed.
3. When new developments are planned, capacity and level of service analysis are needed to identify necessary traffic and roadway changes and to help define cost responsibilities.
4. Studies of operating conditions provide base values for determining changes in road-user costs, fuel consumption, air pollutant emission and noise.

1.1.2 PURPOSE OF MANUAL

The Malaysian Highway Capacity Manual (MHCM) provides transportation practitioner and researchers with a consistent and maintained system of techniques for the evaluation of the quality of service on highway and street facilities especially in urban and suburban area specific to Malaysian Road Condition. The facilities being considered are signalised intersections, unsignalised intersections and, urban and suburban arterials. The parameters and procedures in this manual provide a systematic and consistent method for assessing the capacity and quality of service for the transportation facilities. For each transportation facilities covered in this manual, the best frameworks from the US HCM (TRB, 2000) were applied and the respective correction factors with respect to Malaysian road condition were suggested.

1.1.3 SCOPE OF MANUAL

This manual presents operational, design and planning capacity analysis techniques for three major facilities.

I. Signalised intersections
II. Unsignalised intersections
III. Urban and Suburban arterials

1.1.4 ORGANISATION OF MANUAL

The first edition of the Malaysian Highway Capacity Manual contains 5 chapters and identified in Table 1-1.

Table 1.1 Organisation of Malaysian Highway Capacity Manual

Chapter	Description/Facility Type
1	Introduction, Concepts, and Applications
2	Traffic Characteristics
3	Signalised Intersections
4	Unsignalised Intersections
5	Urban and Suburban Arterials

In Chapter 1, the role and importance of capacity analysis are described, basic concepts are presented, and general guidelines for application are provided.

In Chapter 2, Traffic Characteristics and basic variables related to capacity are identified and their values and relationship as observed throughout Peninsular Malaysia are discussed.

Chapter 3 through 5 are the basic procedural chapters of the manual. They are organised according to the facility types presented in Table 1.1.

Each of the procedural chapters is generally organised in three distinct parts:

1. *Introduction*: The basic characteristics, concepts and philosophies of capacity analysis as applied to the subject facility are described.
2. *Methodologies*: The basic components of the analysis procedure to be applied to the specific facility are presented in the form of tabular and graphical information needed to complete the analysis are included.
3. *Procedure for application*: Step-by-step instructions for applying capacity analysis computations are presented. Procedures are specified for operational analysis,

design and planning, although not all chapters contain these three analysis level. Worksheets are provided for most computational procedures and are explained in detail.

1.2 CONCEPTS

1.2.1 CAPACITY AND LEVEL OF SERVICE

A principal objective of capacity analysis is the estimation of the maximum number of people or vehicles that can be accommodated by a given facility in reasonable safety within a specified time period (TRB, 2000). Facilities are rarely planned to operate at or near capacity.

According to TRB 2000, capacity is defined as the maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental and control condition. Capacity is important information to provide tools for the analysis of existing facilities and for the planning and design of improved or future facilities.

The concept of levels of service is the definition of operational criteria. Ranges of operating conditions are defined for each type of facility and are related to amounts of traffic that can be accommodated at each level.

The concept uses qualitative measures that characterize operational conditions within a traffic stream and their perception by motorist and passengers.

Six levels of service are defined for each type of facility for which analysis procedures are available. They are given letter designations, from A to F, with level of services (LOS) A indicating the best operating conditions and F is the worst. Each level of service represents a range of operating conditions.

The volume of traffic that can be served under the stop-and-go conditions of LOS F is generally accepted as being lower than possible at LOS E; consequently, service flow rate E is the value that corresponds to the maximum flow rate, or capacity, on the facility. Level of service C and D are usually used because to ensure an acceptable quality of service to users.

Service Flow Rates

The service flow rate is the maximum hourly rate at which persons or vehicles can be reasonably be expected to traverse a point or uniform section of a lane or roadway during a given period under prevailing roadway, traffic and control conditions while maintaining a designated level of service. It's generally based on a 15-min period. Typically, the hourly flow rate is defined as four times the peak 15-min volume.

Measures of Effectiveness

Table 1.2 shows the primary measure of effectiveness used in the Malaysian Highway Capacity Manual for the measurement of LOS definition.

Table 1.2 Primary Measures of Effectiveness for Level of Service Definition

TYPE OF FACILITY	MEASURE OF EFFECTIVENESS
Signalised intersections	Average controlled delay (sec/veh)
Unsignalised intersections	Average controlled delay (sec/veh)
Urban and Suburban Arterials	Average running speed (km/h)

Adapted from US HCM 94 and US HCM 2000

1.2.2 FACTORS AFFECTING CAPACITY AND LEVEL OF SERVICE

1.2.2.1 Ideal Conditions

An ideal condition is one for which further improvement will not achieve any increase in capacity. Ideal conditions is the condition represent of good weather, good pavement conditions, users familiar with the facility, and no incidents hold back traffic flow. Specific ideal conditions for each facility are identified.

Predictive adjustments must be included to reflect the absence of ideal conditions in order to evaluate the conditions that are not ideal in most capacity analysis. The prevailing conditions are generally divided into roadway, traffic and control conditions. Variation in vehicle control and technology represent conditions that change in the long term.

1.2.2.2 Roadway Conditions

Roadway factors include geometric conditions and design elements. These factors may affect the capacity of a road, and can change the result in the measurement of effectiveness, such as speed. Roadway factors include the following:

- The type of facility and its development environment.
- Lane width
- Lateral clearances and shoulder widths.

- Design speed.
- Alignments.
- Availability of queuing space at intersections.

The type of facility is critical, because the existence of other major facility type factors significantly affect flow characteristics and capacity. In several cases, the development environment has also been found to affect the performance.

Lane and shoulder widths can have significant impact on traffic flow. The vehicles will travel closer if the lanes are narrow. Motorists compensate by slowing down or observing larger longitudinal spacing for given speed, which effectively reduces capacity and service flow rates.

Narrow shoulders and lateral obstructions also have the effect to capacity and level of service. Motorist will try to move away from roadside or median objects to be safe and not to cause a hazard. Consequently, it will bring them laterally closer to vehicles in adjacent lanes and create the same response as those present in narrow lanes.

According to TRB, 2000, restricted design speeds affect the operations and level of service; drivers are forced to drive their vehicle at somewhat reduced speeds and to be more vigilant in reacting to changes in horizontal and vertical alignments resulting from a reduced design speed.

1.2.2.3 Traffic Conditions

Vehicle type and lane or directional distribution is the traffic conditions that influence the capacities and level of services. The procedures assume that drivers are familiar with the facility. Less efficient use of roadway facilities during weekends or roadways leading to recreation areas is generally attributed mainly to the lack of specific local knowledge.

Vehicle type

The capacity can be affected due to differences in vehicle composition. With respect to the Malaysian traffic characteristics, there are several classes of vehicles types, which can be categorized as in Table 1.3.

Table 1.3 Vehicle classifications in Malaysia

Class	Type of vehicle
1	Passenger car, taxi, pickup, small van
2	Lorry, large van, heavy vehicle with 2 axles
3	Large lorry, trailer, heavy vehicle with 3 axles and more
4	Bus
5	Motorcycle, scooter

The capacity in the traffic stream can be affected if there is the existence of heavy vehicles. Heavy vehicles are defined as vehicles having more than four tyres touching the pavement.

Heavy vehicles in Malaysia contain:

- Large van, lorry with two-axles
- Lorry with three and more axles
- Bus include school bus, factory bus and express bus

Heavy vehicles can have an impact as follows:

- They are larger than passenger cars and therefore occupy more roadway space than passenger car.
- They have poor acceleration, deceleration and the ability to maintain speed on upgrades.

Large gaps formed in the traffic stream because of these large vehicles cannot keep pace with passenger cars, and it is difficult for the passenger car to reduce the gap by undertaken manoeuvres. These gaps create inefficiencies in the use of roadway space.

Heavy vehicles also may affect downgrade operations, particularly where downgrades are steep enough to require operation of such vehicles in a low gear. This manoeuvre will also create large gaps in the traffic stream, due to heavy vehicles must operate at slower speed.

There is considerable variation in the characteristics and performance capabilities of vehicles within each class of heavy vehicle, just as among the passenger car.

In Malaysia, high percentage of motorcycles traverses through traffic stream and currently there is no proper consideration of traffic engineering aspects of motorcycles in the design.

Improper consideration of motorcycles in the design of junctions results in inaccurate design thus possibly can cause significant amount of traffic congestion.

The type of motorcycles prevalent on Malaysian road is that of small size motorcycles where the length of its wheel bases is about 0.5 meter. The size is small as compared to that of passenger cars where the length of its wheel bases is about 1.68 meter. Due to its small size, motorcycles can weave in and out of traffic stream especially when approaching signalised intersection. This enables the motorcyclist to get closer to the stop line, therefore the concept of FIFO (First in First out) is violated.

Motorcyclist's unique characteristic is that they can travel along side other vehicles within a lane. As a result, the flow is not in a structured discipline.

Directional and Lane Distribution

In this manual, the directional and lane distribution will affect the analysis of the signalised, unsignalised and arterial facilities.

1.2.2.4 Control Conditions

For signalised intersection, control in term of the time available for movement of specific traffic flow is a critical element affecting capacity, service flow rates and level of service. The type of control, signal phasing, green time allocation, cycle length and the relationship with the adjacent control measures affects operations at traffic light junction. It will be discussed widely in signalised intersection section.

Stop signs at unsignalised intersection also affect capacity. Motorist travelling on the minor street must find gaps in the major traffic flow. Thus, the capacity of such approaches depends on traffic conditions on the major street.

Restriction of curb parking can increase the number of lanes available on the street or highway. Turn restrictions can eliminate conflicts at intersections and subsequently increase the capacity. Lane use controls can positively allocate available roadway space especially during peak congestion period. Lane use control can be used at intersections and to create reversible lanes on critical arterials.

1.2.2.5 Technology

The factors in the preceding discussion generally relate to immediate conditions that would reduce roadway capacity below ideal conditions. Emerging transportation technologies such as Intelligent Transportation System (ITS) are being developed to enhance the safety and efficiency of road in Malaysia. This technology will allow real-time information to be gathered and used by drivers and traffic control system operators to provide better vehicle navigation, roadway system control, or both.

However, many of roadway improvement related to ITS are system level improvements such as incident response and driver information system, so are not expected to have impact in capacity for individual roadway facility.

1.2.3 SUMMARY

Table 1.4 shows the adjustment factors that are required for the three main type of facilities in the manual to deal with the less-than-ideal conditions. For each facility, the adjustment factors are group into three major categorized roadway, control and traffic characteristics.

The factors are divided into the roadway, control and traffic categories due to the following reasons:

- These variables are important factors involved in the capacity analysis computations.
- These conditions define the parameters that engineers and planners may consider changing in order to improve capacity and level of service.

Improvement to geometric factors such as lane width widening, shoulder width widening, increasing number of lanes, improving horizontal and vertical alignment, and other geometric factors can be done through constructions or spot improvement. The procedure in this manual will give the evaluation of alternative improvement plans based on such changes.

Table 1.4 Adjustment factors used in the analysis

Facility	Factors		
	Roadway	Traffic	Control
Signalised Intersections	Lane width Grade Number of lanes Type of lanes Turning radius	Peak hour factor Heavy Vehicles Right turns Left Turns Pedestrian activity Parking Bus stops	Green time Cycle length Signal progression
Unsignalised intersections	Grade Number of lanes Type of lanes Curb radius Area population Sight distance	Peak hour factor Heavy vehicles Turning movement	Stop control
Urban arterials	Lane width Grade Number of lanes Type of lanes Turning radius Arterial classification	Peak hour factor Heavy Vehicles Right turns Left Turns Pedestrian activity Parking Bus stops Free flow speed	Green time Cycle length Signal progression

Source : Transportation Research Board 1994

1.3 APPLICATION

1.3.1 MODELS OF TRAFFIC FLOW

In this manual, most of the traffic flow models were adopted from the US HCM (TRB, 1994 and TRB, 2000).

It is the job of the analyst to select the appropriate model for solving a given task. For situation that is not been covered in the manual, the analyst may used the model that produce outputs convertible to the measures of effectiveness used in this manual. They must recognize that differences may occur from the used of alternative methodologies, and to present results in the context of the model used.

1.3.2 LEVEL OF ANALYSIS

In this MHCM, most of the procedural chapters addressed three different computational applications:

- Operational
- Design
- Planning

Operational analysis is used for the detailed determination of the operating conditions. Generally, operational analysis identifies the existence and nature of a problem. Hence, in making any decision, an analyst first considers whether a given element, facility, area or system has a potential problem requiring study. In other words, the analyst decides if there is or will be a problem for any given facility.

However, operational analyses often do not end with the confirmation of a problem. Typically, several alternatives for improvement are proposed, leading to the next level of decision. The manual can be used to predict the change in performance measures for each alternative, to help the analyst to select and recommend remedial.

Design procedures, where provided, can be used to determine specific geometric or control parameters to yield the desired level of service, most commonly involve decisions on number of lanes, or the amount of space, needed to operate the facility. MHCM can be used to select among the alternative designs either by comparing the LOS at which the alternative design would operate or by finding the attributes of the design that result in a targeted level of service.

The planning analysis is more general, useful for long-term determination of the type and size of a facility. This kind of analysis is similar to an operating analysis, except it requires less information and field data for the inputs. It usually used the default values that will give the optimum performance to the facility.

It should be noted that for any given facility, the operation, design and planning analysis are based on the same principles and basic method. It is in the hand of the analyst or designer to select an appropriate level of analysis.

1.3.3 SUMMARY

Malaysian Highway Capacity Manual contains the analysis that are required to estimate performance of the three main facilities discuss in this manual, i.e. signalised intersections, unsignalised intersections and suburban arterials. Estimations are carried out based on known roadway, traffic and control conditions under current vehicle technology.

The MHCM is highly technical and complex especially for signalised and unsignalised intersections facilities. For decision making purposes, the result of the analysis can be difficult for the public to understand, unless the data are carefully organised and well presented. Decision makers who are not analytically oriented often prefer to have a single number or letter to represent traffic condition. As in the US HCM, LOS concept is the best way to interpret the analysis using the manual, so that the public and the elected officials can understand and make used of the manual.

Although this manual is an important guide for decision making, the results of capacity analysis do not replace the need to consider local, societal, environmental, behavioural and other specific requirements, constraints and conditions. The need of the professional judgement of the engineer or planner is a necessary input to such decisions.

CHAPTER 2

TRAFFIC CHARACTERISTICS

CHAPTER 2

2.0 TRAFFIC CHARACTERISTICS

CONTENTS

FIGURES

2.1 INTRODUCTION

In this chapter, we will introduce the variables that affect capacity and level of service of signalised intersections, unsignalised intersections and arterials. This chapter also discusses various characteristics of Malaysian traffic conditions. It is important to recognize the impact of these parameters to the facilities, especially some of the impact due to the unique behaviour of local condition.

2.2 BASIC VARIABLES OF TRAFFIC FLOW

2.2.1 VOLUME AND FLOW RATE

Referring to the US HCM (TRB, 2000), volume is stated as the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval. It can be expressed in terms of annual, daily, hourly or sub hourly periods.

Flow rate is the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than 1 hour, usually 15 minutes.

The different between volume and flow rate must be understand carefully. Volume is the number of vehicles observed or predicted to pass a point during a time interval (TRB, 2000). Flow rates represent the number of vehicles passing a point during a time interval (TRB, 2000) less than an hour, but expressed as an equivalent hourly rate. For example, a volume of 100 vehicles observed in a 15-min period implies a flow rate of 100 veh/0.25 hour.

Peak-hour factor or PHF is estimated using the peak flow rates and hourly volumes. Equation 2.1 is used to compute PHF.

$$PHF = \frac{\text{hourly volume}}{\text{Peak flow rate}} \tag{2.1}$$

Equation 2.2 is used to estimate PHF if a 15 minutes period is used.

$$PHF = \frac{V}{4V_{15}} \tag{2.2}$$

Where

PHF = peak-hour factor

V = hourly volume (veh/hr)

V_{15} = volume during the peak 15 min. of the peak hour (veh/15 min)

Equation 2.3 is used to convert a peak-hour volume to a peak flow rate is PHF is known.

$$v = \frac{V}{PHF} \tag{2.3}$$

Where

v = flow rate for a peak 15-min. period (veh/hr)

V = peak-hour volume (veh/hr)

PHF = peak-hour factor

2.2.2 DENSITY

Density is a traffic parameter because it categorizes the quality of traffic operations. It reflects the freedom to manoeuvre within the traffic stream. However, density cannot be directly observed in the field, it can be estimated using Speed-Flow relationship. Sometimes, roadway occupancy is used as a surrogate measure for density because it is easy to calculate and measure.

Occupancy in space

Defined as the proportion of roadway length covered by vehicles.

Occupancy in time

Defined as the proportion of time a roadway cross section is occupied by vehicles.

2.2.3 SPACING

Spacing is defined as the distance between following vehicles in a road stream, as measured from front bumper to front bumper. Spacing can be measured directly in the field by measuring the distance between common points on following vehicles at a particular time. The average vehicle spacing also reflected the density in a traffic stream.

2.2.4 HEADWAY

Headway is the time between successive vehicles as they pass a point on a lane or roadway, usually measured from front bumper to front bumper. Usually being measured using a stopwatch observations as vehicles pass a point on the road.

Average headway of the traffic stream also affected the flow rate of the stream. The smaller the value of the average headway, the higher it is for flow rate of the stream. A small headway indicates very high traffic volume.

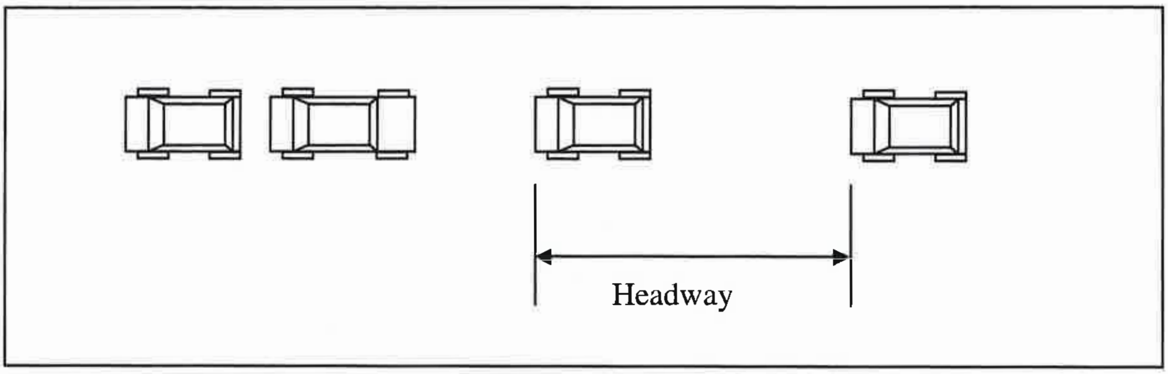


Figure 2.1 The definition of headway

2.2.5 SPEED

Speed is defined as a rate of movement expressed as distance per unit of time. The unit for speed is kilometres per hour (km/hr). For urban and suburban arterials, average travel speed is used as the speed measure because it is easily computed from observation of individual vehicles within the traffic stream. Average travel speed is computed by dividing the length of the highway, street section, or segment under consideration by the average travel time of the vehicles traversing it (TRB, 2000). Equation 2.4 shows how the average travel speed is computed.

$$S = \frac{nL}{\sum_{i=1}^{n} t_i} = \frac{L}{\frac{1}{n}\sum_{i=1}^{n} t_i} = \frac{L}{t_a} \qquad (2.4)$$

Where

S	= average travel speed (km/hr)
L	= length of the highway segment (km)
t_i	= travel time of the i^{th} vehicle to traverse the section (hr)
n	= number of travel time observed
t_a	= average travel time over length of the highway segment, L (hour)

The travel time used in this computation includes stopped delays due to fixed interruptions of traffic congestion. They are total travel times to traverse the defined roadway length.

Several speed parameters can be used in discussing the quality of traffic stream. The definition of each parameter is based from the US HCM 2000.

Average running speed

Average running speed is a traffic stream measure based on the observation of vehicle travel times traversing a section of highway of known length. It is the length of the segment divided by the average running time of vehicles to traverse the segment. Running time only takes into consideration when the vehicle is in motion.

Average travel speed

Measurement based on travel time observed on a known length of highway. It is the length of the segment divided by the average travel time of vehicles traversing the segment, including all stopped delay times.

Space mean speed

A term to represent an average speed based on the average travel time of vehicles to traverse a segment of roadway. It is called a space mean speed because it is the average time each vehicle spends in the defined roadway segment or space.

Time mean speed

The arithmetic average of speeds of vehicles observed passing a point on a highway, also referred to as the average spot speed. The individual speeds of vehicles passing a point are recorded and averaged arithmetically.

Free flow speed

The average speed of vehicles on a given facility, measured under low-volume conditions, when drivers tend to drive at their desired speed and are not constrained by control signal.

2.2.6 RELATIONSHIPS BETWEEN VARIABLES

There are three main parameters for traffic theory i.e. speed, density and flows. The relationship using Greenshield estimation between the three parameter are shown in Figure 2.2, 2.3 and 2.4.

In Figure 2.2, the relationship between speed and density is assumed to be linear. Based on the linear relationship, the flow-density relationship is obtained as shown in Figure 2.3. Similarly, the speed-flow relationship is as shown in Figure 2.4.

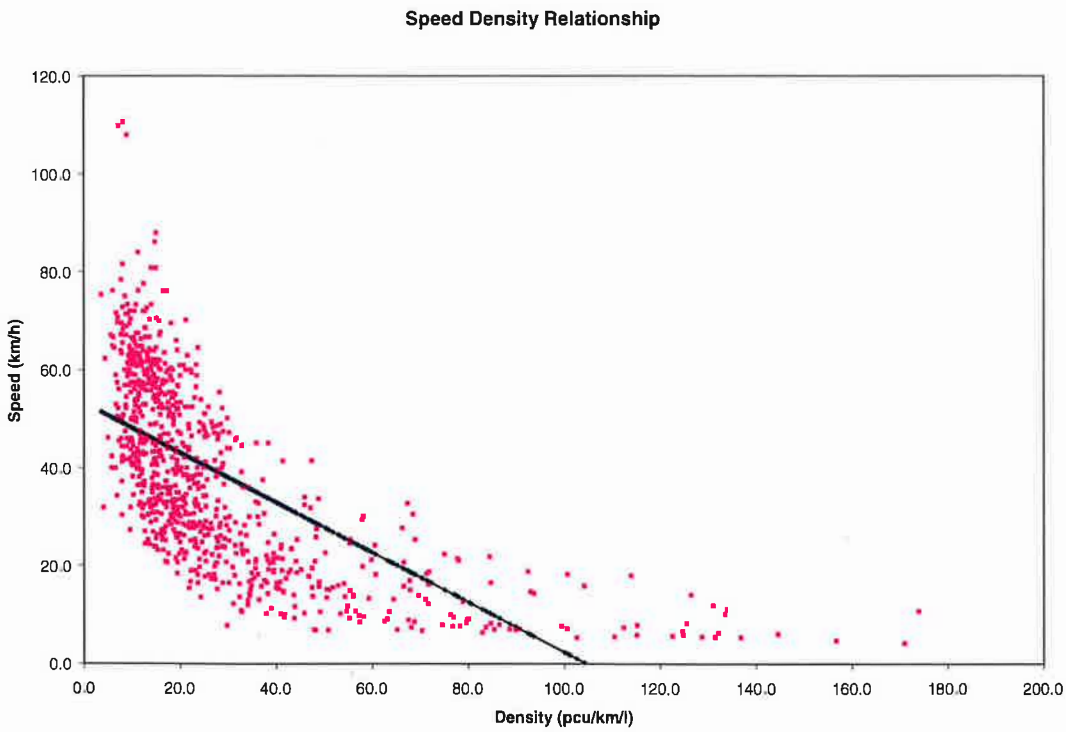


Figure 2.2 The Speed vs. Density Relationship based on Malaysian Road Traffic

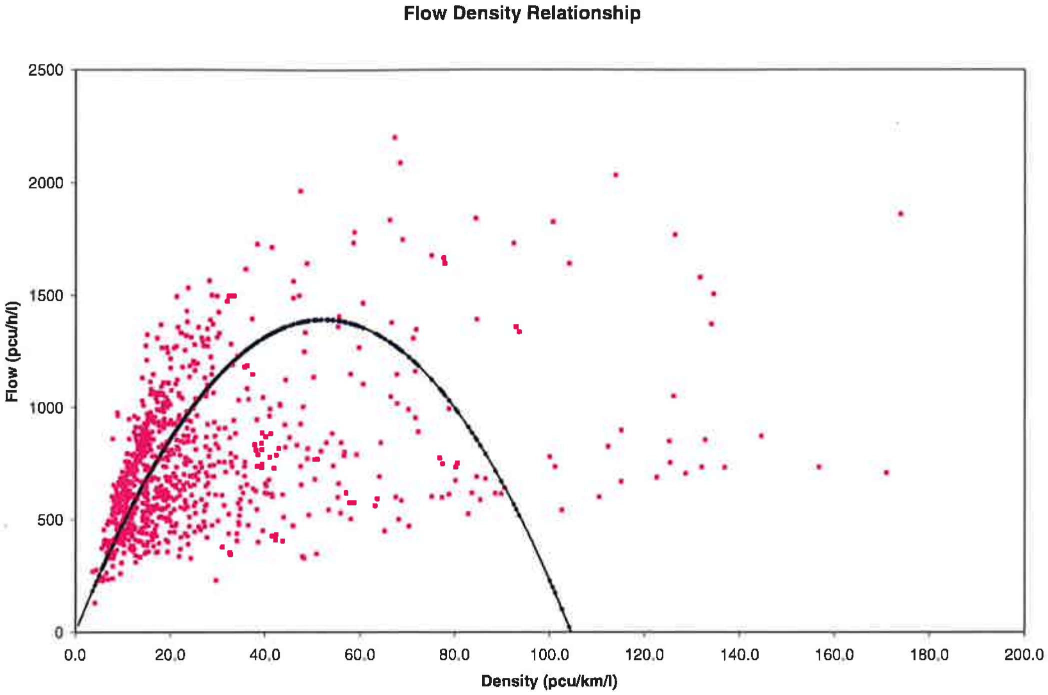


Figure 2.3 The Flow vs. Density Relationship according to real data from site in Malaysia

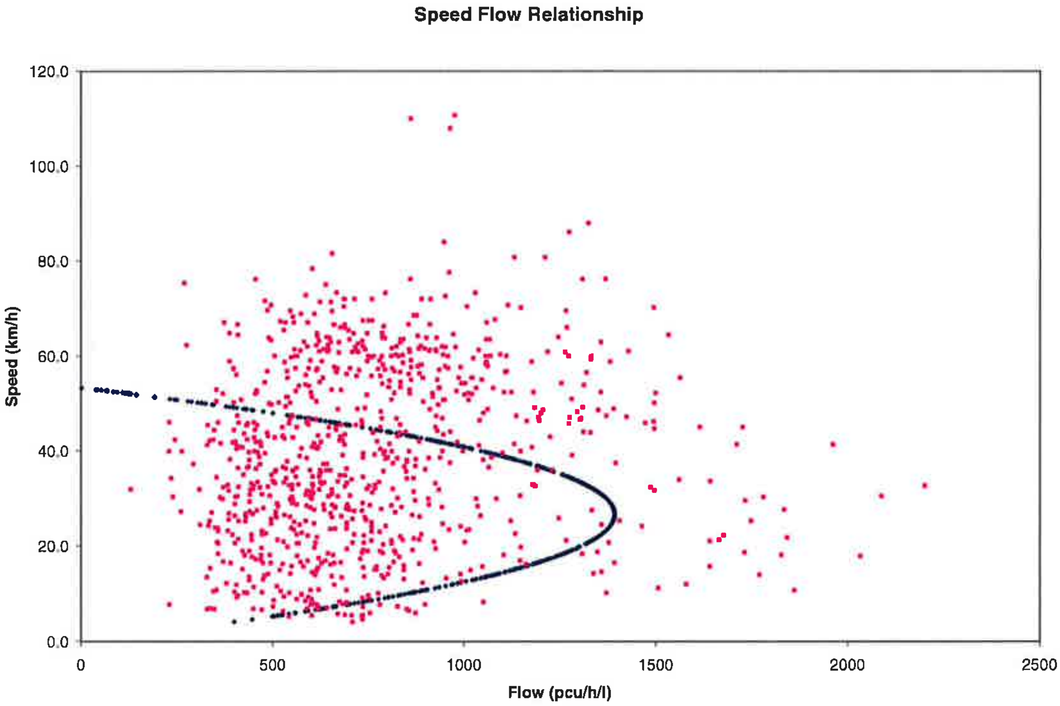
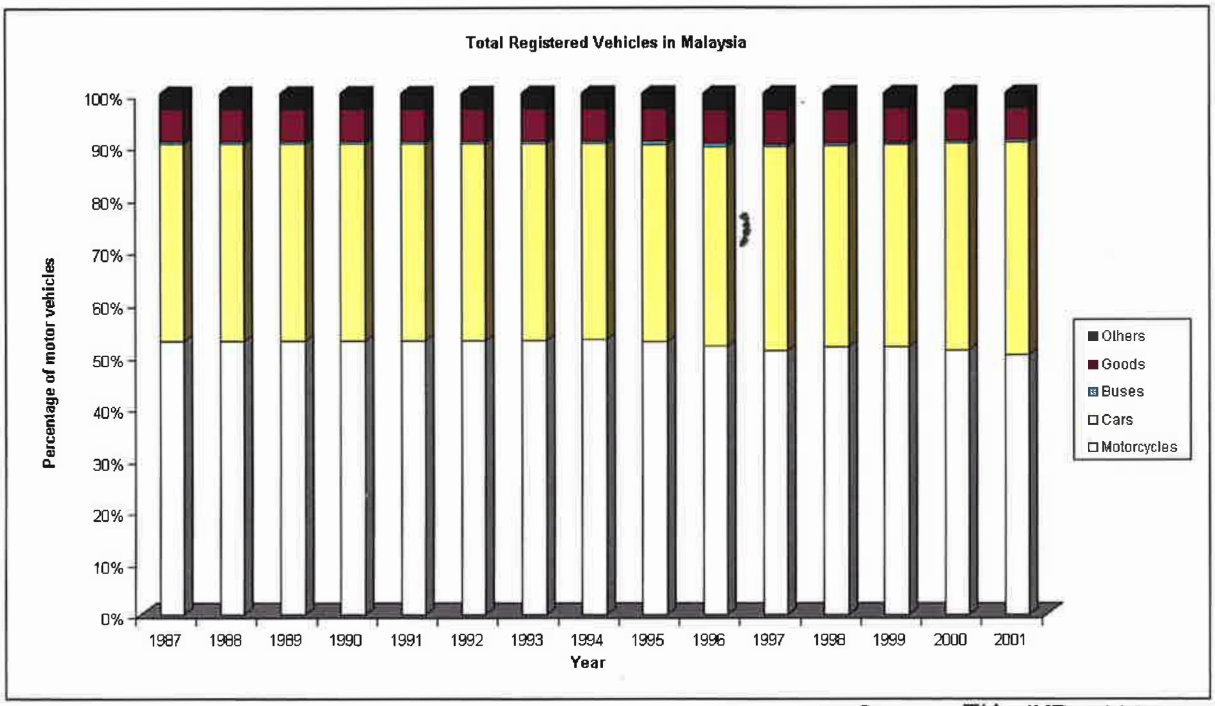


Figure 2.4 The Speed vs. Flow Relationship based on Malaysian Traffic Condition

2.3 OBSERVED VALUE

Malaysia has high number of motorcycles as compared to other western countries. The vehicles composition registered annually in Malaysia consists mainly of passenger cars, motorcycles, buses and goods vehicles as illustrated in Figure 2.5. It clearly shows that the percentage of motorcycles registered annually is about 50% - 60%.



Source : TIA, JKR, 2000

Figure 2.5 Percentage of registered vehicles in Malaysia

Distribution of traffic volume is varied by week, days and hourly. It depends on how motorist travel during weekdays and weekend. Furthermore, the peak hour is varied by day. The typical morning and peak hours are obvious for urban motorist to use the facility on weekdays. Usually, the evening peak is intense than the morning peak.

Figure 2.6 and 2.7 shows the flow pattern along Jalan Masjid Negeri, Georgetown, Penang. It indicates the peak hour based on the highest volume recorded during normal working day on a school holiday.

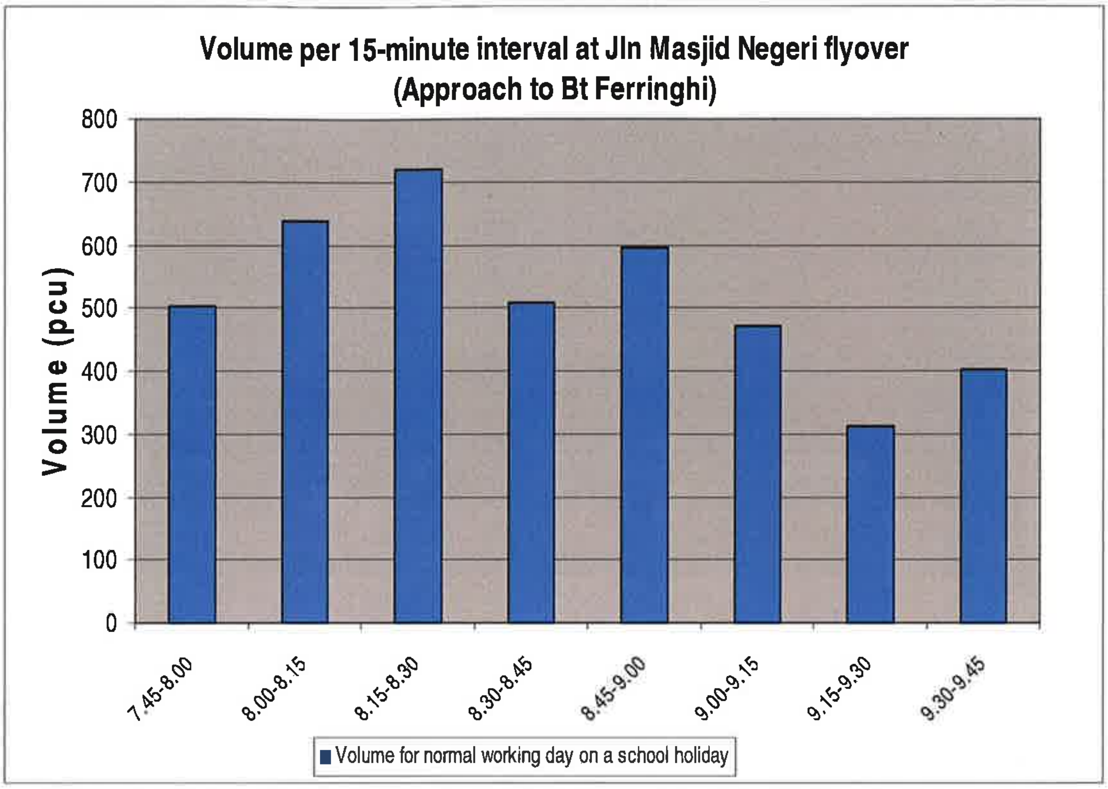


Figure 2.6 Volume (pcu) per 15-minutes interval at Jalan Masjid Negeri, Georgetown – approach to Batu Feringgi for normal working day on a school holiday.

Figure 2.6 shows traffic volume in pcu per hourly interval during morning traffic activities, whereas figure 2.7 shows traffic volume in pcu per hourly interval recorded during evening traffic activities.

Whereas, Figure 2.8 to Figure 2.9 show traffic volume at Jalan Masjid Negeri for normal working days on a school holiday. The trend of traffic volume at Jalan Masjid Negeri during school holiday is higher than during normal days. The result is as expected because many motorists visit Pulau Pinang heading toward Batu Feringghi during school holidays as compared to normal working days.

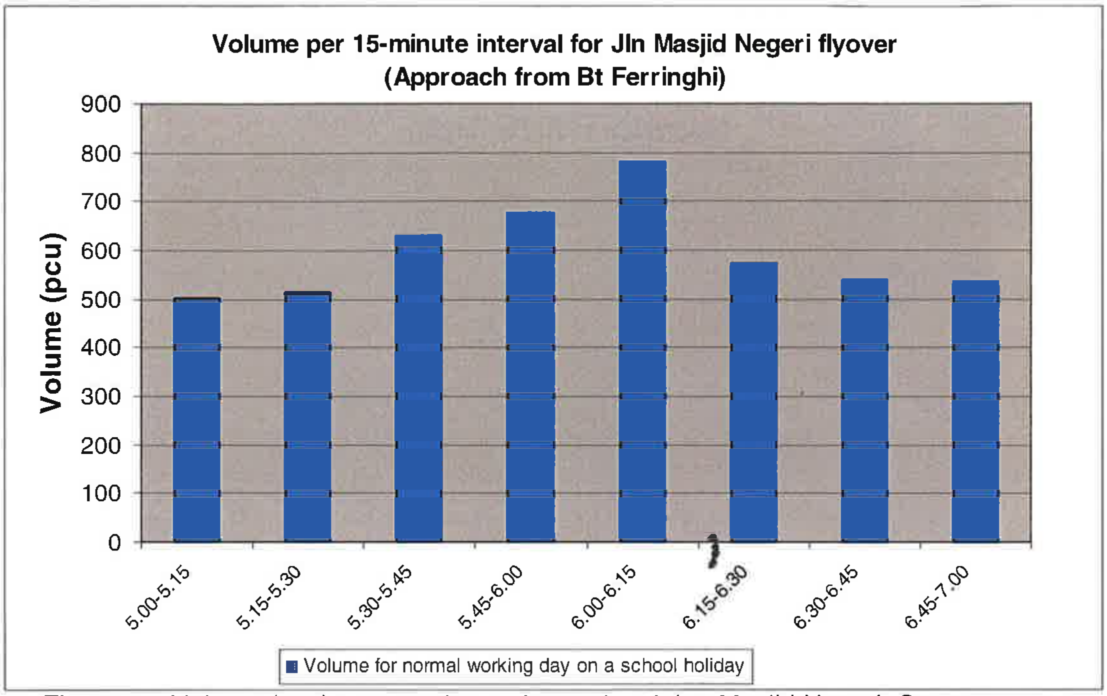


Figure 2.7 Volume (pcu) per 15 minutes interval at Jalan Masjid Negeri, Georgetown – approach from Batu Feringgi for normal working day on a school holiday.

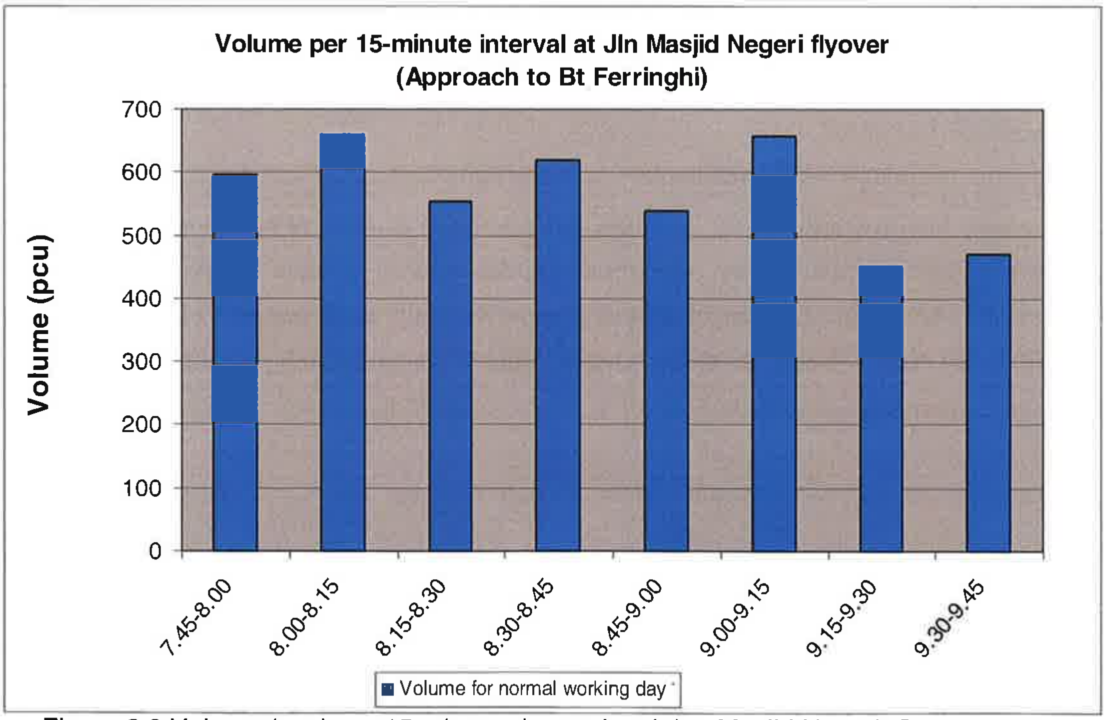


Figure 2.8 Volume (pcu) per 15 minutes interval at Jalan Masjid Negeri, Georgetown – approach to Batu Feringgi on normal working day.

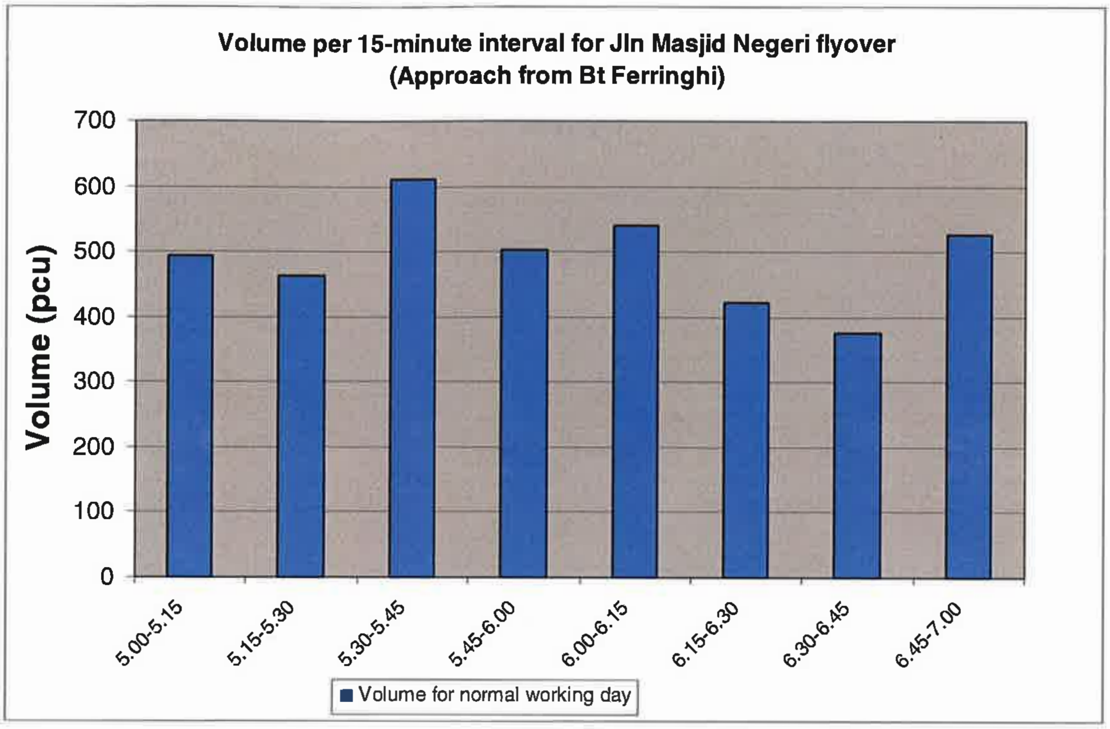


Figure 2.9 Volume (pcu) per 15 minutes interval at Jalan Masjid Negeri, Georgetown – approach from Batu Feringgi on normal working day.

As for Figure 2.10, the differences between the volumes at Jalan Masjid Negeri flyover during normal day and school holiday are as shown. It is clear that during school holiday more vehicles are using the road.

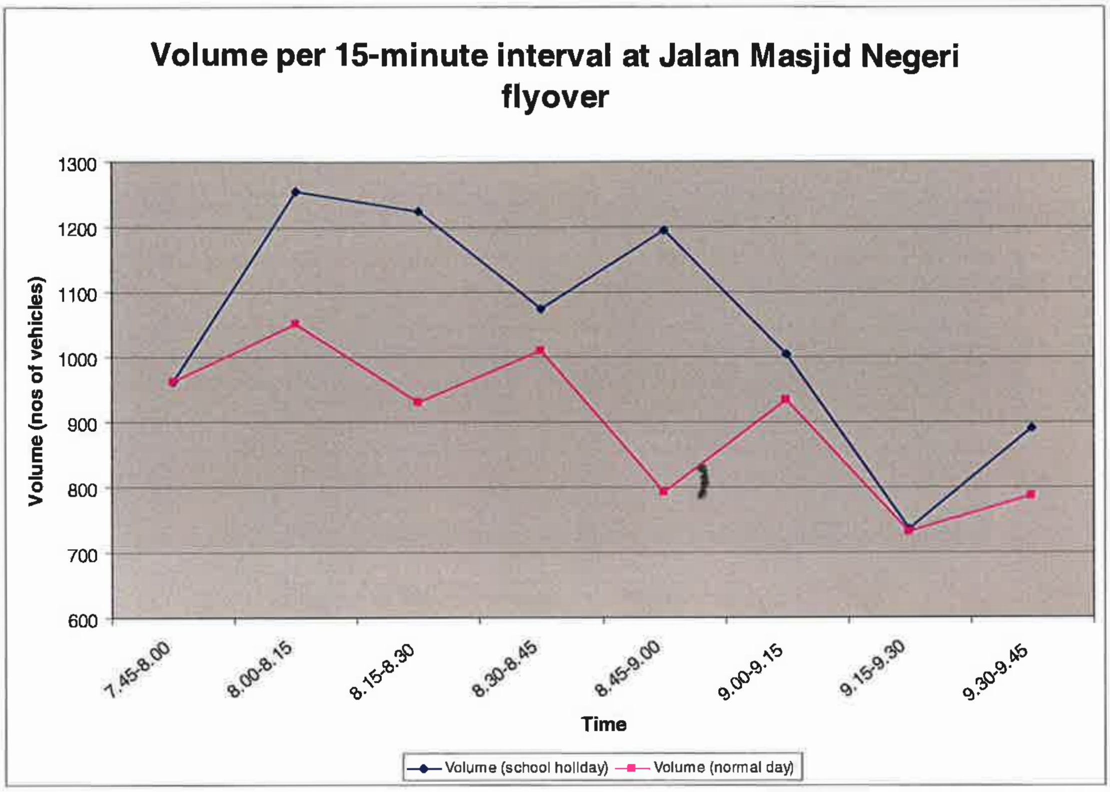


Figure 2.10 Volume (number of vehicle) per 15 minutes interval at Jalan Masjid Negeri, Georgetown

CHAPTER 3

SIGNALISED INTERSECTIONS

CHAPTER 3

3.0 SIGNALISED INTERSECTIONS

CONTENTS

FIGURES

TABLES

3.1 INTRODUCTION

The performance of signalised intersections affects the performance of the road network. The signalised intersections play a crucial role in the performance of arterial streets, particularly in urban areas where traffic congestion has always been a major problem due to the annually increasing number of vehicles in Malaysia. It is imperative to design signalised intersections accurately according to proper design standard in order to maximize the operational efficiency of the intersections and at the same time to ensure safety of the road users.

Capacities of signalised intersections are determined based on individual lane and also based on groups of lanes per approach. One of the parameters for estimating capacities is saturation flow rate. Current practice in estimating saturation flow rates of an intersection approach under prevailing conditions is by applying adjustment factors to account for the effects of roadway, vehicle composition, proportion of turning vehicles and other related factors to the predetermined ideal saturation flow.

The methodology presented here is only applicable for isolated signalised intersections with three or four leg approaches.

3.1.1 LEVEL OF SERVICE

Level of service is directly related to the controlled delay value. The average controlled delay per vehicle is estimated for each lane group and aggregated for each approach and for the intersection as a whole (TRB, 2000). Although v/c affects delay, there are other criteria that more strongly affect it, such as the quality of progression, length of green phases, cycle length and others. Table 3.1 shows the controlled delay threshold used to estimate level of service.

Table 3.1 Levels of Service for Signalised Intersections

Level of service	Controlled delay per vehicle (sec)
A	≤ 10.0
B	> 10.0 – 20.0
C	> 20.0 – 35.0
D	> 35.0 – 55.0
E	> 55.0 - 80.0
F	> 80.0

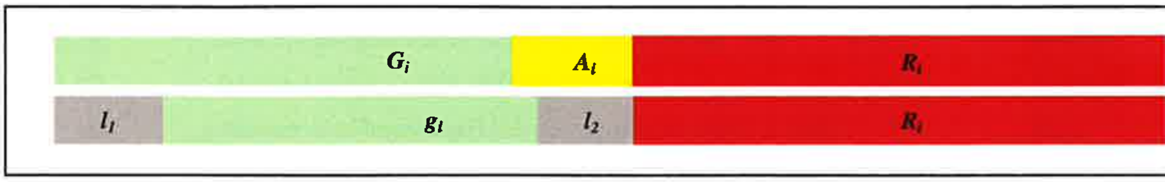
*Source adapted from US HCM 2000

3.1.2 CHARACTERISTICS OF TRAFFIC SIGNAL

For all signalised intersections, only three signal indications are used, specifically green, amber and red. The red indication sometimes includes a short period during which all signal phases are red. This interval is referred to as an "all-red" interval. The sum of amber interval and all-red interval forms the clearance and change interval between two green phases which provides clearance for the intersection before conflicting traffic movements are given the green indication.

For every signal cycle, two lost times are experienced. At the beginning of green interval, the first several vehicles in the queue experience start-up lost time as they accelerate to normal running speed. At the end of green interval, the flow of vehicles declines as there is a portion of the clearance plus change intervals that is not used for vehicular movement. Therefore, the total lost time for a signal cycle is the sum of start-up lost time and clearance lost time. Although the lost times may vary from phase to phase, for practical purposes, they are assumed to be constant.

Hence, effective green time is the time effectively used by the signal for traffic movement. It is determined by taking the actual green time plus the change interval minus the lost time for a signal cycle. The relationship between the actual green time with effective green time is as shown in Figure 3.1.



Source: US HCM 1994

G_i = Actual green time allotted for the signal cycle
A_i = Sum of actual amber time plus all-red interval allotted for the signal cycle
R_i = Actual red time exclusive of the all-red interval allotted for the signal cycle
g_i = Effective green time for the signal cycle
l_1 = Initial lost time for the signal cycle
l_2 = End lost time for the signal cycle

Figure 3.1 Relationship between Actual Green and Effective Green Time

Hence,

$$g_i = G_i + Y_i - (l_1 + l_2) = G_i + Y_i - l_T \quad (3.1)$$

where

$$l_T = \text{Total lost time for a signal cycle} = l_1 + l_2 \quad (3.2)$$

And

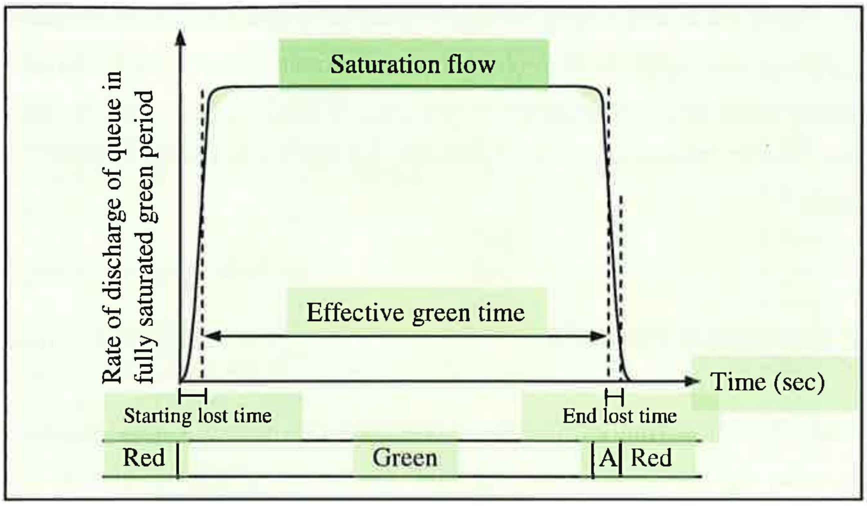
$$r_i = R_i + l_T \quad (3.3)$$

where

r_i = Effective red time for the signal cycle

3.1.3 CONCEPT OF SATURATION FLOW

Saturation flow is the maximum constant departure rate of a queue from the stop line of an approach lane during the green period. A small change in the saturation flow value may result in a relatively large change in the calculated cycle time and the duration of the necessary green intervals. It is the most important single parameter in the capacity analysis of signalised intersections (Akcelik, 1981).



Source: kimber et al (1986)

Figure 3.2: Saturation Flow Concept

The saturation flow concepts can be illustrated using Figure 3.2. It is assumed that after an initial hesitation immediately following the beginning of the green interval, traffic discharges at a constant rate (the saturation flow rate) until the queue is diminished or until shortly after the beginning of amber, when a sharp drop in the flow occurs. The departure rate is lower during the first few seconds, while vehicles accelerate to normal running speed, and after the end of the green interval, as the flow of vehicles declines (Teply & Jones, 1991).

A properly designed signalised intersection usually results in less traffic congestion and accident rates usually will reduce. Performance of a signalised intersection is influenced by several important traffic parameters and one of them is saturation flow. Saturation flow is the basis for determination of traffic signal timings and for the evaluation of intersection performance.

The ability to predict saturation flows is crucial to the design of signalised intersections. For existing signalised intersections, saturation flows can be measured directly in the field using standard methods. However, at the design stage for proposed new intersection, it is necessary to make predictions from other known factors (Kimber et al., 1986). So, in order to design this type of new signalised intersections, generalised predictive formulae are needed for estimating the saturation flow (Kimber et al., 1986). The current practice in estimating saturation flow rates of an intersection approach under prevailing conditions is by applying adjustment factors to account for the effects of roadway, vehicle composition, proportion of turning vehicles and other related factors to the predetermined ideal saturation flow.

Either measured or estimated, the determination of saturation flow involves consideration of both roadway and traffic factors (Asri Hasan & Radin Umar, 1993). Roadway conditions include basic geometric configuration of the intersection, in particular its width, grades and curvatures. Traffic conditions include volumes, vehicle movements (through, right or left) and vehicle types.

3.1.4 SATURATION FLOW RATE

The saturation flow rate is the flow, in vehicles per hour per lane (veh/h/ln), that can be accommodated by the lane assuming that the green phase is always available to the approach. This definition do not mean that there is a continuous hour of green, but imply the usual stopping and moving operation for the normally used range of cycle times and green intervals; thus, saturation flow rate reflects the uniform service rate used in most applications of queuing theory for analyzing of intersection capacity.

3.1.4.1 Ideal saturation flow rate

Ideal saturation flow rate is an important parameter in the saturation flow estimation procedure. It is the saturation flow rate of a twelve-foot (3.66 m), straight through only lane in non-CBD area, with level gradient, no bus blockages and no adjacent parking activities.The ideal saturation flow rate for Malaysian road condition is 1930 pcu/hr/ln.

The value of 1,930 pcu/hr/ln obtained using pce values derived by headway ratio method seems to correspond well with the value of 1,950 pcu/hr/ln used in aaSIDRA (Akcelik, 2000). As shown in Table 3.2, in the U.S. HCM (1985), the ideal saturation flow presented was 1,800 pcu/hr/ln but based on the 1994 and 2000 versions, the ideal saturation flow rate was 1,900 pcu/hr/ln. In Australia, the ideal saturation flow rate adopted in the Research Report No. 123 (Akcelik, 1981) was only 1,850 pcu/hr/ln and the ideal saturation flow rate reported by Webster and Cobbe (1966) in United Kingdom was 1,900 pcu/hr/ln. However, the highest ideal saturation flow rate reported was in the Indonesia HCM (BINKOT, 1996) which was 2,196 pcu/hr/ln.

Table 3.2 Different Ideal Saturation Flow Rate Based On Country

Countries	Sources		Ideal saturation flow rate (pcu/hr/ln)
Malaysia	Malaysian Highway Capacity Manual (2005)		1,930
	Arahan Teknik (Jalan) 13/87 (Ministry of Works, 1987)		1,904
U.S.	Highway Capacity Manual	1985	1,800
		1994	1,900
		2000	1,900
Australia	aaSIDRA (Akcelik, 2000)		1,950
	Research Report ARR No. 123 (Akcelik, 1981)		1,850
U.K.	Webster and Cobbe (1966)		1,900
Indonesia	Indonesia HCM (BINKOT, 1987)		2,196

Saturation flows are very much influenced by the proportion and type of vehicles in the traffic stream. Therefore, it is a usual practice to assign weighting factors or passenger car equivalents (pce) to the various categories of vehicle so that saturation flow can be corrected to the common base of passenger car units per hour per lane (pcu/hr/ln). The pce values currently used in Malaysia were also adopted with slight adjustment to the values obtained by Webster in United Kingdom in the 50's and 60's and have not been revised since the publication of Arahan Teknik (Jalan) 13/87. Due to certain differences such as drivers' behaviour, traffic composition and roadway characteristics, these values may not be representative of local traffic conditions in Malaysia. Hence, this manual will introduce new pce values based on Malaysian road conditions.

3.1.4.2 Passenger Car Equivalent (pce)

Passenger Car Equivalent (pce) is defined as the number of passenger cars displaced in the traffic flow by a truck, bus, motorcycle or any other vehicle, under the prevailing roadway and traffic conditions. PCE is important in delivering the value of saturation flow rate.

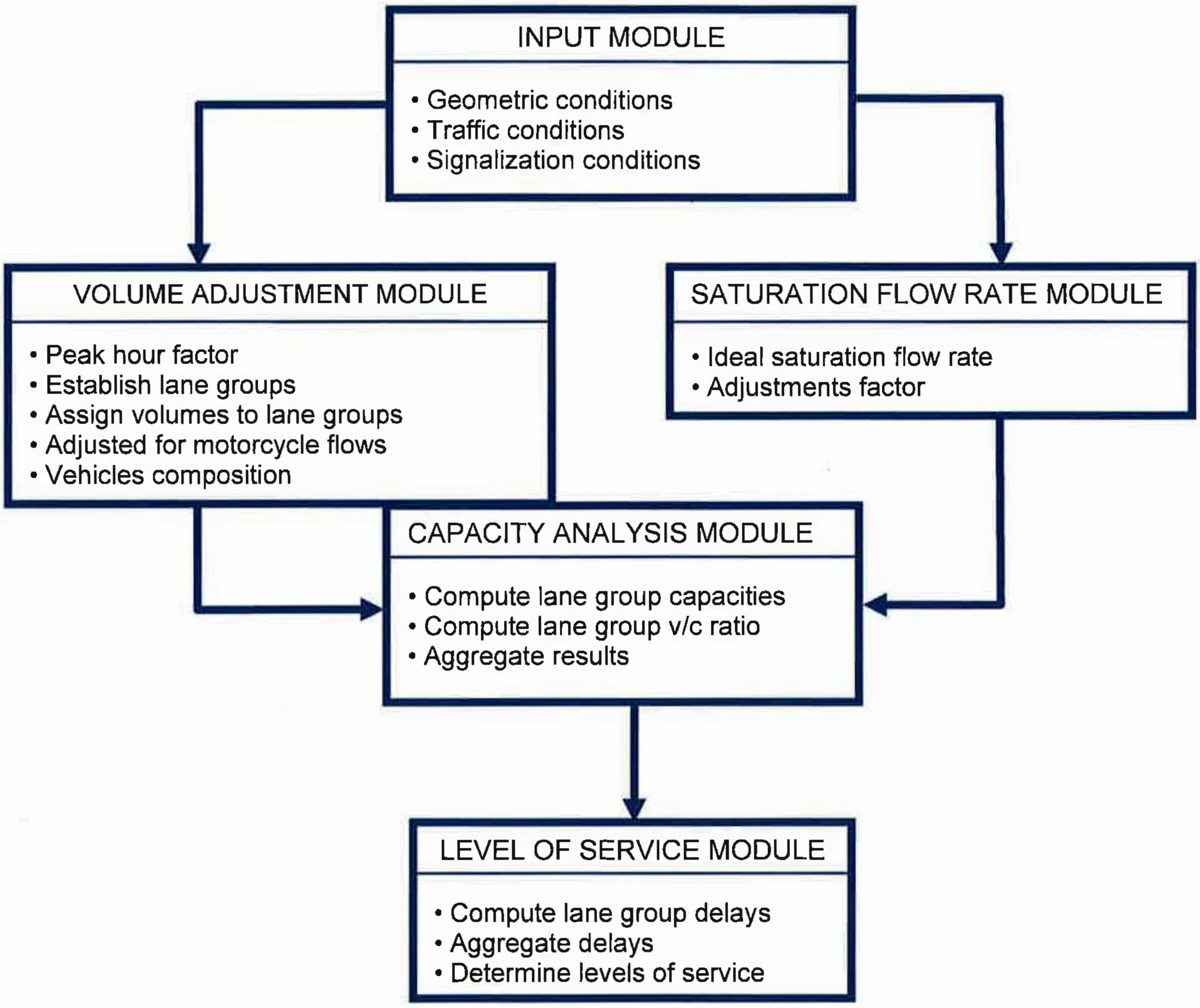
Saturation flow rate measured in vehicles per hour per lane depends on the proportion and type of vehicles in the traffic stream. It is, therefore, usual practice to assign weighting factors or passenger car equivalent (pce), to the various categories of vehicle so that flows can be corrected to the common base of passenger car units per hour (pcu/hr) (Kimber *et al*, 1986). In other words, pce values represent the effect of varying vehicle types on the capacity of an approach in a signalised intersection relative to the passenger car. In Malaysia, the pce values for signalised intersection are as shown in Table 3.3.

Table 3.3 Pce Values for Through Vehicles at Signalised Intersections in Malaysia

Vehicle types	Pce values
Cars, e_{car}	1.00
Motorcycles, e_{motor}	0.22
Lorries, e_{lorry}	1.19
Trailers, $e_{trailer}$	2.27
Buses, e_{bus}	2.08

3.2 METHODOLOGY

Figure 3.3 shows the inputs that are required in determining the capacity and level of service for each lane group as well as the level of service for the entire intersection. The methodology consists of several modules, i.e. input, volume adjustment, saturation flow rate, capacity analysis and level of service.



Adapted: US HCM 1994

Figure 3.3 Signalised Intersection Methodologies

This methodology is an adaptation from the US HCM 1994 methodology. In this manual, the effect of motorcycles is being taken into consideration in analysis of signalised intersection and determining level of service. The different is the way to treats motorcycle in existing flow and how to adjust the volume in each approach.

Input Module

The module includes all necessary information and data on geometry, traffic volumes and conditions, including signalisation at the intersection. Table 3.4 indicates the input data need to be prepared for each analysis.

Volume Adjustment Module

Volume is usually represented in term of vehicles per hour for a peak hour. The module converts these to flow rates for a peak 0.25-hours or 15-minutes analysis period.

Saturation Flow Rate Module

The module computes the saturation flow rate for each of lane groups. The flow rate is based upon the adjustment of an 'ideal' saturation flow rate to reveal a variety of existing conditions.

Capacity Analysis Module

Volumes and saturation flow rate are manipulated to compute the capacity and v/c ratios for each lane group. It also computes the critical v/c ratio for the whole intersection.

Level of Service Module

The module estimates delay for each lane group established for the analysis. Delay is measured for each approach and for the intersection as a whole, and levels of service are determined.

Table 3.4 Input Data Needs for Each Analysis Lane Group

Type of Condition	Parameter	Symbol
Geometric conditions or Characteristic	Area Type	CBD, Non CBD
	Number of Lanes	N
	Average Lane Widths; meter	W
	Grades; %	%G
	Existence of Exclusive RT or LT Lane	None
	Length of Storage Bay, LT or RT Lane	L_s
	Parking Conditions	Yes, No
Traffic Conditions	Volumes by Movement; km/hr	V
	Ideal Saturation Flow Rate; pcphgpl	S_o
	Peak Hour Factor	PHF
	Vehicles Composition	f_c
	Conflicting Pedestrian Flow Rate; peds/hr	PEDS
	Local Buses Stopping in Intersection	N_B
	Parking Activity, pkg manoeuvres/hr	N_m
	Arrival Type	AT
	Proportion of vehicles arriving on green	P
	Approach Speed, km/hr	S_A
Signalised Conditions	Cycle Length, sec	C
	Green Time, sec	G
	Amber-Plus-All-Red Change and Clearance Interval (intergreen), sec	A_i
	Actuated or Pretimed operation	A or P
	Pedestrian push-button	Yes, No
	Minimum Pedestrian green, sec	G_p
	Phase Plan	None
	Analysis Period, hr	T

3.2.1 DETERMINING FLOW RATE

A peak 15-minutes flow rate is derived from an hourly volume by dividing the movement volumes by an appropriate Peak Hour Factor (PHF), which may be defined for the intersection as a whole, for each approach, or for each movement. The flow rate is computed using equation 3.3.

$$v_p = \frac{V}{PHF} \qquad (3.3)$$

The conversion of hourly volumes to peak flow rates using PHF assumes that all movements peak during the same 15-minutes period. So, it is valuable to observe 15-minutes flows directly and select the critical periods for the analysis.

3.2.2 DETERMINING SATURATION FLOW RATE

Saturation flow rate under prevailing conditions is estimated using equation 3.4. The ideal saturation flow rate for Malaysian road condition is 1930 passenger cars per hour of green. The ideal saturation flow is adjusted to take into consideration non-ideal condition. For Malaysian traffic condition, differences in vehicles composition is taken into consideration using f_c factor.

$$S = S_o \times N \times f_w \times f_g \times f_a \times f_{LT} \times f_{RT} \, x \, (1/f_c) \qquad (3.4)$$

where

S = Saturation flow rate under prevailing conditions, expressed in vehicle per hour of effective green time

S_o = Ideal saturation flow rate which is **1,930** passenger cars per hour of green time per lane

N = number of lanes in the lane group

f_w = adjustment factor for lane width (12 ft / 3.66 meter lanes are standard)

f_g = approach grade adjustment factor

f_a = area type adjustment factor

f_{RT} = right turning in the lane group adjustment factor

f_{LT} = left turning in the lane group for adjustment factor

f_c = vehicle composition correction factor ($f_{car} + f_{HV} + f_{motor}$)

f_{HV} = adjustment factor for heavy vehicles (any vehicle having more than four tires touching the pavement)

f_{car} = adjustment factor for passenger cars

f_{motor} = adjustment factor for motorcycles

3.2.3 LANE WIDTH ADJUSTMENT FACTOR

The lane width adjustment factor, f_w, accounts for the narrowing effect of lanes on saturation flow rate and allows for an increased in flow rate, due to wider lanes. As for the lane width adjustment factor, equation 3.5 is used when lane widths under consideration were less than or exceeding 3.66 meter. Table 3.5 shows the tabulation of lane width adjustment factor.

$$f_w = 1 + \frac{w - 3.66}{3.663} \tag{3.5}$$

Table 3.5 Adjustment Factor For Average Lane Width *(f_w)*

Average lane width, w (meter)	Lane width factor (f_w)
2.90	0.793
3.00	0.820
3.10	0.847
3.20	0.874
3.30	0.902
3.40	0.929
3.50	0.956
3.60	0.984
3.66	1.000
3.70	1.011
3.80	1.038
3.90	1.066
4.00	1.093

Note: Applicable only for lane width between 2.9 and 4.0 meter

Comparison of lane width adjustment factors to estimate saturation flow rates are as shown in Table 3.6. The unit of the estimated saturation flow rate is either in pcu/hr or in veh/hr depending on its sources. For the MHCM, the unit for saturation flow rate is veh/hr.

Table 3.6 Comparison Of Lane Width Adjustment Factors Used In Various Saturation Flow Models

Sources	Saturation flow prediction models	Lane width unit	Saturation flow unit
aaSIDRA (Akcelik, 2000)	$S = 1{,}950 \times \left(1 + \frac{w - 3.3}{19.23}\right)$	Meter	pcu/hr
Indonesia HCM (BINKOT, 1996)	$S = 600w$	Meter	pcu/hr
U.S. HCM (1994)	$S = 1{,}900 \times \left(1 + \frac{w - 12}{30}\right) \times f_{HV}$	Feet	veh/hr
U.S. HCM (2000)	$S = 1{,}900 \times \left(1 + \frac{w - 3.66}{9}\right) \times f_{HV}$	Meter	veh/hr
Malaysian Highway Capacity Manual	$S = 1{,}930 \times \left(1 + \frac{w - 3.66}{3.663}\right) \times \left(\frac{1}{f_{car} + f_{hv} + f_m}\right)$	Meter	veh/hr

3.2.4 GRADE ADJUSTMENT FACTOR

Approach grade adjustment factor is separated into uphill and downhill gradient adjustment factor. The grade adjustment factors for signalised intersections in Malaysia are as shown in equation 3.6 and equation 3.7 for downhill and uphill gradient, respectively. Table 3.7 indicates the tabulation of the grade adjustment factor.

Downhill gradient adjustment factor, $$f_g = 1 - \frac{\%G}{26.34} \qquad (3.6)$$

Uphill gradient adjustment factor, $$f_g = 1 - \frac{\%G}{14.39} \qquad (3.7)$$

Table 3.7 Adjustment Factor for Grade *(f_g)*

Downhill (Grade, %G)	$f_g = 1 - \frac{\% G}{26.34}$
-5.0	1.189
-4.0	1.152
-3.0	1.114
-2.0	1.076
-1.0	1.038

Uphill (Grade, %G)	$f_g = 1 - \frac{\% G}{14.39}$
1.0	0.931
1.5	0.896
2.0	0.861
2.5	0.826
3.0	0.792
3.5	0.757

Note: Applicable for gradient from –5.24% to 3.49%

3.2.5 AREA TYPE ADJUSTMENT FACTOR

The corresponding area type adjustment factor for CBD areas in Malaysia was 0.8454 and non CBD is 1.0000. According to US HCM 2000, CBD or Central Business District can be described if the following condition is satisfied:

- Narrow street right-way
- Frequent parking manoeuvres
- Vehicle blockage
- Taxi and bus activity
- Small radius turns
- Limited use of exclusive turn lanes
- High pedestrian activity
- Dense population
- Mid-block curb cuts

The area type adjustment factor takes into consideration relative inefficiency of signalised intersection located at the CBD as compared to other area.

Table 3.8 Adjustment Factor for Area Type (f_a)

Type of Area	Area type factor (f_a)
CBD	0.8454
NON CBD	1.000

3.2.6 LEFT TURN ADJUSTMENT FACTOR

Table 3.9 shows the adjustment factor for left turn movements. The mode of turning operations is significant to calculate the impact of saturation flow rate. It depends whether the lane is made for exclusive or shared lane, type of signal phasing and the proportion of left turning in shared lane. Table 3.10 simplifies the left turn adjustment factor based on proportion of left turning, P_{LT} ,in lane group in shared lane.

Table 3.9 Adjustment Factor For Left Turn (f_{LT})

Case / Lane type	Left turn adjustment factor (f_{LT})
Exclusive	0.76
Shared	$1.0 - 0.243P_{LT}$

Note : P_{LT} = proportion of left turn in lane group

Table 3.10 Proportion of Left Turn in Shared Lane Group, P_{LT}

P_{LT}	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
f_{LT}	1.000	0.976	0.951	0.927	0.903	0.879	0.854	0.830	0.806	0.781	0.757

P_{LT}	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.00	1.00	1.00	0.99	0.99	0.99	0.99	0.98	0.98	0.98
0.1	0.98	0.97	0.97	0.97	0.97	0.96	0.96	0.96	0.96	0.95
0.2	0.95	0.95	0.95	0.94	0.94	0.94	0.94	0.93	0.93	0.93
0.3	0.93	0.92	0.92	0.92	0.92	0.91	0.91	0.91	0.91	0.91
0.4	0.90	0.90	0.90	0.90	0.89	0.89	0.89	0.89	0.88	0.88
0.5	0.88	0.88	0.87	0.87	0.87	0.87	0.86	0.86	0.86	0.86
0.6	0.85	0.85	0.85	0.85	0.84	0.84	0.84	0.84	0.83	0.83
0.7	0.83	0.83	0.83	0.82	0.82	0.82	0.82	0.81	0.81	0.81
0.8	0.81	0.80	0.80	0.80	0.80	0.79	0.79	0.79	0.79	0.78
0.9	0.78	0.78	0.78	0.77	0.77	0.77	0.77	0.76	0.76	0.76
1.0	0.76	0.75	0.75	0.75	0.75	0.74	0.74	0.74	0.74	0.74

3.2.7 RIGHT TURN ADJUSTMENT FACTOR

Turning factors depend on several parameters. Right turns may be operated in either an exclusive or shared lane; the signal phasing can be either permitted or protected. But in Malaysia, the most common signalised intersections setting are shared lane with protected phasing or exclusive lane with protected signal phasing.

Table 3.11 shows the adjustment factor for right turning at a signalised intersection. Table 3.12 simplifies the right turn adjustment factor based on the proportion of right turning in lane group for shared lane with protected phasing.

Table 3.11 Adjustment Factor for Right Turn (f_{RT})

Case / Lane type	Right turn adjustment factor (f_{RT})
Exclusive	0.84
Shared	$\dfrac{1}{1+0.195P_{RT}}$

Note : P_{RT} = proportion of right turn in lane group

Table 3.12 Proportion of Right Turn in Shared Lane Group, P_{RT}

P_{RT}	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
f_{RT}	1.000	0.981	0.962	0.945	0.928	0.911	0.895	0.880	0.865	0.851	0.837

P_{RT}	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.00	1.00	1.00	0.99	0.99	0.99	0.99	0.99	0.98	0.98
0.1	0.98	0.98	0.98	0.98	0.97	0.97	0.97	0.97	0.97	0.96
0.2	0.96	0.96	0.96	0.96	0.96	0.95	0.95	0.95	0.95	0.95
0.3	0.94	0.94	0.94	0.94	0.94	0.94	0.93	0.93	0.93	0.93
0.4	0.93	0.93	0.92	0.92	0.92	0.92	0.92	0.92	0.91	0.91
0.5	0.91	0.91	0.91	0.91	0.90	0.90	0.90	0.90	0.90	0.90
0.6	0.90	0.89	0.89	0.89	0.89	0.89	0.89	0.88	0.88	0.88
0.7	0.88	0.88	0.88	0.88	0.87	0.87	0.87	0.87	0.87	0.87
0.8	0.87	0.86	0.86	0.86	0.86	0.86	0.86	0.85	0.85	0.85
0.9	0.85	0.85	0.85	0.85	0.85	0.84	0.84	0.84	0.84	0.84
1.0	0.84	0.84	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.82

3.2.8 VEHICLE COMPOSITION CORRECTION FACTOR

Malaysia has a wide range of vehicle composition. And each type of vehicle has its own characteristic. Figure 3.4 to Figure 3.8 shows the typical composition in Malaysia.

Figure 3.4 Various Types of Passenger Cars in Malaysia

Figure 3.5 Various Types of Motorcycles in Malaysia

Figure 3.6 Various Types of Trailers in Malaysia

Figure 3.7 Various Types of Lorries in Malaysia

Figure 3.8 Various Types of Buses in Malaysia

In Malaysia, the common type of motorcycles found on the road is that of small size motorcycles. Observation in the field indicated that motorcycles can traverse through a signalised intersection by three different ways due to its small size. During the red light, motorcycles often weave in and out of traffic stream to get as close as possible to the stop line of signalised intersection and due to the high percentage of motorcycles, most of them will stop beyond the stop line. These motorcycles are categorised as the motorcycles in front of stop line. Apart from that, most of the lane widths at traffic light junctions found in Malaysia is about 3.0 to 4.5 meters and with these lane widths, motorcyclists can travel along side other vehicles. Therefore, the second category consists of motorcycles that travel along side other vehicles (such as cars, lorries, bus, etc.) within the same traffic lane. The third category consists of motorcycles following other vehicle types in a structured discipline. Different travel pattern of motorcycles at signalised intersections might have different impact on saturation flow estimation.

Motorcycles can traverse through a signalised intersection by three different ways as shown in Figure 3.9; i.e. motorcycles in front of stop line, motorcycles following other vehicles or motorcycle besides other vehicle. Motorcycles in front of other vehicle and motorcycles beside other vehicle can be grouped into motorcycles outside flow.

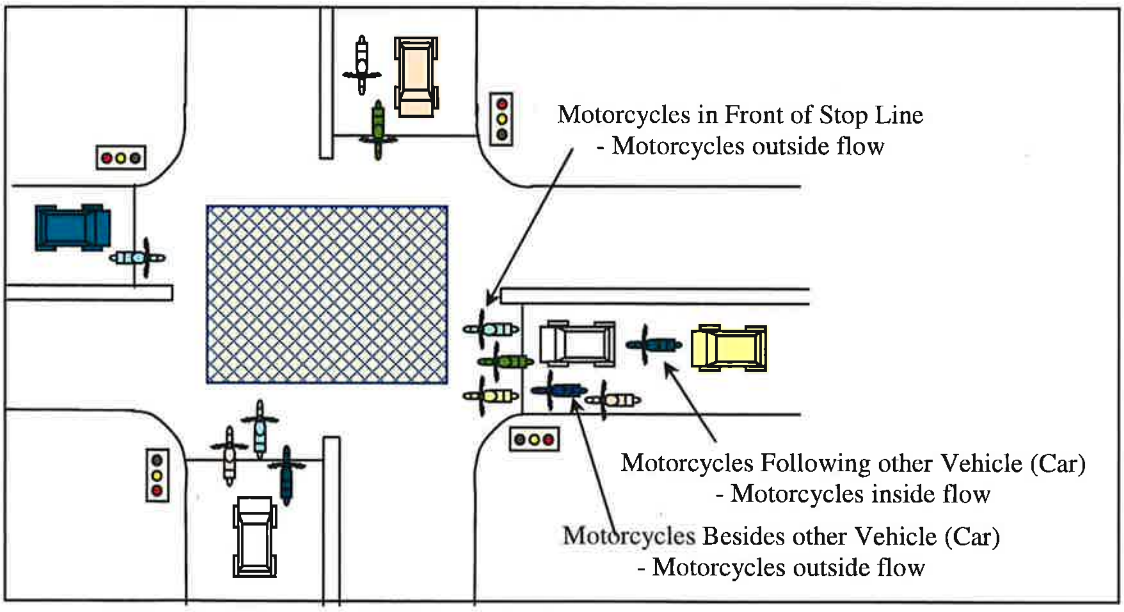


Figure 3.9 Characteristic of Motorcycle at a Signalised Intersection

The difference impact between outside flow and inside flow for motorcycles is as shown in Figure 3.10. Figure 3.10 shows the percentage of motorcycles outside flow and inside flow that are prevalent on Malaysian roads.

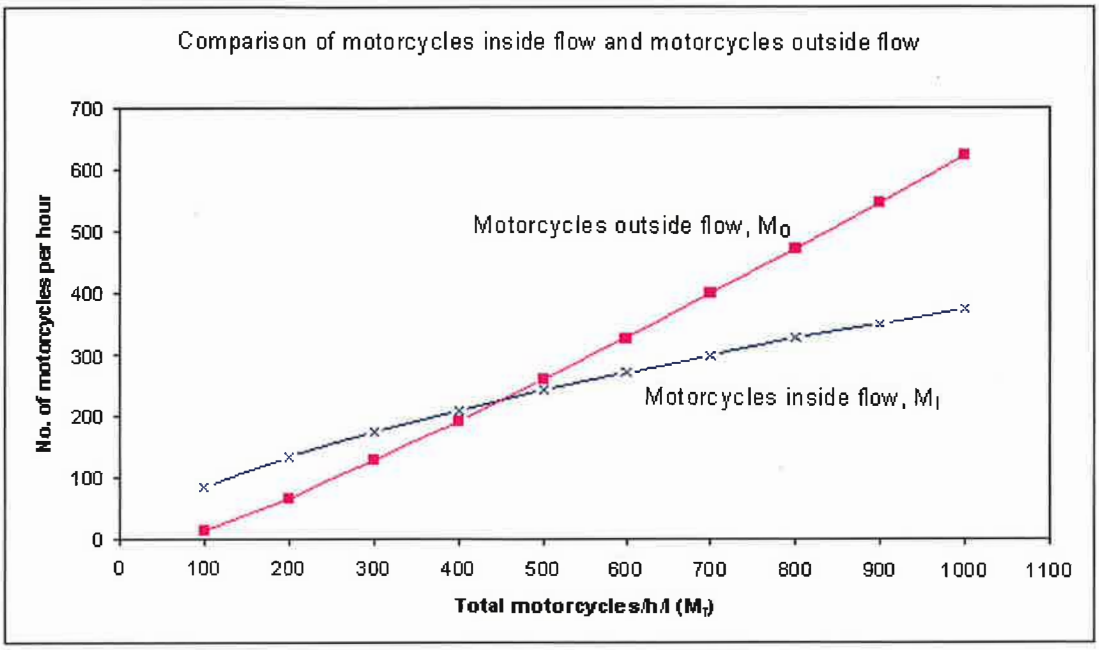


Figure 3.10 Differences between Outside Flow and Inside Flow for Motorcycles

As shown in the Figure 3.10, when total motorcycles is less than 450 motorcycles/h/ln, motorcycle outside flow is lower than motorcycle inside flow. However, as the total number of motorcycle is higher than 450 motorcycles/h/ln, the number of motorcycles outside flow is higher than the number of motorcycle inside flow. The presence of motorcycles inside flow and outside flow affects the value of saturation flow.

Figure 3.11 illustrated the impact of motorcycles on the estimation of saturation flow. Two scenarios are presented; i.e. assuming that motorcycles is not being considered in the estimation of saturation flow and secondly, by assuming that motorcycles are converted to passenger cars. It can be seen from Figure 3.11 that the presence of motorcycles increases the value of saturation flow rates. As the number of motorcycles being reduced, the saturation flow rate is reduced respectively. The phenomena is very significant because it can cause the designer to either over design or under design the signalised intersection.

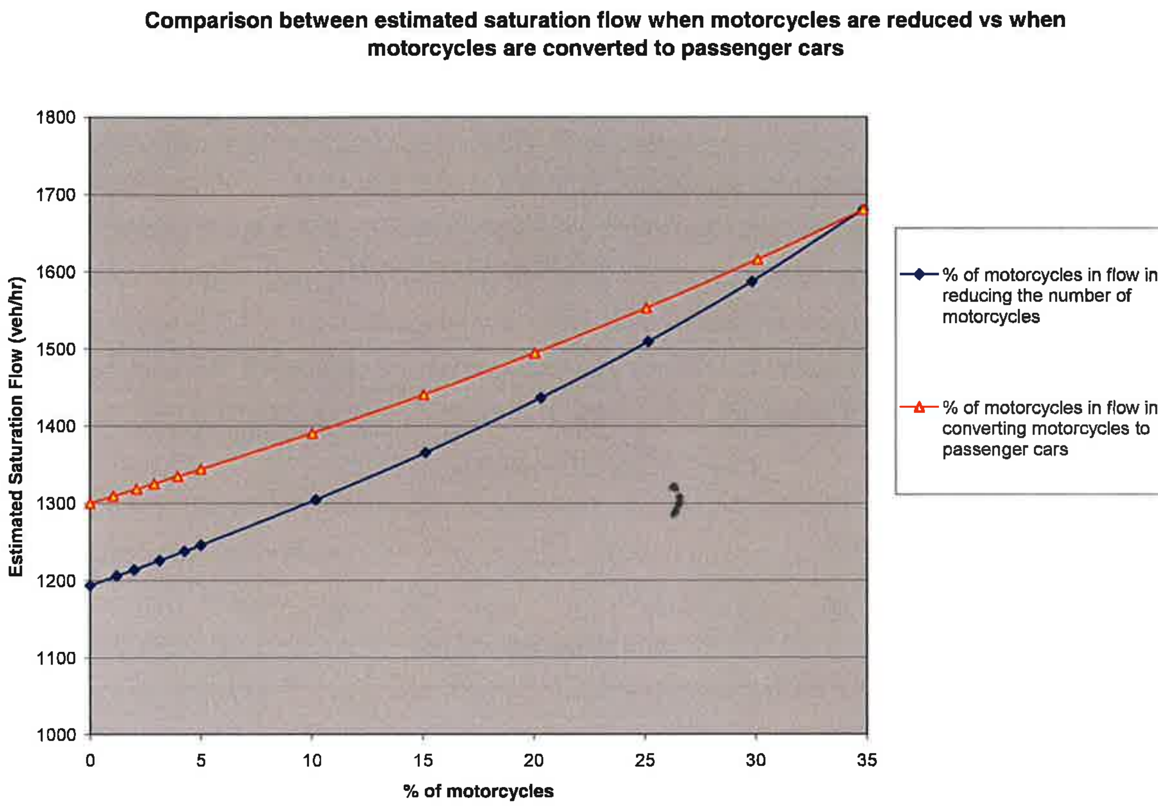


Figure 3.11 Graph to show the influence of motorcycle composition in flow

In Malaysia, the composition of motorcycle in the traffic flow is higher as compared to other countries especially the US. Due to this factor, vehicle composition correction factor is important in the analysis, reflecting the important role of high motorcycle percentage prevalence on Malaysian road. Table 3.13 to Table 3.17 tabulate the values of f_{car}, f_{motor}, $f_{trailer}$, f_{bus}, and f_{lorry}.

Table 3.13 f_{car} based on proportion (%) of cars in flow

q_{car}/Q	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.000	0.010	0.020	0.030	0.040	0.050	0.060	0.070	0.080	0.090
0.1	0.100	0.110	0.120	0.130	0.140	0.150	0.160	0.170	0.180	0.190
0.2	0.200	0.210	0.220	0.230	0.240	0.250	0.260	0.270	0.280	0.290
0.3	0.300	0.310	0.320	0.330	0.340	0.350	0.360	0.370	0.380	0.390
0.4	0.400	0.410	0.420	0.430	0.440	0.450	0.460	0.470	0.480	0.490
0.5	0.500	0.510	0.520	0.530	0.540	0.550	0.560	0.570	0.580	0.590
0.6	0.600	0.610	0.620	0.630	0.640	0.650	0.660	0.670	0.680	0.690
0.7	0.700	0.710	0.720	0.730	0.740	0.750	0.760	0.770	0.780	0.790
0.8	0.800	0.810	0.820	0.830	0.840	0.850	0.860	0.870	0.880	0.890
0.9	0.900	0.910	0.920	0.930	0.940	0.950	0.960	0.970	0.980	0.990

Table 3.14 f_{motor} based on proportion (%) of motorcycles in flow

q_{motor}/Q	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.000	0.002	0.004	0.007	0.009	0.011	0.013	0.015	0.018	0.020
0.1	0.022	0.024	0.026	0.029	0.031	0.033	0.035	0.037	0.040	0.042
0.2	0.044	0.046	0.048	0.051	0.053	0.055	0.057	0.059	0.062	0.064
0.3	0.066	0.068	0.070	0.073	0.075	0.077	0.079	0.081	0.084	0.086
0.4	0.088	0.090	0.092	0.095	0.097	0.099	0.101	0.103	0.106	0.108
0.5	0.110	0.112	0.114	0.117	0.119	0.121	0.123	0.125	0.128	0.130
0.6	0.132	0.134	0.136	0.139	0.141	0.143	0.145	0.147	0.150	0.152
0.7	0.154	0.156	0.158	0.161	0.163	0.165	0.167	0.169	0.172	0.174
0.8	0.176	0.178	0.180	0.183	0.185	0.187	0.189	0.191	0.194	0.196
0.9	0.198	0.200	0.202	0.205	0.207	0.209	0.211	0.213	0.216	0.218

Table 3.15 $f_{trailer}$ based on proportion (%) of trailers in flow

$q_{trailer}/Q$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.000	0.023	0.045	0.068	0.091	0.114	0.136	0.159	0.182	0.020
0.1	0.227	0.250	0.272	0.295	0.318	0.341	0.363	0.386	0.409	0.042
0.2	0.454	0.477	0.499	0.522	0.545	0.568	0.590	0.613	0.636	0.064
0.3	0.681	0.704	0.726	0.749	0.772	0.795	0.817	0.840	0.084	0.086
0.4	0.908	0.931	0.953	0.976	0.999	1.022	1.044	1.067	0.106	0.108
0.5	1.135	1.158	1.180	1.203	1.226	1.249	1.271	1.294	0.128	0.130
0.6	1.362	1.385	1.407	1.430	1.453	1.476	1.498	1.521	0.150	0.152
0.7	1.589	1.612	1.634	1.657	1.680	1.703	1.725	1.748	0.172	0.174
0.8	1.816	1.839	1.861	1.884	1.907	1.930	1.952	1.975	0.194	0.196
0.9	2.043	2.066	2.088	2.111	2.134	2.157	2.179	2.202	0.216	0.218

Table 3.16 f_{lorry} based on proportion (%) of lorries in flow

(q_{lorry}/Q)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.000	0.012	0.024	0.036	0.048	0.060	0.071	0.083	0.095	0.107
0.1	0.119	0.131	0.143	0.155	0.167	0.179	0.190	0.202	0.214	0.226
0.2	0.238	0.250	0.262	0.274	0.286	0.298	0.309	0.321	0.333	0.345
0.3	0.357	0.369	0.381	0.393	0.405	0.417	0.428	0.440	0.452	0.464
0.4	0.476	0.488	0.500	0.512	0.524	0.536	0.547	0.559	0.571	0.583
0.5	0.595	0.607	0.619	0.631	0.643	0.655	0.666	0.678	0.690	0.702
0.6	0.714	0.726	0.738	0.750	0.762	0.774	0.785	0.797	0.809	0.821
0.7	0.833	0.845	0.857	0.869	0.881	0.893	0.904	0.916	0.928	0.940
0.8	0.952	0.964	0.976	0.988	1.000	1.012	1.023	1.035	1.047	1.059
0.9	1.071	1.083	1.095	1.107	1.119	1.131	1.142	1.154	1.166	1.178

Table 3.17 f_{bus} based on proportion (%) of buses in flow

(q_{bus}/Q)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.000	0.021	0.042	0.062	0.083	0.104	0.125	0.146	0.166	0.187
0.1	0.208	0.229	0.250	0.270	0.291	0.312	0.333	0.354	0.374	0.395
0.2	0.416	0.437	0.458	0.478	0.499	0.520	0.541	0.562	0.582	0.603
0.3	0.624	0.645	0.666	0.686	0.707	0.728	0.749	0.770	0.790	0.811
0.4	0.832	0.853	0.874	0.894	0.915	0.936	0.957	0.978	0.998	1.019
0.5	1.040	1.061	1.082	1.102	1.123	1.144	1.165	1.186	1.206	1.227
0.6	1.248	1.269	1.290	1.310	1.331	1.352	1.373	1.394	1.414	1.435
0.7	1.456	1.477	1.498	1.518	1.539	1.560	1.581	1.602	1.622	1.643
0.8	1.664	1.685	1.706	1.726	1.747	1.768	1.789	1.810	1.830	1.851
0.9	1.872	1.893	1.914	1.934	1.955	1.976	1.997	2.018	2.038	2.059

3.2.9 DELAY

Delay in the Levels of Service Module is the average controlled delay. Level of service is directly related to the average controlled delay as shown in Table 3.1. In the LOS Module, average controlled delay is estimated for each of lane group and averaged for approaches and the intersection as a whole.

The average controlled delay per vehicle for a given lane group is given by equation 3.8 which is adapted form the US HCM 2000.

$$d = d_1 PF + d_2 + d_3 \tag{3.8}$$

Where

$$d_1 = 0.5C[1 - g/C]^2 / \{1 - (g/C)[Min(1, X)]\} \tag{3.9}$$

And

$$d_2 = 900T\left\{(X-1) + \left[(X-1)^2 + 8klX/cT\right]^{0.5}\right\} \tag{3.10}$$

Where:

d = controlled delay (sec/veh)

d_1 = uniform controlled delay (sec/veh)

d_2 = incremental delay (sec/veh)

d_3 = initial queue delay

PF = uniform delay progression adjustment factor which accounts for effect of signal progression

X = v/c ratio for lane group

C = cycle length (sec)

c = capacity of lane group (vph)

g = effective green time for lane group (sec)

T = duration of analysis period

k = incremental delay factor that is dependent on controller settings

I = upstream filtering / metering adjustment factor

3.2.9.1 Uniform Delay

Equation 3.9 gives formulation to calculate delay, assuming that the flow is stable and perfectly uniform arrivals.

3.2.9.2 Incremental Delay

Equation 3.10 describes the delay based on non-uniform arrivals and individual cycle failures. The equation can only be used for values of X less than 1.0 but may be used with some caution for values of X not more than 1.2, usual upper limit of delay model, or 1/PHF. In cases where X ≥ 1.0, the delay estimate applies to all vehicles arriving during the first 15-minutes period during which oversaturation usually do happen. The equation does not take into consideration for the cumulative effect of queues remaining from a previous 15-minutes period. Equation 3.10 will be invalid if the value of X exceeds 1/PHF because the hourly volume exceeds the hourly capacity.

3.2.9.3 Progression Adjustment Factor, PF

The progression adjustment factor (PF) explains the impact of control type and signal progression on delay. Table 3.18 shows the appropriate value of PF for existing control mode that can be found in Malaysia.

Adjustment factor for Controller Type or Controller Type Adjustment Factor, CF is important to calculate the controlled delay. Table 3.18 shows the delay adjustment factor for different types of controller.

Table 3.18 Delay Adjustment Factor

Controller-type Adjustment Factor (CF)		
Controller Type	Noncoordinated intersections	Coordinated Intersections
Pretimed (no traffic-actuated lane groups)	1.0	PF as computed below
Semiactuated		
Traffic-actuated lane groups	0.85	1.0
Nonactuated lane groups	0.85	PF as computed below
Fully actuated (all lane groups actuated)	0.85	Treat as semiactuated

Source US HCM 1994

Adjustment factor for Quality or Progression Adjustment Factor, PF is applies to all coordinated lane groups, including pre-timed control and non-actuated lane groups in semi actuated control systems. A good signal progression will give a high proportion of vehicles arriving on the green, whereas poor signal indicates a low percentage of vehicles arriving on the green. The value of PF can be determined by equation 3.11.

Progression Adjustment Factor, $$PF = \frac{(1-P)f_P}{1-(g/C)} \qquad (3.11)$$

Where:

P = proportion of vehicles arriving on the green

g/C = proportion of available green time

f_P = supplemental adjustment factor for when the platoon arrives during green

The default values for f_P are 0.93 for Arrival Type 2, 1.15 for Arrival Type 4, and 1.0 for all other Arrival Type. The default values for f_P, g/C ratio and Arrival Type factor are shown in Table 3.19. Also shown are R_P and Incremental Delay calibration term, m.

Table 3.19 Progression Adjustment Factor

Green Ratio (g/C)	Arrival Type (AT)					
	AT-1	AT-2	AT-3	AT-4	AT-5	AT-6
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.714	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000
Defaul, f_p	1.00	0.93	1.00	1.15	1.00	1.00
Defaul, R_p	0.333	0.667	1	1.333	1.667	2
Incremental delay calibration term, m	8	12	16	12	8	4

Note: 1- Tabulation based on default values of f_p and R_p
2- P = R_p g/C may not exceed 1.0
3- PF may not exceed 1.0 for AT-3 through AT-6

The value of P can be measured at the site or estimated from the arrival type. Arrival type can be divided into 6 types as adapted from the HCM 2000.

1. **Arrival Type 1 (AT-1)**

   Dense platoon, containing over 80 percent of the lane group volume, arriving at the start of the red phase. This AT is representative of network links that may experience very poor progression quality as a result of conditions such as overall network signal optimization.

2. **Arrival Type 2 (AT-2)**

   Moderately dense platoon arriving in the middle of the red phase or dispersed platoon, containing 40 to 80 percent of the lane group volume, arriving throughout the red phase. This AT is representative of unfavorable progression on two-way arterials.

3. **Arrival Type 3 (AT-3)**

   Random arrivals in which the main platoon contains less than 40 percent of the lane group volume. This AT is representative of operations at isolated and non interconnected signalized intersection characterized by highly dispersed platoons. It may also be used to represent coordinated operation in which the benefits of progression are minimal.

4. **Arrival Type 4 (AT-4)**
   Moderately dense platoon arriving in the middle of the green phase or dispersed platoon, containing 40 to 80 percent of the lane group volume, arriving throughout the green phase. This AT is representative of favourable progression quality on a two-way arterial.

5. **Arrival Type 5 (AT-5)**
   Dense to moderately dense platoon, containing over 80 percent of the lane group volume, arriving at the start of the green phase. This AT is representative of highly favourable progression quality, which may occur on routes with low to moderate side-street entries and which receive high-priority treatment in the signal-timing plan design.

6. **Arrival Type 6 (AT-6)**
   This arrival type is reserved for exceptional progression quality on routes with near-ideal progression characteristics. It is representative of very dense platoons progressing over a number of closely spaced intersections with minimal or negligible side-street entries.

The detailed knowledge of offsets, travel speeds and intersection signalization are required in order to use adjustment factor for progression. Arrival Type 4 usually is as the base condition for coordinated lane groups when planning for future situations involving coordination. For all uncoordinated lane groups, Arrival Type 3 should be used in the calculation.

3.2.9.4 Incremental Delay Calibration term, m

The incremental delay calibration term correspond to the effect of arrival type and degree of platooning on the incremental delay term. The values of m are given in Table 3.19 as a function of the arrival type. When a lane group contain movements that have different levels of coordination, a flow-weighted average of m should be used in determining the incremental delay.

3.2.9.5 Incremental Delay Calibration factor, k

The calibration term (k) is included in equation 3.10 to incorporate the effect of controller type on delay. This formula was originally taken from US HCM 2000, which the calibration term, k, did not exist in previous version of US HCM. For k = 0.50, it is used for pretimed signal. The value is based on a queuing process with random arrivals and uniform service time equivalent to the lane group capacity. In contrast, actuated controllers have the ability to modify the green time to traffic demand. As a result, it will reduce the value of incremental delay.

3.2.9.6 Initial Queue Delay

When a residual queue from a previous time period causes an initial queue to occur at the start of the analysis period (T), additional delay is experienced by vehicles arriving in the period since the initial queue must first clear the intersection (TRB, 2000). The estimation of initial queue delay can be done by using equation 3.12.

$$d_3 = \frac{1800 Q_b (1+u) t}{cT} \qquad (3.12)$$

Where

Q_b = initial queue at the start of period T (veh)

c = adjusted lane group capacity (veh/hr)

T = analysis period (hr)

t = duration of unmet demand in T (hr)

u = delay parameter

3.2.9.7 Approach Delay

The delay for an approach is computed using equation 3.13. It is desirable to aggregate these values to provide delay for an intersection approach and for the intersection as a whole. This aggregation is done by computing weighted averages, where the lane group delays are weighted by the adjusted flows in the lane groups. Average delay per vehicle for the intersection is computed using equation 3.14.

$$d_A = \frac{\sum d_i v_i}{\sum v_i} \qquad (3.13)$$

Where

d_A = delay for approach A (sec/veh)

d_i = delay for lane group i (on approach A) (sec/veh)

v_i = adjusted flow for lane group i (veh/hr

$$d_I = \frac{\sum d_A v_A}{\sum v_A} \tag{3.14}$$

Where

d_I = average delay per vehicle for the intersection I (sec/veh)

v_A = adjusted flow for approach A (veh/hr)

3.2.10 CAPACITY

The capacity of each lane group is calculated using equation 3.15.

$$c_i = s_i (g_i / C) \tag{3.15}$$

Where

c_i = capacity of lane group i (veh/hr)

S_i = saturation flow rate for lane group i (veh/hr)

g_i/c = effective green ratio for lane group, i

3.2.11 DETERMINING LEVEL OF SERVICE (LOS)

LOS at a signalised intersection is based on the average controlled delay per vehicle. When delays for each lane group and approach, including the intersection as a whole have been estimated, Table 3.1 is consulted and the appropriate LOS is determined.

Any v/c ratio greater than 1.0 is an indication of actual or potential breakdown (TRB, 2000). According to US HCM 2000, in such cases, it is advisable to make multi-period analysis. When intersection as a whole v/c ratio is less than 1.0 but some critical lane groups have v/c ratios greater than 1.0, it indicates that the green time is generally not appropriately apportioned. So, analyst should attempt to retime the existing phasing in order to overcome such cases.

Under design intersection occurs when a critical v/c ratio is greater than 1.0. Analyst should look into the geometric design of the intersection, the timing and phasing of the traffic signal in order to make improvement. Note that such improvement must be notified to appropriate local authorities.

LOS is a measure of the delay incurred by motorists at a signalised intersection (TRB, 2000). Delay could be higher even the v/c ratios are low. This is due to poor progression or incompatible cycle length or both. Such event will create an intersection with high delays without having a capacity problem. This is usually interpreted as an over design intersection. When the v/c ratio approaches or exceeds 1.0, it is possible for the over design intersection to have acceptable delays. For such cases, the analysis should be consider the results of both the capacity analysis and the LOS analysis to obtain the complete picture of the existing signalised intersection.

3.2.12 SUMMARY

These are the new findings based on Malaysian traffic condition and have been incorporated in this Highway Capacity Manual.

Parameters		MALAYSIA HIGHWAY CAPACITY MANUAL
Pce values	Passenger car, e_{car}	1.00
	Motorcycle, e_{motor}	0.22
	Lorry, e_{lorry}	1.19
	Trailer, $e_{trailer}$	2.27
	Bus, e_{bus}	2.08
Ideal saturation flow rate, S_0		S_0 = 1,930 pcu/hr
Area type adjustment factor, f_a		$f_a = 0.8454$
Lane width adjustment factor, f_w		$f_w = 1 + \frac{w - 3.66}{3.663}$
Gradient adjustment factor, f_g	Downhill	$f_g = 1 - \frac{\%G}{26.34}$
	Uphill	$f_g = 1 - \frac{\%G}{14.39}$
Right-turn adjustment factor, f_{RT}	Shared-lanes	$f_{RT} = \frac{1}{1 + 0.195 P_{RT}}$
	Exclusive	0.84
Left-turn adjustment factor, f_{LT}	Shared-lanes	$f_{LT} = 1.0 - 0.243 P_{LT}$
	Exclusive	0.76

3.3 APPLICATION

There are four important components in analyzing signalised intersection; i.e. flow rates, signalisation, geometric characteristic and delay or LOS. Each having a target output, this method can be used for each of four operational and design analysis types, through the remaining parameters are known or been assumed for use as inputs.

Operational (LOS)
Determine LOS when details of intersection flows, signalisation and geometric characteristics are known.

Design (v_p)
Determine acceptable service flow rates for selected LOS when the details of signalisation and geometrics are known.

Design (Sig)
Determine signal timing when the desired LOS, flows and geometric characteristics are known.

Design (Geom)
Determine basic geometrics, i.e. number and allocation of lanes; when the desired LOS and details of flows and signalisation are known.

Planning analysis needed only of approximation of input data. However, in operational analysis the analysis being produced is not much diferent from the design phase and analysis. For planning purposes, the data needed are the traffic volumes and number of lanes for each movement together with a minimal description of the signal design and related operating parameters.

Operational analysis is divided into five modules, i.e. input, volume adjustment, saturation flow rate, capacity analysis and LOS. The computations for each module are conducted on the appropriate worksheet. For more complex computations, some worksheets are provided with supplementary worksheets. The flow of this module operation is described in Figure 3.3 at the early part of this chapter.

3.3.1 INPUT PARAMETERS

Based on Input Module Worksheet, the input parameters needed to perform the calculations are geometric characteristics, volume and signalisation information. Most of these data are obtained from site studies, especially if the existing intersection is under study. However, for the purpose of future consideration or planning purposes, traffic data will be forecasted. Therefore geometric characteristic and signal designs will be based on existing conditions or will be proposed. The input worksheet is based from the US HCM 2000, and is shown in Figure 3.12.

The worksheet consists of general information about the site, which are Intersection Geometry, Volume and Timing Input and Signal Phasing Plan. The lane geometric characteristics should be shown within the intersection diagram. It should include:

- Number of lanes
- Lane widths
- Grade
- Traffic movements using each lane – must be shown by arrows
- Existence and location of curb parking lanes (if available)
- Existence and location of bus stops (if available)
- Existence and length of storage bays

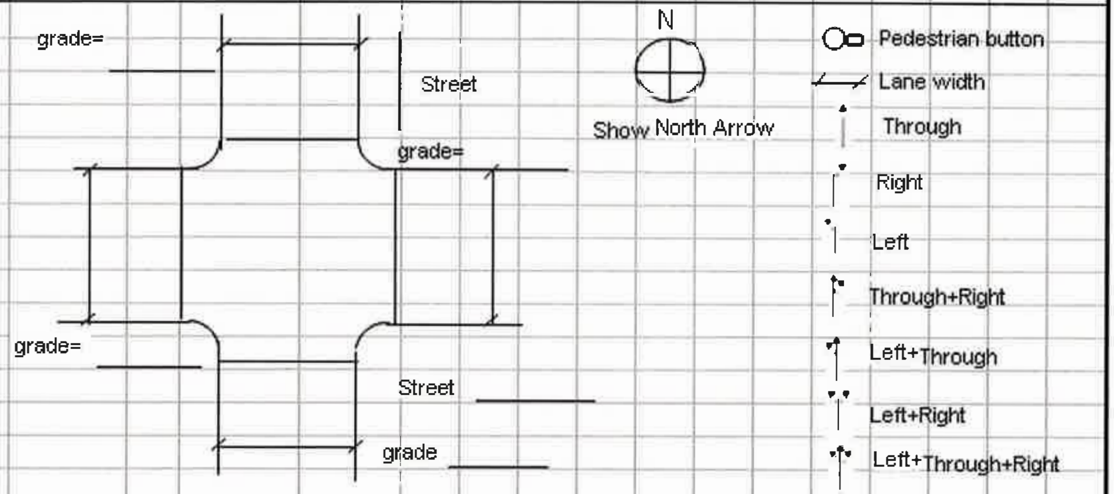
For design purposes, when a geometric condition is not available, a design based on state or local practice should be proposed.

The next portion of the worksheet consists of a table containing volume and timing data for each approach. Hourly volume or a 15-minutes flow rate and volume related parameters are entered into the table. Separate entries are required for each approach and for each movement if applicable.

INPUT WORKSHEET

General Information		Site Information	
Analyst		Intersection	
Agency or Company		Area Type	❑ CBD ❑ Other
Date Performed		Jurisdiction	
Analysis Time Period		Analysis Year	

Intersection Geometry



Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]
Volume, (V(veh/h)												
% heavy vehicles, %HV												
Peak-hour factor, PHF												
Pretimed (P) or actuated (A)												
Start-up lost time, l_1(s)												
Extension of effective green time, e(s)												
Arrival Type, AT												
Parking (Y or N)												
Parking maneuvers, N_m (maneuvers/h)												
Bus stopping, N_B(buses/h)												

Signal Phasing Plan

	1	2	3	4	5	6	7	8
DIAGRAM								
Timing	G= 45.0	G= 30.0	G= 22.0	G= 45.0	G=	G=	G=	G=
	Y= 5.0	Y= 5.0	Y= 5.0	Y= 5.0	Y=	Y=	Y=	Y=

Protected turns — Permitted turns — Pedestrian — Cycle length, C=________ s

Figure 3.12 Input Worksheet

For percent heavy vehicles and peak-hour factor (PHF), an average value for an entire approach can be used for all movements. For arrival type, either the value for P or the description of type (AT-1 to AT-6) is entered. Each approach movement is identified either pretimed (P) or actuated (A). Enter the start-up lost time (l_1) and extension time (e) for each movement. If pedestrians and bicycles exist, enter the appropriate value.

Default values can be used if the data is not available or forecasting cannot be done adequately. Default value can be found in the manual pertaining to the subject.

The next table is actually a space to draw diagram of signal phasing. The table consist of eight boxes. Each box is used to show a single phase or sub phase during which the allowable movement remain constant. The actual green time (G) and the actual amber-plus-all-red time (Y) are shown. However, in some cases the signal timing and phasing will not be known. The signal timing and phasing manipulate the determination of lane groups.

3.3.2 VOLUME ADJUSTMENT AND SATURATION FLOW RATE

The next analysis module is concerning on adjustment of hourly movement volumes to flow rates for a peak 15-minutes period within the hour and on establishment of lane groups. The worksheet is shown in Figure 3.13, containing computation of saturation flow rate, making volume adjustments and allocations of lane groups.

The hourly volume (V) for each approach and movement in this worksheet is taken from the Input Worksheet, for the calculation of flow rate (v_p). Lane groups are established and associated flow rates and turn proportion are noted. For saturation flow rate, adjustment factors are identified and an adjusted saturation flow is computed for each lane group.

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET												
General Information												
Project Description ______												
Volume Adjustment												
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V(veh/h)												
Peak hour factor, PHF												
Adjusted flow rate, v_p=V/PHF(veh/h)												
Lane Group												
Adjusted flow rate in lane group, v (veh/h)												
Proportion[1] of LT or RT(P_{LT} or P_{RT})												
Saturation Flow Rate												
Base saturation flow, s_o(pc/h/ln)												
Number of lanes, N												
Lane width adjustment factor, f_w												
Vehicle composition adjustment factor, f_c												
Grade adjustment factor, f_g												
Area type adjustment factor, f_a												
Left-turn adjustment factor, f_{LT}												
Right-turn adjustment factor, f_{RT}												
Adjusted saturation flow, s(veh/h) $s=s_o N f_w f_g f_a f_{LT} f_{RT} (1/f_c)$												
Notes												
1. P_{LT} = 1.000 for exclusive left-turn lanes, and P_{RT}=1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group												

Figure 3.13 Volume Adjustment and Saturation Flow Rate Worksheet

3.3.3 CAPACITY ANALYSIS

Figure 3.14 shows the Capacity and LOS Worksheet. The results from computational of input worksheet, volume adjustment and saturation flow rate modules are combined to perform the calculation of the capacity, v/c ratio of each lane group, delay, LOS for each lane group and approach and certainly the whole intersection.

The first portion of the worksheet is to compute capacities. Phase type is included to accommodate right turns that have both protected and permitted phases. The different type of phase must be separated by the column entries in this worksheet. In the lower part of the worksheet, the sum of certain quantities, such as lane group capacity must be computed as the sum of the different phases. Primary phase entries should be noted as P in this row, while secondary phase should be noted as S.

The flow rate and the adjusted flow rate for each lane group is obtained from the Volume Adjustment and Saturation Flow Rate Worksheet and entered in this worksheet. The flow ratio for each lane group is computed as v/c and entered in columns representing primary and secondary phases.

The next stage is to compute the movement lost time for all phases. By using the start-up lost time (l_1), extension time (e) and amber-plus-all-red time (Y), the movement lost time could be determined. Effective green times are calculated using actual green time (G) and movement lost time (t_L). Then the g/C ratio for each lane group, the effective green time divided by the cycle length, is computed. The result is used to compute lane group capacity. Effective green time can be taken as equal to the actual green time plus the change and clearance interval minus the lost time for the movement.

The capacity is calculated using equation 3.4. Then the v/c ratio for each lane is computed. At this point, critical lane groups and lost time per cycle can be identified. A critical lane group is defined as the lane group with the highest flow ratio in each phase or set of phases (TRB,2000). If an overlapping phase occurred, all possible combinations of critical lane groups must be examined for the combination leading to the highest total of flow ratios in the worksheet, critical lane groups are identified with checkmark. The flow ratios for critical lane groups are summed. Lastly in this portion, the v/c ratio, X_c, which indicates the degree of saturation is computed.

CAPACITY AND LOS WORKSHEET

General Information												
Project Description ____________________												
Capacity Analysis												
Phase number												
Phase type												
Lane group												
Adjusted flow rate, v(veh/h)												
Saturation flow rate, s(veh/h)												
Lost time, t_L(s), $t_L = I_1 + Y - e$												
Effective green time, g(s), $g=G+Y-t_L$												
Green ratio, g/C												
Lane group capacity,[1] c=s(g/C),(veh/h)												
v/c ratio, X												
Flow ratio, v/s												
Critical lane group/phase (√)												
Sum of flow ratios for critical lane groups, Y_c												
$Y_c = \sum$ (critical lane group, v/s)												
Total lost time per cycle, L(s)												
Critical flow rate to capacity ratio, X_c												
$X_c = (Y_c)(C)/(C-L)$												
Lane Group Capacity, Control Delay, and LOS Determination												
		EB			WB			NB			SB	
Lane group												
Ajusted flow rate,[2] v(veh/h)												
Lane group capacity,[2] c(veh/h)												
v/c ratio,[2] X = v/c												
Total green ratio,[2] g/C												
Uniform delay, $d_1 = \frac{0.50C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)												
Incremental delay calibration,[3] k												
Incremental delay,[4] d_2 $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8klX}{cT}}\right]$ (s/veh)												
Initial queue delay, d_3 (s/veh)												
Uniform delay, d_1(s/veh)												
Progression adjustment factor, PF												
Delay, $d=d_1(PF)+d_2+d_3$ (s/veh)												
LOS by lane group (Table 3.1)												
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v_A}$ (s/veh)												
LOS by approach (Table 3.1)												
Approach flow rate, v_A (veh/h)												
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)					Intersection LOS (Table 3.1)							

Notes

1. For permitted left turns, the minimum capacity is $(1 + P_L)(3600/C)$
2. Primary and secondary phase parameters are summed to obtain lane group parameters
3. For pretimed or nonactuated signals, k=0.50.
4. T = analysis duration (h); typically T = 15 minutes.
   I = upstream filtering metering adjustment factor; I = 1 for isolated intersections.

Figure 3.14 Capacity and LOS Worksheet

3.3.4 DELAY AND LEVEL OF SERVICE

In Capacity and LOS worksheet shown in Figure 3.14, the last portion of the worksheet is the calculation of Lane Group Capacity, Controlled Delay and the determination of LOS. Using the result from the volume adjustment, saturation flow rate and capacity analysis, the average controlled delay per vehicle in each lane group can be determined and resulting in the determination of LOS. In this section, the lane group capacities and flow rate are the sum of the phases from the capacity analysis section.

Delay can be estimated using equation 3.8, 3.9, 3.10, 3.12 and 3.13. The uniform delay is multiplied by the progression adjustment factor (PF) to account for the impact progression (TRB, 2000). The value of PF can be found from Table 3.19.

Incremental delay is the delay over and above uniform delay due to random arrivals rather than uniform arrivals and also because of the fail cycles (TRB, 2000). The incremental delay calibration term can be obtained from Table 3.19.

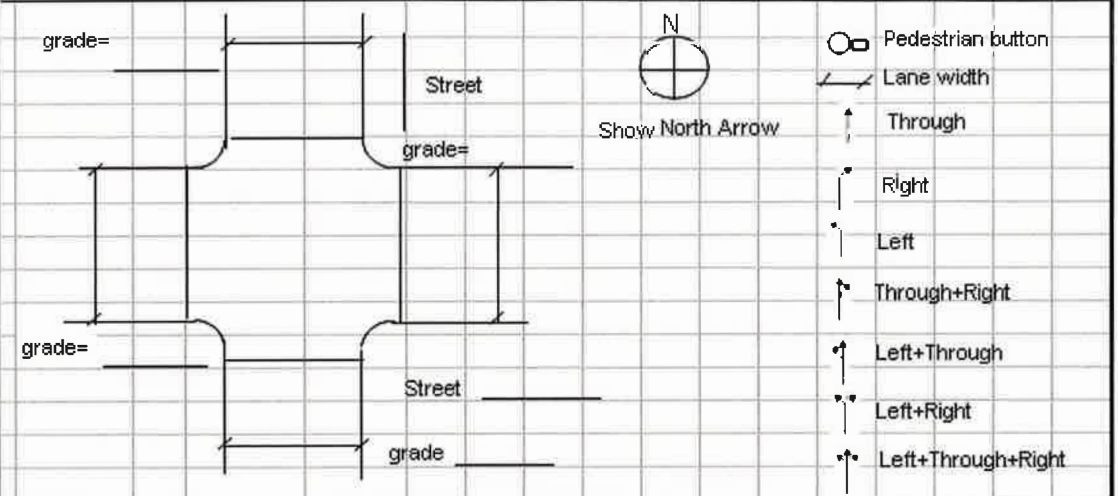
Delay and LOS is calculated using equation 3.8 and accordance to Table 3.1. The result is entered on this worksheet.

3.4 WORKSHEETS

INPUT WORKSHEET

General Information		Site Information	
Analyst	______	Intersection	______
Agency or Company	______	Area Type	☐ CBD ☐ Other
Date Performed	______	Jurisdiction	______
Analysis Time Period	______	Analysis Year	______

Intersection Geometry



Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, (V(veh/h)												
Peak-hour factor, PHF												
Pretimed (P) or actuated (A)												
Start-up lost time, l_1(s)												
Extension of effective green time, e(s)												
Arrival Type, AT												
Parking (Y or N)												
Parking maneuvers, N_m (maneuvers/h)												
Bus stopping, N_B(buses/h)												

Signal Phasing Plan

	1	2	3	4	5	6	7	8
DIAGRAM								
Timing	G= Y=	G= Y=	G= Y=	G= Y=	G= Y=	G= Y=	G= Y=	G= Y=

Protected turns — Permitted turns — Pedestrian — Cycle length, C=________ s

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

General Information

Project Description ____

Volume Adjustment												
		EB			WB			NB			SB	
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V(veh/h)												
Lane Group												
Total Cars												
fcar												
Total Motors												
fmotor												
Total Trailers												
ftrailer												
Total Lorries												
florry												
Total Buses												
fbus												
Adjusted flow rate in lane group, v (veh/h)												
Proportion[1] of LT or RT(P_{LT} or P_{RT})												
Saturation Flow Rate												
Base saturation flow, s_0(pc/h/ln)												
Number of lanes, N												
Lane width adjustment factor, f_w												
Vehicle composition adjustment factor, f_c												
Grade adjustment factor, f_g												
Area type adjustment factor, f_a												
Left-turn adjustment factor, f_{LT}												
Right-turn adjustment factor, f_{RT}												
Adjusted saturation flow, s(veh/h), s=s0, $s = s_0 \, N \, f_w f_g f_a f_{LT} f_{RT} (1/f_c)$												

Notes

1. P_{LT} = 1.000 for exclusive left-turn lanes, and P_{RT}=1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group

CAPACITY AND LOS WORKSHEET

General Information												
Project Description ____________												
Capacity Analysis												
Phase number												
Phase type												
Lane group												
Adjusted flow rate, v(veh/h)												
Saturation flow rate, s(veh/h)												
Lost time, t_L(s), $t_L = l_1 + Y - e$												
Effective green time, g(s), $g = G + Y - t_L$												
Green ratio, g/C												
Lane group capacity,[1] c=s(g/C),(veh/h)												
v/c ratio, X												
Flow ratio, v/s												
Critical lane group/phase (√)												
Sum of flow ratios for critical lane groups, Y_c												
$Y_c = \sum$ (critical lane group, v/s)												
Total lost time per cycle, L(s)												
Critical flow rate to capacity ratio, X_c												
$X_c = (Y_c)(C)/(C-L)$												
Lane Group Capacity, Control Delay, and LOS Determination												
		EB			WB			NB			SB	
Lane group												
Ajusted flow rate,[2] v(veh/h)												
Lane group capacity,[2] c(veh/h)												
v/c ratio,[2] X = v/c												
Total green ratio,[2] g/C												
Uniform delay, $d_1 = \frac{0.50C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)												
Incremental delay calibration,[3] k												
Incremental delay,[4] d_2 $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right]$ (s/veh)												
Initial queue delay, d_3 (s/veh)												
Uniform delay, d_1(s/veh)												
Progression adjustment factor, PF												
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)												
LOS by lane group (Table 3.1)												
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v_A}$ (s/veh)												
LOS by approach (Table 3.1)												
Approach flow rate, v_A (veh/h)												
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)				Intersection LOS (Table 3.1)								
Notes												
1. For permitted left turns, the minimum capacity is $(1 + P_L)(3600/C)$												
2. Primary and secondary phase parameters are summed to obtain lane group parameters												
3. For pretimed or nonactuated signals, k=0.50.												
4. T = analysis duration (h); typically T = 15 minutes.												
I = upstream filtering metering adjustment factor; I = 1 for isolated intersections.												

INITIAL QUEUE DELAY WORKSHEET

General Information

Project Description ________________________________

Input Parameters

Period (i) ______________

Duration, T ______________ h

Cycle length, C ______________ s

	EB	WB	NB	SB
Lane group				
Initial queue Q_b (veh)				
Green ratio, g/C				
v/c ratio, X				
X = v/c				
Adjusted lane group capacity, c (veh/h)				
Duration of unmet demand in T (h) $t = \min\left\{T, \frac{Q_b}{c[1-\min(1,X)]}\right\}$				
Case				
Case I and II (Q_b = 0)				
Initial queue delay, d_3 = 0, and uniform delay, d1 is as shown on Capacity and LOS Worksheet				
Case III (Qb>0) (X<=1.0) (t<T)				
Initial queue delay, d3(s) $d_3 = \frac{1800\,Q_b t}{cT}$				
Uniform delay, d_1 (s) $d_1 = 0.50C(1-g/C)\frac{t}{T} + \frac{0.5C(1-g/C)^2}{1-g/C\min[1,X]}\left[\frac{[T-t]}{T}\right] * PF$				
Case IV (Qb>0) (X<=1.0) (t=T)				
Delay parameter, u $u = 1 - \frac{cT}{Q_b}[1-\min(1,X)]$				
Initial queue delay, d_3(s) $d_3 = \frac{1800Q_b(1+u)}{c}$				
Uniform delay, d_1 (s) $d_1 = 0.50C\,(1 - g/C)$				
Case V (Qb > 0) (X > 1.0) (t = T)				
Initial queue delay, d_3 (s) $d_3 = \frac{3600Q_b}{c}$				
Uniform delay, d_1 (s) $d_1 = 0.50C\,(1 - g/C)$				

BACK-OF-QUEUE WORKSHEET												
General Information												
Project Description ____________												
Average Back of Queue												
		EB			WB			NB			SB	
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Lane group												
Initial queue per lane at the start of analysis period, Q_{bL}												
Flow rate per lane, v_L (veh/h)												
Saturation flow rate per lane, s_L (veh/h)												
Capacity per lane, c_L (veh/h)												
Flow ratio, v_L/s_L												
v/c ratio, $X_L = v_L/c_L$												
Effective green time, g(s)												
Green ratio, g/C												
Upstream filtering factor, I												
Proportion of vehicles arriving on green, P												
Platoon ratio, R_p $R_p = \frac{P}{g/C}$												
Effects of progession adjustment factor, PF_2 $PF_2 = \frac{\left(1-R_p\frac{g}{C}\right)\left(1-\frac{v_L}{s_L}\right)}{\left(1-\frac{g}{C}\right)\left[1-R_p\left(\frac{v_L}{s_L}\right)\right]}$												
First-term queued vehicles, Q_1 (veh) $Q_1 = PF_2 \frac{\frac{v_L C}{3600}\left(1-\frac{g}{C}\right)}{\left[1-\min\left(1.0, X_L\left(\frac{g}{C}\right)\right)\right]}$												
Second-term adjustment factor, k_B $k_B = 0.12\, I\left(\frac{s_L g}{3600}\right)^{0.7}$ pretimed signals $k_B = 0.10\, I\left(\frac{s_L g}{3600}\right)^{0.6}$ actuated signals												
Second-term queued vehicles, Q_2 $Q_2 = 0.25 c_L T\left[(x_L-1)+\sqrt{(xL-1)^2+\frac{8k_B X_L}{c_L T}+\frac{16 k_B Q_{bL}}{(c_L T)^2}}\right]$												
Average number of queued vehicles, Q $Q = Q_1 + Q_2$												
Percentile Back of Queue	☐ 70%		☐ 85%		☐ 90%			☐ 95%		☐ 98%		
Percentile back-of-queue factor,[1] $f_B\%$												
Percentile back-of-queue, Q% (veh), $Q\% = Qf_{B\%}$												
Queue Storage Ratio												
Average queue spacing, L_h (m)												
Average queue storage, L_a (m)												
Average queue storage ratio, $R_Q = \frac{L_h Q}{L_a}$												
Percentile queue storage ratio, $R_{Q\%} = \frac{L_h Q_\%}{L_a}$												
Notes												
1 $f_{B\%} = p_1 + p_2 e^{\left(\frac{-Q}{p_1}\right)}$												

INTERSECTION CONTROL DELAY WORKSHEET

General Information

Analyst	________	Intersection	________
Agency or Company	________	Area Type	❑ CBD ❑ Other
Date Performed	________	Jurisdiction	________
Analysis Time Period	________	Analysis Year	________

Input Initial Parameters

Number of lanes, N	________	Total vehicles arriving, V_{tot}	________
Free-flow speed, FFS (km/h)	________	Stopped-vehicle count, V_{stop}	________
Survey count interval, Is(s)	________	Cycle length, C(s)	________

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue — Count Interval 1	2	3	4	5	6	7	8	9	10
Total											

Computations

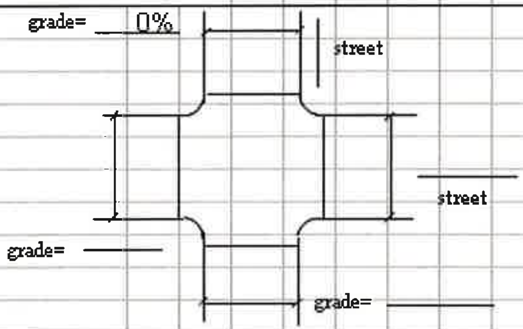
Total vehicles in queue, $\sum V_{iq} =$	________	Number of cycle surveyed, $N_c =$	________
Time-in-queue per vehicle, $d_{vq} = \left(I_s \times \frac{\sum V_{iq}}{V_{tot}}\right) \times 0.9$	________	Fraction of vehicles stopping $\frac{V_{stop}}{V_{tot}} =$	________
No. of vehicles stopping per lane each cycle $= \frac{V_{stop}}{(N_c x N)}$	________	Accel/Decel correction delay, d_{ad}=FVS*CF	________ s
Accel/Decel correction factor, CF	________	Control delay/vehicle, $d = d_{vq} + d_{ad}$	________ s

FIELD SATURATION FLOW RATE STUDY WORKSHEET

General Information

Analyst	________________	Intersection	________________
Agency or Company	________________	Area Type	❑ CBD ❑ Other
Date Performed	________________	Jurisdiction	________________
Analysis Time Period	________________	Analysis Year	________________

Lane Movement Input



Movements Allowed
❑ Through
❑ Right turn
❑ Left turn

Identify all lane movements and the lane studied

Input Field Measurement

Veh. in	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
queue	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		
18																		
19																		
20																		
End of saturation																		
End of green																		
No. veh.>20																		
No.veh. on yellow																		

Glossary and Notes

HV = Heavy vehicles (vehicles with more than 4 tires on pavement)

T = Turning vehicles (L=Left, R=Right)

Pedestrians and buses that block vehicles should be noted wih the time that they block traffic, for example,

P12 = Pedestrians blocked traffic for 12 s

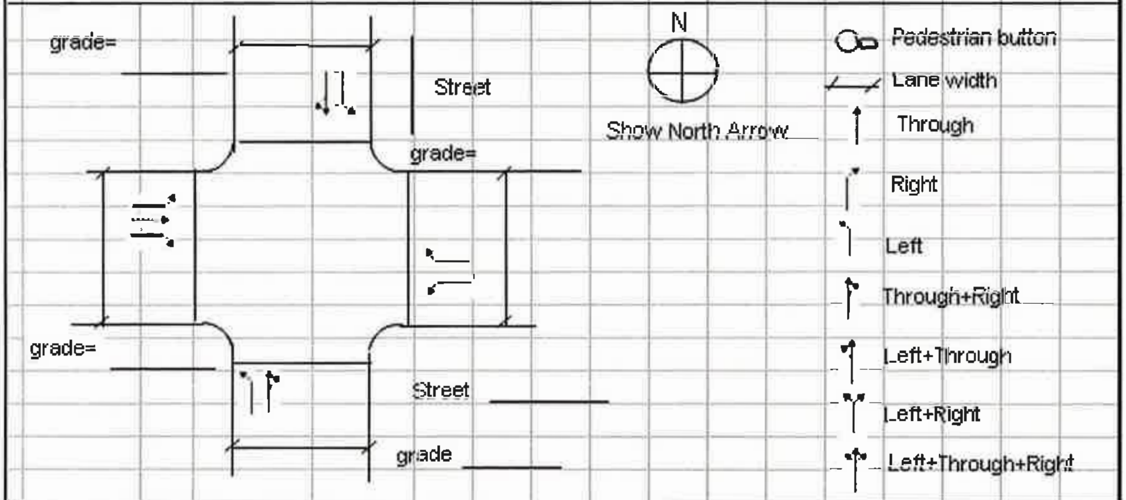
B15 = Bus blocked traffic for 15 s

3.5 EXAMPLE

INPUT WORKSHEET

General Information		**Site Information**	
Analyst	________	Intersection	________
Agency or Company	________	Area Type	☐ CBD ☐ Other
Date Performed	________	Jurisdiction	________
Analysis Time Period	________	Analysis Year	________

Intersection Geometry



Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, (V(veh/h)	18	269	307	85	165	170	46	112	53	38	149	43
Peak-hour factor, PHF	0.89	0.89	0.89	0.80	0.80	0.80	0.84	0.84	0.84	0.75	0.75	0.75
Pretimed (P) or actuated (A)	p	p	p	p	p	p	p	p	p	p	p	p
Start-up lost time, l_1(s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green time, e(s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival Type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Parking (Y or N)	N	N	N	N	N	N	N	N	N	N	N	N
Parking maneuvers, N_m (maneuvers/h)	0	0	0	0	0	0	0	0	0	0	0	0
Bus stopping, N_B(buses/h)	0	0	0	0	0	0	0	0	0	0	0	0

Signal Phasing Plan

	1	2	3	4	5	6	7	8
DIAGRAM								
Timing	G= 45.0	G= 30.0	G= 22.0	G= 45.0	G=	G=	G=	G=
	Y= 5.0	Y= 5.0	Y= 5.0	Y= 5.0	Y=	Y=	Y=	Y=

Protected turns — Permitted turns — Pedestrian — Cycle length, C= 162 s

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET												
General Information												
Project Description ______												
Volume Adjustment												
		EB			WB			NB			SB	
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume,V(veh/h)	18	269	307	85	165	170	46	112	53	38	149	43
Lane Group	L	T	R	L	T	R	L	TR		L	TR	
Total Cars	4	79	166	51	71	93	20	88		11	88	
fcar	0.33	0.28	0.71	0.69	0.48	0.61	0.54	0.71		0.43	0.74	
Total Motors	2.2	26	23.3	5.28	12.8	10.3	4.18	13.6		4.62	21.6	
fmotor	0.04	0.02	0.02	0.02	0.02	0.01	0.02	0.02		0.04	0.04	
Total Trailers	2.27	102	13.6	9.08	25	6.81	2.27	4.54		2.27	2.27	
ftrailer	0.43	0.84	0.13	0.28	0.38	0.1	0.14	0.08		0.2	0.04	
Total Lorries	3.57	55.9	23.8	5.95	15.5	19	2.38	11.9		3.57	4.76	
florry	0.35	0.24	0.12	0.1	0.12	0.15	0.08	0.11		0.17	0.05	
Total Buses	0	14.6	6.24	2.08	25	22.9	8.32	6.24		4.16	2.08	
fbus	0	0.11	0.06	0.06	0.35	0.31	0.47	0.1		0.34	0.04	
Adjusted flow rate in lane group,v (veh/h)	20.2	302	345	106	206	213	54.8	133	63.1	50.7	199	57.3
Proportion[1] of LT or RT(P_{LT} or P_{RT})	1.00	-	1.00	1.00	-	1.00	1.00	0.32	-	1.00	0.22	-
Saturation Flow Rate												
Base saturation flow, s_o(pc/h/ln)	1930	1930	1930	1930	1930	1930	1930	1930		1930	1930	
Number of lanes, N	1	1	1	1	1	1	1	1		1	1	
Lane width adjustment factor, f_w	1.07	0.96	0.96	1.01	0.96	0.9	0.96	0.9		0.74	0.68	
Vehicle composition adjustment factor,f_c	1.15	1.49	1.04	1.15	1.35	1.19	1.24	1.03		1.17	0.91	
Grade adjustment factor, f_g	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Area type adjustment factor, f_a	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Left-turn adjustment factor, f_{LT}	0.76	1.00	1.00	0.76	1.00	1.00	0.76	1.00		0.76	1.00	
Right-turn adjustment factor,f_{RT}	1.00	1.00	0.84	1.00	1.00	0.84	0.94	1.00		0.96	1.00	
Adjusted saturation flow, s(veh/h), s=s0, $s=s_o\ N\ f_w f_g f_a f_{LT} f_{RT} (1/f_c)$	1355	1239	1484	1293	1371	1229	1128	1600		922	1407	
Notes												
1. P_{LT} = 1.000 for exclusive left-turn lanes, and P_{RT}=1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group												

CAPACITY AND LOS WORKSHEET												
General Information												
Project Description												
Capacity Analysis												
Phase number												
	EB			WB			NB			SB		
Lane group	L	T	R	L	T	R	L	T	R	L	T	R
Adjusted flow rate, v(veh/h)	20.22	302.2	344.9	106.3	206.3	212.5	54.76	133.3	63.1	50.67	198.7	57.33
Saturation flow rate, s(veh/h)	1355	1239	1484	1293	1371	1229	1128	1600		922	1407	
Lost time, t_L(s), $t_L = l_1 + Y - e$	5.0	5.0	5.0	0.0	5.0	5.0	5.0	5.0		5.0	5.0	
Effective green time, g(s), $g = G + Y - t_L$	45.0	45.0	45.0	162.0	45.0	45.0	80.0	30.0		22.0	22.0	
Green ratio, g/C	0.278	0.278	0.278	1	0.278	0.278	0.494	0.185		0.136	0.136	
Lane group capacity,[1] c=s(g/C),(veh/h)	376.4	344.2	412.2	1293	380.8	341.4	557	296.3		125.2	191.1	
v/c ratio, X	0.054	0.878	0.837	0.082	0.542	0.622	0.098	0.45		0.405	1.04	
Flow ratio, v/s	0.015	0.244	0.232	0.082	0.15	0.173	0.049	0.083		0.055	0.141	
Critical lane group/phase (√)												
Sum of flow ratios for critical lane groups, Y												
$Y_c = \sum$ (critical lane group, v/s)												
Total lost time per cycle, L(s)												
Critical flow rate to capacity ratio, X_c												
$X_c = (Y_c)(C)/(C-L)$												
Lane Group Capacity, Control Delay, and LOS Determination												
	EB			WB			NB			SB		
Lane group	L	T	R	L	T	R	L	T	R	L	T	R
Adjusted flow rate,[2] v(veh/h)	20.22	302.2	344.9	106.3	206.3	212.5	54.76	133.3	63.1	50.67	198.7	57.33
Lane group capacity,[2] c(veh/h)	376.4	344.2	412.2	1293	380.8	341.4	557	296.3		125.2	191.1	
v/c ratio,[2] X = v/c	0.054	0.878	0.837	0.082	0.542	0.622	0.098	0.45		0.405	1.04	
Total green ratio,[2] g/C	0.278	0.278	0.278	1.00	0.278	0.278	0.494	0.185		0.136	0.136	
Uniform delay, $d_1 = \frac{0.50C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	58.5	58.5	55.04	0	58.5	58.5	41	66		70	70.44	
Incremental delay calibration,[3] k	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50		0.50	0.50	
Incremental delay,[4] d_2 $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8klX}{cT}}\right]$ (s/veh)	0.272	32.8	20.9	0.125	5.547	8.584	0.352	4.946		9.684	173.3	
Initial queue delay, d_3 (s/veh)	0	0	0	0	0	0	0	0		0	0	
Uniform delay, d_1 (s/veh)												
Progression adjustment factor, PF	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	58.77	91.3	75.95	0.125	64.05	67.08	41.35	70.95		79.68	243.7	
LOS by lane group (Table 3.1)	E	E	F	A	E	E	D	E		F	F	
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v_A}$ (s/veh)	82.4			52.3			46.7			171.0		
LOS by approach (Table 3.1)	F			E			D			F		
Approach flow rate, v_A (veh/h)	667.4			525.0			251.2			306.7		
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)	83.8			Intersection LOS (Table 3.1)						F		
Notes												
1. For permitted left turns, the minimum capacity is $(1 + P_L)(3600/C)$												
2. Primary and secondary phase parameters are summed to obtain lane group parameters												
3. For pretimed or nonactuated signals, k=0.50												
4. T = analysis duration (h); typically T = 15 minutes.												
I = upstream filtering metering adjustment factor; I = 1 for isolated intersections.												

CHAPTER 4

UNSIGNALISED INTERSECTIONS

CHAPTER 4

4.0 UNSIGNALISED INTERSECTIONS

CONTENTS

FIGURES

TABLES

4.1 INTRODUCTION

Unsignalised intersections are most common intersection type. Although their capacities may be lower than other intersection types, they do play an important part in the control of traffic flow in a network. A poorly operating unsignalised intersection may affect a signalised network or operation of an Intelligent Transport System. The theory of the operation of unsignalised intersections is fundamental to many elements of theory used for other intersections. For instance, queuing theory in traffic engineering used to analyze other unsignalised intersections is also been used to analyze other intersection type.

Concerning vehicle movements in intersections, there will be a number of conflicts, which influence traffic safety. The most common way to resolve such conflicts is by introducing priority controls i.e. a give way or stop rule, at the unsignalised intersection. The rules are implemented at T-intersection or 4-way junction. In Malaysia, majority of the unsignalised intersections is of the T-intersection. Most of the 4-way unsignalised intersection are either converted to signalized intersection.

Unsignalised intersection is the type of intersections commonly constructed in urban and also suburban areas. Currently the design of unsignalised intersection is based on the procedure available in the *Arahan Teknik (Jalan) 11/87*. The procedure is based on the methodologies adopted from the U.S. Highway Capacity Manual 1985.

4.2 METHODOLOGY

There are several parameters affecting the capacity of the unsignalised intersection i.e. junction geometric, critical gap, follow up time and vehicle characteristics. Nevertheless, two of the most important parameters affecting the capacity and performance of unsignalised intersection are the critical gap and the follow-up time.

The overall methodology for analyzing unsignalised intersection is shown in Figure 4.1. The methodology takes into consideration the Malaysian traffic condition such as the prevalence of motorcycles in the traffic stream. The presence of motorcycles is taken into consideration in the estimation of critical gap and follow-up times, and also in the computation of the potential capacity. The procedure for estimating control delays and threshold for level of service are adapted from TRB, 2000.

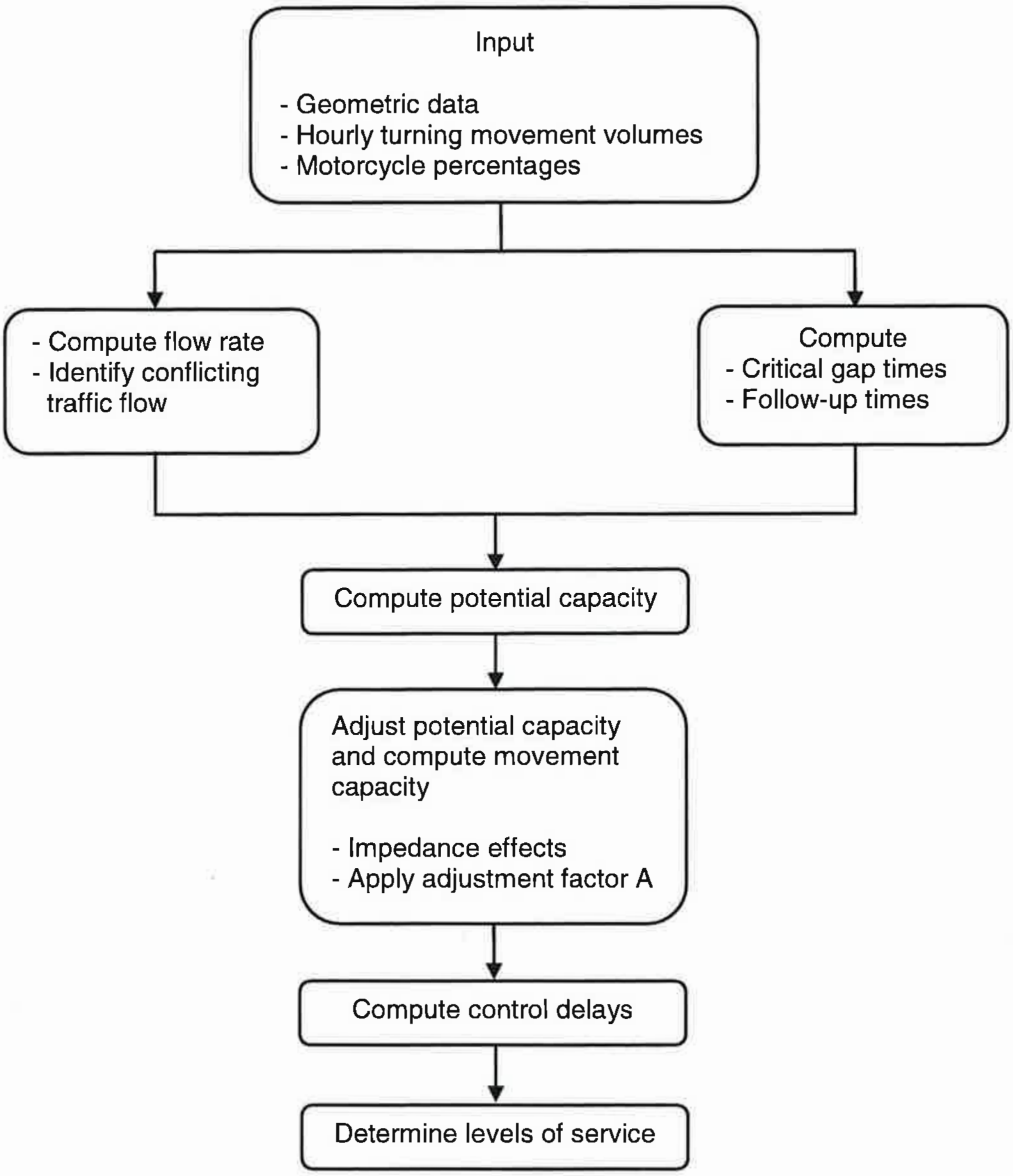


Figure 4.1 Operational Analysis Procedures

This section will be presenting the methodology to analyze capacity and level of service of two-way stop-controlled (TWSC) intersections. Capacity analysis at TWSC intersections depends on a clear description and understanding of the interaction of drivers on the minor or stop-controlled approach with the drivers on the major street. Procedures described in this chapter rely on gap acceptance model developed by Troutbeck (1992) where the data were collected at several TWSC intersections in Malaysia.

4.2.1 LEVEL OF SERVICE

Level of service (LOS) for an unsignalised intersection is determined by the estimation of control delay or measured control delay for each movement. LOS is not defined for the intersection as a whole. LOS criteria are given in Table 4.1.

Table 4.1 Level of Service Criteria for Unsignalised Intersections

Level of service	Average controlled delay (sec/veh)
A	0 – 10
B	> 10 – 15
C	> 15 – 25
D	> 25 – 35
E	> 35 – 50
F	> 50

*Note : Adapted from US HCM 2000

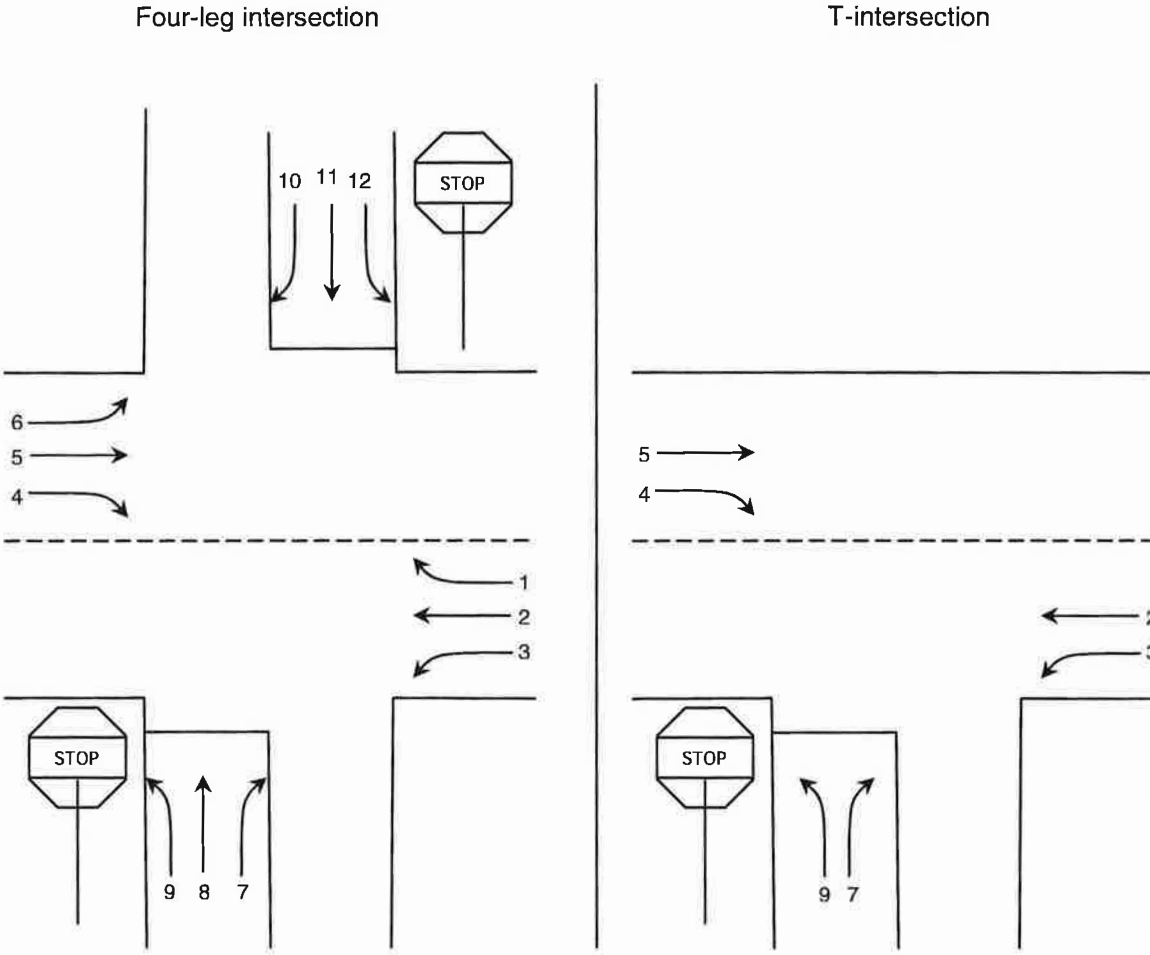
4.2.2 PRIORITY STREAMS

The priority of traffic stream at unsignalised intersection must be correctly identified. Some of the streams have absolute priority, while others have to give way or yield to higher rank stream. Figure 4.2 shows the comparative priority in the stream for two ways stop control intersection.

In the four-leg intersection, the priority of movement is described in four level ranks. Movement of rank 1 include through traffic on the major street and left turning from the major street. For movement rank 2, it includes right-turning from the major street and left-turning from the minor street. Rank 3 movement include through traffic on the minor street, as for the case of T-junction, right turning from the minor road will be included. There are no rank 4 in T-junction, but in four-leg intersection, rank 4 include right-turning from the minor street.

To demonstrate the priority stream concept and application, it is best to see how a four-leg intersection operates in such manner. However in Malaysia, the concept of two-way stopped control at a four-leg intersection is rarely found, especially in urban area. Most of the two way stop control at four-leg intersections are upgraded to signalised intersection especially in urban area.

The important criteria in priority stream is to identify the availability of gap. For example, if a right-turning vehicle on the major street and turning vehicle from minor street are waiting to cross the major stream, the first available gap of acceptable size would be taken priority to the right-turning vehicle on the major street. The minor street turning traffic must wait for the next available gap.



Rank	Traffic stream
1	2, 3, 5, 6
2	1, 4, 9, 12
3	8, 11
4	7,10

Rank	Traffic stream
1	2, 3, 5
2	4, 9
3	7

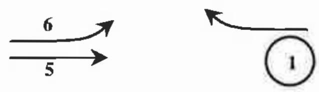
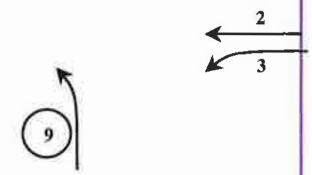
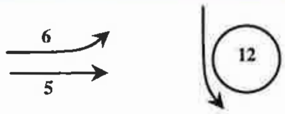
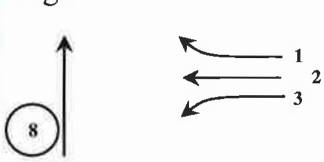
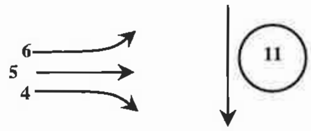
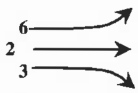
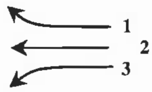
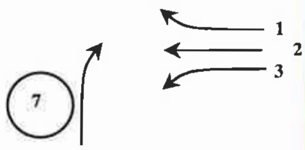
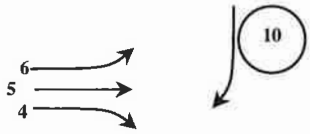
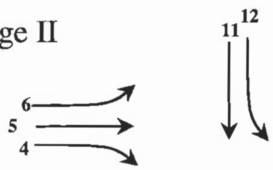
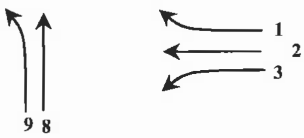
Figure 4.2 Traffic Streams at A Two Way Stop Control (TWSC) intersection

4.2.3 CONFLICTING TRAFFIC

Each movement at a TWSC intersection faces a different set of conflicts that are related to the nature of movement computation of conflict flows is as shown in Figure 4.3, which illustrates the computation of parameter $v_{c,x}$, the conflicting flow rate for movement x, that is, the total flow rate that conflicts with movement x (veh/h).

A typical unsignalised T-junction is as illustrated in Figure 4.4. There are six different types of traffic movements at the T-junction. The hierarchy of the unsignalised T-junction has three levels of conflicting streams that should be considered; i.e. movement Z, Y and X. The conflicting streams mean that the movements cannot cross the junction except the driver gives priority to other movement which has simple potential conflict and high saturation flow. Right turn movement from major road, i.e. movement Z, is the first conflicting stream because this movement is from the major stream where major stream is a priority stream that minor stream should be aware of it and must be given priority.

Second conflicting stream is left turn movement from minor stream. The last conflicting stream is right turn movement from minor stream. For left turn movement from minor stream, the driver has to give way only to movement 'A', but for right turn movement from minor stream, the driver has to give way to movements `A' 'B' and 'Z' which are more complicated.

Subject Movement	Subject and Conflicting Movements Conflicting Traffic Flows, $v_{c,x}$	
Major RT (1,4)	 $v_{c,1} = v_5 + v_6$ [a]	 $v_{c,4} = v_2 + v_3$ [a]
Minor LT (9,12)	 $v_{c,9} = \frac{v_{2[b]}}{N} + 0.5v_{3[c]}$	 $v_{c,12} = \frac{v_{5[b]}}{N} + 0.5v_{6[c]}$
Minor TH (8,11)	Stage I  $v_{c,I,8} = 2v_1 + v_2 + 0.5v_3$ [c]	 $v_{c,I,11} = 2v_4 + v_5 + 0.5v_6$ [c]
	Stage II  $v_{c,II,11} = 2v_6 + v_5 + v_6$ [a]	 $v_{c,II,11} = 2v_1 + v_2 + v_3$ [a]
Minor RT (7,10)	Stage I  $v_{c,I,7} = 2v_1 + v_2 + 0.5v_3$ [c]	 $v_{c,I,10} = 2v_4 + v_5 + 0.5v_6$ [c]
	Stage II  $v_{c,II,7} = 2v_4 + \frac{v_5}{N} + 0.5v_{6[d]} + 0.5v_{12[e,f]} + 0.5v_{11}$	 $v_{c,II,10} = 2v_1 + \frac{v_2}{N} + 0.5v_{3[d]} + 0.5v_{9[e,f]} + 0.5v_8$

*adapted from US HCM 2000

Figure 4.3 Definitions and Computation of Conflicting Flows

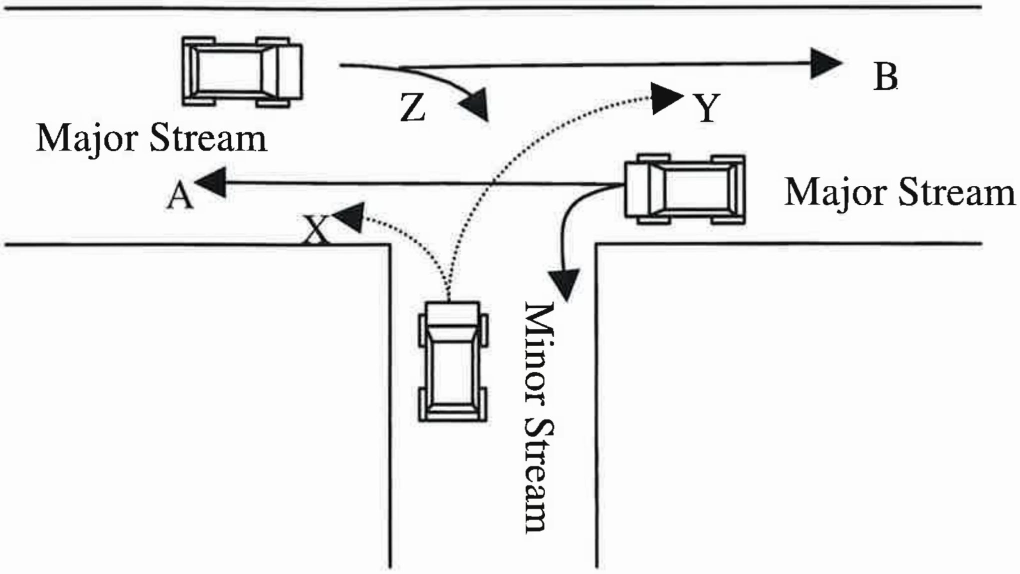


Figure 4.4 Vehicles Movement at A T-Junction (TWSC)

4.2.4 CRITICAL GAP AND FOLLOW-UP TIME

The critical gap, t_c, is the minimum time interval between the vehicles of the major stream that is necessary for the vehicles in the minor stream to enter the conflict area.

In addition, several vehicles of the minor stream can only follow one behind the other within a certain time space, which is defined as follow-up time. The follow-up time is the average time gap between two vehicles of the minor stream being queued and entering the same major stream gap one behind the other. Some of the well-known estimation procedures for critical gaps have been checked for consistency in Miller (1972) and Troutbeck (1992).

The estimation procedures should be consistent. The procedures should be able to reproduce the average critical gap reliably, without being dependent on other parameters such as traffic volumes on the major or the minor street, delay experienced by the drivers, and other external influences. The generalised control gap formula taking into consideration the influence of motorcycles is as shown by equation 4.1.

$$t_{c,x} = t_{c,base} - t_{c,M} P_{M,x} \tag{4.1}$$

where

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

$t_{c,base}$ = base critical gap from Table 4.2

$t_{c,M}$ = adjustment factor for motorcycle referring to Table 4.3

$P_{M,x}$ = proportion of motorcycles for movement x.

The follow-up time is computed for each minor movement using equation 4.2. As for the critical gap, adjustments are made for the presence of motorcycle.

$$t_{f,x} = t_{f,base} - t_{f,M} P_M \tag{4.2}$$

where

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

$t_{f,base}$ = base follow-up time from Table 4.2

$t_{f,M}$ = adjustment factor for motorcycle referring to Table 4.3

$P_{M,x}$ = proportion of motorcycles for movement x.

The value of base critical gap and follow-up time are as shown in Table 4.2. The value of the critical gap and follow up time influence the capacity at unsignalised intersection. However, more accurate capacity estimates will be produced if field measurement can be acquired.

Table 4.2 Base Critical Gap and Follow-Up Time for TWSC Intersections

Vehicle Movement	Base Critical Gap, $T_{c,\ base}$ (second)		Base Follow-up Time, $T_{f,\ base}$ (second)	
	Single Lane	Multi Lane	Single Lane	Multi Lane
Right turn from major	3.5	3.7	2.0	2.1
Left turn from minor	3.2	3.3	1.9	2.1
Right turn from minor	4.0	4.2	2.2	2.4

Table 4.3 Adjustment Factor for Motorcycle

$t_{c,m}$		$t_{f,m}$	
Single-lane	Multi-lane	Single-lane	Multi-lane
0.424	0.252	0.738	0.815

Table 4.3 shows the value of $t_{c,m}$ and $t_{f,m}$ to be included in equation 4.1 and 4.2, in order to get critical gap and follow-up time.

4.2.5 POTENTIAL CAPACITY

The potential capacity is defined as the "ideal" capacity for a specific subject movement, assuming the following conditions:

1. Traffic on the major roadway does not block the minor road.
2. Traffic from nearby intersections does not back up into the intersection under consideration.
3. A separate lane is provided for the exclusive use of each minor street movement under consideration.
4. No other movements impede the subject movement.

When traffic becomes congested in a high-priority movement, it can impede the potential capacity. These impedance effects can be derived by multiplying the potential capacity to the series of impedance factor for every impeded movement. The impedance effect will be discussed later in this chapter.

The gap acceptance method employed in the procedure used in determining the capacity of these intersections computes the potential capacity of each minor traffic stream in accordance with equation 4.3. The use of small critical gap and follow-up time based on Table 4.2 resulting in the increase in potential capacity of the unsignalised intersection. In order to obtain a realistic value, the potential capacity need to be calibrated in the field and the potential capacity adopted from the US HCM 2000 is adjusted by using an adjustment factor, A_x. The adjustment factor will ensure that the estimated potential capacity will be according to the Malaysian traffic condition. The value for adjustment factor, A_x is shown in Table 4.4.

$$c_{mp,x} = A_x \left(V_{c,x} \frac{e^{-v_{c,x} t_{c,x}/3600}}{1 - e^{-v_{c,x} t_{f,x}/3600}} \right) \tag{4.3}$$

where

$c_{mp,x}$ = potential capacity for movement x (veh/h)
$v_{c,x}$ = conflicting flow rate for movement x (veh/h)
$t_{c,x}$ = critical gap for movement x (sec)
$t_{f,x}$ = follow-up time for movement x (sec)
A_x = adjustment factor for movement x (refer Table 4.4)

Table 4.4 Adjustment Factor for Capacity, A

Vehicle maneuver	A value	
	Single lane	Multi lane
Right-turn from major street	1.000	1.000
Left turn from minor street	0.4846	0.5181
Right turn from minor street	0.4375	0.4864

4.2.6 RIGHT TURN FROM MAJOR

Based on equation 4.1 and equation 4.2, the critical gaps and follow up times for right turning into minor road from major road have been tabulated according to the proportion of motorcycles in that particular movement. Table 4.5 and Table 4.6 are the critical gaps and follow-up times for single lane and multilane facility, respectively.

Table 4-5 Critical Gap and Follow Up Time for Right Turn from Major Movement (Single Lane Approach).

Proportion of motorcycles for movement x, $P_{M,x}$	Critical gap for movement x (sec), $t_{c,x}$	Follow up times for movement x (sec), $t_{f,x}$
0.0	3.500	2.000
0.1	3.458	1.926
0.2	3.415	1.852
0.3	3.373	1.779
0.4	3.330	1.705
0.5	3.288	1.631
0.6	3.246	1.557
0.7	3.203	1.483
0.8	3.161	1.410
0.9	3.118	1.336
1.0	3.076	1.262

Table 4-6 Critical Gap and Follow Up Time for Right Turn from Major Movement (Multilane Approach)

Proportion of motorcycles for minor movement x, P_M	Critical gap for minor movement x (sec), $t_{c,x}$	Follow up times for minor movement x (sec), $t_{f,x}$
0.0	3.700	2.100
0.1	3.658	2.019
0.2	3.615	1.937
0.3	3.573	1.856
0.4	3.530	1.774
0.5	3.488	1.693
0.6	3.446	1.611
0.7	3.403	1.530
0.8	3.361	1.448
0.9	3.318	1.367
1.0	3.276	1.285

4.2.7 LEFT TURN FROM MINOR

Based on equation 4.1 and equation 4.2, the critical gaps and follow up times for left turning from major road have been tabulated according to the proportion of motorcycles in that particular movement. Table 4.7 and Table 4.8 are critical gaps and follow-up times for single lane and multilane, respectively.

Table 4-7 Critical Gap and Follow Up Time for Left Turn from Minor Movement (Single Lane Approach)

Proportion of motorcycles for movement x, P_M	Critical gap for movement x (sec), $t_{c,x}$	Follow up times for movement x (sec), $t_{f,x}$
0.0	3.200	1.900
0.1	3.158	1.826
0.2	3.115	1.752
0.3	3.073	1.679
0.4	3.030	1.605
0.5	2.988	1.531
0.6	2.946	1.457
0.7	2.903	1.383
0.8	2.861	1.310
0.9	2.818	1.236
1.0	2.776	1.162

Table 4-8 Critical Gap and Follow Up Time for Left Turn Minor Movement (Multilane Approach)

Proportion of motorcycles for movement x,P_M	Critical gap for movement x (sec), $t_{c,x}$	Follow up times for movement x (sec), $t_{f,x}$
0.0	3.300	2.100
0.1	3.258	2.019
0.2	3.215	1.937
0.3	3.173	1.856
0.4	3.130	1.774
0.5	3.088	1.693
0.6	3.046	1.611
0.7	3.003	1.530
0.8	2.961	1.448
0.9	2.918	1.367
1.0	2.876	1.285

4.2.8 RIGHT TURN FROM MINOR

Based on equation 4.1 and equation 4.2, the critical gaps and follow up times for right turning from minor road have been tabulated according to the proportion of motorcycles in that particular movement. Table 4.9 and Table 4.10 are the critical gaps and follow-up times for single lane and multilane facilities.

Table 4-9 Critical Gap and Follow Up Time for Right Turn from Minor Movement (Single Lane Approach)

Proportion of motorcycles for minor movement x, P_M	Critical gap for minor movement x (sec), $t_{c,x}$	Follow up times for minor movement x (sec), $t_{f,x}$
0.0	4.000	2.200
0.1	3.958	2.126
0.2	3.915	2.052
0.3	3.873	1.979
0.4	3.830	1.905
0.5	3.788	1.831
0.6	3.746	1.757
0.7	3.703	1.683
0.8	3.661	1.610
0.9	3.618	1.536
1.0	3.576	1.462

Table 4-10 Critical Gap and Follow Up Time for Right Turn from Minor Movement (Multilane Approach)

Proportion of motorcycles for minor movement x, P_M	Critical gap for minor movement x (sec), $t_{c,x}$	Follow up times for minor movement x (sec), $t_{f,x}$
0.0	4.200	2.400
0.1	4.158	2.319
0.2	4.115	2.237
0.3	4.073	2.156
0.4	4.030	2.074
0.5	3.988	1.993
0.6	3.946	1.911
0.7	3.903	1.830
0.8	3.861	1.748
0.9	3.818	1.667
1.0	3.776	1.585

4.2.9 IMPEDANCE EFFECT

Vehicle Impedance

Vehicle use gaps at a two way stopped control (TWSC) intersection in a prioritized manner. When traffic becomes congested in a high-priority movement, it can be impeded by lower-priority movements from using gaps in the traffic stream thus reducing the capacity of these movements.

Minor traffic streams of Rank 2 (including right turns from the major street and left turns from the minor street) must yield only to the major-street through and left-turning traffic streams of Rank 1. There are no additional impedances from other minor traffic streams, and so the movement capacity of each Rank 2 traffic stream is equal to its potential capacity as indicated by equation 4.4.

$$C_{m,j} = C_{p,j} \qquad (4.4)$$

where j denotes movements of Rank 2 priority.

For TWSC T-intersection, the probability that the major-street right-turning traffic will operate in a queue-free state is computed using equation 4.5 as adapted from the HCM 2000.

$$P_{0,j} = 1 - \frac{v_j}{C_{m,j}} \qquad (4.5)$$

Where

j = 3 (major-street right-turn movements of Rank 2).

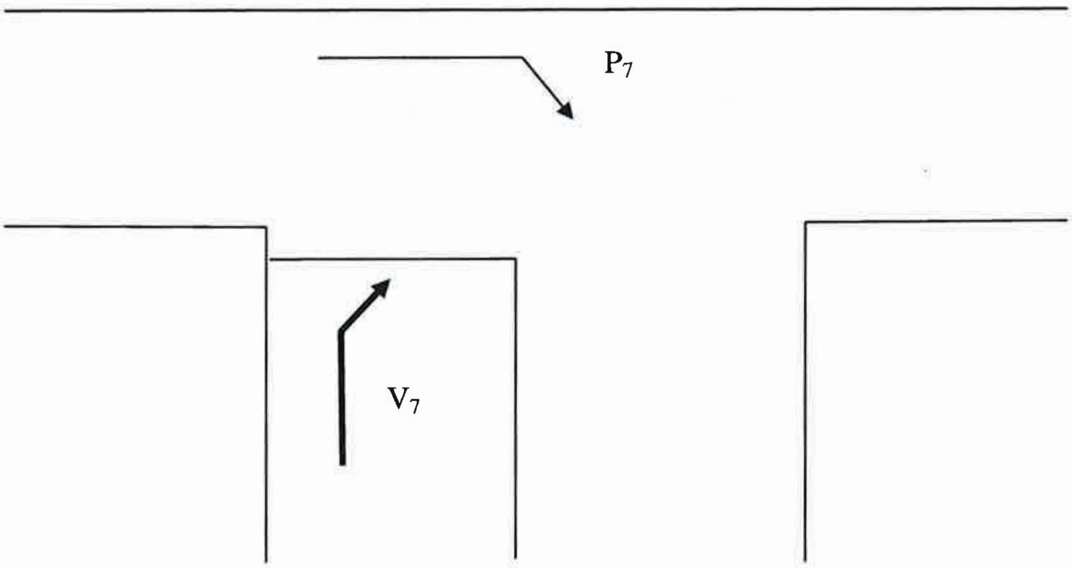


Figure 4.5 Example of Impedance Effect

Referring to Figure 4.5, right turn movement is impeded by queue of the major-street right turning.

The movement capacity for right turn from minor is computed using equation 4.6.

$$C_{m,7} = C_{p,7} \times P_{o,7} \qquad (4.6)$$

4.2.10 MOVEMENT CAPACITY

The potential capacity, $c_{p,x}$, of minor street movements is given in Figure 4.6 for a single lane street and in Figure 4.7 for a multilane street. These graphs show the application of the values in Table 4.2, Table 4.4 and the equation 4.3. The potential capacity is presented as vehicles per hour (veh/h). The figure indicate that the potential capacity is a function of the conflicting flow rate, $v_{c,x}$ expressed as an hourly rate, as well as the minor-street movement.

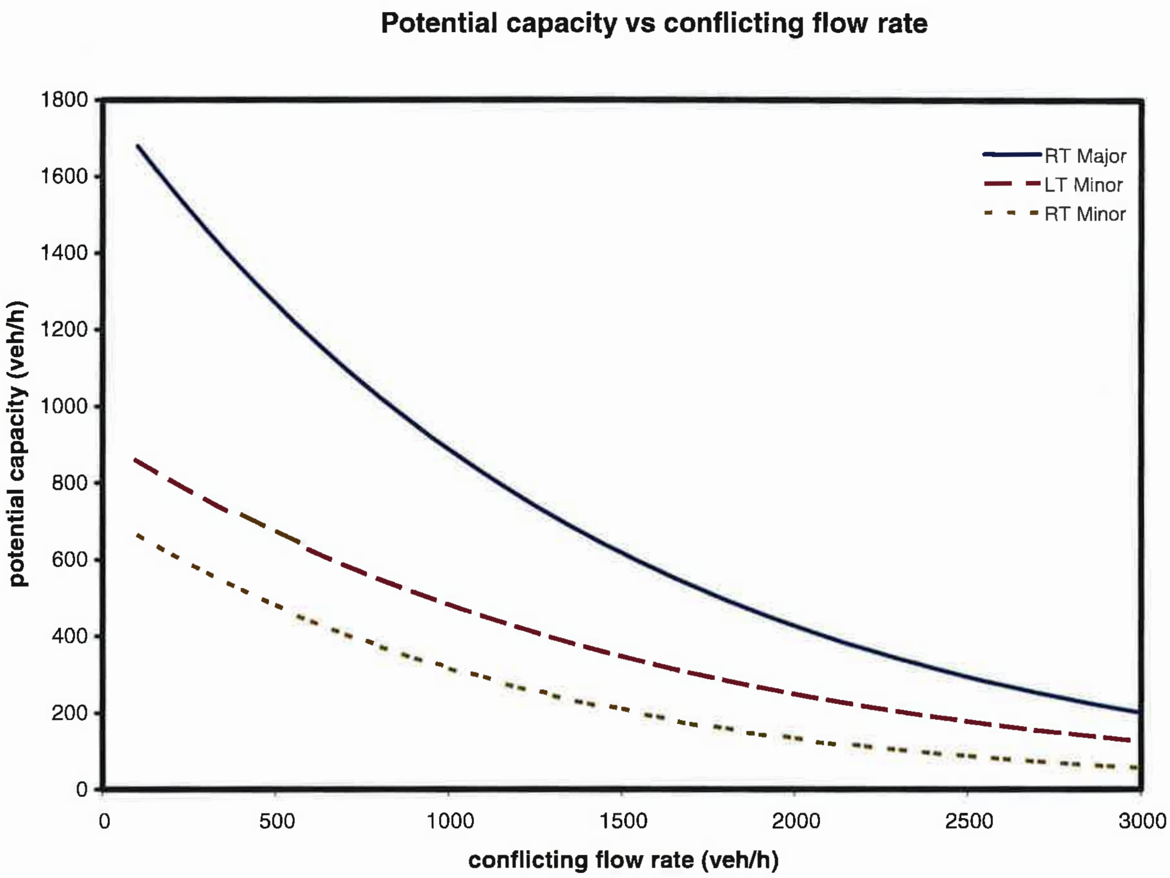


Figure 4.6 Potential Capacity for Single Lane

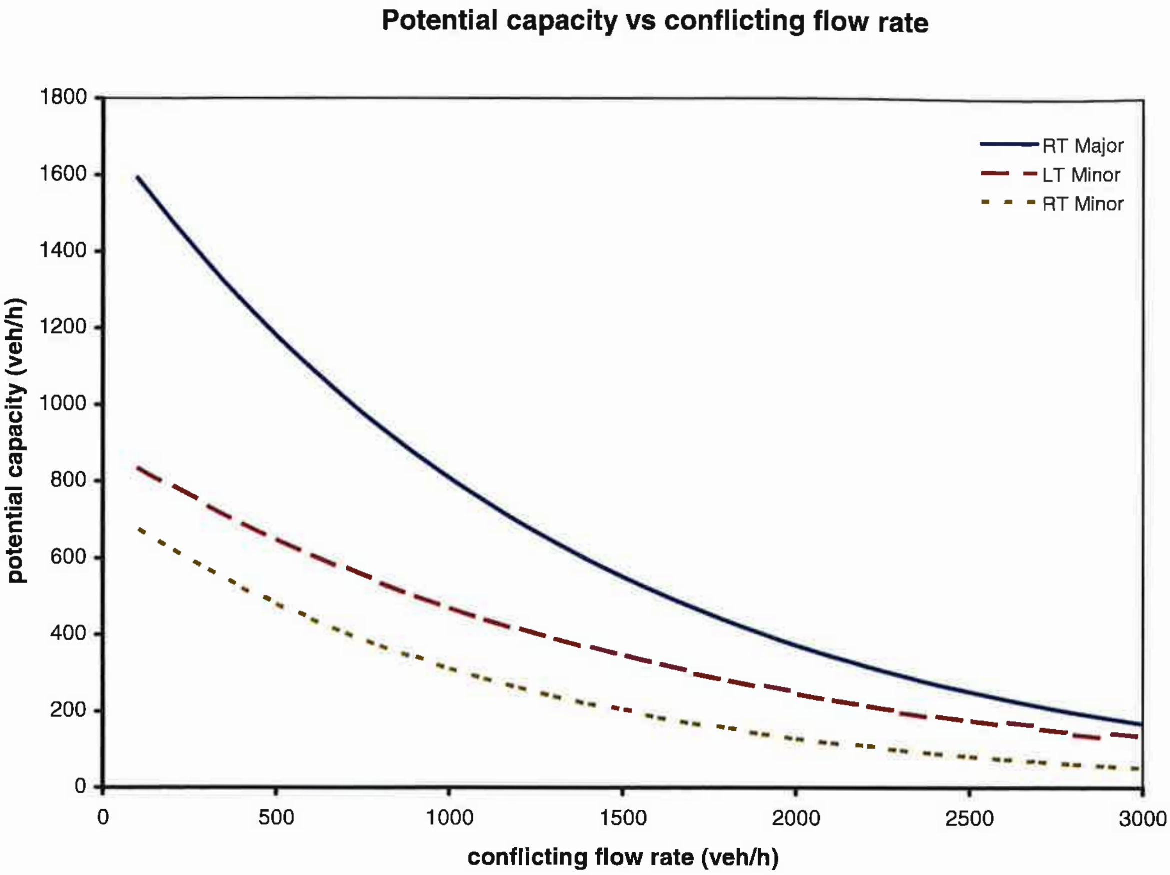


Figure 4.7 Potential Capacity for Multi lanes

4.2.10.1 Capacity for Right Turn from Major Road

Capacity calculation for right turn from major road is based on equation 4.3. It is important to know the conflicting flow rate in order to calculate the potential capacity for each movement. Table 4.11 and 4.12 simplify the potential capacity for each single lane and multilane for right turn from major road, respectively.

Table 4.11 Potential Capacity for Right Turn from Major for Single Lane (movement x = 4)

P_M	Conflicting Flow Rate, $v_{c,4}$														
	100	**200**	**300**	**400**	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**	**1300**	**1400**	**1500**
	Potential Capacity, $c_{mp,4}$														
0.0	1679	1566	1460	1361	1268	1181	1100	1024	954	887	826	768	714	664	617
0.1	1744	1626	1517	1414	1318	1228	1144	1065	992	924	860	800	744	692	644
0.2	1813	1692	1578	1471	1372	1278	1191	1110	1034	963	896	834	777	723	672
0.3	1889	1762	1644	1533	1430	1333	1243	1158	1079	1005	936	872	812	755	703
0.4	1971	1839	1716	1601	1493	1393	1298	1210	1128	1051	979	912	850	791	736
0.5	2061	1923	1795	1675	1562	1457	1359	1267	1181	1101	1026	956	891	830	773
0.6	2159	2015	1881	1755	1638	1528	1426	1330	1240	1156	1078	1005	936	872	813
0.7	2266	2116	1976	1844	1721	1606	1499	1398	1304	1216	1134	1058	986	919	857
0.8	2385	2228	2080	1942	1813	1692	1579	1474	1375	1283	1197	1116	1041	970	905
0.9	2518	2351	2196	2051	1915	1788	1669	1558	1454	1357	1266	1181	1102	1028	958
1.0	2665	2490	2326	2172	2029	1894	1769	1651	1542	1439	1343	1253	1170	1091	1018

P_M	Conflicting Flow Rate, $v_{c,4}$														
	1600	**1700**	**1800**	**1900**	**2000**	**2100**	**2200**	**2300**	**2400**	**2500**	**2600**	**2700**	**2800**	**2900**	**3000**
	Potential Capacity, $c_{mp,4}$														
0.0	573	533	495	459	427	396	367	341	316	293	272	252	233	216	200
0.1	598	556	517	480	446	414	384	357	331	307	285	264	245	227	210
0.2	625	581	540	502	467	434	403	374	347	322	299	278	258	239	222
0.3	654	608	566	526	489	455	423	393	365	339	315	292	271	252	234
0.4	685	638	594	552	514	478	444	413	384	357	331	308	286	266	247
0.5	720	670	624	580	540	503	468	435	404	376	350	325	302	281	261
0.6	757	705	657	611	569	530	493	459	427	397	369	344	319	297	276
0.7	798	744	693	645	601	560	521	485	452	420	391	364	339	315	293
0.8	843	786	733	683	636	593	552	514	479	446	415	387	360	335	312
0.9	894	833	777	724	675	629	586	546	509	474	442	411	383	357	332
1.0	950	886	826	771	719	670	625	582	543	506	471	439	409	381	355

Table 4.12 Potential Capacity for Right Turn from Major for Multi Lane (movement x = 4)

P_M	Conflicting Flow Rate, $v_{c,4}$														
	100	**200**	**300**	**400**	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**	**1300**	**1400**	**1500**
	Potential Capacity, $c_{mp,4}$														
0.0	1592	1479	1373	1274	1182	1097	1017	943	874	810	750	694	643	595	551
0.1	1656	1537	1427	1324	1228	1138	1055	978	906	840	777	720	666	617	570
0.2	1725	1601	1485	1377	1277	1184	1097	1016	942	872	807	747	692	640	592
0.3	1800	1669	1548	1435	1330	1233	1142	1058	980	907	840	777	719	665	615
0.4	1882	1745	1617	1499	1389	1287	1192	1104	1022	946	875	810	749	693	641
0.5	1972	1827	1693	1569	1453	1346	1246	1153	1068	988	914	846	782	723	669
0.6	2071	1918	1777	1645	1523	1410	1305	1208	1118	1034	957	885	818	756	699
0.7	2180	2019	1869	1730	1601	1482	1371	1269	1174	1086	1004	928	858	793	733
0.8	2302	2130	1972	1825	1688	1562	1445	1336	1236	1143	1056	977	903	834	771
0.9	2438	2256	2087	1930	1785	1651	1527	1412	1305	1206	1115	1031	952	880	813
1.0	2591	2397	2216	2050	1895	1752	1619	1497	1383	1278	1181	1091	1008	931	860

P_M	Conflicting Flow Rate, $v_{c,4}$														
	1600	**1700**	**1800**	**1900**	**2000**	**2100**	**2200**	**2300**	**2400**	**2500**	**2600**	**2700**	**2800**	**2900**	**3000**
	Potential Capacity, $c_{mp,4}$														
0.0	509	471	435	402	372	343	317	293	270	249	230	212	196	180	166
0.1	528	488	451	417	385	356	329	303	280	258	238	220	203	187	172
0.2	547	506	468	432	400	369	341	315	291	268	247	228	210	194	179
0.3	569	526	486	449	415	383	354	327	302	279	257	237	219	202	186
0.4	592	548	506	468	432	399	369	340	314	290	268	247	228	210	194
0.5	618	571	528	488	451	416	384	355	327	302	279	257	237	219	202
0.6	646	597	552	510	471	435	402	371	342	316	291	269	248	229	211
0.7	678	626	578	534	493	456	421	388	358	331	305	282	260	240	221
0.8	712	658	608	561	518	478	442	408	376	347	320	296	273	252	232
0.9	751	693	640	591	546	504	465	429	396	366	337	311	287	265	244
1.0	794	733	677	625	577	533	492	454	419	386	356	329	303	280	258

4.2.10.2 Capacity for Left Turn from Minor

Table 4.13 Potential Capacity for Left Turn from Minor for Single Lane (movement x = 9)

P_M	Conflicting Flow Rate, $v_{c,9}$														
	100	***200***	***300***	***400***	***500***	***600***	***700***	***800***	***900***	***1000***	***1100***	***1200***	***1300***	***1400***	***1500***
	Potential Capacity, $c_{mp,9}$														
0.0	862	810	760	714	670	628	589	553	518	486	455	427	400	374	350
0.1	897	843	792	743	698	655	614	576	540	507	475	445	417	391	366
0.2	935	879	825	775	728	683	641	601	564	529	496	465	436	409	383
0.3	977	918	862	810	760	714	670	629	590	554	520	487	457	428	402
0.4	1022	960	902	848	796	748	702	659	619	581	545	511	480	450	422
0.5	1071	1007	946	889	835	785	737	692	650	610	573	537	504	473	444
0.6	1126	1058	995	935	878	825	775	728	684	642	603	566	531	499	468
0.7	1186	1115	1048	985	926	870	818	768	722	678	637	598	562	527	495
0.8	1253	1178	1108	1042	979	920	865	813	764	718	674	634	595	559	525
0.9	1328	1249	1175	1105	1039	977	918	863	811	762	716	673	632	594	558
1.0	1412	1329	1250	1176	1106	1040	978	919	864	812	764	718	675	634	596

	Conflicting Flow Rate, $v_{c,9}$														
	1600	***1700***	***1800***	***1900***	***2000***	***2100***	***2200***	***2300***	***2400***	***2500***	***2600***	***2700***	***2800***	***2900***	***3000***
	Potential Capacity, $c_{mp,9}$														
0.0	328	307	287	269	251	235	220	205	192	179	167	156	146	136	127
0.1	343	321	300	281	263	246	230	215	201	188	176	164	153	143	134
0.2	359	336	315	295	276	258	242	226	212	198	185	173	162	151	141
0.3	376	353	330	310	290	271	254	238	223	208	195	182	171	159	149
0.4	395	371	347	326	305	286	268	251	235	220	206	193	180	169	158
0.5	416	390	366	343	322	302	283	265	248	232	218	204	191	179	167
0.6	439	412	387	363	340	319	299	280	263	246	231	216	202	190	178
0.7	465	436	409	384	360	338	317	297	279	261	245	230	215	202	189
0.8	493	463	434	408	383	359	337	316	297	278	261	245	229	215	202
0.9	524	492	462	434	408	383	359	337	317	297	279	262	245	230	216
1.0	560	526	494	464	436	409	384	361	339	318	299	280	263	247	232

Table 4.14 Potential Capacity for Left Turn from Minor for Multi Lane (movement x=9)

P_M	Conflicting Flow Rate, $v_{c,9}$														
	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500
	Potential Capacity, $c_{mp,9}$														
0.0	834	783	735	690	648	607	569	534	500	469	439	411	385	360	337
0.1	868	814	764	717	672	630	591	554	519	486	455	426	399	373	349
0.2	904	848	795	746	699	656	614	576	539	505	473	442	414	387	362
0.3	943	884	829	777	729	683	640	599	561	525	492	460	431	403	377
0.4	986	924	866	812	761	713	667	625	585	548	512	479	448	419	392
0.5	1033	968	907	850	796	745	698	653	611	572	535	501	468	438	409
0.6	1085	1016	952	891	834	781	731	684	640	599	560	524	490	458	428
0.7	1142	1069	1001	937	877	821	768	718	672	629	588	550	514	480	449
0.8	1206	1129	1056	988	925	865	809	757	708	662	618	578	540	505	472
0.9	1277	1195	1118	1046	978	914	855	799	747	699	653	610	570	533	497
1.0	1358	1270	1187	1110	1038	970	907	848	792	740	692	646	603	564	526

P_M	Conflicting Flow Rate, $v_{c,9}$														
	1600	1700	1800	1900	2000	2100	2200	2300	2400	2500	2600	2700	2800	2900	3000
	Potential Capacity, $c_{mp,9}$														
0.0	315	295	276	257	241	225	210	196	183	171	159	148	138	129	120
0.1	327	305	285	267	249	233	217	203	189	177	165	154	143	134	125
0.2	339	317	296	277	258	241	225	211	197	183	171	160	149	139	129
0.3	352	329	308	287	269	251	234	219	204	191	178	166	155	144	134
0.4	367	343	320	299	280	261	244	228	212	198	185	173	161	150	140
0.5	383	358	334	312	292	272	254	237	222	207	193	180	168	157	146
0.6	400	374	349	326	305	285	266	248	231	216	202	188	175	164	153
0.7	419	392	366	342	319	298	278	260	242	226	211	197	184	171	160
0.8	441	412	385	359	335	313	292	273	255	238	222	207	193	180	168
0.9	465	434	405	378	353	330	308	287	268	250	233	218	203	189	177
1.0	492	459	429	400	373	349	325	304	283	264	247	230	214	200	186

4.2.10.3 Capacity for Right Turning from Minor

Table 4-15 Potential Capacity for Right Turn from Minor for Single Lane (movement x=7)

P_M	Conflicting Flow Rate, $v_{c,7}$														
	100	***200***	***300***	***400***	***500***	***600***	***700***	***800***	***900***	***1000***	***1100***	***1200***	***1300***	***1400***	***1500***
	Potential Capacity, $c_{mp,7}$														
0.0	660	609	561	517	477	439	404	372	342	315	290	266	245	225	207
0.1	683	630	581	536	494	455	419	386	355	327	301	276	254	234	187
0.2	708	653	602	555	512	472	435	400	369	339	312	287	264	243	195
0.3	735	678	625	577	532	490	452	416	383	353	325	299	275	253	203
0.4	763	704	650	599	553	510	470	433	399	367	338	312	287	264	212
0.5	794	733	676	624	576	531	489	451	416	383	353	325	299	276	222
0.6	828	764	705	651	600	554	511	471	434	400	369	340	313	288	232
0.7	864	798	737	680	627	579	534	492	454	419	386	356	328	302	243
0.8	904	835	771	711	657	606	559	516	476	439	405	373	344	317	255
0.9	947	875	808	746	689	636	587	542	500	461	425	392	362	333	268
1.0	995	920	849	784	724	669	617	570	526	485	448	413	381	352	283

	Conflicting Flow Rate, $v_{c,7}$														
	1600	***1700***	***1800***	***1900***	***2000***	***2100***	***2200***	***2300***	***2400***	***2500***	***2600***	***2700***	***2800***	***2900***	***3000***
	Potential Capacity, $c_{mp,7}$														
0.0	190	174	160	147	134	123	113	104	95	87	80	73	67	61	56
0.1	197	181	166	153	140	128	118	108	99	91	83	76	70	64	51
0.2	205	189	173	159	146	134	123	113	104	95	87	80	73	67	54
0.3	214	197	181	166	153	140	129	118	108	99	91	84	77	70	56
0.4	223	205	189	174	160	147	135	124	114	104	96	88	81	74	59
0.5	233	215	198	182	167	154	141	130	119	109	101	92	85	78	62
0.6	244	225	207	190	175	161	148	136	125	115	106	97	89	82	66
0.7	256	236	217	200	184	169	156	143	132	121	111	102	94	87	69
0.8	269	248	228	210	194	178	164	151	139	128	118	108	100	91	73
0.9	283	261	241	222	204	188	173	159	147	135	124	114	105	97	78
1.0	299	276	254	234	216	199	183	169	155	143	132	121	112	103	83

Table 4-16 Potential Capacity for Right Turn from Minor for Multi Lane (movement x=7)

P_M	Conflicting Flow Rate, $v_{c,7}$														
	100	***200***	***300***	***400***	***500***	***600***	***700***	***800***	***900***	***1000***	***1100***	***1200***	***1300***	***1400***	***1500***
	Potential Capacity, $c_{mp,7}$														
0.0	671	617	567	521	479	440	403	370	340	311	285	261	239	219	201
0.1	694	638	587	539	495	454	417	382	350	321	294	270	247	226	207
0.2	719	661	607	558	512	470	431	395	362	332	304	278	255	233	214
0.3	746	685	629	578	530	486	446	409	375	343	315	288	264	241	221
0.4	775	712	653	599	550	504	462	424	388	356	326	298	273	250	229
0.5	807	740	679	623	571	524	480	440	403	369	338	309	283	259	237
0.6	841	771	707	649	595	545	499	457	419	384	351	321	294	269	246
0.7	878	805	738	676	620	568	520	476	436	399	365	334	306	280	256
0.8	918	842	771	707	647	593	543	497	455	416	381	349	319	292	267
0.9	963	882	808	740	678	621	568	520	476	435	398	364	333	305	278
1.0	1012	927	849	777	711	651	596	545	499	456	417	381	349	319	291

	Conflicting Flow Rate, $v_{c,7}$														
	1600	***1700***	***1800***	***1900***	***2000***	***2100***	***2200***	***2300***	***2400***	***2500***	***2600***	***2700***	***2800***	***2900***	***3000***
	Potential Capacity, $c_{mp,7}$														
0.0	183	168	153	140	128	117	107	97	89	81	74	67	61	56	51
0.1	189	173	158	145	132	121	110	101	92	84	76	70	63	58	53
0.2	195	179	163	149	136	125	114	104	95	86	79	72	66	60	54
0.3	202	185	169	154	141	129	118	107	98	89	81	74	68	62	56
0.4	209	191	175	160	146	133	122	111	101	92	84	77	70	64	58
0.5	217	198	181	165	151	138	126	115	105	96	87	80	73	66	60
0.6	225	206	188	172	157	143	131	119	109	99	91	83	75	69	63
0.7	234	214	195	179	163	149	136	124	113	103	94	86	78	72	65
0.8	244	223	203	186	170	155	142	129	118	108	98	90	82	75	68
0.9	254	232	212	194	177	162	148	135	123	112	102	94	85	78	71
1.0	266	243	222	203	185	169	155	141	129	117	107	98	89	81	74

4.2.10.4 Shared Lane Capacity

Minor street Approaches

Equation 4.7 is used to compute shared lane capacity.

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}} \right)} \tag{4.7}$$

Where

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

v_y = flow rate for movement y in a shared lane (veh/hr)

$C_{m,y}$ = movement capacity for movement y in a shared lane (veh/hr)

Major Street Approaches

Equation 4.8 is a derived equation from equation 4.6 for potential capacity for queues on a major street with shared lane right-turn lanes may be taken into consideration.

$$P^{*}_{0,j} = 1 - \frac{1 - p_{0,j}}{1 - \left(\frac{v_{i1}}{s_{i1}} + \frac{v_{i2}}{s_{i2}} \right)} \qquad (4.8)$$

Where

$P_{0,j}$	= probability of queue-free state for movement j assuming an exclusive right-turn lane on major street
j	= 1,4 (major street right turning traffic streams)
i1	= 2,5 (major street through traffic streams)
i2	= 3,6 (major street left turning traffic streams)
s_{i1}	= saturation flow rate for the major street through traffic streams (veh/hr) – this parameter can be measured in the field
s_{i2}	= saturation flow rate for the major street left turning traffic (veh/hr) – this parameter can be measured in the field
v_{i1}	= major street through flow rate (veh/hr)
v_{i2}	= major street left turning flow rate (0 if an exclusive left turn lane is provided) (veh/hr)

4.2.11 ESTIMATING QUEUE LENGHTS

Queue length estimation is an important calculation for unsignalised intersections. It is stated in TRB, 2000, theoretical studies and empirical observations have demonstrated that the probability distribution of queue lengths for any minor movement at an unsignalised intersection is a function of the capacity of the movement and the volume of traffic being served during the analysis period.

Equation 4.9 and Figure 4.8 can be used to calculate and to predict the 95th percentile queue length for any minor movement at an unsignalised intersection during the 15 minutes peak hour period.

$$Q_{95} \approx 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\right]\left(\frac{c_{m,x}}{3600}\right) \quad (4.9)$$

Where

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

v_x = flow rate for movement x (veh/hr)

$C_{m,x}$ = capacity of movement x (veh/hr)

T = analysis time period (h) (T=0.25 for a 15 minutes period)

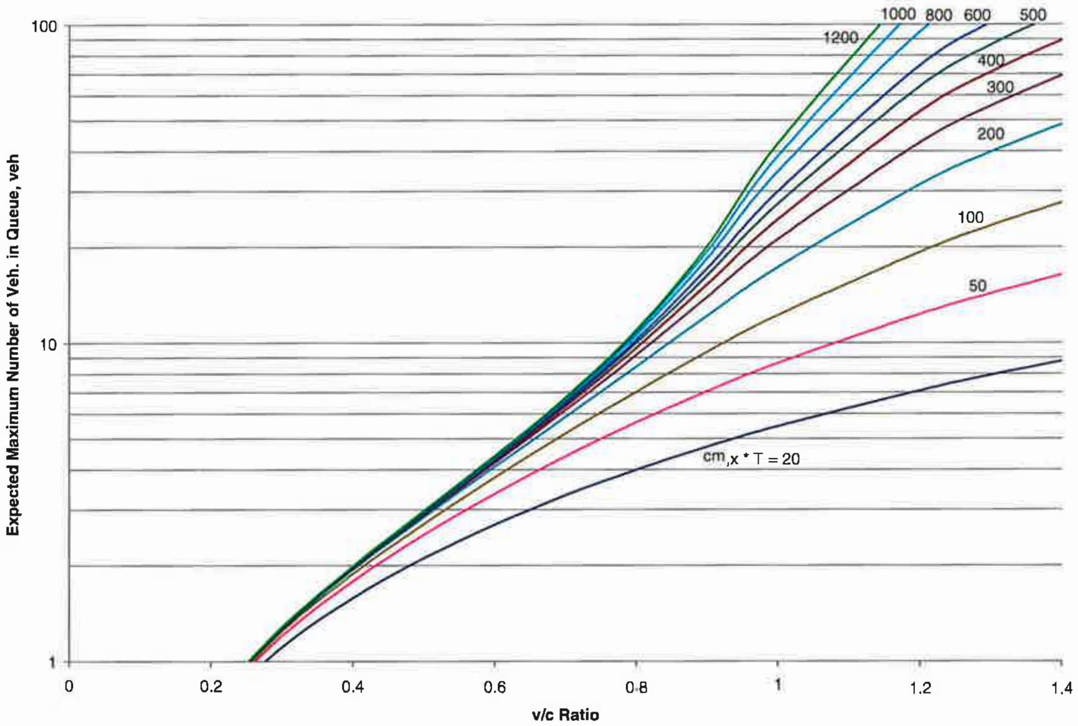


Figure 4.8 95th Percentile Queue Length

4.2.12 CONTROLLED DELAY

Total delay is the difference between the travel time actually experienced and the reference travel time that would result during base conditions, in the absence of incident, control traffic, or geometric delay. It includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. Controlled delay in field measurements is defined as the total elapsed time from the time a vehicle stops at the end of queue to the time the vehicle departs from the stop line. This total elapsed time includes the time required for the vehicle to travel from the last-in-queue position to the first-in-queue position, including deceleration of vehicles from free-flow speed to the speed of vehicles in queue.

Average controlled delay for any particular minor movement is a function of the capacity of the approach and the degree of saturation. Equation 4.10 is the analytical model used to estimate controlled delay. It assumes that the demand is less than capacity for the period of analysis. If the degree of saturation is greater than 0.9, average controlled delay is significantly affected by the length of the analysis period. Mostly, a 15 minutes period is recommended. During the 15 minutes period, if demand exceeds capacity, the delay calculated may not be accurate. In this case, the period of analysis should be extended to include the period of oversaturation.

$$D = \frac{3600}{C_{m,x}} + 900T\left[\frac{V_x}{C_{m,x}} - 1 + \sqrt{\left(\frac{V_x}{C_{m,x}} - 1\right)^2 + \frac{\left(\frac{V_x}{C_{m,x}}\right)\left(\frac{3600}{C_{m,x}}\right)}{450T}}\,\right] + 5 \qquad (4.10)$$

where

D	= controlled delay (sec/veh)
V_x	= movement volume (veh/hr)
$C_{m,x}$	= movement capacity (veh/hr)
T	= analysis time period in hours = 0.25

The constant value of 5 s/veh is added to the equation 4.10 to take into consideration the acceleration and deceleration of vehicles. This equation is graphically shown in Figure 4.9, based on a 15 minutes analysis and for a discrete range of capacities.

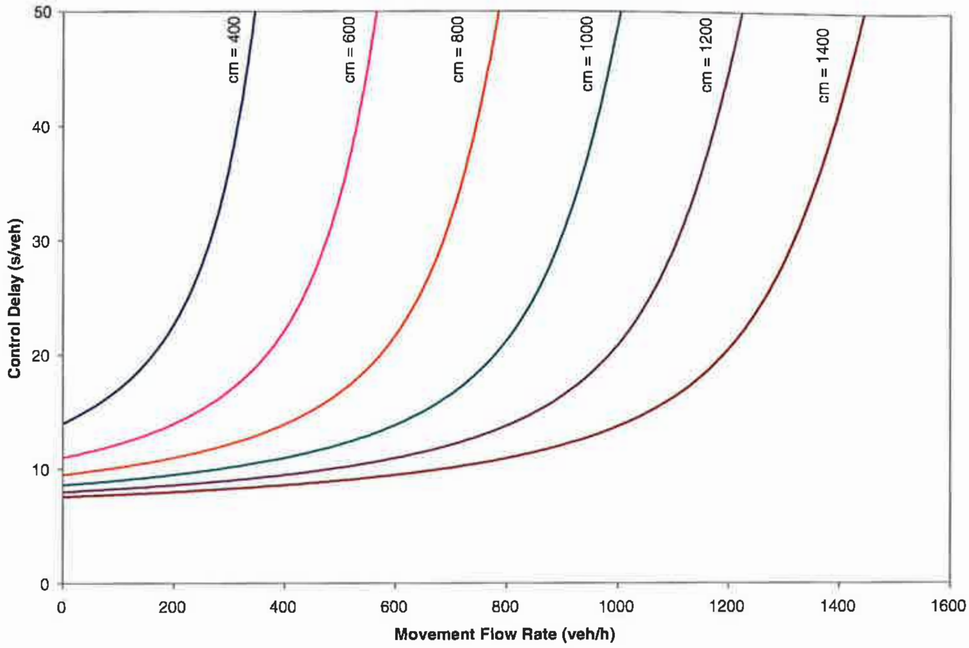


Figure 4.9 Controlled Delay and Flow Rate

4.3 APPLICATION

The analysis of TWSC intersections usually being used for existing intersection either to evaluate operational conditions or to estimate the effects under current traffic demands.

The procedure for analyzing unsignalised intersection is divided into three modules.

Initial Calculations
Worksheet 1 through 5 are used to record input conditions, compute the critical gap and follow-up time, and determine the flow patterns that result from any upstream signalised intersections that may affect the capacity of the subject intersection.

Capacity Calculations
Worksheet 6 through 9 are used to compute the capacity of each movement and make adjustments for the effects of two stage gap acceptance, shared lanes, or flared minor street approaches.

Delay and LOS Calculations
In this module, worksheets are used to calculate the delay, queue length and LOS for each approach.

4.3.1 SEQUENCE OF CAPACITY CALCULATIONS

Priority of gap use and movements are important to the computation of capacity. The computational sequence is as follows:

1- Left turns from the minor street
2- Right turns from the major street
3- Through movements
4- Right turns from the minor street

4.3.2 COMPUTATIONAL STEPS

The following steps describe each computations and summarized using a worksheets. According to TRB, 2000 there are several worksheets to be filled for the computational.

Geometrics and Movements (Worksheet 1)

The sketch shows designated movement numbers, v_1 through v_2 denoting major-street movements and v_7 through v12 denoting minor-street movements. Lane arrangement, grade, street particular and geometric data are entered in appropriate fields and columns.

Volume Adjustment (Worksheet 2)

Measured or forecast volumes (veh/hr) for each movement are used to compute hourly flow rates by dividing volume by PHF. Proportion of heavy vehicles (HV) is the percentage of heavy vehicles divided by 100 and is used to compute the critical gap and follow up time.

Site Characteristics (Worksheet 3)

Information about lanes and traffic movements is entered in this worksheet.

Critical Gap and Follow up Time (Worksheet 4)

Table 4.2, equation 4.1 and equation 4.2 are used to calculate the critical gap and follow-up time, which is used in equation 4.3 to determine potential capacities. The presence of motorcycle will take into consideration with some adjustment according to Table 4.3.

Impedance and Capacity Calculation (Worksheet 6)

The capacity for each movement is calculated using this worksheet. There will be some equations as shown in the worksheet for calculation convenience. Flow rates are entered to worksheets 1 and 2.

Calculation must be made in order according to the turning priority, considering first the left turns from minor street, followed by right turns from the major street, etc.

Shared Lane Capacity (Worksheet 8)

Equation 4.7 is used to compute shared lane capacity in this worksheet.

Controlled Delay, Queue Length, Level of Service (Worksheet 10)

Worksheet 10 is used to compute control delay, average queue length, and level of service. Controlled delay for each movement can be estimated from Figure 4.11 and equation 4.10. The 95^{th} percentile queue can be estimated from equation 4.9 and LOS can be determined from Table 4.1.

4.3.3 PLANNING AND DESIGN APPLICATIONS

In order to plan, analyst requires geometric and traffic flow data, and base values of critical gap and follow up time are used (refer Table 4.2). Because the effect of upstream signals, two-stage gap acceptance and flared left-turn approaches are normally not take into consideration in planning analysis, Malaysian HCM exclude the methodology of these part.

Planning stage uses the similar worksheets with some exceptions as noted below:

- Worksheet 1 is used to describe basic conditions.
- Worksheet 2 is used to summarize the vehicles volumes.
- Worksheet 3 is used to note the lane designation for each movement.
- Worksheet 4 in planning analysis usually unused, as the base values are used without adjustment.
- Worksheet 5 is for analyzing upstream signals, therefore it is not applicable for planning analysis.
- Worksheet 6 is used to compute the movement capacities.
- Worksheet 7 is used to include the effects of two-stage gap acceptance when there is a divided roadway.
- Worksheet 8 is used to compute shared-lane capacities.
- Worksheet 9 is unused, since the effect of compute flared minor-street approaches is not applicable.
- Worksheet 10 is not used because the impedance and delay for major through movements are not accounted for in planning analysis.
- Worksheet 11 is used to compute capacity, delay and LOS for rank 1 vehicles.

Malaysian Highway Capacity Manual only provides the essential worksheet, and suitable to the condition. These worksheets originally taken from US HCM 2000, with some adjustment to take into consideration of Malaysia road condition.

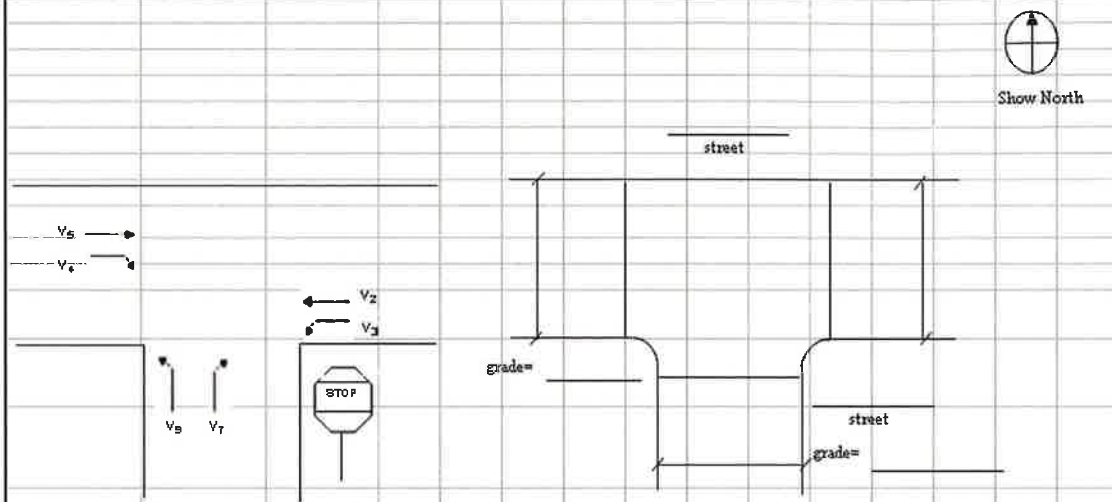
4.4 WORKSHEET

TWSC - UNSIGNALIZED INTERSECTIONS WORKSHEET

Worksheet 1

General Information		**Site Information**	
Analyst	__________	Intersection	__________
Agency or Company	__________	Jurisdiction	__________
Date Performed	__________	Analysis Year	__________
Analysis Time Period	__________		

Geometrics and Movements



Length of study period = __________ hr

Worksheet 2

Vehicle Volumes and Adjustments

	Vehicle Volumes and Adjustments											
Movement	1	2	3	4	5	6	7	8	9	10	11	12
Volume (veh/hr)												
Peak-hour factor,PHF												
Hourly flow rate (veh/hr)												
Proportion of motorcycles, P_m												

s_5	protected RT						
	TH						

Computing Delay to Major-Street Vehicles

Data for Computing Effect of Delay to Major-Street Vehicles	s_2 Approach	s_5 Approach
Shared-lane volume, major-street through vehicles, v_{i1}, blocked by RT		
Shared-lane volume, major-street right-turn vehicles, v_{i1}, blocked by RT		
Saturation flow rate, major-street through vehicles, s_{i1}		
Saturation flow rate, major-street right-turn vehicles, s_{i2}		
Number of major-street through lanes		
Length of study period, T(h)		

TWSC - UNSIGNALISED INTERSECTIONS WORKSHEET								
Worksheet 4								
General Information								
Project Description								
Critical Gap and Follow-Up Time								

$$t_{c,x} = t_{c,base} - t_{c,M} P_{M,x}$$

		Major RT		Minor LT		Minor TH		Minor RT	
Movement		1	4	9	12	8	11	7	10
$t_{c,base}$ (Table 4.2)									
$t_{c,M}$									
P_M (from Worksheet 2)									
t_c									

$$t_{f,x} = t_{f,base} - t_{f,M} P_M$$

		Major RT		Minor LT		Minor TH		Minor RT	
Movement		1	4	9	12	8	11	7	10
$t_{f,base}$ (Table 4.2)									
$t_{f,M}$									
P_M (from Worksheet 2)									
t_f									

TWSC - UNSIGNALISED INTERSECTIONS WORKSHEET	
Worksheet 6	
General Information	
Project Description	
Impedance and Capacity Calculation	
Step 1: LT from Minor Street	v_9
Conflicting flows (Figure 4.3)	$v_{c,9}$ =
Adjustment factor for capacity (Table 4.4)	A_9 =
Potential capacity (Equation 4.3)	$c_{mp,9}$ =
Movement capacity	$c_{mm,9} = c_{mp,9}$ =
Prob of queue-free state (Equation 4.8)	$p_{0,9}$ =
Step 2: RT from Major Street	v_4
Conflicting flows (Figure 4.3)	$v_{c,4}$ =
Adjustment factor for capacity (Table 4.4)	A_4 =
Potential capacity (Equation 4.3)	$c_{mp,4}$ =
Movement capacity	$c_{mm,4} = c_{mp,4}$ =
Prob of queue-free state (Equation 4.8)	$p_{0,4}$ =
Step 3: RT from Minor Street (T-intersections only)	v_7
Conflicting flows (Figure 4.3)	$v_{c,7}$ =
Adjustment factor for capacity (Table 4.4)	A_7 =
Potential capacity (Equation 4.3)	$c_{mp,7}$ =
Capacity adjustment factor due to impeding movement	$f_7 = P_{0,4}P_{0,1}$ =
Movement capacity (Equation 17-7)	$c_{mm,7} = c_{mp,7}\ f_7$ =

TWSC - UNSIGNALISED INTERSECTIONS WORKSHEET							
Worksheet 10							
General Information							
Project Description							
Control Delay, Queue Length, Level of Service							
Lane	v (veh/h)	cm (veh/h)	v/c	Queue Length	Control Delay	LOS	Delay and LOS
				(Equation 17-37)	(Equation 17-38)	(Exhibit 17-2)	
1 7 8 9							
2 7 8 9							
3 7 8 9							
Movement	v (veh/h)	c_m (veh/h)	v/c	Queue Length	Control Delay	LOS	
				(Equation 17-37)	(Equation 17-38)	(Exhibit 17-2)	
4							

4.5 SAMPLE CALCULATION

Example 1

UNSIGNALIZED INTERSECTION

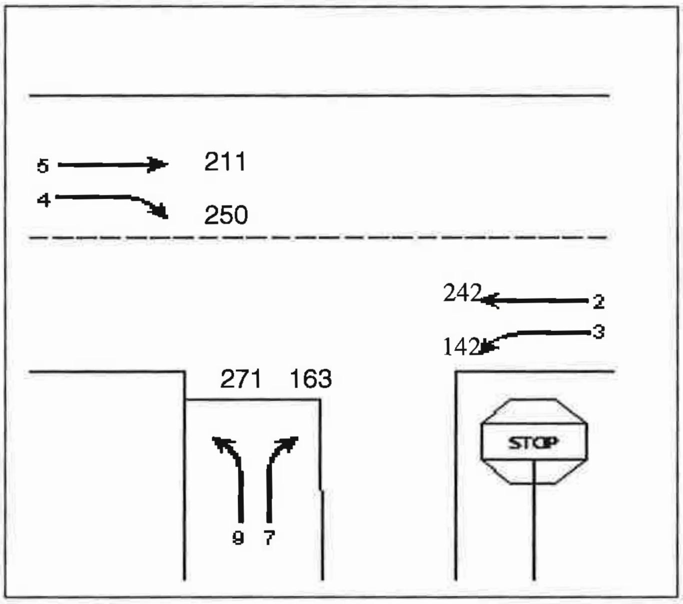
General Information

Analyst ______
Agency or Company Universiti Sains Malaysia
Date Performed ______
Analysis Time Period ______

Site Information

Intersection Jln Suka Menanti-Jln Gunung Keriang
Jurisdiction Majlis Bandaraya Kota Star
Analysis Year 2005

Geometrics and Movements



Vehicle Volumes and Adjustments

Movement	1	2	3	4	5	6	7	8	9	10	11	12
Volume (veh/hr)		242	142	250	211		163		271			
Peak-hour factor, PHF		1.0	1.0	1.0	1.0		1.0		1.0			
Hourly flow rate (veh/hr)		242	142	250	211		163		271			
Proportion of motorcycles, P_M				0.38			0.28		0.40			

Critical Gap and Follow-up Time

$$t_{c,x} = t_{c,base} - \alpha_{c,M} P_M$$

Movement	7	9	4
$t_{c,base}$	4.0	3.2	3.5
$t_{c,M}$	0.424	0.424	0.424
P_M	0.28	0.40	0.38
t_c	2.94	2.28	2.79

$$t_{f,x} = t_{f,base} - \alpha_{f,M} P_M$$

Movement	7	9	4
$t_{f,base}$	2.2	1.9	2.0
$t_{f,M}$	0.738	0.738	0.738
P_M	0.28	0.40	0.38
t_f	1.99	1.60	1.72

Impedance and Capacity Calculation

LT from Minor Street	v_9
Conflicting flows	$v_{c,9} = 313$
Adjustment factor for capacity	$A_9 = 0.4846$
Potential capacity	$c_{mp,9} = 895$
Movement capacity	$c_{mm,9} = c_{mp,9} = 895$
Prob. of queue-free state	$P_{O,9} = 0.6972$ (tak pakai pon)

RT from Major Street	v_4
Conflicting flows	$v_{c,4} = 384$
Adjustment factor for capacity	$A_4 = 1.0000$
Potential capacity	$c_{mp,4} = 1538$
Movement capacity	$c_{mm,4} = c_{mp,4} = 1538$
Prob. of queue-free state	$P_{O,4} = 0.8374$

RT from Minor Street	v_7
Conflicting flows	$v_{c,7} = 313$
Adjustment factor for capacity	$A_7 = 0.4375$
Potential capacity	$c_{mp,7} = 614$
Capacity adjustment factor due to impeding movement (shared lane use P*)	$f_7 = P_{O,4} P_{O,1} = 0.8374$
Movement capacity	$c_{mm,7} = c_{mp,7} f_7 = 514$

Shared-Lane Capacity

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}} \right)}$$

Movement	v(ve h/h)	c_{mm} (veh/h)	c_{SH} (veh/h)
7	163	514	700
9	271	895	

Control Delay and Level of Service

Movement	v (veh/h)	c_{mm} (veh/h)	c_{SH}	Control Delay (SH)	Control Delay	LOS(SH)	LOS
7	163	514	700	18	15	B	B
9	271	895			11		B
4	250	1538			8		A

CHAPTER 5

URBAN AND SUBURBAN ARTERIALS

CHAPTER 5

5.0 URBAN AND SUBURBAN ARTERIALS

CONTENTS

FIGURES

TABLES

5.1 INTRODUCTION

Suburban Arterials are signalised streets that primarily serve through traffic and provide access to abutting properties as a secondary function. Arterial is defined as a road that consist of several traffic light junctions and the distance between the two junctions is usually 3 km or less and the main function of the arterial is to provide through movement while the second main purpose of the road is for access to adjacent road (TRB,2000).

Basically, the arterial will be classified by functional category and design category. The functional category must be a principal arterial or a minor arterial. A principal arterial serves major through movements between important centres of activities in a metropolitan area. A minor arterial is a facility that connects the principal arterial system and serves trips of limited length and services smaller geographical areas. The design category are divided into three parts; typical suburban design, intermediate design and typical urban design

From previous study, indicates that Malaysia has its own classification of road arterials. But in the hierarchy of urban highway transportation facilities adapted from the US HCM, arterial streets are ranked between collector and downtown streets and multilane suburban highways and rural roads. The differences are mainly determined by their function and the character and intensity of roadside development.


Figure 5.1 Urban and Suburban Arterial Roads

5.1.1 APPLICATIONS

In this chapter, the manual described and gave the proper approached to evaluate the level of service of an existing or proposed facility, especially for those who are concerns with the planning, design and operation of arterials.

In general, user can best conduct an arterial capacity analysis by analysing the capacity of signalised intersections and other such points, although the methodology of urban streets does not address arterial capacity. It is important to make capacity analysis in signalised intersections because when demand volume exceeds the capacity at any point along the arterial, the arterial evaluation methodology based on average travel speed becomes unsuitable.

The methodology is oriented toward the evaluation of an existing operations situation or a specific design proposal by a level-of service determination. Investigation should cover the effect of signal phasing, arterial classifications and traffic flow. The methodology uses signalised intersection procedure in Chapter 3 for the lane group containing the through-traffic. By redefining lane arrangement, the analyst may influence which traffic flow is in the through-traffic lane group and the capacity of the lane group. This redefinition in turn, influences the arterial level-of-service determination by changing the intersection evaluation and possibly the arterial classification.

The arterial methodology may be used in a straightforward manner for those who are interested in planning but somewhat in simplified way by computing stopped delay using certain values as outlined in the Signalised Intersections chapter. Signal timing and quality of progression is very important in order to get meaningful estimation of arterial level-of-service, even during the planning stage.

Level-of-service criteria can be applied when travel time and delay runs are used to assess the impact of optimizing signal timing or other improvements to the arterial and to periodically evaluate the entire arterial system in an urban area. Alternative arterial traffic models also can estimate arterial level of service, provided that

1. Input parameter such as running speeds and saturation flow rates are determined in a manner consistent with the procedures in this manual.
2. The delay calculated or estimated by the model is defined consistent with definition in this manual (i.e., average stopped delay per vehicle)
3. The delay outputs from the model are based on the delay equations in this manual or have been validated with field data.

The above applications of the methodology always require the determination of level of service and associated measures of effectiveness (e.g., travel time, delay, speed).

5.1.2 CHARACTERISTICS OF ARTERIAL FLOW

The operation of the vehicles on arterial streets is influenced by three main factors:

- The arterial environment
- The interaction among vehicles
- Effect of traffic signals

These factors contribute to the capacity of an arterial street and quality of service offered to its users. The arterial environment includes the geometric characteristics of the facility and adjacent land uses. The number of lanes, type of median, spacing between signalised intersections are among the environmental factors, as well as the existence of parking, level of pedestrian activity, speed limit and population of the city. The arterial environment affects a driver notion of safe speed.

The interaction among vehicles is determined by traffic density, the proportion of lorries and buses, and the turning movements. This interaction affects the operation of vehicles at intersection and to a lesser extent, between signals. Most of the time, the presence of other vehicles restricts the speed of a vehicle in motion because of the differences in desired speeds among drivers. Therefore, the average of a vehicle in motion over a certain length of road, or a running speed, is usually lower than the desired speed of its driver because of the effect of vehicle intersections. Traffic signals forced vehicles to stop and to remain stopped for a certain time and then release them in platoons. The delay and speed changes caused by traffic signal operation considerably reduce the quality of traffic flow on arterial streets.

5.1.3 ARTERIAL LEVEL OF SERVICE

Arterial level of service is based on average through-vehicle travel speed for the segment, section, or entire arterial under consideration. The average travel speed is computed from running time on the arterial segment or segments and the intersection total delay for through movements at all intersections. To ensure that the arterial is of sufficient length so that average travel speed is a reasonable measure of effectiveness (MOE), its length generally should be at least 1,609 km in downtown areas and at least 3,218 km in other cases. Arterial level of service is defined in terms of average travel speed of all through vehicles on the arterial. It is strongly influenced by the number of signals per kilometre and the average intersection delay.

The following general statement may be made regarding arterial level of service (LOS):

LOS A

- Free-flow operations at average travel speeds, usually about 90% of the free flow speed for the arterial classification.
- Vehicles are completely unimpeded in their ability to manoeuvres within the traffic stream.
- Stopped delay at signalized intersections is minimal.

LOS B

- Reasonably unimpeded operations at average travel speed, usually about 70% of the free flow speed for the arterial classification.
- The ability to manoeuvre within the traffic stream is only slightly restricted and stopped delays are not bothersome.
- Drivers are not generally subjected to appreciable tension

LOS C

- Stable operations; however, ability to manoeuvre and change lanes in midblock locations may be more restricted than at LOS B.
- Longer queues, adverse signal coordination, or both may contribute to lower average travel speeds of about 50% of the average free flow speed for the arterial classification.
- Motorists will experience appreciable tension while driving.

LOS D

- Borders on a range in which small increases in flow may cause substantial in delay and hence decreases in arterial speed.
- May be due to adverse signal progression, inappropriate signal timing, high volumes, or some combination of these factors.
- Average travel speeds are about 40% of free flow speed.

LOS E

- Characterized by significant delays and average travel speeds of 1/3 of the free flow speed or less.

- Such operations are caused by some combination of adverse progression, high signal density, high volumes, extensive delays at critical intersections, and inappropriate signal timing.

LOS F

- Characterizes arterial flow at extremely low speeds below 1/3 to 1/4 of the free flow speed.
- Intersection congestion is likely at critical signalized locations, with high delays and extensive queuing.
- Adverse progression is frequently a contributor to this condition.

Table 5.1 contains the arterial level of service definitions, which are based on average travel speed over the arterial segment being considered. Intersection volume to capacity ratios greater than 1.0 will probably result in an unacceptable level of service on the arterial.

Table 5.1 Arterial Classifications and Level Of Service

Arterial Classifications	I	II	III	IV
Range of Free-Flow Speed (km/hr)	90 to 70 km/hr	70-55 km/hr	55 to 50 km/hr	55 to 40 km/hr
Typical Free Flow Speed	80 km/hr	65 km/hr	55 km/hr	45 km/hr
LOS	Average travel speed (km/hr)			
A	>72	>59	> 50	> 41
B	> 56-72	> 46-59	> 39-50	> 32-41
C	> 40-56	> 33-46	> 28-39	> 23-32
D	>32-40	> 26-33	> 22-28	> 18-23
E	>26-32	> 21-26	> 17-22	> 14-18
F	≤ 26	≤ 21	≤ 17	≤14

Source: US HCM 2000

5.2 METHODOLOGY

This methodology provides the framework for arterial evaluation. If field data available, this framework can be used to determine the level of service of a given arterial without references to running time and intersection delay estimates. Rather than considering field evaluation to be a lesser methods, the transport analyst is using the speed-flow relationship for transportation planning analysis. From the speed-flow relationship we can easily find out the speed of the arterial and if the speed of the arterial is known then it is easy to determine the LOS of the arterial. On two way arterials, the methodology should be applied twice if it is desired to assess the level of service in each direction.

The procedure to determine arterial level-of-service (LOS) involves six steps, as shown in Figure 5.2.

4. Establish the location and length of the arterial to be considered.
5. Determine the arterial classification using the classification scheme presented here in conjunction with the measurement of free-flow speed.
6. Divide the arterial into sections for the purpose of the evaluation, where each section contains one or more arterial segments.
7. Compute the running time.
8. Compute the traffic flow and average travel speed of the vehicles (by section and entire facility).
9. Establish the Speed-flow relationship.
10. Determine the level of service (LOS) by referring to Table 5.1

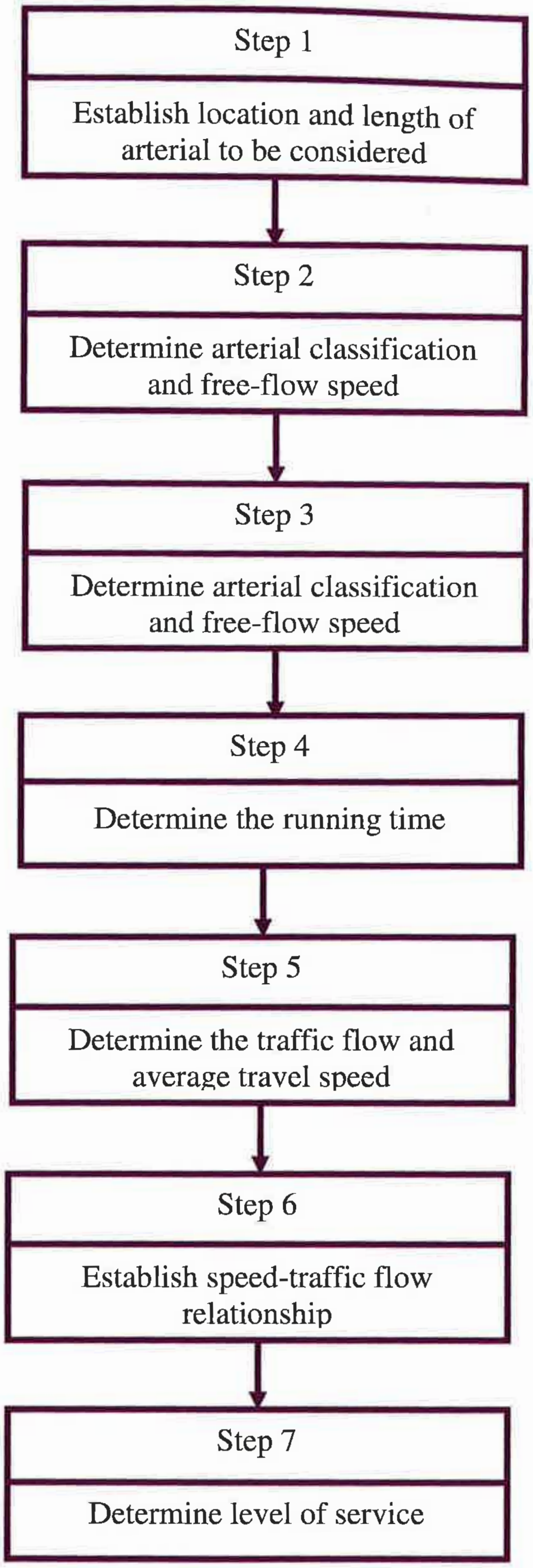


Figure 5.2 Arterial LOS Methodologies

Step 1 – Establish location and length of arterial to be considered

At the start of the analysis, it is useful to define the location and length of the arterial to be considered and identify all relevant physical, signalised and traffic data. Consideration should be given to whether the extent of the arterial being analysed is sufficient (at least 1.609 km long in downtown areas and at least 3.218 long in other areas), or whether additional sections should be considered.

Step 2 – Determine arterial classification and free-flow speed

The free flow speed (FFS) observation is a part of Urban and Suburban Arterials study. Free flow speed was the average speed of motorist over those portions of arterial sections that were not close to signalised intersections, as observed during very low traffic volume conditions (200 veh/h/ln) while driver were not constrained by other vehicles or by traffic signals (US HCM 1994). Free flow speed represents the average desired speed at which drivers would like to travel. Therefore, off-peak hours were generally is a good time to observe free-flow condition. The free-flow speed is relatively easy to estimate in the field and generally lies between the speed limit and the design speed of the road.

First, the free-flow speed can be categorized as a spot speed. A spot speed is the instantaneous measure of speed at specific location on a roadway. A spot speed study is carried out by recording the speeds of a sample of vehicles at specific location in the arterial stream. The spot speed data are collect by direct or indirect measurements (Robertson et. al, 1994). Free-flow speed data for each study location should be analysed. The entire data are group into speed-class intervals to avoid excessive computations at later stage. A frequency distribution curve is drawn by the spot speed data. From the graph, the value of mean speed, median speed and 85^{th} percentile speed are calculated. The mean speed value is chosen for free-flow speed. The 85^{th} percentile speed is commonly used as the speed limit on the road. Table 5.2 and Table 5.3 show the arterial classification according to various criteria.

Table 5.2 Aid in Establishing Arterial Classification

Criteria	Design Category			
	Suburban		Urban	
Location	Between city center and expressway		City center with high business activity	
Number of Driveways	More than 10	Less than 10	More than 10	Less than 10
Turning movements	High	Low	High	Low
Number of signals	> 2	< 2	> 2	< 2
Range of free-flow speed	55 to 80 kmph		30 to 55 kmph	
Roadside development	High	Low	High	Low

Source: US HCM 1994

Table 5.3 Arterial Classification According To Their Design Category

Design Category	Arterial
Typical suburban	I
Typical urban	II

Source: US HCM 1994

Step 3 – Divide arterial into sections

The basic unit of arterial is the segment, which is the one-directional distance from one signalised intersection to the next. If two or more consecutive segments are comparable in arterial classification, segment length, speed limit and general land used and activity, the analyst may wish to aggregate the arterial into sections. All results should be focused on the section rather than on smaller components, if the segments are aggregated into a section.

Step 4 - Compute arterial running time

In this procedure, a test vehicle travels with the existing traffic stream to take time measurements based on the flow of normal traffic. The vehicle moves with the traffic along the corridor. The starting time, ending time and total stop time of the vehicle are recorded in the field. The running time is calculated by journey time minus total stop time.

Step 5 - Compute the traffic flow and average travel speed of the vehicles

The traffic volume is computed by manual traffic counting and video camera. The classified traffic volume in both directions is recorded in 15 minutes intervals for the segments of each arterial road. The observation of travel time and traffic volume must be done simultaneously. The numbers of vehicles counted were then converted to Passenger Car Unit (PCU) using MHCM. This is due to the fact that vehicles in arterial roads were influenced by the traffic signal. The arterial road can also be divided into section or zone. The time taken by the vehicle travelling from one zone to another were recorded in order to obtain the flow for every zone. The distance of every zone is then recorded and the average travel speed value for every zone can be estimated using equation 5.1.

$$S = 3600\frac{L}{T} \qquad (5.1)$$

where

S = Average travel speed in km/h,

L = Length of study route or section in km

T = Travel time in seconds.

For current analysis of existing facility, in order to calculate the running time, flow rate per lane of each segment must be determined. From the speed-flow curve, determine the average speed. Using equation 5.1, the running time can be estimated.

Step 6 – Establish Speed-Flow relationship

There has been a lot of works being carried out to establish the speed flow relationship within the past years (Hall, 1994). Researchers in many countries have investigated the relationship between traffic speed and flow in the last 60 years. Various speed-flow relationship have been developed such as 1994 HCM Speed-Flow curve, 1985 HCM Speed-Flow curve, MTC (Metropolitan Transportation Commission Bay Area) Curve and Akcelik Speed-Flow function, but they are mainly for uninterrupted flow streams. Not much can be found in the literature regarding speed-flow relationship of interrupted flow such that on an arterial road (Lum et al., 1998). This study has been carried out in Singapore which then became a major reference to the study of establishing speed flow relationship especially in Malaysia.

Nowadays, an economical analyst will use the speed flow relationship for transportation planning analysis. The speed-flow relationship curve has been used for highway capacity analysis. This curve has been used for design and planning analyses. These two variables are the traffic stream characteristics most often measured and have been traditionally use in the assessment of traffic operations (HCM 1994). Here the speed-flow relationship for both interrupted and uninterrupted flow will be discussed. Nowadays, the speed-flow relationship can be used for road pricing system analysis and traffic assignment.

The pioneer researcher in the speed flow relationships is Greenshields. He had developed speed-flow models based on field observation at Ohio two-lane roadway (Greenshields, 1935).Greenberg model has been developed in 1959 by using the fluid dynamic theory (Greenberg, 1959). The traffic data were collected at Linclon Tunnel. Lastly, the remaining two models were developed by Underwood (1961) and Drake (1967).

The following equations as shown in Table 5.4 have been derived from field data of different urban and suburban arterial roads in Malaysia.

Table 5.4 Speed-Traffic Flow Relationships In Urban and Suburban Arterial

Arterial Cases	Conditions	Equation
Urban	Low side friction, traffic light >2	$Q = 150.5 v \ln(\frac{30.7}{v})$
	Low side friction, traffic light <2	$Q = 47.90 v \ln(\frac{62.4}{v})$
	High side friction, traffic light >2	$Q = 54.20 v \ln(\frac{41.6}{v})$
	High side friction, traffic light <2	$Q = 88.8 v \ln(\frac{49.8}{v})$
Suburban	Low side friction, traffic light <2	$Q = 81.10 v \ln(\frac{54.3}{v})$
	High side friction, traffic light >2	$Q = 76.90 v \ln(\frac{54.2}{v})$
	High side friction, traffic light <2	$Q = 43.40 v \ln(\frac{74.6}{v})$

Figure 5.3 to Figure 5.9 show the speed-flow relationship of different urban and suburban arterial roads according to side frictions, traffic light and number of driveways. The term low side friction represents the number of driveway that is less than 10 with low turning movement and located at a suburban area. As for high side friction, the number of driveways is more than 10 with high turning movement and located in a business center. Table 5.5 shows the summary regarding the following graphs.

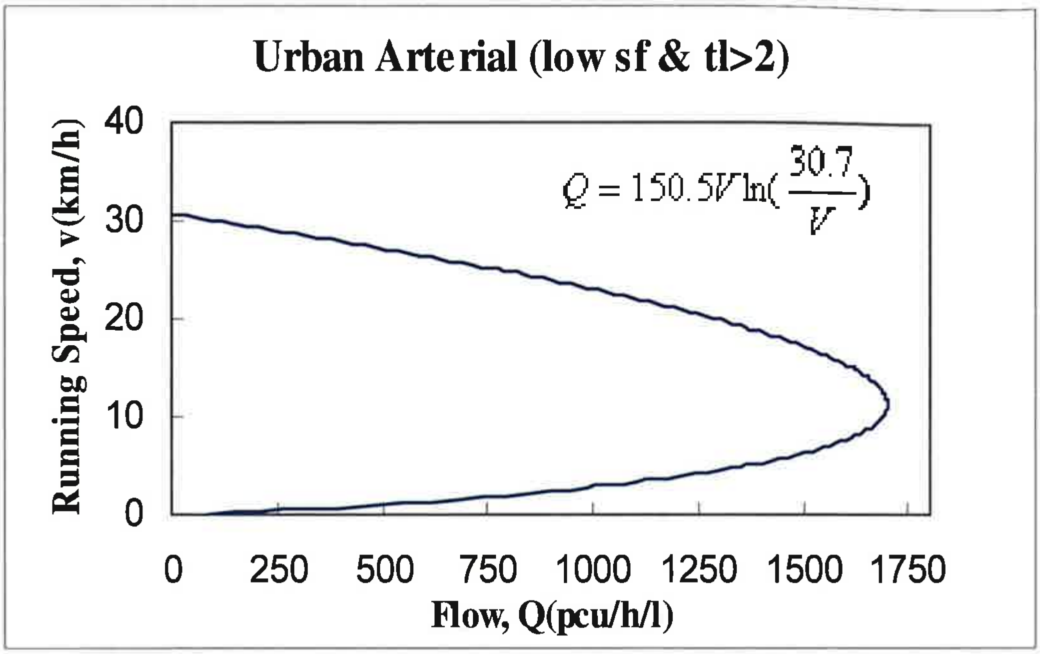


Figure 5.3 Urban Arterial of Low Side Friction With Traffic Light More Than Two

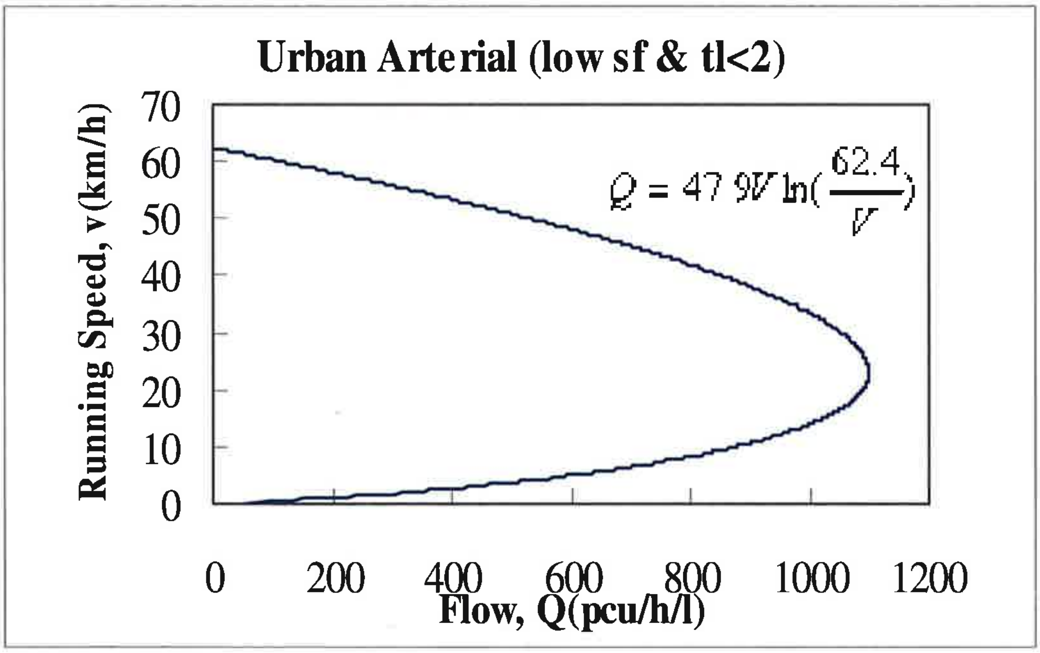


Figure 5.4 Urban Arterial of Low Side Friction With Traffic Light Less Than Two

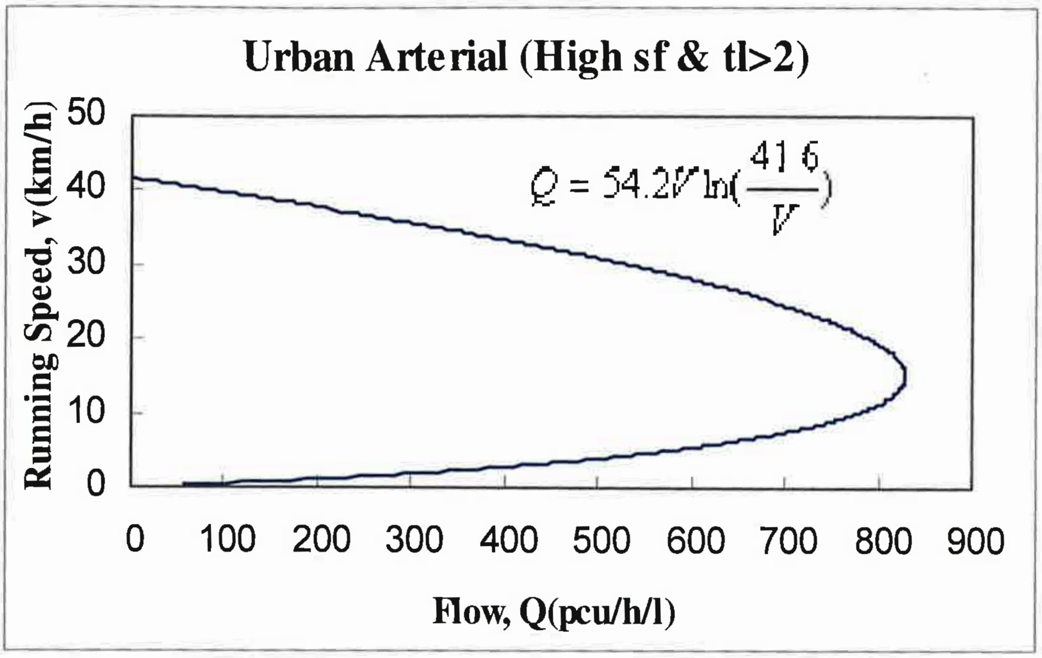


Figure 5.5 Urban Arterial of High Side Friction With Traffic Light More Than Two

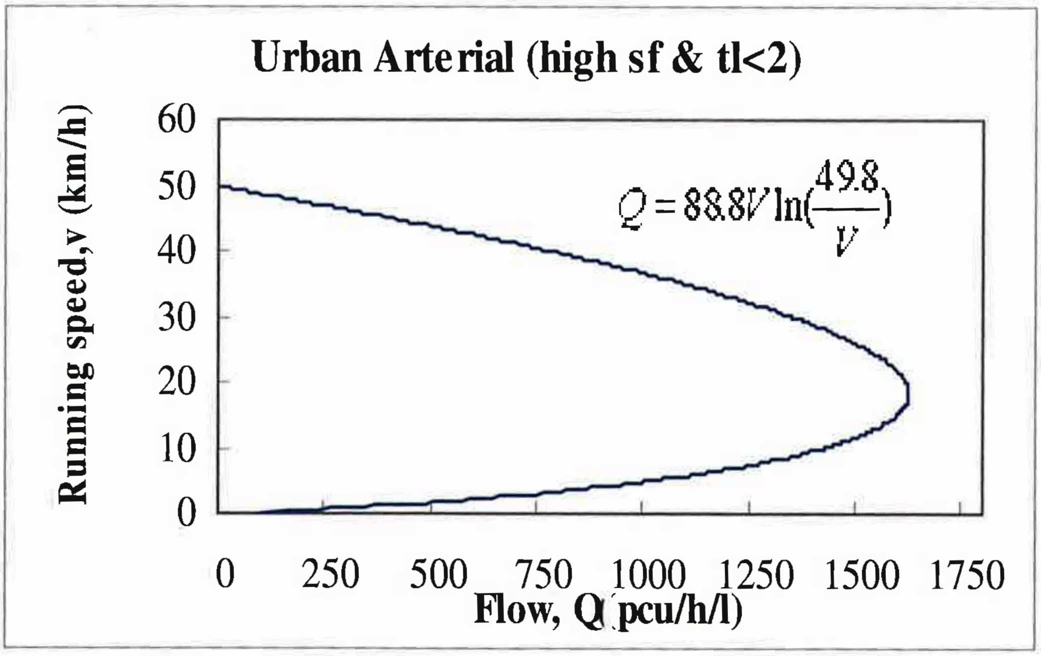


Figure 5.6 Urban Arterial of High Side Friction With Traffic Light Less Than Two

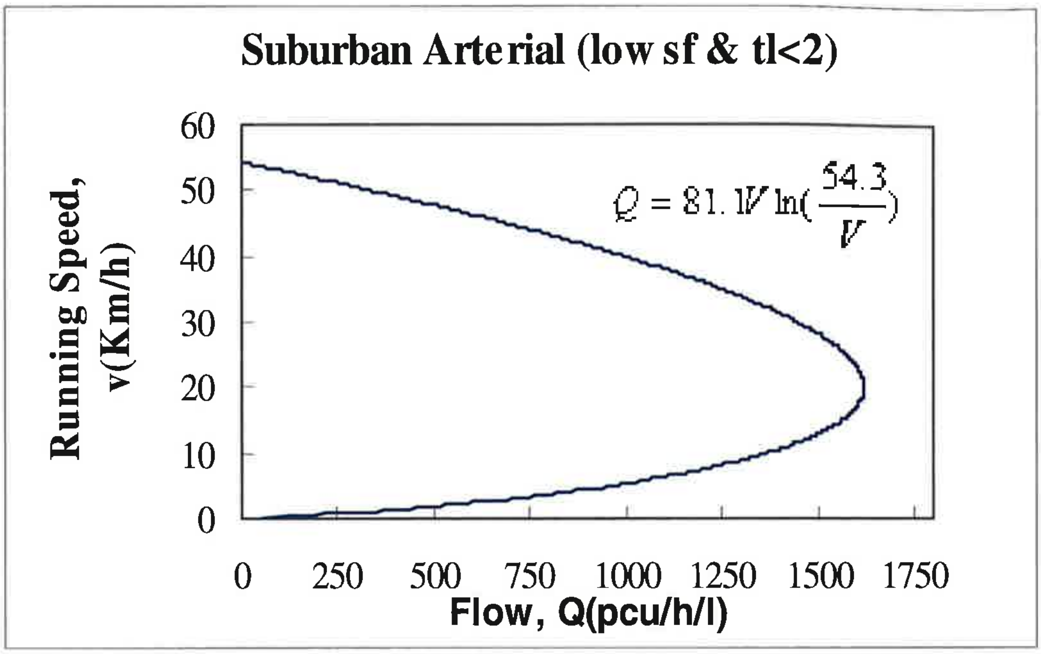


Figure 5.7 Suburban Arterial of Low Side Friction With Traffic Light Less Than Two

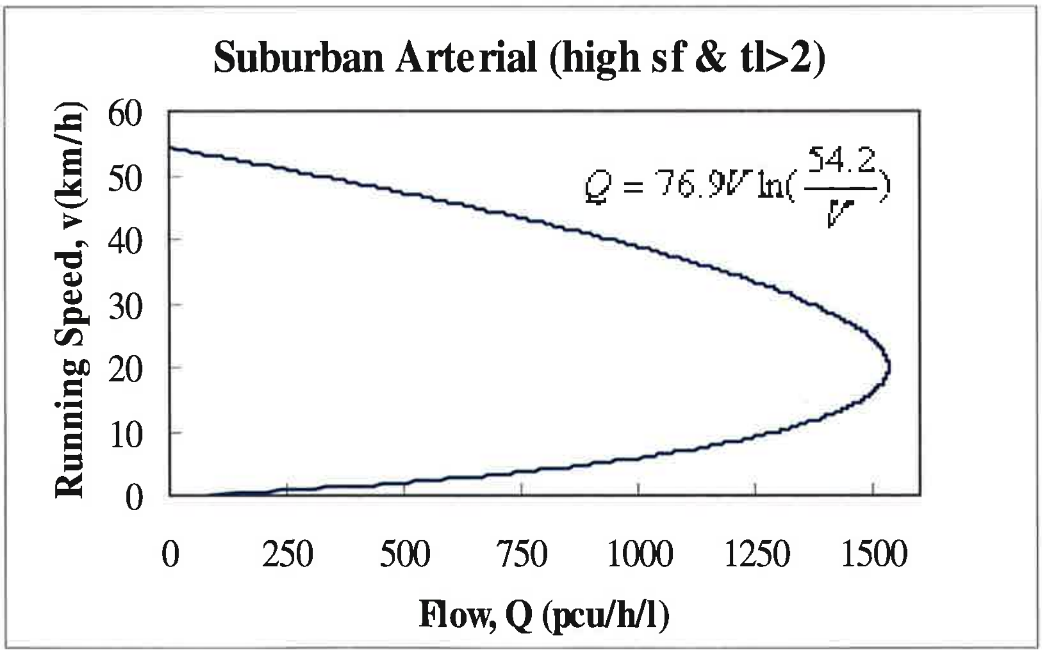


Figure 5.8 Suburban Arterial of High Side Friction With Traffic Light More Than Two

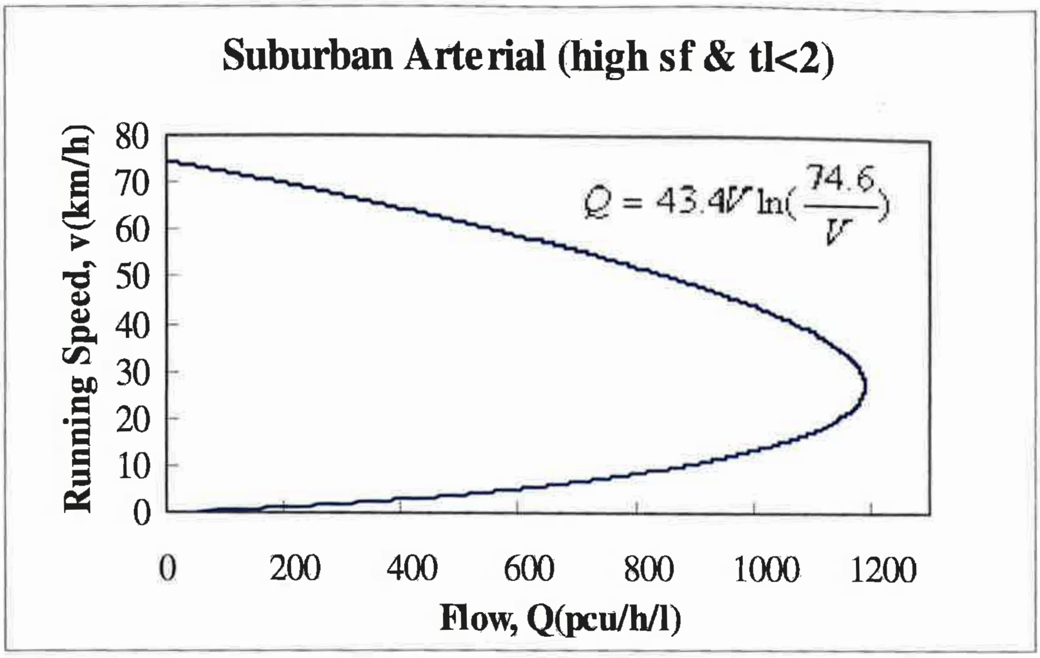


Figure 5.9 Suburban Arterial of High Side Friction With Traffic Light Less Than Two

Table 5.5 Summary Of Urban And Suburban Arterial Running Speed vs. Flow Equation

URBAN

High side friction Traffic light<2		High side friction Traffic light>2		Low side friction Traffic light>2		Low side friction Traffic light<2	
V(kmph)	Q(pcuphpl)	V(kmph)	Q(pcuphpl)	V(kmph)	Q(pcuphpl)	V(kmph)	Q(pcuphpl)
5	1020.56	5	574.16	5	1365.65	5	604.53
10	1425.62	10	772.63	10	1688.12	10	877.04
15	1598.35	15	829.30	15	1616.85	15	1024.23
20	1620.21	20	793.89	20	1289.88	20	1090.04
25	1529.89	25	689.99	25	772.77	25	1095.34
30	1350.16	30	531.54	30	104.14	30	1052.41
35	1096.09	35	327.71			35	969.38
40	778.37	40	85.03			40	852.02
45	405.00					45	704.64
50	--					50	530.59
						55	332.56
						60	112.72

SUBURBAN

High side friction Traffic light<2		High side friction Traffic light>2		Low side friction Traffic light<2	
V(kmph)	Q(pcuphpl)	V(kmph)	Q(pcuphpl)	V(kmph)	Q(pcuphpl)
5	586.49	5	916.36	5	967.15
10	872.15	10	1299.68	10	1372.16
15	1044.26	15	1481.82	15	1564.99
20	1142.64	20	1533.31	20	1620.04
25	1186.20	25	1487.64	25	1572.63
30	1186.05	30	1364.55	30	1443.56
35	1149.57	35	1177.08	35	1246.60
40	1081.98	40	934.49	40	991.51
45	987.20	45	643.72	45	685.60
50	868.25	50	310.13	50	334.54
55	727.57				
60	567.14				
65	388.60				
70	193.35				

V: running speed in kmph, Q: Flow in pcuphpl

Step 7 – Determine the Level of Service

A distinct set of arterial LOS criteria has been established for each arterial classification. In the definition of level of service, both the free-flow speed of the arterial classification and intersection LOS definition were taken into account. In general, the arterial LOS criteria are based on the smooth and efficient movement of through traffic along an entire arterial. Therefore, it is necessary to expect less delay per segment than for the corresponding intersection level of service. Table 5.2 gives the arterial LOS criteria for each of the two arterial classifications. The lower the arterial classification, the lower the drivers expectation while driving on that facility and the lower the speed associated with a given level of service. It is also noted that the concept of an overall arterial level of service is generally meaningful only when all segments on the arterial are of the same classification. If different arterial classifications are represented, the LOS criteria are different.

5.2.1 DELAY

Intersection controlled delays are required to compute the urban or section speed. And the lane group for through traffic is used to characterize the urban arterial. Based from US HCM 2000, the controlled delay is the best way to be use in urban street evaluation. Equation 5.2 can be used to determine control delay.

$$d = d_1(PF) + d_2 + d_3 \tag{5.2}$$

And equation 5.3 and equation 5.4 are the equation used to determine uniform delay and incremental delay, respectively.

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]} \tag{5.3}$$

$$d_2 = 900T\left[(x-1)+\sqrt{(x-1)^2+\frac{8klX}{cT}}\right] \tag{5.4}$$

where

d	= controlled delay (s/veh)
d_1	= uniform delay (s/veh)
d_2	= incremental delay (s/veh)
d_3	= initial queue delay (s/veh)
PF	= progression adjustment factor
X	= volume to capacity (v/c) ratio for the lane group
C	= cycle length (s)
c	= capacity of lane group (veh/h)
g	= effective green time for lane group (s)
T	= duration of analysis period (h)

k = incremental delay adjustment for the actuated control

I = incremental delay adjustment for the filtering or metering upstream signals

Uniform Delay

Equation 5.3 gives an estimate of controlled delay assuming a perfect uniform arrivals and a stable flow.

Incremental Delay

Equation 5.4 estimates the incremental delay caused by the non-uniform arrivals

Initial Queue Delay

When a queue from the previous period is present at the start of the analysis, newly arriving vehicles experience initial queue delay. This delay results from the additional time required to clear the initial queue.

Arrival Type and Platoon Ratio

Arrival type for urban arterials is a critical characteristic that must be quantified. This parameter approximates the quality of progression by defining six types of dominant arrival flow. These arrival types are previously defined in signalised intersections chapter, which are originally adopted from TRB 2000.

Arrival type is best observed in the field but can be approximated by examining time-space diagrams for the street. The arrival type has a significant impact on delay estimation and LOS determination. Due to that, it is important to get arrival type as precise as possible. Equation 5.5 is useful to quantify the arrival type.

$$R_p = P(C / g) \tag{5.5}$$

where

R_p = platoon ratio

P = proportion of all vehicles arriving during green

C = cycle length (s)

g = effective green time for movement (s)

The value of P can be calculated or taken from the field, while the value of C and g can be computed from the signal timing. The value of P cannot exceed 1.0. Table 5.6 shows the relationship between arrival type and platoon ratio.

Table 5.6 Relationship between arrival type and platoon ratio

Arrival Type	Range of Platoon Ratio (R_p)	Default Value (R_p)	Progression Quality
1	≤ 0.50	0.333	Very Poor
2	> 0.50 – 0.85	0.667	Unfavourable
3	> 0.85 – 1.15	1.000	Random Arrivals
4	> 1.15 – 1.50	1.333	Favourable
5	> 1.50 – 2.00	1.667	Highly Favourable
6	> 2.00	2.000	Exceptional

*Adapted from US HCM 2000

Progression Adjustment Factor

The progression adjustment factor, PF, applies to all coordinated lane groups, whether the control is pretimed or non-actuated in a semiactuated system. Usually this factor affects uniform delay,d_1 where the value of PF can be determined by equation 5.6.

$$PF = \frac{(1-P)f_{PA}}{\left(1-\frac{g}{C}\right)} \qquad (5.6)$$

Where

PF = progression adjustment factor

P = proportion of all vehicles arriving during green

g/C = effective green-time ratio

f_{PA} = supplemental adjustment factor for platoon arrival during the green

The value of P may be estimated from time-space diagram or can be measured in the field. The value of PF can also be computed using the default value of f_{PA}, in order to come up with a measured value of P.

Table 5.7 shows progression adjustment factors for uniform delay calculation. From the table, the value of PF can be determined as a function of the arrival type based on the default values for P and f_{PA} which are associated with each arrival type.

The progression adjustment factor, PF, requires knowledge of offsets, travel speeds and intersection signalisation.

Table 5.7 Progression Adjustment Factors for uniform delay calculation

Green Ratio (g/C)	Arrival Type (AT)					
	AT 1	AT 2	AT 3	AT 4	AT 5	AT 6
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.174	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000
f_{PA}	1.00	0.93	1.00	1.15	1.00	1.00
Default, R_P	0.333	0.667	1.000	1.333	1.667	2.000

Notes:
PF = $(1-P)f_{PA}/(1-g/C)$
Tabulation is based on default values of f_P and R_P
P = RP * g/C (may not exceed 1.0)
PF may not exceed 1.0 for AT 3 through AT 6.

*Adapted from US HCM 2000

Incremental Delay Adjustment for Actuated Controls

The effect of controller type on delay can be measure in equation 5.4. For pre-timed signals, a k-value of 0.50 is used. The value is used based on queuing with arrival type 3 (AT 3) and on uniform service equivalent to the lane group capacity. In the case of actuated control type, the green time can be tailor according to the current demand, and reducing the overall incremental delay.

Table 5.8 shows the appropriated k-value for controller type. From the table, there are two criteria effect the delay reduction; controller's unit extension and the degree of saturation. When the degree of saturation approaches 1.0, an actuated controller will act like a pre-timed controller, producing a k-values of 0.50 at degree of saturation greater than or equal to 1.0.

Unit Extension (s)	Degree of Saturation (X)					
	≤ 0.50	0.60	0.70	0.80	0.90	≥ 1.0
≤ 2.0	0.04	0.13	0.22	0.32	0.41	0.50
2.5	0.08	0.16	0.25	0.33	0.42	0.50
3.0	0.11	0.19	0.27	0.34	0.42	0.50
3.5	0.13	0.20	0.28	0.35	0.43	0.50
4.0	0.15	0.22	0.29	0.36	0.43	0.50
4.5	0.19	0.25	0.31	0.38	0.44	0.50
5.0[a]	0.23	0.28	0.34	0.39	0.45	0.50
Pretimed or Nonactuated Movement	0.50	0.50	0.50	0.50	0.50	0.50

Table 5.8 k-Value for Controller Type

Notes:
For a unit extension and its k_{min} value at X=0.5; $k=(1-2k_{min})(X-0.5)+k_{min}$, where $k \geq k_{min}$, and $k \leq 0.5$.
a. For a unit extension more than > 5.0,extrapolate to find k, keeping k ≤ 5.0

*Adapted from US HCM 200

Upstream Filtering or Metering Adjustment Factor, I

Equation 5.4 mentioned I or the incremental delay adjustment term is for the effect of filtered arrivals from upstream signals. Value of 1 is used for an isolated intersection, is based on a random number of vehicles arriving per cycle so that the variance in arrivals equal to mean. An isolated intersection is an intersection that is 1.6 km or more from the nearest upstream signalised intersection

Value less than 1.0 is used for non-isolated intersections, and shows the upstream signals decrease the variance in the number of arrivals per cycle at the site intersection. Therefore, the amount of delay due to random arrivals is reduced.

Table 5.9 Recommended I-Values for Lane Groups with Upstream Signals

	Degree of Saturation at Upstream Intersection, X_u						
	0.40	0.50	0.60	0.70	0.80	0.90	≥ 1.0
I	0.922	0.858	0.769	0.650	0.500	0.314	0.090

Note:
$I = 1.0 - 0.91X_u^{2.68}$ and $X_u \leq 1.0$.

*Adapted from US HCM 2000

Table 5.9 lists I-Values for non-isolated intersections. X_u is the weighted v/c ratio of all upstream movements contributing to the volume in the intersection lane group.

5.2.2 DETERMINING TRAVEL SPEED

Equation 5.7 is used on each segment and on the entire section to computer the travel speed.

$$S_A = \frac{3600L}{T_R + d} \tag{5.7}$$

Where

S_A	= average travel speed of through vehicles in the segment (km/hr)
L	= segment length (km)
T_R	= total of running time on all segments in defined section (s)
d	= controlled delay fro through movements at the signalised intersection (s)

5.2.3 DETERMINING LOS

Table 5.1 shows a distinct set of arterial LOS criteria. Both free-flow speed of the street and the intersection LOS are taken into consideration.

5.3 PLANNING APPLICATIONS

5.3.1 OBJECTIVES

The objective of an arterial street LOS analysis at a planning level is to approximate the operating conditions of the facility. The accuracy of the planning of LOS analysis is dependent on the degree of generalization of input data.

The major differences between planning analysis of signalized intersections and the arterial is the treatment of turning vehicle. The purpose of an arterial is to move vehicles over a reasonable length of roadway.

5.3.2 DATA REQUIREMENTS

For the planning analysis traffic characteristics, roadway characteristics and signal characteristics values are needed for the following:

1. Traffic characteristics
   Annual average daily traffic (AADT)
   Planning analysis peak hour factor
   Directional distribution factor
   Peak hour factor (PHF)
   Adjusted saturation flow rate
   Percentage of turns from exclusive lanes

2. Roadway characteristics
   Number of through lanes (N)
   Free-flow speed
   Arterial classification
   Medians
   Exclusive right-turn lanes

3. Signal characteristics
   Signal type
   Cycle length (C
   Effective green ratio (g/C)

5.3.2.1 Adjusted Saturation Flow Rate

Many factors affect the saturation flow rate per lane. For planning analysis, these adjustments factors should be multiplied by the ideal saturation flow rate. The ideal saturation flow rate for Malaysia is 1930 passenger cars per hour of green time per lane (pcphgpl).

5.3.2.2 Percentage of Turns from Exclusive Lanes

Exclusive turn lanes represent the percentage of vehicles turn left or right-turning at signalized intersections from lanes absolutely dedicated to turning movements. It is assumed in this manual that the right-turns are accommodated by separate lanes and phase so that they have minimal effect on through vehicles.

5.3.2.3 Free-flow Speed

Arterial's free-flow speed should be based on actual studies of similar road and must be consistent with arterial classifications for planning purpose.

5.3.2.4 Medians

For planning purposes, the adjusted saturation flow rate may be reduced 5 percent for roadways that do not have medians.

5.3.2.5 Effective Green Ratio (g/C)

The parameter g/C is the ratio of the time at signalized intersections allocated for through traffic movement to the cycle length (C). Arterial's through g/C for each intersection is desirable. Weighted g/C of an arterial may be appropriate for broad planning purpose. The weighted g/C of an arterial is the average of the critical intersection through g/C and the average intersection through g/C.

5.3.3 COMPUTATION STEPS

The calculation process for determining arterial level of service consists of the following steps:

1. Convert daily volumes to the planning analysis hour by an appropriate planning analysis peak hour factor.
2. Adjust the hourly volumes based on PHF for 15 minutes service flow rates.
3. Calculate the running time on the basis of starting time, ending time and total stop of the vehicle.
4. Calculate the traffic flow and average travel speed on the basis of length of the route, travel time, space mean speed, number of test runs.

5. Calculate the average travel speed using the speed-flow graphs for know traffic flow in the flow at different conditions (high side friction, low side friction, traffic signal more than two or less than two, urban and suburban).
6. Calculate the arterial level of service on the basis of average travel speed.

5.4 WORKSHEET

URBAN SUBURBAN ARTERIAL WORKSHEET

General Information		**Site Information**	
Analyst		Urban Street	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
Analysis Time Period		Analysis Year	

[ ] Operational(LOS) [ ] Design (V_P) [ ] Planning (LOS) [ ] Planning (V_P) Analysis Period, T= ______ hour

Input Parameters	segments 1	2	3	4	5	6	7	8
Cycle length, C (s)								
Effective green-to-cycle length ratio, g/C								
v/c ratio for lane group, X								
Capacity of lane group, c (veh/hr)								
Arrival type, AT								
Segment Length, L (km)								
Initial queue, Qb (veh)								
Volume (veh/hr)								
Side Friction								
No. Traffic Lights								
Running Speed, S_R (km/hr); from speed flow relationship chart (Figure 5.3-5.9)								
Running Time, T_R (s)								
Delay Calculation								
Uniform Delay, $d_1 = \frac{0.50C[1-(g/C)]^2}{1-[\min(1,X)g/C]}\ (s/veh)$								
Signal control adjustment factor, k								
Upstream filtering adjustment factor, I								
Incremental delay, $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right](s/veh)$								
Initial queue delay, d_3								
Progression adjustment factor, PF								
Controlled delay, d (s)								
Segment LOS Determination								
Segment Travel Time, ST (s); $ST = T_R + d$ + other delay								
Segment travel speed, S_A (km/hr); $SA = \frac{3600(L)}{ST}$								
Segment LOS (Table 5.1)								

Arterial LOS determination

Total travle time = ______ s

Total length = ______ km

Total travle speed, S_A = ______ km/hr

Total arterial LOS (Table 5.1) = ______

5.5 SAMPLE CALCULATIONS

Example 1:

- **The Urban Street** The total length of a divided multilane urban street is 2.5 km, with five signalised intersections at 0.5-km spacing.

- **The Question** What is the LOS by segment and for the entire length for one direction of flow for through lane groups?

- **The Facts**
  Field-measured FFS = 45 km/h, Urban road
  Cycle length = 70 s (all signals),
  g/C = 0.60 (all through lane groups),
  Lane group capacity = 1,800 veh/h,
  Arrival Type 3 for Segment 1,
  v/c ratio as shown on the worksheet,
  Arrival Type 5 for all other segments, and
  Analysis period = 1.0 h,
  Pretimed signals.

URBAN SUBURBAN ARTERIAL WORKSHEET

General Information		**Site Information**	
Analyst		Urban Street	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
Analysis Time Period		Analysis Year	

[/] Operational(LOS) [ ] Design (V_P) [ ] Planning (LOS) [ ] Planning (V_P) Analysis Period, T= 1 hour

Input Parameters	segments							
	1	2	3	4	5	6	7	8
Cycle length, C (s)	70	70	70	70	70			
Effective green-to-cycle length ratio, g/C	0.6	0.6	0.6	0.6	0.6			
v/c ratio for lane group, X	0.583	0.611	0.611	0.611	0.597			
Capacity of lane group, c (veh/hr)	1800	1800	1800	1800	1800			
Arrival type, AT	3	5	5	5	5			
Segment Length, L (km)	0.5	0.5	0.5	0.5	0.5			
Initial queue, Qb (veh)	-	-	-	-	-			
Volume (veh/hr)	1048	1898	1100	1100	1075			
Side Friction	low	low	low	low	low			
No. Traffic Lights	1	1	1	1	1			
Running Speed, S_R (km/hr); from speed flow relationship chart (Figure 5.3-5.9)	30.19	29.5	29.5	29.5	29.8			
Running Time, T_R (s)	59.62	61.02	61.02	61.02	60.4			
Delay Calculation								
Uniform Delay, $d_1 = \frac{0.50C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	8.6	8.8	8.8	8.8	8.7			
Signal control adjustment factor, k	0.5	0.5	0.5	0.5	0.5			
Upstream filtering adjustment factor, I	1	0.786	0.757	0.757	0.757			
Incremental delay, $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right]$ (s/veh)	1.4	1.2	1.2	1.2	1.1			
Initial queue delay, d_3	-	-	-	-	-			
Progression adjustment factor, PF	1	0	0	0	0			
Controlled delay, d (s)	10	1.2	1.2	1.2	1.1			
Segment LOS Determination								
Segment Travel Time, ST (s) ST = T_R + d + other delay	69.62	62.22	62.23	62.22	61.5			
Segment travel speed, S_A (km/hr) $SA = \frac{3600(L)}{ST}$	25.85	28.93	28.93	28.93	29.27			
Segment LOS (Table 5.1)	C	C	C	C	C			

Arterial LOS determination		
Total travel time =	317.8	s
Total length =	2.5	km
Total travle speed, S_A =	28.32	km/hr
Total arterial LOS (Table 5.1) =	C	

REFERENCES

REFERENCES

Akcelik, R (1981); Traffic Signals: Capacity and Timing Analysis; *Research Report ARR No. 123, Australian Road Research Board*; 12–22.

Akcelik, R (2000); aaSIDRA User Guide; *Akcelik & Associates Pty Ltd.*

Al-Masaeid, Hashem R . Date:6/1999 vol. 69, no. 6. Empirical and gap-acceptance approaches were used in a study in Jordan to help identify specific guidelines for estimating capacity and delay of U-turn movement at median openings. http://www.ite.org/membersonly/itejournal/pdf/jfa99a28.pdf

Antje Weinert (1991). Estimation of Critical Gaps and Follow-Up Times at Rural Unsignalised Intersections in Germany. www.nas.edu/trb/publications/ec018/35_37.pdf

Arahan Teknik (Jalan) 13/87; A Guide to the Design of Traffic Signals; *Jabatan Kerja Raya Malaysia; Chapter 6.*

Arahan Teknik (Jalan) 8/86; A Guide on Geometry Design of Roads; *Jabatan Kerja Raya Malaysia;* (1986).

Asri Hasan and Radin Umar Radin Sohadi (1993); Passenger Car Unit (pcu) of Through Vehicles Under Malaysian Conditions; *The Monthly Bulletin of The Institution of Engineers Malaysia*; 5–9.

Behnam, Jahanbakhsh Date:6/1972 vol. 42. Gap Acceptance as a Criterion for Left Turn Phasing. http://www.ite.org/membersonly/itejournal/pdf/JFA72A40.pdf

Brown, G. E. and Ogden, K. W. (1988); The Effect of Vehicle Category on Traffic Signal Design: A Re-Examination of Through Car Equivalents; *Proceedings 14th ARRB Conference, Part 2*; 27–37

Cuddon, A. P. and Bennett, D. W. (1988); Methods of Saturation Flow Measurement; *Proceedings 14th ARRB Conference, Part 2*; 109–117.

Cuddon, A. P. and Odgen, K. W. (1992); The Effect of Heavy Vehicles on Saturation Flows at Signalised Intersections; *Proceedings 16th ARRB Conference, Part 5*; 1–18.

Duncan, N.C (1976); A Note on Speed/Flow/Concentration Relations; *Traffic Engineering and Control*; 34-35.

Drew, Donald R. (1968) *Traffic Flow Theory and Control*, Texan : Mc Graw-Hill

Hall, F.L.(1992); *Traffic Stream Characteristics, Traffic Flow Theory and Characteristics*; Transportation Research Board.

HCM (1985). *Special Report 209:Highway Capacity Manual* , Transportation Research Board.

J. L. Gattis and Sonny T. Low. pp. 201-207. Gap Acceptance at Atypical Stop-Controlled Intersections. http://ojps.aip.org/journal_cgi/getpdf?KEY=JTPEDI&cvips=JTPEDI000125000003000201000001

Kimber, R. L.; McDonald, M. and Hounsell, N. B. (1985); Passenger Car Units in Saturation Flows: Concept, Defination, Derivation; *Transportation Reseach , Volume 19B;* 39–61.

Kimber, R. L.; McDonald, M. and Hounsell, N. B. (1986); The Prediction of Saturation Flows for Road Junctions Controlled by Traffic Signals; *Research Report 67; Department of Transport, Transportation and Road Research Laboratory, HMSO, London;* 1–14.

Lam, William H. K. (1994); Saturation Flows and Passenger Car Equivalents at Signalised Intersections in Hong Kong; *In Proceedings 2nd International Symposium on Highway Capacity (Akcelik, R., ed.), Volume 2*; 385–394.

Lum K. M.,Fan H. S. L., Lam S. H. & Olszewski P. (1998). Speed Flow Modelling of Arterial Roads in Singapore, *Journ. Of Transpn. Eng.* 124(3),213-222

Manual Kapasitas Jalan Indonesia (1996). Republik Indonesia Direktorat General Bina Marga Directoratate of Urban Development Road Development (BINKOT).

May Adolf D. (1990). *Traffic Flow Fundamentals,* Englewood Cliffs, New Jersey: Prentice Hall

M.M. Hamed and S. M. Easa (1997). Disaggregate Gap-Acceptance Model for Unsignalised T-Intersections. Journal of TransportAtion Engineering January/February 1997.

Olszewski, P., Fan, H.S.L and Tan, J.W. (1995), Area Wide Traffic Speed-Flow Model for Singapore CBD, Transportation Research A, Vol. 29A, No. 4; 273-281

Papacostas, C. S., (1987), *Fundamentals of Transportation Engineering*, Prentice Hall.

Perunding Lee dan Rakan, Transportation Research Engineering Ltd. And Institute for Transport Studies, Leeds University (1996), Traffic Study for Malaysia (TSM) Final Report, Kuala Lumpur, HPU.

Radwan, Ahmed Essam and Sinha, Kumares C. Date:3/1980 vol.50, no.3. Gap Acceptance and Delay at Stop Controlled Intersections on Multi-Lane Divided Highways. http://www.ite.org/membersonly/itejournal/pdf/JCA80A38.pdf

Road Research Laboratory (1963); A Method for Measuring Saturation Flow at Traffic Signals; *Department of Scientific and Industrial Research, Road Note 34/196, HMSO, London;* 1–13.

Road Traffic Volume Malaysia (1996), *Highway Planning Unit, Ministry of Works, Malaysia.*

Robertson, H.D., Hummer, J.E., & Nelson, D.C. (1994). *Manual of Transportation Engineering Studies,* New Jersey : Prentice Hall.

Rod J. Troutbeck, Werner Brilon, and Wegmann (1991) Unsignalised Intersection Theory. www.tfhrc.gov/its/tft/chap8.pdf

Rod J. Troutbeck, Werner Brilon, and Wegmann (1991) Unsignalised Intersection Theory. www.tfhrc.gov/its/tft/chap8.pdf

Sinha, Kumares C. and Tomiak, Walter W.Date:4/1971 vol. 41. Section Gap Acceptance Phenomenon at Stop-Controlled Intersections. http://www.ite.org/membersonly/itejournal/pdf/JDA71A28.pdf

SPSS Inc; SPSS Base 9.0 Application Guide, United States of America (1999)

Teply, S. and Jones, A. M. (1991); Saturation Flow: Do We Speak the Same Language?; *Transportation Research Record 1320*; 144–153.

Transportation Research Board. (1985). Transportation Research Board, Special Report 209, Washington, 1985.

Transportation Research Board (1994); Highway Capacity Manual; *Special Report 209, Chapter 9: Signalised Intersections, third ed. Washington D.C.*

Velan, Shane M. dan Van Aerde, Michael. Date:3/1996 vol. 66, no. 3. Gap acceptance and approach capacity at unsignalised intersections. http://www.ite.org/membersonly/itejournal/pdf/JCA96A40.pdf

William C. Taylor; Ahmad S. Abdel-Rahim (1998). Analysis of Corridor Delay Under SCATS Control (Orchard Lake Road Corridor), University of Michigan.

Zong Z. Tian, Rod Troutbeck, Michael Kyte, Werner Brilon, Mark Vandehey, Wayne Kittelson and Bruce Robinson (1996). A Further Investigation on Critical Gap and Follow-Up Time. http://www.nas.edu/trb/publications/ec018/34_45.pdf